OSM/TR-82/2

Surface Mining Water Diversion Design Manual

September 1982

United States Department Of The Interior

Office Of Surface Mining

Technical Services & Research

OSM/TR-82/2

Surface Mining Water Diversion Design Manual

September 1982

Prepared For:

U.S. Department Of The Interior

Office Of Surface Mining

Contract No. J5101050

By: Simons, Li & Associates, Inc.
P.O. Box 1816
Fort Collins, Colorado 80522

United States Department Of The Interior
Office Of Surface Mining

Technical Services & Research

CONTRIBUTORS

D. B. Simons, Principal Engineer

R. M. Li, Principal Hydraulic Engineer

J. D. Schall, Senior Hydraulic Engineer

M. R. Peterson, Hydraulic Engineer

B. A. Anderson, Hydraulic Engineer

R. M. Summer, Geomorphologist

W. T. Fullerton, Senior Hydraulic Engineer

For sale by the Superintendent of Documents, U.S. Government Printing Office Washington, D.C. 20402

TABLE OF CONTENTS

PART 1: DESIGN PROCEDURES FOR CONDITIONS TYPICAL OF THE EASTERN COAL PROVINCE

TABLE OF CONTENTS (continued)

TABLE OF CONTENTS (continued)

TABLE OF CONTENTS (continued)

TABLE OF CONTENTS (continued)

TABLE OF CONTENTS (continued)

TABLE OF CONTENTS (continued)

PART 2: ADDITIONAL DESIGN PROCEDURES FOR CONDITIONS TYPICAL OF WESTERN COAL REGIONS

TABLE OF CONTENTS (continued)

TABLE OF CONTENTS (continued)

TABLE OF CONTENTS (continued

LIST OF FIGURES

LIST OF FIGURES (continued)

LIST OF FIGURES (continued)

LIST OF FIGURES (continued)

LIST OF FIGURES (continued)

LIST OF TABLES

LIST OF TABLES (continued)

ACKNOWLEDGEMENTS

This manual was prepared for the Office of Surface Mining by Simons, Li & Associates, Inc. under contract No. J5101050. The contract was initiated September, 1980, with Mr. Robert Carpenter as the Contracting Officer and Mr. Jonathan Ventura, OSM Region I, as the Technical Project Officer. The cooperation of Mr. Carpenter and the guidance and direction given to the project by Mr. Ventura are greatly appreciated. Additionally, Mr. Ventura made arrangements during the early stages of the project to visit several mine sites. Access to the mine sites and the time and effort spent by OSM field inspectors and mining company personnel significantly contributed to the project. Finally, the contribution to the geotechnical section of the report by project specialist Dr. Robert Johnson, Colorado State University, and the efforts of the Simons, Li & Associates, Inc. administrative, clerical and drafting staff in producing the Design Manual in its final form are greatly appreciated.

This report is based upon work supported by the Office of Surface Mining, Department of the Interior, under Grant No. J5101050.

Conversion Factors: English to International System (SI)*

Unit and Symbol	Multipliers	Unit and Symbol
Inches (in)	25.40	Millimeters (mm)
Inches (in)	2.540	Centimeters (cm)
Inches (in)	0.0254	Meters (m)
Feet (ft)	0.305	Meters (m)
Miles (mi)	1.61	Kilometers (km)
Yards (yd)	0.91	Meters (m)
Square inches (in^2)	6.45	Square centimeters (cm^2)
Square feet (ft^2)	0.093	Square meters (m^2)
Square yards (yd^2)	0.836	Square meters (m^2)
Acres (a)	4,047	Square meters (m^2)
Acres (a)	0.04047	Hectares (ha)
Square miles (mi^2)	2.59	Square kilometers (km^2)
Pint (pt)	0.4732	Liters (l)
Gallon (gal)-	3.785	Liters (l)
Cubic feet (ft^3)	0.0283	Cubic meters (m^3)
Acre-feet (a-ft)	1,233	Cubic meters (m^3)
Cubic inches (in^3)	16.4	Cubic centimeters (cm^3)
Cubic feet (ft^3)	0.028	Cubic meters (m^3)
Cubic yards (yd^3)	0.765	Cubic meters (m^3)
Ounce (oz)	28.35	Grams (g)
Pounds (lb)	0.4536	Kilograms (kg)
Tons (short (ts)	907.2	Kilograms (kg)
Tons (short) (ts)	0.9072	Metric tons (tm)
Pounds per square foot (lb/ft^2)	47.9	Newtons per square meter (N/m^2)
Pounds per square inch (lb/in^2)	6.9	Kilonewtons per square meter (kN/m^2)
Gallons (gal)	3.8	Liter (l)
Acre-feet (a-ft)	1,233	Cubic meters (m^3)
Gallons per minute (gal/min)	0.004	Cubic meters per minute (m^3/min)
Miles per hour (mi/h)	0.447	Meters per second (m/s)
Feet per second (ft/s)	0.3048	Meters per second (m/s)
Pounds per square foot (lb/ft^2)	0.4885	Grams per square centimeter (g/cm^2)
Tons (short) per square foot (ts/ft^2)	0.9765	Kilograms per square centimeter (kg/cm^2)
Tons (short) per acre (ts/a)	0.2241	Kilograms per square meter (kg/m^2)
Tons (short) per square mile (ts/mi^2)	350.2	Kilograms per square kilometer (kg/km^2)
Pounds per cubic foot (lbs/ft^3)	16.02	Kilograms per cubic meter (kg/m^3)
Cubic feet per second (ft^3/s)	0.0283	Cubic meters per second (m^3/s)
Cubic feet per second for one day ($ft^3/s\text{-}d$)	2,446	Cubic meters (m^3)

English Conversion Factors for Water Hydrology

Unit and Symbol	Multipliers	Unit and Symbol
Inches per hour (in/h) --------	1.008	Cubic feet per second (ft^3/s)
Inches per hour (in/h) --------	645.3	Cubic feet per second per square mile ($ft^3/s/mi^2$)
Surface inches (in) -----------	0.0833	Acre-feet (a-ft)
Surface inches (in) -----------	645.3	Cubic feet per second per hour ($ft^3/s/h$)
Surface inches (in) -----------	26.89	Cubic feet per second per day ($ft^3/s/d$)
Square miles (mi^2) ------------	640	Acres (a)
Acres (a) ----------------------	43,560	Square feet (ft^2)
Square feet (ft^2) -------------	144	Square inches (in^2)
Cubic feet per second-days (ft^3/s-d)	1.983	Acre-feet (a-ft)
Miles per hour (mi/h) ---------	1.467	Feet per second (ft/s)

*Example: To convert from inches to millimeters multiply by 25.40.

I. INTRODUCTION

1.1 The Problem

The design and implementation of an adequate drainage system before, during and after surface mining operations is essential to minimize adverse environmental impacts. Surface mining operations are often a source of water pollution. Water pollution from a surface mine generally occurs in two forms: chemical and physical. Chemical pollution is the result of minerals exposed to leaching or oxidation, producing undesirable concentrations of dissolved materials. Physical pollution is the increased sediment loading from excessive erosion. Since sediment is also a major carrier of many chemical pollutants, the two forms of pollution often occur simultaneously.

Compared to industrial pollution, which is usually a result of by-products created while the industrial process is actively pursued, water pollution from surface disturbances can be a continuous source of pollution for years after the mine becomes inactive. Runoff from abandoned mine sites can continue to carry large volumes of sediment and concentrations of chemicals to downstream bodies of water. Sediment in a stream or reservoir, above certain "natural" levels, constitutes pollution and reduces the usefulness of the water. The deposition of sediments reduces storage volumes, complicates flood control and power generation, destroys aquatic life habitat, decreases the value of floodplain areas for recreational and agricultural purposes, and leads to obstructions to navigation in larger rivers. Additionally, the dynamic nature of drainage systems can result in dramatic responses in downstream channel alignment, shape and type to increased sediment loading.

Drainage pollution of any type affects nearly every type of water use. It increases costs to industrial, municipal and navigation water users, for instance, by corroding equipment or by requiring special water treatment. Additionally, relatively small amounts of pollution can prevent the use of surface waters for some recreational uses, as well as for fish and aquatic life production. According to the Appalachian Regional Commission (1969) in House Document 91-160,

> "Over the last 100 years, coal mining in the Appalachian Region has caused increased amounts of acid, sediment, sulfates, iron, manganese, and hardness in the Region's streams, thus substantially altering water quality. This alteration has occurred in approximately 10,500 miles of streams, primarily in the northern half of the Region."

The actual pattern of the streams affected by mining operations in the Appalachian Region generally corresponds to the historical and present patterns of mining. The distribution of streams affected by all types of mine is uneven among the eight states in the Region that are affected. Throughout the Region, the incidence of affected streams decreases from northeast to southwest. This decreasing trend of affected streams toward the southwestern parts of the Region is primarily the result of compositional changes in the coal and adjacent strata, the mining techniques used, and smaller amounts of mining.

1.2 Control of Drainage On a Mine Site

Due to the magnitude and extent of the pollution problems that have arisen as a result of previous mining activities, and the desire to minimize further proliferation of such pollution, drainage abatement and control techniques have been developed. In general, abatement and control techniques can be grouped into the categories of source control to prevent the formation of polluted water, treatment processes to handle water that has become polluted, dispersion and dilution of polluted water by its controlled addition to unpolluted flows, and permanent containment or isolation of contaminated waters by injection into deep disposal wells.

The two major categories which encompass the majority of the preferred abatement and control techniques are treatment and source control. Use of water treatment during mining has no effect on the levels of water pollution after treatment ceases and the mine is abandoned. Thus, preplanning and implementation of source controls to reduce water pollution, both present and future, is considered preferable.

Compared to other categories of pollution control, source control measures such as revegetation and properly engineered drainage structures are relatively inexpensive. Source control measures are further desirable in that they help minimize the formation of polluted water by preventing contact of unpolluted water with areas disturbed by mining operations, thus limiting erosion and sediment loads carried by runoff, contact of runoff with acid- or toxic material-producing materials, and decreasing quantities of water needing treatment.

Diversion practices are particularly suitable in that they provide for a measure of control over the watershed. Using sound engineering design, diversion structures, channel modification and relocations can be constructed in a manner that provides pollution and erosion control through runoff management over a wide range of seasonal and climatic conditions. Such structures can prevent runoff into active mine sites, thus helping to reduce ponding along the bench or in the pit and the resultant downtime. Properly engineered diversions reduce erosion by preventing flow over unstabilized soil in waterways above highwalls, and over disturbed areas where vegetation has not been established.

Diversion is by no means a complete pollution control measure, but simply an integral part of an overall plan. The need for a complete drainage design plan for the permit area based on sound engineering knowledge is necessary to minimize potential environmental damage from surface mining activities. Further, it is essential that the designer realize that the drainage basin in the permit area is only one part of a larger, more complex drainage system. The drainage network in the permit area interacts with other parts of the larger drainage system in a complex fashion. Over time this complicated system has established a state of balance or quasi-equilibrium. The mining operation, or any other large-scale disturbance, will affect this balance or equilibrium and can result in dynamic responses throughout the system. The designer must recognize this phenomenon in order to restore the disturbed topography and drainage to a condition where it will again properly function as part of the larger system.

1.3 OSM Regulations Concerning Water Diversions

General provisions of OSM standards pertaining to surface mine drainage specify the best technology currently available should be used to minimize disturbances of the prevailing hydrologic balance, water quantity, and water quality. This standard is applicable to the mine site as well as outlying areas that would be affected by runoff from the mined region.

Of particular importance in the aforementioned specification is the phrase "best technology currently available," defined by the OSM as (30 CFR 701.5):

> "equipment, services, systems, methods, or techniques which will (a) prevent, to the extent possible, additional contributions of suspended solids to stream flow or runoff outside the permit area, but in no event result in contributions of suspended solids in excess of requirements set by applicable state or federal laws; and

> (b) minimize, to the extent possible, disturbances and adverse impacts on fish, wildlife and related environmental values and achieve enhancement of these resources where practicable. The term includes equipment, devices, systems, methods or techniques which are currently available anywhere as determined by the Directors, even if they are not in routine use."

Other federal laws controlling discharges are the Federal Water Pollution Control Act and the Clean Water Act administered by the EPA.

Specifically, OSM regulations require that diversions of overland flow or flow in ephemeral streams are to be undertaken in a manner which prevents erosion, avoids contact with acid-forming and toxic material-forming materials, and reduces the amount of suspended solids entering receiving streams or other off-site bodies of water.

Standard practices and criteria related to the diversion of overland flow, and flow in ephemeral, perennial, or intermittent streams, are summarized in Table 1.1. It is important to note that the diversion channel itself does not necessarily have to be large enough to pass the design flow (Table 1.1, part a). Regulations allow that the combination of channel, bank and floodplain configurations be adequate to pass the required flows. However, the capacity of the channel itself should at least be equal to the capacity of the unmodified stream channel immediately upstream and downstream of the diversion.

1.4 Applications of Water Diversion Structures

Water diversion structures are temporary or permanent water handling structures used to control and manage the drainage above and through the disturbed area of a mine site including the channels diverting and conveying the runoff, grade control structures, erosion control structures, etc. In this manual, diversions are defined as those channels used to intercept and divert surface runoff, and those used to relocate or reestablish ephemeral, intermittent or perennial streams. Perennial streams normally carry water throughout the year because they either drain areas of heavy rainfall or intersect the ground water table at some point. Intermittent streams flow

Table 1.1. Design Requirements by Technologies.

Considerations*	Overland Flows, Shallow Groundwater Flows, Ephemeral Streams	Perennial and Intermittent Streams
<u>Hydrology</u>		
(a) Recurrence Interval-Design Event		
Permanent	10-year, 24-hour	100-year, 24-hour
Temporary	2-year, 24-hour	10-year, 24-hour
<u>Hydraulics</u>		
(b) Channel Capacity	Peak runoff from design event, 0.3 ft freeboard minimum. Protection of critical areas can be more stringent.	Must equal adjacent unmodified stream channel (floodplain capacity can be used for passing design event), but not less than (a).
(c) Channel Lining	Suitable to control and minimize water pollution.	To control erosion, must be stable and only require infrequent maintenance.
(d) Slope or Gradient	Appropriate for sediment control.	Longitudinal profile of the stream to remain stable and to prevent erosion.
(e) Velocities	Regulated to control and minimize water pollution.	Regulated to control and minimize water pollution.
<u>Geotechnical</u>		
(f) Backslopes	Stable	Stable
<u>Ecological</u>		
(g) Restoration		
Permanent	None	Restore or maintain natural riparian vegetation, including aquatic habitats (riffles, pools, drops, etc.) that approximate premining characteristics.
Temporary	Remove regrade topsoil & revegetate.	Same as ephemeral stream
(h) Enhancement	None	"Where practicable" enhance natural riparian vegetation.
(i) Shape	None	Establish or restore natural meandering shape of an environmentally acceptable gradient.
(j) Longitudinal Profile and Cross Section	(see slopes and capacity)	Establish or restore to approximate premining stream channel characteristics (including aquatic considerations below).
(k) Aquatic Habitats	None	"Establish or restore...usually a pattern of pools, riffles and drops...that approximate premining characteristics."

*Where not specifically indicated, temporary and permanent requirements would be the same.

steadily for only a part of the year and are seasonally dry. Ephemeral streams are normally dry and flow only in response to precipitation or snowmelt.

Diversion of surface runoff (overland flow), shallow ground water flow and ephemeral streams helps to control erosion, reduces contact time with acid-and toxic material-forming materials, and reduces the suspended sediments entering downstream bodies of water. Diversion of intermittent and perennial streams into new channels is performed to reduce seepage into or flooding of the work area, and to allow the mining operation in the region of the original stream channel. Diversion of intermittent or perennial streams is allowed only with specific approval from the regulatory authority due to the potential adverse environmental impacts.

Diversion ditches above the highwall or open cut are often portrayed in surface mining publications as a common method of diversion; however, this application is not always practiced. Water diversion channels can also occur within and through the disturbed area to reestablish drainage patterns. Application of diversion channels and relocations to contour, area and mountaintop removal methods are illustrated in Figures 1.1, 1.2 and 1.3, respectively. Diversions are also used below spoil slopes to direct runoff to sediment ponds and as conveyances of natural drainage around excess spoil slopes and on roadways to protect the lower portions of the hillside or roadway from highly erosive flows. Figure 1.4 illustrates the typical surface drainage control techniques used for an excess spoil fill.

Usually the channel is located in the groin area as shown. Due to the steep slope topography typical of the Eastern Coal Province, these channels must be carefully designed and constructed in order to remain stable.

Another diversion technique for excess spoil fills found unique to the Eastern Coal Province is the use of an internal rock core drain. This technique is employed in lieu of a surface diversion to convey runoff from the surface of the fill and from areas above the fill where the fills are not carried to the ridge line. The internal drain is used for handling both the surface and subsurface water. The surface water percolates through the rock core much like a french drain.

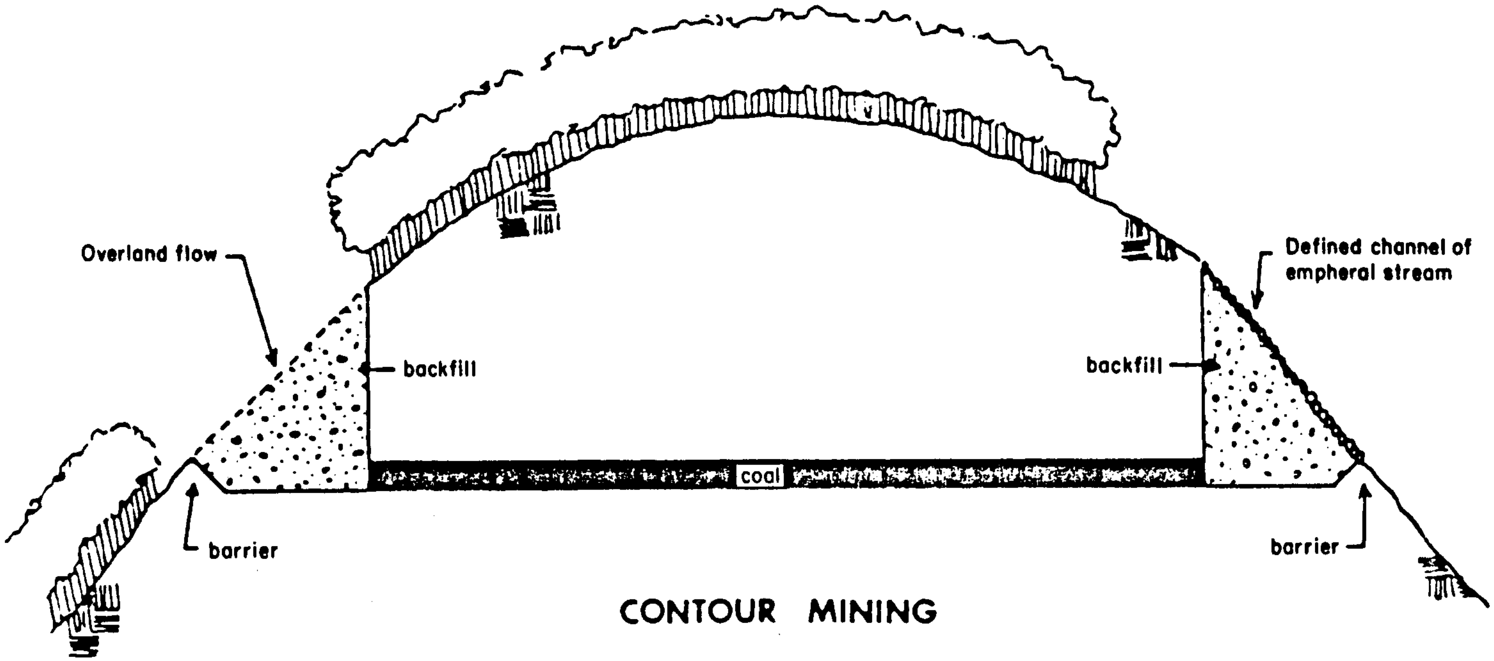


Figure 1.1. Drainage in contour mining.

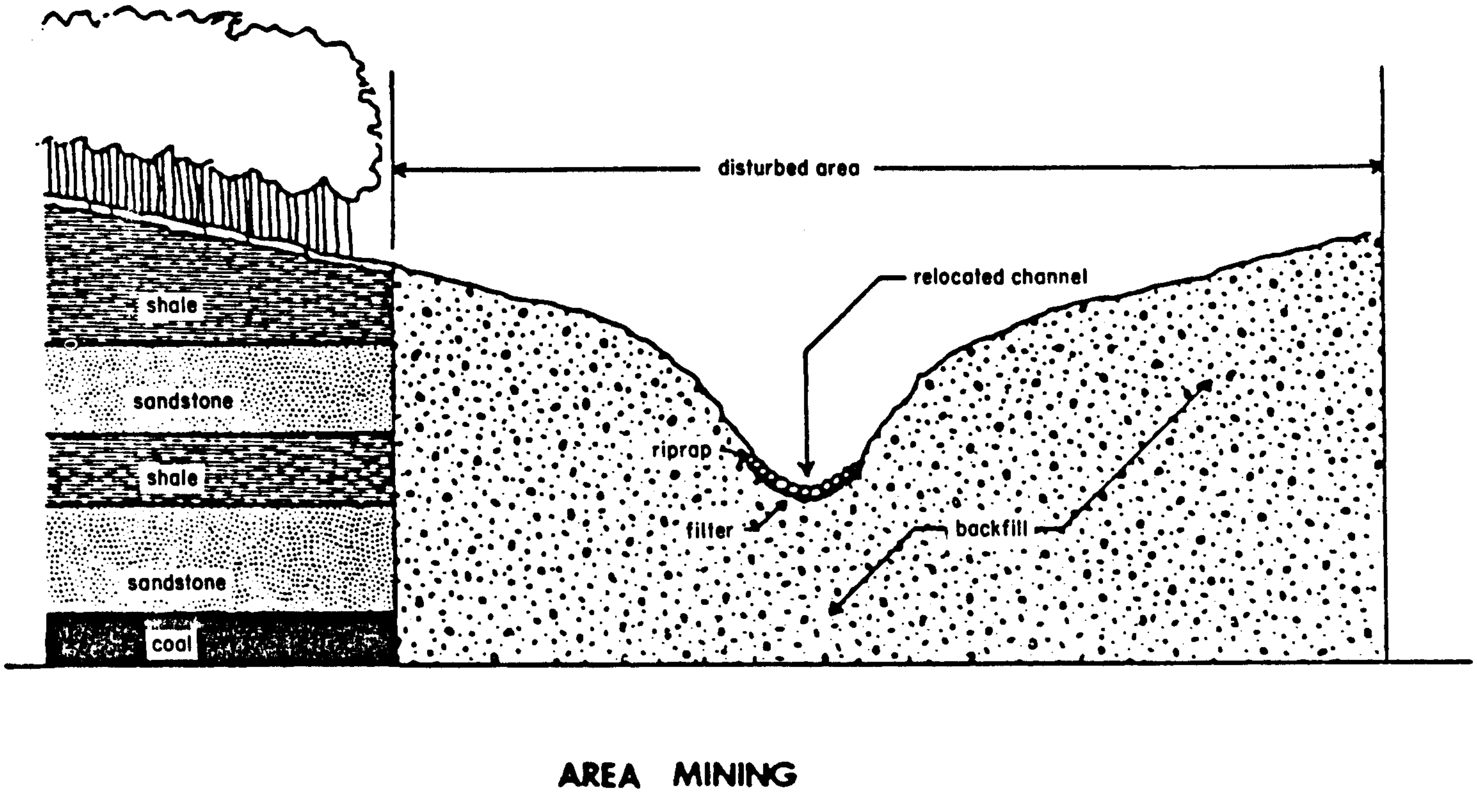


Figure 1.2. Relocated channel in area mining.

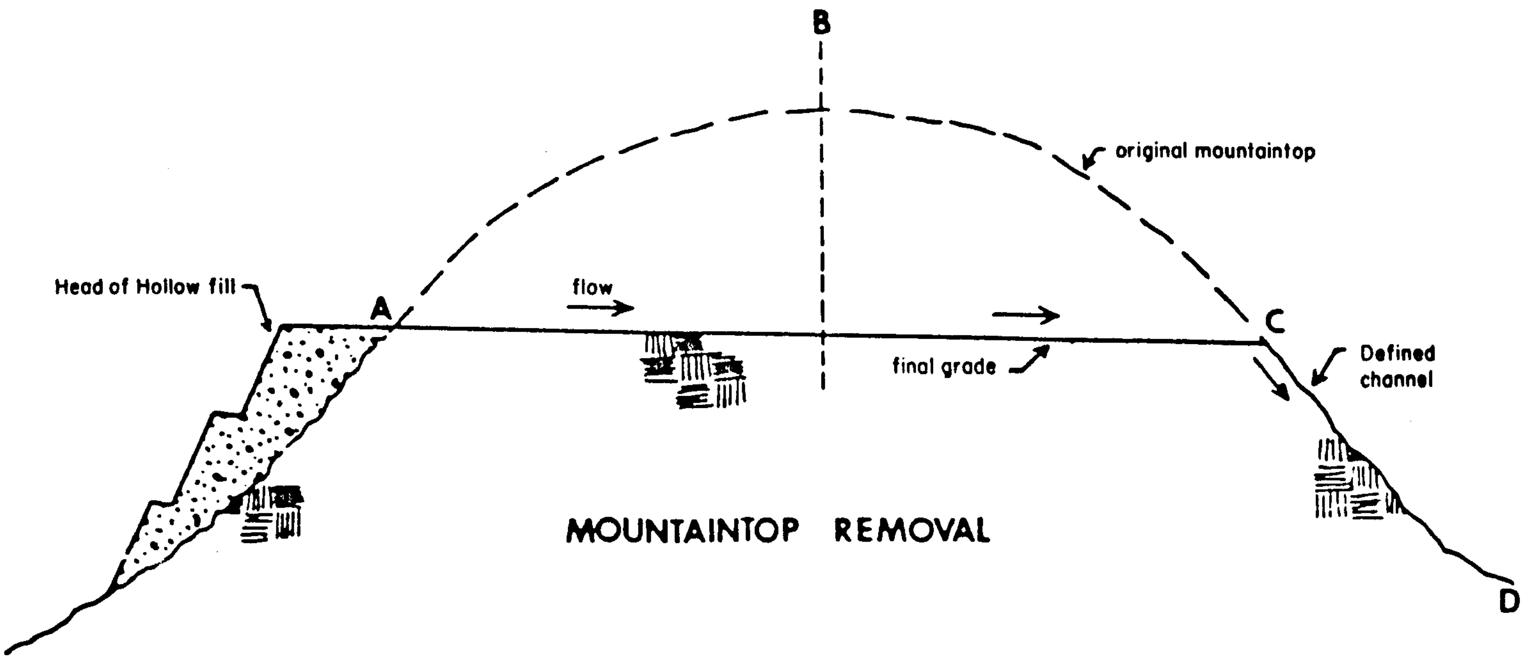


Figure 1.3. Drainage patterns in the mountain removal method.

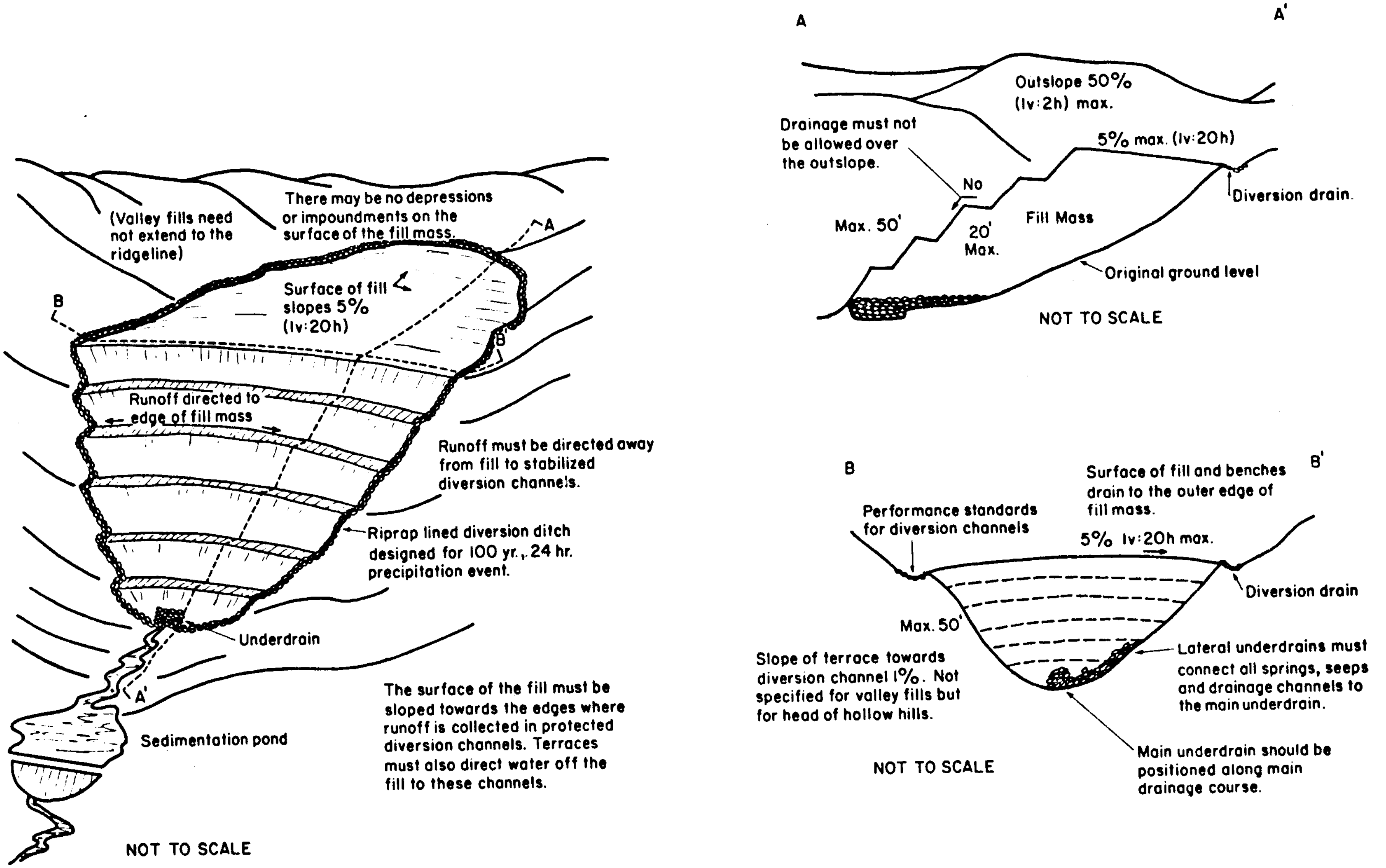


Figure 1.4. Design requirements for valley fills.

Based on current practices of surface conveyance construction observed on surface mine operations, at least for the short term, rock core drains may be a more feasible method for diverting water around spill fills. However, there were no standardized methods of design or analysis found in literature reviews. The construction techniques are also mostly left up to the operator. Additionally, many concerns have been raised as to their long-term functioning. No published research was found to confirm or dispute this concern. Therefore, no design technology for rock core drains in spoil fills is presented in this manual.

In most cases, diversion is an economical form of erosion control. It is not meant as a complete erosion control, but as an integral part of an erosion control plan. The following are some of the factors that can significantly affect the cost of diversion systems:

1. Topography - Unusually steep topography, dense forest cover or rock formations may increase the cost of surface trenching.

2. Equipment - Availability of adequate equipment can significantly affect the cost of diversion ditches.

3. Condition of Soil - Rock fissures and highly permeable soil may necessitate the use of an impermeable material for trench construction.

Small temporary diversion structures may include the use of straw bales, tires, downpipes, brush and other temporary measures in addition to some permanent measures to achieve grade and erosion control. However, after an operation is complete, temporary diversion channels and structures must be converted to permanent standards or removed and the affected land regraded, topsoiled and revegetated in the same way as other disturbed areas of the site.

Permanent diversion structures used for the reestablishment of a drainage system affected by surface mining must perform adequately without the assured benefit of periodic maintenance. Serious environmental problems have resulted in many previously mined areas, particularly in steep slope regions, due to inadequate design of the drainage network and associated diversion structures. Plate 1.1 illustrates the result of improper diversion channel design in fill material. Combined with steep slope conditions and relatively large annual precipitation, channels that are not properly designed or protected become deeply incised in the erodible fill material, thus becoming a con-

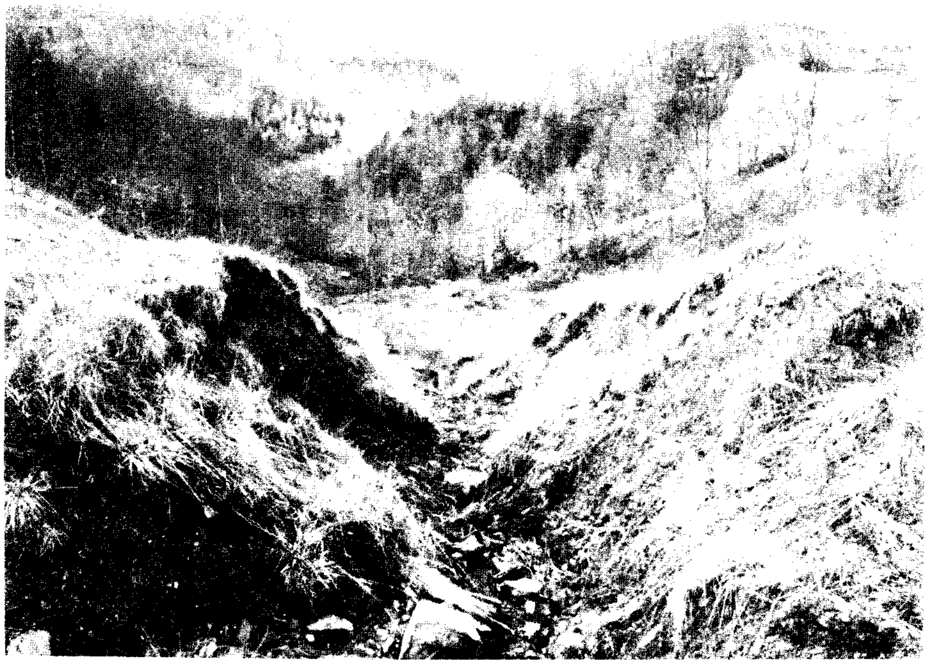


Plate 1.1

Plate 1.2

tinuous source of water pollution. However, even moderately sloped channels (Plate 1.2) can erode and become incised, illustrating the need for proper drainage in all cases. The incision shown in Plate 1.2 is not serious in its current condition; however, it has probably resulted from relatively small flows. If a 100-year event did occur, a much greater incision could be expected.

Typical drainage systems are shown in Figures 1.5 and 1.6 illustrating the relationship of ephemeral, intermittent and perennial streams. Reestablishment of ephemeral streams is the beginning and perhaps the most critical step in restoring the hydrologic balance of steep slope mining areas. If ephemeral streams do not perform adequately, significant erosion can occur in the upslope regions, resulting in large volumes of sediment being delivered to downstream intermittent and perennial streams. The increased sediment loading can cause a dynamic response and readjustment of these streams as the entire drainage network moves to reestablish a balance or quasi-equilibrium.

1.5 Problems Unique to OSM Regions I and II (Eastern Coal Province)

1.5.1 Geographic Considerations

The diversity in terrain, climate, biologic, chemical and physical conditions throughout mining regions of the country was recognized in the Surface Mining Control and Reclamation Act of 1977. The most dramatic differences are apparent between the mining operations of the humid eastern states and those of the arid and semiarid western states. The definitions of humid, arid and semiarid are generally based on precipitation, although a variety of climatological and environmental factors is involved. According to OSM regulations, the 100th Meridian is defined as the legal boundary of the "arid and semi-arid area." This definition is commonly accepted, although it ignores the humid Pacific coast and the snowfall of high mountain regions.

Dryland landscapes are quite different from those of more humid regions. The topography and landforms are more abrupt, the soils are thinner, the bedrock exposures are usually more pronounced and the streams are smaller and are likely to be dry for at least part of the year. Overall, the physical environment reflects the lack of water and the predominance of mechanical weathering and erosion over chemical weathering and solution.

In a humid environment high precipitation produces vegetation and soils that are well developed and stabilized. Under these natural conditions,

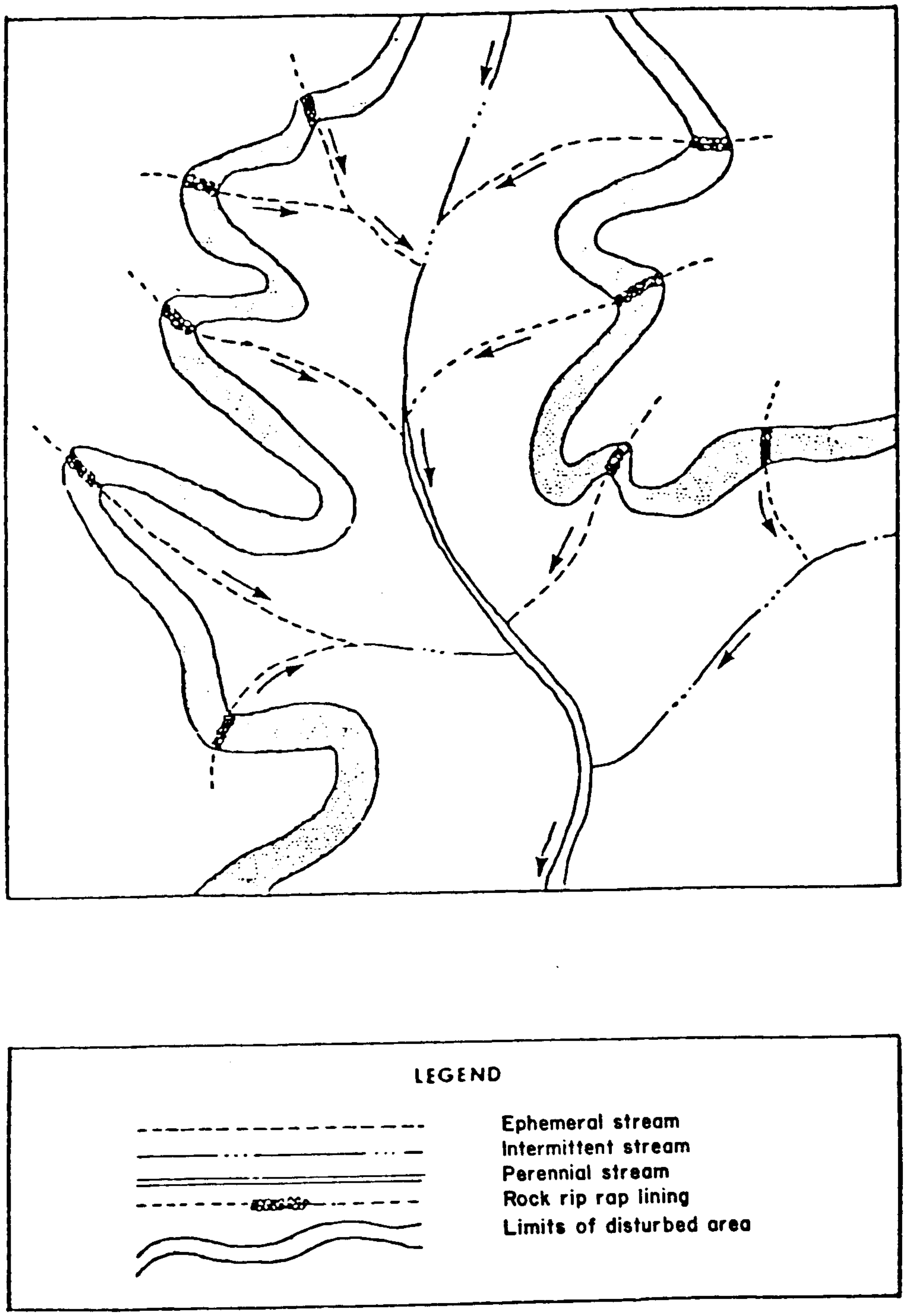


Figure 1.5. Plan view of contour mining drainage plan.

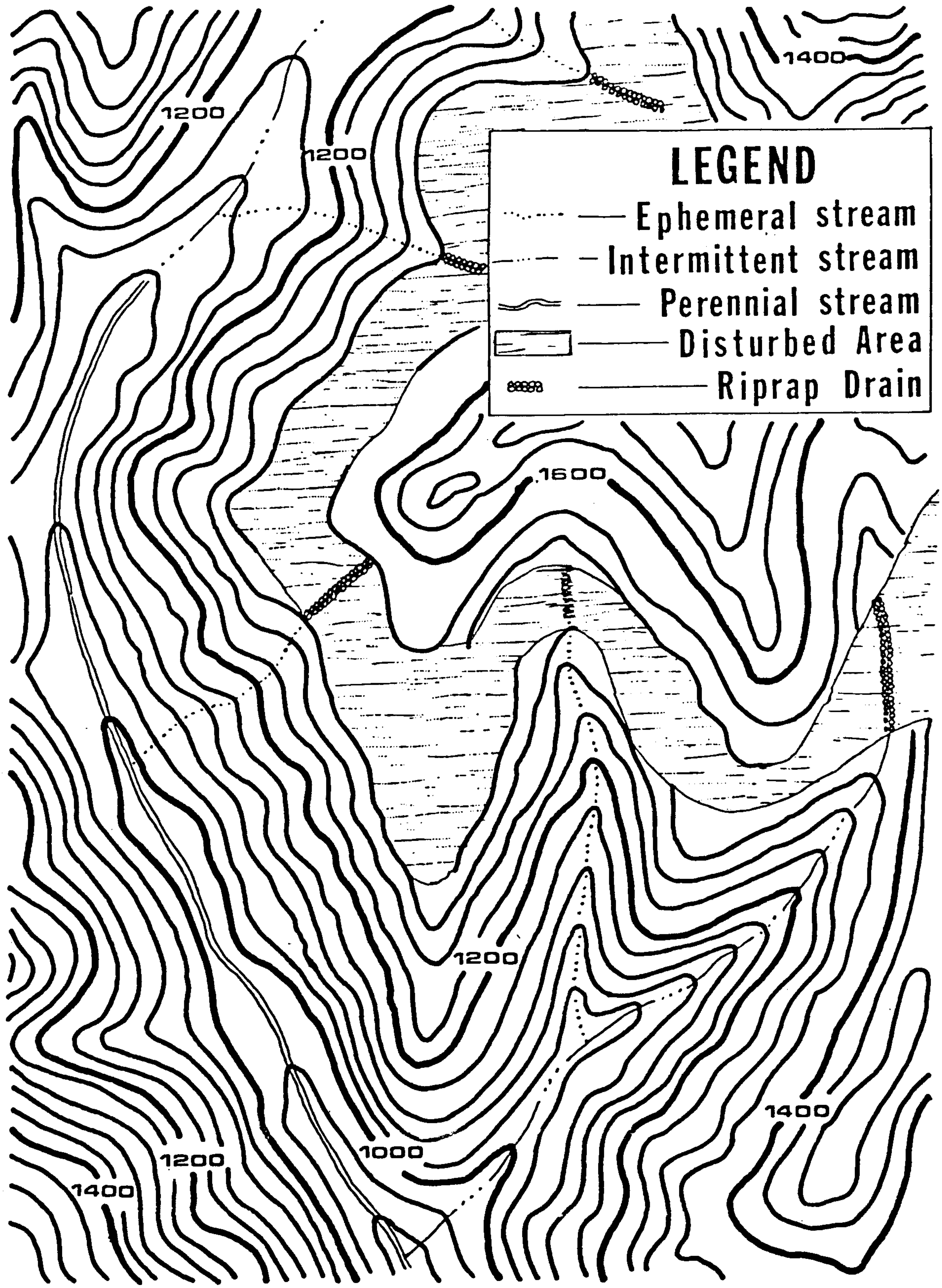


Figure 1.6. Plan view of a contour mine drainage plan.

streams generally carry low suspended sediment loads, reflecting this stability in the upland watersheds. Additionally, high precipitation produces a dilution effect on the sediments that are eroded.

In contrast, dryland streams normally carry quite high sediment loads from erosion by both wind and water. The precipitation generating the erosion in a dryland environment usually results from small storm cells that may be limited in areal extent, but can produce high intensity and rainfall energy. This type of storm produces "flashy" runoff with pronounced capacity for sediment removal and transportation. Only rarely does a single storm produce runoff in all parts of a dryland stream basin, and extended periods may pass with no streamflow at all. Many dryland streams flow only during the spring runoff and immediately after major storms. Therefore in drylands even streams draining large basins are often intermittent or ephemeral, while in a humid region most larger streams would be perennial.

However, in any climate one of the most important factors affecting sediment yield is land use. Wilson (1972) shows that the impact of surface mining on the humid eastern states produces sediment yields from the affected areas that are similar to those of arid regions. Therefore, higher sediment yields from disturbed mining sites in OSM Regions I and II relative to the natural stability of these watersheds must be considered in diversion channel design.

1.5.2 Specific Problems Observed in OSM Regions I and II (Eastern Coal Province)

Based on mine site visits during Phase I of this project, some general observations were made and specific problems identified. In most of the steeper sloped areas, surface mining is relatively high in the watershed, so diversions above the highwall controlling upper watershed drainage are not utilized. However, even in more moderately sloped areas where larger upper watershed drainages exist, such diversions are not utilized for drainage control. Water and sediment control is usually below the strip in the form of diversion structures leading to sediment ponds. The use of diversion channels to route flow through or around fill areas (head-of-hollow or valley fills) is a common application. Culverts and other closed conduit structures were not observed for diversion applications.

In general, a common problem with diversion channels appears to be construction techniques and the lack of proper supervision and inspection of construction work. For example, when riprap was used, it often was not properly designed or placed. Filter blankets beneath the riprap were not observed in any application. Additionally, the riprap layer was typically mounded in channels rather than being placed in a manner that maintained a basic channel shape (Plate 1.3). Consequently, the channel design capacity was greatly reduced, forcing larger flows outside the channel boundary (Plate 1.4). The lack of gradational particle sizes in the riprap layer was also a common problem. Typically, only rock of large diameter was used for riprap.

Part of the problem appears to be the difficulty in placing riprap on steep slopes. Catepillar D-9 bulldozers with 14-foot blades are used for earthmoving work, including diversion channel construction. In many instances, this piece of machinery is too large for effective or efficient channel construction based on permit design. For example, the typical technique for steep-slope riprap placement is end-dumping at the upslope end of the channel and then working a D-9 downslope to position the riprap. However, the result is usually a channel that is level or mounded and full of rock. More effective construction techniques will have to be implemented to insure adequate drainage in these situations. For example, the diversion channels could be built concurrently with the fill instead of after completion of the fill. As each lift is completed, the channel and riprap lining could be constructed, thus simplifying construction.

Plates 1.5 and 1.6 show the use of large boulders placed parallel to the channel to prevent meandering. Under the low flow conditions existing when the pictures were taken, the design may appear reasonable. However, under higher flow conditions, these boulders would probably not contain the flow. Although the boulders themselves would not move, the stream would move out of the riprapped channel by cutting a new course between any given pair of boulders.

Plate 1.7 illustrates another case where excessively large rocks have been used. The picture shows the entrance to a fill slope diversion. Under high flows most of the water would probably be forced around these boulders, thereby missing the riprapped channel entirely.

Given the difficulties in designing and placing riprapped diversions, some attempts have been made to avoid the problem entirely. Plate 1.8 illustrates a case where the natural drainages were completely blocked. The reasoning behind this design is that water caught behind the blockage will

Plate 1.3

Plate 1.4

Plate 1.5

Plate 1.6

Plate 1.7

Plate 1.8

slowly infiltrate and move through the fill as groundwater. Forcing overland flow from the upper watershed to groundwater flow can result in slope stability problems on the fill. This design could also create serious problems in the future. If, through time, the ground surface is sealed by deposition of fine silts and clays, ponded water could overtop the barrier, causing excessive erosion on the fill slopes.

Some examples of properly designed and constructed channels were observed. Plate 1.9 shows a small channel leading to a sediment pond. The riprap appears to have been hand-placed; however, the important concept is that the basic shape of the channel has been maintained. This basic concept is critical to a successful, long-lasting channel. Plate 1.10 shows another example of a larger channel on a steeper slope. Again, the riprap was probably hand-placed, which is not economically feasible on a larger scale. Regardless of the placement technique, the basic channel shape must be maintained in order for the channel to operate as designed.

Some examples of larger-scale channels that have been properly designed and constructed were observed in the mountains of Colorado. Plate 1.11 is a steep drainage channel along Interstate 70 on Vail Pass. Note how the rocks are placed, probably by machinery (i.e., not hand-placed), in a manner that maintained the basic channel shape. Plate 1.12 shows another approach using wire gabions to stabilize a steep slope drainage.

1.6 Design Manual Organization and Use

1.6.1 Design Manual Organization

The Design Manual is organized in two parts. Both parts are contained in this volume and the chapters are consecutively numbered for easy reference. Part II begins with Chapter XI. Part I considers design methodologies that are primarily applicable to the Eastern Coal Province. Additionally, it presents the other general information and background knowledge required to complete a good design. Part II contains supplemental design information required to design channels in sandy soil regions.

Due to the predominantly steep slope conditions that exist in the Eastern Coal Province, Part I gives special consideration to design procedures for steep slope channels (Chapter V). However, mild slope channels are also considered (Chapter VI). Rock riprap channels are emphasized due to their current widespread use.

Plate 1.9

Plate 1.10

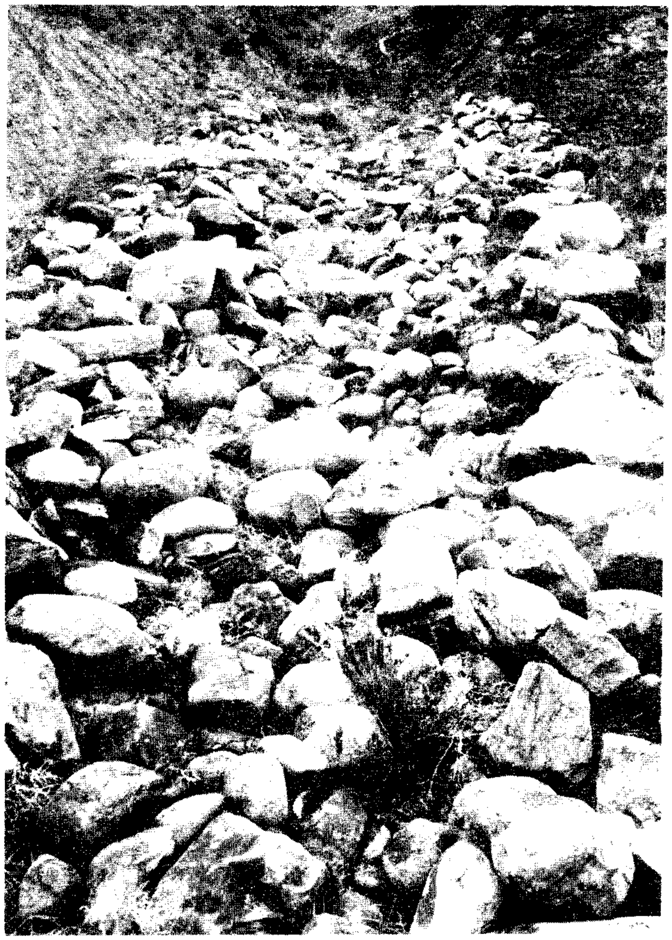


Plate 1.11

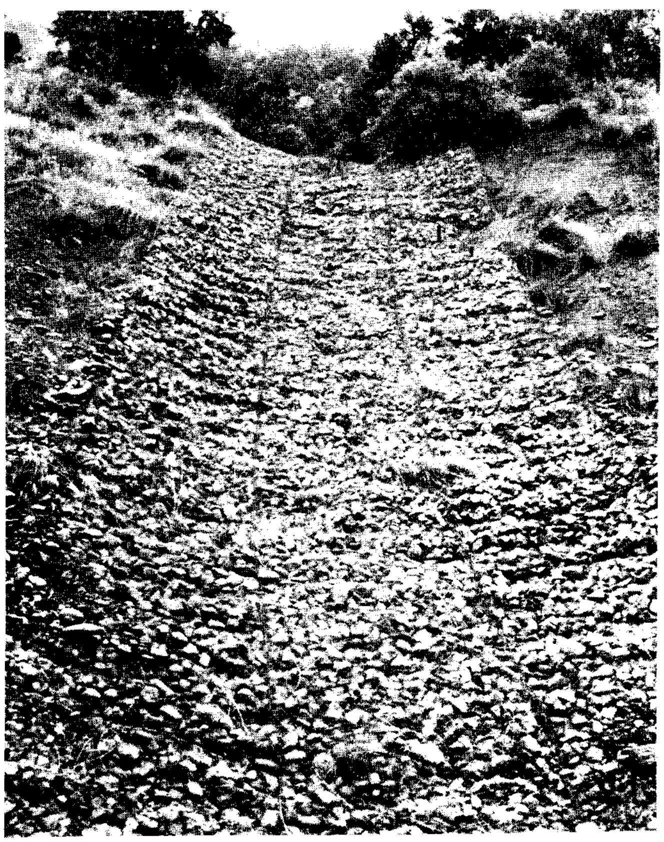


Plate 1.12

Due to the significant differences in stream morphology between the humid eastern states and the dryland environment of the western states, Part II presents the additional information required to design channels in sandy soils. Bed form conditions common in sand-bed channels and their effect on resistance to flow are discussed in Chapter XII. Chapter XIII discusses alluvial channel concepts, particularly movable boundary hydraulics. The design of large rock riprap drop structures is discussed in Chapter XVI and the design of dikes in Chapter XVII.

Assessment of the best technology currently available for application to surface mine operations as presented in this Design Manual was based on a comprehensive literature review. For those methodologies applicable to the Eastern Coal Povince a written literature review was prepared as the Phase I report for this project. Selection criteria for inclusion in the Design Manual from the broad range of design methodologies available included consideration of the physical environment at surface mine operations, current design procedures employed, the problems with existing diversion structures, and the level of effort required to produce an adequate diversion design. Many of the state-of-the-art procedures that provide the best possible design are too complicated and laborious to be used and are not included in the manual. In contrast, many of the simplified procedures, including some methods in common use, produce inadequate designs that probably would not survive the high flows required for permanent structures by OSM regulations. Therefore, the objective was to produce a usable document that provides reliable, accurate design procedures for conditions that exist on a surface mine operation.

Information in both the Phase I report and the Phase II Design Manual, Part 1 and Part 2, concentrates on permanent water diversion structures due to their greater long-term importance. Many publications are available providing design guidelines for temporary diversion design. Culverts and other closed conduit conveyance structures were also not considered. These structures may represent viable alternatives as a short-term, temporary water conveyance method; however, their permanent, long-term use cannot be considered reliable due to maintenance requirements. Additionally, due to concrete requirements, their construction can be very labor intensive, making them relatively infeasible for use on surface mine sites.

1.6.2 Design Manual Use

At first glance the Design Manual may appear difficult to use with complex and elaborate design procedures; however, upon closer examination the user should realize that throughout the manual an effort has been made to simplify the design procedures as much as reasonably possible. It may be that the Design Manual appears complex only relative to the procedures currently used. To overcome this potential problem, a flow chart has been prepared and numerous examples are given. The flow chart (Figure 1.7) illustrates the overall organization and decision-making process involved in the design of diversion channels and relocations. Familiarity with this flow chart will greatly aid the designer in utilizing the manual. For example, the flow chart indicates the first and perhaps most important decision in the design process is whether or not the slope condition is hydraulically steep or mild. Note that this definition of slope is in the hydraulic and not the topographic sense. This slope definition is assumed throughout the Design Manual. The hydraulic slope is based on the Froude number (Section 4.2.5) which depends on velocity and depth of flow. However, both velocity and depth of flow depend on the channel size and roughness. Therefore, the designer must first assume a slope condition based on topographic considerations and proceed with the design. At a later point in the design process this initial assumption is checked to insure the correct procedures have been followed. A good assumption to make is if the slope is greater than 10 percent the steep slope procedures should be followed; however, the designer must realize that in some cases slopes as low as four to five percent may be hydraulically steep.

The comprehensive examples given in Chapters X and IXX illustrate the application of the design procedures to conditions typical of eastern and western state mining operations, respectively. Users of the Design Manual are encouraged to carefully review these examples and the other examples within each chapter to better understand the design methodologies. With a little practice the complete design process will become familiar and straightforward.

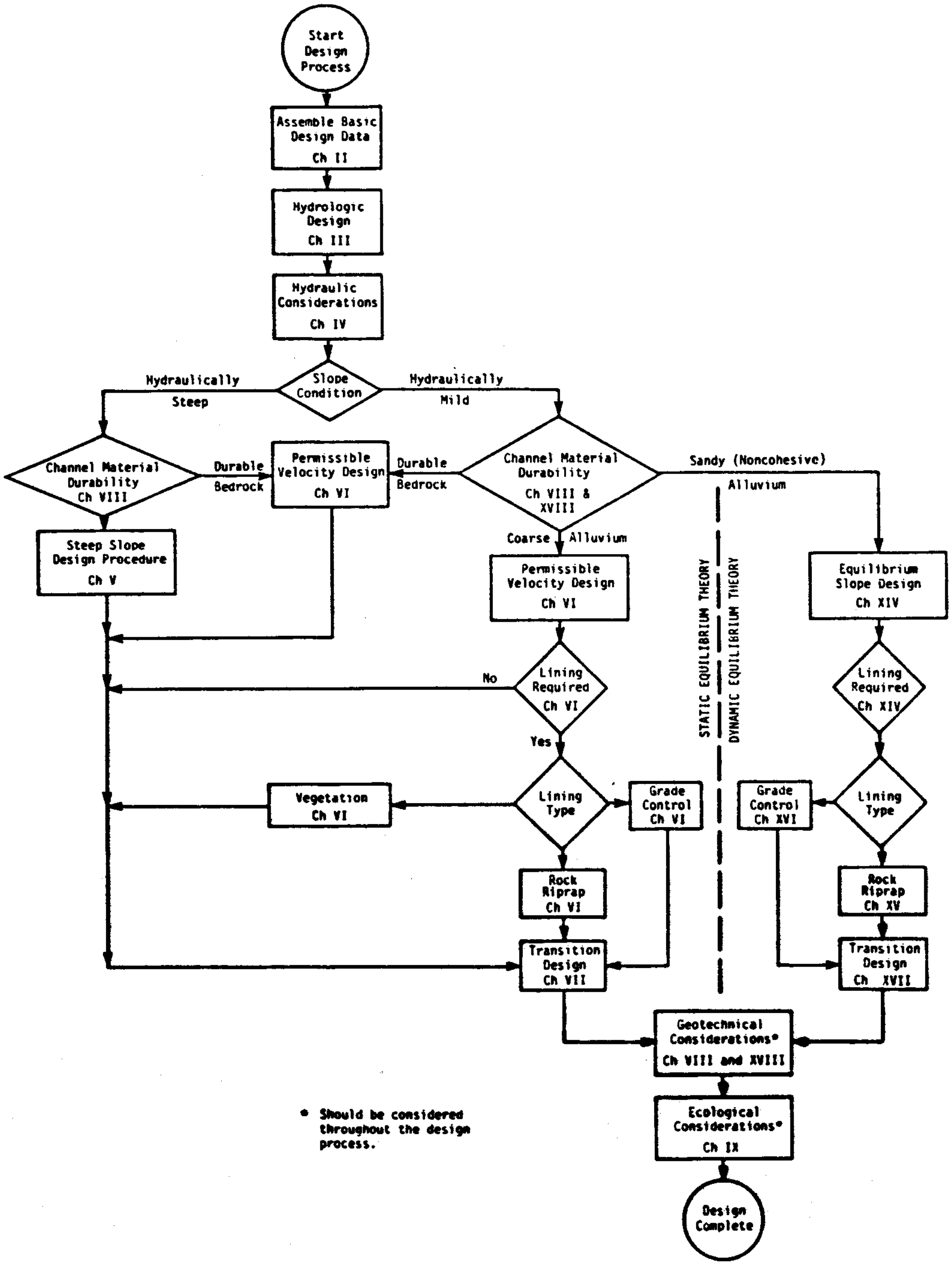


Figure 1.7. Flow chart illustrating design manual organization.

1.7 References

Appalachian Regional Commission, 1969, "Acid Mine Drainage in Appalachia," House Document 91-160.

Wilson, Lee, 1972, "Seasonal Sediment Yield Patterns of United States Rivers," Water Resources Research, Vol. 8, pp. 1470-1479.

This page intentionally left blank.

II. BASIC INFORMATION REQUIRED FOR DIVERSION DESIGN

Many case histories of poorly designed surface mine channels reveal the need for more thorough planning and design methodologies. The first step in accomplishing this is to consider diversions in the early stage of mine plan formulation and then effectively integrate them into the plan. Slight alterations in mine plans before implementation can often greatly enhance the usefulness of diversions and simplify their design; however, if diversions are viewed as devices which can be added when the need appears, their effectiveness and efficiency are jeopardized. Even when diversions are properly located from the point of view of effectiveness, many fail or perform poorly because a thorough understanding of all factors related to their design is absent. The following discussion provides insight to the initial data required for efficient diversion design. Application of this information to the design of diversion structures is illustrated in the following chapters.

Proper design of water diversion structures relies on a well-developed data base. The data can result from actual measurements or they can be synthesized by various computational techniques. Information required for diversion design can be classified into the categories of topographic, hydrologic, hydraulic, geotechnical and ecological.

Topographic data on slopes, contours, distances, areas, etc. are considered as preliminary data and are essential to any diversion design. Topographic data are used throughout the entire design process and must be accurate and readily available. The U.S. Geological Survey has mapped nearly the entire United States, and topographic maps of a given region are usually available. County maps and aerial photographs can also be of value.

Hydrologic data are required for the analysis and estimation of the design discharge. For minor structures, such as diversion channels, an estimate of the peak discharge for a given frequency is sufficient for design.

Hydraulic data are used to size the structure required to carry the design discharge. The basic principles of open-channel flow are used to determine specific hydraulic data such as flow depth, velocity, energy dissipation requirements, etc.

Geotechnical considerations govern the design of rock and soil sideslopes. Of particular importance is the stability of the soils where the diversion will be located. In steep slope areas, coordination between geotechnical and hydraulic engineers is essential to produce a stable channel that minimizes the long-term land disturbance and the potential for landslides. Ultimately, geotechnical considerations may govern the channel slope, shape and lining.

Ecological data are necessary to assess any potential adverse environmental impacts resulting from a given design. Whenever the waters of any stream are proposed to be impounded, diverted, channelized or otherwise controlled, consideration must be given to applicable state and federal laws regarding fish and wildlife habitats. Most state game and fish agencies have guidelines concerning stream improvement or restoration related to minimum flow standards, stream bottom materials and stream type (riffles and pools, etc.).

Table 2.1 provides general guidelines concerning the information required for design. Much of the data base for design is often available as a result of filing for mining permits and meeting other regulations.

Table 2.1. Possible Data Required for Channel Design.

<u>Topographic Data</u>

- Drainage area
- Stream slope
- Watershed slope
- Watershed shape
- Longitude
- Latitude
- Topographic maps
- Aerial photographs
- Land characteristics

<u>Hydrologic Data</u>

Precipitation:

- 2-year, 24-hour rainfall amount
- 10-year, 24-hour rainfall amount
- 100-year, 24-hour rainfall amount

<u>Hydraulic</u>

- Average velocity
- Boundary roughness
- Flow depth
- Top width
- Hydraulic radius
- Wetted perimeter
- Backwater profile
- Bedform configuration

<u>Geotechnical</u>

Soils:

- Type
- Structure
- Particle size
- Permeability
- Infiltration
- Percent organic matter
- Chemical composition
- Aggregate index
- Soil maps

Table 2.1 (continued)

Geology:

- Geographical region
- Rock formations

Ecological

- Fish and wildlife species present
- Water quality
- Water temperature
- Minimum flow requirements
- Siltation
- Streambed composition
- Vegetation
- Feeding areas
- Limitation of mobility
- General habitat requirements

III. HYDROLOGIC ANALYSIS

3.1 Introduction

A hydrologic analysis is used to establish the design flow of a hydraulic structure. For diversion structures it is adequate to estimate the magnitude of the peak flow or flood at the required frequency for a particular drainage area. The frequency or return periods for ephemeral, intermittent and perennial streams were given in Table 1.1 and are summarized in Table 3.1.

If an adequate floodplain exists, regulations allow designing the channel and floodplain to carry the required flow, provided the channel capacity is at least equal to the capacity of the unmodified stream channel immediately upstream and downstream of the diversion. Therefore, a reasonable approach in some cases is to compute the design discharge for a lesser event than stated in Table 3.1 and compare it to the estimated unmodified channel capacity. If the values are reasonably close, then it is realistic to design based on this value. If not, a larger event must be used.

Development of the entire runoff hydrograph required for some hydraulic structures is somewhat involved. However, estimation of peak discharge required for diversion channel design is generally easier and there are a number of relatively simple methods available for use. Determining the method to use depends on the available data and the applicability of a given relationship to the design conditions. For a gaged watershed, the estimate is made by a hydrologic analysis of the drainage area characteristics, climatic characteristics and the accumulated streamflow data; however, most smaller drainages associated with diversion design are ungaged and an estimate of the design flow must be made from limited topographic and climatic data.

The primary physical characteristics that must be considered in selecting an applicable method for surface mine operations in the Eastern Coal Province are steep slopes and relatively small watershed area. Many of the available methods for peak flow estimation have been developed for moderately sloped, large-area watersheds. No single method examined was exactly tailored to meet the requirements for surface mine operations. However, since most diversion structures are small engineering structures dealing with acreages typically less than 200, and located in rural areas, the application of any one of these methods is probably acceptable.

The recommended methods are the Rational Method and the method described in SCS TP-149. The Rational Method is probably the most commonly used method

Table 3.1. Hydrologic Recurrence Interval and Design Event.

	Overland Flows, Shallow Groundwater Flows Ephemeral Streams	Perennial and Intermittent Streams
Permanent	10-year, 24-hour	100-year, 24-hour
Temporary	2-year, 24-hour	10-year, 24-hour

in the Eastern Coal Province. The SCS TP-149 is considered to be a more accurate method, and yet a relatively simple alternative to the Rational Method. While only these two methods are described in this manual, it is expected that designers will apply sound judgement and where warranted, apply more complex methods. Designers frequently determine peak rates by more than method and use judgment for selection of the design value. In some states the regulatory agencies may even dictate the preferred method.

3.2 Rational Method

The Rational Method is a common method for peak flow estimation; however, it has many limitations that must be considered. These limitations are discussed by McPherson (1969) and others. The basic problem is the great oversimplification by the equation of a complicated runoff process; however, because of the simplicity of the Rational Method, it continues to be a widely used technique.

The Rational formula is

$$Q = C\ i\ A \tag{3.1}$$

where Q is the peak flow rate in cubic feet per second (cfs), C is a dimensionless coefficient, i is the rainfall intensity in inches per hour (iph) for the design return period, and A is the drainage area in acres. Values for i can be determined from U.S. Weather Bureau Technical Paper 40 (1961). Appendix A provides these figures for the geographical area of OSM Regions I and II and for the design return periods required in diversion design. To be dimensionally correct, a conversion factor of 1.008 should be included in Equation 3.1 to convert acre-inches per hour to cfs; however, this factor is generally neglected.

The assumptions used in developing the Rational Method are:

1. The rainfall occurs at a uniform intensity over the entire watershed.

2. The rainfall occurs at a uniform intensity for a duration equal to the time of concentration, (t_c).

3. The frequency of the runoff equals that of the rainfall used in the equation.

The time of concentration (t_c) is defined as the time required for water to flow from the most remote (in time of flow) point in the watershed to the point in consideration once the soil has become saturated and minor depressions are filled. To satisfy assumption 2, the time of concentration must be known to establish the proper value of intensity in formula 3.1. Accurately evaluating the time of concentration is one of the major problems in using the Rational formula. Table 3.2 presents t_c values for small watersheds based on a commonly used formula (Kirpich, 1940). Figures 3.1 and 3.2 provide nomographs for computing t_c that incorporate the character of the ground surface.

After estimating the time of concentration, the rainfall intensity for a duration equal to that time must be established. The U.S. Weather Bureau TP 40 charts in Appendix A are only for durations of 1 and 24 hours. The rainfall amount for a different duration can be established from these charts. If the duration established by the time of concentration is longer than one hour, the procedure is to plot the one- and 24-hour amounts for the given return period on semilog paper and interpolate by a straight line the required value. If the duration is less than one hour, the rainfall amount from the one-hour chart is multiplied by the conversion factor in Figure 3.3. The intensity in iph can then be established.

The coefficient C, called the runoff coefficient and defined as the ratio of the peak runoff rate to the rainfall intensity, is also difficult to determine. Studies have shown that C depends on the infiltration rate, surface cover, channel and surface storage, antecedent conditions, and rainfall intensity. The difficulty in lumping all these factors into one coefficient is apparent. If one C value is not applicable to the entire drainage area, an area weighted average can be established from

$$\bar{C} = \frac{\Sigma C_i A_i}{\Sigma A_i} \tag{3.2}$$

where Σ is the symbol for a summation and C_i is the coefficient applicable to the incremental area A_i. Numerous tables of C values for urban areas exist. Table 3.3 presents a table of C values developed for rural areas (Schwab et al., 1971). This table is the one found most applicable to surface mine conditions. In reviewing some of the mine plans submitted in OSM Regions I and II, it was observed that C values of 0.15 and 0.50 were commonly used for undisturbed and disturbed mine areas, respectively. According to Table 3.3, these values are reasonable only in relatively flat terrain.

Table 3.2. Time of Concentration for Small Watersheds[a]

Maximum Length of Flow (ft)	Time of Concentration (min) Watershed Gradient (percent) 0.05	0.1	0.5	1.0	2.0	5.0
500	17	13	7	6	4	3
1,000	30	23	12	9	7	5
2,000	51	39	21	16	12	9
4,000	86	66	36	27	21	15
6,000	118	90	49	37	29	20
8,000	147	113	61	46	36	25
10,000	175	134	72	55	42	30

[a]Computed from $t_c = 0.0078\ L^{0.77}\ S^{-0.385}$, where L is the maximum length of flow in feet, S is the watershed gradient in feet per foot, and T_c is the time concentration in minutes (Kirpich, 1940).

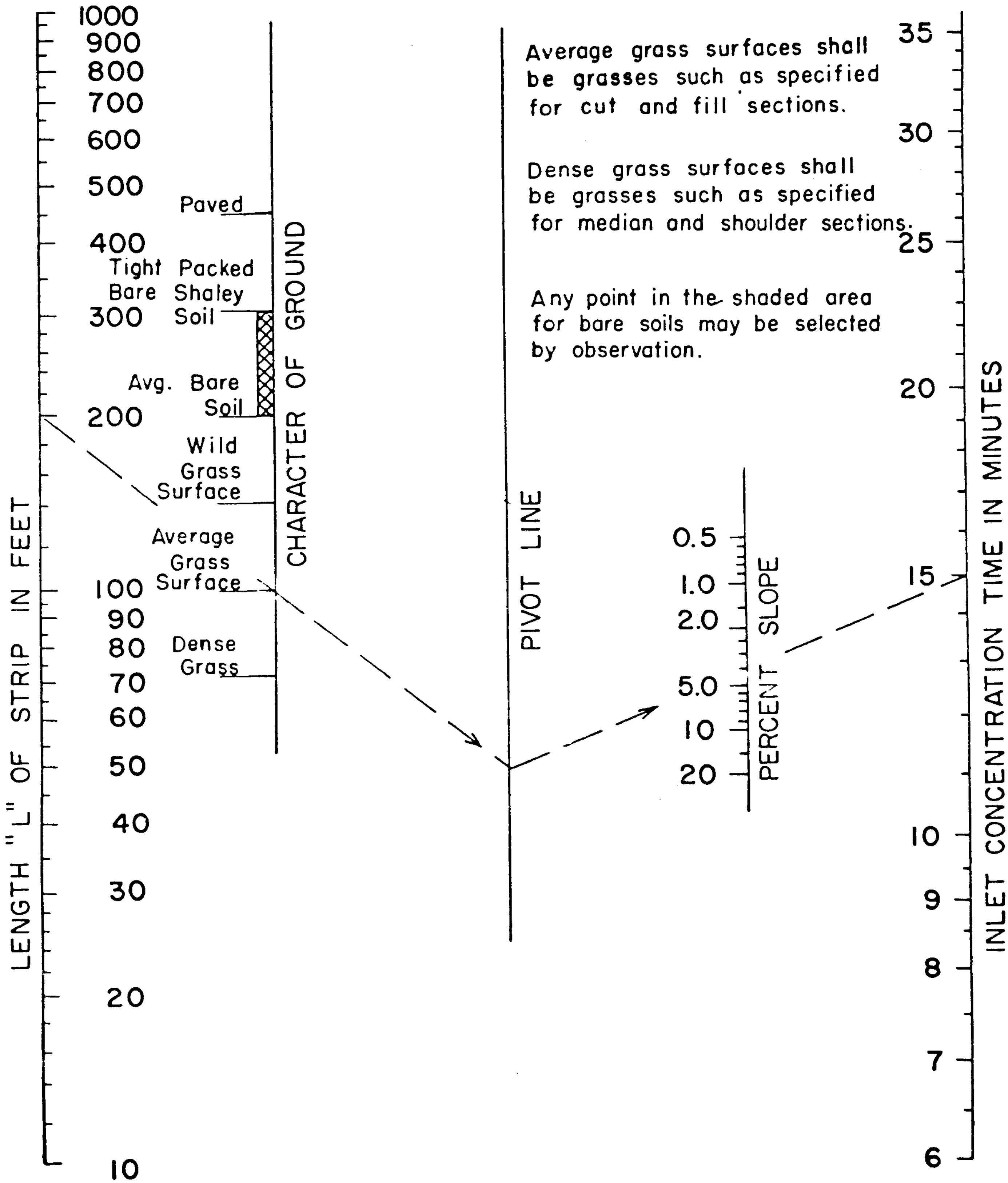


Figure 3.1. Nomograph for computing t_c (from West Virginia Department of Highways).

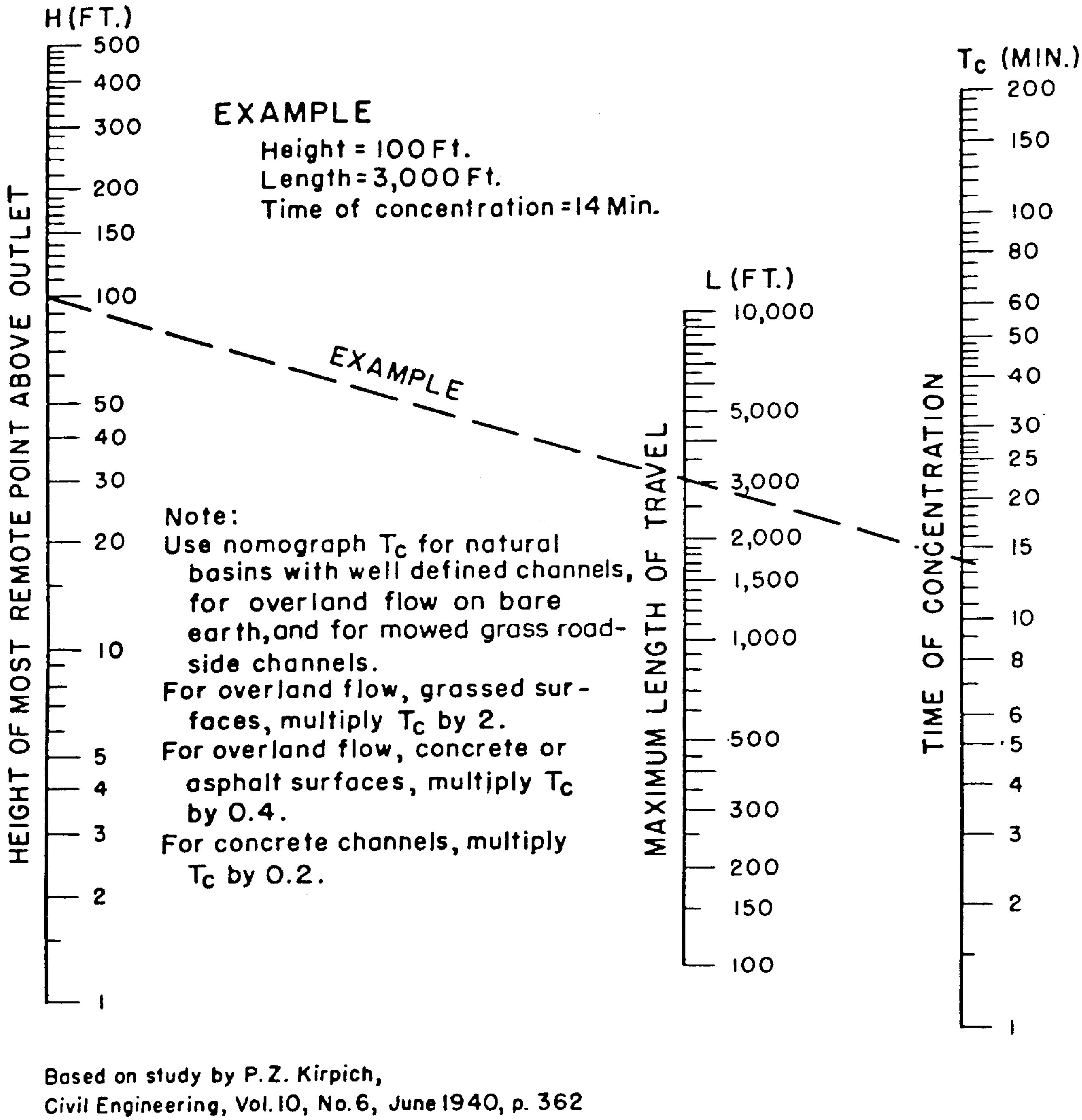


Figure 3.2. Time of concentration of small rural drainage basins (from West Virginia Department of Highways).

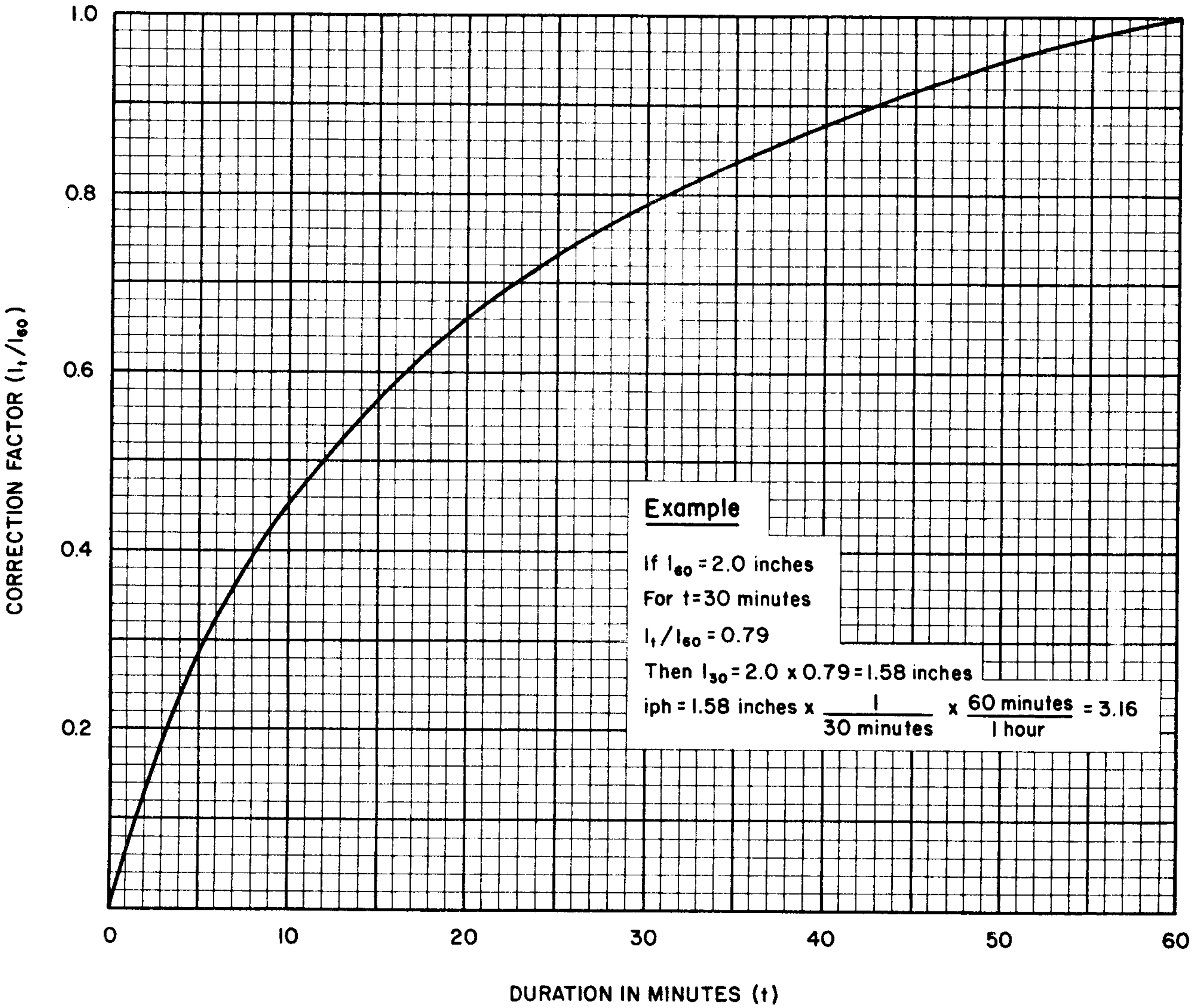


Figure 3.3. Conversion factors for durations less than one hour.

Table 3.3. Rational Runoff Coefficients (after Schwab et al., 1971).

Topography and Vegetation	Values of C in Q = CiA — Soil Texture: Open Sandy Loam	Clay and Silt Loam	Tight Clay
Woodland			
Flat 0-5% slope	0.10	0.30	0.40
Rolling 5-10% slope	0.25	0.35	0.50
Hilly 10-30% slope*	0.30	0.50	0.60
Pasture			
Flat	0.10	0.30	0.40
Rolling	0.16	0.36	0.55
Hilly	0.22	0.42	0.60
Cultivated			
Flat	0.30	0.50	0.60
Rolling	0.40	0.60	0.70
Hilly	0.52	0.72	0.82

*Values are not available for steeper slope conditions, so when applying the Rational Formula to steeper slopes this limitation must be realized.

Considerable care should be exercised in using the tabulated values to insure reasonable results. From a review of the table it is apparent that vegetation has a considerable influence on the peak runoff rate. A surface mine operation will, at least temporarily, change a substantial portion of the drainage basin vegetation type, for example from woodland to cultivated (disturbed). Therefore, a weighted C value will most often be required. Additionally, the designer must realize that the most critical period of channel stability is immediately after regrading, before the vegetation has become established. Not only are potential erosion rates the greatest, but also the peak runoff rate due to the lack of resistance to overland flow. Since there can be no assurance of when the design storm will occur, the designer may want to assume the worst condition to prevent potential failure of the drainagae system.

3.3 SCS TP-149 Method

Discrepancies between some of the commonly used methods of peak rate estimation under similar conditions prompted the SCS to develop the method presented in SCS TP-149 (Kent, 1968). The primary differences in results between the methods (i.e., Rational formula, Cook's Method, etc.) were due to the choice of coefficients and factors inherent in each method rather than to the method itself.

The development of the TP-149 method was based on the need to produce a reliable procedure for making quick, on-the-spot estimates of peak discharge rates. The method is a simplified graphical approach that is closely allied with the SCS NEH-4 procedure. The graphs were computer generated using average conditions for the various parameters and correction factors involved in NEH-4. Therefore, the peak discharge of a small watershed with unusual characteristics can be more accurately computed using NEH-4 if necessary. Graphs were prepared for two distributions of storm rainfall with time. Type I represents Hawaii, Alaska, and the coastal side of the Sierra Nevada and Cascade Mountains in California, Oregon and Washington. The Type II distribution represents the rest of the United States, Puerto Rico and the Virgin Islands. Therefore, most all surface mining areas in the United States are covered by the Type II graphs.

The graphs have also been prepared for curve numbers (CN) of 60, 65, 70, 75, 80, 85 and 90. Identifying the representative CN requires sound judgment based upon the specific information. Table 3.4 provides the association of CN's with various hydrologic soil-cover complexes for average antecedent moisture conditions (AMC II). Soils are divided into four hydrologic soil groups: A, B, C and D. The groups are included in NEH-4 (Soil Conservation Service, 1972). Group A soils have a high infiltration rate even when thoroughly wet. When thoroughly wet, group B soils have a moderate infiltration rate, group C soils have a slow infiltration rate, and group D soils have a very slow infiltration rate. Typically, most soils in the Eastern Coal Province are in the Type C group. Table 3.5 describes the properties of the four soil group classifications. NEH-4 lists more than 4000 soils and their hydrologic group classifications. For greater detail or accuracy in defining the CN, refer to NEH-4.

The graphs also involve average watershed slope as defined by flat, moderate or steep. Table 3.6 gives the values assumed in the computer generated graphs for TP-149 and the slope ranges assigned to each slope factor. If a closer estimate of peak discharge is required for a specific slope, curvilinear interpolation can be used between 1, 4 and 16 percent. Ordinarily the peak discharge values for one of the three slope categories will be adequate without interpolating between slope categories.

The method is generally limited to drainages less than 2000 acres and average slopes less than 30 percent. For watersheds exceeding these limits SCS recommends that the methods in NEH-4 be followed (Kent, 1968). Additionally, since the graphs were developed for typical or average conditions, a watershed with unusual characteristics should be evaluated by NEH-4. However, as previously mentioned, the method probably yields reasonable results for design in surface mine situations. Appendix B provides the required graphs for Type II rainfall distribution.

3.4 Calculation Procedures

3.4.1 Rational Formula

1. Estimate runoff coefficient C or an area-weighted C, from Table 3.3.
2. Evaluate the time of concentration t_c (min) from Table 3.2, Figure 3.1 or Figure 3.2, depending on available data.
3. Determine the rainfall intensity i (iph) for a duration equal to t_c. If t_c is less than one hour, obtain the one-hour rainfall amount for the design return period from the charts in Appendix A. Then evaluate

Table 3.4. Runoff Curve Numbers for Hydrologic Soil-Cover Complexes. [Antecedent Moisture Condition (AMC) II and $I_a = 0.25$]

Land Use and Treatment or Practice	Hydrologic Condition	Hydrologic Soil Group A	B	C	D
Fallow					
Straight row	----	77	86	91	94
Row crops					
Straight row	Poor	72	81	88	91
Straight row	Good	67	78	85	89
Contoured	Poor	70	79	84	88
Contoured	Good	65	75	82	86
Contoured and terraced	poor	66	74	80	82
Contoured and terraced	Good	62	71	78	81
Small grain					
Straight row	Poor	65	76	84	88
Straight row	Good	63	75	83	87
Contoured	poor	63	74	82	85
Contoured	Good	61	73	81	84
Contoured and terraced	Poor	61	72	79	82
Contoured and terraced	Good	59	70	78	81
Close-seeded legumes or rotation meadow					
Straight row	Poor	66	77	85	89
Straight row	Good	58	72	81	85
Contoured	Poor	64	75	83	85
Contoured	Good	55	69	78	83
Contoured and terraced	Poor	63	73	80	83
Contoured and terraced	Good	51	67	76	80
Pasture or range					
No mechanical treatment	Poor	68	79	86	89
No mechanical treatment	Fair	49	69	79	84
No mechanical treatment	Good	39	61	74	80
Contoured	Poor	47	67	81	88
Contoured	Fair	25	59	75	83
Contoured	Good	6	35	70	79
Meadow	Good	30	58	71	78
Woods	Poor	45	66	77	83
	Fair	36	60	73	79
	Good	25	55	70	77
Farmsteads	----	59	74	82	86
Roads[1]					
Dirt	----	72	82	87	89
Hard surface	----	74	84	90	92

[1] Including rights of way.

Table 3.5. Soil Conservation Service Soil Group Classifications.

Soil Group	Description
A	Lowest Runoff Potential. Includes deep sands with very little silt and clay, also deep, rapidly permeable loess.
B	Moderately Low Runoff Potential. Mostly sandy soils less deep than A, and loess less deep or less aggregated than A, but the group as a whole has above-average infiltration after thorough wetting.
C	Moderately High Runoff Potential. Comprises shallow soils and soils containing considerable clay and colloids, though less than those of Group D. The group has below-average infiltration after presaturation.
D	Highest Runoff Potential. Includes mostly clays of high swelling percent, but the group also includes some shallow soils with nearly impermeably subhorizons near the surface.

From U.S. Soil Conservation Service, National Engineering Handbook, Hydrology, Section 4, Part I, Watershed Planning (1964).

Table 3.6. Slope Factors for Peak Discharge Computations in the TP-149 Method.

Slope Factor	Slope for Which Computations Were Made (%)	Average Slope Range (%)
Flat[1]	1	0 to 3
Moderate	4	3 to 8
Steep	16	8 or more

[1]Level to nearly level.

the required correction factor from Figure 3.3 to obtain the rainfall amount for the t_c. The intensity i required in the Rational Formula is then

$$iph = t_c \text{ rainfall amount (in)} \times \frac{1}{t_c \text{ (min)}} \times \frac{60 \text{ min}}{1 \text{ hr}}$$

If t_c is greater than one hour, obtain the one- and 24-hour rainfall amounts for the design return period and evaluate the amount for t_c by interpolation on semilog paper. Compute the iph as described above.

4. Establish the contributing drainage area A (acres).
5. Evaluate Q = CiA (cfs).

3.4.2 SCS TP-149 Method

1. Estimate the curve number (CN) from Table 3.4.
2. Determine the 24-hour duration rainfall for the required return period from the charts in Appendix A.
3. Enter the appropriate chart in Appendix B to determine Q in cfs.

3.5 Example Using Step-By-Step Procedures Outlined Above

From a topographic map the drainage area is estimated as 70 acres and the maximum flow length is 2750 ft. The terrain is hilly with an average 20 percent slope and consists primarily of woodland with a clay and silt loam soil. What is the discharge for the ten-year event near Charleston, West Virginia?

3.5.1 Rational Formula

1. From Table 3.3 C = 0.50.
2. From formula given in Table 3.2,

$$t_c = 0.0078\ (2750)^{0.77}\ (0.20)^{-0.385}$$

$$t_c = 6.5 \text{ min.}$$

3. From Appendix A the rainfall amount for the ten-year, one-hour event is 2.0 inches. From Figure 3.3 the correction factor for duration equal to 6.5 minutes is 0.30.

$$I_{6.5} = 2.0\ (0.30) = 0.60 \text{ inches}$$

$$\text{iph} = 0.60 \text{ inches} \times \frac{1}{6.5 \text{ min}} \times \frac{60 \text{ min}}{1 \text{ hour}} = 5.5 \text{ iph}$$

4. Contributing area given as 70 acres.

5. From $Q = CiA$

$= 0.50\ (5.5)\ (70)$

$Q_p = 192$ cfs

3.5.2 SCS TP-149 Method

1. For group C soils, the CN from Table 3.4 is 73 for woods in fair condition.

2. From Appendix A, the ten-year, 24-hour event is 4.2 inches.

3. From charts in Appendix B,

for CN = 70 $Q_p = 90$ cfs

CN = 75 $Q_p = 130$ cfs

By linear interpolation, CN = 73 $Q_p = 114$ cfs.

3.6 References

Kent, K. M., 1968, "A Method for Estimating Volume and Rate of Runoff in Small Watersheds," USDA SCS TP-149, January.

Kirpich, P. Z., 1940, "Time of Concentration of Small Agricultural Watersheds," Civil Engineering, 10.

McPherson, M. B., 1969, "Some Notes in the National Method of Storm Drainage Design," Technical Memorandum No. 6, ASCE Urban Water Resources Program, ASCE.

Schwab, G. O., R. K. Frevert, T. W. Edminster, K. K. Barnes, 1966. Soil and Water Conservation Engineering, John Wiley and Sons, Inc., New York.

U.S. Weather Bureau, 1961, "Rainfall Frequency Atlas of the United States," Technical Paper No. 40, May.

U.S. Soil Conservatin Service, 1964, "Hydrology," National Engineering Handbook, Section 4, Part I, Watershed Planning.

U.S. Soil Conservation Service, 1972, "Hydrology," National Engineering Handbook, Section 4, August.

West Virginia Department of Highways, 1963, Drainage Manual, developed by the Design Division, June.

This page intentionally left blank.

IV. BASIC CONCEPTS OF OPEN-CHANNEL FLOW

4.1 Introduction

Understanding the basic concepts of open-channel flow is necessary to properly design the channels required on surface mine operations. In open-channel flow, the water surface is not confined. Surface configuration, flow pattern and pressure distribution within the flow depend on gravity. In rigid-boundary open-channel flow, no deformations or movements of the bed and banks are considered, whereas in mobile-boundary hydraulics the bed configuration depends on the flow. Discussions in this chapter pertain primarily to rigid-boundary open-channel flow, since they are most applicable to diversion channel design in the Eastern Coal Province. Part II presents the movable boundary hydraulics necessary for diversion channel design in sandy soils. For greater detail than what is presented here, refer to any basic fluid mechanics textbook.

4.2 Parameters Describing the Hydraulics of Open-Channel Flow

4.2.1 General

All variables used in fluid mechanics and hydraulics fall into one of three classes: those describing the boundary geometry, those describing the flow, and those describing the fluid (Rouse, 1976). Various combinations of these variables define parameters that describe the state of flow in open channels. Understanding these variables and parameters is necessary background knowledge to future discussions of equations and formulas applicable to open-channel flow. Some of the more common variables and parameters are defined below.

4.2.2 Variables Describing the Boundary Geometry

1. Depth of Flow: The depth of flow d is defined as the perpendicular distance from the bed of the stream to the water surface. For channels on mild slopes the depth of flow is often approximated by the vertical distance from the bed.

2. Stage: The stage h is the vertical distance from any selected and defined datum to the water surface.

3. Top Width: The top width T is the width of a stream section at the water surface and it varies with stage in most natural channels.

4. Cross-Sectional Area: The cross-sectional area A is the area of a cross section of the flow normal to the direction of flow.

5. Wetted Perimeter: The wetted perimeter P is the length of wetted cross section normal to the direction of flow.

6. Hydraulic Radius: The hydraulic radius R is the ratio of the cross-sectional area to wetted perimeter,

$$R = A/P \tag{4.1}$$

7. Hydraulic Depth: The hydraulic depth d_h is the ratio of the cross-sectional area to the top width,

$$d_h = A/T \tag{4.2}$$

8. Water Surface Slope: The slope of the water surface or hydraulic gradient is denoted by S_w.

9. Slope of the Energy Grade Line: The energy grade line is a graphical representation with respect to a selected datum of the total head or energy possessed by the fluid. For an open channel, the energy gradient is located a distance $V^2/2g$ above the free water surface. The slope of the energy grade line is designated by the symbol S_f or S_E.

10. Bed Slope: The bed slope S_b is the longitudinal slope of the channel bed.

General formulas for determining area, wetted perimeter, hydraulic radius, and top width in trapezoidal, rectangular, triangular, circular, and parabolic sections are given in Table 4.1 (Soil Conservation Service, 1954).

4.2.3 Variables Describing the Flow

1. Discharge: The discharge Q is the volume of a fluid or solid passing a cross section of a stream per unit time.

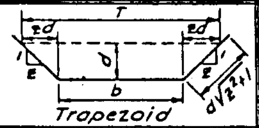
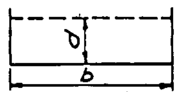
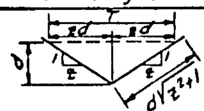
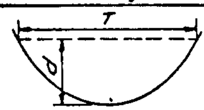
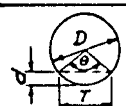
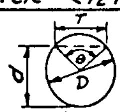
2. Mean Velocity: The mean velocity $V = Q/A$ is the discharge divided by the area of the water cross section.

3. Drag Force: The drag force F_d is the force component exerted by a moving fluid on any object submerged in the fluid. The direction of the force is the same as that of the free stream of fluid.

4. Lift Force: The lift force F_L is the force component exerted on a body submerged in a moving turbulent fluid. The force acts in the direction normal to the free stream of fluid.

5. Shear Force: The shear force is the shear developed on the wetted area of the channel and it acts in the direction of flow. This force per unit wetted area is called the shear stress τ_o and can be expressed as

Table 4.1. Elements of Channel Sections (from Soil Conservation Service, 1954).

Section	Area a	Wetted Perimeter p	Hydraulic Radius r	Top Width T
 Trapezoid	$bd+zd^2$	$b+2d\sqrt{z^2+1}$	$\frac{bd+zd^2}{b+2d\sqrt{z^2+1}}$	$b+2zd$
 Rectangle	bd	$b+2d$	$\frac{bd}{b+2d}$	b
 Triangle	zd^2	$2d\sqrt{z^2+1}$	$\frac{zd}{2\sqrt{z^2+1}}$	$2zd$
 Parabola	$\frac{2}{3}dT$	$T+\frac{8d^2}{3T}$ [1]	$\frac{2dT^2}{3T^2+8d^2}$ [1]	$\frac{3a}{2d}$
 Circle - < ½ full [2]	$\frac{D^2}{8}\left(\frac{\pi\theta}{180}-\sin\theta\right)$	$\frac{\pi D\theta}{360}$	$\frac{45D}{\pi\theta}\left(\frac{\pi\theta}{180}-\sin\theta\right)$	$D\sin\frac{\theta}{2}$ or $2\sqrt{d(D-d)}$
 Circle - > ½ full [3]	$\frac{D^2}{8}\left(2\pi-\frac{\pi\theta}{180}+\sin\theta\right)$	$\frac{\pi D(360-\theta)}{360}$	$\frac{45D}{\pi(360-\theta)}\left(2\pi-\frac{\pi\theta}{180}+\sin\theta\right)$	$D\sin\frac{\theta}{2}$ or $2\sqrt{d(D-d)}$

[1] Satisfactory approximation for the interval $0 < \frac{d}{T} \leq 0.25$
When $d/T > 0.25$, use $p = \frac{1}{2}\sqrt{16d^2+T^2} + \frac{T^2}{8d}\sinh^{-1}\frac{4d}{T}$

[2] $\theta = 4\sin^{-1}\sqrt{d/D}$
[3] $\theta = 4\cos^{-1}\sqrt{d/D}$
Insert θ in degrees in above equations

$$\tau_o = \gamma RS \tag{4.3}$$

where γ is the specific weight, R is the hydraulic radius, and S is a representative slope.

4.2.4 Variables Describing the Fluid

1. Density: The density ρ (kg-sec^2/m^4 or lb-sec^2/ft^4) of a fluid or solid is the mass that possesses per unit volume. Both the density of the water-sediment mixture and the density of sediment are important variables.

2. Specific Weight: The specific weight γ (kg/m^3, T/m^3, lb/ft^3) is the weight per unit volume. It is related to the density by

$$\gamma = \rho g \tag{4.4}$$

where g is the gravitational acceleration in m/sec^2, ft/sec^2.

3. Specific Gravity: The specific gravity G_s is the ratio of the specific weight of a fluid, solid or fluid-solid mixture, to the specific weight of water at 4° C or 39.2° F.

4. Viscosity: Viscosity is the property of a fluid that resists relative motion and deformation in the fluid and causes internal shear. Therefore, viscosity is a property exhibited only under dynamic conditions. According to Newton, the shear τ at a point within a fluid is proportional to the velocity gradient du/dy at that point or

$$\tau = \mu \frac{du}{dy} \tag{4.5}$$

where μ, in kg-sec/m^2 or lb-sec/ft^2, is the dynamic viscosity. When divided by the density ρ, it is the kinematic viscosity $\nu = \mu/\rho$ in m^2/sec or ft^2/sec. Under ordinary conditions of pressure, viscosity varies only with temperature. The viscosity of a liquid decreases with increasing temperature; the reverse is true for gasses.

4.2.5 Parameters Describing Open-Channel Flow

1. Reynolds Number: The Reynolds number is

$$R_e = \frac{VL}{\nu} \tag{4.6}$$

where V is the velocity, L is a characteristic length and ν is the kinematic viscosity. The Reynolds number relates the inertia forces to the viscous forces and is usually involved wherever viscosity is important, such as in slow movement of fluid in small passages or around small objects.

2. Froude Number: The Froude number is

$$F_r = \frac{V}{\sqrt{gL}} \tag{4.7}$$

where g is the gravitational acceleration. The Froude number relates the inertia forces to the gravitational effects and is important wherever the gravity effect is dominating, such as with water waves, flow in open channels, sedimentation in lakes and reservoirs, salt water intrusions, and the mixing of air masses of different specific weights. If the Froude number is less than one, equal to zero, or greater than one, the flow is defined as subcritical, critical and supercritical, respectively. Additionally, if the Froude number is less than one, the slope is considered hydraulically mild and when it is greater than one it is considered hydraulically steep.

4.2.6 Parameters Describing Boundary Roughness Conditions

1. Relative Roughness: The relative roughness is

$$k/R \tag{4.8}$$

where k is the height of the roughness element and R is the hydraulic radius. The inverse of the relative roughness is often encountered in resistance formulas.

2. Particle Size: The boundary of a natural earthen channel or a rock riprap channel is composed of a variety of particle sizes. The size distribution of these particles is often measured and expressed as the particle diameter for which a given percentage of the mixture is finer. For example, the D_{50} is a common measure of the representative particle size of the channel and it is equal to the minimum diameter for which 50 percent of the sediment mixture is finer.

4.3 Governing Equations

In rigid-boundary open-channel flow the equations of continuity, energy and momentum govern all flow processes. For diversion channel design the continuity and energy equations are most often applied.

The continuity equation in its simplest form is

$$Q = V_1A_1 = V_2A_2 \tag{4.9}$$

where Q is the discharge in cfs and V is the mean velocity in the cross section of area A for locations 1 and 2. This equation applies only to steady, two-dimensional, incompressible flow. These conditions are assumed to exist in most open-channel design procedures.

The most common form of the energy equation for open-channel flow is the Bernoulli equation

$$\frac{V_1^2}{2g} + \frac{P_1}{\gamma} + Z_1 = \frac{V_2^2}{2g} + \frac{P_2}{\gamma} + Z_2 + h_L \tag{4.10}$$

where V is the mean velocity, P is the hydrostatic pressure and Z is the elevation of the channel bed (relative to datum) at sections 1 and 2. The head loss term h_L represents the loss of energy by friction from Sections 1 to 2. The terms of the Bernoulli equation are commonly referred to as the velocity head, pressure head and elevation head, respectively.

The important concepts of the energy grade line and hydraulic grade line are encompassed in the Bernoulli equation. The energy grade line was defined in Section 4.2.2, item 10, as a graphical representation of the total energy possessed by the flow. Therefore, at any section the elevation of the energy grade line (EGL) relative to a datum is

$$EGL = \frac{V^2}{2g} + \frac{P}{\gamma} + Z \tag{4.11}$$

From Section 1 to Section 2 the energy grade line slopes downward by an amount equal to the head loss h_L between the two sections.

The hydraulic grade line lies below the energy line a distance equal to the velocity head. Therefore, the elevation of the hydraulic grade line (HGL) at any section is

$$HGL = \frac{P}{\gamma} + Z \tag{4.12}$$

which is simply the water-surface elevation relative to a datum. These concepts are illustrated in Figure 4.1.

In many cases the objective of hydraulic computations in open channels is to determine the curve of the water surface. These problems involve three general relationships between the hydraulic gradient and the energy gradient. For uniform flow the hydraulic gradient and the energy gradient are parallel and the hydraulic gradient becomes an adequate basis for the determination of friction loss, since no conversion between kinetic and potential energy is involved. In accelerated flow where velocity is increasing in the downslope direction, the hydraulic gradient is steeper than the energy gradient. This flow condition typically exists in steep slope diversions. In retarded flow

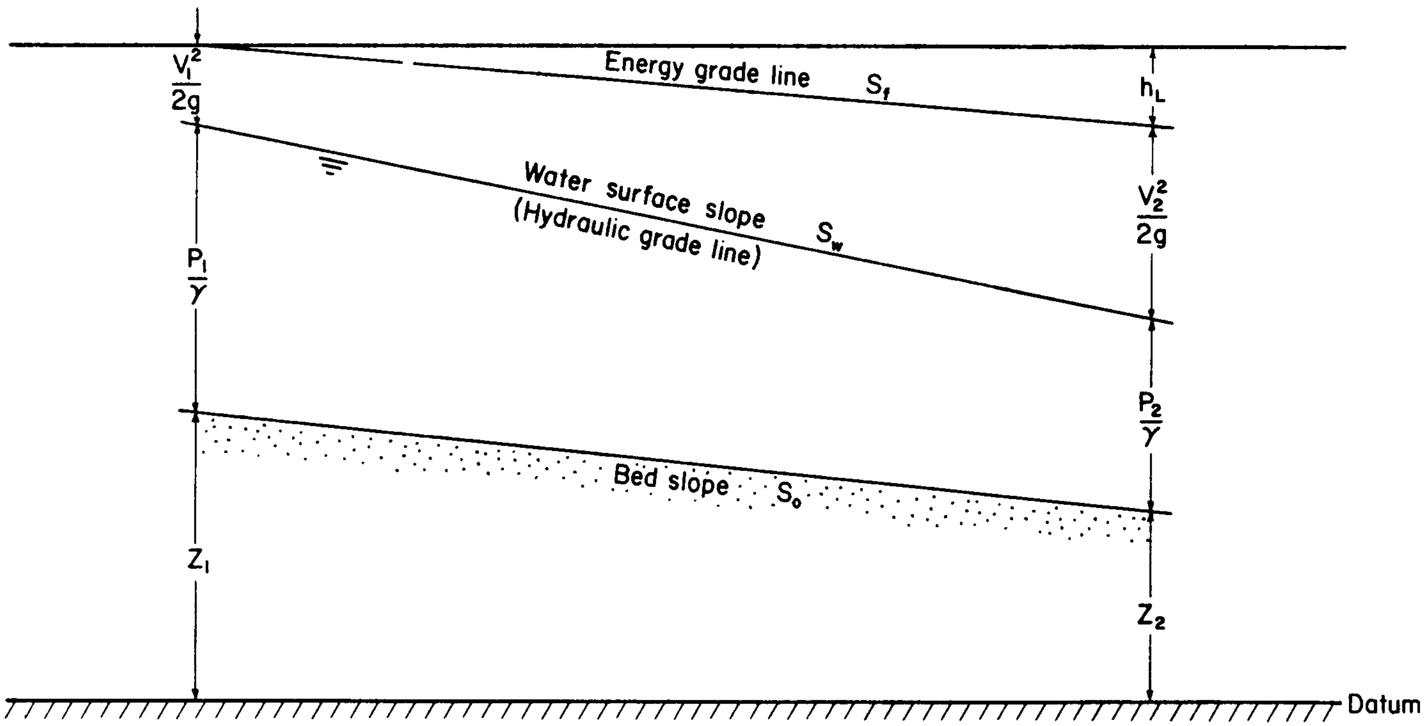


Figure 4.1. Definition sketch of the energy and hydraulic grade lines in open-channel flow.

where velocity is decreasing in the downslope direction, the energy gradient is steeper than the hydraulic gradient. An adequate anlaysis of flow under both these conditions cannot be made without consideration of both the energy and the hydraulic gradient.

4.4 Steady and Uniform Flow Formulas for Open Channels - The Manning Equation

Many formulas have been proposed to determine the mean characteristics of flow. However, the Manning relation remains the most commonly used. The Manning Equation in English units is

$$V = \frac{1.49}{n} R^{2/3} S^{1/2} \tag{4.13}$$

where n is defined as the Manning roughness coefficient with the dimension $L^{1/6}$. Since the flow rate (discharge) of a stream is defined by the continuity equation as

$$Q = VA \tag{4.9}$$

where Q is the discharge, V is the mean velocity and A is the cross-sectional area normal to the flow, the Manning equation can be expressed as

$$Q = \frac{1.49}{n} A R^{2/3} S^{1/2} \tag{4.14}$$

Because of simplicity of form and satisfactory results for practical applications, the Manning formula has become one of the most widely used of all open-channel uniform-flow formulas.

The depth of a uniform flow in an open channel is the normal depth. Although uniform flow seldom occurs in nature, most hydraulic computations are simplified and approximated by assuming uniform flow conditions. Therefore, the Manning Equation must often be solved for the normal depth d. However, since both velocity (or discharge) and the hydraulic radius depend on the flow depth, the equation cannot be directly solved for the normal depth. Many charts and nomographs have been developed to solve the Manning equation. Additionally, procedures have been developed for small programmable calculators providing efficient solution. Charts given in Appendix C provide solution of the Manning equation for trapezoidal channels of 2:1 side slope and bottom widths of 2, 4, 6, 8, 10, 12 and 14 feet. For additional charts see

"Design Charts for Open Channel Flow" by the U.S. Department of Transportation (1979).

4.5 Resistance to Flow

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

1. The Darcy-Weisbach friction factor, f
2. The Manning roughness coefficient, n
3. The Chezy resistance factor, C

A study of friction factors in open channels by the ASCE Task Force Committee (1963) provided specific recommendations on the use of these three resistance parameters. The results of the study are summarized below.

The Darcy-Weisbach f has great utility in expressing resistance to steady, fully developed flow in uniform channels. Experimental measurements of friction in open channels over a wide range of conditions appear to be better correlated and understood through the use of f. Additionally, f is commonly used in other branches of engineering, particularly closed conduit flow, and therefore provides a basis for pooling all experience and knowledge on frictional resistance. The friction factor f may be defined from

$$S = \frac{f}{4R}\frac{V^2}{2g} \tag{4.15}$$

where S is the slope of the hydraulic gradeline and channel bed, R is the hydraulic radius, V is the mean velocity and g is the acceleration of gravity. The dimensionless friction factor f depends on the Reynolds number and bed roughness, and therefore must be evaluated separately for each open-channel flow condition. The relationships between f, n and C are

$$C = \sqrt{\frac{8g}{f}} \tag{4.16}$$

$$n = 1.49\, R^{1/6} \sqrt{\frac{f}{8g}} \qquad \text{(English units)} \tag{4.17}$$

The Manning n has traditionally been widely used to evaluate resistance in open-channel flows. The ASCE report indicates that, when applied with judgment, n and f are probably equally effective in the solution of practical problems. The following recommendations were given for obtaining specific values of f or n for design purposes:

1. For roughnesses typical of those found in pipe flow (i.e. concrete), pipe resistance diagrams (i.e. Moody-type diagram) may be used to estimate f if the pipe diameter D is replaced by 4R.

2. For roughnesses found in unlined channels and for high Reynolds numbers, f (as defined by Equation 4.11) is independent of the Reynolds number and depends only on the hydraulic radius, R. Under these conditions f is nearly proportional to $1/R^{1/3}$, and Manning's n (as defined by Equation 4.13) is nearly constant. For this "fully rough" condition the constant values of Manning's n can be taken from the literature, such as Chow's book (1959). A detailed listing of values applicable to surface mine conditions in OSM Regions I and II is given in Appendix C. A shorter listing of values is given in Table 4.2. If desired, the value of f may then be computed. (It should be noted that not all turbulent flows are "fully rough.")

3. For other than fully rough flow, f or n may be larger or smaller than for the fully rough case. Therefore, caution should be used when using the Manning formula and n to insure that fully rough conditions exist.

4. For movable boundary conditions none of the formulas or methods discussed apply. However, the concept of expressing resistance with a friction factor is still valid. For a movable boundary condition, the fixed-bed friction factor may be increased by the bed form (pattern) that develops or decreased by the sediment carried in suspension. The final estimate of roughness relies greatly on individual judgment. Simons and Senturk (1976) review some of the available methods for estimating roughness in a movable boundary.

For most situations encountered in diversion channel design, the Manning n is the easiest and most appropriate estimate of flow resistance. A useful formula for evaluating n based on data from laboratory channels to large rivers is (Highway Research Board, 1970)

$$n = 0.0395\, D_{50}^{1/6} \tag{4.18}$$

where D_{50} is in feet. Grain sizes used in developing this formula ranged from 0.001 ft to nearly 1.0 ft.

4.6 Selection of Channel Cross Section

Typical channel cross sections are triangular, trapezoidal and parabolic (see Table 4.1). A triangular channel is a special type of trapezoidal with a bottom width of zero, and their application is limited to relatively low flow conditions. Trapezoidal channels of varying bottom widths and side slopes are

Table 4.2. Manning's Coefficients of Channel Roughness.

Constructed Channel Condition	Values of n Minimum	Maximum	Average
Earth channels, straight and uniform	0.017	0.025	0.0225
Dredged earth channels	0.025	0.033	0.0275
Rock channels, straight and uniform	0.025	0.035	0.033
Rock channels, jagged and irregular	0.035	0.045	0.045
Concrete lined, regular finish	0.012	0.018	0.014
Concrete lined, smooth finish	0.010	0.013	--
Grouted rubble paving	0.017	0.030	--
Corrugated metal	0.023	0.025	0.024

Natural Channel Condition	Value of n
Smoothest natural earth channels, free from growth with straight alignment.	0.017
Smooth natural earth channels, free from growth, little curvature.	0.020
Average, well-constructed, moderate-sized earth channels in good condition.	0.0225
Small earth channels in good condition, or large earth channels with some growth on banks or scattered cobbles in bed.	0.025
Earth channels with considerable growth, natural streams with good alignment and fairly constant section, or large floodway channels well maintained.	0.030
Earth channels considerably covered with small growth, or cleared but not continuously maintained floodways.	0.035
Mountain streams in clean loose cobbles, rivers with variable cross section and some vegetation growing in banks, or earth channels with thick aquatic growths.	0.050

Table 4.2 (continued)

Natural Channel Condition	Value of n
Rivers with fairly straight alignment and cross section, badly obstructed by small trees, very little underbrush or aquatic growth.	0.075
Rivers with irregular alignment and cross section, moderately obstructed by small trees and underbrush.	0.100
Rivers with fairly regular alignment and cross section, heavily obstructed by small trees and underbrush.	0.100
Rivers with irregular alignment and cross section, covered with growth of virgin timber and occasional dense patches of bushes and small trees, some logs and dead fallen trees.	0.125
Rivers with very irregular alignment and cross section, many roots, trees, large logs, and other drift on bottom, trees continually falling into channel due to bank caving.	0.200

the most commonly constructed channels. Parabolic channels are generally used only when a vegetated lining is required, although other cross sections are also used in vegetated channels.

The selection of the channel sideslope largely depends on the angle of repose of the parent material or channel lining. The angle of repose is the slope angle formed by particulate material under the critical equilibrium condition of incipient sliding (Simons and Senturk, 1977). For a stable channel the side slope must be smaller than the angle of repose. If θ is the side slope angle of the channel design and ϕ is the angle of repose, then a general relation for the maximum side slope angle for a stable channel is $\theta \leq \phi - 5$ (degrees). The relationship between the side slope angle θ and the side slope value z is depicted in Figure 4.2. Table 4.3 lists suggested z values to be used in designing channels. For channels lined with riprap, the angle of repose can be determined from Figure 4.3. In this case, the suggested value for z must be less than the angle of repose of the riprap.

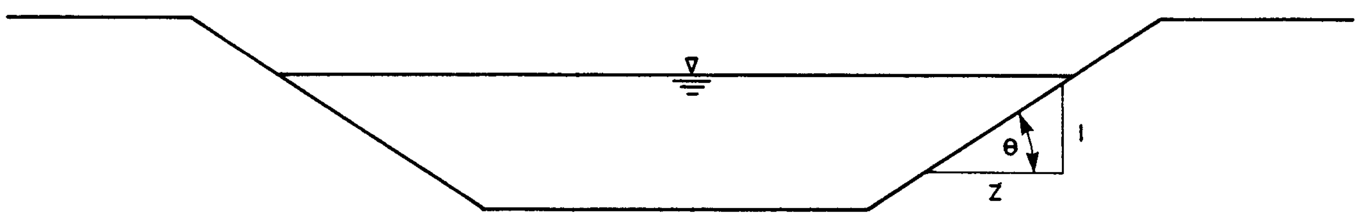
The selection of the channel bottom width for trapezoidal channels depends on the hydraulic conditions and the available equipment for construction. Specific guidelines or recommendations for selection of the channel cross section are given in later sections.

4.7 Variances in Flow Conditions

In open-channel flow, problems have been encountered with maintaining the flow within the designed channel, that is, the flow overtops the channel lining, resulting in serious erosion problems and possible failure. Many of these problems can be prevented when design consideration is given to (1) centrifugal forces that occur where the channel is turned and (2) additional channel depth to account for debris accumulation or variance in construction that results in differences in roughness coefficients. These two design concepts are known as superelevation and freeboard. For smaller channels the freeboard is often sufficient to account for centrifugal forces and superelevation need not be considered.

4.7.1 Superelevation

Because of the change in flow direction that results in centrifugal forces, there is a superelevation of the water surface in river bends (Figure 4.4). The water surface is higher at the concave bank and lower at the convex



$$\tan \theta = \frac{1}{z}$$

Figure 4.2. Relationship between side slope value, z and slope angle, Θ.

Table 4.3. Suggested Sideslope z Values.

Nature of Bank Material	z
Rock	0.2
Smooth or weathered rock, shell	0.5 ~ 1.0
Soil (clay, silt and sand mixtures)	1.5
Sandy soil	1.5
Silt and loam (loose sandy earth)	2
Fine sand	3
Flowing fine and other very fine material	>3
Compacted clay	1.5

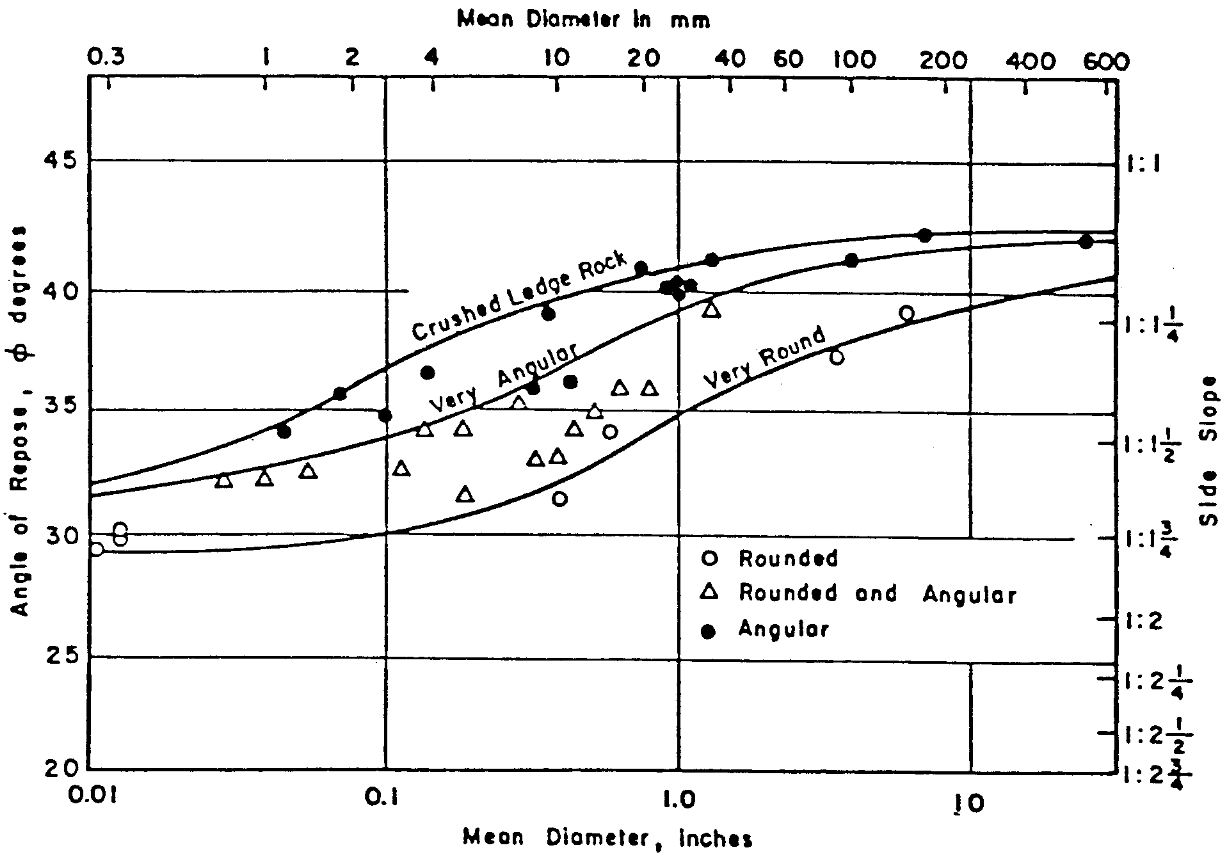


Figure 4.3. Angle of repose.

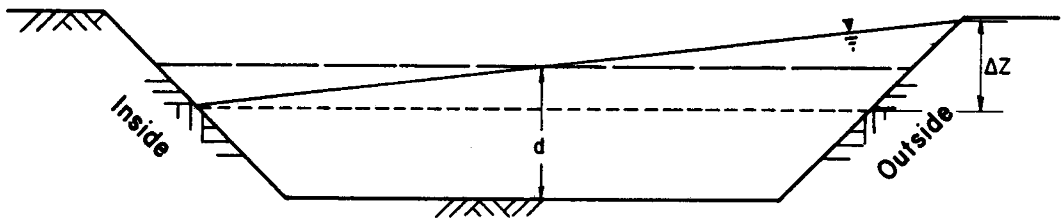


Figure 4.4. Definition sketch of superelevation in a channel bend.

bank than what the water surface would be under uniform conditions. The resulting transverse slope can be evaluated quantitatively. Several different equations have been proposed for evaluating superelevation. However, the differences between results by using the various equations is not significant. Therefore, one of the simpler procedures is suggested. Woodward (1920) assumed V equal to the average velocity Q/A and r equal to the radius to the center of the stream r_c and obtained

$$\Delta Z = Z_o - Z_i = \frac{V^2}{gr_c}(r_o - r_i) \tag{4.19}$$

in which Z_i and r_i are the water surface elevation and the radius at the inside of the bend, and Z_o and r_o are the water surface elevation and the radius at the outside of the bend. The value $(r_o - r_i)$ can be taken as the top width W of the channel.

4.7.2 Freeboard

Freeboard is the vertical distance from the water-surface elevation of the design flow to the top of the channel. Freeboard is used as a safety measure to prevent overtopping as a result of sedimentation, additional depth due to a rougher friction coefficient than used in the design, or wave action. The freeboard for a channel will depend on a number of factors such as size of channel, velocity of water, channel curvature, and transition conditions. In normal channel designs, the wave action due to wind is not usually significant. Freeboard is commonly defined as a percentage of the depth of flow. The Soil Conservation Service (1977) recommends that freeboard for trapezoidal channels at subcritical flow (mild slope) should be equal to or greater than 20 percent of the flow depth at the design discharge, but not less than one foot. For supercritical flow (steep slope) the recommended value is 25 percent of flow depth. These values are in addition to any other increase in channel depth required for superelevation or extremely turbulent flow. Therefore, for riprap-lined channels using large rock, the values should be greater due to anticipated turbulent flow conditions. The recommended freeboard for diversion channels on a surface mine operation is

$$F.B. = c_{fb}d + \frac{1}{2}\Delta Z \tag{4.20}$$

where c_{fb} is a coefficient defined according to Table 4.4 and ΔZ is defined in Figure 4.4. In all cases the recommended minimum freeboard is 1.0 foot (Soil Conservation Service minimum) plus one-half superelevation. The 1.0 foot minimum is greater than the 0.3 feet minimum specified in OSM Regulations. However, the regulations state that if necessary, the design freeboard may be increased by the regulatory authority. In this manual it is recommended that the 1.0 foot minimum be adopted.

4.8 Example

Given the design flow computed by the SCS TP-149 method in the example of Section 3.4, determine the following:

1. If the flow passes through a 14-foot trapezoidal channel of 2:1 side slopes, bottom slope of ten percent, on a cobble bed with $D_{50} = 1.5$ inches, what is the flow depth?

$$n = 0.0395\ (1.5/12)^{1/6} \qquad \text{Equation 4.18}$$

$$n = 0.028$$

$$Qn = 114\ (0.028) = 3.18$$

Therefore from Appendix C charts $d = 0.70$ feet and

$$Vn = 0.33, \quad V = \frac{Vn}{n} = 11.8 \text{ fps}$$

2. What is the Froude number?

$$Fr = \frac{V}{\sqrt{gL}} \qquad \text{Equation 4.6}$$

where the characteristic length L is usually taken as the flow depth. Therefore,

$$Fr = \frac{11.8}{\sqrt{32.2(0.70)}} = 2.5$$

Since the Froude number is greater than 1 the flow is supercritical and the slope condition is hydraulically steep (see Section 4.2.5).

3. If the channel contains a bend with an inside radius of 50 feet and an outside radius of 65 feet, what is the superelevation?

$$\Delta Z = \frac{V^2}{gr_c}\ (r_o - r_i) \qquad \text{Equation 4.19}$$

Table 4.4. Freeboard Coefficients.

Flow Condition	Minimum Freeboard Coefficient (c_{fb})
Subcritical (mild slope), unlined or vegetation-lined	0.20
supercritical (steep slope), unlined or vegetation-lined	0.25
subcritical (mild slope), rock riprap-lined	0.25
supercritical (steep slope), rock riprap-lined	1.00

Therefore

$$\Delta Z = \frac{(11.8)^2}{32.2(57.5)} (65-50) = 1.1 \text{ ft}$$

4. What are the freeboard requirements?

a. For supercritical flow, Table 4.4 gives

$$C_{fb} = 0.25$$

$$0.25(d) = 0.25 (0.70) = 0.18 < 1.0 \text{ ft; use } 1.0 \text{ ft}$$

$$F.B. = 0.25(d) + \frac{1}{2} \Delta Z = 1.0 + 0 = 1.0 \qquad \text{Equation 4.20}$$

b. In the channel bend?

$$F.B. = 1.0 + \frac{1}{2} (1.1) = 1.55 \text{ ft}$$

4.9 References

ASCE Task Force Committee, 1963, "Friction Factors in Open Channels," Journal of the Hydraulics Division, Proc. ASCE, Vol. 89, No. HY2, March.

Chow, V. T., 1959, Open-Channel Hydraulics, McGraw-Hill, New York, NY, 680 pp.

Highway Research Board, 1970, "Tentative Design Procedure for Riprap-Lined Channels," National Cooperative Highway Research Program Report 108.

Rouse, H. R., 1976, Advanced Mechanics of Fluids, Robert Krieger Publishing Co., New York, NY.

Simons, D. B., and F. Senturk, 1976, Sediment Transport Technology, Water Resources Publications, Fort Collins, CO.

Soil Conservation Service, 1954, National Engineering Handbook, Section 5, Hydraulics, U.S. Department of Agriculture, Washington, D.C.

Soil Conservation Service, 1977, "Design of Open Channels," Technical Release No. 25.

U.S. Department of Transportation, 1979, "Design Charts for Open-Channel Flow," Hydraulic Design Series No. 3, reprinted from 1961.

Woodward, S. M., 1920, "Hydraulics of the Miami Flood Control Project," Technical Reports, Miami Conservancy District, Dayton, OH, Part VII.

V. STEEP SLOPE CHANNEL DESIGN

5.1 Introduction

The design of steep conveyance channels is a critical step in developing an adequate drainage network on a surface mine site. Steep slope conditions are typical of the natural watersheds in the mining regions of Appalachia. Additionally, man-made spoil fill slopes involve the conveyance of water on slopes of steep angle. The success or failure of these conveyances, typically intermittent or ephemeral streams, determines the sediment load delivered to the downstream environment. Therefore, steep slope conveyances must be adequately designed to insure the long-term success of the drainage network.

Achieving channel stability on steep slopes usually requires some type of channel lining. The only exception is a channel constructed in durable bedrock. In this case, the permissible velocity design approach given in Chapter VI can be utilized. Rock riprap is the most commonly used channel lining on surface mine sites, but adequate procedures for its design are not readily available. Most all riprap design procedures were developed for mild slope channels (typically less than ten percent) and their use in the steep slope conditions of Appalachia does not produce a reliable design.

Therefore, in this chapter a riprap design procedure developed exclusively for steep slope conditions is presented. The graphical design procedure is simple to apply and provides an accurate design based on the conditions that exist. The success of any riprap-lined channel depends on factors other than simply sizing the rock required. Of particular importance to a successful riprap-lined channel is the gradation and placement of the riprap (section 5.2.6). These two criteria are probably the most difficult to follow for surface mine operators in the Appalachia region. The blast rock typically used for riprap in steep channels is large enough, but usually not adequately graded to the smaller sizes. Placement of the large rock on the steep slopes is difficult to accomplish, particularly when constructing relatively small channel cross sections with the large rock. The steep slope riprap design procedure presented herein provides an adequate design, however, these other two criteria may determine the success or failure of a channel, and perhaps even the feasibility of designing and constructing such channels. The West Virginia alternative of chimney drains, even with the potential problems of plugging by sediment deposition, provides a viable alternative.

However, it is felt that additional research is needed to establish their long-term functioning.

Prior to presenting the steep slope design procedure, consideration is given to the other important concepts of riprap design. These concepts are applicable to both mild and steep slopes and are an integral part of any riprap design.

5.2 General Riprap Considerations

5.2.1 Definition of Riprap

Riprap consists of a layer of discrete fragments of durable rock possessing sufficient size to withstand the dynamic, erosive forces generated by the flow of water. The protective qualities of a riprap-lined channel lie somewhere between those of a grassed waterway and a concrete-lined channel. Dumped riprap is extensively used on surface mine sites due to the availability of rock, and the conduciveness of this method to placement by readily available mechanized equipment.

Proper design of a riprap channel lining requires consideration of many factors. The desired level of protection may not be provided by the riprap if design criteria concerning rock gradation, riprap thickness, and filter design are not considered. This section discusses general riprap design considerations applicable to riprap protection on both mild and steep slopes.

5.2.2 Types of Riprap

There are many means and methods by which riprap protection can be constructed and placed. Hydraulic Engineering Circular No. 11 (Searcy, 1967) entitled "Use of Riprap for Bank Protection" provided the following categorization of riprap materials and methods of placement:

- Dumped riprap
- Hand-placed riprap
- Wire-enclosed riprap
- Grouted riprap
- Concrete riprap in bags
- Concrete slab riprap

5.2.3 General Considerations

The important factors to be considered in designing rock riprap protection are:

1. Durability of the rock (Chapter VIII)

2. Density of the rock

3. Velocity (both magnitude and direction) of the flow in the vicinity of the rock

4. Slope of the bed or bankline being protected

5. Angle of repose for the rock

6. Size and weight of the rock

7. Shape and angularity of the rock

8. Filter considerations

Rock riprap should (Soil Conservation Service, 1977):

1. Assure stability of the protected bank as an integral part of the channel as a whole. For this major objective, the ideal condition for stability is a straight channel or a gently curved channel with its outer bank rougher and more erosion resistant than the inner bank.

2. Tie to stable natural bank or other fixed improvements with transitions designed to ease differentials in alignment, grade, slope and roughness of banks.

3. Eliminate or ease local irregularities so as to streamline the protected bank.

When available in sufficient size, dumped rock riprap is usually the most economical material for bank protection. Dumped riprap has many advantages over other types of protection, including its flexibility and the ease of local damage repair. Construction must be accomplished in a prescribed manner but is not complicated. When proper consideration is given to filter requirements and grading of the bed foundation problems will be minimal. Appearance of dumped riprap is natural, and after a time vegetation will grow between the rocks. Wave runup on rock slopes is usually less than on other types. Finally, in temporary channels when the usefulness of the protection is finished, the rock is salvageable. Additionally, most riprap used on surface mining operations is dumped riprap. Therefore the following discussion concentrates on this type of riprap.

5.2.4 Properties of Rock Used as Riprap

Riprap should be hard, dense and durable to withstand long exposure to weathering. Visual inspection by a knowledgeable inspector is most often adequate to judge quality, but laboratory tests may be made to aid the judgment of the field inspector (see section on rock durability).

Rocks used for riprap should be blocky in shape, as they will tend to "nest" together, providing greater resistance to movement. Riprap consisting of angular stones is more suitable than that consisting of rounded stones due to their greater angle of repose (see Figure 4.3). These criteria are reflected in stone shape limitations given by the U.S. Army Corps of Engineers (1970) which specify:

1. The stone shall be predominantly angular in shape.

2. Not more than 25 percent of the stones reasonably well distributed throughout the gradation shall have a length more than 2.5 times the breadth or thickness.

3. No stone shall have a length exceeding 3.0 times its breadth or thickness.

These limitations apply only to the stone within the riprap gradation and not to any durable spalls and waste that may be allowed. When a high percentage of durable wastes or spalls is allowed or quarry run stone is used that does not meet the above-listed limitations, an increase in riprap thickness should be provided.

5.2.5 Riprap Gradation and Placement

Lack of a proper riprap gradation is one of the most common causes of riprap failure. Riprap gradation simply implies that the riprap should be composed of a distributed size range of rock. With distributed size range, the interstices formed by the larger stones are filled with the smaller sizes in an interlocking fashion that prevents formation of open pockets. Open pockets in a riprap layer allow jets of water to contact the underlying soil, resulting ultimately in the erosion of material supporting the riprap layer.

Riprap gradation guidelines are commonly given by defining the D_{max} and the D_{10} or D_{20} size as some percentage of the D_{50} size. These three points (D_{max}, D_{50} and D_{10} or D_{20}) are then plotted on semilog graph paper and a smooth S-shaped curve drawn through them to define the entire gradation range. Recommended values for the D_{max} size range from 1.3 to 2.0 times the D_{50} size, with 2.0 being the most commonly used. The value for the D_{10} to D_{20} size ranges from 0.20 to 0.30 times the D_{50} size.

These guidelines were developed for use with mild slope riprap design procedures where the D_{50} size is usually relatively small. Consequently, they produce adequate designs with a well distributed size range and D_{max} sizes that are reasonable. However applying this D_{max} criteria to a steep slope riprap design does not produce reasonable results. Due to the large D_{50} sizes typical of the steep slope riprap design procedure (see Section 5.3), the resultant D_{max} sizes are unrealistic and impractical particularly considering placement and channel excavation requirements (see Section 5.2.6). Additionally, the recommended steep slope riprap design procedure produces a conservative estimate of the D_{50} size. Therefore, the stability of the riprap should not require a large gradation above this size. In this situation it is more important to establish a smooth gradation to the smaller sizes to avoid large voids.

Therefore, the recommended riprap gradation for protection on steep slopes is that the maximum rock size D_{max} be no larger than 1.25 times the median size D_{50}. On mild slopes the upper limit for D_{max} should be increased to two times D_{50}. To maintain a large safety factor and a conservative design the ratio between the median diameter and the 10 to 20 percent size should be in a range of two to three for both steep and mild. Table 5.1 gives the suggested riprap gradation limits. Figure 5.1 qualitatively presents the gradation curves that result by using these guidelines and indicate their reasonableness. Control of the gradation of riprap is almost always made by visual inspection.

Improper placement of a properly designed riprap is another cause of failure. Riprap placement is usually by dumping directly from trucks. If riprap is placed during construction of the embankment, rocks can be dumped directly from trucks from the top of the embankment. To prevent segregation of sizes, rock should never be placed by dropping down the slope in a chute or pushed downhill with a bulldozer. With proper equipment dumped riprap can be placed with a minimum of expensive hand work. Poorly graded riprap with slab-like rocks requires more work to form a compact protective blanket without large holes or pockets. Draglines with orange peel buckets, backhoes and other power equipment can also be used advantageously to place the riprap.

Table 5.1. Recommended Riprap Gradation Limits*.

	Steep Slope	Mild Slope
$\frac{D_{max}}{D50}$	1.25	2
$\frac{D_{50}}{D_{10-20}}$	2-3	2-3

*i.e., if D_{50} is 12 inches for a steep slope riprap, then

$$\frac{D_{max}}{D_{50}} = 1.25; \quad D_{max} = 1.25\ (D_{50})$$

$$= 1.25\ (12) = 15 \text{ inches.}$$

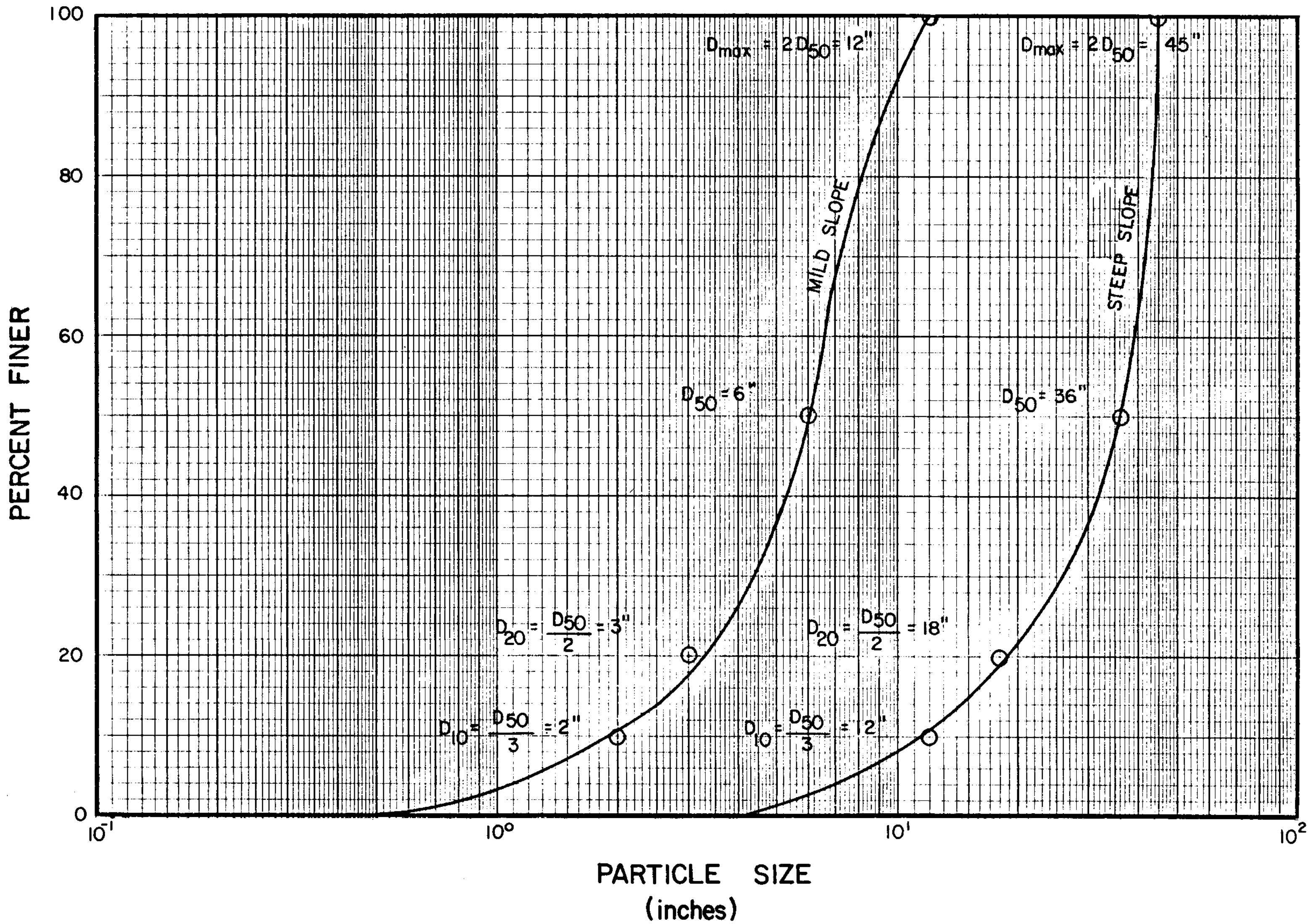


Figure 5.1. Definition sketch illustrating steep and mild slope riprap gradation based on recommended guidelines.

5.2.6 Riprap Thickness

The thickness of the riprap layer should be sufficient to accommodate the largest rock in the riprap material. Maximum resistance to the erosive forces of flowing water is obtained when individual stones are contained within the riprap layer thickness. Oversize stones that protrude above the riprap layer should be avoided since they reduce capacity and influence riprap stability. According to the Corps of Engineers (1970), "Oversize stones, even in isolated spots, may cause riprap failure by precluding mutual support between individual stones, providing large voids that expose filter and bedding materials, and creating excessive local turbulence that removes smaller stones." Where few oversize stones exist, these should be removed individually and replaced by appropriately sized rock. In instances where many oversized rocks exist, consideration should be given to remedial measures. Corrective actions could include using methods to remove the oversize stone, obtaining the stone from another source, or increasing the riprap layer thickness to contain the larger stone (Corps of Engineers, 1970).

The actual thickness of a riprap channel lining depends to a large extent upon experience and engineering judgment. Similar to gradation specifications, recommendations of layer thickness vary from about 1.3 to 2.0 times the median rock diameter D_{50} of the riprap in order to accommodate the D_{max} size. Therefore, for riprap linings on steep slopes a design value of 1.25 is recommended. A conservative riprap layer thickness of 2.0 times D_{50} is recommended for riprap applications on mild slopes.

In constructing the channel the depth of excavation must be adequate to accommodate the thickness of the riprap and filter layers. Therefore, any unnecessary increase in riprap thickness can significantly increase excavation and overall construction costs. If the design has been carefully and properly completed, there should be no need for the designer to recommend, nor the construction crew to build, a thicker lining.

5.2.7 Filter Layers

It is usually necessary to place a filter layer beneath riprap material to prevent leaching of the underlying soil and possible resultant bank stability problems. Without a filter layer the loss of underlying soil can occur in several ways. During high flows the lift and drag forces created by movement of water through the channel can create an uplift pressure. This pressure can generate enough suction to draw soil particles vertically through the voids in the riprap. Also during high flows, turbulent eddies and jets can penetrate the riprap lining through the voids in the riprap causing detachment and erosion of the underlying soil. Finally, during all flows, but more visable during low flows, water can move at the interface between the riprap layer and the underlying soil. If large enough voids are present, erosion can be significant, particularly if the underlying material is an erodible fill. In all cases the loss of soil can result in the formation of cavities with potential failure from the loss of support. The use of a properly designed filter layer will minimize this occurrence and greatly increase riprap stability.

The actual need for a filter layer is dependent upon the gradation and cohesion of the bank material as well as the relative size difference between the riprap particles and soil particles. However, in most surface mine situations a filter layer will be necessary. Suitable material for the filter can usually be obtained from the same strata as the rock riprap since the blasting operation creates a variation in rock sizes (Plate 5.1). Figure 5.2 illustrates typical particle size gradation curves for three-inch diameter blastholes in hard shale and sandstone (U.S. Army Engineer Waterways Experiment Station, 1975). The larger rocks can be scalped off first and the remaining fines material saved and used as the filter material. In addition to natural graded filters, plastic filter cloth is also available. Guidelines for filter design are given in the following section.

5.2.7.1 <u>Granular Filters</u>

A layer or blanket of well-graded gravel should be placed over the channel bed prior to riprap placement. Sizes of durable particles in the filter blanket should be from 3/16 inches (5 mm) to an upper limit depending on the gradation of the riprap. Thickness of the filter may vary depending upon the riprap thickness, but should not be less than 6 to 9 inches (15-23 cm). It is recommended that the filter thickness equal the D_{max} of the filter with a minimum of nine inches.

Suggested filter gradation specifications have been developed on the basis of tests by Terzaghi's the Bureau of Reclamation, and the Corps of Engineers. Terzaghi's criteria relating gradation of filter cover to the particle size distribution of the underlying base or bed material is as follows:

Plate 5.1. Blasted durable rock along highwall. Note the gradation of particle sizes available for both riprap and filter materials.

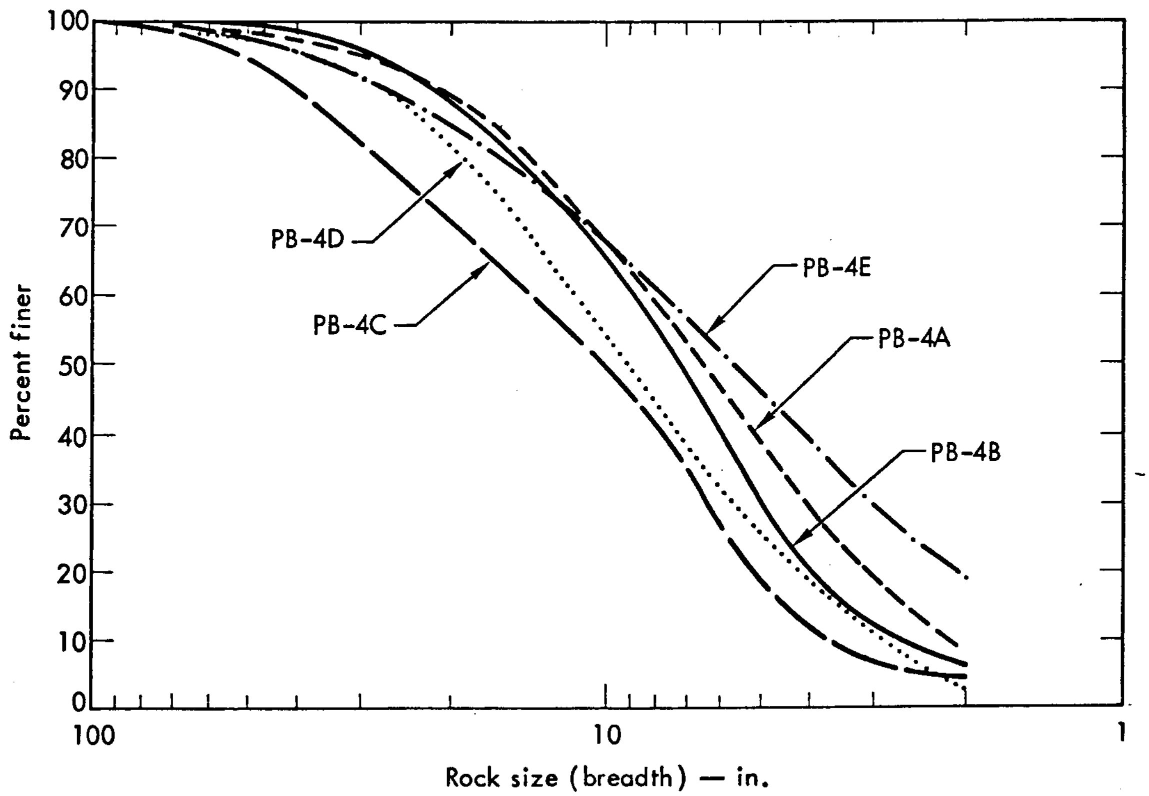


Figure 5.2. Gradation curves for three-inch diameter blast holes in hard shale and sandstone (U.S. Army Engineer Waterways Experiment Station, 1975).

$$\frac{D_{15}\ \text{(Filter)}}{D_{85}\ \text{(Base)}} < 5$$

$$4 < \frac{D_{15}\ \text{(Filter)}}{D_{15}\ \text{(Base)}} < 20 \tag{5.1}$$

$$\frac{D_{50}\ \text{(Filter)}}{D_{50}\ \text{(Base)}} < 25$$

where the subscripts denote percentage of particles finer by weight.

The Corps of Engineers and Bureau of Reclamation filter gradation specifications have different limits, specifying (Anderson et al., 1970):

$$\frac{D_{15}\ \text{(Filter)}}{D_{85}\ \text{(Base)}} < 5$$

$$5 < \frac{D_{15}\ \text{(Filter)}}{D_{15}\ \text{(Base)}} < 40 \tag{5.2}$$

$$\frac{D_{50}\ \text{(Filter)}}{D_{50}\ \text{(Base)}} < 40$$

Use of these criteria to evaluate the need for filter layers and their design is illustrated in the examples. Filter layers should not contain too many fines. A general guideline is that no more than five percent by weight of the particles in a filter sample should pass through a 200-mesh sieve. If more than one filter layer is required to adequately protect the base bed material, then the gradation criteria must be met by successive filter layers. When multiple granular filter layers are required, for example when the bed and banks are composed of very fine particles, the use of plastic filter cloths should be investigated. It may be possible to achieve the desired level of protection with one layer of filter fabric in place of several layers of granular filter material. Additionally, channel excavation costs would be decreased since the channel section would not have to be enlarged to contain several filter layers. The following section discusses the properties and design criteria for plastic filter cloths.

5.2.7.2 Plastic Filter Cloths

Plastic filter cloths are being used beneath riprap and other revetment materials, such as articulated concrete blocks, with considerable success. However, filter fabric is by no means a total substitute for granular filters. Filter fabric has the limitations of (1) filtering action is provided in only one direction, (2) only one Equivalent Opening Size (EOS) is maintained between the bed material and riprap, (3) fabric is less resistant to stone movement because of its relatively smooth surface, (4) additional of care must be exercised in placing riprap over plastic cloth filters to prevent damage, and (5) long term durability has not been proven.

The Denver Urban Flood Control District Drainage Design Criteria Manual (in print) specifies that plastic filter cloths should not be used when slopes are greater than 2.5 to 1. This is due to the reduced resistance to movement afforded by the smooth fabric. A six- to nine-inch layer of granular material is also recommended to be placed over fabric cloths to prevent tearing during placement of the riprap.

Care must be exercised in filter fabric installations where the seepage forces could be oriented parallel to the fabric. This could result in piping along the underside of the fabric and possible stability problems. Durability of filter cloths has not yet been established because they have been in use only since around 1967. However, inspections at various installations indicate little or no deterioration had occurred in the few (one to four) years that have elapsed since test installations.

Applications for filter fabric should be evaluated in terms of the specific advantages and/or disadvantages as compared to granular bedding. The economics associated with granular material availability, excavation, and placement of granular filters should be weighed against the economics of filter fabric. Obviously numerous site-specific factors determine the relative merits of each method; however, for long term design on a surface mine granular filter layers are generally preferred.

The following design criteria for plastic filter cloths were given by Normann (1975) in Federal Highway Administration Hydraulic Engineering Circular No. 15.

For filter cloths adjacent to granular materials containing 50 percent or less by weight fines (minus No. 200 material):

1. $\frac{\text{85 percent size of material (mm)}}{\text{EOS (mm)}} > 1$ (5.3)

2. Open area not to exceed 36 percent.

For filter cloths adjacent to all other soils:

1. EOS no larger than the opening in the U.S. Standard Sieve No. 70.

2. Open area not to exceed ten percent.

Note: No cloth specified should have an open area less than four percent or an EOS with openings smaller than the opening in a U.S. Standard Sieve Size No. 100. When possible, it is preferable to specify a cloth with openings as large as allowable by the criteria. It may not be possible to obtain a suitable cloth with the maximum allowable openings which also meets the strength requirements including tear resistance, however, due to the limited number of cloths available. Hydraulic Engineering Circular No. 15 describes the procedures for determination of Equivalent Opening Size (EOS) and open area.

5.3 Steep Channel Riprap Design

5.3.1 Introduction

The design of riprap for stabilizing steep conveyance channels with significant flow presents unique problems generally not encountered in riprap design study. Considering normal riprap design, the accepted methods are usually limited to slopes less than ten percent, where the velocities are slow enough and the depth of flow large enough (relative to the riprap size) that a reasonable estimate for the resistance to flow can be made. This resistance is generally characterized by Manning's n. However, on steep slopes the riprap size required to stabilize the channel is on the same order of magnitude or greater than the depth of flow. This creates difficulties in estimating roughness since Manning's relation is no longer valid. Without knowledge of the resistance to flow, an estimate of the velocity, needed for the design of the riprap, cannot be accurately determined.

Bathurst (1979) studied the hydraulics of mountain rivers where large roughness elements often exist in the flow. Based on flume studies, he was able to derive relationships for the resistance in channels where roughness elements are on the same order of magnitude as the depth of flow, often breaking through the water surface. Using the general resistance equation developed by Bathurst, the velocity in a channel with a given size riprap can be determined. This velocity can then be used to evaluate the stability of the riprap. This approach is physically realistic since the hydraulics of a mountain river are similar to those of a steep slope diversion channel. In both situations the steep slopes require large diameter rock for stability since the flow tends to cascade around the rocks rather than flow over them. Consequently, the design procedure recommended in the follwoing section produces rock sizes and therefore, channel excavation requirements that may appear unrealistic for the design discharge and flow depth. However, the designer need only remember the hydraulics of the situation and the similarity to a steep mountain river to bring perspective to the design.

5.3.2 Simplified Design Procedures

Five sets of design curves (Figures 5.3-5.7) have been developed from Bathurst's relationship to simplify riprap design for steep conveyance channels. The design curves were developed for trapezoidal channels with 2 to 1 side slopes and bottom widths of 0, 6, 10, 14 and 20 feet. The 2 to 1 side slope was selected by considering angle of repose (Section 4.6) and channel excavation requirements. The 2 to 1 side slope is sufficiently less than the angle of repose for any particle size or shape. Side slopes less than 2 to 1 (i.e., 3 to 1) greatly increase excavation amounts without significantly increasing flow capacity for a given bottom width. Therefore, the 2 to 1 side slope was selected and is recommended for all steep slope channels. For a given channel bottom width, each curve represents the design for a given channel slope. The design curves were terminated at the point where flow velocities for a specific channel configuration and slope exceeded 15 ft/sec. A median rock diameter could be determined that would be stable at higher flows; however, rock durability at high flow velocity becomes of greater concern.

The procedure for entering the design curves with a known Q is illustrated on Figures 5.3 and 5.4. For practical engineering purposes, the D_{50} size specified for the design should be given in 0.25-foot increments. Therefore, the final design size is determined using Table 5.2. For the case where the bed slope (S) is not given on one of the design curves, linear interpolation is used to determine the riprap design. This can be done graphically by extending the horizontal line from the known discharge through

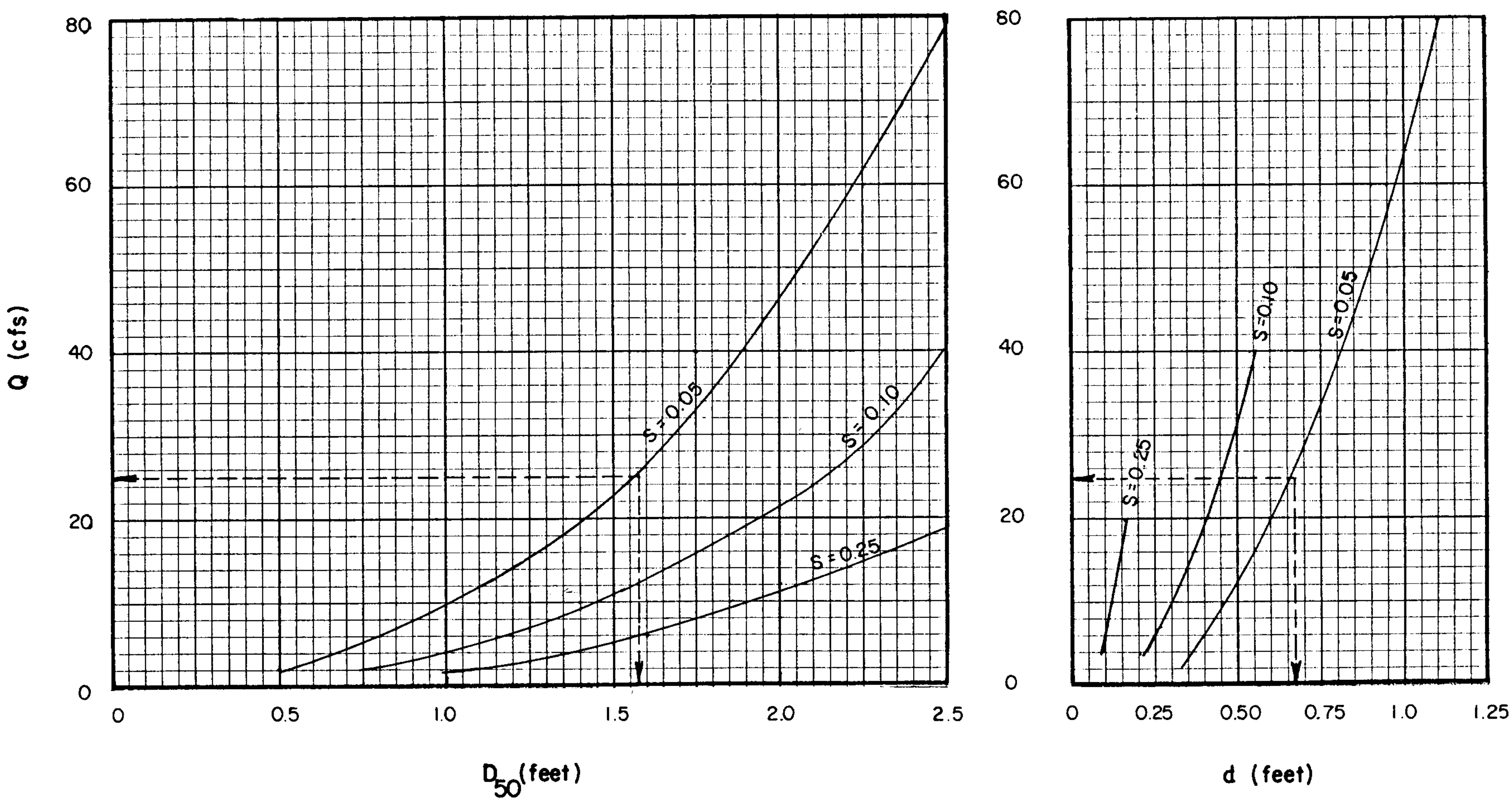


Figure 5.3. Steep slope riprap design, triangular channels, 2:1 sideslopes.

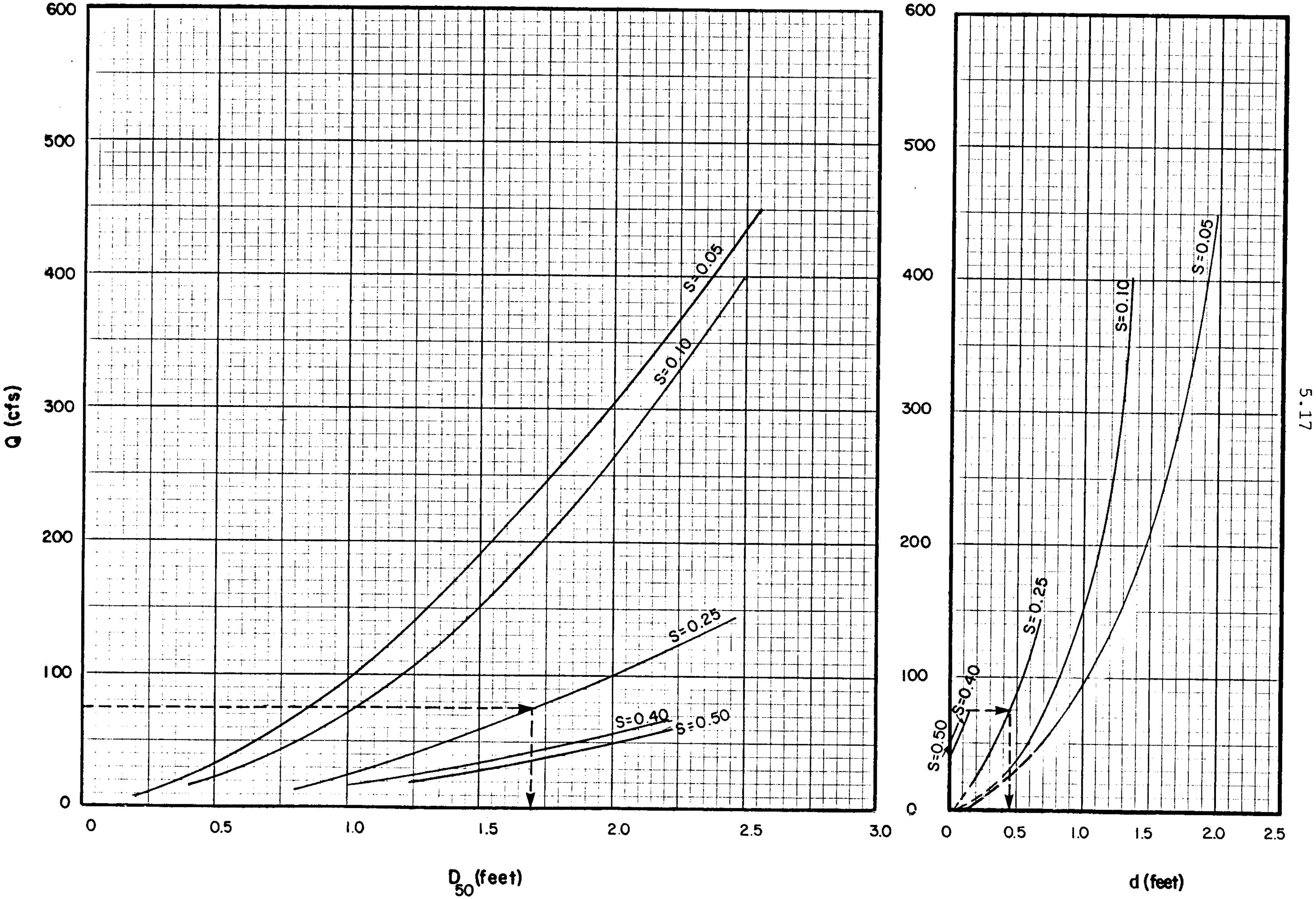


Figure 5.4. Steep slope riprap design, trapezoidal channels, 2:1 sideslopes, 6 ft base width.

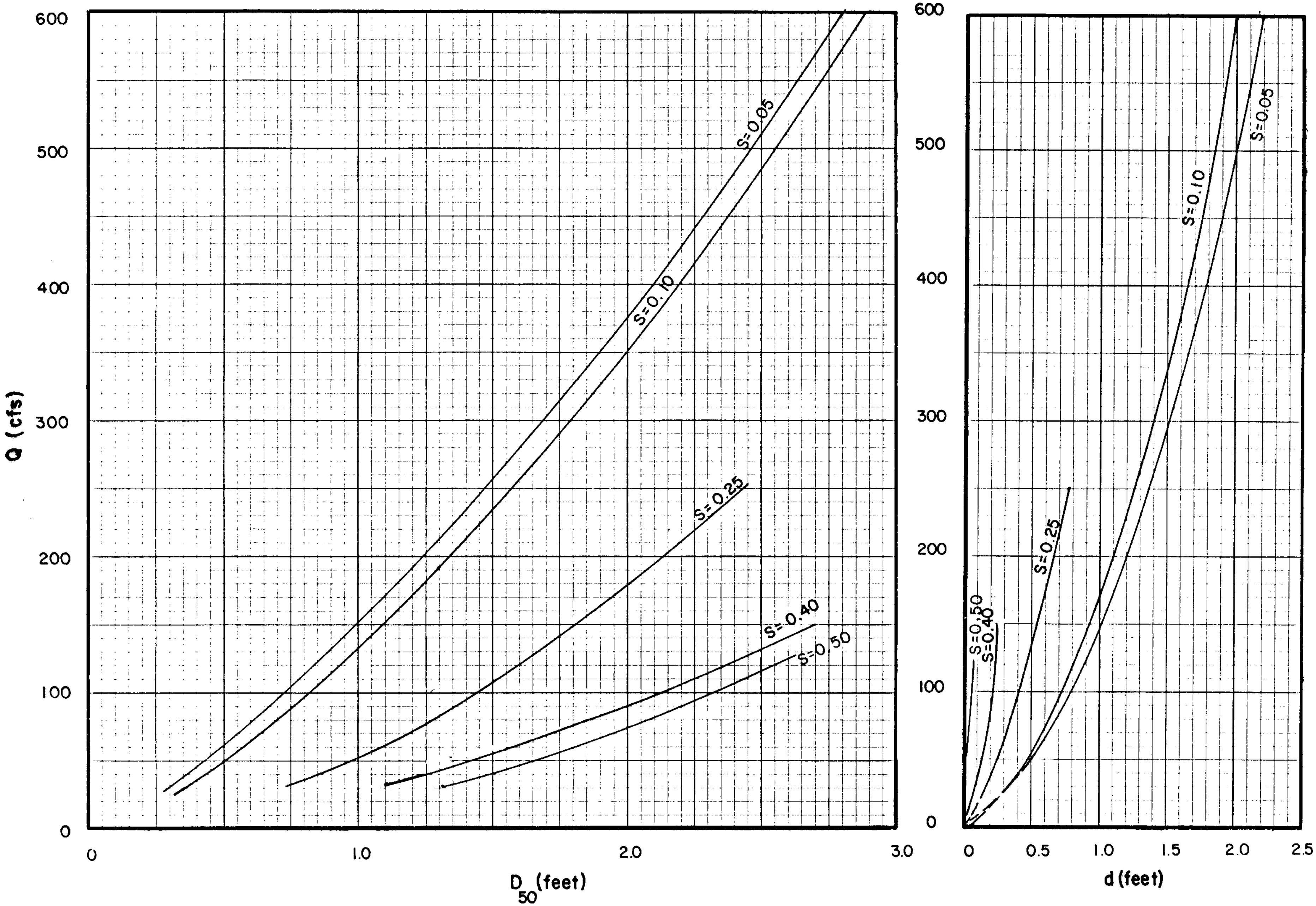


Figure 5.5. Steep slope riprap design, trapezoidal channel, 2:1 sideslopes, 10 ft base width.

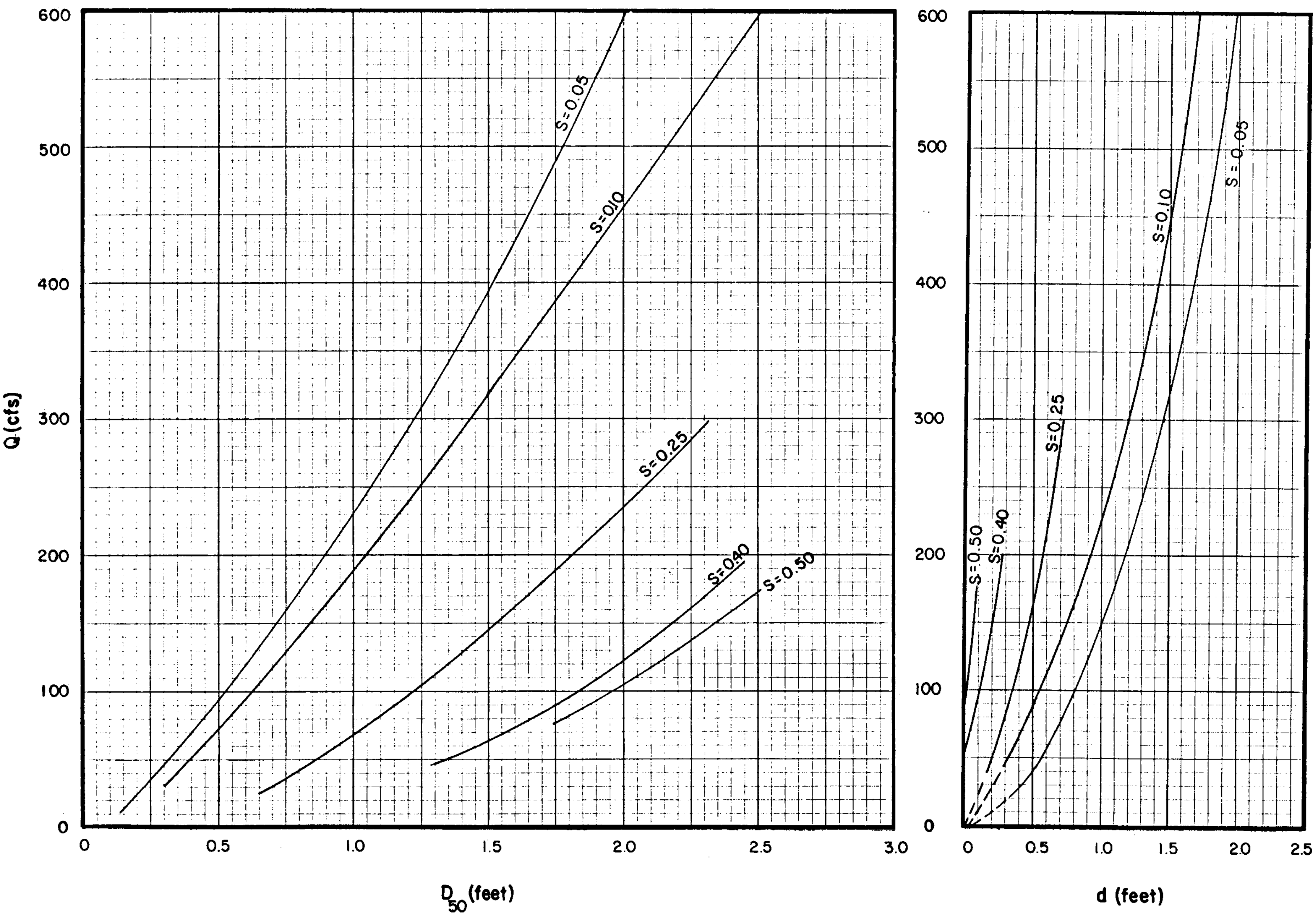


Figure 5.6. Steep slope riprap design, trapezoidal channels, 2:1 sideslopes, 14 ft base width.

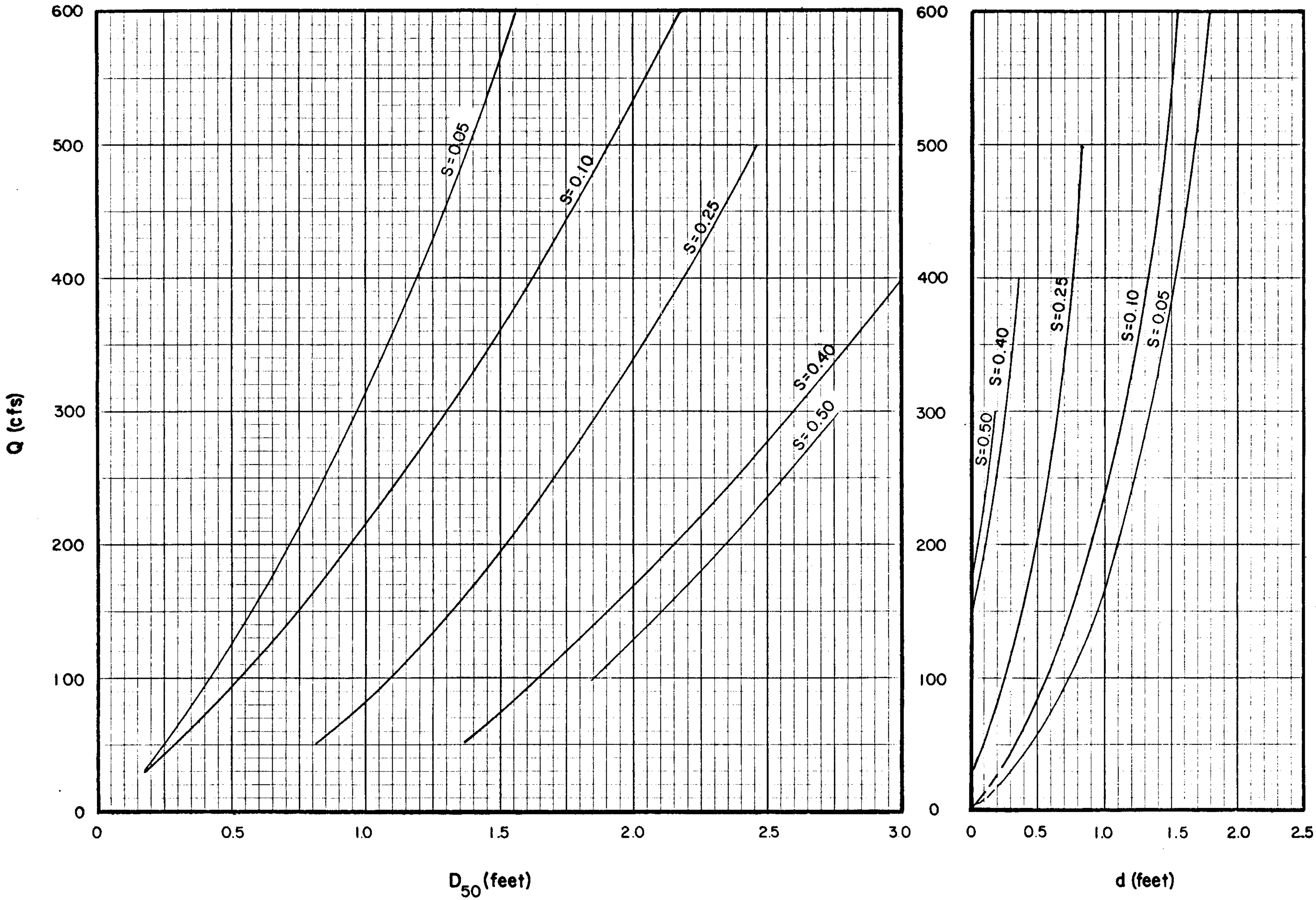


Figure 5.7. Steep slope riprap design, trapezoidal channels, 2:1 sideslopes, 20 ft base width.

Table 5.2. Design D_{50} Values.

D_{50} Determined From Design Curve (ft)	Design D_{50} (ft)
< 0.25	0.25
0.26 - 0.50	0.50
0.51 - 0.75	0.75
0.76 - 1.00	1.00
1.01 - 1.25	1.25
1.26 - 1.50	1.50
1.51 - 1.75	1.75
1.76 - 2.00	2.00
2.01 - 2.25	2.25
2.26 - 2.50	2.50
2.51 - 2.75	2.75
2.76 - 3.00	3.00

the curves with slopes bracketing the design slope. A point is then determined between the design curves by visual interpolation, and the D_{50} size is determined by a vertical line extended from this point. Similarly, for bottom widths other than the ones the graphs were prepared for, it is adequate to linearly interpolate using values from the graphs that bracket the desired bottom width.

5.4 Channel Entrances and Exits

When there is a transition from a steep to a mild slope, the flow changes from a supercritical (rapid) to a subcritical (tranquil) condition (see Section 4.2.5). Due to this change in the state of flow there is the possibility of formation of a hydraulic jump in the transition reach. The occurrence of a hydraulic jump is related to the Froude number (see Equation 4.7).

For the range of velocities and flow depths established in a channel by the steep slope riprap design procedure, Froude numbers on the steep slope will be less than four. In this Froude number range, violent hydraulic jumps do not form. For Froude numbers in the range of 2.5 to 4.5, surface waves are usually generated and a slight increase in water-surface elevation occurs; however, the violent hydraulic jump experienced at the base of a smooth surfaced spillway does not occur. Protection is still required in the transition length between a steep slope riprapped channel and a mild sloped channel to protect against local scour. Protection is required in the entrance due to the drawdown and increased velocity that results as flow transitions from a mild to steep slope. For simplicity, the length of protection estimated for the more critical exit section is also specified for entrance protection. A general rule for the length of protection required when Froude numbers are less than four is that transition length should be five times the uniform depth of flow computed for the downstream channel section; however, in no case should this length be less than 15 feet.

5.5 Design Procedures Summary

5.5.1 Criteria for Riprap Design

The design of riprap-lined channels with steep conveyance using the design curves requires the following steps:

1. Estimate the design flow based on hydrologic computations (Chapter III).

2. Determine the channel design slope based on topographic considerations. The channel design slope should be the uniform slope required to allow the channel to be constructed through slight changes in grade. If excavation amounts are too great too allow a uniform channel slope through changes in terrain slope, the channel can be designed to follow the changes in grade. For ease in construction, a single channel cross section adequate for each slope can be designed using the maximum slope to size the riprap required and the minimum slope to establish flow depth and freeboard requirements (transition requirements must be considered if this procedure is followed).
3. Determine a channel bottom width based on engineering judgment and available equipment.
4. Using the appropriate design curve, determine the required D_{50} size and flow depth.
5. Determine gradation and riprap thickness (Sections 5.2.5 and 5.2.6).
6. Evaluate filter requirements (Section 5.2.7).

   Granular Filter Design

   a. Determine gradation of channel bed and banks and gradation of granular material available.
   b. Evaluate adequacy of filter material by criteria in Section 5.2.7.
   c. Determine acceptable gradation for filter material.
   d. Determine thickness of filter layer.

   Plastic Filter Cloths

   a. Determine gradation of bed and bank material.
   b. Evaluate percent by weight of fines in bed material.
   c. Calculate allowable Equivalent Opening Size (EOS) of plastic filter cloth using criteria in Section 5.2.7.2.
   d. Select from available filter fabric meeting criteria.
7. Estimate freeboard required from Equation 4.20 and finalize cross section dimensions.
8. Evaluate entrance and exit protection required.

5.6 Design Examples - Using Step-By-Step Procedures Outlined Above

5.6.1 Design Example for Steep Slope Protection

1. Design discharge $Q = 75$ cfs
2. Assume a uniform slope of 0.25

3. Channel base width (b) selected as 6 feet

4. From Figure 5.4

   $D_{50} = 1.70$ ft   Use $D_{50} = 1.75$ ft (Table 5.2)

   $d = 0.45$ ft

5. $D_{max} = 1.25\ D_{50} = 2.2$ ft
   Riprap Thickness = $D_{max} = 2.2$ ft

6. Evaluate filter requirements as discussed below.

7. Evaluate freeboard requirements. For steep slopes with riprap lining, Table 4.4 gives $c_{fb} = 1.0$.

   $c_{fb}(d) = (1)(0.45) = 0.45 \text{ ft} < 1.0 \text{ ft}$

   Therefore use 1.0 ft

   $$F.B. = c_{fb}(d) + \frac{1}{2}\Delta Z = 1.0 + 0 = 1.0 \text{ ft}$$

8. Figure 5.8 shows the channel section designed, including the granular filter (see next section).

5.6.2 Design Example for Granular Filter Layer

This example details procedures for determination and selection of an appropriate filter layer. The U.S. Army Corps of Engineers filter criteria are used because the limits are somewhat less restrictive than the Terzaghi filter criteria. The characteristics of the channel bed material and riprap protection are assumed to be:

$D_{85} = 0.27$ in

$D_{50} = 0.1$ in

$D_{15} = 0.036$ in

Riprap properties (determined from plotting gradation given in step 5 on semi-log paper; see Figure 5.9)

$D_{85} = 26$ in

$D_{50} = 21$ in

$D_{15} = 9.5$ in

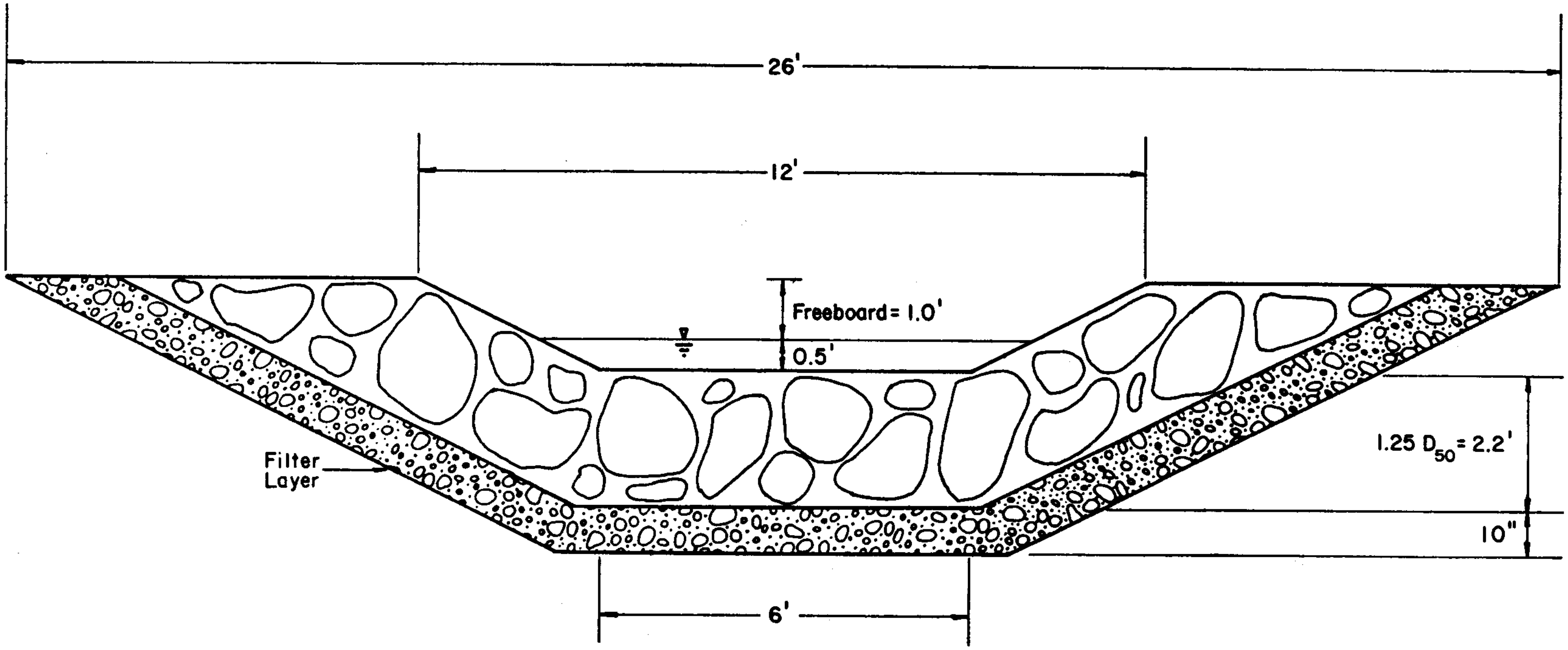


Figure 5.8. Final channel dimensions.

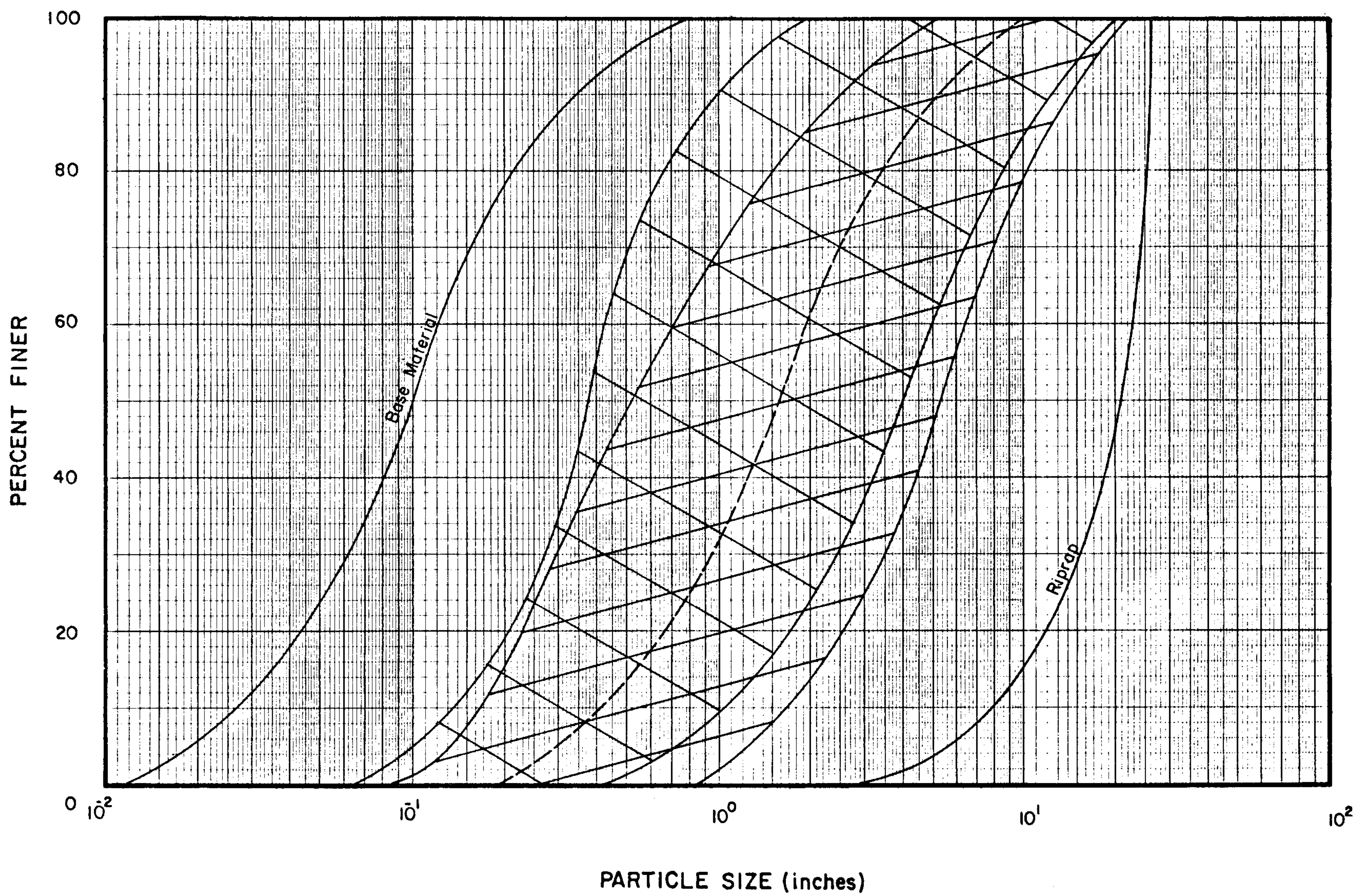


Figure 5.9. Gradations of filter blanket for design example.

<u>Solution</u>

1. Evaluate the need for a filter layer

$$\frac{D_{15}\ (\text{RIPRAP})}{D_{85}\ (\text{BASE})} = \frac{9.5}{0.27} = 35 < 5$$

Equation 5.2

$$\frac{D_{15}\ (\text{RIPRAP})}{D_{15}\ (\text{BASE})} \quad \frac{9.5}{0.036} = 264 < 40$$

Since filter criteria are exceeded, a filter layer is required.

2. The properties of the filter can be determined as follows

$$\frac{D_{50}\ (\text{FILTER})}{D_{50}\ (\text{BASE})} < 40, \quad \text{so} \quad D_{50}(\text{FILTER}) < 40 \text{ X } 0.1 = 4 \text{ in}$$

$$\frac{D_{15}\ (\text{FILTER})}{D_{15}\ (\text{BASE})} < 40, \quad \text{so} \quad D_{15}(\text{FILTER}) < 40 \text{ X } 0.036 = 1.4 \text{ in}$$

$$\frac{D_{15}\ (\text{FILTER})}{D_{85}\ (\text{BASE})} < 5, \quad \text{so} \quad D_{15}(\text{FILTER}) < 5 \text{ X } 0.27 = 1.35 \text{ in}$$

$$\frac{D_{15}\ (\text{FILTER})}{D_{15}\ (\text{BASE})} > 5, \quad \text{so} \quad D_{15}(\text{FILTER}) > 5 \text{ X } 0.036 = 0.18 \text{ in}$$

Therefore, with respect to the base material, the filter must satisfy

$$0.18 \text{ in} < D_{15}(\text{FILTER}) < 1.35 \text{ in}$$

$$D_{50}(\text{FILTER}) < 4 \text{ in}$$

3. Considering the riprap and a filter, the properties of the filter must satisfy

$$\frac{D_{50}\ (\text{RIPRAP})}{D_{50}\ (\text{FILTER})} < 40, \quad \text{so} \quad D_{50}(\text{FILTER}) > \frac{21}{40} = 0.5 \text{ in}$$

$$\frac{D_{15}\ (\text{RIPRAP})}{D_{15}\ (\text{FILTER})} < 40, \quad \text{so} \quad D_{15}(\text{FILTER}) > \frac{9.5}{40} = 0.2 \text{ in}$$

$$\frac{D_{15}\ (\text{RIPRAP})}{D_{85}\ (\text{FILTER})} < 5, \quad \text{so} \quad D_{85}(\text{FILTER}) > \frac{9.5}{5} = 1.9 \text{ in}$$

$$\frac{D_{15}\ (\text{RIPRAP})}{\text{D15}\ (\text{FILTER})} > 5, \quad \text{so} \quad D_{15}(\text{FILTER}) < \frac{9.5}{5} = 1.9 \text{ in}$$

Therefore, with respect to the riprap, the filter must satisfy

$$0.2 \text{ in} < D_{15} \text{ (FILTER)} < 1.9 \text{ in}$$

$$D_{50} \text{ (FILTER)} > 0.5 \text{ in}$$

$$D_{85} \text{ (FILTER)} > 1.9 \text{ in}$$

4. Figure 5.9 shows the limits of the filter material with respect to both the base and riprap material. The gradation curves for the filter layer have been extrapolated somewhat arbitrarily beyond the computed points. The ranges of suitable filter for both the riprap and the base have been crosshatched; any filter material that falls within the region where the cross-hatching overlaps will meet the criteria of both the riprap and the base material and will thus be suitable for the filter blanket.
5. The thickness of the filter layer can be determined for an assumed value of D_{max} of the filter. If filter material was available that had a gradation shown by the dotted line in Figure 5.9 then

   Assuming $D_{max} = 10$ in

   and filter layer thickness = $D_{max} = 10$ in

   The channel section including the filter layer, was shown in Figure 5.8.

5.6.3 Plastic Filter Cloth Design Example

It is desired to design a plastic filter cloth suitable for application to base material having the gradation shown in Figure 5.8. Since no fines are present in the base material the design criteria are:

$$\frac{\text{85 percent size of material (mm)}}{\text{EOS (mm)}} > 1 \tag{5.3}$$

Open area not to exceed 36 percent.

<u>Solution</u>

1. From Figure 5.8 $D_{85} = 0.27$ in = 6.8 mm
2. A filter cloth should be chosen that has:

   Equivalent Opening Size (EOS) < 6.8 mm of Plastic Filter Cloth

3. A layer of gravel should be placed over the filter cloth to provide protection during riprap placement.

5.6.4 Entrance and Exit Design Example

Determine the length of protection required above and below the channel section of the previous example. The downstream channel properties are:

1. $b = 6$ ft
2. $S = 0.002$
3. $D_{50} = 2$ in $= 0.17$ ft

<u>Solution</u>

Compute normal depth downstream

1. $n = 0.0395\ D_{50}^{1/6} = 0.0395(0.17)^{1/6} = 0.03$ Equation 4.18
2. From Figure C-1 in Appendix C for $Q_n = 2.25$ and

   $S = 0.002$

   $d = 2.4$ ft

   $V_n = 0.089;\ V = \frac{V_n}{n} = 3.0$ fps

3. Compute protection required

   length of protection

   $L = 5d = 12 \text{ ft} < 15 \text{ ft}$

   Therefore, length of protection = 15 ft

4. Figure 5.10 shows the entrance and exit protection

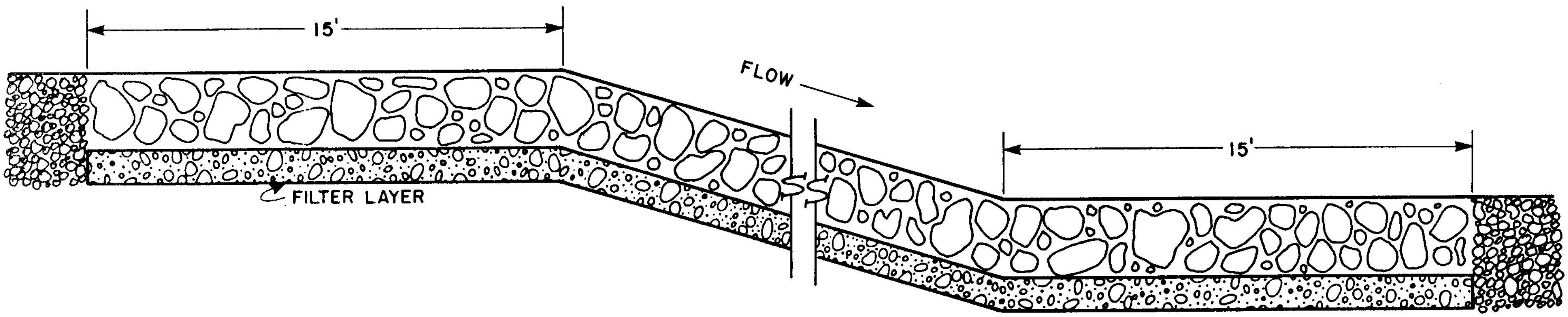


Figure 5.10. Entrance and exit protection on steep conveyance channel.

5.7 References

Anderson, A. G., A. S. Paintal, and J. T. Davenport, 1970, Tentative design procedure for riprap lined channels, Project Report No. 96, St. Anthony Falls Hydraulics Laboratory, Minneapolis, Minnesota, NCHRP Report 108, Highway Research Board, Washington, D.C.

Bathurst, J. C., R. M. Li, D. B. Simons, 1979, Hydraulics of Mountain Rivers. Civil Engineering Department, Colorado State University, CER78-79JCB-RML-DBS55.

Denver Urban Drainage and Flood Control District, Urban Storm Drainage Criteria Manual, Chapter V, Volume 2.

Normann, J. M., 1975, "Design of Stable Channels with Flexible Linings," Hyd. Eng. Circ. No. 15, USDT, FHA, pp. 1-136.

Searcy, J. K., 1967, "Use of riprap for bank protection," Hydraulic Engineering Circular No. 11, Hydraulics Branch, Bridge Division, Office of Engineering and Operations, Bureau of Public Roads, Washington, D.C.

USDA, Soil Conservation Service, 1977, Design of Open Channels Technical Release No. 25, Washington, D.C.

U.S. Army Corps of Engineers, 1970, Hydraulic Design of Flood Control Channels. Department of the Army, Office of the Chief of Engineers, Engineer Manual EM1110-2-1601.

U.S. Army Engineer Waterways Experiment Station, 1975, Project R.D. Bailey Experimental Excavation Program, Technical Reprint E-75-2, Explosive Excavation Research Laboratory.

This page intentionally left blank.

VI. MILD SLOPE CHANNEL DESIGN

6.1 Introduction

In many areas of the Eastern Coal Province the mine support facilities are often located in stream or river bottom areas where mild slope conditions exist. To accommodate the facilities or to provide larger storage areas, larger streams or rivers have been relocated using mild slope channel design procedures. Additionally, the slopes around the upper perimeter of a backfill or spoil fill area are often mild slope channels. Mild slope channel design involves the concepts of alluvial channels unless the channel is constructed in durable bedrock. An alluvial channel is a waterway flowing through a natural alluvium consisting of clay, silt, sand, gravel or boulders. Under these conditions, the boundary of the channel can easily change its configuration. Therefore, in alluvial channel design problems the concepts of movable-boundary hydraulics as well as rigid-boundary hydraulics must be applied. The concepts of movable-boundary hydraulics apply to small unlined diversion ditches as well as large stream systems, since both can qualify as alluvial channels. Plate 6.1 illustrates an unlined diversion channel around the edge of a backfill area. The channel appears relatively stable and is a good example of stable channel design based on alluvial channel concepts. However, if the channel is not properly designed and overbank flow occurs, excessive rilling and gullying can be expected on the steeper face of the fill area (Plate 6.2).

This section of the manual is presented not only for the purposes of designing channels in mild slope areas, but also to give the designer an understanding of the entire drainage system that will be affected by an operation. Additionally, if an operation is not properly reclaimed to near natural conditions, there will be the potential for a long-term increase of unnatural sediment load in a stream. This increase in sediment will have many downstream consequences. Further, if a channel is placed on fill materials and the designed lining fails, the channel will function as a movable boundary channel. In steep slope areas the channel will become deeply incised in the embankment with continual adjustments until some stability is achieved or until natural bed rock is reached. This will most likely not result until after tremendous erosion has occurred.

The flow of water along an alluvial channel bottom produces forces that initiate sediment motion. The amount of sediment entrained depends on the

Plate 6.1

Plate 6.2

characteristics of these forces, referred to as hydrodynamic forces in literature on channel stability. For a given sediment particle a critical or threshold value of the hydrodynamic forces must be reached before sediment motion begins. The magnitude of force necessary to initiate motion depends on grain size and bed-material properties. After traveling some distance downstream, sediment entrained with the flow can also settle back to the bed surface. The process of sediment transport is characterized by this cycle of motion and rest. The rates and frequencies at which the cycle occurs are random variables depending on sediment characteristics, flow conditions, channel shape, turbulent velocity fluctuations and many other factors. The complexity of the problem makes design of a stable alluvial channel and prediction of geomorphic changes in a stream bed difficult.

Stable alluvial channel design involves the concepts of static and dynamic equilibrium. Static equilibrium exists when the bed and banks of the alluvial channel are not in motion and it can be considered as a rigid boundary system. This condition exists as long as the hydrodynamic forces are less than the critical or threshold values. Dynamic equilibrium exists when the channel boundary is in motion such that the sediment tansporting capacity is equal to the sediment supply rate. According to Lane (1953), "A stable channel is an unlined earth channel (a) which carries water, (b) the banks and bed of which are not scoured objectionably by moving water, and (c) in which objectionable deposits of sediment do not occur." This definition is based on dynamic equilibrium concepts.

Economies in cost can often be realized by designing the channel considering the processes of erosion and sedimentation, rather than attempting to create stability through expensive riprap or other channel stabilization measures. Static equilibrium concepts are applicable primarily to gravel-cobble bed channel systems, while dynamic equilibrium concepts must be utilized in sand-bed channel systems. The stable alluvial channel design methods discussed in this chapter are based primarily on the static equilibrium concept since stream beds the Eastern Coal Province are typically gravel-cobble type systems. Part 2 will present design guidelines for sand bed systems.

6.2 Determination of Drainage Patterns and Diversion Alignment

Channel alignment is an important feature of channel design. Careful consideration must be given to all factors affecting location, including comparison of alternate alignments. Location of a diversion channel depends on the application and motive for its use. Diversions can be used to control and manage the drainage of a mine site (such as interception and diversion of surface runoff), or to relocate and re-establish stream channels (see Section 1.4). If the motive is management of surface drainage during mining the existing drainage patterns must be established. This can be accomplished with a good topographic map. If the motive is relocating or re-establishing a stream channel, experience and engineering judgment combined with a careful study of the local conditions is required.

Many factors affect the planned alignment of a channel. Topography, the size of the proposed channel, the existing channel, tributary junctions, geologic conditions, channel stability, rights of way, required stabilization measures, and other physical features enter into this decision. General rules to follow in determining diversion channel alignment include: (1) follow the general direction of natural drainageways, (2) provide relatively straight channels with gradual curves, (3) make use of natural or existing channels when possible, and (4) avoid unstable soils and other natural conditions that increase construction and maintenance costs (Schwab et al., 1966). Channel alignment and the use of gradual curves are particularly important. Gradual curves minimize superelevation and possible bank erosion. Further guidelines for channel alignment are given by Soil Conservation Service (1977) Technical Release No. 25.

The shortest alignment between two points may provide the most efficient hydraulic layout, but it might not meet all the objectives of the channel improvement or give due consideration to the limitations imposed by certain physical features. The shortest, well planned alignment should be used in flat topography if geologic conditions are favorable and if physical and property boundaries permit.

Alternate alignment should be considered in areas where geologic conditions present a stability problem. An alternate alignment may locate the channel in more stable soils. In some cases, the alignment of the existing channel may be satisfactory with only minor changes. An alignment resulting in a longer channel may, to a minor degree, help to alleviate stability problems. A longer channel will decrease the energy gradient which, in turn, will decrease the velocities and tractive forces.

6.3 Alluvial Channel Concepts

The fluvial system, composed of watersheds and alluvial channels, is a highly complex system involving the processes of erosion and sedimentation. A conceptual drawing of the fluvial system is given in Figure 6.1. Erosion in the watersheds supplies primarily fine sediments that are transported by overland flow to the alluvial channel system. Within the alluvial channel system, consisting of streams, rivers, and reservoirs, these fine sediments are transported downstream, in addition to the transport of coarser sediments eroded from the bed and banks of the alluvial channel.

Alluvial channel systems are very dynamic in nature and generally experience significant changes in depth, width, alignment and stability with time, particularly during the floods of long duration. The dynamic nature of watershed and channel systems requires that local problems and their solutions be considered in terms of the entire system. Natural and man-induced changes in a channel frequently initiate responses that can be propagated for long distances both upstream and downstream (Simons and Senturk, 1977). Successful stream and river utilization and water resources development require a general knowledge of the entire watershed and river system and the processes affecting it. Understanding potential changes requires a knowledge of the principles of erosion, sedimentation, and sediment transport processes.

6.3.1 General Sediment Transport Theory

The amount of material transported, eroded, or deposited in an alluvial channel is a function of sediment supply and channel transport capacity. Sediment supply includes the quality and quantity of sediment brought to a given reach. Transport capacity involves the size of bed material, flow rate, and geometric and hydraulic properties of the channel. Both the supply rate and the transport capacity may limit the actual sediment transport rate in a given reach.

The total sediment load in a stream is the sum of the bed-material load and wash load. The bed-material load is that part of the total sediment discharge which is composed of grain sizes found in the bed. The wash load is that part composed of particle sizes finer than those found in appreciable quantities in the bed (Simons and Senturk, 1977). Wash load can increase bank stability, reduce seepage and increase bed-material transport and can be easily transported in large quantities by the stream, but is usually limited

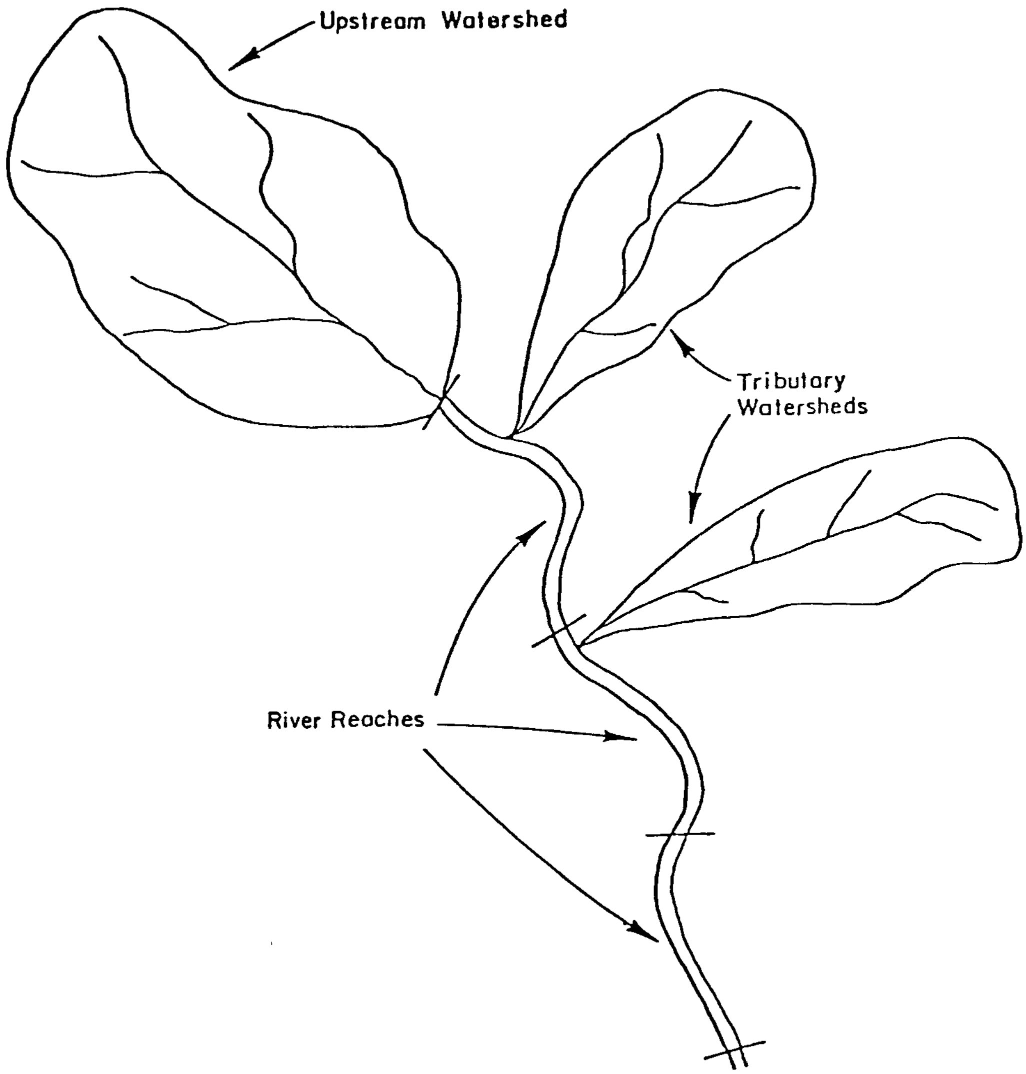


Figure 6.1. Watershed-river system.

by availability from the watershed and banks. The bed-material load is more difficult for the stream to move and is limited in quantity by the transport capacity of the channel.

Sediment particles are transported by the flow in one or more of the following ways: (1) surface creep, (2) saltation, and (3) suspension. Surface creep is the rolling or sliding of particles along the bed. Saltation is the cycle of motion above the bed with resting periods on the bed. Suspension involves the sediment particle being supported by the water during its entire motion. Sediments transported by surface creep, sliding, rolling and saltation are referred to as bed load, and those transported by suspension are called suspended load. The suspended load consists of sands, silts, and clays. The bed-material load is the sum of bed load and suspended bed-material load.

6.3.2 Stream Form and Classification

Streams and rivers can be classified broadly in terms of channel pattern, that is, the configuration of the river as viewed on a map or from the air. Patterns include straight, meandering, braided, or some combination of these (Figure 6.2).

6.3.2.1 Straight Channels

A straight channel can be defined as one that does not follow a sinuous course. Leopold and Wolman (1957) have pointed out that truly straight channels are rare in nature. Although a stream may have relatively straight banks, the thalweg, or path of greatest depth along the channel, is usually sinuous (Figure 6.2b). As a result, there is no simple distinction between straight and meandering channels.

The sinuosity of a stream or river, the ratio of the thalweg length to down valley distance, is most often used to distinguish between straight and meandering channels. Sinuosity varies from a value of unity to a value of three or more. Leopold, Wolman, and Miller (1964) took a sinuosity of 1.5 as the division between meandering and straight channels. It should be noted that in a straight reach with a sinuous thalweg developed between alternate bars (Figure 6.2b) a sequence of shallow crossings and deep pools is established along the channel.

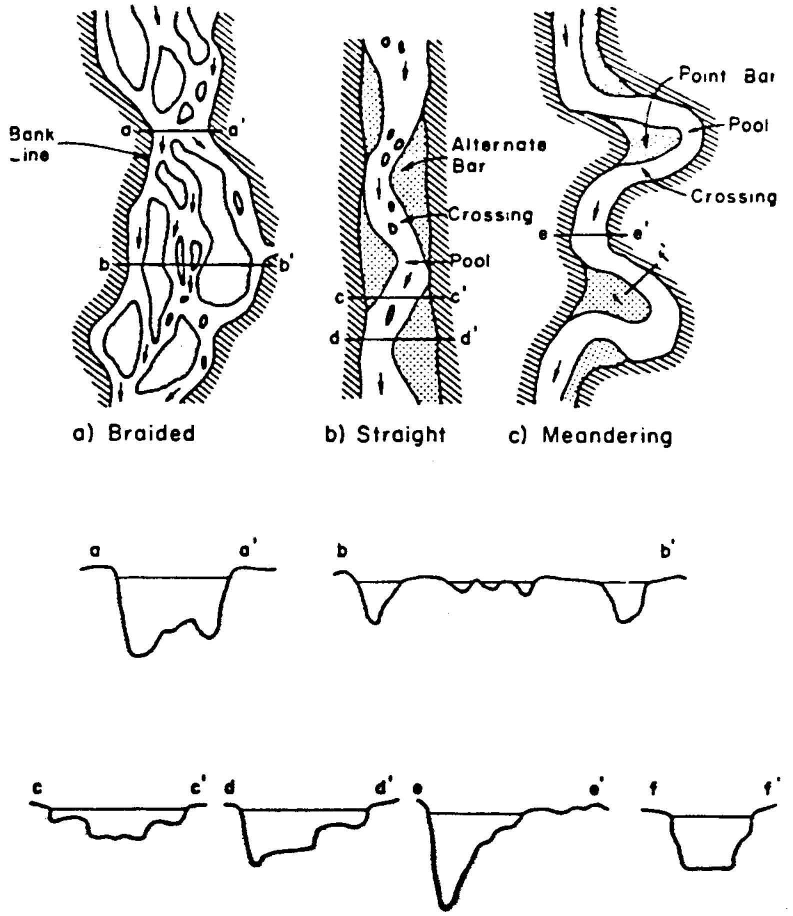


Figure 6.2. River channel patterns.

6.3.2.2 The Braided Stream

A braided stream or river is generally wide with poorly defined and unstable banks, and is characterized by a steep, shallow course with multiple channel divisions around alluvial islands (Figure 6.2a). Braiding was studied by Leopold and Wolman (1957) in a laboratory flume. They concluded that braiding is one of many patterns which can maintain quasi-equilibrium among the variables of discharge, sediment load, and transporting ability. Lane (1957) concluded that, generally, the two primary causes that may be responsible for the braided condition are: (1) overloading, that is, the stream may be supplied with more sediment than it can carry, resulting in deposition of part of the load, and (2) steep slopes, which produce a wide shallow channel where bars and islands form readily.

Either of these factors alone, or both in concert, could be responsible for a braided pattern. If the channel is overloaded with sediment, deposition occurs, the bed aggrades, and the slope of the channel increases in an effort to maintain a graded condition. As the channel steepens, the velocity increases, multiple channels develop and cause the overall channel system to widen. The multiple channels, which form when bars of sediment accumulate within the main channel, are generally unstable and change position with both time and stage.

Another cause of braiding is easily eroded banks. If the banks are easily eroded, the stream widens at high flow and at low flow bars form which become stabilized, forming islands. In general, then, a braided channel has a steep slope, a large bed-material load in comparison with its suspended load, and relatively small amounts of silts and clay in the bed and banks.

6.3.2.3 The Meandering Channel

A meandering channel is one that consists of alternating bends, giving an S-shaped appearance to the plan view of the stream or river (Figure 6.2c). More precisely, Lane (1957) concluded that a meandering stream is one whose channel alignment consists principally of pronounced bends, the shapes of which have not been determined predominantly by the varying nature of the terrain through which the channel passes. The meandering stream or river consists of a series of deep pools in the bends and shallow crossings in the short straight reach connecting the bends. The thalweg flows from a pool through a crossing to the next pool, forming the typical S curve of a single meander loop.

As shown schematically in Figure 6.2, the pools tend to be somewhat triangular in section with point bars located on the inside of the bend. In the crossing the channel tends to be more rectangular, widths are greater and depths are relatively shallow. At low flows the local slope is steeper and velocities are larger in the crossing than in the pool. At low stages the thalweg is located very close to the outside of the bend. At higher stages, the thalweg tends to straighten. More specifically the thalweg moves away from the outside of the bend, encroaching on the point bar to some degree. In the extreme case, the shifting of the current causes chute channels to develop across the point bar at high stages.

6.3.3 Bed and Bank Material

Bed material is the sediment mixture of which the streambed is composed. Bed material ranges in size from huge boulders many feet in diameter to fine clay particles. The erodibility or stability of a channel largely depends on the size of particles in the bed. It is often not sufficient to just know the median bed-material size (D_{50}) in determining the potential for degradation; knowledge of the bed-material size distribution is important. As water flows over the bed, smaller particles that are more easily transported are carried away, while larger particles remain, armoring the bed. The armoring process is an important concept for understanding alluvial channel response.

The armoring process begins as the nonmmoving coarser particles segregate from the finer material in transport. The coarser particles are gradually worked down into the bed, where they accumulate in a sublayer. This generally represents the lowest level to which the bed is turned over by the bed form movement that accompanies the transport process. Fine bed material is leached up through this coarse sublayer to augment the material in transport. As movement continues and degradation progresses, an increasing number of non-moving particles accumulate in the sublayer. This accumulation interferes with the leaching of fine material so that the rate of transport over the sublayer is not maintained at its former intensity. Eventually, enough coarse particles accumulate to shield, or "armor" the entire bed surface (Plate 6.3). When fines can no longer be leached from the underlying bed, degradation is

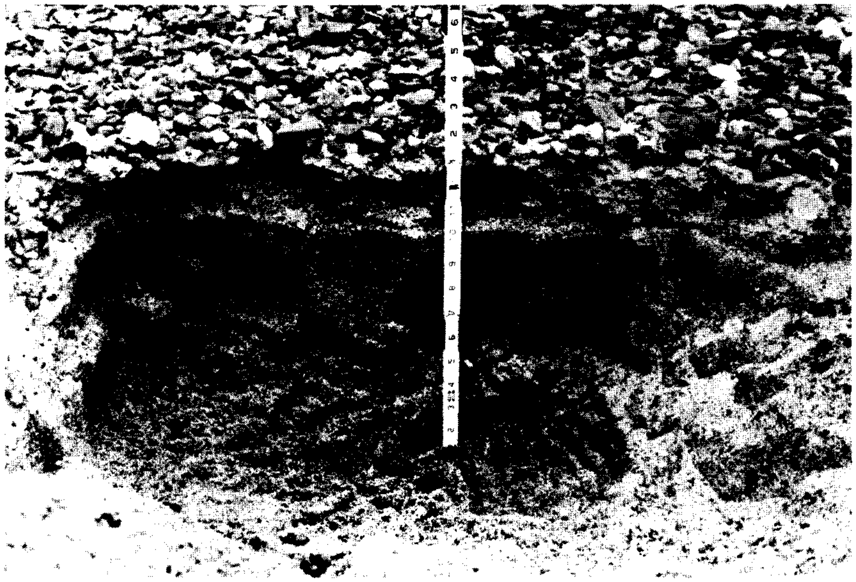


Plate 6.3. Typical armoring situation.

arrested. This final condition is similar to a riprapped channel with a granular filter layer.

Examination of typical armor layers reveals several important characteristics:

1. Less than a single complete covering layer of larger gravel particles seems to suffice for a total armoring effect for a particular discharge.

2. A natural "filter" apparently develops between the larger surface particles and the subsurface material to prevent leaching of the underlying fines.

3. The shingled arrangement of surface particles is not restricted to the larger material but seems evident throughout the gravel gradation.

An armor layer sufficient to protect the bed against moderate discharges can be disrupted during high flow, but may be restored as flows diminish. It is evident that an armor layer will tend to accumulate in areas of natural scour in the channel or stream, such as on the upstream end of islands and bars.

Bank material is in general made up of smaller or the same sized particles as the bed. Thus, banks are often more easily eroded than the bed unless protected by vegetation, cohesiveness, or some type of man-made protection. Stream banks can be classified according to stability by consideration of vegetation, cohesiveness, frequency of protection, lateral migration tendencies of the stream, etc.

6.3.4 Lane Relation

A basic physical process that occurs in a stream is its pursuit, in the long run, of a balance between the product of water flow and channel slope and the product of sediment discharge and size. The most widely known geomorphic relation embodying the equilibrium concept is known as Lane's principle (Figure 6.3).

Lane (1953) studied the changes in river morphology caused by modifications of water and sediment discharges. Similar but more comprehensive treatments of channel response to changing conditions in rivers have been presented by Leopold and Maddock (1953), and others. All research results support the following general statements:

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

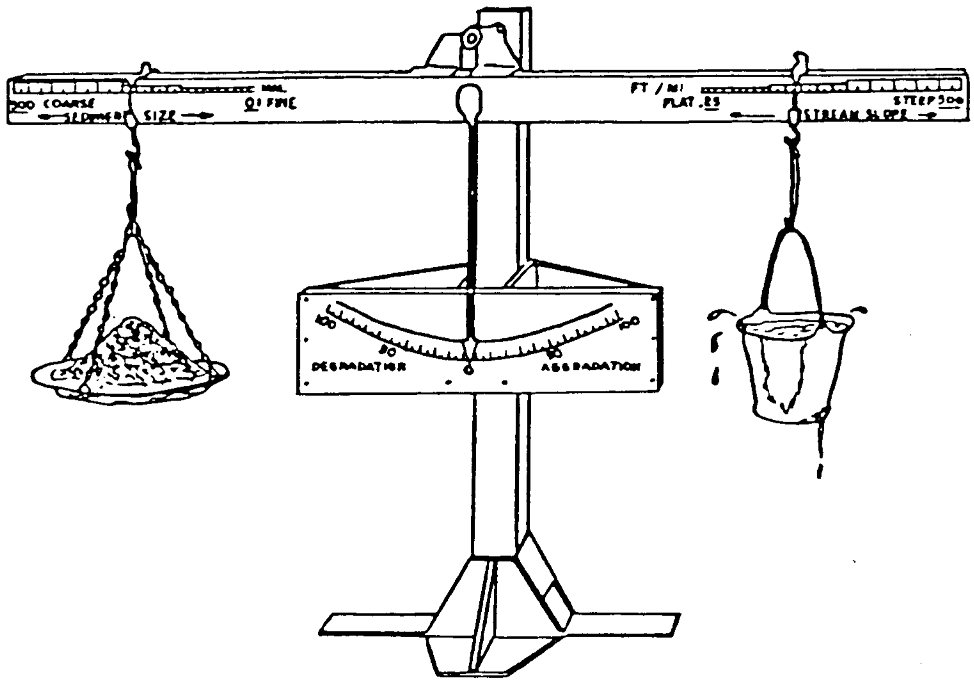


Figure 6.3. Schematic of the Lane relationship for qualitative analysis.

2. Width of channel is directly proportional to water discharge and to sediment discharge.

3. Shape of channel expressed as width-depth ratio is directly related to sediment discharge.

4. Meander wavelength is directly proportional to water discharge and to sediment discharge.

5. Slope of stream channel is inversely proportional to water discharge and directly proportional to sediment discharge and grain size.

6. Sinuosity of stream channel is proportional to valley slope and inversely proportional to sediment discharge.

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

A mathematical statement of the above principles is (Lane, 1953):

$$QS \qquad Q_s D_{50} \tag{6.1}$$

where Q is the water discharge, S is the channel slope, Q_s is the sediment discharge and D_{50} is the median diameter of the bed material.

6.3.5 Shields' Relation

An evaluation of relative stability can be made by evaluating the incipient motion parameters. The definition of incipient motion is based on the critical or threshold condition where the hydrodynamic forces acting on the grain of sediment particles have reached a value that, if increased even slightly, will move the grain. Under critical conditions, or at incipient motion, the hydrodynamic forces acting on the grain are just balanced by the resisting forces of the particle. The initiation of motion is involved in many geomorphic and hydraulic problems such as local scour, slope stability, stable channel design, etc. These problems can only be handled when the threshold of sediment motion is fully understood.

The beginning of motion of bed material is known to be a function of the dimensionless number (see Simons and Senturk, 1977).

$$F_* = \tau_c/(\gamma_s - \gamma)\ D_s \tag{6.2}$$

where τ_c is the critical boundary shear stress, γ_s and γ are the specific weights of the sediment and water, respectively, and D_s is a characteristic diameter of the sediment particle. This parameter (F_*) is often referred to as the Shields parameter. Shields determined a graphical relationship between this parameter and the shear velocity Reynolds number (R_*) for defining incipient motion (Figure 6.4). This relationship, known as the Shields diagram, was developed by measuring bed-load transport for various values of $\tau/(\gamma_s - \gamma)\ D_s$ at least twice as large as the critical value and then extrapolating to the point of vanishing bed load. This indirect procedure was used to avoid the implications of the random orientation of grains and variations in local flow conditions that may result in grain movement even when $\tau/(\gamma_s - \gamma)\ D_s$ is considerably below the critical value.

In the region where R_* is 70-500 the boundary is completely rough and F_* is considered independent of R_*. Numbers for the constant value of F_* in this region range from 0.047 to 0.060, or

$$\frac{\tau_c}{(\gamma_s - \gamma)\ D_s} = 0.047 \rightarrow 0.060 \qquad (6.3)$$

6.3.6 Sediment Transport Equations

Sediment transport equations are used to determine the sediment transport capacity for a specific set of flow conditions. Many formulas have been developed since DuBoys first presented his tractive force equation in 1879. The first step in evaluating sediment transport is to select one or more of the available equations for use in solving the given problem. The selection is not straightforward, since the results of different formulas can give drastically different results, and it is usually not possible to positively determine the one providing the best result. Additionally, some of the methods are considerably more complex than others. The initial consideration is to decide what portion of the sediment transport needs to be estimated. If it is desirable to know the contribution of the bed load and the suspended load to the bed-material discharge, formulas for each are available. Other formulas provide direct determination of the bed-material discharge. A common feature of all sediment transport equations is that the washload is not included since it is governed by upstream supply.

A second consideration in deciding what formula(s) to use is the type of stream or river conditions that exist. It is important to select a formula that was developed under conditions similar to those of the given problem. For example, some formulas were developed from data collected in sand-bed

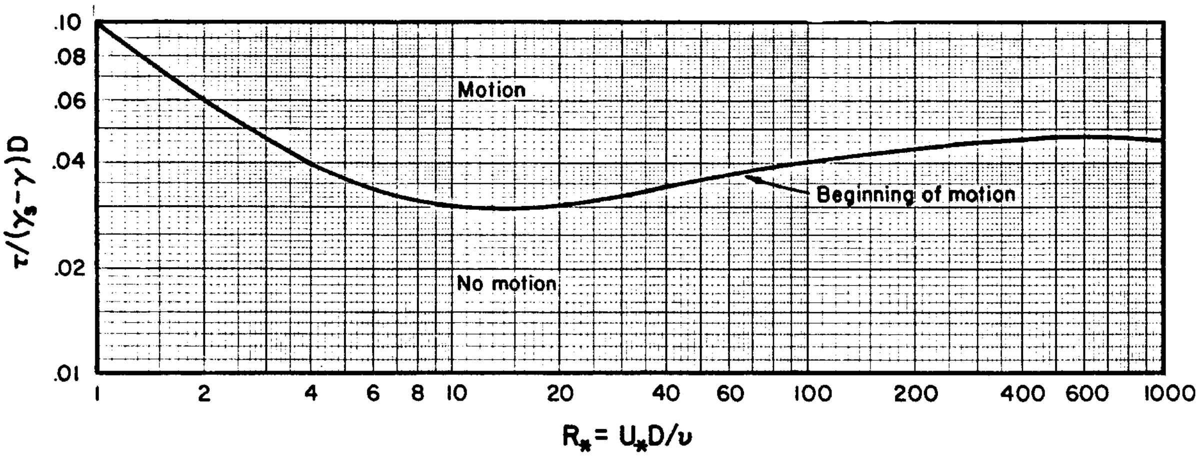


Figure 6.4. Shields' Diagram.

streams where most of the sediment transport was suspended load, while other equations are based on conditions of predominantly bed-load transport.

In addition to the use of purely analytical or empirical formulas, there are methods available for evaluating the bed-material discharge based on measured suspended load and other normal stream flow measurements. By use of observed data these results are usually more accurate and reliable than those given by other formulas. Unfortunately, measured data are often not available for the desired stream location, or the data are not recent enough or of long enough duration to provide sufficient accuracy.

Considering these factors, the relationship is presented below is recommended for application in OSM Regions I and II. It is a commonly used and well accepted method for computing the bed-material discharge in a cobble-bed stream. In using any sediment transport methodology, consideration should be given to solution by size fraction. Different transport capacities can be expected for different sediment sizes and some loss in accuracy may result from a calculation based on a single representative grain size. Solution of the total bed-material discharge by size fraction analysis is based on weighted average of the sediment transport for each given size.

Meyer-Peter, Muller Equation. The Meyer-Peter, Muller Equation (MPM) is a simple and commonly used equation for evaluating the bed material transport in a cobble-bed stream. Most of the data used in developing the equation were obtained in flows with little or no suspended load. A common form of the equation is (U.S. Bureau of Reclamation, 1960):

$$q_b = \frac{12.85}{\sqrt{\rho}\ \gamma_s} (\tau_o - \tau_c)^{1.5} \tag{6.4}$$

where q_b is the bed-load transport rate in volume per unit width for a specific size of sediment, τ_o is the tractive force (boundary shear stress), τ_c is the critical tractive force, ρ is the density of water and γ_s is the specific weight of sediment. The critical tractive force is defined by the Shields parameter (Equation 6.2). The tractive force or boundary shear stress acting under the given flow conditions is defined by

$$\tau_o = \frac{1}{8} \rho\ f\ V^2 \tag{6.5}$$

where ρ is the density of the flowing water and f is the Darcy-Weisbach friction factor.

A general form of the MPM equation was presented by Shen (1971) as

$$q_b = a_4 (\tau_o - \tau_c)^{b_4} \tag{6.6}$$

in which a_4 and b_4 are constants. When the constants in this equation are calibrated with field data, good results are usually obtained.

6.4 Stable Alluvial Channel Design - Method of Maximum Permissible Velocity

6.4.1 General Procedure

Two major variables affecting channel design and sediment transport are velocity and shear stress. In reality, determining shear stress is usually difficult. Therefore, velocity is often accepted as the most important factor when designing stable alluvial channels using the static equilibrium approach. The procedure is based on the condition that if the adopted mean velocity is lower than maximum permissible velocity (or the nonerodible velocity), the channel is assumed to be stable (Fortier and Scobey, 1926).

Appreciable work has been devoted to developing the permissible velocity approach. Many limits have been suggested for the permissible velocity under given conditions; however, experience has identified discrepancies in these values. For example, channels carrying sediment may be stable at velocities higher than the given limiting velocity. Consequently Fortier and Scobey (1926) introduced a certain increase in their listed values of maximum permissible velocities when water was transporting colloidal silt. The authors emphasized the importance of exercising judgment on each particular problem. Subsequently these limits were recommended by a Special Committee on Irrigation Research, ASCE. Since then many designs have been based on their suggested permissible velocities.

Table 6.1a summarizes the permissible velocities given by Fortier and Scobey. Other tabular listings of permissible velocity are given in Tables 6.1b, 6.1c and 6.1d.

The design procedure for a trapezoidal channel using the maximum permissible velocity consists of the following steps (Chow, 1959):

Table 6.1a. Maximum Permissible Velocities Tables by Fortier and Scobey (1926).

Original Material Excavated For Canals	n	Mean velocity of canals after aging (d≲3 ft): Clear water, no detritus ft/sec	Clear water, no detritus m/sec	Water transporting colloidal silt ft/sec	Water transporting colloidal silt m/sec	Water transporting noncolloidal silts, sands gravels or rock fragments ft/sec	Water transporting noncolloidal silts, sands gravels or rock fragments m/sec
1. Fine sand (colloidal)	0.02	1.5	0.46	2.50	0.76	1.50	0.46
2. Sandy loam (noncolloidal)	0.02	1.45	0.53	2.50	0.76	2.00	0.61
3. Silt loam (noncolloidal)	0.02	2.00	0.61	3.00	0.91	2.00	0.61
4. Alluvial silt when noncolloidal	0.02	2.00	0.61	3.50	1.07	2.00	0.61
5. Ordinary firm loam	0.02	2.50	0.76	3.50	1.07	2.25	0.69
6. Volcanic ash	0.02	2.50	0.76	3.50	1.07	2.00	0.61
7. Fine gravel	0.02	2.50	0.76	5.00	1.52	3.75	1.14
8. Stiff clay (very colloidal)	0.025	3.75	1.14	5.00	1.52	3.00	0.91
9. Graded, loam to cobbles, when noncolloidal	0.03	3.75	1.14	5.00	1.52	5.00	1.52
10. Alluvial silt when colloidal	0.025	3.75	1.14	5.00	1.52	3.00	0.91
11. Graded, silt to cobbles, when colloidal	0.03	4.00	1.22	5.50	1.68	5.00	1.52
12. Coarse gravel (noncolloidal)	0.025	4.00	1.22	6.00	1.83	6.50	1.98
13. Cobbles and shingles	0.035	5.00	1.52	5.50	1.68	6.50	1.98
14. Shales and hard pans	0.025	6.00	1.83	6.00	1.83	5.00	1.52

Table 6.1b. Maximum Permissible Velocities Tables by Etcheverry (1916).

Material	Mean Velocity (fps)
Very light pure sand of quicksand character	0.75- 1.00
Very light loose sand	1.00- 1.50
Coarse sand or light sandy soil	1.50- 2.00
Average sandy soil	2.00- 2.50
Sandy loam	2.50- 2.75
Average loam, alluvial soil, volcanic ash soil	2.75- 3.00
Firm loam, clay loam	3.00- 3.75
Stiff clay soil, ordinary gravel soil	4.00- 5.00
Coarse gravel, cobbles, shingles	5.00- 6.00
Conglomerates, cemented gravel, soft slate, tough hard-pan, soft sedimentary rock	6.00- 8.00
Hard rock	10.00-15.00
Concrete	15.00-20.00

Table 6.1c. Maximum Permissible Velocities Tables[1] by U.S. Army Office (1970).

Channel Material	Mean Channel Velocity (fps)
Find sand	2.0
Coarse sand	4.0
Fine gravel[2]	6.0
Earth	
Sandy silt	2.0
Silt clay	3.5
Clay	6.0
Grass-lined earth (slopes < 5%)[3]	
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

[1] Based on TM 5-886-4 and CE Hydraulic Design Conferences of 1958-1960.

[2] For particles less than fine gravel (about 20 mm = 3/4 in.).

[3] Keep velocities less than 5.0 fps unless good cover and proper maintenance can be obtained.

*May be used with judgment in durable bedrock.

Table 6.1d. Formulas for Maximum Permissible Velocity for Canals Constructed in Alluvium.

1. Mavis, et al. (1937)

$$(V_b) = \frac{1}{2} D^{4/9} \sqrt{\frac{\rho_s}{\rho} - 1}$$

D = size of particle in millimeters
(V_b) = Maximum permissible velocity at the bottom, ft/sec
ρ_s = density of particle in lb-sec^2/ft^4
ρ = density of water in lb-sec^2/ft^4

2. Carstens (1966)

$$\frac{V_b^2}{(\frac{\rho_s}{\rho} - 1)\ gD} = 3.61\ (\tan\phi\ \cos\alpha - \sin\alpha)$$

α = slope of plane bed, English units.
ϕ = natural angle of repose

3. Neill (1967)

$$\frac{V_{per}^2}{(\frac{\rho_s}{\rho} - 1)\ gD} = 2.5\ (\frac{D}{d})^{-0.20}$$

d = flow depth, ft
English units.

4. Mirtskhulava, T. E.

$$V_{per} = (\log \frac{8.8d}{D}) \sqrt{\frac{2g}{0.44\sqrt{n}}\ (\gamma_s - \gamma)\ D}$$

Metric units are required,
D > 2 mm

$$n = 1 + \frac{D}{0.00005 + 0.3D}$$

and

V_{per} = Maximum permissible mean velocity in mps

1. For the given kind of material forming the channel body, estimate the roughness coefficient n (Section 4.5), side slope z (Table 4.3), and the maximum permissible velocity V.

2. Compute the hydraulic radius R by the Manning formula (Equation 4.13).

3. Compute the water area required by the given discharge and permissible velocity, or $A = Q/V$.

4. Compute the wetted perimeter, or $P = A/R$.

5. Using the expressions for A and P from Table 4.1, solve simultaneously for b and y.

6. Add a proper freeboard, and modify the section for practicability.

6.4.2 Evaluating the Channel for Reasonable Shape

Following the design procedure using maximum permissible velocity can result in a very shallow, wide channel, as illustrated in the example at the end of the chapter. This type of cross section is clearly not desirable since the water would probably not flow uniformly across the entire width. Rather, it would tend to concentrate in one area by scouring a new deeper, narrower channel within the limits of the broader channel. Therefore, consideration must be given to the computed channel dimensions to insure they represent a practical design. Empirical formulas have been developed that provide guidance in assessing the practicality of a channel design. Some of the formulas used to evaluate depth of flow or the width-to-depth (b/d) ratio are given below.

1. U.S. Bureau of Reclamation procedure

$$d = 0.5\sqrt{A} \qquad (6.7)$$

A = Area in ft^2

and for a trapezoidal cross section

$$\frac{b}{d} = 4 - z \qquad (6.8)$$

2. Irrigation Service Procedure, India

$$d = \sqrt{A/3} \qquad (6.9)$$

and for a trapezoidal cross section

$$\frac{b}{d} = 3 - z \tag{6.10}$$

It should be noted that the preceding empirical formulas are simply guidelines. These equations do not apply to all conceivable flow conditions, nor do they differentiate between practical and impractical channel configurations.

Channel designs having width-to-depth (b/d) ratios significantly different from the empirical rules (Equations 6.7-6.10) should be evaluated further. It may be possible to improve the channel design by using a properly designed lining or installing grade control structures. These methods will be discussed in the following section.

6.4.3 Evaluation of the Need for Rock Riprap or Grade Control Structures

If the cross section determined from the stable channel design procedure (Section 6.4.1) is not economical or acceptable according to the b/d ratio (Section 6.4.2), then a more practical cross section can be designed by using a channel lining and/or grade control structures. A channel lining allows designing for a larger permissible velocity without scour or erosion of the channel. For instance, if the bedrock of the natural ground is a poor sedimentary rock strata with a low permissible velocity, a smaller channel lined with a durable riprap may be more economical and stable. Additionally, channel linings can be used to reduce or eliminate seepage losses from the channel. The reduction of seepage is not usually a major concern in a surface mine operation; however, it may become important in areas of the mine site where seepage could cause water quality or stability problems. Possible stability problems from seepage include slippage along backfill areas, mass or surface sloughage of waste sites and bank sloughing in otherwise stable channels due to seepage pore pressure. Ideally, channel linings for diversion structures should be maintenance free and have a long design life, since they will have to remain "forever" after bond is released.

Grade control structures can reduce the velocity upstream of the structure to a nonerosive value. Multiple grade control structures can be used to control long reaches of a stream. The design procedures for channel linings and grade control structures follow.

6.5 Vegetative Linings

6.5.1 General

Vegetative linings can be a practical, economical method of channel protection in regions where the vegetation can be grown. Minor erosion damage to a vegetative lining often repairs itself where a rigid-type lining would progressively deteriorate unless repaired; however, it is well known that vegetative linings do not withstand large shear forces, nor do they easily survive long periods of submergence. Therefore, under these conditions, vegetative linings may be impractical and other linings such as rock riprap should be utilized. Often composite linings consisting of rock riprap in areas of high shear or long term submergence and vegetation in the remainder of the cross section can be utilized to reduce costs. Intermittently spaced vegetative diversions are commonly used on surface mine operations for long slopes of backfill areas and waste sites to collect drainage without gully erosion.

6.5.2 Design Procedure - Maximum Permissible Velocity

Since about 1935, many flow tests over common American and Australian grasses have been performed and summarized by Cox and Palmer (1948), Ree and Palmer (1949), and Eastgate (1966). In each test depth scour and general appearance of the channel was noted. Whenever conditions were such that unacceptable rates of scour and destruction of the channel lining occurred, the mean velocity of flow was noted. Then the maximum mean velocity the channel withstood without significant damage was suggested as the maximum permissible velocity. Velocities tabulated in Table 3 of the "Handbook of Channel Design for Soil and Water Conservation" are reproduced in Table 6.2.

It should be noted that maximum permissible velocity is generally less for steeper slopes. Also, velocities stated were often exceeded without damaging the experimental channels from which the data were derived. Of course, these channels were usually prepared with great care and under ideal conditions, resulting in vegetative linings of greater density and uniformity than those found in the field. Therefore, the designer should typically use slightly lower velocities to provide for a margin of error.

Design of vegetated channels is complicated by the fact that the relative roughness is a function of depth or hydraulic radius. The Soil Conservation Service has identified the degree of retardance by vegetation height according to data given in Table 6.3. Design charts given in Figures 6.5a to 6.5e can

Table 6.2 Permissible Velocities for Channels Lined with Vegetation.[1] The values apply to average uniform stands of each type of cover (Soil Conservation Service, 1954).

Cover	Slope Range[2] (percent)	Permissible velocity (fps) Erosion resistant soils	Easily eroded soils
	0-5	8	6
Bermudagrass	5-10	7	5
	over 10	6	4
Buffalograss			
Kentucky bluegrass . . .	0-5	7	5
Smooth brome	5-10	6	4
Blue grama	over 10	5	3
	[2]0-5	5	4
Grass mixture	5-10	4	3
Lespedeza sericea			
Weeping lovegrass			
Yellow bluestream	[3]0-5	3.5	2.5
Kudzu			
Alfalfa			
Crabgrass			
Common lespedeza[4]	[5]0-5	3.5	2.5
Sudangrass[2]			

[1]Use velocities exceeding 5 feet per second only where good covers and proper maintenance can be obtained.

[2]Do not use on slopes steeper than 10 percent except for side slopes in a combination channel.

[3]Do not use on slopes steeper than 5 percent except for side slopes in a combination channel.

[4]Annuals--used on mild slopes or as temporary protection until permanent covers are established.

[5]Use on slopes steeper than 5 percent is not recommended.

Table 6.3. Guide to Selection of Vegetal Retardance*.

Average height of vegetation (inches)	Degree of Retardance	
	Good Stand	Fair Stand
More than 30	A	B
11 to 24	B	C
6 to 10	C	D
2 to 6	D	D
Less than 2	E	E

*From U.S. Soil Conservation Service (1954).

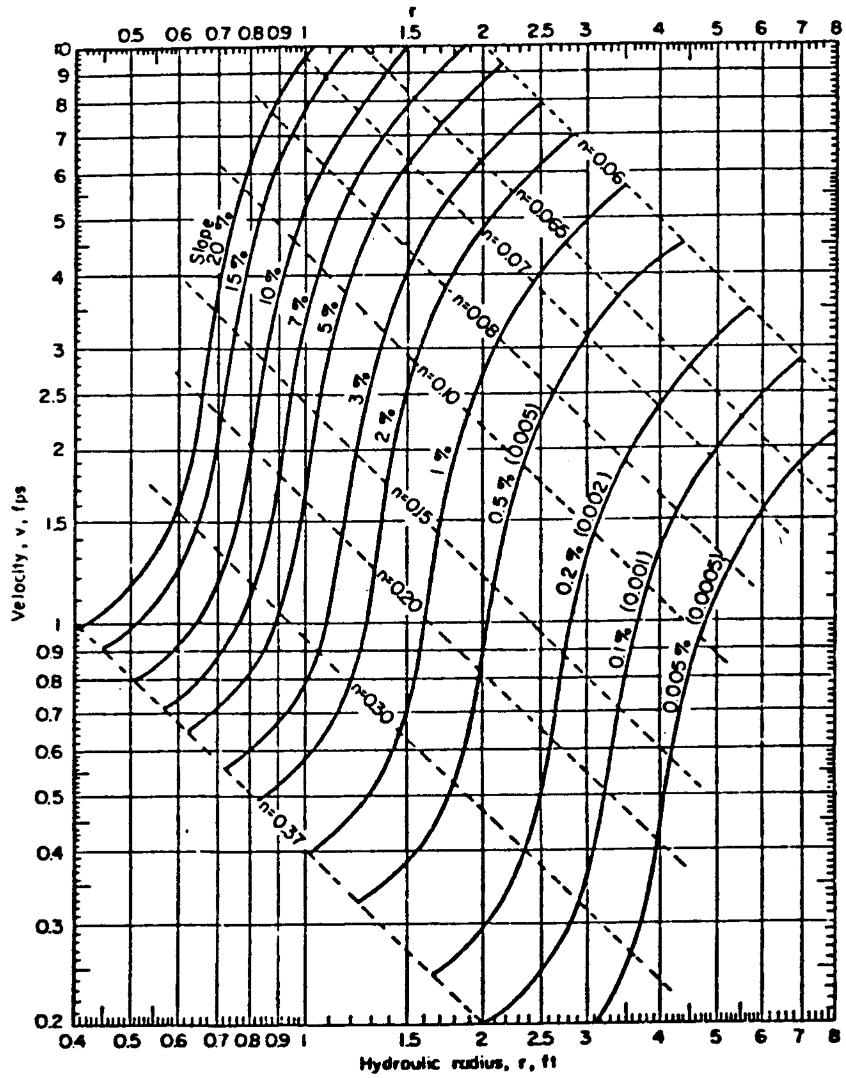


Figure 6.5a. Solution of the Manning equation for retardance A (very high vegetal retardance) (U.S. Soil Conservation Service).

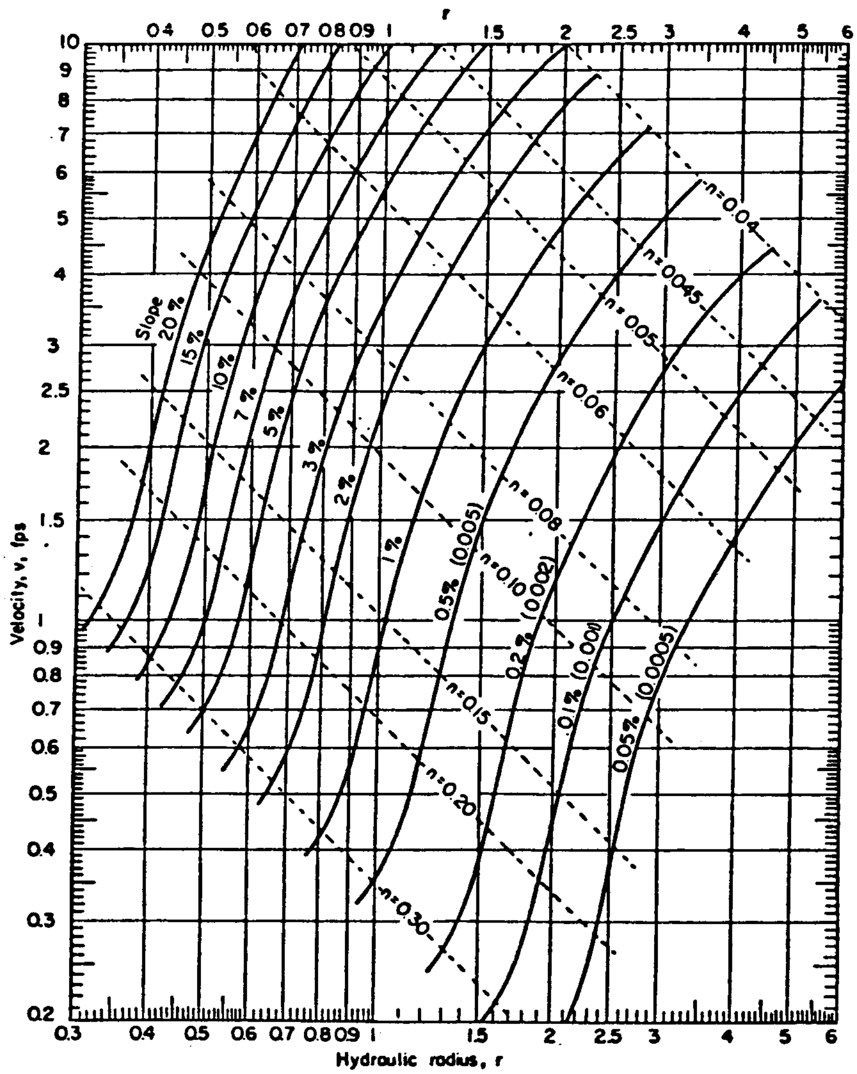


Figure 6.5b. Solution of the Manning equation for retardance B (high vegetal retardance). (U.S. SCS)

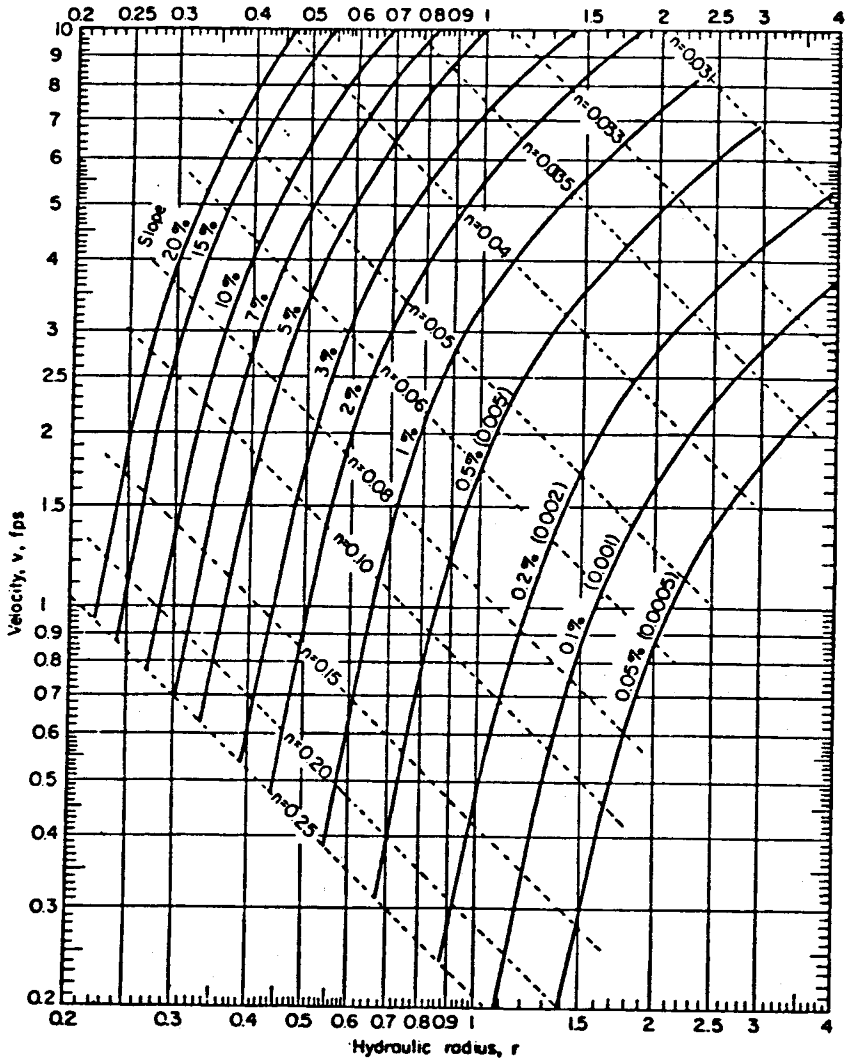


Figure 6.5c. Solution of the Manning equation for retardance C (moderate vegetal retardance). (U.S. SCS)

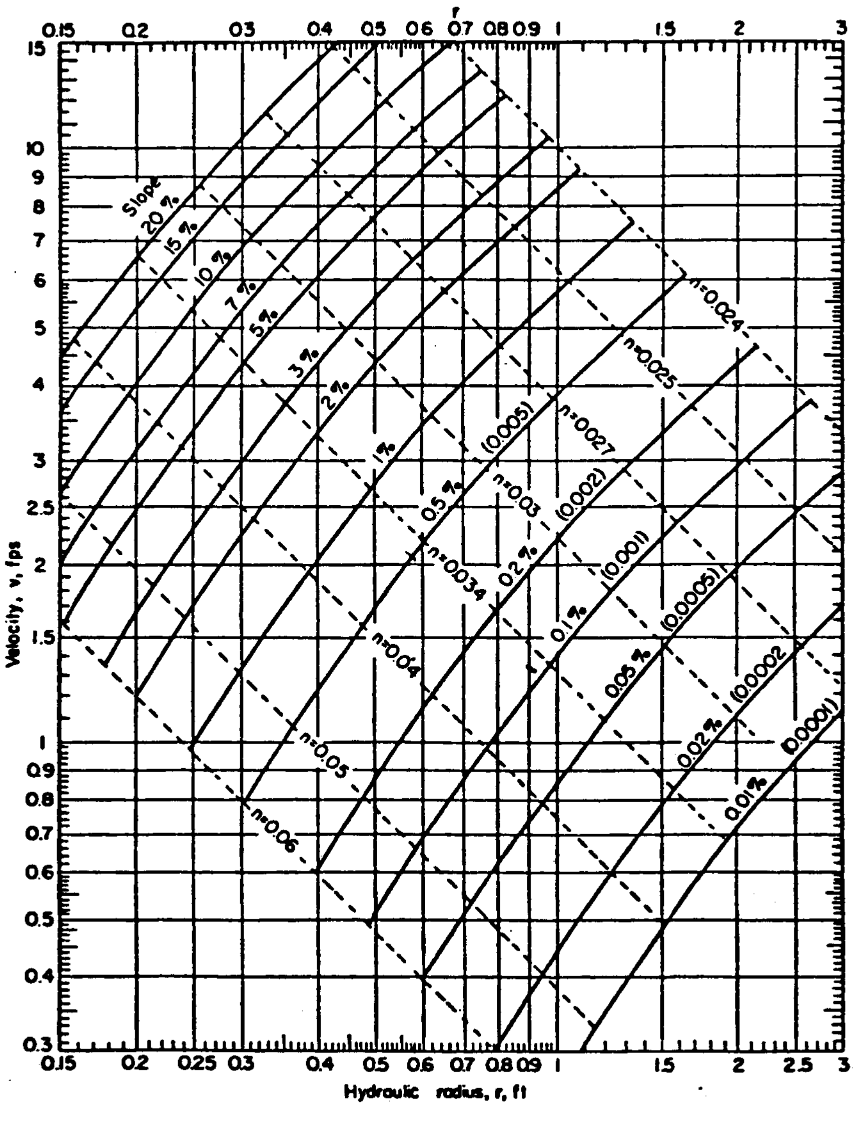


Figure 6.5d. Solution of the Manning equation for retardance D (low vegetal retardance). (U.S. SCS)

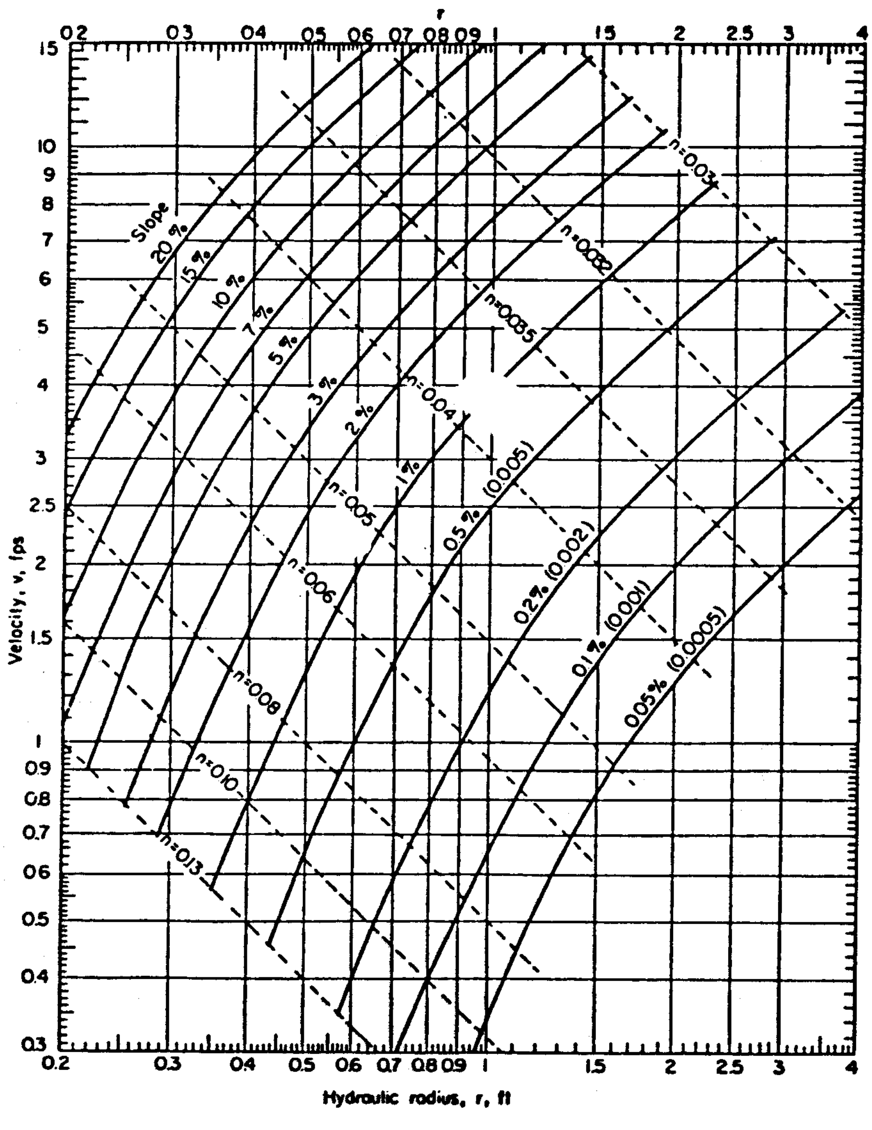


Figure 6.5e. Solution of the Manning equation for retardance E (very low vegetal retardance). (U.S. SCS)

then be used to solve the Manning equation, using the maximum permissible velocity for the given vegetation (Table 6.2). The design procedure involves two steps. First, the bottom width of the vegetated channel is determined so that the velocity is less than the maximum permissible velocity for the mowed condition of minimum retardance. Second, the channel depth is determined by the need to provide the design capacity under conditions of maximum retardance. The procedure is summarized in Section 6.8 and illustrated by an example in Section 6.9.

6.5.3 Composite Linings

Vegetation is also particularly suited for use in combination with other rigid lining materials to produce a composite lining. Velocities in a straight, uniform channel are generally greatest in the upper part of the middle portion. Velocities decrease toward the channel sides and bottom. Although the mean velocity might exceed the permissible value for a grass lining and thus require a higher cost lining, the mean velocity in the triangular section embracing the upper edge of the bank slope might be low enough for grass. The most economical solution would probably be the combination of a rigid-type lining in the lowest part of the channel and grass lining on the upper bank slopes.

Combination linings are also used where the channel bottom requires protection which could be furnished by a grass lining, but low flows of long duration, from snow melt or seepage, retard or prevent the growth of grass. In such a situation, the channel could be paved with a rigid-type lining to carry the low flow and with grass above the elevation of the continued low flow. Ree (1951) describes tests on composite linings in a channel on a ten-percent slope. Figure 6.6 is a reproduction of Ree's figure showing the dimensions and velocity distribution. Ree concluded that the usual practice of summing calculated discharge rates for the component parts of the cross section to give the capacity of a composite channel seems a valid method. Furthermore, he found that high velocities in the gutter section do not carry over appreciably to the grassed portion of the waterway and therefore concluded that observed scour at the junction cannot be attributed to excessively high velocities. However, based on the earlier discussion regarding the probability of high-velocity eddies intermittently reaching the bed and causing erosion, it seems prudent to provide some sort of apron. The apron

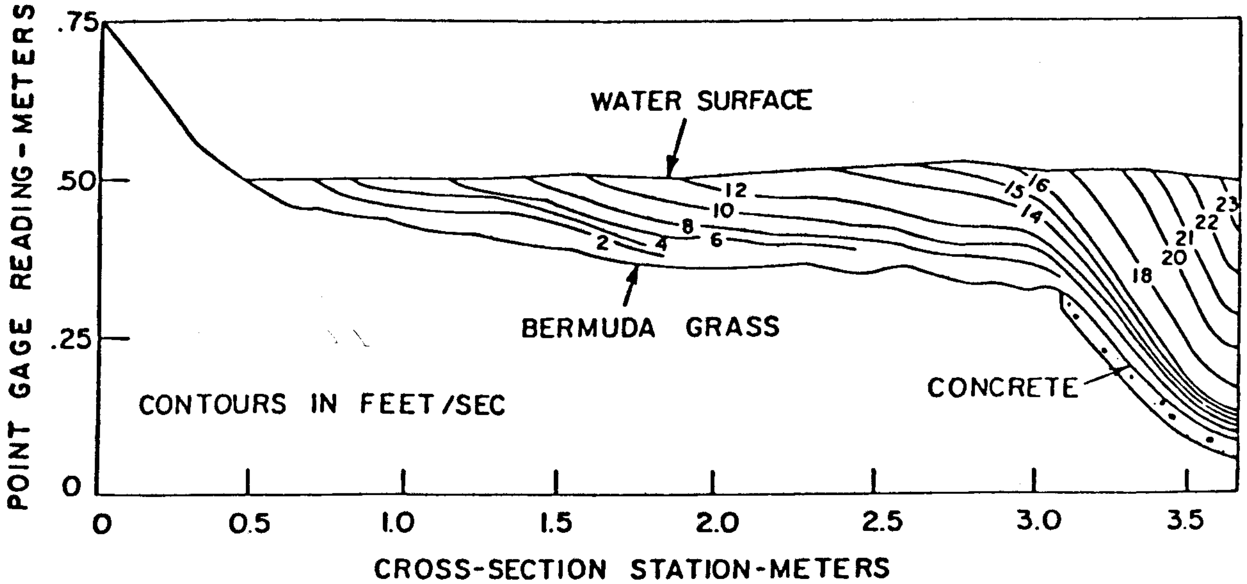


Figure 6.6. Dimensions and velocity distribution, Ree (1951).

should be designed so that the velocity profile will be continuous across the joint, thus preventing the formation of a shear zone and its resulting turbulence. Figure 6.7 shows two examples of such junctions. The surface of the rigid lining should line up with y', the velocity intercept of the flowing water, at design depth. The design of the riprap part of the cross section should be according to procedures outlined in Section 6.6.

6.5.4 Establishing Vegetative Linings

Temporary linings are flexible coverings used to protect a channel until permanent vegetation can be established. The lining materials are usually biodegradable and do not require removal after the vegetation becomes established. Some typical temporary linings tested by Mississippi State University in 1968 for the Mississippi State Highway Department are:

1. Erosionet 315 - a paper yarn with openings approximately 7/8 inch by 1/2 inch. Normally used to hold other materials such as straw. Secured with steel pins.

2. Jute mesh - a woven mat of coarse jute yarn with openings about 3/8 inch by 3/4 inch. Held in place with steel pins.

3. Stranded fiberglass roving with Erosion 315 - fine glass fibers blown onto the channel bed using compressed air and a special nozzle, and held in place with steel pins and Erosionet (see No. 1 above).

4. 3/8-inch fiberglass mat - a fine glass fiber mat similar to furnace air filter material held in place with steel pins.

5. 1/2-inch fiberglass mat - same as No. 4 above, except thicker and more dense. May retard seed germination and vegetation growth.

6. Excelsior mat - dried shredded wood held together with a fine paper net and secured with steel pins.

7. Straw with erosionet - chopped straw held in place with Erosionet and steel pins.

Chemical soil stabilizers are another means of protecting a channel until vegetation can be established. Chemical soil stabilizers are designed to coat and penetrate the soil surface and bind the soil particles together. They can be used both in lieu of temporary mulch material and in conjunction with the material to act as a mulch tack and soil binder. Chemical stabilizers generally work best on dry, highly permeable spoil, or in-place soils subject to sheet flow rather than concentrated flow.

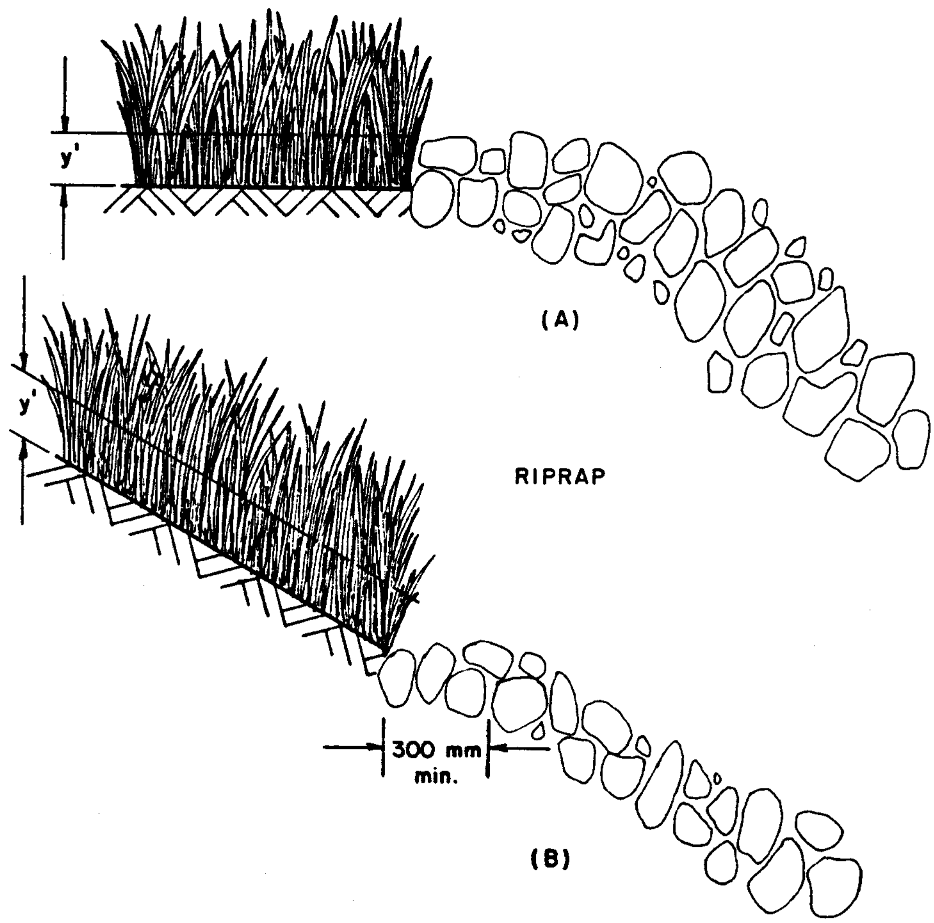


Figure 6.7. Detail of suggested grass to riprap junction.

6.6 Rock Riprap Design

6.6.1 General

Many procedures are available for designing rock riprap for mild slope channels. In this context the definition of mild is in the hydraulic sense (where the Froude number is less than one) and not in the topographic sense. The Froude number is based on velocity and flow depth, which both depend on channel size and roughness (i.e., riprap size); therefore, the designer must first assume the channel condition will be mild and proceed with the design (unless experience dictates otherwise). After evaluating the channel size and D_{50} riprap size, the designer must check the Froude number to insure that the mild slope assumption was correct and consequently that the procedure applied was valid. Regardless of the procedure used, the general concepts related to riprap design given in Section 5.1 must be followed.

A riprap design procedure adopted by the Denver Urban Drainage and Flood Control District provides a simple means for determination of riprap protection. The design procedure is based on the flow velocity V and hydraulic radius R. Defined riprap classes are selected according to the channel side slope and computed quantity $V^2/R^{0.33}$, where V is the velocity and R is the hydraulic radius. The primary advantage to this design methodology is the quick, simple determination of a stable channel lining. A limitation to this procedure is that it is only valid for subcritical flows where the Froude number (see Section 4.2.5) is less than 0.8. For mild slope Froude numbers between 0.8 and 1.0 the designer should use the steep slope design procedure (Section 5.3) which will give an adequate design, although slightly conservative.

Other simplified riprap design procedures include the methodology presented in National Cooperative Highway Research Program Report (NCHRP) No. 108, (Highway Research Board, 1970). This riprap design was developed from research performed at the University of Minnesota. One advantage to the NCHRP No. 108 design procedure is that is allows for design of the entire channel section based only upon design discharge Q and slope S. For this riprap design method, charts have been developed to provide solution for both a hydraulically efficient cross section as well as the riprap size required for stabilization. The Federal Highway Administration utilized the NCHRP riprap design methodology in its Hydraulic Engineering Circular No. 15 (Federal Highway Administration, 1975). However, the riprap design procedure was modified to conform with the concept of maximum permissible depth of flow, as used in the circular. Again, figures and charts have been developed to aid in design.

6.6.2 Recommended Riprap Design Procedure

Only the Denver Urban Drainage and Flood Control District riprap design methodology is presented in this manual. This method was selected due to its ease of understanding and application. Table 6.4 indicates the required riprap type for specific value of the parameter $V^2/R^{0.33}$. Ordinary riprap is classified and a gradation specified, according to criteria shown in Table 6.5. To insure the method is applicable to the given conditions the designer must check the Froude number criteria ($Fr < 0.8$) after determining the D_{50} size and channel dimensions. If the Froude criteria are not met, the steep slope riprap design procedure given in Section 5.3 must be used. Section 6.9.4 provides an example that illustrates the procedure.

6.6.3 Riprap Protection in Channel Bends

Flow around a bend in a channel generates secondary currents which in turn modify the velocity profile and shear stress distribution through the bend. The result of this modification in stresses is that the banks on the outside of the bend become more susceptible to erosion. For this reason, additional protection measures are often necessary in channel bends.

The Denver Urban Drainage and Flood Control District Drainage Manual specifies that riprap-lined channel bends should have a radius of curvature of at least two times the top width but no less than 50 feet.

For a specific ratio of channel top width to bend radius, Figure 6.8 can be used to determine the ratio of shear stress in a bend to shear in a straight channel. The ratio is then applied directly to the parameter $V^2/R^{0.33}$ used in the riprap design procedures. The riprap protection provided in the curve should be extended both upstream and downstream of the bend for a distance at least equal to the bend length.

6.7 Riprap Design with Grade Control Structures

6.7.1 Application

Where a long channel is to be constructed in an erodible material a more economical riprap design may be achieved through the use of strategically

Table 6.4. Riprap Requirements for Channel Linings in Mild Slope Channels ($F_r < 0.8$).

$V^2/R^{0.33}$	Channel Side Slope			
	4:1	3:1	2.5:1	2:1
20- 70	Type L	Type L	Type L	Type L
70- 90	Type L	Type L	Type L	Type M
90- 95	Type L	Type L	Type L	Type M
95-100	Type L	Type L	Type L	Type H
100-105	Type L	Type L	Type M	Type H
105-110	Type L	Type M	Type M	Type H
110-115	Type M	Type M	Type M	Type H
115-120	Type M	Type M	Type M	Type VH
120-125	Type M	Type M	Type M	Type VH
125-130	Type M	Type M	Type H	Type VH

Type L riprap should be buried to reduce vandalism.
Side slopes steeper than 2:1 should be designed as retaining walls.
Table valid for Froude numbers less than 0.8.

Table 6.5. Classification and Gradation of Ordinary Riprap for Mild Slope Channels ($F_r < 0.8$).

Riprap Designation	% Smaller Than Given Size by Weight	Minimum Dimension (inches)	K_m* (inches)
	100	9**	
Type VL	35-55	6	6***
	10	2	
	100	12**	
Type L	35-55	9	9***
	10	2	
	100	18**	
Type M	35-55	12	12
	10	3	
	100	24**	
Type H	35-55	18	18
	10	6	
	100	36**	
Type VH	35-55	24	24
	10	6	

*K_m = mean particle size, equivalent to D_{50}

**At least 30% of all stones by weight shall be this dimension.

***Bury types VL and L with native soil to protect from vandalism damage.

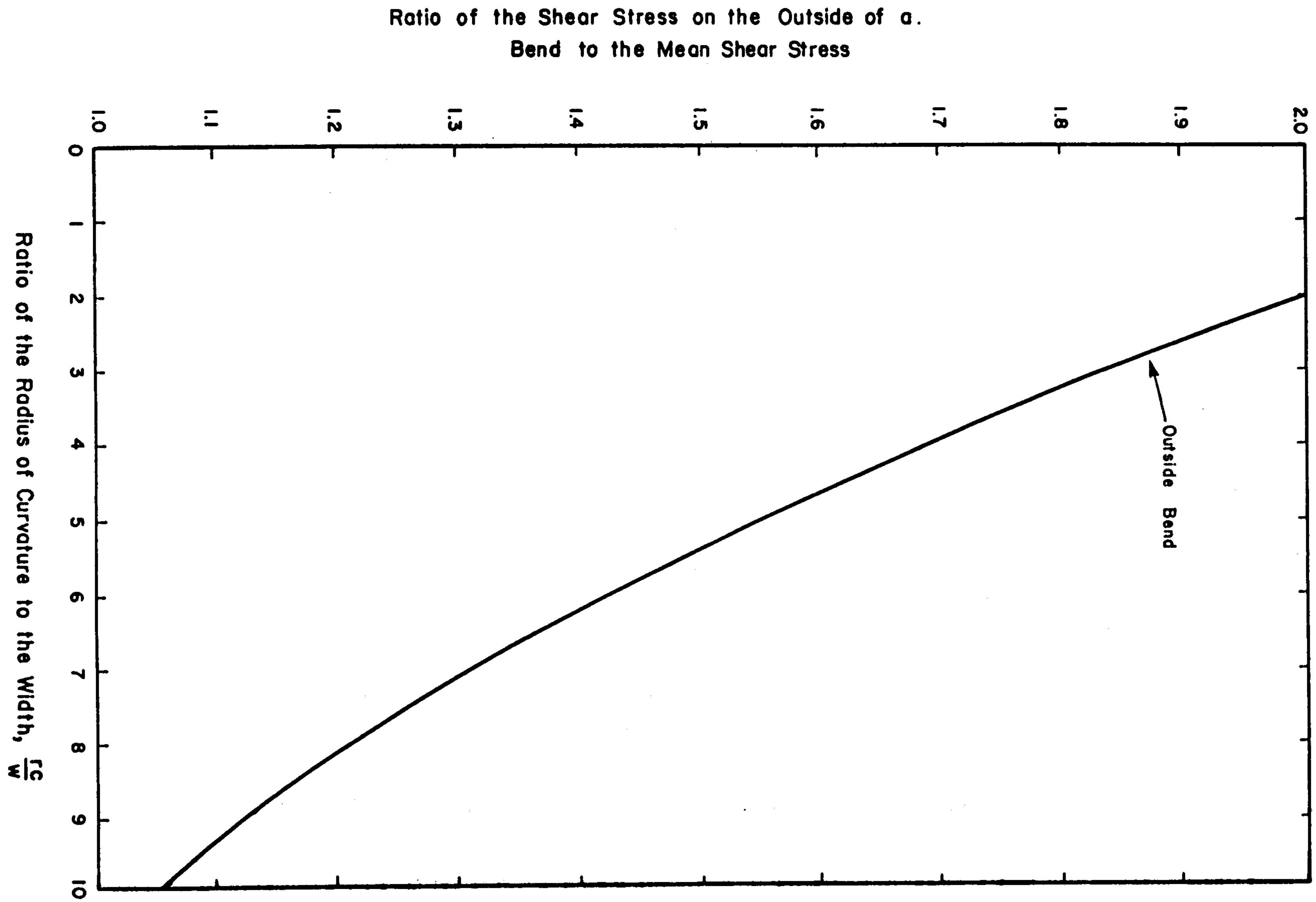


Figure 6.8. Effect of bend on boundary shear stress (after Soil Conservation Service design manual).

placed grade control structures. A grade control structure can be used to decrease the gradient of a channel to some slope where a smaller size rock will be stable. If sufficient coarse material exists in the natural alluvium, it may be possible to develop an armor layer (see Section 6.3.3) and avoid the need for riprapping entirely.

The design procedure is based on the static equilibrium slope for the given particle size. The static equilibrium slope is that slope where the particles remaining on the bed and banks of the channel are not transportable by the flow. For example, if a certain rock size is available for riprap at the mine site, the maximum slope (static equilibrium slope) at which that rock will be stable for the design flow can be determined. If the slope of the natural terrain is greater than the static equilibrium slope, then drop structures can be used to achieve the required static equilibrium slope. Similarly, if gravel-cobble type material exists in the natural alluvium, the slope at which the D_{50} of this material will be stable can be determined. If this slope is obtainable through grade control structures, then riprap will not be necessary.

To determine the feasibility of grade control structures, the costs of riprapping the channel with large rock at the natural slope of the terrain must be compared to: (1) costs of excavation to achieve a smaller slope, (2) installation of drop structures, and (3) riprapping with a smaller size rock. Additionally, the ecological impacts of grade control structures on fish habitat in perennial streams must be considered. The primary concern with the installation of many closely spaced grade control structures is the restriction they might have on fish movement. One additional ecological consideration is necessary if grade control structures are being used to achieve a static equilibrium slope based on the development of an armor layer. This procedure implicitly assumes that channel stability is attainable at some reduced slope by allowing limited degradation to occur. The degradation process involves sorting of the particles comprising the natural alluvium to achieve the armor layer. The downstream sediment loading resulting from this process must be compared to background sediment concentrations to establish if adverse environmental impacts will occur.

6.7.2 Types of Grade Control Structures

Grade control structures can range in complexity from simple rock riprap type drop structures to concrete structures with baffled aprons and stilling basins. For the range of discharges and velocities typically expected on a surface mine site, and considering the construction techniques typically employed, only the design of rock riprap structures is covered in this manual. Figure 6.9 illustrates a loose rock drop structure.

General guidelines for construction of loose rock drop structures constructed in mild slope channels are similar to stone check dams. The following specific recommendations are made:

1. Maximum drop height of three feet (guidelines for designing loose rock drop structures for drop heights greater than three feet are given in the Part 2.

2. Top width no less than five feet.

3. Downstream slope of 2 horizontal to 1 vertical.

4. 25 percent of the rock by volume will be 18 inches or larger. The remaining 75 percent shall be well graded material consisting of sufficient rock small enough to fill the voids between the larger rocks.

5. Energy dissipation should be provided at the downstream toe of a structure with a small plunge pool and large rocks.

6.7.3 Design Procedure Involving Grade Control Structures

Development of the graphical design procedure presented below is detailed in Appendix D. The design procedure is based on an application of Shields' relation (Equation 6.3) and the Manning equation (Equation 4.13). The primary design relationship is

$$S = \frac{0.047\ (G_s - 1)\ D_{50}}{R} \qquad (6.11)$$

where S is the static equilibrium slope, G_s is the specific gravity of the bed and bank material, often assumed to be 2.65, D_{50} is the median riprap size available or the armor particle size present in the natural alluvium, and R is the hydraulic radius.

The relationship defining R for a given combination of Manning's n, discharge Q and D_{50} is given in Figures 6.10a to 6.10c, where K is defined as

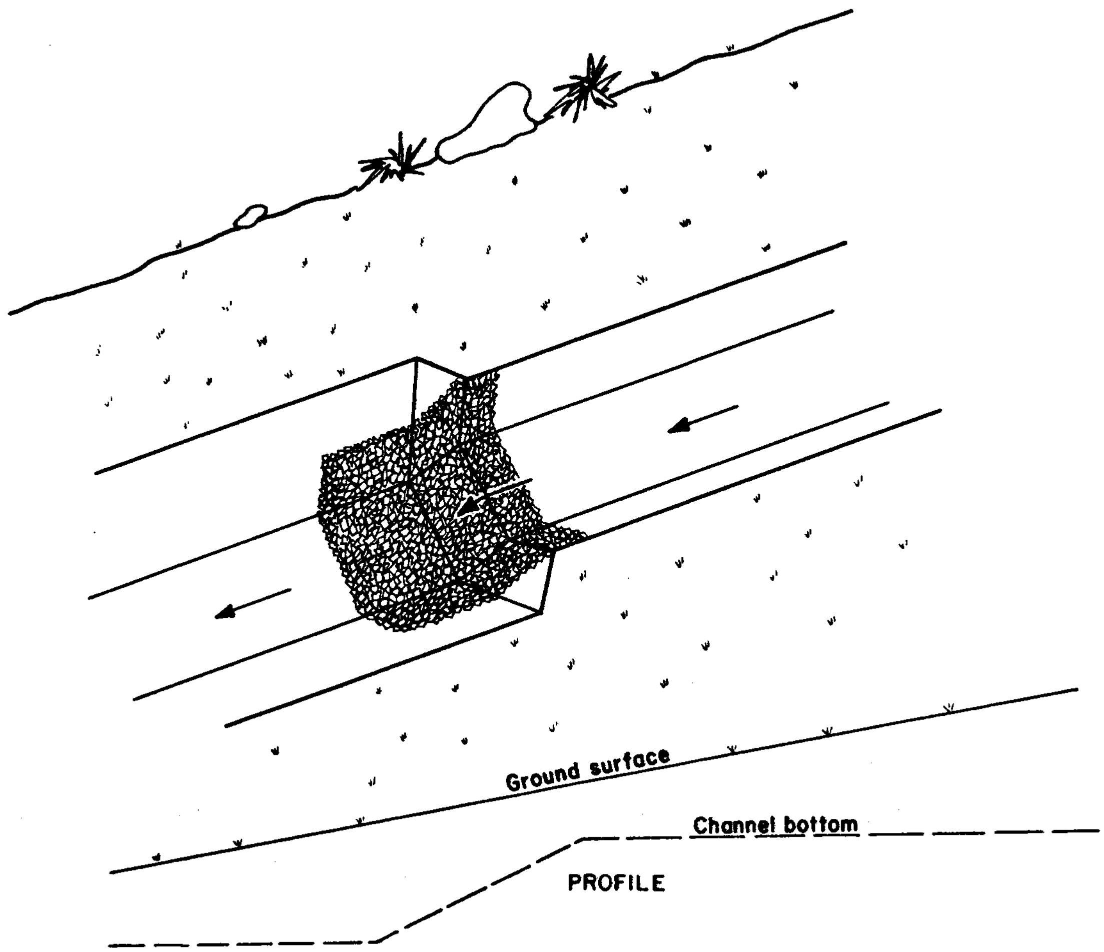


Figure 6.9. Definition sketch of a rock riprap drop structure (protection upstream and downstream according to Section 5.4).

$$K = \left(\frac{Qn}{0.323\sqrt{(G_s-1)\ D_{50}}}\right)^6 \tag{6.12}$$

For values of K beyond the limits given in the figures, Equation D.9 in Appendix D must be solved.

The design procedure using these figures is simple to apply. After establishing the D_{50} of the available riprap, or the natural alluvium for development of an armor layer, the value of K is computed for the design flow Q and the representative Manning n. For gravel-cobble size rock Equation 4.18 gives a good estimate of the Manning n. With K established, the value of R is determined from the graphs. Equation 6.11 can then be solved for the static equilibrium slope required to maintain stability for the given D_{50} and flow conditions. If the natural terrain slope is less than the computed static equilibrium slope, the riprap will be stable without the need for drop structures. Otherwise, drop structures will be needed to establish the required slope.

6.7.4 Spacing of Grade Control Structures

If the above computation indicates grade control structures are required, the number and spacing of the structures must be determined. The vertical height that must be controlled for the given reach to achieve the required static equilibrium slope can be evaluated from

$$\Delta H = (S_o - S)\ \Delta X \tag{6.13}$$

where ΔH is the total height requiring structural control, S_o is the original channel slope, S is the estimated static equilibrium slope, and ΔX is the length of channel to be controlled.

To prevent highly erosive velocities at the base of a rock riprap drop structure, the maximum allowable height of the structure is three feet. Therefore, the number of structures N required to control the total vertical height is

$$N = \frac{\Delta H}{3} \tag{6.14}$$

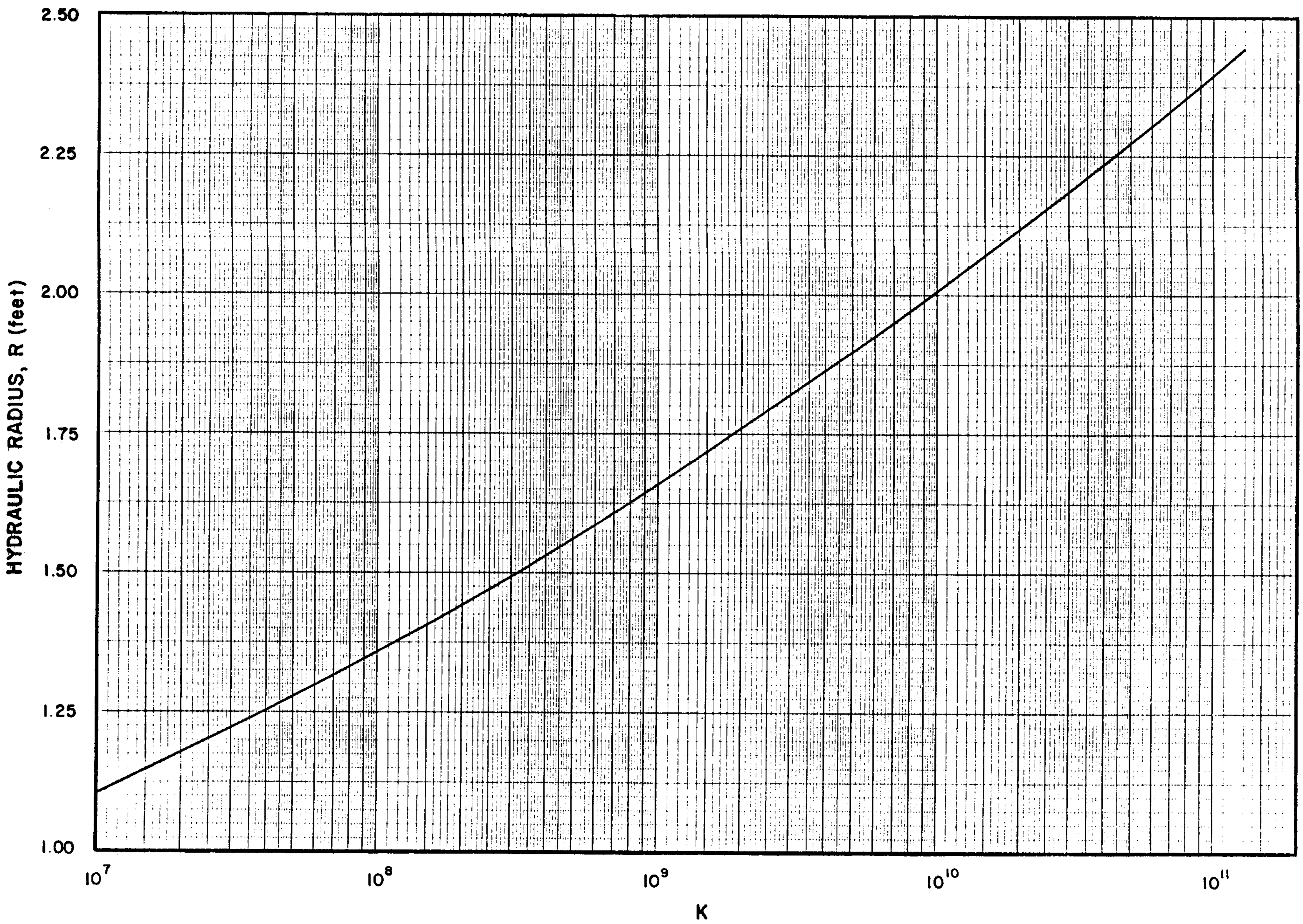


Figure 6.10a. Relationship between hydraulic radius R and K for trapezoidal channel with 2:1 side slopes and 6-foot base width.

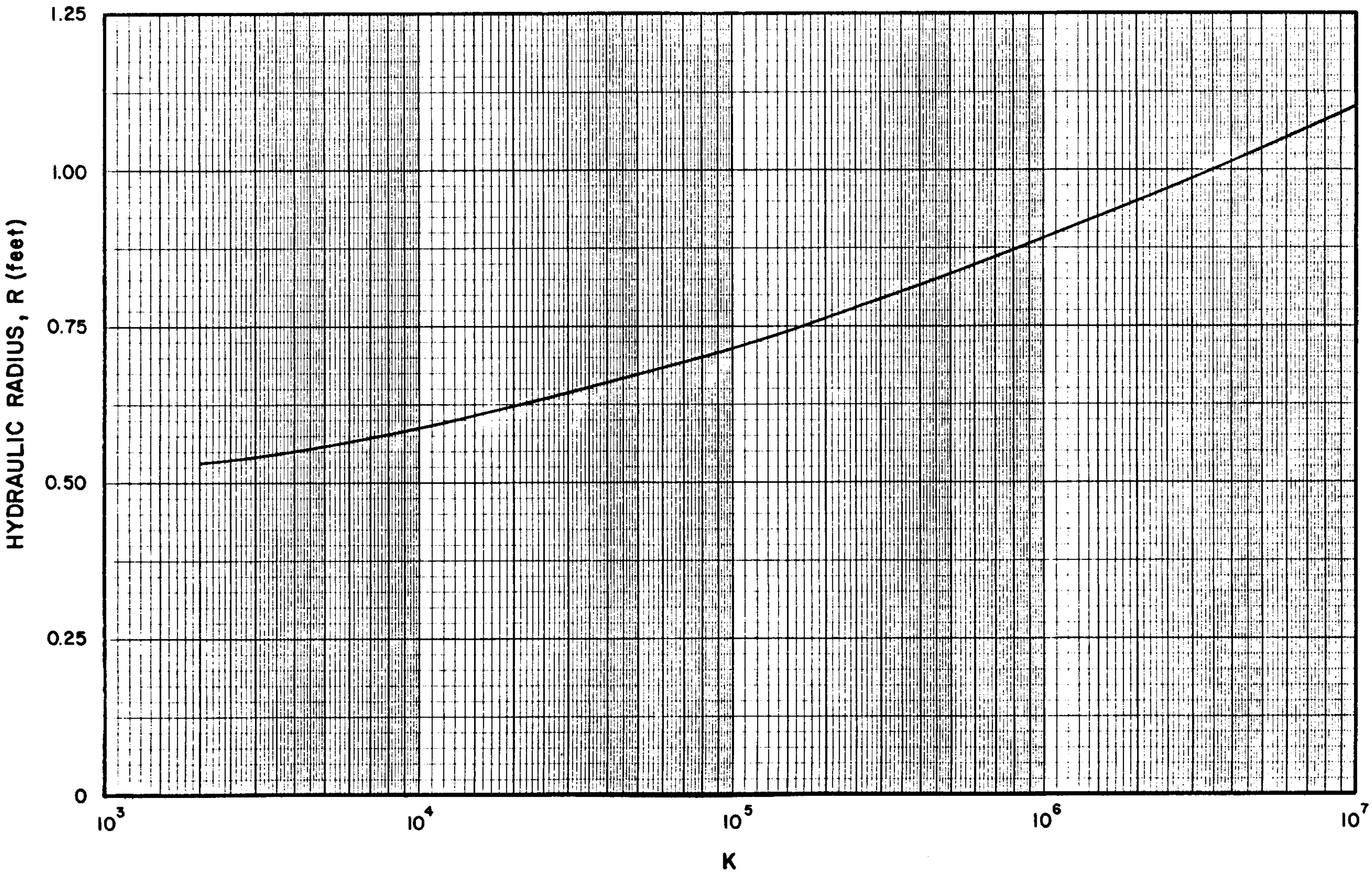


Figure 6.10a. Relationship between hydraulic radius R and K for trapezoidal channel with 2:1 side slopes and 6-foot base width. (continued).

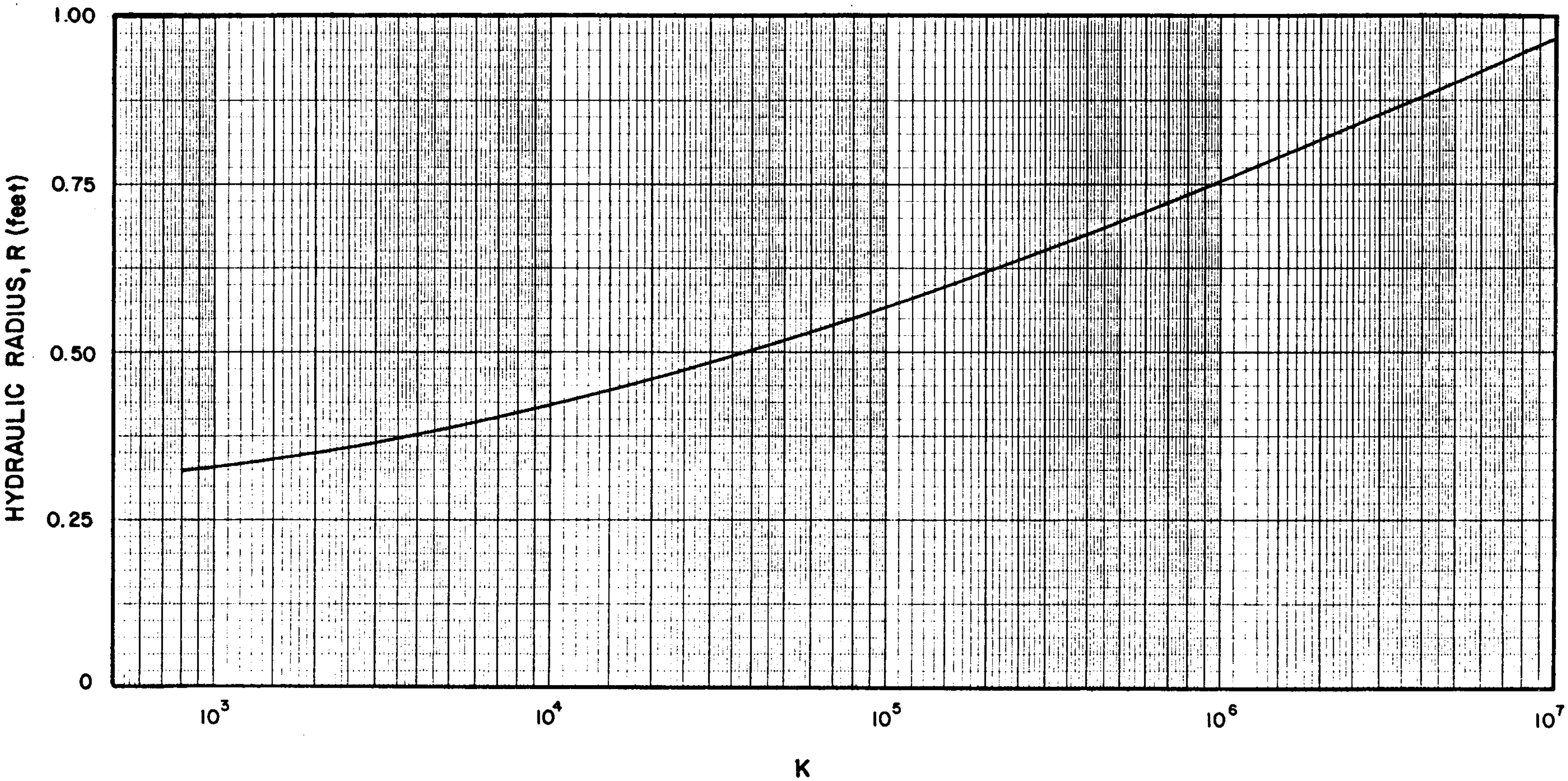


Figure 6.10b. Relationship between hydraulic radius R and K for trapezoidal channel with 2:1 side slopes and 10-foot base width.

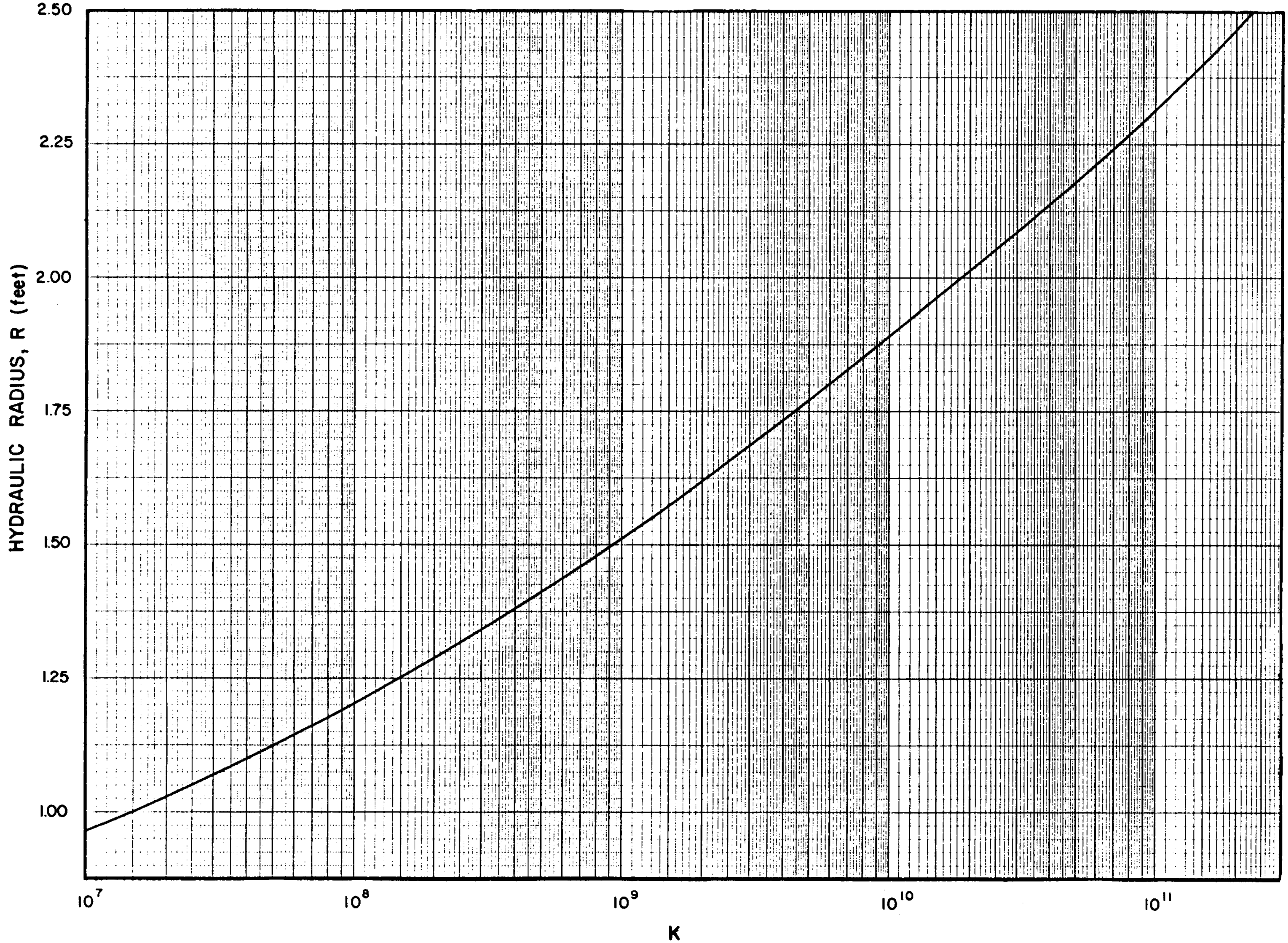


Figure 6.10b. Relationship between hydraulic radius R and K for trapezoidal channel with 2:1 side slopes and 10-foot base width (continued).

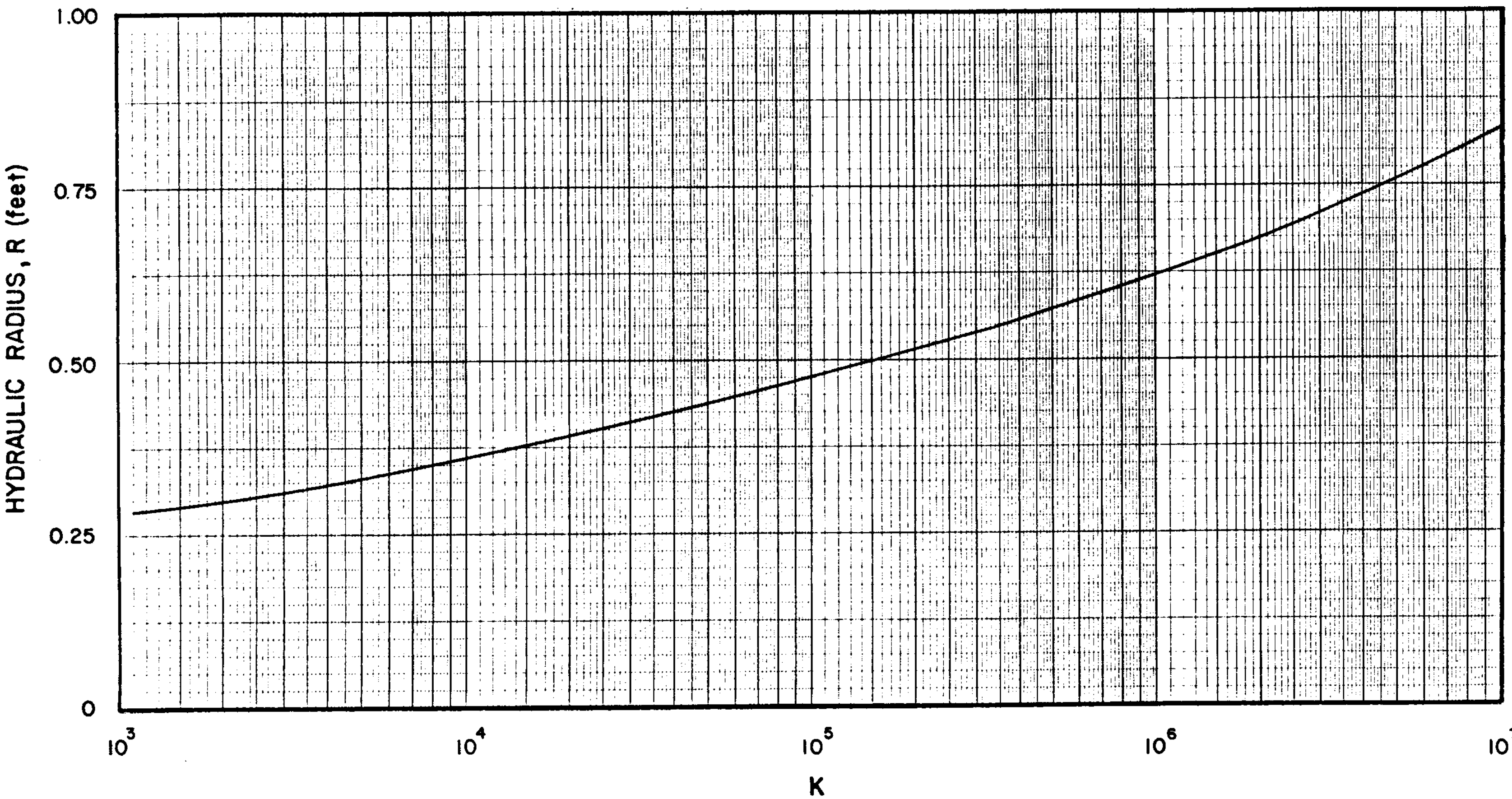


Figure 6.10c. Relationship between hydraulic radius R and K for trapezoidal channels with 2:1 side slopes and 14-foot base width.

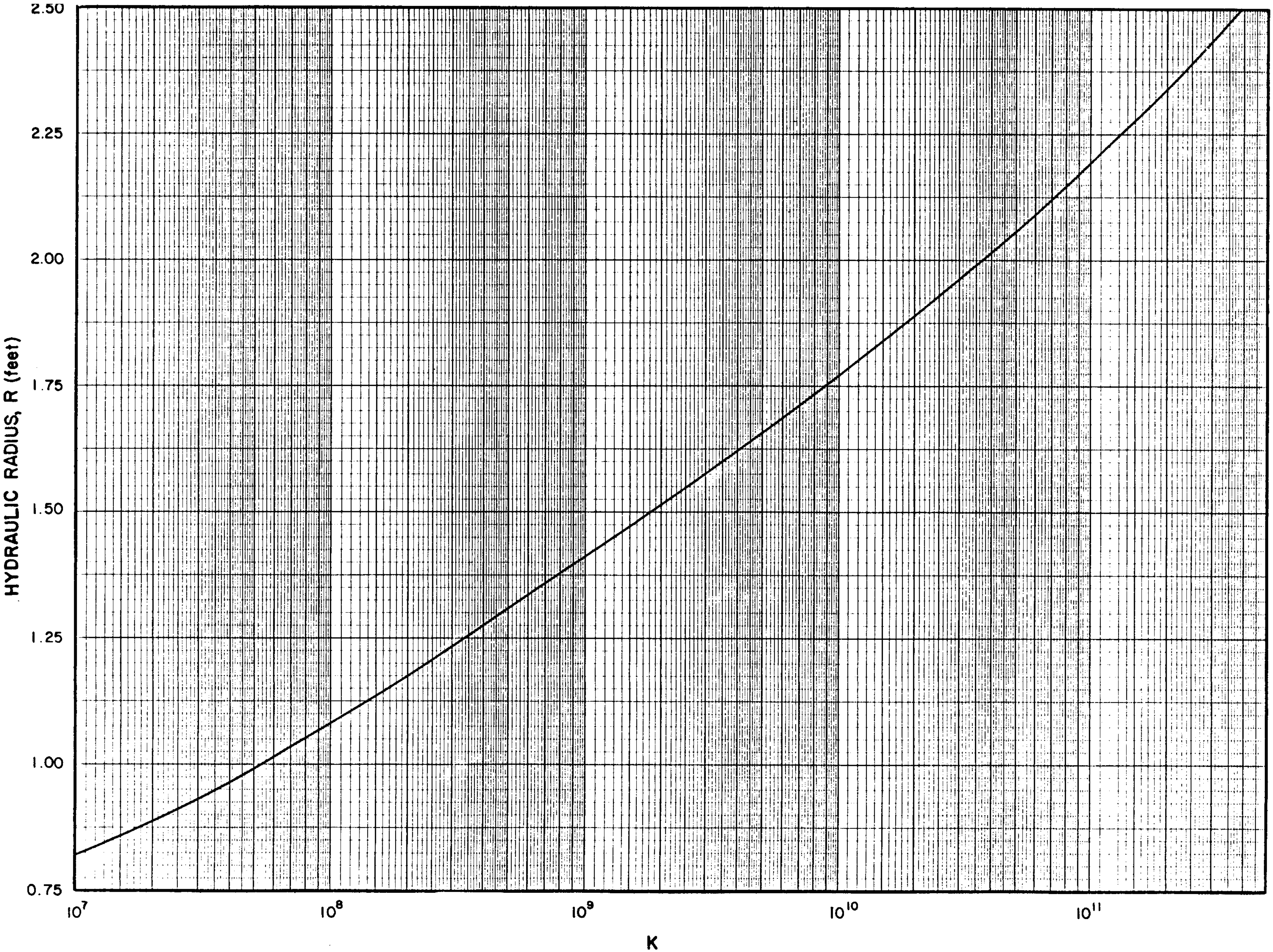


Figure 6.10c. Relationship between hydraulic radius R and K for trapezoidal channels with 2:1 side slopes and 14-foot base width (continued).

The spacing L of the drop structures is then

$$L = \frac{\Delta X}{N} \tag{6.15}$$

6.7.5 Protection of Grade Control Structures

The velocity of flow on the downstream side of a drop structure can be quite high, creating the potential for local scour at the toe and possible undercutting of the structure. Therefore, a riprap transition between the toe and the downstream channel must be provided with adequate energy dissipation measures.

The method for determining the length of protection required below a grade control structure is identical to the procedure for protection below steep slopes presented in Section 5.4. A riprap layer should be extended below the structure for a distance equal five times the downstream depth of flow, but never less than 15 feet. Additionally a small plunge pool can be provided at the downstream toe to help dissipate energy.

6.8 Design Procedure Summary

1. Design channel based on maximum permissible velocity method according to steps 1-6, Section 6.4.1.
2. Evaluate the channel for reasonable shape using Equations 6.7-6.10, and engineering judgment.
3. If a more hydraulically efficient channel is desired, evaluate the use of linings (vegetation or riprap) or grade control structures. Table 6.6 will aid in this evaluation.
   a. Vegetation
      1) Determine maximum permissible velocity for given vegetation type from Table 6.2.
      2) To design for stability, assume vegetation is mowed and identify retardance class from Table 6.3.
      3) Enter Figures 6.5a-e for given velocity, retardance and design slope to establish R.
      4) Calculate $A = Q/V$.
      5) Determine d for given b such that

Table 6.6. Application Conditions for Various Types of Channel Lining.

Lining Type	Velocity	Flow Duration	Slope
Vegetation	Less than 5 fps	Short-term	Mild
Riprap	Less than 12 fps	Year-round	Mild or Steep
Composite Vegetation & riprap	According to above	Short-term	According to above
Riprap & drop structures	Less than 12 fps	Year-round	Mild or Steep

$$R = \frac{A}{P} = \frac{bd + zd^2}{b+2d\ (z^2+1)^{0.5}}$$

Then check $A = bd + zd^2$

6) The design depth must now be increased to carry the flow when the grass is long - identify retardance class for uncut condition from Table 6.3.

7) Assume new depth and calculate R for the given bottom width.

8) Enter Figures 6.5a-e with computed R and design S to determine V.

9) Compute $Q = VA$ and compare to design Q. Iterate if calculated Q less than design Q.

10) Add proper freeboard (Equation 4.20).

b. Riprap

1) Assume a K_m size (6, 9, 12, 18 or 24 in.) and calculate Manning's n from Equation 4.18.

2) Evaluate V, d and R for the design Q, S, and channel geometry from charts in Appendix C. The channel design slope should be the uniform slope required to allow the channel to be constructed through slight changes in grade. If excavation amounts are too great to allow a uniform channel slope through changes in terrain slope, the channel can be designed to follow the changes in grade. For ease in construction, a single channel cross section adequate for each slope can be designed by using the maximum slope to size the riprap required, and the minimum slope to establish flow depth and freeboard requirements (transition requirements must be considered if this procedure is used).

3) Compute $V^2/R^{0.33}$ and determine the riprap type from Table 6.4 and K_m from Table 6.5.

4) Check the K_m determined from calculation with the assumed value.

5) Iterate until convergence occurs.

6) Check Froude number criteria ($F_r < 0.8$); if acceptable continue with design.

7) Determine gradation from Table 6.5.

8) Determine filter requirements.

9) Add proper freeboard. If the channel design is for a reach with slight changes in grade, the mildest slope should now be used to evaluate flow depth and freeboard requirements.

c. Drop Structure

1) Establish D_{50} of available riprap or bed material.

2) Compute K according to Equation 6.12 using Equation 4.18 for Manning's n.

3) Determine R from Figures 6.10a-c.

4) Solve Equation 6.11 for the static equilibrium slope.

5) If the slope of the natural terrain is less than the static equilibrium slope, no drop structures are required.

6) If drop structures are required, evaluate the number and spacing necessary from Equations 6.14 and 6.15, respectively.

6.9 Design Examples - Using Step-By-Step Procedures Outlined Above

6.9.1 Example of the Lane Relation Evaluation of Disturbances to Alluvial Channels

The impact of a new surface mine operation on a stream or river can be qualitatively predicted using the Lane Relation. Assuming that the watershed was relatively undisturbed for a long period of time, streams and rivers would have achieved a state of approximate equilibrium. This condition is commonly referred to as "graded" by geologists and "poised" by engineers, implying insignificant aggradation or degradation is occurring. With the large-scale land disturbance and clearing of the mine operation, the production of sediment is greater, and consequently the sediment discharge Q_s would increase to Q_s+. Assuming the particle size (D_{50}) and water discharge (Q) do not change, the channel gradient S must increase to maintain the proportionality of the Lane Relation.

$$Q_s^+ \, D_{50}^o \propto Q^o \, S^+$$

This will occur due to aggradation of sediment in the upper reaches of the channel(s) due to the overloaded sediment condition.

A second application of the Lane Relation is the qualitative analysis of the impact of a sediment pond on the downstream channel. Assuming the sediment pond is extremely effective, then the Q_s from the pond to the channel may be less than what originally existed in the channel in its graded or poised state. Under these conditions, and assuming Q and D_{50} do not change, the channel slope must decrease downstream of the pond to maintain the proportionality of the Lane Relation.

$$Q_s^- \, D_{50}^o \propto Q^o \, S^-$$

Therefore, the relatively clear water discharge from the sediment pond induces scour in the channel immediately downstream. Additionally, the channel banks may become unstable due to the degradation. With time the sediment pond may fill and sediment would once again be available to the downstream channel. Then, except for local scour, the channel gradient would again increase to transport the increased sedimei.: load.

6.9.2 Example of the Method of Maximum Permissible Velocity (Alluvial or Bedrock Channel)

Compute the bottom width and the flow depth of a trapezoidal channel laid on a slope of 0.02 and carrying a design discharge of 75 cfs. Assume the channel is to be excavated in earth containing noncolloidal coarse gravels and pebbles and no additional protection will be required, that is, the channel will be designed to be in static equilibrium without use of a lining. The design procedure would be identical if the channel were being cut in bedrock. Only the value of the permissible velocity would change.

Solution

1. For the given conditions, the following are estimated: $n = 0.025$, $z = 2$, and maximum permissible velocity = 4.0 fps.
2. Using the Manning formula, solve for R.

$$4.0 = \frac{1.49}{0.025} R^{2/3} \sqrt{0.02}$$

or $R = 0.33$ ft.

3. Then $A = 75/4.0 = 18.7$, $A = (b + zd)\,d = (b + 2d)\,d = 18.7 \text{ ft}^2$
4. $P = A/R = 18.7/0.33 = 56.7$ ft.

$$P = b + 2\sqrt{1 + z^2}\, d = 56.7 \text{ ft.}$$

5. Solving the two equations simultaneously,

$$b + 2\sqrt{5}\ d = 56.7 \qquad \text{or} \qquad b = 56.7 - 2\sqrt{5}\ d$$

$$(b + 2d)\ d = 18.7$$

Substituting for b in the second equation yields

$$67.7 - 2\sqrt{5}\ d^2 + 2d^2 = 18.7$$

or $-2.47\ d^2 + 56.7\ d - 18.7 = 0$

The latter equation is of the form

$$Ad^2 + Bd + C = 0$$

which can be solved by the quadratic equation:

$$d = \frac{-B \pm \sqrt{B^2 - 4AC}}{2A}$$

Using the appropriate values of A, B and C produces the result

$$d = \frac{-56.7 \pm \sqrt{(-56.7)^2 - 4(-2.47)(-18.7)}}{2(-2.47)} = 0.33\ \text{ft}$$

$$b = 12.6\ \text{ft}$$

Note that in this case the depth and hydraulic radius are equal (to the second decimal) as a result of the channel being hydraulically wide ($b/d > 10$).

6. Add freeboard. First evaluate if the flow is subcritical or supercritical:

$$Fr = \frac{V}{\sqrt{gL}} = \frac{4.0}{\sqrt{32/2(0.33)}} = 1.3;$$ supercritical (where the flow depth d is used for the characteristic length length L).

Equation 4.7

Therefore, from Table 4.4

$$c_{fb} = 0.25 \quad \text{and} \quad 0.25(d) = 0.08 < 1.0 \quad \text{use } 1.0\ \text{ft}$$

$$F.B. = 1.0 + \frac{1}{2}\Delta Z = 1.0 + 0 = 1.0$$ Equation 4.20

Therefore, a bottom width of $b = 12.6$ ft and a channel depth of $d = 1.33$ ft are required for a static equilibrium channel in the natural excavated earth of this example.

6.9.3 Example of Vegetated Channel Design

Assuming the channel described in the example of Section 6.9.2 does not flow for long durations, design a trapezoidal vegetated waterway for this location. Use a grass mixture as the vegetation and assume an easily eroded soil.

1. Determine design velocity from Table 6.2 as 3 fps.
2. Determine retardance class from Table 6.3 as D for the mowed condition.
3. Determine R as 0.37 for two percent slope, from Figure 6.5d.
4. Calculate $A = Q/V$.

$$A = \frac{75}{3} = 25.0 \text{ ft}^2$$

5. Determine b and d such that

$$A = bd + zd^2 = 25.0$$

$$R = \frac{bd + zd^2}{b+2d(z^2+1)^{0.5}} = 0.37$$

A good assumption for channels that must be designed with a low permissible velocity is that the final cross section will be hydraulically wide, therefore, the flow depth d will approximately equal the hydraulic radius R . The area relation can then be solved for the bottom width b and this value assumed for design. Therefore, use $b = 30$ assume $d = 0.8$ and iterate until $R = 0.8$.

d	R
0.8	0.74
1.0	0.90
0.9	0.8

Therefore, $A = 29$, $R = 0.8$ and actual capacity $Q = 87$ cfs.

6. From Table 6.3 the retardance class for unmowed is B.
7. Assume $d = 1.5$ ft, then $R = 1.3$.
8. From Figure 6.5b, with $R = 1.3$ and $S = 2$ percent, $V = 4.0$ fps. which is too high for the vegetation. Therefore try lower d

   $d = 1.2$ ft, then $R = 1.1$

   From Figure 6.5b $V = 3.0$ fps
9. $Q = VA = 3.0\ [30(1.2) + 3(1.2^2)] = 121$ cfs. Since 121 > 75 cfs, try a lower d. Try $d = 1.1$ then $R = 1.0$. From Figure 6.5b, $V = 2.3$ fps and

$Q = VA = 2.3\ [30(1.1) + 3(1.1)^2] = 84$ cfs--close enough to 75 cfs.

10. Freeboard

First, determine if the flow is subcritical or supercritical for both conditions (mowed, unmowed)

$$Fr = \frac{V}{\sqrt{gL}} = \frac{3.0}{\sqrt{32.2(0.9)}} = 0.55;\ \text{subcritical} \qquad \text{Equation 4.7 (mowed)}$$

$$Fr = \frac{V}{\sqrt{gL}} = \frac{2.3}{\sqrt{(32.2)(1.1)}} = 0.39;\ \text{subcritical} \qquad \text{Equation 4.7 (unmowed)}$$

Therefore, from Table 4.4

$c_{fb} = 0.20$ for unmowed and mowed conditions

$c_{fb}(d) = 0.20(1.6) = 0.32 < 1.0$; use 1.0 ft

$$\text{F.B. } 1.0 + \frac{1}{2}\Delta Z = 1.0 \text{ ft} \qquad \text{Equation 4.20}$$

Therefore, use F.B. = 1.0 ft.

The channel dimensions are then $b = 30$ ft, channel depth = 2.1 ft with a capacity for 84 cfs.

6.9.4 Example of Riprap Design

If a vegetated lining is not feasible for the channel of the previous example, rock riprap can be used. The channel dimensions for static equilibrium were (Example of Section 6.8.3) $b = 12.6$ ft and $d = 0.33$ ft. Therefore, for a lined channel assume $b = 8$ ft.

1. Assume K_m size of 9 inches, therefore

$$n = 0.0395\ (9/12)^{1/6} \qquad \text{Equation 4.18}$$

$$= 0.038$$

2. From charts in Appendix C for $Qn = 75(0.038) = 2.85$ on a two percent slope

$$Vn = 0.21;\ V = \frac{Vn}{n} = 5.5 \text{ fps}$$

$d = 1.3$ ft.

Therefore,

$$R = \frac{8(1.3) + 2(1.3^2)}{8 + 2(1.3)(2^2+1)^{0.5}} = 1.0 \text{ ft.}$$

3. $$\frac{V^2}{R^{0.33}} = \frac{5.5^2}{1.0^{0.33}} = 30$$

From Table 6.4 required riprap is Type L.

4. For Type L, $K_m = 6$ in. Therefore, must recalculate.

5. $n = 0.0395\ (6/12)^{1/6}$

$= 0.035$

$Qn = 75(0.035) = 2.62$

from Appendix C

$$Vn = 0.20;\ V = \frac{Vn}{n} = 5.7 \text{ fps}$$

$$d = 1.25$$

$$\text{Therefore } R = \frac{8(1.25) + 2(1.25^2)}{8+2(1.25)(2^2+1)^{0.5}} = 0.97$$

$$\frac{V^2}{R^{0.33}} = \frac{(5.7)^2}{(0.97)^{0.33}} = 33$$

and from Table 6.4 the required riprap is Type L. Therefore, the required riprapped channel to convey 75 cfs on a two percent slope has an eight-foot bottom width, a flow depth of 1.25 ft, and a median riprap size of six inches.

6. Check Froude Number

$$Fr = \frac{V}{\sqrt{gL}} = \frac{5.7}{\sqrt{32.2(1.25)}} = 0.90;\ \text{subcritical} \qquad \text{Equation 4.7}$$

Therefore, since the Froude Number is greater than 0.8 the steep slope riprap design procedure must be used.

From Section 5.5.1

1) Design discharge = 75 cfs

2) Channel slope = 0.02

3) Use 8 ft bottom width

4) Since the lowest slope shown on the design curves (Figures 5.3 to 5.7) is 0.05, this value will be used. This will provide a slightly conservative design. Since there is not a graph for 8 ft bottom widths, use the 6- and 10-ft graphs and linearly interpolate.

6 ft $D_{50} = 0.85$

10 ft $D_{50} = 0.58$

Therefore for 8 ft bottom width $D_{50} = 0.72$. From Table 5.2 the design $D_{50} = 0.75$.

5) Gradation

$$D_{max} = 1.25 \ D_{50} = 1.25(0.75) = 0.94 \text{ ft}$$

$$D_{10} = \frac{D_{50}}{3} = \frac{0.75}{3} = 0.25 \text{ ft}$$

Thickness

$$1.25 \ D_{50} = 0.94 \text{ ft}$$

6) Evaluate filter requirements as previously discussed.

6.9.5 Grade Control Structures

If the available riprap on a mine site consists of rock with a D_{50} = 6 in. and it is required to design a channel to transport 200 cfs on a slope of four percent for 500 ft, will grade control structures be required? Assume a trapezoidal channel with a bottom width of 10 ft and 2:1 side slopes.

1. As given, the D_{50} of the available riprap is six inches.

2. $$K = \left\{\frac{200\ [0.0395\ (6/12)^{1/6}]}{0.323\ [(2.65-1)\ 6/12]^{0.5}}\right\}^{6} \qquad \text{Equation 6.12}$$

$$= 1.9 \times 10^{8}$$

3. From Figure 6.11b $R = 1.28$ ft.

4. $$S = \frac{0.047\ (2.65-1)(6/12)}{1.28} \qquad \text{Equation 6.11}$$

$$= 0.030$$

5. Since $0.04 > 0.03$, grade control structures are required.

6. From Equation 6.13 the elevation to be controlled is

$$\Delta H = (0.04 - 0.03)\ 500$$

$$\Delta H = 5 \text{ ft}$$

and the required number of structures

$$N = \frac{\Delta H}{3} = \frac{5}{3} = 1.6$$

Therefore, use two structures spaced

$$L = \frac{\Delta X}{N} = \frac{500}{2} = 250 \text{ ft apart}$$

Therefore, the first structure is 250 ft downstream and the second is at 500 ft.

6.10 References

Carstens, M. R., 1966, "An Analytical and Experimental Study of Bed Ripples Under Wake Waves," Quart, Reports 8 and 9, Georgia Inst. of Technology, School of Engineering, Atlanta, GA.

Chen, V. T., 1959, Open Channel Hydraulics, McGraw-Hill, New York, N.Y.

Cox, M. B. and V. J. Palmer, 1948, "Results of Tests on Vegetated Waterways and Method of Field Application," Oklahoma Agricultural Experiment Station, Misc. Pub. No. MP-12, January, pp. 1-43.

Eastgate, W. I., 1966, "Vegetated Stabilization of Grassed Waterways and Dam Bywashes," M. Eng. Sc. Thesis, Department of Civil Engineering, University of Queensland, St. Lucia, Queensland, Australia.

Etcheverry, B. A., 1916, "Irrigation Practice and Engineering, Vol. II, The Conveyance of Water," 57 pp.

Federal Highway Administration, 1975. "Design of Stable Channels with Flexible Linings," Hydraulic Engineering Circular No. 15, October.

Fortier, S., and F. C. Scobey, 1926, "Permissible Canal Velocities," Trans., ASCE, Vol. 89, p. 940-956.

Highway Research Board, 1970, Tentative design procedure for riprap lined channels, Project Report No. 96, St. Anthony Falls Hydraulics Laboratory, Minneapolis, Minnesota, NCHRP Report 108, Washington, D.C.

Lane, E. W., 1953, "Design of Stable Channels," ASCE Proc., Separate No. 280, September.

Lane, E. W., 1957, "A study of the shape of channels formed by natural streams flowing in erodible material," Missouri River Division Sediments Series No. 9, U.S. Army Engineer Division, Missouri River, Corps of Engineers, Omaha, Nebraska.

Leopold, L. B., and Wolman, M. G., 1957, "River channel patterns: Braided, meandering, and straight," USGS Prof. Paper 282-B, 85 p.

Leopold, L. B., Wolman, M. G., and Miller, J. P., 1964, Fluvial Processes in Geomorphology, W. H. Freeman and Company, San Francisco.

Mavis, F. T., T. Liu, and E. Soneck, 1937, "The Transportation of Detritus by Flowing Water," Univ. of Iowa, Studies in Engineering, No. 341.

Mirtskhulava, T. W., (*), "Studies on Permissible Velocities for Soil and Facings."

Neill, C. R., 1967, "Mean Velocity Criterion for Scour of Coarse Uniform Bed Material," IAHR, 12th Congress, Fort Collins, CO.

Ree, W. O., 1951, "Preliminary Report of Tests on a Grass Lined Channel with a Center Concrete Gutter Section," U.S. Department of Agriculture, Soil Conservation Service, pp. 1-12, Unpublished Report.

Ree, W. O. and Palmer, V. J., 1949, Flow of water in channels protected by vegetative linings, U.S. Soil Conservation Bulletin No. 967, February, pp. 1-115.

Schwab, G. O., R. K. Frevert, T. W. Edminster, K. K. Barnes, 1966. Soil and Water Conservation Engineering, John Wiley and Sons, Inc., New York.

Shen, H. W. 1971, River Mechanics, Proceedings of the Institute on River Mechanics, Colorado State University, June.

Simons, D. B. and Senturk, F., 1977, "Sediment Transport Technology," Water Resources Publications, Fort Collins, Colorado.

Soil Conservation Service, 1977, "Design of Open Channels" Technical Release No. 25, October.

U.S. Army, Office, Chief of Engineers, 1970, Engineering and Design: Hydraulic Design of Flood Control Channels, EM 1110-2-1601, Washington, D.C., 1 July.

U.S. Bureau of Reclamation, 1960, Investigation of Meyer-Peter, Muller Bed Load Formulas" Sedimentation Section, Hydrology Branch.

USDA, Soil Conservation Service, 1954, "Handbook of Channel Design for Soil and Water Conservation, SCS-TP-61, Washington, D.C., pp. 1-34.

VII. TRANSITION DESIGN

7.1 Basic Considerations

Transitions may be defined as a change in either direction, slope or cross section of the channel that produces a change in the state of flow (Henderson, 1966). Transition design is a critical step in design of open-channel flow networks since the design capacity of the system can be significantly lowered if the transitions do not perform properly. Some of the possible problems that can develop with poorly designed transitions include backwater effects, local scour and wave formation.

There are several conditions where tansitions will be required on surface mine sites. Diversion channels will seldom be identical in shape with the natural waterway above and below. Economic considerations usually dictate designing a smaller, more hydraulically efficient diversion channel than the natural waterway. This is particularly true when riprap is being used to stabilize the channel. Transition sections are also required at changes in grade, such as the inlet and outlet to a spoil fill diversion. These transitions typically represent a change from a mild to a steep slope and from a steep to a mild slope, respectively. The recommended design for the inlet and outlet of a steep slope diversion on a spoil fill was given in Section 5.4. However, other changes in grade, such as the transition from a mild slope to a milder slope, require consideration. In this case a potential backwater condition exists that could cause overtopping of the upstream channel. Conversely, if a mild slope transitions to slightly steeper slope, the potential for local scour exists.

Transitions must be properly designed to avoid the potential adverse effects discussed above. Transitions are sometimes designed to conserve head, however, this consideration is not particularly relevant to diversion channel design on surface mine operations. Additionally, transitions from one geometric shape to another, such as trapezoidal to rectangular, are relevant in canal design, but not diversion channel design on surface mine sites. Riprap-lined channels on surface mine sites are typically geometrically similar trapezoidal shapes. Therefore, consideration of the more complicated curved or warped transition section is not included in this manual.

The final section of this chapter presents general guidelines related to channel junctions, such as a diversion channel discharging into a natural stream. This condition can be considered as a special type of transition.

7.2 General Design Principles

Transition design is based on the Bernoulli and continuity equations (Equations 4.9 and 4.10); however, experience plays an important part. The Soil Conservation Service (1977) has provided some general rules to follow in designing transitions. They are:

1. The water surface should be smoothly transitioned to meet end conditions.

2. The water surface edges should not at any section converge at an angle greater than 28° with the center line, nor diverge at an angle greater than 25°.

3. In well designed transitions, losses in addition to friction should not exceed 0.10 h_v for convergence and 0.20 h_v for divergence, where h_v is the velocity head.

4. In general it is desirable to have bottom grades and side slopes meet end conditions tangentially.

Transition design is based on a modified form of Bernoulli's equation, (Section 4.3) derived by grouping the various head terms from Sections 1 and 2, or

$$\left(\frac{P_1}{\gamma} + Z_1\right) - \left(\frac{P_2}{\gamma} + Z_2\right) = \frac{V_2^2}{2g} - \frac{V_1^2}{2g} + h_L \tag{7.1}$$

For sufficiently flat slopes $(P/\gamma) + Z$ equals the water-surface elevation, and taking the fall of the water surface in the downstream direction as positive, Equation 7.1 equals

$$\Delta W.S. = \Delta h_v + h_L \tag{7.2}$$

where $\Delta W.S.$ is the change in the water-surface elevation, Δh_v is the change in the velocity heads and h_L is the head loss term. In a transition section, additional head loss is usually involved over the friction head loss. The additional losses, referred to as conversion losses, can be defined simply as those due to a change in direction of the stream lines resulting in both converging and diverging transitions. In relatively short transitions, the conversion losses are usually significantly greater than the friction losses and so the friction losses can be neglected. The head loss term h_L is then defined as (Chow, 1959)

$$\text{inlets:}\quad h_L = C_i\, \Delta h_v \tag{7.3a}$$

$$\text{outlets:} \quad h_c = C_o \, \Delta h_v \tag{7.3b}$$

where C_i and C_o are the inlet and outlet loss coefficients, respectively. For inlet (converging) structures, the entrance velocity is less than the exit velocity and it is necessary to design the transition with a drop in the water-surface sufficient to provide the required increase in velocity head and to overcome head losses. For outlet (diverging) structures the velocity is reduced and the water surface theoretically rises an amount equal to the reduction in velocity head. The actual rise, referred to as the recovery head, is less than theoretical due to head losses. These relationships can be stated mathematically by incorporating Equation 7.3 into Equation 7.2,

$$\text{inlets:} \quad \Delta W.S. = \Delta h_v + C_i \, \Delta h_v \tag{7.4a}$$

$$\text{outlets:} \quad \Delta W.S. = \Delta h_v - C_o \, \Delta h_v \tag{7.4b}$$

These simple relationships, plus the continuity equation, form the basis for all transition design. The objective in designing a transition is then to achieve the water-surface change specified by Equations 7.4.

7.3 Simplified Design Procedure

Given two channel cross sections, it is required to design the transition. A simplified transition design procedure where conservation of head is not critical is given by the Soil Conservation Service (1977). The elevation of the water-surface and the flow velocity at the end points are known from the design discharge and channel geometries. However, in the design procedure no attempt is made to trace out the water surface at intermediate points in the transition. The design objective is only to insure that the proper overall change in water surface elevation exists.

In the absence of more specific knowledge the length of the transition should be such that a straight line joining the flow line at the two ends of the transition will make an angle of about 12 1/2 degrees with the axis of the structure. The recommended values of the coefficients C_i and C_i are taken as 0.15 and 0.25, respectively, therefore Equation 7.4 becomes

$$\text{inlets:} \quad \Delta W.S. = 1.15 \, \Delta h_v \tag{7.5a}$$

$$\text{outlet:} \quad \Delta W.S. = 0.75 \, \Delta h_v \tag{7.5b}$$

With the known velocities the change in water surface can then be computed.

For an inlet, the drop in bed elevation through the transition necessary to achieve the required ΔW.S. is

$$\Delta B.E. = d_2 - d_1 + \Delta W.S. \tag{7.6a}$$

where d_1 and d_2 are the flow depths at the entrance and outlet of the transition, respectively. For an outlet the required rise in elevation through the transition is

$$\Delta B.E. = d_1 - d_2 + \Delta W.S. \tag{7.6b}$$

Therefore, following the general rules given in Section 7.2 and the computed length and elevation change, the transition design is complete.

7.4 Transition Protection

The transition design procedure presented in the previous section is a simplified method. Therefore, to ensure that transition stability is maintained, it is recommended that riprap protection be provided in transition reaches. Using the values for flow velocity (V) and depth of flow (d), values of the Froude number can be determined (Section 4.2.5).

If the transition is accomplished on a mild slope where the Froude number does not exceed 0.8, transition protection can be determined from the mild slope riprap design procedure in Section 6.7. To account for the turbulence in the transition section, the value for velocity V used in the parameter $V^2/R^{0.33}$ should be increased by the following amounts:

V = 1.05 times channel velocity for converging channels (accelerating flow)

V = 1.10 times channel velocity for diverging channels (decelerating flow)

Protection should also be provided both up and downstream of the transition reach. Recommended distances are at least three feet upstream of the entrance and at least five feet downstream of transition exit.

For steep sloping transitions where Froude numbers exceed 0.8, transition protection should be evaluated from the steep slope riprap design procedure given in Section 5.3. Transition slopes less than five percent (the minimum slope value in the steep slope design curves) can be designed using the ten percent curves consequently providing conservative rock protection. Protection should be provided at the transition entrance and exit according to the criteria in Section 5.4.

7.5 Special Considerations

The simplified method discussed in Section 7.3 is probably adequate for most transition designs required for diversion channels in a surface mine situation. The method gives a satisfactory design for relatively low velocity, small transition sections. Under higher velocity flows involving supercritical flow and hydraulic jump situations, more detailed design procedures are required. Shock wave formation and other complicating factors must be considered. References for these design procedures include Hinds (1928) and Henderson (1966).

7.6 Channel Junctions

Where a confluence between a major diversion channel and a natural stream channel occurs, the junction should be oriented to provide a good transition of the diversion channel flow into the natural stream flow. If the diversion channel is brought in perpendicular to the stream channel, significant turbulence and waves may be generated at the junction point. This in turn can result in scour and problems of channel instability. Orientation of the diversion channel exit more in the direction of the natural stream flow helps reduce velocity and momentum components (which cause waves normal to the direction of the combined flow) (Soil Conservation Service, 1977).

The natural angle of juncture between tributary streams and main streams has been observed to be in a range of 45 to 55 degrees. At a junction between a diversion channel and a natural stream channel, the diversion channel is essentially a tributary. Therefore, orientation of major diversion channel junctions at angles no larger than 45 to 55 degrees should provide for reasonable transition and assimilation of diverted flows into stream channels. Figure 7.1 shows the recommended orientation.

A major diversion channel, such as one carrying the flow of a diverted tributary stream, should be constructed so that the diversion enters at the invert level of the natural channel. Smaller diversions may be brought in at some point on the channel bank above the stream bed level. When this is the case, adequate protection of the channel banks must be provided by placement

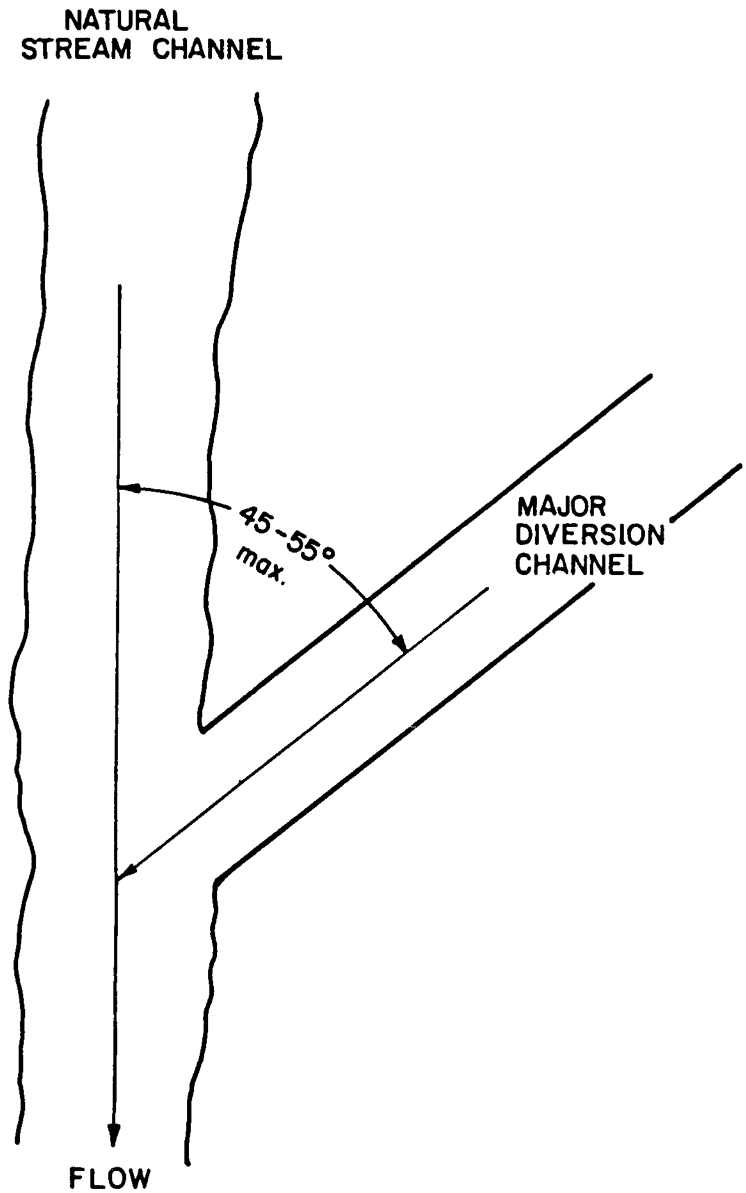


Figure 7.1. Recommended junction angle between a major diversion and a natural stream channel.

of localized riprap. Overland flows entering a diversion channel should be concentrated and brought in at selected locations. Where it is not practical to concentrate overland flows, the channel bank should be protected or vegetated.

7.7 Example of Transition Design

The following example illustrates the transition design procedure. It is required to design a transition between two trapezoidal channels with different cross sections. The characteristics of each channel are:

Upstream Channel Section	Downstream Channel Section
natural smooth earth channel	Riprap lined
$n = 0.025$	$D_{50} = 0.5$ ft
$S = 0.003$	$S = 0.01$
Base width $b = 10$ ft	Base width $b = 6$ ft

flow rate $Q = 150$ cfs

Solution

1) Evaluate upstream channel section properties from data for upstream channel: compute Qn

$$Qn = 150\ (0.025) = 3.75$$

From the charts in Appendix C

$$Vn = 0.112 \qquad V = \frac{Vn}{n} = 4.5 \text{ fps}$$

$$d = 2.3 \text{ ft}$$

2) Evaluate flow properties in downstream channel section

Estimate $n = 0.0395\ D_{50}^{1/6}$ (Equation 4.18)

$$n = 0.0395\ (0.5)^{1/6} = 0.035$$

$$Qn = 150\ (0.035) = 5.25$$

From the charts in Appendix C

$$Vn = 0.20;\ V = \frac{Vn}{n} = 5.7 \text{ fps}$$

$d = 2.5$ ft

3) Compute change in water surface profile (Equation 7.5a)

$$\Delta B.E. = 1.15 \left(\frac{5.7^2}{2g} - \frac{4.5^2}{2g}\right) = 0.22 \text{ ft}$$

4) Compute necessary change in elevation (ΔZ) between transition entrance and exit (Equation 7.6a)

$$\Delta B.E. = d_1 - d_2 + \Delta W.S.$$

$$\Delta B.E. = 2.5 - 2.3 + 0.22 = 0.42 \text{ ft}$$

5) Compute length of transition using maximum angle of convergence equal to 25° between the water surface.

$$\tan (12.5) = \frac{(9.6-8.0)}{L}$$

$$L = 7.2 \text{ ft}$$

Figure 7.2 illustrates the design.

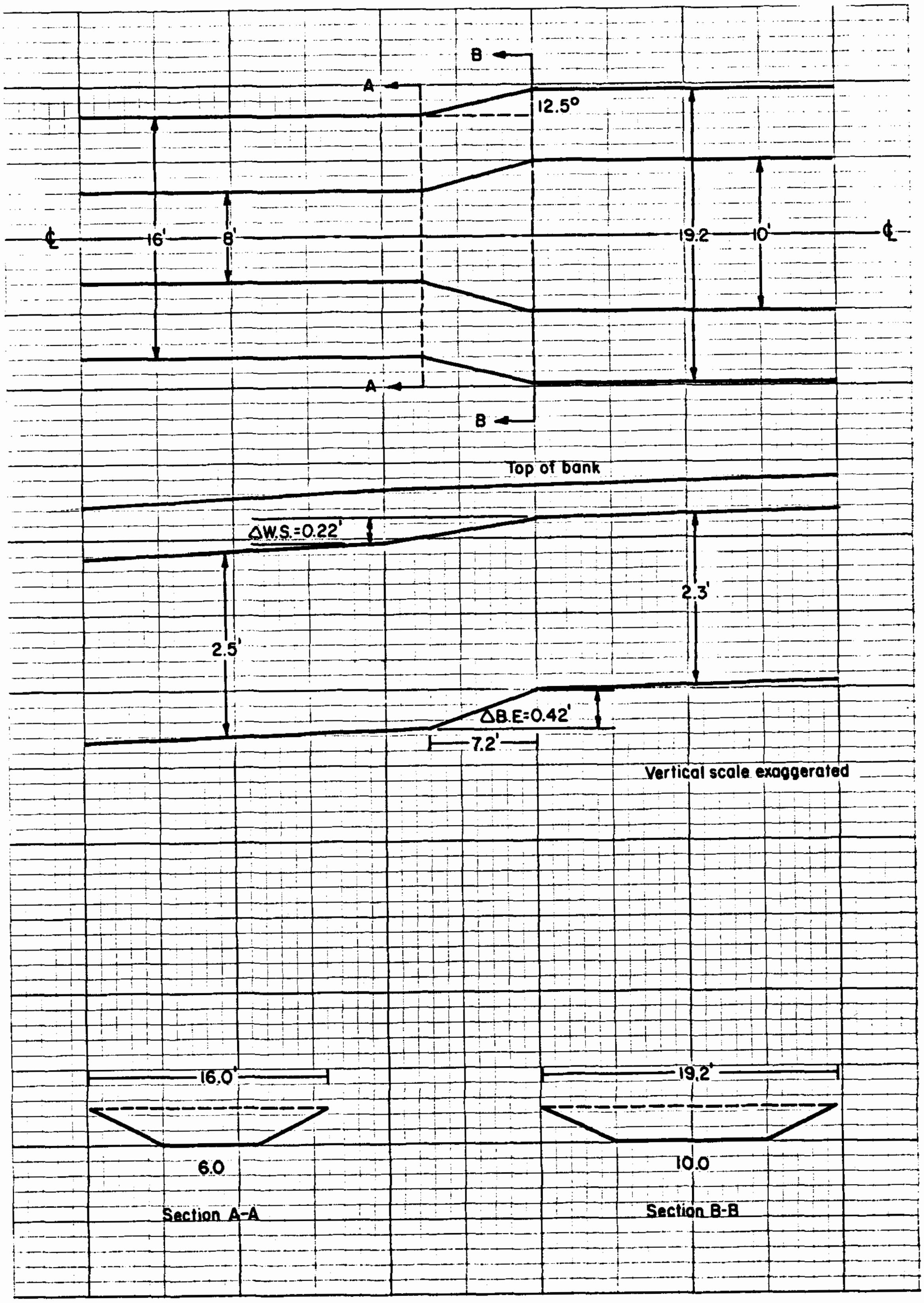


Figure 7.2. Converging transition design example.

7.8 References

Chow, V. T., 1959. Open Channel Hydraulics, McGraw-Hill, New York, NY.

Denver Urban Drainage and Flood Control District, 1980.

Henderson, F. M., 1966, Open Channel Flow, MacMillan Publishing, New York.

Hinds, J., 1928, "The Hydraulic Design of Flume and Siphon Transitions," American Society of Civil Engineers, Transactions, Vol. 92, Paper 1690.

Soil Conservation Service, 1977. "Design of Open Channels," Technical Release No. 25, October.

VIII. ROCK DURABILITY AND SLOPE STABILITY EVALUATIONS

8.1 Introduction

Basic geotechnical techniques and geological concepts are essential to the proper design and maintenance of diversions and channels on a mining site. In this section of the design manual, guidelines for evaluating engineering and geological characteristics of rocks which are important in assessing rock durability as related to riprap and bedrock channels are given. Following the detailed assessment and guidelines, problems of geotechnical stability and site-specific conditions associated with diversions are addressed.

8.2 Rock Durability Evaluation

8.2.1 Purpose and Scope

The purpose of this section is to define and demonstrate the utility of a systematic method of quantifying durability of earth materials to predict their physical behavior if used as riprap diversion channels or for other mining-related activities. It is designed predominantly for sedimentary rock types which overlie coal seams in the eastern coal region. The procedure was developed with the primary goal of producing a reliable and adaptable method for evaluating rock durability which can be conducted economically and time-efficiently, mainly in the field. It is essentially a three-fold procedure based primarily on field observations, field tests, and selected laboratory tests. Options to perform the laboratory tests are included and their use depends on environmental demands, economic considerations at a particular site, and availability of suitable riprap at each site.

In considering rock durability relative to the service life of riprap, it is important to recognize that "nothing lasts forever". Therefore, the designer must take a very conservative approach, although most riprap linings still may require periodic maintenance, this manual has chosen a conservative approach. The recommended procedure is based on conservative values designed for a probable lengthy in-service performance of riprap. This translates into a higher factor of safety than is sometimes recommended for minimum performance. Albeit, it is considered expedient to be conservative considering the long term exposure and life of a diversion channel.

With this in mind, the durability evaluation method is described and discussed below. The step-by-step procedure, beginning with general field considerations and progressing to specific laboratory tests, is also presented in a simple flow chart in Figure 8.1. Reference to this chart may be helpful in understanding the sequence of the procedure as detailed points are described in the text.

Following this, an evaluation format for on-site evaluation is provided. Then, several rock types from Kentucky are actually evaluated using the prescribed format to illustrate and clarify proper use of the method.

8.2.2 General Considerations of Test Procedure

The durability or weatherability of rock is a critical factor whether stream diversion is being constructed with a channel in unlined bedrock or a channel with riprap as a lining. Rock properties to be considered will include composition of the rock fragment or rock outcrop and the presence of bedding, joints, etc. at the rock mass (rock outcrop) level. The term "discontinuity" will be used as a non-genetic term for bedding, joints, etc. except where benefit is derived from a more specific terminology. Aside from visual evaluations that may be made at the site, testing will be primarily that used for determining the abrasion resistance and degradation properties of aggregates. Climatic conditions at the site have a bearing on the choice of tests. Estimates of annual or seasonal rainfall and number of freeze-thaw cycles will normally be sufficient indicators of climatic conditions. In the area of interest, the eastern coal region, the climate is a temperate, humid continental-type where temperatures rarely exceed 100°F or drop below 0°F (Huddle, et al. 1963). Forty to 50 inches of precipitation are received annually, most of which is rainfall and the frost-free period is approximately 175 days between April 25th and October 15th.

In addition to these considerations, estimates must be made of the volumes of water to be carried by the diversions and the nature and amount of sediment transported.

8.2.3 Site Investigations

8.2.3.1 <u>General</u>

Much can be learned of the durability of rock from an investigation of the site. A first step in evaluating a site is to obtain a listing of the rock types occurring in the area. This listing may be obtained from geologic maps, geologic reports for the area and drill hole data from exploration drilling. Brief descriptions of the rocks normally accompany such listings.

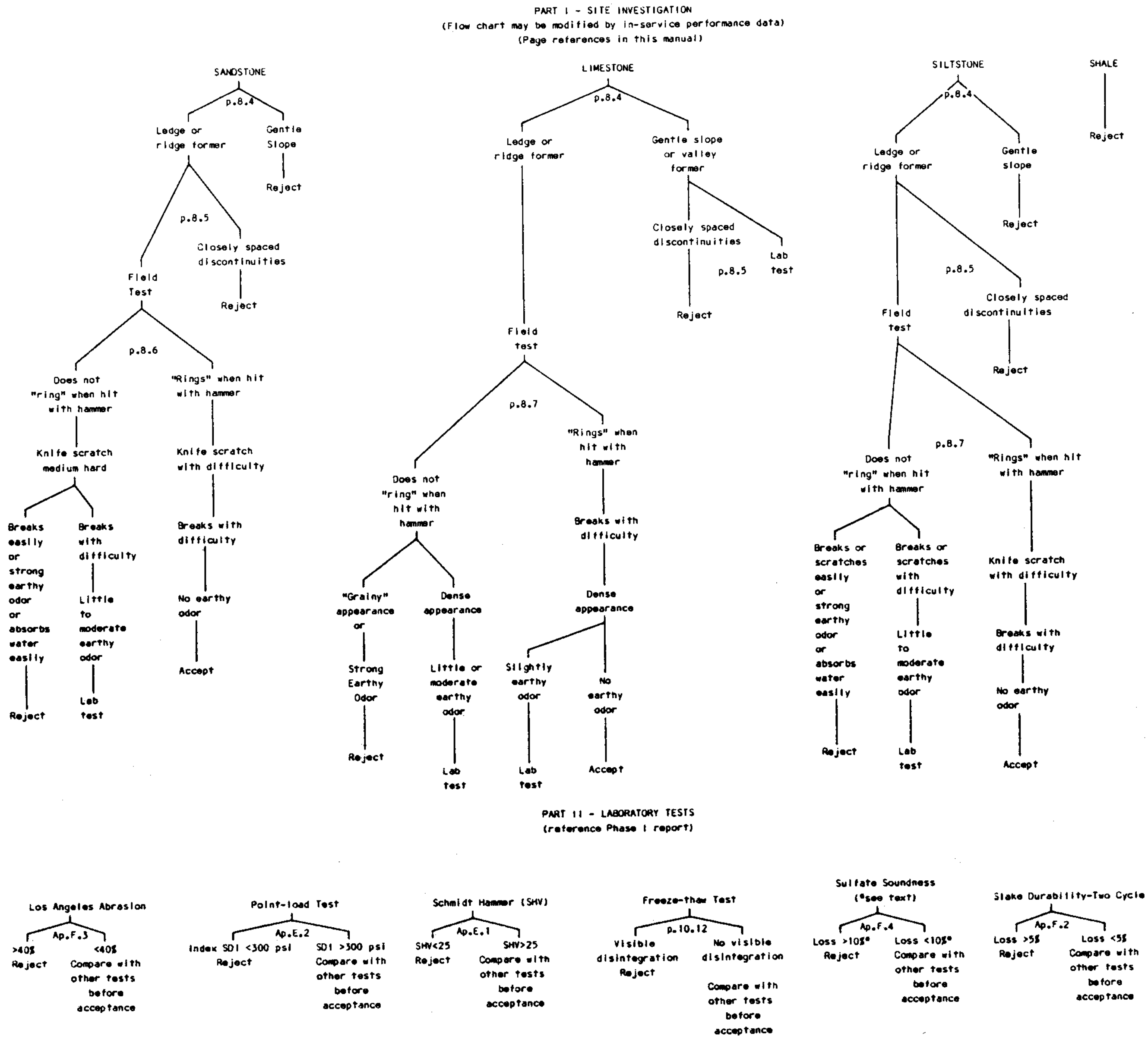


Figure 8.1. Rock durability flow chart: Procedure for evaluating rock suitable as riprap and channel lining. Part I - Site investigation; Part II - Laboratory investigation.

If such data are not available, the rock types and any sequential occurrence of them must be determined at the site from exposures. The site investigation provides the opportunity to observe the rocks in their natural setting, i.e., climate and landforms. If air photos are available for the site area, their use should be an integral part of the investigation. The site features to be considered are: landform characteristics, rock type or types, rock mass conditions, performance record and simple field tests.

8.2.3.2 Landform Characteristics

Most rocks resistant to the mechanical and chemical weathering conditions at a site will in turn be durable for the conditions prevailing in diversion channels. The correlation of ridges and benches or rock-cored terraces with known rock units in the site area is a major first step in finding durable rock. This is done by the combined use of topographic maps, air photos and observation of known rock types occurring in the area. Thus, the evaluation of the topographic control exerted by the rocks at a site is of prime importance. A geologic map is a useful tool for further designating naturally durable rock units from those unsuitable for use in constructing diversion channels.

8.2.3.3 Rock Type

In coal-producing areas the rock types in which diversion channels will be excavated or the rocks which will be used for riprap will be of sedimentary origin (see Section 8.2.6). Of these only the well-cemented sandstones and relatively clay-free limestones will have long-term durability. Shales, claystones and mudstones are not suitable for either channel construction or as riprap because of poor abrasion resistance and the tendency to slake and weather rapidly. Siltstones may fall in this category also if they are clay-rich and/or poorly cemented. As stated in the Earth Manual published by the U.S. Bureau of Reclamation (1974), any sedimentary rock with clay must be suspected of poor performance as riprap.

8.2.3.4 Rock Mass Conditions

The most durable rock as determined by testing is only as good as the rock mass from which it was obtained. Sedimentary rocks by nature have discontinuities known as bedding surfaces. In addition, all rocks regardless of origin have joints or relatively planar discontinuities caused by volume reduction or response to tectonic stresses. Joints occur in sets or many planar discontinuities parallel to one another. When these sets intersect a bedding plane, they typically form blocks which can be of similar size or an array of sizes and shapes.

Closely spaced bedding and joints are a problem for both excavated channels and riprap sources. Their orientations and spacings may be measured. In excavated channels the smaller the block the easier the hydraulic quarreying action will be. Riprap sources obviously must have block dimensions greater than the design dimensions to be suitable. Any tendency to have incipient sedimentary pactings will influence breakage of blocks into smaller blocks when blasted, transported and placed as channel lining. Riprap material should not exhibit discontinuities with spacings less than the predetermined dimension of the riprap required for a given channel size and flow.

Other significant characteristics of the rocks are a direct result of the sedimentary environment in which coal and the adjoining rock units were formed. In addition to being thinly bedded, the rocks may consist of interbedded shale and sandstone. Interbedded weak and strong layers of rock broken by joints will only be as durable as the weakest rock exposed. Also, channel deposits are prevalent in this sedimentary environment. Because of their irregular, linear geometry, durability of the rock mass may exhibit significant vertical and horizontal variability. Careful examination of the local, seemingly durable units must be made for these spatial controls where a channel is excavated or where riprap is being quarried.

8.2.3.5 Performance

A factor that should not be overlooked is the in-service performance of any local rock unit which has been exposed to the elements. There also may be local excavated sites where the performance over several seasons of both the rock in the excavated face and stockpiled material can be evaluated. Since these are the conditions prevailing at the site, such performance may be a better indicator of durability or weatherability than many of the laboratory tests. Evaluation of in-service performance of several years or more is recommended. The wearability or resistance to abrasion common to exposed bedrock in a channel can be assertained qualitatively from examination of exposures of the rock in local stream channels. If the rock forms rapids or falls or is sufficiently resistant to have localized abrasion-formed potholes rather than complete removal by erosion, the rock should perform well for the service life of a diversion channel.

8.2.4 Field Testing

Either the inherent strength of fresh rock or the weakened state from weathering processes may be judged from several field tests. A hammer, pocket knife and a 10-power hand lens are required for most simple tests. Since these are all empirical tests, it is not feasible to standardize on one size or kind of hammer, although this has been attempted.

Broken pieces of sandstone (hand specimens) may be examined with a hand lens. If the sand-sized particles are composed of glassy-appearing quartz, the chemical stability and the resistance to abrasion of the primary mineral grains will be excellent. If the grains appear to be composed of other material two possibilities exist. The grains may be fairly soft fragments of the minerals that compose limestone (calcite/dolomite); they can be scratched easily with a knife. Otherwise, they may be a mixture of minerals derived from preexisting rocks. These minerals may have decomposed, at least in part, to clays. Then, they will scratch easily and the rock will have an earthy smell when breathed upon. Only the last case can be categorically removed from further consideration.

The sand-sized quartz and limey minerals composing a "sandstone" may provide satisfactory material if the cement holding the grains together is of sufficient quantity and a durable kind. The sand-sized grains may be cemented or at least adhered together by such common cements as clay, limey cement (calcite), iron oxide (red), or varieties of quartz. To determine the degree of cementation, place a drop of water on the fresh surface. If it is absorbed, the cementing of the grains is not complete and the rock would not be very durable.

If the water is not absorbed, check for clay by breathing on the specimen and by scratching the surface with a knife. When viewed under the hand lens, clay cement will appear to allow hardy or individual mineral grains to be released from the rock matrix when the surface is scratched by a knife. Such rocks will not be durable because of the clay. Limey, iron oxide and quartz cements may permit some clay-bearing rocks to perform well and should be tested further in the laboratory. Scratch tests range from "easy" for limey cement to "impossible" for quartz. Another qualitative measure of durability will be obtained by examining the freshly broken surface with the hand lens. Tougher, more durable rocks will break across the grains and cement, while less durable but perhaps still useful rocks will tend to break around the grains through the cementing material.

The hammer is useful in several ways. The hardest and probably most durable rock will have a distinct "ring" when struck with a hammer. With a change in either kind of cement, amount of cement or weathering of the rock minerals and/or cement, the hammer blow will be either a dull ring or a "thud." Since these are qualitative measures, the terms should be defined by the field inspector by experimenting on rocks of known durability.

The hammer is also useful as a means of breaking a rock sample held in the hand. If the rock can be broken by hammer blows only with considerable difficulty, the durability of the intact sample should be good. The durability will decrease with increasing ease of breakage. Any rock that breaks easily when hand held should be considered non-durable and of no value in diversion channel construction. Additionally, because sedimentary rocks may be deposited in a layered fashion, it is wise to carry out the breakage test with several sample orientations in the hand. Subsequently, the weakest orientation will control one's choice of rock, especially if the use is riprap material.

Williamson (1978) of the USDA Forest Service has devised a simple impact test using a 1 lb. ball peen hammer. If the hammer rebounds completely with no damage to the rock, the rock quality should be high with an estimated compressive strength >15,000 psi. If fragments are formed at the point of impact, the quality will still be good (durable) with a strength estimated between 8,000 and 15,000 psi. Any impact that results in denting of the surface or complete fragmentation will reveal rocks not suitable for riprap channel construction.

8.2.5 Laboratory Testing

8.2.5.1 General

Rocks which will be eliminated as suitable riprap by visual inspection or simple field tests are those rocks which are cemented with clay or weathering to clay, poorly-cemented sandstone, or clay-rich rocks such as shale, claystone and mudstone. The remaining rock types will range from marginal to excellent in performance. Laboratory testing of these rocks will provide a basis for judging in-service durability.

A literature review (Phase I report) indicates that measures of rock durability defining durable rock for the uses described in this manual are not well defined. However, tests for resistance to abrasion, freeze-thaw cycles and strength used for other engineering applications are adaptable to durability assessment (Appendices E and F, Phase I report). Abrasion resistance is a major concern when the channel is constructed in rock without a lining, although response to water and freeze-thaw cycles is of additional concern. The durability of riprap will be defined more by resistance to water and freezing and thawing than by abrasion resistance. The tests described in the following sections address these aspects of durability. A sufficient number of tests are described so that the need for a large array of specific test equipment is reduced. While all are typically to be run in the laboratory, those that may also be conducted in the field will be identified.

8.2.5.2 Los Angeles Abrasion Test

The Los Angeles Abrasion Test or LA test is used to estimate aggregate abrasion susceptibility. As stated in the procedure outlined in Appendix F.3, a percentage loss or LA number less than 40 percent is considered as resistant or durable. This test follows the ASTM standards (1980).

8.2.5.3 Point Load Test

Durability, whether to abrasion or with regard to natural weathering under climatic conditions, is directly proportional to uniaxial compressive strength of an intact or solid rock sample. The Point Load Test which is portable and may be used in the field on hand specimens, determines the tensile strength which is converted to uniaxial compressive strength. As defined in Appendix E.2 of Phase I report, the acceptable threshold value is a value, >300, which is approximately 7,000 psi. This, in turn, is roughly equivalent to an LA number of 30 percent, indicating that the point load index value is conservative relative to the upper index value of the LA test (40).

8.2.5.4 Schmidt Hammer

The Schmidt Hammer is an alternative device for measuring unconfined compressive strength in the field. A detailed description of the method for defining strength values as indicators of durability is given in Appendix E.1 of the Phase I report. A Schmidt Hammer Value (SHV) of <25 is approximately equivalent to 7,000 psi or to an LA number of 30 percent.

8.2.5.5 Freeze-Thaw Testing

If the rock to be used in diversion channel construction or as riprap to line the channel is subject to freeze-thaw cycles, some estimate must be made of the rock resistance to mechanical breakdown from the cycles. This is a most significant factor in even short-term disintegration of exposed rock. Susceptibility of rock is the result of clay content and porosity. If the rock in question is naturally exposed in an area visual examination of the products of many cycles of freezing and thawing can serve to eliminate the rock and the test. However, where rock is to be quarried testing is recommended (Phase I report, p. 10.12).

The equipment required to conduct freeze-thaw tests is not readily available in mobile labs or in many fixed-based testing labs. Testing may be done on a contract basis making certain that ASTM Standard C666 for either freezing and thawing in water or freezing in air and thawing in water is followed. Any change in the gradation of the test can be taken as deleterious for rocks exposed, as in diversion channel service. Local weather data may be used to obtain the average freezing cycles for comparison with test results having been produced by a specified number of cycles. Judgment must also be used concerning in-service contact with moisture, i.e., rainfall, snowmelt, exposure to sun, channel flow conditions, etc. before a rock can be accepted or eliminated on the basis of the test results.

8.2.5.6 Sulfate Soundness Test

The Sulfate Soundness Test is a satisfactory substitute for the freeze-thaw test and is less time consuming and requires less costly equipment. The test procedures are defined in Appendix F.4 (Phase I report). Conservative threshold values for assessing durability are <5 percent loss for sodium sulfate tests and <10 percent for magnesium sulfate tests.

8.2.5.7 Slake Durability

In the event clay-rich siltstones, sandstones and limestones are the only available materials and they marginally meet the field criteria described earlier and the preceding index tests, slake durability testing is in order. The durability or weatherability of clay-rich rocks presents problems because the degree of induration may mislead the observer to make a performance estimate better than what will actually occur.

The Two-Cycle Durability Test is recommended because it is already required and/or commonly used by mine engineers to evaluate durability of rocks for use as rockfill embankments. Detailed explanation and procedures are given in Appendix F.2 of Phase I report. The recommended threshold value for durable riprap is an index of >95 percent.

An alternative test for assessing slake durability is the Jar-Slake Test (Appendix F.1), although this test should not be substituted unless absolutely necessary because it is highly qualitative in nature. The investigator must be aware that this test is not a definitive indicator nor is it proven to be as reliable as the Two-Cycle Durability Test.

8.2.6 Application of Procedure

The rock durability evaluation procedure is designed mainly for field use. To illustrate the step-by-step method of predicting the durability of overburden, a field inspection at a coal mine site is reviewed and potential riprap material is analyzed. Hopefully, these examples will demonstrate the practical and efficient function of the durability evaluation.

8.2.6.1 General Inspection

Roughly 25 mine operations in eastern Kentucky were observed, some of which were active and others were inactive and reclaimed, partially reclaimed, or not reclaimed. Field reconnaissance of overburden characteristics suggest that:

(a) overburden "dump rock" is commonly used as riprap,

(b) much less overburden is used compared to the available spoil following coal excavation,

(c) excavation by blasting usually produces a range of rock sizes for use as riprap.

8.2.6.2 Durability Flow Chart Evaluation

To get more detailed information on the overburden, selected rock types were described and analyzed using the durability evaluation procedure. The data sheets used to compile necessary information to assess durability are given in Figure 8.2. The flow chart is subdivided into sandstone, siltstone-shale, and limestone field evaluations, followed by a flow chart of laboratory tests.

Two rock types in eastern Kentucky have been collected and analyzed. First, a typical section of overburden exposed in a highwall was described in detail, as illustrated in Figure 8.3. Then, characteristics of the light gray sandstone (lower sandstone in Figure 8.3) were evaluated and the results are illustrated in the field flow chart (Figure 8.4). This sandstone is a ledge former, the discontinuities are not closely spaced, and it does not ring when struck with a hammer. The rock sample is moderately hard to scratch, smells very earthy and absorbs water rapidly, so it is "rejected" as material for riprap.

The upper buff sandstone described in Figure 8.3 is even more crumbly than the gray sandstone and its characteristics follow a similar flow line as the gray sandstone (Figure 8.4). However, at certain sections of the outcrop, this sandstone has closely spaced continuities so it would be rejected at an earlier point on the sandstone flow chart.

Field observations of riprapped channels indicate that some of these sandstones appear to be slowly breaking down and crumbling into smaller rock fragments and single grains, even though they have been in-service for only several years. Sediment-production and erosion within the diversion channel can potentially increase, resulting in complete failure, although further study and observation of longer stretches of channel are required to verify these suspected consequences.

A durability test was conducted on sandstones which are stratigraphically equivalent to these sandstones. The samples, taken from a nearby mining site, were tested for possible use in rock-fill embankments using the Slake Durability test as defined by the Kentucky Bureau of Highways (Phase I report, Appendix F.2). Results indicated that both sandstones meet the durability specifications for rock-fill embankments which is an index exceeding 90 percent. However, in assessing riprap durability in this mining area, it is strongly recommended that (1) a durability index of >95 be used as the limit for evaluating suitable <u>riprap</u> material and (2) results of durability tests of a rock type should <u>not</u> be extrapolated from one mining section to another

(Flow Chart may be modified by in-service performance data)
Check the appropriate boxes along flow chart lines to define the durability of rock in question.

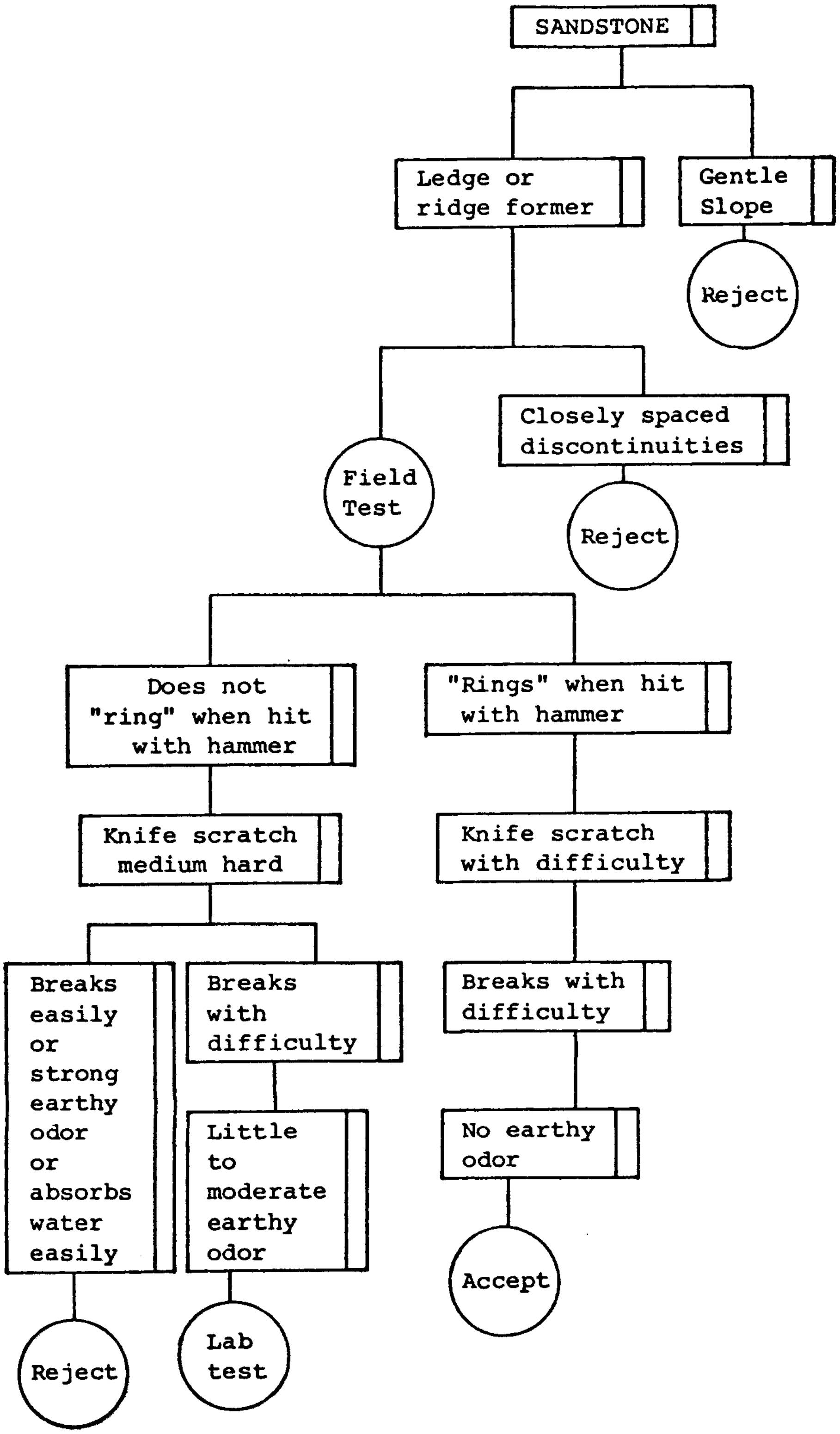


Figure 8.2a. Rock durability field flow chart.

(Flow Chart may be modified by in-service performance data)
Check the appropriate boxes along flow chart lines to define the durability of rock in question.

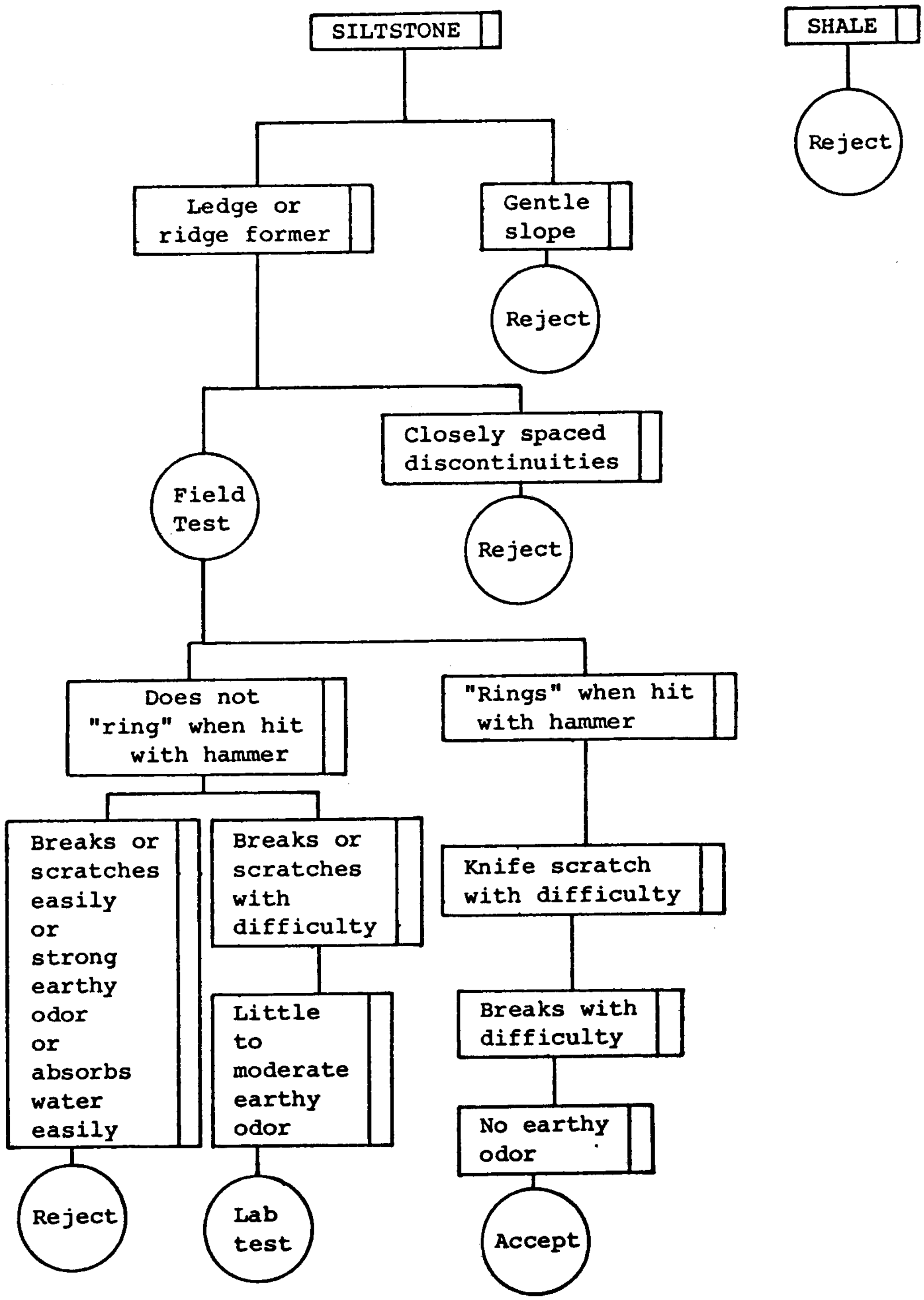


Figure 8.2a (continued).

(Flow Chart may be modified by in-service performance data)
Check the appropriate boxes along flow chart lines to define the durability of rock in question.

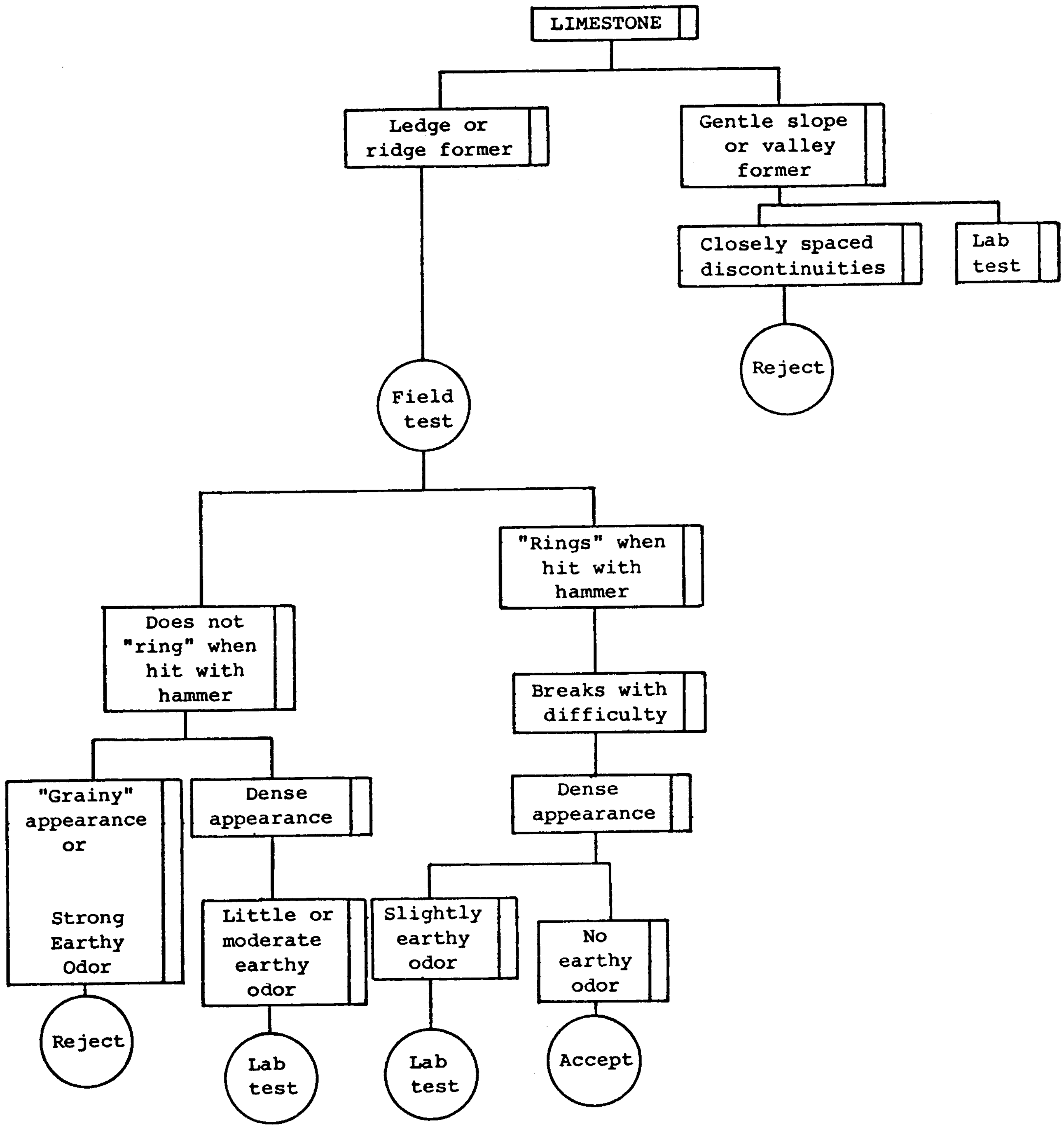


Figure 8.2a (continued)

(Flow Chart may be modified by in-service performance data)
Check the appropriate boxes along flow chart lines to define the durability of rock in question.

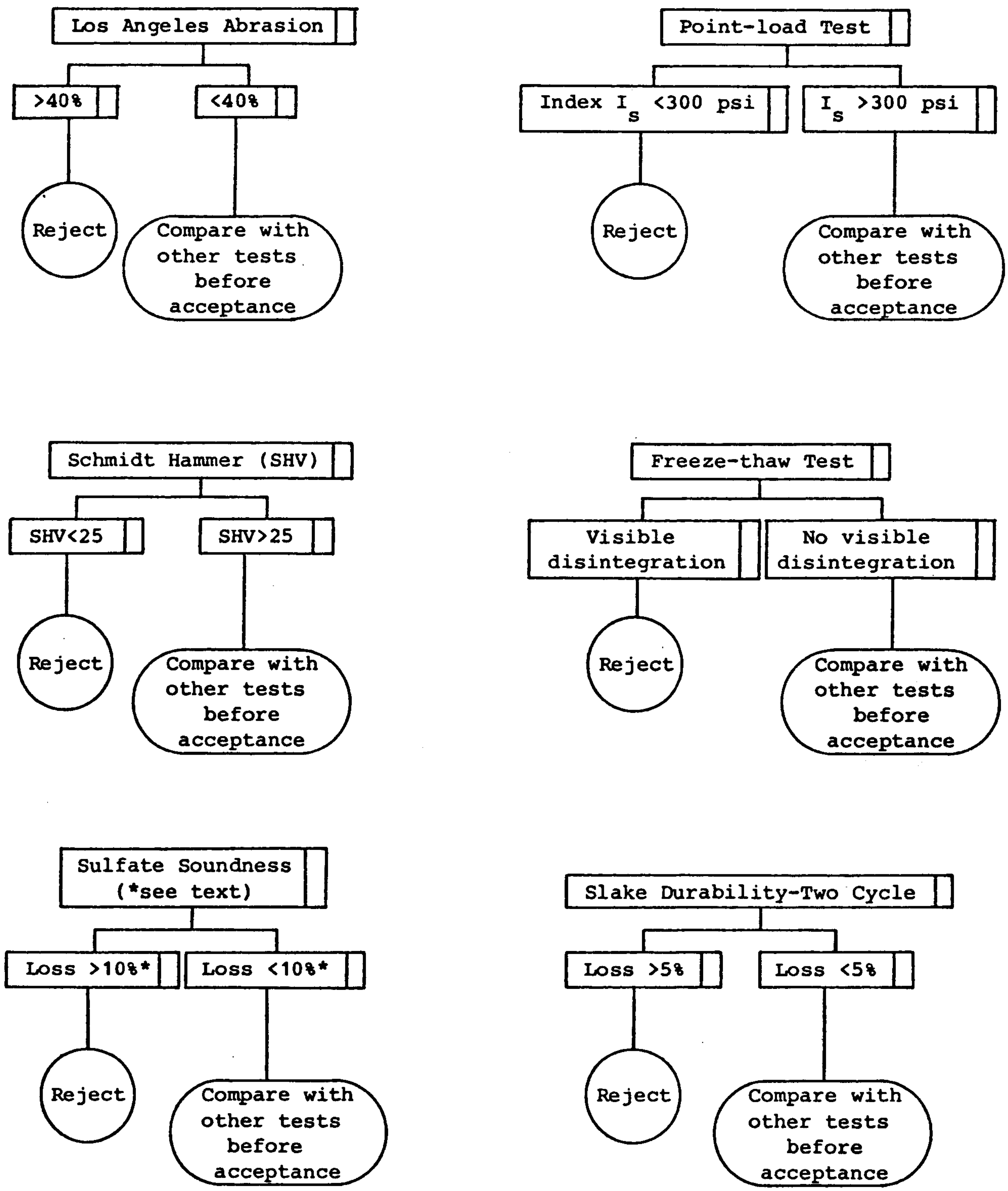


Figure 8.2b. Rock durability laboratory flow chart.

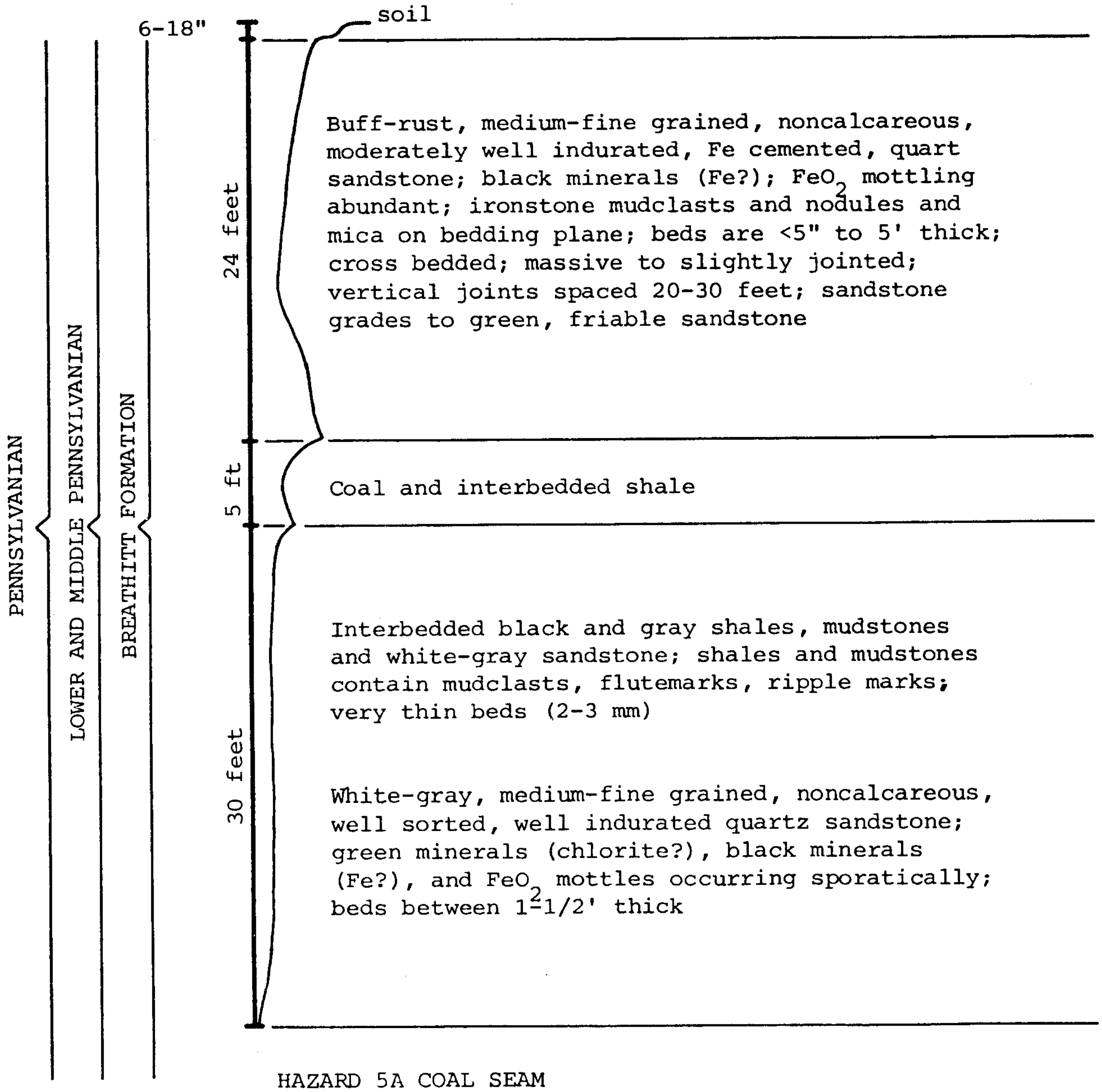


Figure 8.3. Stratigraphic description of overburden at a mine site in eastern Kentucky.

(Flow Chart may be modified by in-service performance data)
Check the appropriate boxes along flow chart lines to define the durability of rock in question.

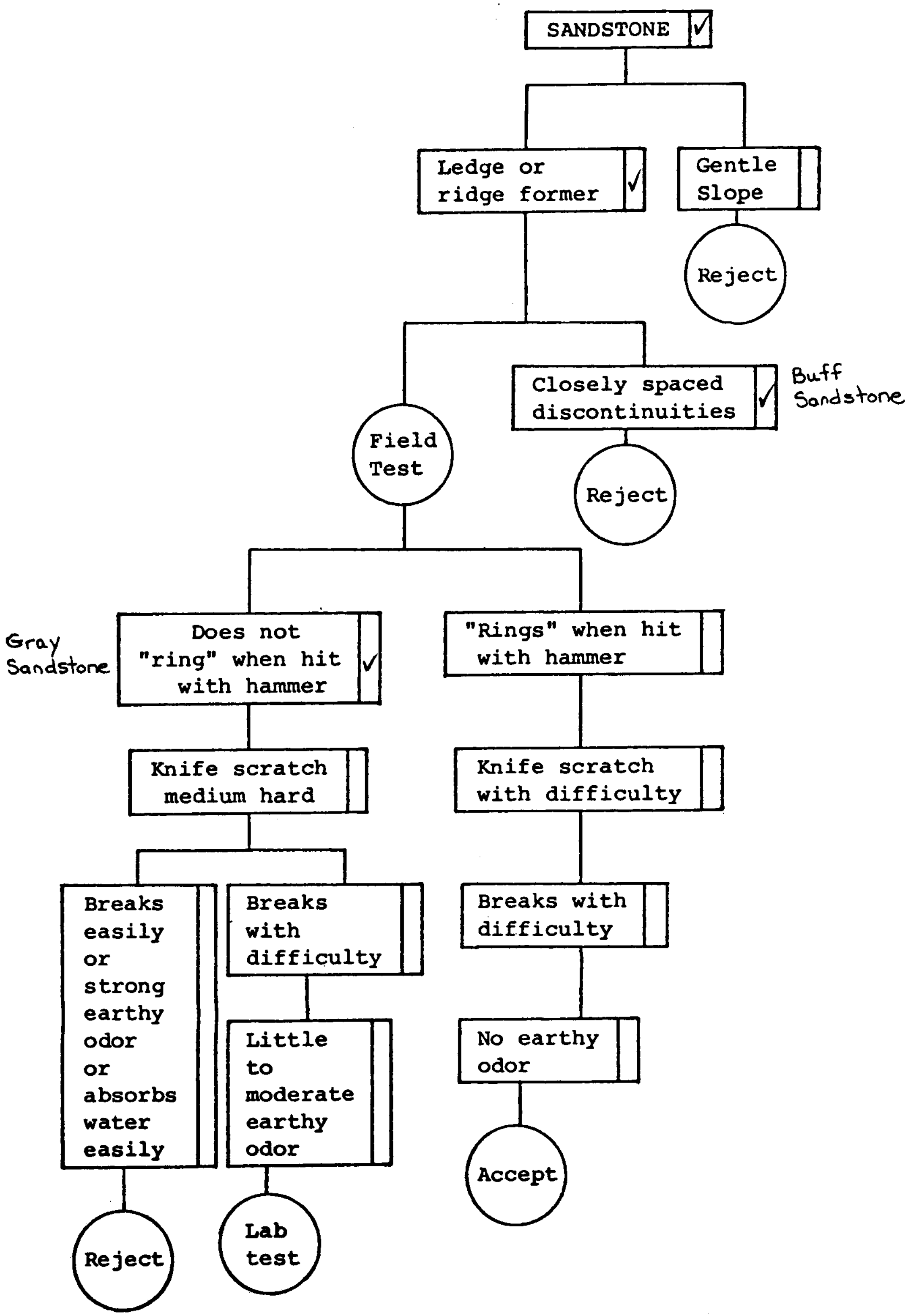


Figure 8.4. Rock durability field flow chart: example.

because rocks can vary significantly (both laterally and vertically) within very short distances.

The second rock type exemplifying the use of the durability flow chart is a gray limestone used as riprap at an inactive, partially reclaimed site. As shown in the chart (Figure 8.5), this specimen rings when hit with a hammer, is difficult to break, and has a dense appearance, but it has a slightly earthy smell suggesting clayey matrix material. Therefore, further laboratory tests should be conducted, such as the durability test and LA abrasion test (Figure 8.1).

The field performance of this particular limestone is not known. As indicated by the top of the flow charts, in-service performance over an extended period may override the results of the flow chart and these field data should be carefully evaluated if available.

8.2.7 Summary and Conclusions

Of equal importance to sizing rock riprap is the determination of durability. Suitable material is often available on site and effort should be made to select the most durable rock types.

A three-fold procedure for evaluating rock durability on coal mining sites has been presented. Rock types suitable as riprap can be identified by incorporating field observations with simple geotechnical information. In reviewing the detailed procedure, one significant component is the "in-service performance" evaluation over an extended period because it allows judgment of a rock type based on its "actual" field performance, rather than "probable" performance predicted by the other evaluations.

It must be recognized that considerable judgment is required for use of site evaluations and laboratory testing procedures described herein. All evaluations should utilize all of the site investigations. However, selected laboratory or index tests should be run on rock types that have been judged to be marginal during site investigations. The greatest number of tests should be run on these marginal rock types and generally there is sufficient interaction among the various tests described to provide a basis for judging durability on a minimum of tests.

It must be emphasized that index tests are designed for solid or intact samples. High spacing frequency and number of discontinuity sets can make a rock exposure useless for either an exposed rock channel or for riprap.

(Flow Chart may be modified by in-service performance data)
Check the appropriate boxes along flow chart lines to define the durability of rock in question.

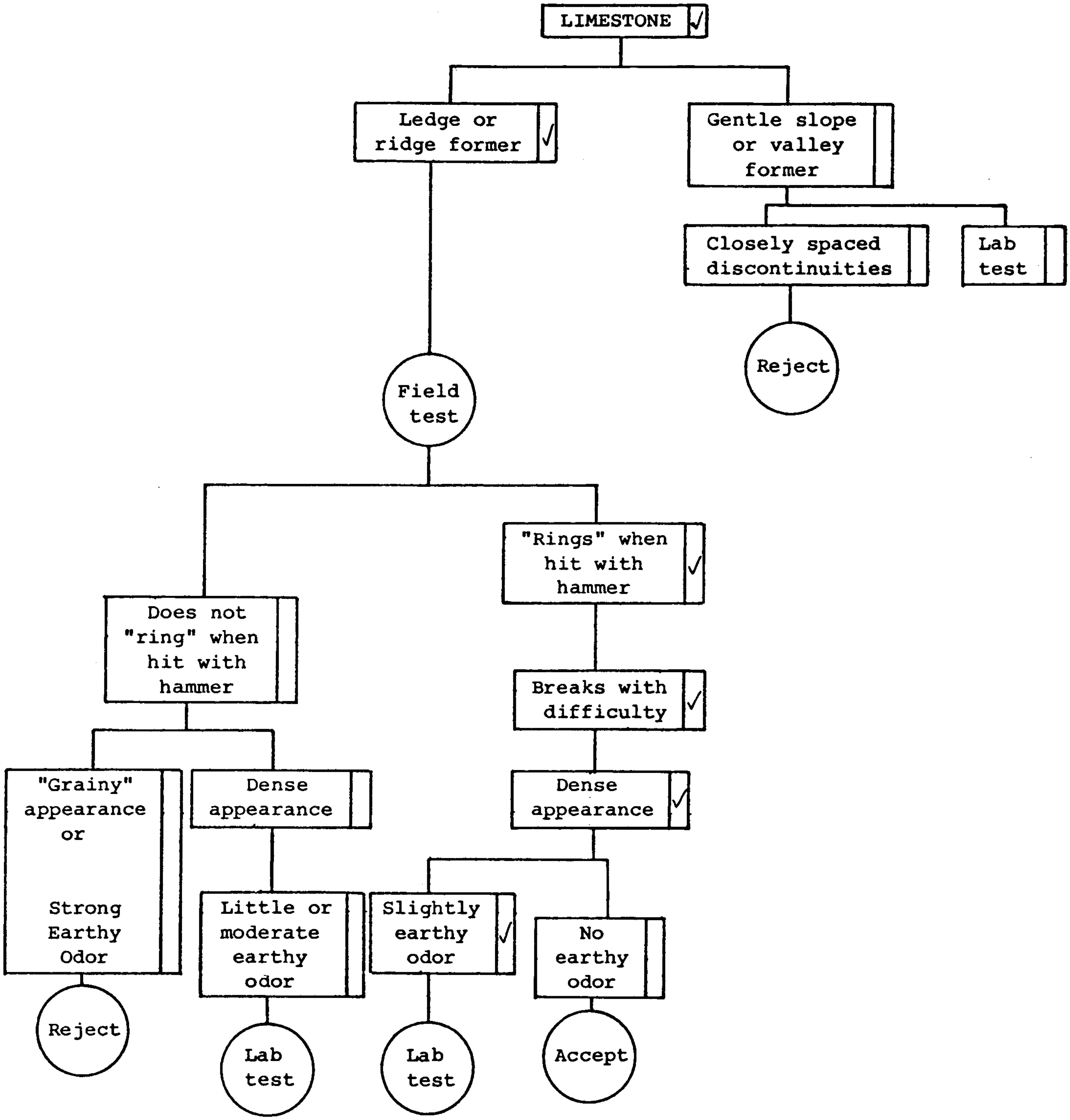


Figure 8.5. Rock durability field flow chart: example.

The ultimate goal of any of these tests or methods is to accurately predict durability and prevent consequential erosion hazards. The method recommended using information in Figure 8.1 is an effort to relate field, as well as laboratory tests, to actual service life of riprap or channels. This procedure will facilitate determination of suitable riprap for use at mining sites as economic and environmental demands intensify.

8.3 Geotechnical Stability Considerations

8.3.1 Introduction

In the following section, major factors causing slope failure are described which may influence stability of rock and soil slopes of channels. Potential problems that may be encountered in the field are addressed. Hence, it is hoped that slope failure mechanisms at a particular site can be identified to facilitate the design and maintenance of stable channels.

8.3.2 Slope Stability Factors

It is difficult to specify exact procedures to predict slope stability because stability will vary on a site specific basis as a function of natural weathering processes under a particular climate, type of rock, and use; however, general factors are known to interact with each other and determine stability of slopes. Assessment of these factors can be a valuable tool in the preliminary analysis of design channels and diversion ditches.

The following questions are important to consider prior to and during initial field observations and evaluation of slope stability at a particular diversion site:

1. Are slopes at the site currently stable?

2. Will the proposed construction activity influence slope stability at the site?

3. Could potential future changes in land use or the environment decrease slope stability at the site?

4. If the site is currently unstable, what countermeasures can be implemented that would make the site suitable for diversion channels?

5. If the site is currently stable but construction or future activities will make the site unstable, what countermeasures are necessary?

8.3.2.1 Natural Ground Surface Slope

One of the major factors to consider is the existing ground surface slope. As previously mentioned, the slope steepness of natural, in situ rock or soil material is usually a reliable indication of the inherent stability or angle of repose of the rock or soil.

Before a natural slope face is cut, the angle of repose should be measured. After the cut the "new" slope should be graded to the original slope. A guide for determining the horizontal or vertical extent of a properly graded cut slope after a 10-foot cut is made and is illustrated in Figure 8.6.

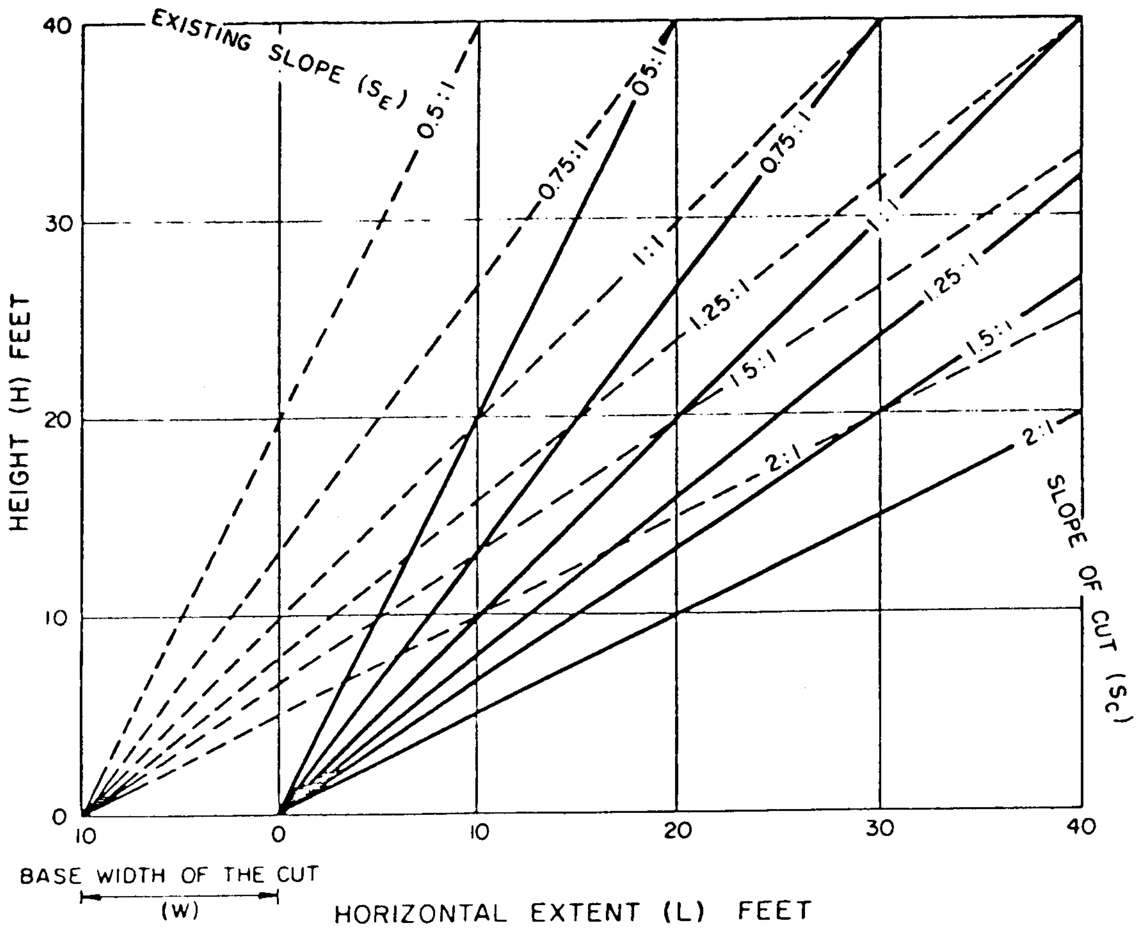
8.3.2.2 Earth Material Type

An important factor interacting with the natural slope is the type, structure, and stratigraphy of rock or surficial material. Detailed analysis and evaluation of durable and "stable" rock types for use as riprap or bedrock channels are given in the rock durability section. Figures 8.7 and 8.8 illustrate situations of potential slope failures on favorable and unfavorable bedding plane orientations. In considering slopes constructed in unconsolidated deposits and soils, generally the greater the backslope gradient and the more ground water present, the less stable will be the slope. Table 8.1 gives general guidelines for suitable side slopes of channels built in unlined and lined unconsolidated materials. General guidelines for backslopes of cut sections through rock are given in Table 8.2.

In steep slope areas in the Appalachian Basin, most soils are very shallow and larger cuts will consist mostly of shale, sandstone or limestone backslopes. Where the backslope consists of intermittent layers of shale, sandstone or limestone, it is possible that the exposed shale strata will weather and erode, thereby undercutting the more durable layers. Gentler slopes designed for the most incompetent material are required to avoid these conditions.

8.3.2.3 Ground Water

Near-surface flow of water can induce (1) excessive pressures along bedding planes or discontinuities, (2) erosion of rock by chemical solution or mechanical abrasion, and (3) increased weathering rates and disintegration by



(1) $S_E > S_C$

(2) $L = W \dfrac{S_C}{S_E - S_C}$

(3) $H = \dfrac{L}{S_C}$

FOR $W \neq 10$ FEET MULTIPLY THE VALUE L AND H OBTAINED FROM THE GRAPH BY COEFFICIENT $C = W/10$.

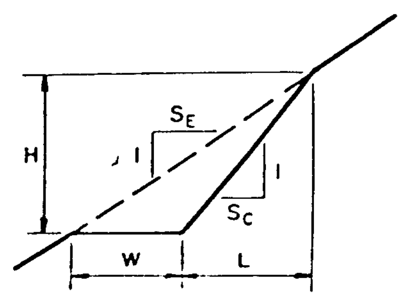
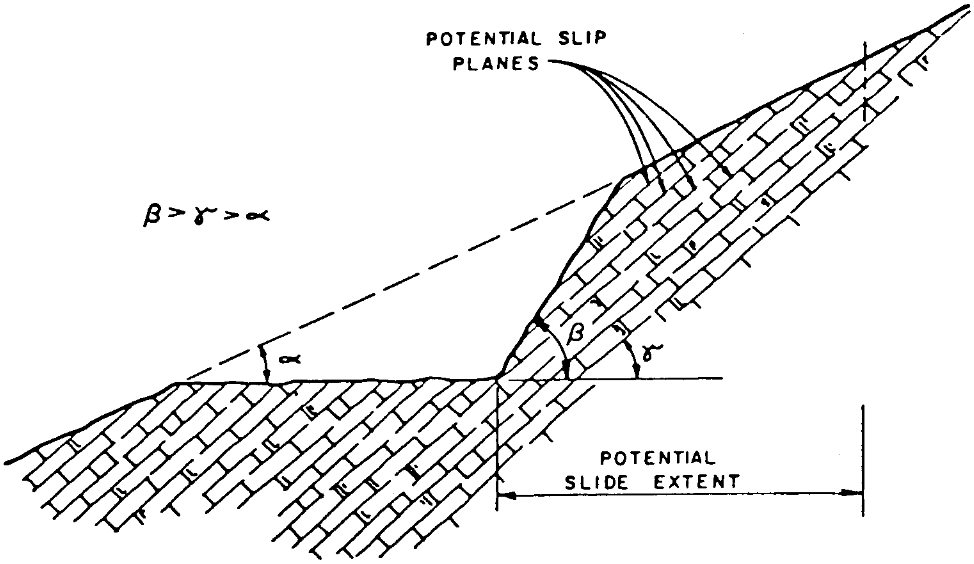
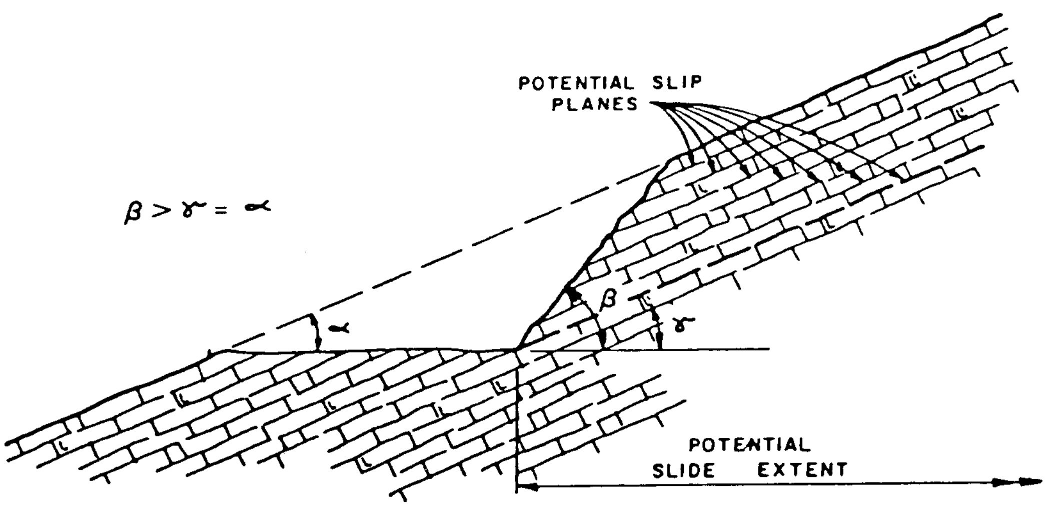


Figure 8.6. Horizontal and vertical extent of cuts for base width of 10 feet (D'Appolonia, Inc.).

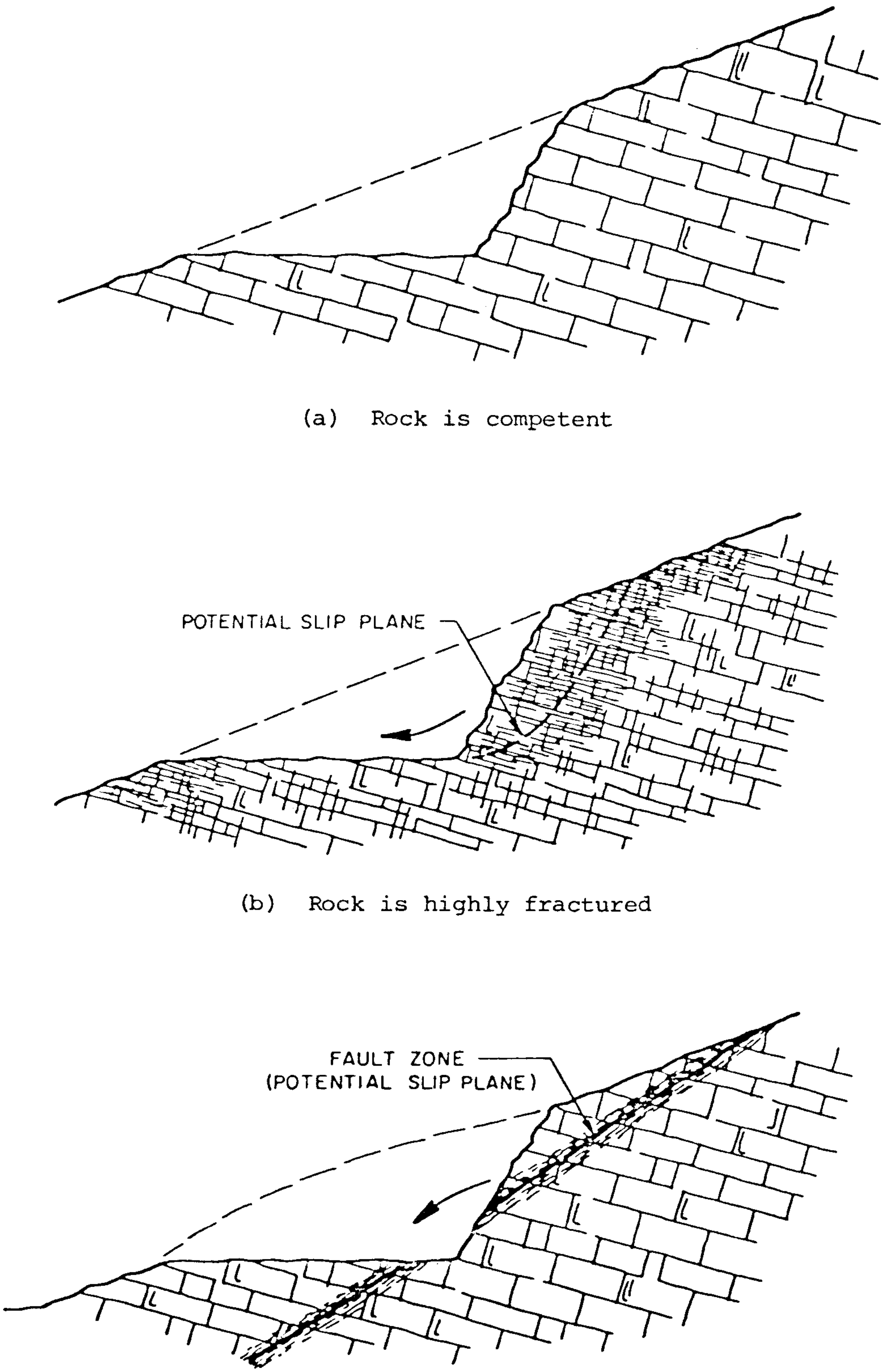


(a) Extent of potential slide small



(b) Extent of potential slide severe

Figure 8.7. Unfavorable orientation of bedding planes (D'appolonia, Inc.).



(a) Rock is competent

(b) Rock is highly fractured

(c) Fault zone is the governing feature

Figure 8.8. Favorable orientation of bedding planes (D'appolonia, Inc.).

Table 8.1. Stable Side Slopes for Channels Built in Various Kinds of Materials (from Chow, 1959).

Material	Side Slope
Stiff clay or earth with concrete lining	1/2:1 to 1:1
Earth with stone lining, or earth for large channels	1:1
Firm clay or earth for small ditches	1 1/2:1
Loose sandy earth	2:1
Sandy loam or porous clay	3:1

Table 8.2. General Guidelines for Cut Sections Through Rock.

Type of Rock	Back Slope
Hard and medium sandstone and limestone	1/4:1
Soft sandstone, medium hard shale, limestone, siltstone	1/2:1
Soft shale interbedded with siltstone or limestone having AASHO M-145-49 granular classification	1:1 (1 1/2:1 if height of cut < 25 ft)
Soft shale having AASHO M-145-49 silty clay classification	1 1/2:1 (2:1 if height of cut < 25 ft)

freeze-thaw and wet-dry cycles. Therefore, water control is an important consideration and must be evaluated on a site-specific basis. Field investigations should include observations of seepage and surface drainage patterns, along with utilization of data from well logs. This is important in assessing possible changes in the water table and seepage from loading (produced by diversion structures and spoil placement).

8.3.2.4 Design Flow

The above factors are, in turn, modified by the amount of water which is to be carried by the diversion channel. In general, the higher the conveyance factor, the lower the channel gradient should be to resist abrasive and other forces and remain stable. Additionally, the velocity of flow is critical to stability. In general, velocities should be maintained less than 15 fps to insure long-term stability in a bedrock or rock riprap channel.

8.3.2.5 Other Stability Factors

Several other features and environmental conditions should be considered when evaluating the degree of stability within channels. These factors include: (1) the amount of precipitation or peak discharge of storms and extended wet periods, (2) infiltration characteristics of surrounding terrain, (3) vibration from blasting or earthquake, and (4) loading of the head or toe of the slope. These factors can be assessed by contacting the local or state weather service, the District Soil Conservation Service and by evaluating operations at the mining site.

8.3.3 Stability Problems Unique to the Appalachian Basin

Certain stability problems are unique to the Appalachian region due to the lithology in the basin. These problems should be considered when designing diversions through these kinds of materials.

8.3.3.1 Shale

Shales have been cited by numerous sources as a cause of slope instability and slide activity in the Eastern Coal Province. These shales are highly susceptible to chemical and physical weathering and breakdown. Slaking is a common process occurring in mining areas when these shales are wetted and loaded (Shamburger, Patrick and Lutton, 1975; Fisher, Fanaff, Picking, 1968).

8.3.3.2 Sandstone

Weakly cemented sandstones have been recognized as a problem in West Virginia and Kentucky because they are very susceptible to weathering and disintegrate into loose sands after placement. An example evaluation of a sandstone from eastern Kentucky is given in Figure 8.4.

8.3.3.3 Colluvial Deposits

Colluvium is deposited by gravity and is generally loosely deposited and has a low shear strength. This is of particular concern when constructing sedimentation control structures which often produce changes in the groundwater level. Data from test drill cores at a mining site may provide information on the depth and geometry of colluvium. Typically, terraced slopes fail at the colluvium/rock interface and failure is dependent on the depth of colluvium. Existing slopes which are <30° most often result in failure along the fill/colluvium interface regardless of the depth of colluvium.

8.3.3.4 Aquifers and Underclays

Aquifers or water-bearing seams often lead to stability problems, especially when associated with underclays and shales. The mining engineer should be aware of the presence of water-bearing units and note any springs and seeps at the coal/underclay boundary. Dewatering may be a necessary part of the mining operation to avoid adverse mining conditions and slide activity. Water well logs and domestic well information should be accessed prior to mining and post-mining diversion construction.

8.3.3.5 Existing Landslides

Existing landslides or areas undergoing creep (which can occur on slopes as gentle as 5°) may reactivate or accelerate when incised by diversion structures. The mining regulations comment against the siting of diversions in a manner such that the potential for landslides is increased. To evaluate landslide hazards judiciously before construction of diversions, the following points summarize criteria indicating past or present slope movement or suggesting potential instability. These criteria can be assessed both in the field and by use of aerial photography. Further explanation can be found in Piteau and Peckover (in press) and Phase I report, Section 10.3.2.

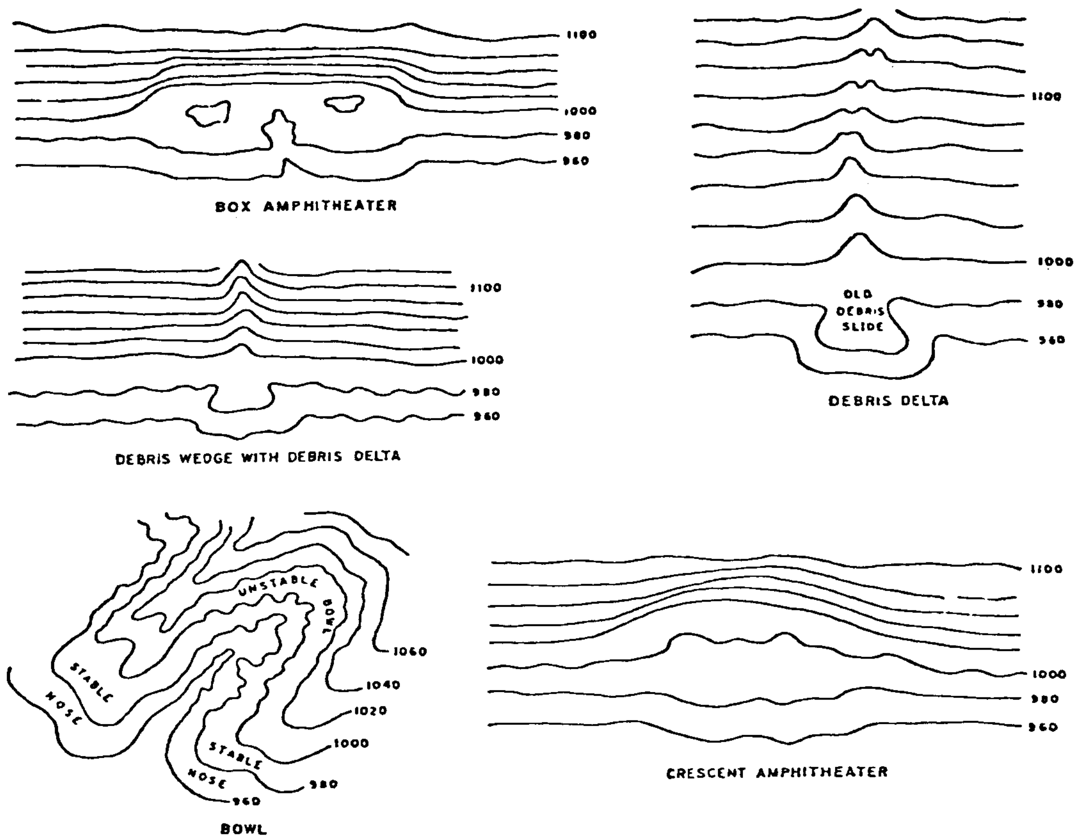


Figure 8.9. Schematic topographic diagrams of five landforms that are highly susceptible to landslides.

1. Existence of an old escarpment; indicated by vegetative and topographic patterns (Figure 8.9).

2. Existence of cracks at the top and near the toe of the slope.

3. Springs and seeps.

4. Erosion near the toe of the slope.

5. Soil piping.

8.4 References

American Society for Testing and Materials, 1980, Book of ASTM Standards, Part 14, Concrete and Mineral Aggregates, Am. Soc. for Testing Materials, Spec. C88, C131, C535, C666, 834 p.

Chow, V. T., 1959, Open Channel Hydraulics, McGraw-Hill, New York, 680 pp.

E. D'appolonia Consulting Inc., Engineering Design Manual Coal Refuse Disposal Facilities: U.S.D.I., Mining Enforcement and Safety Administration.

Fisher, S. P., A. S. Fanaff, L. W. Picking, 1968, "Landslides of Southeastern Ohio," Ohio Journal of Science, Vol. 68, No. 2, pp. 65-80.

Huddle, J. W., E. J. Lyons, H. L. Smith, and J. C. Ferm, 1963, "Coal Reserves of Eastern Kentucky," Bulletin 1120, U.S. Department of the Interior, Geological Survey, Washington, D.C., 247 pp.

Piteau, D. R., F. L. Peckover (in press), Landslides: Analysis and Control, Special Report 176, Transportation Research Board, National Academy of Sciences, Washington, D.C.

Shamburger, J. H., D. M. Patrick and R. J. Lutton, 1975, Design and Construction of Compacted Shale Embankments, August, Report No. FHWA-RD-75-61.

U.S. Department of the Interior, Bureau of Reclamation, 1974, Earth Manual, 2nd Edition, U.S. Bureau of Reclamation, Dept. of Interior, 810 pp.

Williamson, D. A., 1980, Uniform rock classification geotechnical engineering purpose: Transportation Research, Record 783, National Research Council, p. 9-14.

This page intentionally left blank.

IX. ECOLOGICAL CONSIDERATIONS

9.1 General

Ecological considerations are important for the proper design, construction, maintenance and/or removal of diversions.

The following discussion provides criteria and guidelines for evaluating factors influencing the ecological environment of a mining area. First, water quality and physical characteristics are described (Soil Conservation Service, 1977), then stream study and investigation procedures are briefly considered and finally recommended reconstruction techniques and considerations are given and the potential short- and long-term effects of diversion structures are evaluated.

9.2 Water Quality

Water quality is a limiting factor for fish production and is an important element in determining the biological community. The following items affect fish species suitability, production, and survival.

1. Temperature is an important physical factor. Summer water temperatures commonly vary as much as 10° in a 24-hour period. In general, summer temperatures should be between 50° and 70° for cold-water species. Egg hatching success is best for trout between 45° and 55°. Warm-water species need summer temperatures between 70° and 90°.

   Removal of shade tends to raise water temperatures while the maintenance of vegetation for shade may keep the water cooler. Water temperatuare is raised when velocity and depth are reduced. Water temperature may be affected by release of water from upstream impoundments.

2. Turbidity caused by inorganic material, such as clay, is detrimental to fish production. Such material destroys spawning areas by sedimentation and reducing growth of bottom organisms. Adult fish generally can withstand high levels of turbidity for short periods of time, but prolonged exposure may cause mortality.

It is reported that turbidity as high as 245 mg/l is not harmful to fish. In fact, fish thrive in water with turbidities that range over 400 mg/l and average 200 mg/l. Turbidities of 3,000 mg/l are considered dangerous to fish when maintained over a ten-day period. Trout eggs were destroyed with 2,000 mg/l turbidity for six days. Symptoms of fish stress appear as turbidity approaches 20,000 mg/l; death between 50,000 and 200,000 mg/l. At turbidities causing death, the opercular cavities were found to be matted with soil and the gills had a layer of soil in them.

3. Oxygen requirements for subsidence of fingerling and adult salmon and trout are about 6 ppm dissolved oxygen. Incubating eggs require a minimum of 8 ppm. Warm-water species require about 3 ppm. Water at or near oxygen saturation, for its temperature and elevation, is always satisfactory. Oxygen is put into water by direct absorption from the atmosphere, photosynthesis of growing plants, and by tumbling action of stream or waterfalls and turbulence generated at drop inlets or drop spillways. Turbidity, reduced flow, and nontumbling action reduce oxygen.

4. Carbon dioxide is another of the basic factors determining productivity of waters. It is necessary in photosynthesis and for keeping minerals, such as calcium, in solution. High carbon dioxide levels reduce the ability of fish to take up oxygen and to dispose of carbon dioxide from the body. Concentrations of carbon dioxide should be kept below 25 ppm.

   Carbon dioxide is put in water by direct absorption from the atmosphere, decomposing organic matter, and respiration of plants and animals. It is removed by photosynthesis, agitation of water, evaporation, and rise of bubbles from depths.

5. pH is a measure of the acid intensity in water. The scale of reading is from 0 to 14. Optimum fish production lies between 6.5 and 8.5. Values below 5 and above 9 affect the ability of fish to take oxygen from the water source. Water pH is changed if an acid layer of soil is exposed in stream bottom or sides.

9.3 Physical Characteristics

These stream channel features affect fish production, species suitability, and survival.

1. Bottom material - The bottom material of a stream is important from the standpoint of food production and natural spawning. The following yield in grams of food per square foot in terms of different stream bottom materials has been recorded: silt - 3.07; cobble - 2.47; coarse gravel - 1.51; fine gravel - 0.93; and sand - 0.1.

   Coarse and fine gravel beds in riffles are best for trout to deposit their spawn successfully. Most warm-water fish spawn in sand or silt beds in water less than three feet deep and with little or no current.

2. Water types

a. Riffle - Section of stream containing gravel and/or rubble, in which surface water is at least slightly turbulent and current is swift enough that the surface of the gravel and cobble is kept fairly free of sand and silt.

Riffles are essential for trout spawning and food production. Riffles should occur at intervals equal to every five to seven channel widths. The current in the riffle should be swift enough to carry away sediment. The bed material in riffles should be larger than in pools so as to provide for aeration of the water. A water depth of six inches is desirable.

b. Pool - Section of stream deeper and usually wider than normal with appreciably slower current than immediate upstream or downstream areas and possessing adequate cover (sheer depth or physical condition) for protection of fish. Stream bottom usually is a mixture of silt and coarse sand.

Pools are valuable as resting and refuge areas. Some surface feeding is also done.

c. Flat - Section of stream with current too slow to be classed as riffle and too shallow to be classed as a pool. Stream bottom usually composed of sand and finer materials with coarse cobbles, boulders, or bedrock occasionally evident.

d. Cascades or bedrock - Section of stream without pools, the bottom consisting primarily of bedrock with little cobble, gravel, or other such material present. Current usually faster than in riffles.

3. Stream side vegetation - This item pertains to the relation of vegetation to stream shade and fish shelter. Low shrubs and grasses provide shade for small streams, but do not over-shade them. Such vegetation does not clog streams by falling in the water, and it provides hiding cover for fish if allowed to hang over the bank into the water.

Trees are necessary for shade along streams over 30 feet wide since low shrubs and grasses shade only a small portion of this width.

An ideal situation, along small streams, is enough trees for aesthetic purposes and low shrubs and grasses providing shade and cover. Along large streams, trees for about 40 percent of the stream length, on both sides, should be present. There probably are situations where the presence of trees well back from the water's edge furnishes shade almost as good as comparabale ones closer to the stream. This would be true especially on the east side of north-south flowing stream and the south side of east-west streams.

4. Velocities - Tolerable water velocity for fish is governed by several factors, chiefly by the species of fish, size of fish, and the distance and frequency of resting areas. Boulders, pools, deflectors, etc. provide resting areas.

9.4 Stream Study and Investigation Procedures

The objectives of any stream study should be clearly defined prior to being undertaken. Neglect of this essential preliminary step may result in failure to obtain critical information or conversely in expenditures of needless and wasteful amounts of time, effort and money. The major objective of studies designed for stream location is to determine the biological community present within the existing streams, and conditions existing within the stream to support the community.

This will involve one or more sampling stations on the stream system. Sampling may be occasional, perhaps at weekly, monthly, or even quarterly intervals. Sampling should be designed to obtain quantitative or qualitative nature and will usually include physical, chemical and biological data.

Investigations involving the definition of a biologically active stream may necessitate the sampling of the three basic stream habitats (riffle, pool, run), if present. Study sites should be representative of the stream areas most likely to be affected or impacted by mining activity. Stream condition must be considered since unusual stream conditions such as high water can make biological sampling impractical and/or lead to erroneous conclusions based on limited or incomplete samples. Extremely low water stages can also make the results of sampling efforts of limited value. In general, in the Eastern Coal Province the greatest measurable aquatic invertebrate family diversities will be found in late spring and summer. For a long-term biological survey program it is best that collections be made at least once during each of the annual seasons.

The series approach is used to document water quality and biological changes throughout a reach of river or stream. The pattern of changing quality reflected by the relationship among the several stations is more important than the isolated biological or physiochemical quality at any one station. The assessment of the relationship among the stations therefore depends on the collection of data representative of the stream at each station.

Establishment of sites that are physically similar is desirable. However, when dissimilar sites are to be compared, care should be taken so that data comparisons do not lead to false conclusions concerning the biological communities occurring in these study sites. The establishment of one or more control stations can allow for the comparison of water and biological quality above and below the point of alterations. A control station upstream of the source of impact is as important as the stations within or below the impact area and should be chosen with equal care to ensure representative results. The distance between the sampling sites should be sufficient to permit accurate measurement of potential changes.

Bridges should be avoided when sampling for bottom organisms. Benthic populations may have been altered or destroyed by bridge construction activities. In addition, the physical environment of the stream near bridges is often altered and may be unrepresentative of the stream in general. If sampling near a bridge is necessary, then it should be limited to the upstream side. Bridges frequently shade the stream beneath them and reduce light exposure and penetration.

A survey sheet has been included in Appendix E so that the investigator may accurately and concisely document the physical, chemical and biological properties observed at each sampling site or locality. Detailed instructions and suggestions for completing these survey sheets are also included so that some consistency might be obtained when data are recorded by the user. Individual survey sheets allow the investigator to record the precise stream conditions at each station or site at the time of survey. The field sheets can be used to document, in a comparative manner, the changes that occur from site to site at any given sample date or at one site over an extended time period. Additionally, the latest available reference on actual sampling techniques has been duplicated and placed in Appendix E for ready reference.

9.5 Reconstruction

If a stream must be relocated, various stream improvement techniques may be employed to create suitable habitats for the desired fish species. Many types of devices may be installed to produce more favorable conditions for fish production. They may be classified as (1) dams, (2) deflectors, and (3) covers. Small dams may be built, creating ponds behind the dams and deep holes on the downstream side. The dams are generally constructed of logs, boulders, and rocks, or sticks and sod, depending on the stream conditions.

The ecological disadvantages of damming are: (1) the water is exposed to warming, (2) sand may fill in the pool above, and (3) fish movement may be blocked. Creation of ponds increases the space of the aquatic habitat and provides resting places, while formation of deep holes allows fish to survive severe winters and dry periods (Figure 9.1). Gard (1961) found that suitable trout habitat could be created by the placement of dams in a California stream. Saunders and Smith (1962) used dams, deflectors, and shelters to provide more hiding places for young trout in a stream on Prince Edward Island.

Deflectors are partial barriers which accelerate and direct the current, creating both local scour and deposition. The deflector confines the flow to a narrower channel, causing a long, deep hole to be scoured. By narrowing and deepening the channel, overheating is prevented. The accelerated current exposes the gravel substrate required for trout spawning and washes finer particles to quiet waters, where they settle. Weed beds tend to form in these silt deposits, providing food and shelter for young fish. Shetter et al. (1949) described deflectors used to improve trout habitat in a Michigan stream. He used rectangular log cribs filled with gravel, soil, and sod and anchored into the bank at a 35°-45° angle from the downstream bank. Deflectors must be strategically located and angled properly to insure that they will effectively direct the current to produce the desired scour and deposition. Various deflector designs include wing deflectors, V-deflectors, Y-deflectors, I-deflectors, A-deflectors, and underpass deflectors (Hubbs et al., 1932). Tarzwell (1937) quantified habitat improvement resulting from deflectors placed in six Michigan streams. He found the following improvements in trout habitat: (1) increased number and depth of pools, (2) increased aquatic habitat due to increased pool width, (3) exposed gravel for trout reproduction, (4) riffle areas, (5) submerged plant beds, and (6) mucky areas. The deflectors created a more diverse substrate capable of producing a larger quantity of food organisms. "Food production in a sandy section may be increased by a deflector which uncovers gravel and produces mucky areas for plant beds," (Tarzwell, 1937).

Good cover is required to offer the fish hiding places and resting places sheltered from the current. Cover devices can be designed to catch naturally drifting debris, thus enhancing their effectiveness. Undercut banks and submerged tangles of roots, brush, or limbs provide good cover. Some types of cover are especially designed to protect erodible banks as well as to provide fish cover. A boom cover is such a structure and is designed to protect the outside bank of a bend while creating fish cover. A boom cover consists of old logs and stumps floating behind a fixed log barrier and placed at the out-

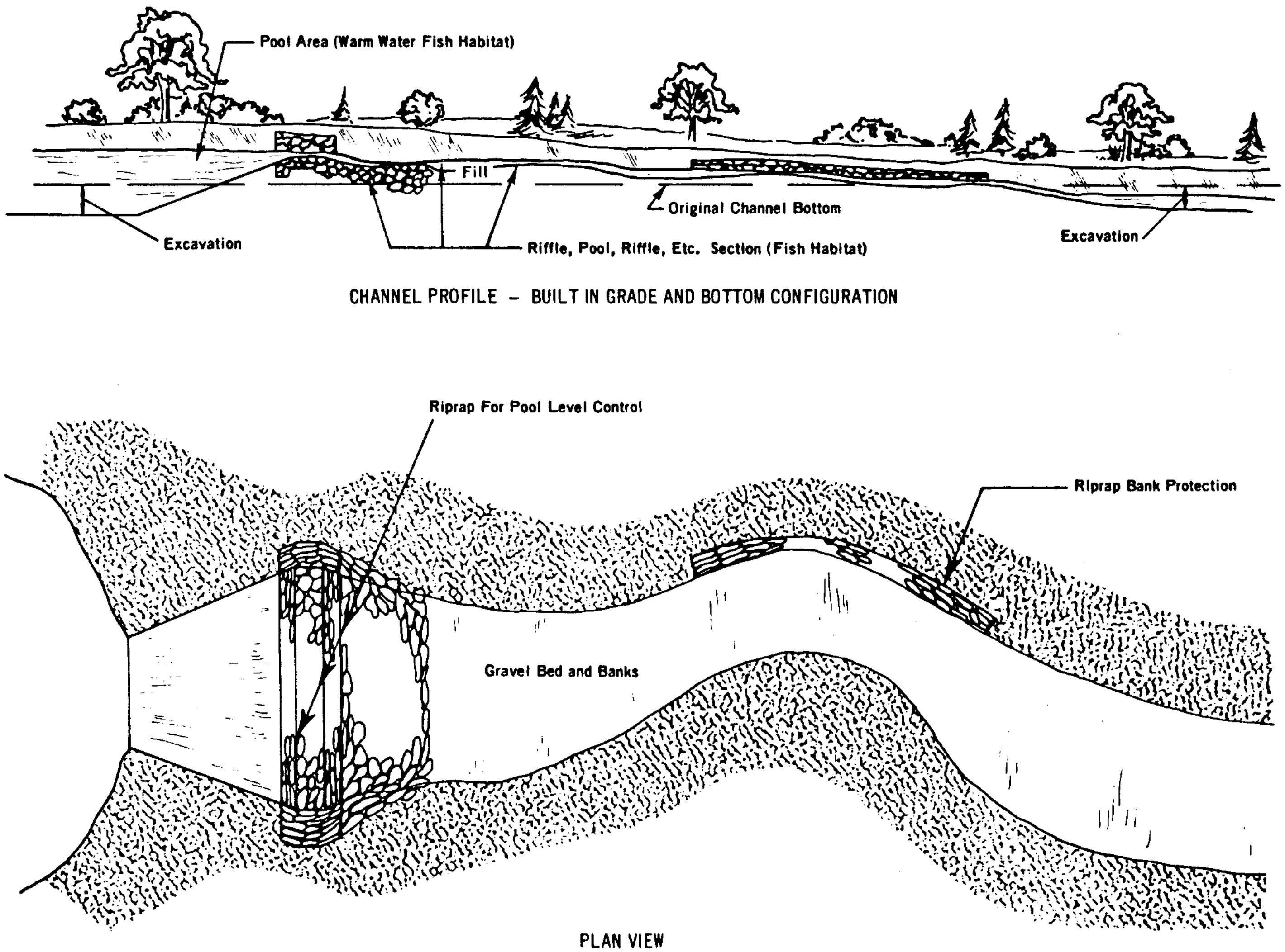


Figure 9.1. Channel segment with pond, riffle, pool layout (from Soil Conservation Service, 1977).

side of a bend. Several other types of structures used for cover include the bend raft, the bank cover, log platforms, the teepee cover, bridge covers, brush shelters within the stream, and overhanging shelters. Hubbs et al. (1932) discusses the construction and use of a variety of different stream improvement devices. Planting of streamside shrubs and trees also develops stream cover and protective shade. Figure 9.2 illustrates some of these concepts.

Research has been conducted to determine the success of stream reconstruction after rechanneling a trout stream in Pennsylvania (Bradt and Wieland, 1978). Rechannelization of the stream was required following highway construction in the stream's vicinity. Reconstruction efforts included: (1) installation of gabions to narrow and deepen the bed, (2) placement of large rocks and small dams to diversify the substrate, and (3) planting of trees, shrubs, and ground cover along the banks. The authors concluded "It is possible to restore a stream after it has been rechanneled, but, even with the most intensive efforts, it will take ten to twenty years before the streambed will again be shaded. Rechannelization should be pursued only as a last resort, because of the length of the recovery time. When it is absolutely necessary to damage existing stream ecosystems it must be kept in mind the amount of time, effort, and money involved in stream restoration and the many years that must pass before the stream ecosystem will recover from the damage." (Bradt and Wieland, 1978.)

9.6 Conclusion

To conclude, if a perennial or in some cases an intermittent stream must be relocated and reconstructed, a combination of stream improvement techniques should be used to restore the stream habitat. The ecological and engineering techniques employed depend on the specific stream involved, such as the bed and bank material, winter ice conditions, flooding, and the species for which the management is being designed. Selection and placement of habitat improvement devices must be carefully planned for maximum effectiveness.

Most state game and fish agencies have guidelines for stream improvement and restoration. A partial list of references and handbooks covering this subject is given in Section 9.7.

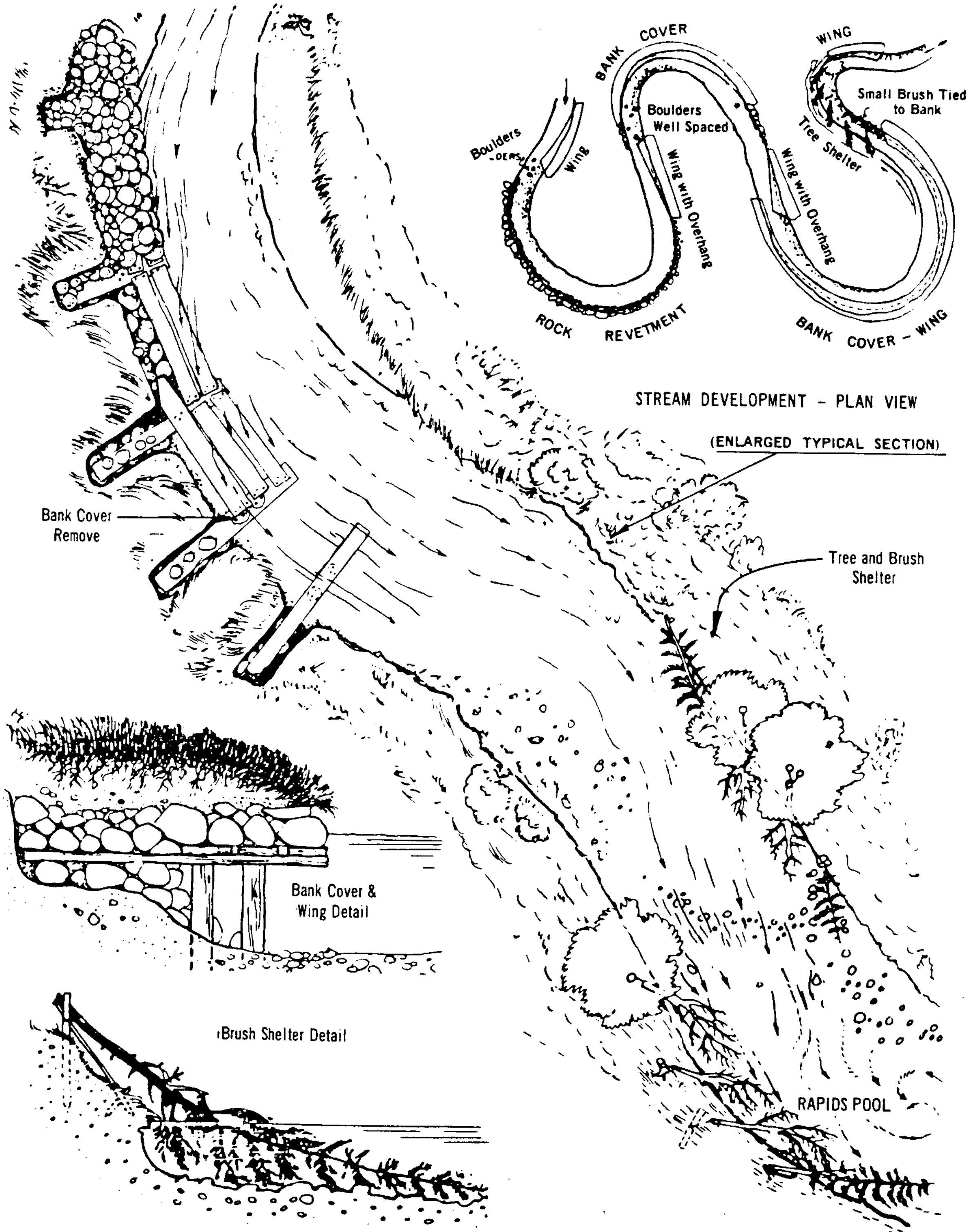


Figure 9.2. Fish habitat development (from Soil Conservation Service, 1977).

9.7 References

Alexander, H. E., "Stream Values, Recreational Use and Preservation," 1960. Twenty-fifth North American Wildlife and Natural Resources Conference, pp. 192-201.

Alvord, W. and J. D. Peters, 1963. "Channel Change in 13 Montana Streams," Montana Fish and Game Commission, 22 pp.

American Public Health Association (APHA), 1976. "Standard Methods for the Examination of Water and Wastewater," American Public Health Assoc., Washington, D.C.

Bradt, P. T. and G. E. Wieland III, 1978. "The Impact of Stream Reconstruction and a Gabion Installation on the Biology and Chemistry of a Trout Stream, U.S.D.I. Office of Water Research and Technology Report.

Fullner, R. W., 1971. "A Comparison of Macroinvertebrates Collected by Basket and Modified Multiple Samplers," J. Water Poll. Contr. Fed., 43: 494-499.

Gard, R., 1961. "Creation of Trout Habitat by Constructing Small Dams," J. Wildl. Mgmt., 25:384-390.

Hubbs, C. L., J. R. Greeley and C. M. Tarzwell, 1932. "Methods for the Improvement of Michigan Trout Streams," Bulletin of the Institute for Fisheries Research, No. 1, Michigan Dept. of Conserv., Ann Arbor, MI.

Saunders, J. W. and M. W. Smith, 1962. "Physical Alteration of Stream Habitat to Improve Brook Trout Production," Trans. Amer. Fish. Soc., 91:185-188.

Shetter, D. S., O. H. Clark and A. S. Hazzard, 1949. "The Effect of Deflectors in a Section of a Michigan Trout Stream, Trans. Amer. Fish. Soc., 76:248-278.

Smith, R. L., 1974. Ecology and Field Biology, Harper and Row Publishers, New York, NY.

Soil Conservation Service, 1977. "Design of Open Channels," Technical Release No. 25, October.

Southwood, T. R. E., Ecological Methods, Halsted Press, John Wiley and Sons, New York, NY.

Surber, E. W., 1937. "Rainbow Trout and Fauna Production in One Mile of Stream," Trans. Amer. Fish. Soc., 66:193-202.

Tarzwell, C. M., 1937. "Experimental Evidence on the Value of Trout Stream Improvement in Michigan, Trans. Amer. Fish. Soc., 66:177-187.

X. COMPREHENSIVE DESIGN EXAMPLE

10.1 General Description

Permanent diversion structures must be designed for conveying water around the spoil fill and across the reclaimed strip bench as shown in Figure 10.1. Four diversion channels must be designed. Channels A and B collect runoff from above the fill and convey it around the perimeter of the fill. The watershed above the fill is a reclaimed mountain top removal and the contributing drainage area to Channels A and B is 40 acres. Channel C is a reconstructed natural drainage extending from the toe of fill to the junction with a natural channel draining 16 acres. Diversion Channel D is a reconstructed channel passing over the reclaimed strip bench. All the channels are intermittent. The mine site is assumed to be located in southern West Virginia.

10.2 Hydrologic Design

The hydrologic design involves estimating the peak discharge rates for the two contributing areas, subwatersheds 1 and 2. For permanent diversions draining intermittent streams, Table 3.1 gives the required design event as the 100-year, 24-hour storm. Since the drainages involved in this design are relatively high in the watershed, there is no floodplain available to carry part of the design flows. Therefore, the channel must be capable of carrying the predicted peak discharges. Both the Rational Formula and the SCS TP-149 method will be used and the results compared to determine the design value.

10.2.1 Rational Formula

Subwatershed 1

Step 1: One C value is assumed adequate to represent the 40 acres existing within a reclaimed mountain top removal. From Table 3.3, the estimated C is 0.42 for a hilly pasture in a clay and silt loam.

Step 2: From topographic maps the maximum length of flow is estimated to be 2000 ft over an average slope of 30 percent. Using the formula given in Table 3.2,

$$t_c = 0.0078\ (2000)^{0.77}\ (0.30)^{-0.385}$$

$$t_c = 4.3 \text{ min}$$

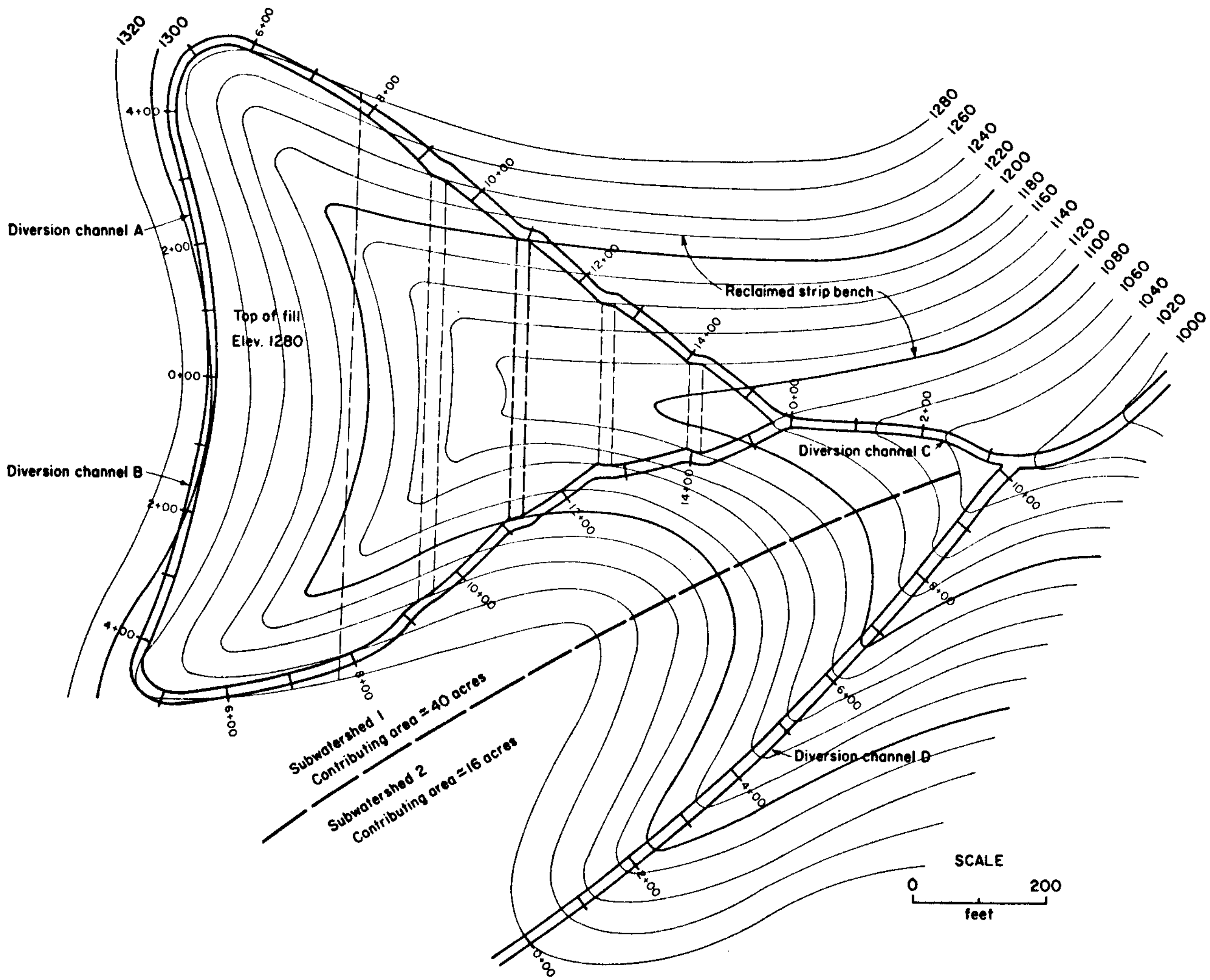


Figure 10.1. Topographic map of the design example.

Step 3: Since t_c is less than one hour, the 100-year, one-hour duration storm gives 3.0 inches of rain (Appendix A). From Figure 3.3 the required correction factor is 0.26, therefore

$$I_{4.3} = 3.0 \times 0.26 = 0.78 \text{ in}$$

$$\text{iph} = 0.78 \text{ in} \times \frac{1}{4.3 \text{ min}} \times \frac{60 \text{ min}}{\text{hour}} = 10.9 \text{ iph}$$

Step 4: From $Q = CiA$ (Equation 3.1),

$$= (0.42)\ (10.9)\ (40)$$

$$Q = 183 \text{ cfs}$$

Subwatershed 2

Step 1: Of the 16 contributing acres, about 5.0 acres represent the reclaimed strip bench. The remaining 11.0 acres exist as undisturbed woodland. For the reclaimed strip bench a C of 0.42 is again used. For the undisturbed area, a C of 0.50 is used. The composite C is then given by Equation 3.2 as

$$C = \frac{0.42\ (5) + 0.50\ (11)}{16} = 0.48$$

Step 2: From topographic maps the maximum length of flow is estimated to be 1200 feet for the average 30 percent slope. From the formula in Table 3.2,

$$t_c = 0.0078\ (1200)^{0.77}\ (0.30)^{-0.385}$$

$$t_c = 2.9 \text{ min}$$

Step 3: The 100-year, one-hour duration storm is again 3.0 inches. From Figure 3.3 the required correction factor is 0.20, therefore

$$I_{2.9} = 3.0 \times 0.2 = 0.60 \text{ in}$$

$$\text{iph} = 0.60 \times \frac{1}{2.9 \text{ min}} \times \frac{60 \text{ min}}{\text{hour}} = 12.4 \text{ iph}$$

Step 4: From $Q = CiA$ (Equation 3.1),

$$Q = (0.48)\ (12.4)\ (16)$$

$$Q = 95 \text{ cfs}$$

10.2.2 SCS TP-149 Method

Subwatershed 1

Step 1: From Table 3.5 the soil group for the sandy-clay loam soil is C. From Table 3.4 the CN for pasture in fair condition with no mechanical treatment is 79.

Step 2: From the charts in Appendix A, the 100-year, 24-hour duration storm gives 5.7 inches of rain.

Step 3: From charts in Appendix B for steep terrain and curve numbers 75 and 80, the Q is, respectively,

$$Q = 140 \text{ cfs} \qquad \text{and} \qquad Q = 185 \text{ cfs}$$

Using linear interpolation, Q for a CN = 79 is

$$Q = 172 \text{ cfs}$$

Subwatershed 2

Step 1: An area weighted curve number is required for this drainage, similar to the area weighted C for the Rational Formula. Using a CN of 79 for the reclaimed strip bench and 73 for the undisturbed watershed (woods in fair condition), the composite CN is

$$CN = \frac{79\ (5)\ +\ 73\ (11)}{16}$$

$$CN = 75$$

Step 2: The 100-year, 24-hour event is again 5.7 inches.

Step 3: From charts in Appendix B for steep terrain and curve number 75, the Q is 62 cfs.

10.2.3 Design Values

Using an average (rounded to the nearest 5 cfs) of the estimates by the two methods, the recommended design discharges are given in Table 10.1.

10.3 Diversion Channel Designs

10.3.1 Channel A

The design flow for Channels A and B together is 180 cfs. Assuming the flow is equally divided between each channel, the design flow for each channel is 90 cfs. From the topographic map given in Figure 10.1, the longitudinal profile of the channel was evaluated (Figure 10.2). For stations 0+00 to 7+80

Table 10.1. Recommended Design Discharges.

Method	Subwatershed 1	Subwatershed 2
Rational Formula	183	95
SCS TP-149	172	62
Design Value	180	80

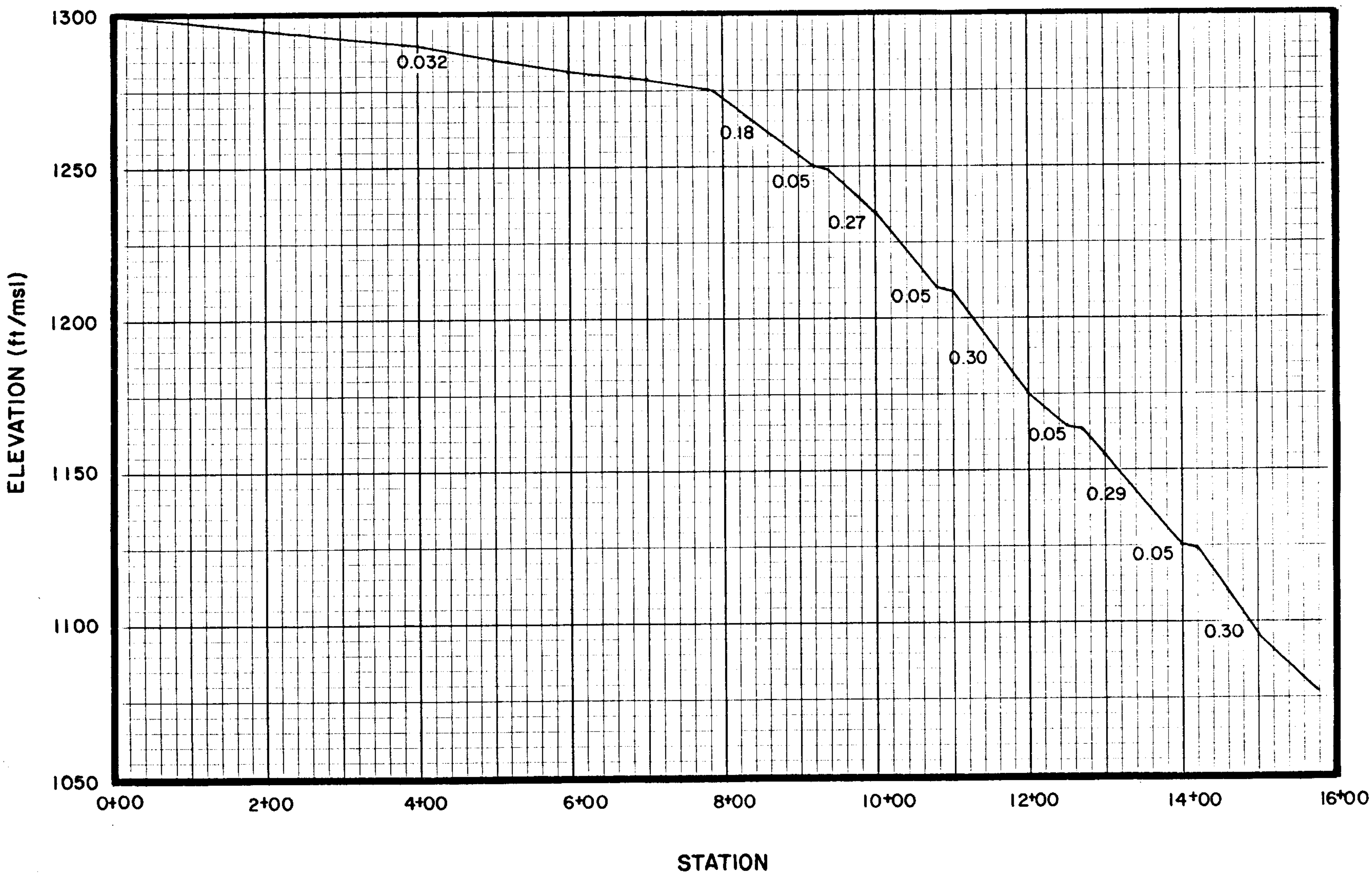


Figure 10.2. Longitudinal profile of diversion channel A.

the average slope is 0.032. As an initial assumption, assume the slope is mild. Since the channel is intermittent, a vegetative lining could be utilized. However, the material the channel will be constructed in cannot be considered erosion resistant, and a low permissible velocity would have to be used in design (Table 6.2). For the relatively large design discharge in this channel, this would result in a very wide, shallow cross section. Therefore, it is more practical to use rock riprap. The mild slope riprap design procedures can be used (Chapter VI).

For stations 7+80 to 15+80, the steep slopes require that only riprap be used for stabilization. The steep slope riprap design procedure given in Chapter V should be used.

<u>Channel Design Station 0+00 to 7+80</u>

1. Assume $K_m = 9$ in. From Equation 4.18,

$$n = 0.0395\ (0.75)^{1/6} = 0.038$$

2. From the charts in Appendix C for $Qn = 90\ (0.038) = 3.4$, a slope 0.032 and a bottom width of 6 ft,

$$Vn = 0.27 \text{ fps} \qquad V = Vn/n = 7.1 \text{ fps}$$

$$d = 1.5 \text{ ft, and}$$

$$R = \frac{A}{P} = \frac{6(1.5) + 2(1.5)^2}{6 + 2(1.5)(2^2+1)^{0.5}} = 1.06 \text{ ft}$$

3. $$\frac{V^2}{R^{0.33}} = \frac{6.8^2}{1.06^{0.33}} = 49$$

Therefore, from Table 6.4 the required riprap is Type L.

4. For Type L the K_m size is 9 in., therefore the initial assumption is OK.

5. No iteration is required.

6. $$F_r = \frac{7.1}{\sqrt{32.2\ (1.5)}} = 1.0$$

Since the Froude number is greater than 0.8, the mild slope assumption for a six-foot bottom width channel was not valid. Two options are now available. The bottom width can be increased to reduce the velocity, and consequently the Froude number, or the six-foot bottom width can be retained and the steep slope design procedure used.

Try the steep slope procedure for a six-foot bottom width.

a. The design flow remains 90 cfs.

b. Bed slope = 0.032.

c. As discussed above, use a trapezoidal channel with six-foot bottom width and 2:1 side slopes.

d. From Figure 5.4 using 0.05 slope,

$D_{50} = 0.95$ ft use 1.0 ft.

$d = 0.96$ ft.

This D_{50} value compares favorably with the mild slope design result and substantiates the use of the steep slope procedure for Froude numbers greater than 0.8.

e. Gradation

$$D_{max} = 1.25\ D_{50}$$

$$= 1.25 \text{ ft.}$$

$$D_{10} = \frac{D_{50}}{3} = 0.33 \text{ ft}$$

Riprap thickness = $1.25\ D_{50} = 1.25$ ft.

7. Filter Evaluation

a. The assumed base material gradation is

$D_{85} = 0.27$ in.

$D_{50} = 0.10$ in.

$D_{15} = 0.036$ in.

The riprap charateristics are determined graphically by plotting the recommended gradation on semilog paper (Figure 10.3).

$D_{85} = 14$ in.

$D_{50} = 12$ in.

$D_{15} = 4$ in.

b. Using Army Corps of Engineers and U.S. Bureau of Reclamation filter design criteria (Equations 5.2 and 5.3):

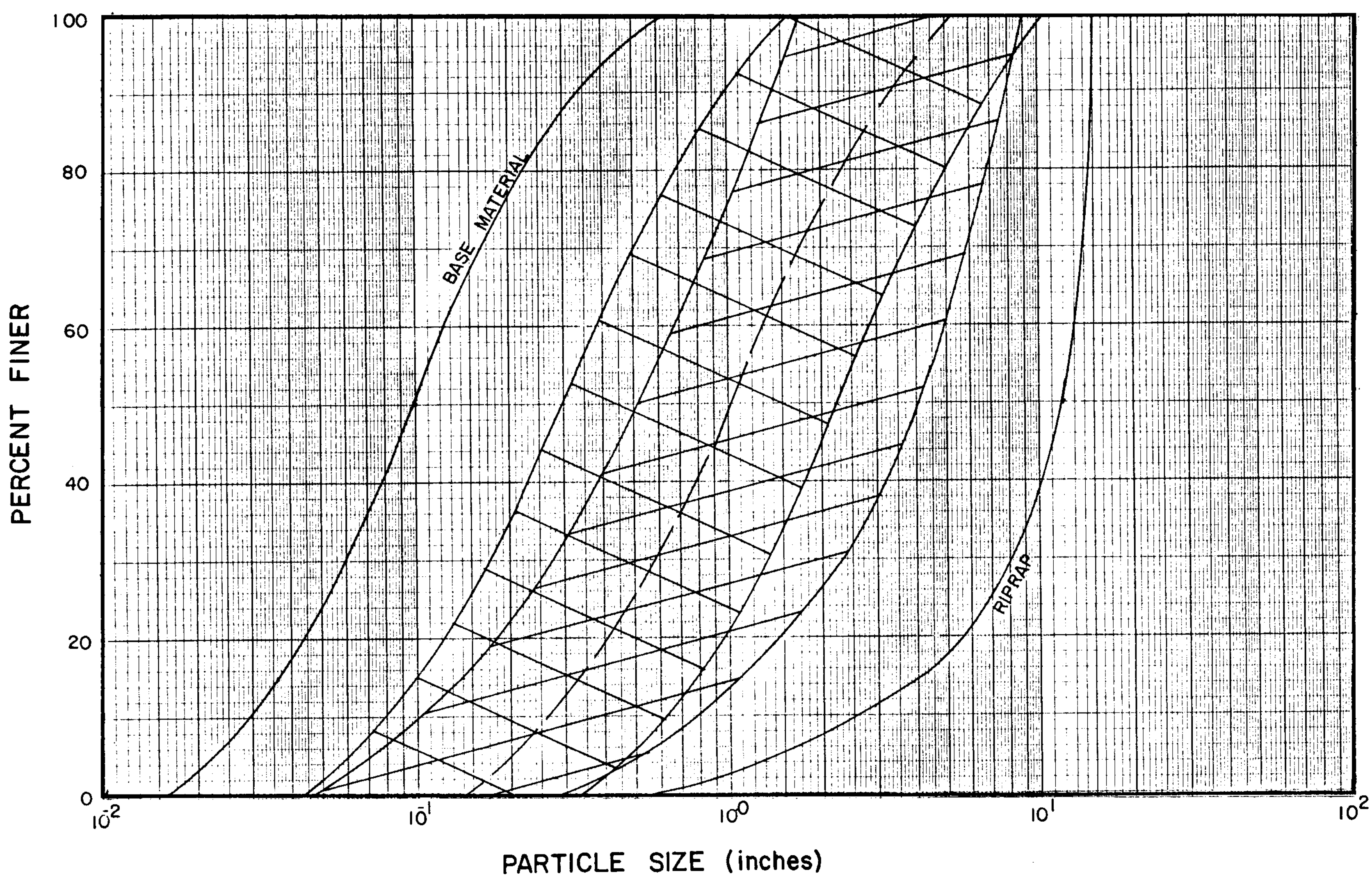


Figure 10.3. Particle size characteristic for diversion channel A from stations 0+00 to 7+80.

$$\frac{D_{15}\ (\text{riprap})}{D_{85}\ (\text{base})} = \frac{4}{0.27} = 15;\ 15 > 5$$

$$\frac{D_{15}\ (\text{riprap})}{D_{15}\ (\text{base})} = \frac{4}{0.036} = 111;\ 111 > 40$$

Therefore, a filter is necessary.

c. Properties of the filter relative to the base material:

$$\frac{D_{50}\ (\text{filter})}{D_{50}\ (\text{base})} < 40, \text{ so } D_{50}\ (\text{filter}) < 40(0.10) = 4.0 \text{ in.}$$

$$\frac{D_{15}\ (\text{filter})}{D_{15}\ (\text{base})} < 40, \text{ so } D_{15}\ (\text{filter}) < 40(0.036) = 1.4 \text{ in.}$$

$$\frac{D_{15}\ (\text{filter})}{D_{85}\ (\text{base})} < 5, \text{ so } D_{15}\ (\text{filter}) < 4(0.27) = 1.1 \text{ in.}$$

$$\frac{D_{15}\ (\text{filter})}{D_{15}\ (\text{base})} > 5, \text{ so } D_{15}\ (\text{filter}) > 4(0.036) = 0.14 \text{ in.}$$

Therefore, with respect to the base material, the filter must satisfy

$0.14 \text{ in.} < D_{15}\ (\text{filter}) < 1.1 \text{ in.}$

$D_{50}\ (\text{filter}) < 4.0 \text{ in.}$

Considering the riprap and filter:

$$\frac{D_{50}\ (\text{riprap})}{D_{50}\ (\text{filter})} < 40, \text{ so } D_{50}\ (\text{filter}) > \frac{12}{40} = 0.30 \text{ in.}$$

$$\frac{D_{15}\ (\text{riprap})}{D_{15}\ (\text{filter})} < 40, \text{ so } D_{15}\ (\text{filter}) > \frac{4}{40} = 0.10 \text{ in.}$$

$$\frac{D_{15}\ (\text{riprap})}{D_{85}\ (\text{filter})} < 5, \text{ so } D_{85}\ (\text{filter}) > \frac{4}{5} = 0.8 \text{ in.}$$

$$\frac{D_{15}\ (\text{riprap})}{D_{15}\ (\text{filter})} > 5, \text{ so } D_{15}\ (\text{filter}) < \frac{4}{5} = 0.8 \text{ in.}$$

Therefore, with respect to the riprap layer, the filter must satisfy

0.10 in. $< D_{15}$ (filter) < 0.8 in.

D_{50} (filter) > 0.30 in.

D_{85} (filter) > 0.8 in.

The limits of the filter material with respect to both the riprap and base material are shown on Figure 10.3. The selected size distribution of the filter is indicated by the dashed line. The filter thickness should be equal to D_{max} (filter) but not less than 6-9 in. Therefore, use 9 in. as thickness.

8. Freeboard

$$F.B. = c_{fb}d + \frac{1}{2}\Delta Z \qquad \text{(Equation 4.20)}$$

Since the channel has a relatively short bend between Station 4+00 and 6+00, superelevation must be included in the freeboard calculation. From the topographic map, the radius of curvature (r_c) is estimated as 80' and $r_o - r_c$ (equal to the top width of the channel) is 12.0 ft. Froude number

$$V = \frac{Q}{A} = \frac{90}{6(0.96) + 3(0.96^2)} = 10.5 \text{ fps}$$

$$F_r = 10.5/\sqrt{32.2\ (0.96)} = 1.9$$

The freeboard coefficient c_{fb} from Table 4.4 is 1.0.

$$c_{fd}(d) = 1.0(0.96) = 0.96 \text{ ft} < 1.0 \text{ ft}; \quad \text{use } 1.0 \text{ ft}$$

$$F.B. = 1.0 + \frac{1}{2g}\,\frac{10.5^2}{80}(12.0) = 1.25 \text{ ft} \qquad (4.20)$$

The final channel dimensions are b = 6 ft and channel depth = 2.2 ft. Figure 10.4 illustrates the channel dimensions and excavation amounts required to accommodate the riprap and filter layer.

<u>Channel Design for Station 7+80 to 15+80</u>

Steep Slope Riprap Design

1. The design flow, as determined above, is 90 cfs.

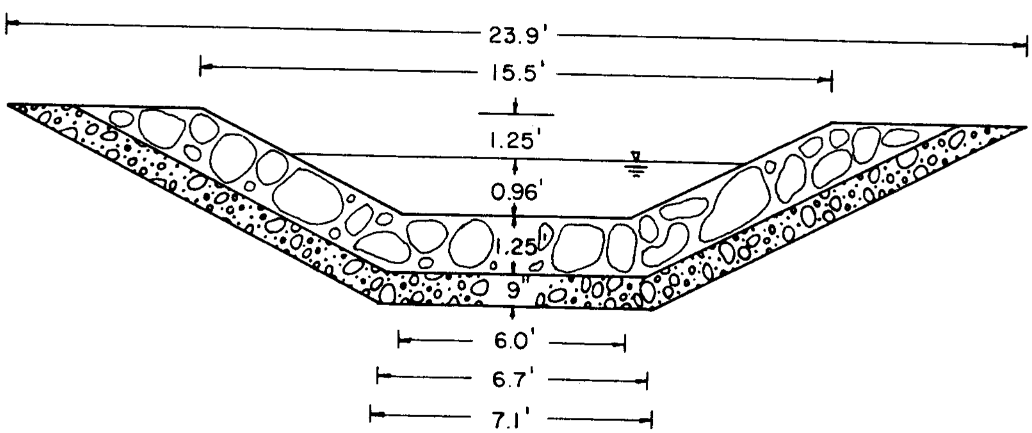


Figure 10.4. Diversion channel A dimensions from station 0+00 to 7+80.

2. The bed slope selected for design is based on an examination of Figure 10.2. The channel must collect runoff from each bench and therefore, must follow slope of fill. Using the steepest section, the bed slope is determined to be 0.30.

3. Channel size and shape. Try a trapezoidal channel with a 2:1 side slope and 6 foot base width.

4. From Figure 5.4, $D_{50} = 2.1$ ft, use $D_{50} = 2.25$ ft.

$$d = 0.4 \text{ ft.}$$

5. Gradation of riprap is

$$D_{max} = 1.25\ D_{50}$$

$$= 2.8 \text{ ft.}$$

$$D_{10} = \frac{D_{50}}{3} = 0.75 \text{ ft}$$

Riprap thickness = 1.25 D_{50} = 2.8 ft.

6. Granular filter design

a. The assumed base material gradation is

$D_{85} = 0.27$ in.

$D_{50} = 0.10$ in.

$D_{15} = 0.036$ in.

The riprap properties are determined by plotting the recommended gradation on semilog paper (Figure 10.5).

$D_{85} = 33$ in.

$D_{50} = 27$ in.

$D_{15} = 12$ in.

b. Using the Army Corps of Engineers and U.S. Bureau of Reclamation filter design criteria,

$$\frac{D_{15}\ \text{(riprap)}}{D_{85}\ \text{(base)}} = \frac{12}{0.27} = 44\ ;\ 44 > 5$$

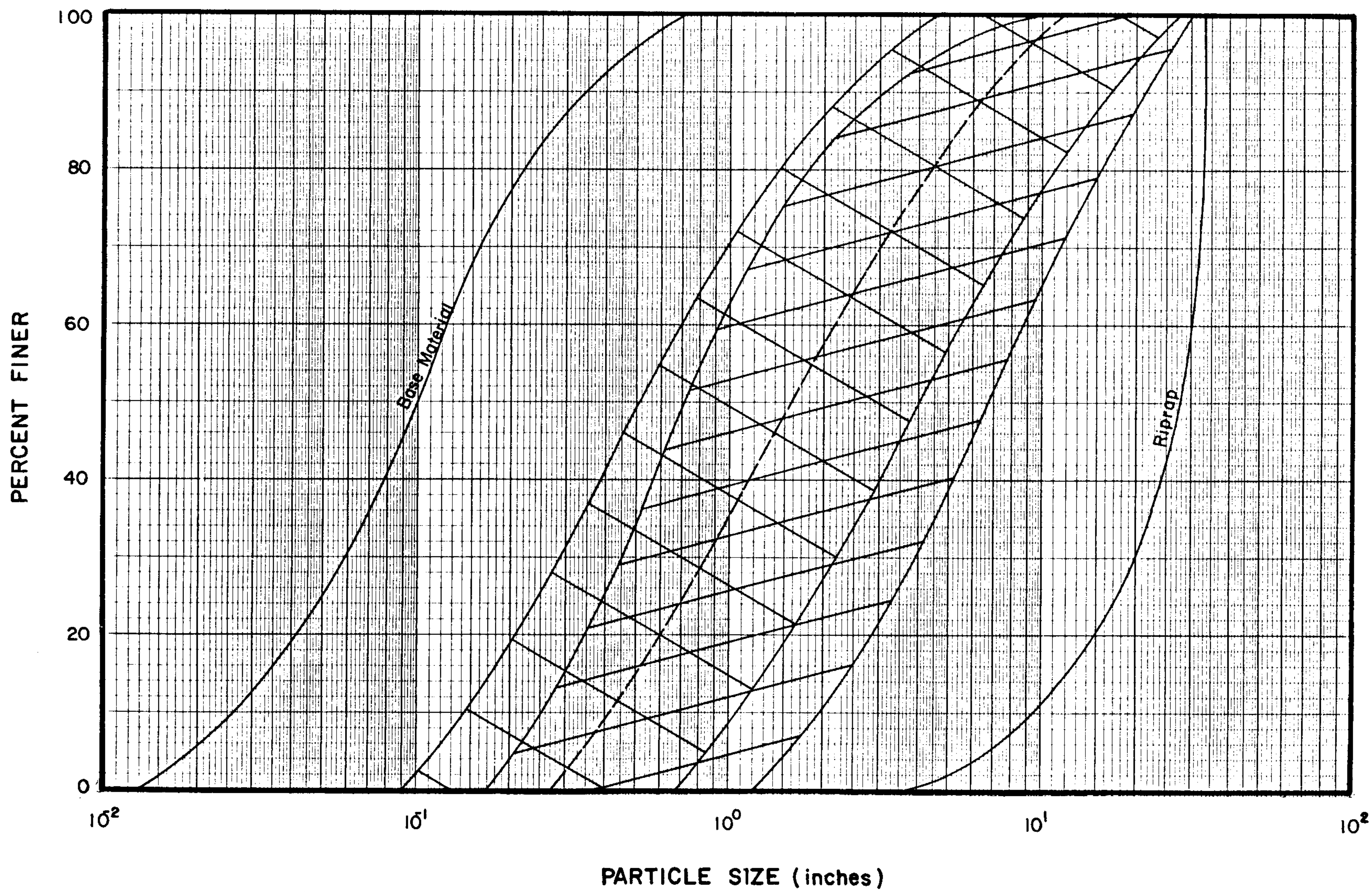


Figure 10.5. Particle size characteristics for diversion channel A stations 7+80 to 15+80.

$$\frac{D_{15}\text{ (riprap)}}{D_{15}\text{ (base)}} = \frac{12}{0.036} = 333 \; ; \; 333 > 40$$

Therefore, a filter layer is necessary.

c. Properties of the filter relative to the base material are:

$$\frac{D_{50}\text{ (filter)}}{D_{50}\text{ (base)}} < 40, \text{ so } D_{50}\text{ (filter)} < 40(0.1) = 4.0 \text{ in.}$$

$$\frac{D_{15}\text{ (filter)}}{D_{15}\text{ (base)}} < 40, \text{ so } D_{15}\text{ (filter)} < 40(0.036) = 1.4 \text{ in.}$$

$$\frac{D_{15}\text{ (filter)}}{D_{85}\text{ (base)}} < 5, \text{ so } D_{15}\text{ (filter)} < 5(0.27) = 1.3 \text{ in.}$$

$$\frac{D_{15}\text{ (filter)}}{D_{15}\text{ (base)}} > 5, \text{ so } D_{15}\text{ (filter)} > 5(0.036) = 0.18 \text{ in.}$$

With respect to the base material, the filter must satisfy

$0.18 \text{ in.} < D_{15}\text{ (filter)} < 1.3 \text{ in.}$

$D_{50}\text{ (filter)} < 4.0 \text{ in.}$

Considering the riprap and filter material

$$\frac{D_{50}\text{ (riprap)}}{D_{50}\text{ (filter)}} < 40, \text{ so } D_{50}\text{ (filter)} > \frac{27}{40} = 0.6 \text{ in.}$$

$$\frac{D_{15}\text{ (riprap)}}{D_{15}\text{ (filter)}} < 40, \text{ so } D_{15}\text{ (filter)} > \frac{12}{40} = 0.30 \text{ in.}$$

$$\frac{D_{15}\text{ (riprap)}}{D_{85}\text{ (filter)}} < 5, \text{ so } D_{85}\text{ (filter)} > \frac{12}{5} = 2.4 \text{ in.}$$

$$\frac{D_{15}\text{ (riprap)}}{D_{15}\text{ (filter)}} > 5, \text{ so } D_{15}\text{ (filter)} < \frac{12}{5} = 2.4 \text{ in.}$$

With respect to the riprap layer, the filter must satisfy

$0.30 \text{ in} < D_{15}\text{ (filter)} < 2.4 \text{ in.}$

D_{50} (filter) > 0.68 in.

D_{85} (filter) > 2.4 in.

The limits of the filter material with respect to both the riprap and base material are shown in Figure 10.5. The selected size distribution of the filter material is indicated by the dashed line. Thickness of the filter material is equal to the maximum size of the filter material if more than nine inches. In this case, thickness is equal to eleven inches.

7. Freeboard

The riprap size necessary for stability is determined by considering the steepest reach of the channel. However, in the steepest channel reach the flow depth will be a minimum relative to the milder-sloped reaches. To ensure adequate freeboard and overall channel depth, the depth of flow must be evaluated for the mildest-sloped reach. For the steep channel design between stations 7+80 and 15+80, neglecting the short bench sections of channel, the mildest slope is 15%.

From Figure 5.4

$d = 0.7$ ft.

$c_{fb} = 1.0$ (Table 4.4)

$c_{fb}(d) = 1.0(0.7) = 0.7 < 1.0$; use 1.0 ft.

$$F.B. = c_{fb}(d) + \frac{1}{2}\Delta Z = 1.0 + 0 = 1.0 \text{ ft.} \qquad \text{(Equation 4.20)}$$

The constructed channel section dimensions are therefore $b = 6$ ft. and total channel depth = F.B. + d = 1.7 ft. Figure 10.6 illustrates the final channel dimensions.

10.3.2 Diversion Channel B

As determined above, the design flow for Channel B is 90 cfs. From the topographic map (Figure 10.1), the longitudinal profile was evaluated and plotted in Figure 10.7. For stations 0+00 to 7+80 the average slope is 0.044 and from 7+80 to 15+60, the maximum slope is 33 percent. Therefore, based on the results of Diversion Channel A, assume both slopes are steep.

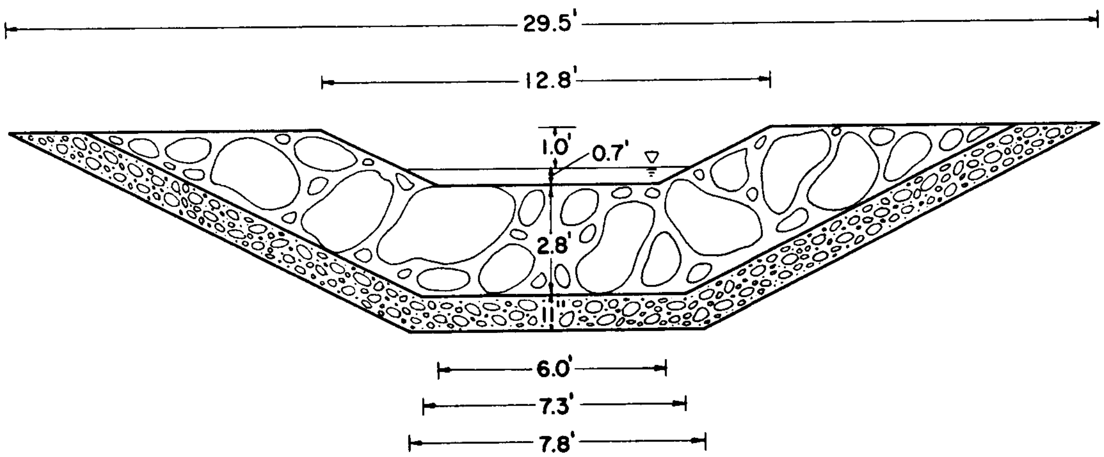


Figure 10.6. Diversion channel A dimensions from station 7+80 to 15+80.

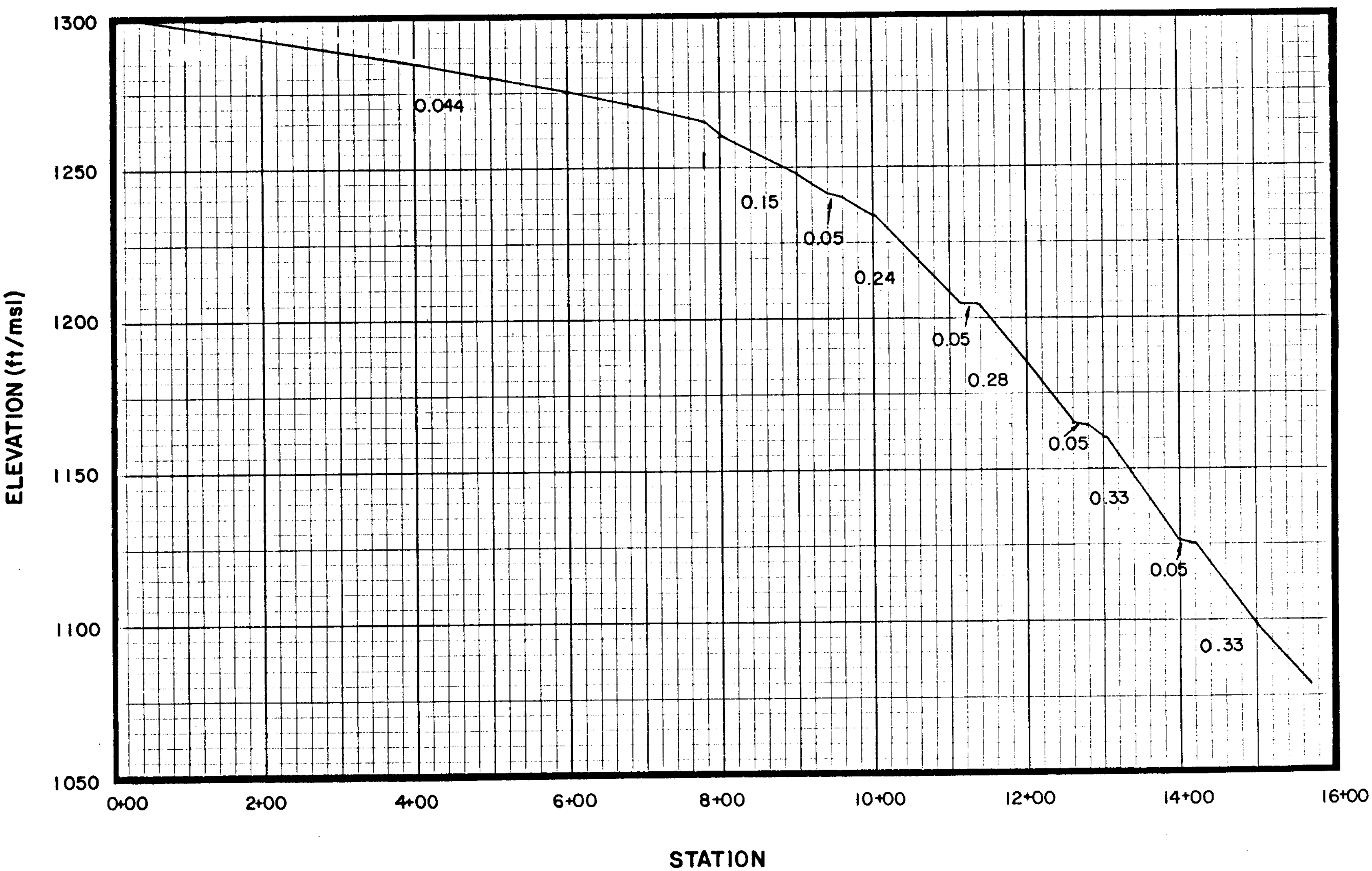


Figure 10.7. Longitudinal profile for diversion channel B.

Station 0+00 to 7+80

Since the design discharge is the same as Channel A and the 0.05 slope line will again be used on Figure 5.4, the riprap design will be identical to Channel A station 0+00 to 7+80. Also due to the similarity in alignment, the freeboard is nearly the same. It is adequate to use the same cross section for Channel B, station 0+00 to 7+80 as Channel A, station 0+00 to 7+80 (Figure 10.4).

Channel Design for Station 7+80 to 15+60

<u>Steep Slope Riprap Design</u>

1. The design flow is 90 cfs.
2. The bed slope selected for design is based on an examination of Figure 10.7. Using the steepest section, the bed slope is determined to be 0.33.
3. Channel size and slope. Try a trapezoidal channel with 2:1 side slope and 6 foot base width.
4. From Figure 5.4, $D_{50} = 2.2$ ft; use $D_{50} = 2.25$ ft.

   $d = 0.3$ ft.

5. Gradation of riprap is

   $D_{max} = 1.25\ D_{50} = 2.8$ ft.

   $D_{10} = \frac{D_{50}}{3} = 0.75$ ft

   Riprap thickness $= 1.25\ D_{50} = 2.8$ ft.

6. Granular filter design. Since the base material and riprap are the same for the steep sections of both Channel A and Channel B, the granular filter remains the same. The limits of the filter material with respect to the riprap and base material are shown on Figure 10.5. The selected size distribution of the filter material is indicated by the dashed line. Thickness of the filter material is eleven inches.
7. Freeboard

   $C_{fb} = 1.0$ (Table 4.4)

   $C_{fb}(d) = 1.0(0.3) = 0.3 < 1.0$; use 1.0 ft.

$$F.B. = C_{fb}(d) + \frac{1}{2}\Delta Z = 1 + 0 = 1.0 \text{ ft.}$$

The channel design will be $b = 6$ ft and channel depth = 1.3 ft. Comparing this with the steep slope design of Channel A reveals a difference of 0.10 feet in design depth. This small difference in depth does not justify a new channel design. Therefore use the same channel as previously illustrated in Figure 10.6.

10.3.3 Diversion Channel C

Diversion Channel C collects runoff from both Channels A and B and therefore must be designed for 180 cfs. From the topographic map (Figure 10.1) the longitudinal profile was determined (Figure 10.8). The slope conditions require the steep slope design procedure.

1. The design flow is 180 cfs, as determined above.
2. The bed slope selected for design is based on examination of Figure 10.8. The amount of excavation required to design a uniform slope channel is prohibitive. Therefore, for ease in construction, a single channel cross section will be designed that is adequate for each section. Designing for riprap stability using the steepest section, the bed slope is determined to be 0.18, therefore use steep slope procedure.
3. The bottom widths of Channels A and B were equal to 6 ft. Therefore, try a bottom width of 10 ft for Channel C. Use 2:1 side slopes.
4. From Figure 5.5, $D_{50} = 1.65$ ft, use 1.75 ft.

   $d = 0.8$ ft

5. Gradation of the riprap is

   $D_{max} = 1.25(1.75) = 2.2$ ft

   $D_{10} = 1.75/3 = 0.6$ ft

   riprap thickness $1.25\ (D_{max}) = 2.2$ ft

6. Filter gradation design.

   a. The assumed base material gradation is

   $D_{85} = 0.27$ in.

   $D_{50} = 0.10$ in.

   $D_{15} = 0.036$ in.

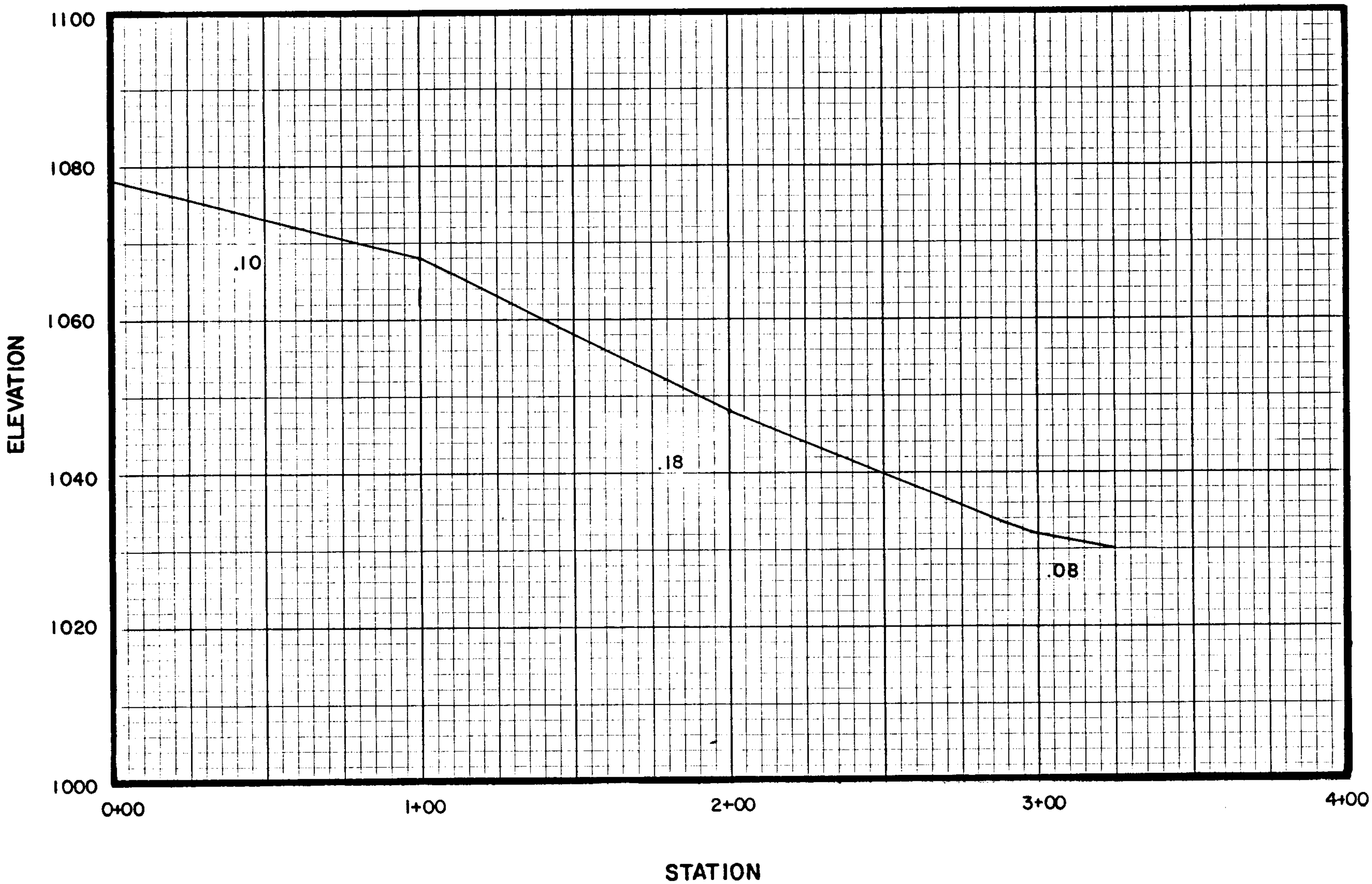


Figure 10.8. Longitudinal profile of diversion channel C.

The riprap characteristics are determined graphically by plotting the recommended gradation on semi-log paper (Figure 10.9).

$D_{85} = 26.0$ in.

$D_{50} = 21$ in.

$D_{15} = 8.5$ in.

b. Using the Army Corps of Engineers and U.S. Bureau of Reclamation filter design criteria (Equations 5.2 and 5.3)

$$\frac{D_{15}(\text{riprap})}{D_{85}(\text{base})} = \frac{8.5}{0.27} = 31 \ ; \ 31 > 5$$

$$\frac{D_{15}(\text{riprap})}{D_{15}(\text{base})} = \frac{8.5}{0.036} = 236 \ ; \ 236 > 40$$

Therefore, a filter is necessary.

c. Properties of the filter relative to the base material are:

$$\frac{D_{50}(\text{filter})}{D_{50}(\text{base})} < 40 \ , \text{ so } \ D_{50}(\text{filter}) < 40(0.1) = 4.0 \text{ in.}$$

$$\frac{D_{15}(\text{filter})}{D_{15}(\text{base})} < 40 \ , \text{ so } \ D_{15}(\text{filter}) < 40(0.036) = 1.4 \text{ in.}$$

$$\frac{D_{15}(\text{filter})}{D_{85}(\text{base})} < 5 \ , \text{ so } \ D_{15}(\text{filter}) < 5(0.27) = 1.3 \text{ in.}$$

$$\frac{D_{15}(\text{filter})}{D_{15}(\text{base})} > 5 \ , \text{ so } \ D_{15}(\text{filter}) < 5(0.030) = 0.18 \text{ in.}$$

With respect to the base material, the filter must satisfy

$0.18 < D_{15}(\text{filter}) < 1.3$ in.

$D_{50}(\text{filter}) < 4.0$ in.

Considering the riprap and filter material

$$\frac{D_{50}(\text{riprap})}{D_{50}(\text{filter})} < 40 \ , \text{ so } D_{50}(\text{filter}) > \frac{21}{40} = 0.52 \text{ in.}$$

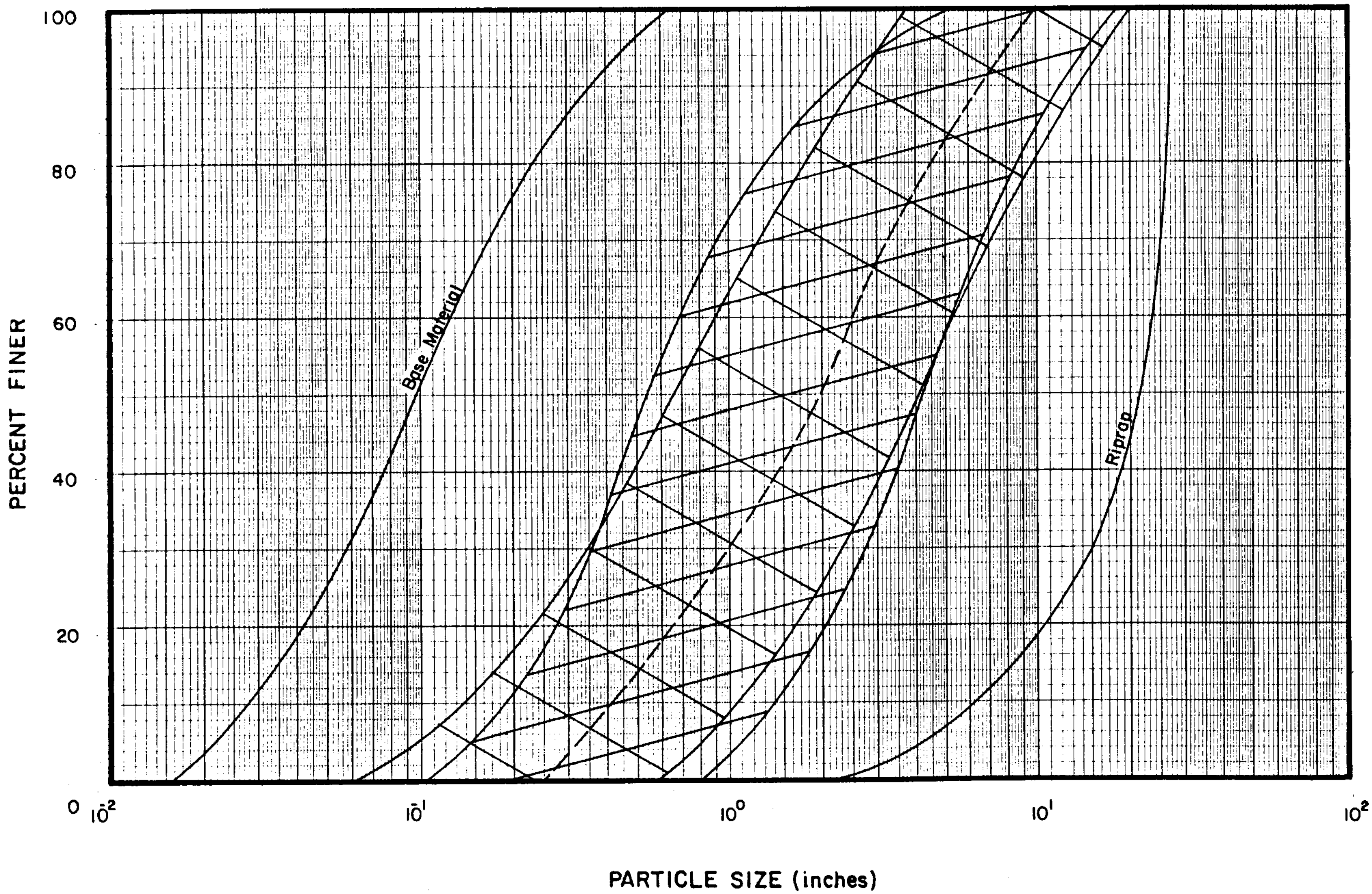


Figure 10.9. Particle size characteristics for diversion channel C.

$$\frac{D_{15}(\text{riprap})}{D_{15}(\text{filter})} < 40 \text{ , so } D_{15}(\text{filter}) > \frac{8.5}{40} = 0.21 \text{ in.}$$

$$\frac{D_{15}(\text{riprap})}{D_{85}(\text{filter})} < 5 \text{ , so } D_{85}(\text{filter}) > \frac{8.5}{5} = 1.7 \text{ in.}$$

$$\frac{D_{15}(\text{riprap})}{D_{15}(\text{filter})} < 5 \text{ , so } D_{15}(\text{filter}) < \frac{8.5}{5} = 1.7 \text{ in.}$$

With respect to the riprap layer, the filter must satisfy

0.21 in. $< D_{15}(\text{filter}) < 1.7$ in.

$D_{50}(\text{filter}) > 0.52$ in.

$D_{85}(\text{filter}) > 1.7$ in.

The limits of the filter material with respect to both the riprap and base material are shown in Figure 10.9. The selected size distribution of the filter material is indicated by the dashed line. The thickness of the filter material is equal to the maximum size of the filter, is more than nine inches. In this case, thickness is equal to 10 inches.

7. Freeboard.

The riprap size was determined in Step 4 by considering the critical flow conditions occurring in the steepest (S = 0.18) reach of Channel C. To evaluate freeboard and total channel depth required, the milder sloping reach from Station 0+00 to 1+00 with a slope of 10 percent is considered. The 8 percent channel reach starting at Station 3+00 is neglected because it is only 25 ft long.

Using $S = 0.10$

$Q = 180$ cfs

Figure 5.5 indicates

$d = 1.1$ ft.

From Table 4.4 $C_{fb} = 1.0$

$C_{fb}(d) = 1.0(1.1) = 1.1 > 1.0$ ft. ; use 1.1 ft.

$$F.B. = C_{fb}(d) + \frac{1}{2}\Delta Z = 1.1 + 0 = 1.1 \text{ ft.}$$

The final channel dimensions as shown in Figure 10.10 are $b = 10$ ft., total channel depth = 2.2 ft.

10.3.4 Diversion Channel D

Diversion Channel D is a reconstructed channel over a reclaimed strip bench. From the topographic map the channel extends from Stations 1+85 to 6+70. The profile of the waterway is shown in Figure 10.11. The slope conditions require utilizing the steep slope riprap design procedure.

1. The design flow as determined in Section 10.2 is 80 cfs.
2. From Figure 10.11 the bed slope is 0.24.
3. From consideration of upstream and downstream natural waterways, a channel bottom width of 6 ft is selected with a 2:1 side slope.
4. From Figure 5.1, $D_{50} = 1.75$ ft.

   $d = 0.46$ ft.

5. Gradation of riprap is

   $D_{max} = 1.25(1.75) = 2.2$ ft

   $D_{10} = 1.75/3 = 0.60$ ft.

   Riprap thickness $1.25\ D_{50} = 2.2$ ft.

6. Filter design. The riprap has the same particle size as Channel C. The particle size of the base material is also assumed nearly the same. Therefore, the filter requirement for channel D is identical to the filter designed for Channel C.
7. Freeboard

   From Table 4.4 $C_{fb} = 1.0$

   $C_{fb}(d) = 1.0(0.46) = 0.46 < 1.0$; use 1.0 ft.

   $$F.B. = C_{fb}(d) + \frac{1}{2}\Delta Z = 1.0 + 0 = 1.0 \text{ ft.}$$

The final channel dimensions are given in Figure 10.12.

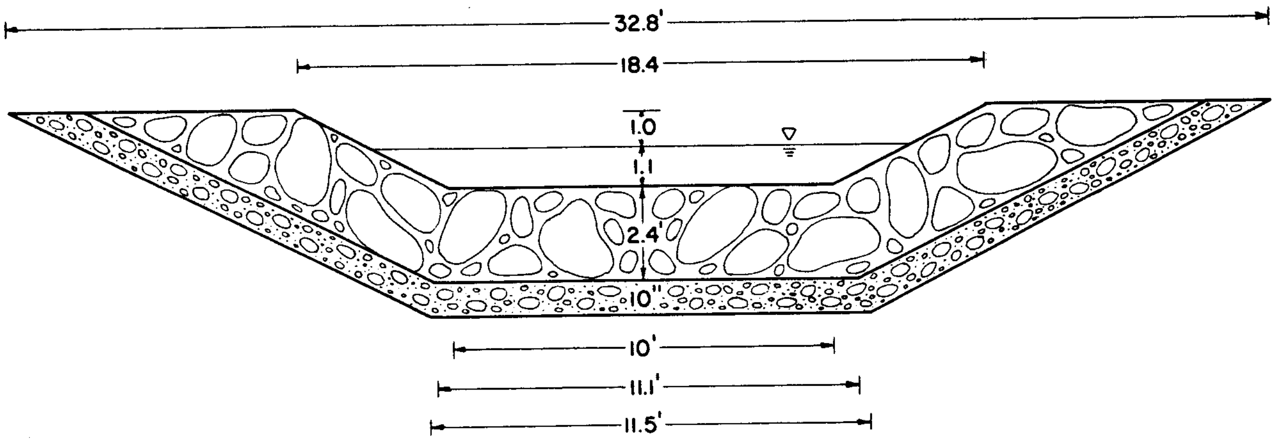


Figure 10.10. Diversion channel C dimensions.

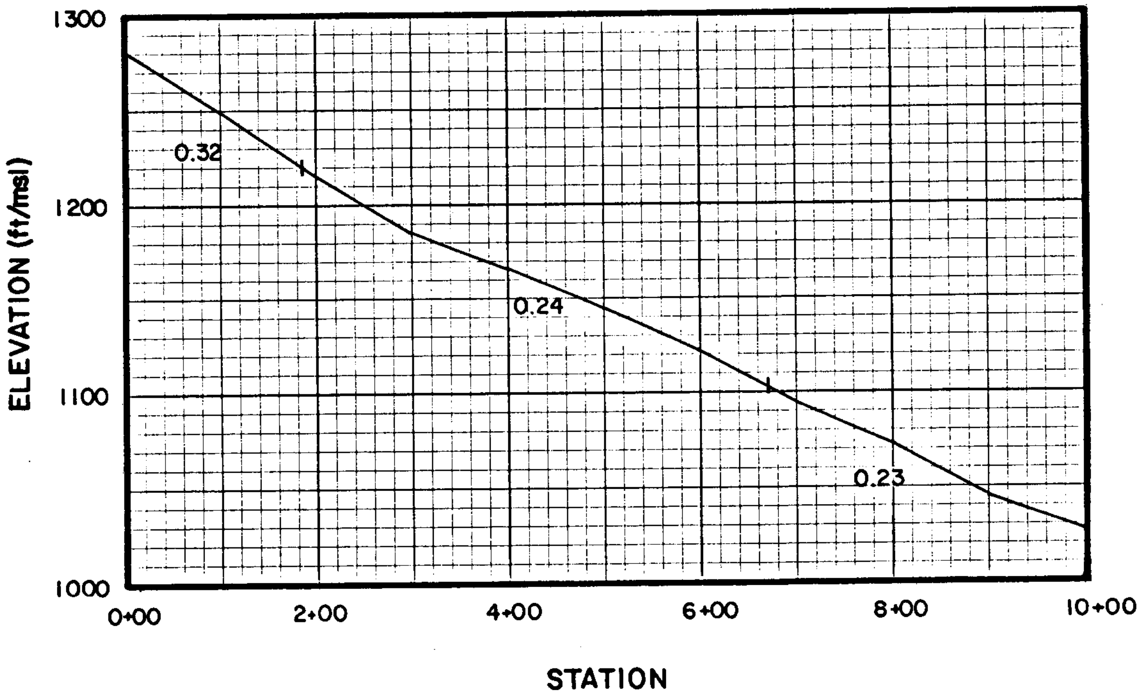


Figure 10.11. Longitudinal profile of diversion channel D.

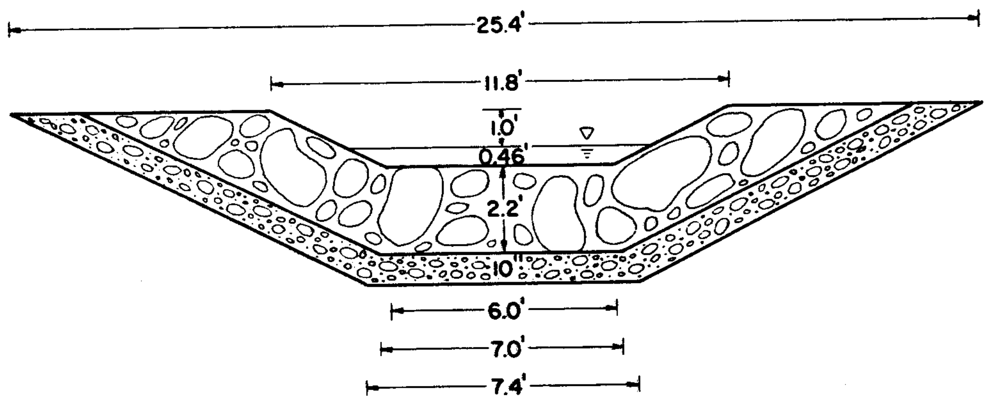


Figure 10.12. Diversion channel D dimensions.

10.4 Transition Design

10.4.1 Channels A and B

In both Channels A and B, transition design is required for the entrance to and exit from the steep slope channel. The transition length should be protected with the same riprap size used on the steep slope. The transition length is computed from the guidelines given in Section 5.4.

The downstream depth of flow, used to evaluate the length of protection needed at the change in grade, is computed for the 10 percent channel reach between Stations 0+00 and 1+00 in Channel C. From Figure 5.2 for $Q = 180$ cfs and $S = 0.10$

$$d \cong 1.1 \text{ ft.}$$

$$\text{Length of protection} = 5d = 5.5 \text{ ft.}$$

Since 5.5 ft < 15 ft, the minimum protection of 15 feet is required. The riprap size calculated for the steep slope sections of Channels A and B should be extended for a distance of 15 ft in Channel C (i.e., from Station 0+00 to 0+15). In this example the riprap size in both steep channels sections of Channels A and B is identical. If the rock size was different, protection below the chanel in grade should be provided by using the larger riprap gradation. Protection should also be provided at the entrance sections from mild to steep slopes in Channels A and B. Steep slope riprap protection should be extended 15 ft above the steep slope sections. There will be no other significant transition problems since the cross section geometry (6 ft bottom width and 2:1 side slope) is the same for the steep and mild channels.

10.4.2 Channels C and D

The entrance protection for Channel C has already been established by the exit protection required for Channels A and B. The exit of Channel C is to the existing natural waterway downstream of the junction of the natural waterway containing diversion Channel D. It is recommended that the riprap of Channel C be extended through the junction to insure the stability of this area. The entrance of the natural waterway containing diversion Channel D should also be riprapped in this region.

The bottom width of Channel D was chosen to approximate the natural waterway upstream and downstream. Although the change in roughness may cause a greater depth in the riprapped diversion channel than in the natural waterway, the steep slope conditions will minimize this effect. Therefore, considering these factors, no transition problems are anticipated in Channel D.

XI. INTRODUCTION

11.1 Purpose of Part 2

The design of water diversion channels for surface mine operations in sandy soil regions presents unique problems not encountered at mine sites with more well developed soils. In this context, sandy is used in the engineering sense to describe loose, cohesionless soils. Sandy soil conditions typically exist in coal mining regions of the semiarid western states.

In contrast, the more humid environment of the eastern state surface mines produces a more stable topography with well developed soils and vegetation and natural stream beds of gravel cobble substrate. The highly erodible sandy soil conditions require special consideration and design procedures for diversion channels to avoid problems of bank stability, lateral migration, aggradation and degradation. Design techniques must necessarily involve sediment transport principles to account for the movable boundary conditions.

Design procedures applicable to regions with well developed soils and gravel cobble type streams were presented in Part 1 of this Design Manual. Although Part 1 was prepared primarily for application in the Eastern Coal Province, many of the basic concepts and design procedures are sufficiently general to allow more widespread application. For example, even in sandy soil regions some channels may be constructed in bedrock or on cobbled spoils material where resistance to flow is high. However, due to the significant differences in overall stream morphology between sandy soil regions and regions with well developed soils, certain methodologies recommended in Part 1 are not applicable to sandy soil conditions. Therefore, to develop nationwide guidelines for water diversion channel design on surface mine operations, Part 2 was developed by SLA as a supplement to Part 1. The two parts are intended to be used together in developing water diversion designs on sandy soils.

11.2 Special Problems of Sandy Soils

11.2.1 Sediment Yield

Sandy soil conditions typically exist in the semi-arid and arid regions of the country. In these regions the vegetation and landforms reflect the lack of water. Compared with more humid regions, topogaphy is more abrupt and hill slopes are usually steeper and shorter. The soils are thinner with little organic content, and bedrock exposures are common.

Dryland streams are usually incised channels and are likely to be dry for long periods of time. When the channels do flow, it is usually in response to small storm cells of limited areal extent producing high intensity, short-duration storms. These storms can generate excessive erosion in upland watersheds. Due to high drainage density (number of channels per unit area), water and sediment runoff occurs very efficiently. Peak discharge is high and the time to peak and flow duration are short. Therefore, normal sediment yields in dryland areas are large due to highly erodible soils, well developed drainage networks and widely ranging climatic characteristics.

11.2.2 Stream Response

The combination of large sediment yields from upland watersheds and highly erodible stream channels can cause rapid changes in stream channel configuration. These changes result from lateral migration, degradation and aggradation and include changes in stream form, bedform, flow resistance and other geometric and hydraulic characteristics. The dynamic conditions that exist require that local problems and their solutions be considered in terms of the entire system. Natural and man-induced changes in any channel, particularly a highly erodible sandy soil channel, initiate responses that can be propagated for long distances both upstream and downstream (Simons and Senturk, 1977).

11.2.3 Stable Channel Design

Designing a stable alluvial channel (one without a channel lining) or a stable lined channel under the dynamic conditions described above requires an understanding of sediment transport and stream channel response. For example, unlined channels must be designed to minimize excessive scour while lined channels must be designed to prevent deposition of sediments. Channel linings in dryland areas are typically some type of riprap due to the difficulties in growing the required type of vegetation. Unlined channels are most successful when designed under the concept of dynamic equilibrium, which simply allows for sediment transport conditions. These topics and others are presented in detail in the following chapters.

11.3 Organization

Information presented in Part 2 pertains primarily to the design of water diversion channels in sandy soils. Basic information and design guidelines needed from Part 1 are referenced by chapter or section number. For example, the hydrologic design and ecological considerations were given in Part 1 and are not repeated herein but cited by reference.

Information presented in Part 2 includes the specialized design considerations and procedures necessary for water diversion design in sandy soils. This includes some additional hydraulic considerations (Chapter XII), and alluvial channel concepts (Chapter XIII). Chapters XIV to XVII provide the specialized design information and Chapter XVIII is additional rock durability information pertinent to western mine regions. The comprehensive example in Chapter XIX illustrates the use of the recommended procedures. Other examples are given in each chapter. Users of the manual are encouraged to review these examples to better understand the design procedures.

This page intentionally left blank.

XII. OPEN CHANNEL FLOW CONCEPTS FOR SANDY SOIL DIVERSIONS

12.1 Introduction

Basic concepts of open channel flow were presented in Chapter IV; however, in sandy soil diversion channels some additional considerations are necessary. For example, in sand bed channels the bed and bank material is easily eroded and continually being moved, shaped and reworked by the flow. The development of bed forms by this process increases the resistance to flow by the effects of form roughness. This effect must be accounted for in designing diversion channels in sandy soils. This chapter provides the basic hydraulic concepts pertinent to sandy soil diversions that are necessary for application of design procedures given in the following chapters.

12.2 Selection of Channel Shape

In natural sand-bed channels, the most frequently observed channel shape is trapezoidal. Based on this observation, the sand-bed channels designed throughout this report will assume a trapezoidal shape. The angles of repose for material composed largely of sands is presented in Table 12.1. For conservative design a side slope z of 3 is recommended for all unlined sand-bed channels.

12.3 Normal Depth Calculation

The Manning Equation, defined in Section 4.4, will again be used to determine the normal depth. Charts given in Appendix C provide solution of the Manning equation for trapezoidal channels of 2:1 side slope and bottom widths of 2, 4, 6, 8, 10, 12, and 14 feet. Based on the relatively small angle of repose for sands, a side slope of 2:1 represents the maximum side slope for design. Commonly, sand-bed channels are designed with 2.5:1 and 3:1 side slopes. Thus, for side slopes other than 2:1, the charts in Appendix C of the Design Manual cannot be used to directly solve for the normal depth, but can be used to provide an initial estimate of the normal depth in an iterative procedure for solving the Manning equation. The iterative procedure will be illustrated by example at the end of this chapter.

12.4 Resistance to Flow

The bed of a channel composed of sandy soils seldom forms a smooth regular boundary, but is characterized instead by shifting forms that vary in

Table 12.1. Angles of Repose for Sand Classifications.

Classification	Angle of Repose (degrees)	z
Uniform fine to medium sand	26 to 30	2 1.75
Well-graded sand	30 to 34	1.75 1.50
Sand and gravel	32 to 36	1.60 1.40

size, shape, and location. These bed forms are influenced by changes in flow, temperature, sediment load, and other variables. Additionally, they constitute a major part of the resistance to flow exhibited by a sand-bed channel, and exert a significant influence on flow parameters such as depth, velocity and sediment transport. Understanding the different types of bed forms that can occur and the resistance to flow and sediment transport associated with each bed form will aid in the hydraulic design of sand-bed channels.

12.4.1 Bed Configuration, Flow Phenomena, and Resistance to Flow

The bed configuration is the array of bed forms, or absence thereof, generated on the bed of an alluvial channel by the flow. The bed configurations that may form in an alluvial channel are plane bed without sediment movement, ripples, dunes, plane bed with sediment movement, antidunes, and chutes and pools. These bed configurations are listed in their order of occurrence with increasing values of stream power for bed materials having D_{50} less than 0.6 mm. Stream power is defined as the product of the bed shear stress, τ_0, and the average velocity, V. Sketches of typical bed configurations are shown in Figure 12.1.

12.4.1.1 Plane Bed Without Sediment Movement

If the bed material of a stream moves at one discharge but not at a smaller discharge, the bed configuration at the smaller discharge will exist as a remnant of the bed configuration formed when sediment was moving. For bed configuration without sediment movement, the problem of resistance to flow is one of rigid-boundary hydraulics. Plane bed without sediment movement has been studied to determine the flow conditions for the beginning of motion and the bed profiles that would form after beginning of motion. For a plane sand bed without bed-material transport, Manning's roughness coefficient varies from 0.012 to 0.016.

12.4.1.2 Ripples

In fine sand, ripples will usually occur as soon as particles show appreciable movement. The separation zone downstream from a ripple causes very little disturbance on the water surface. The concentation of sediment is small, ranging from 10 to 200 ppm. Resistance to flow is large, with

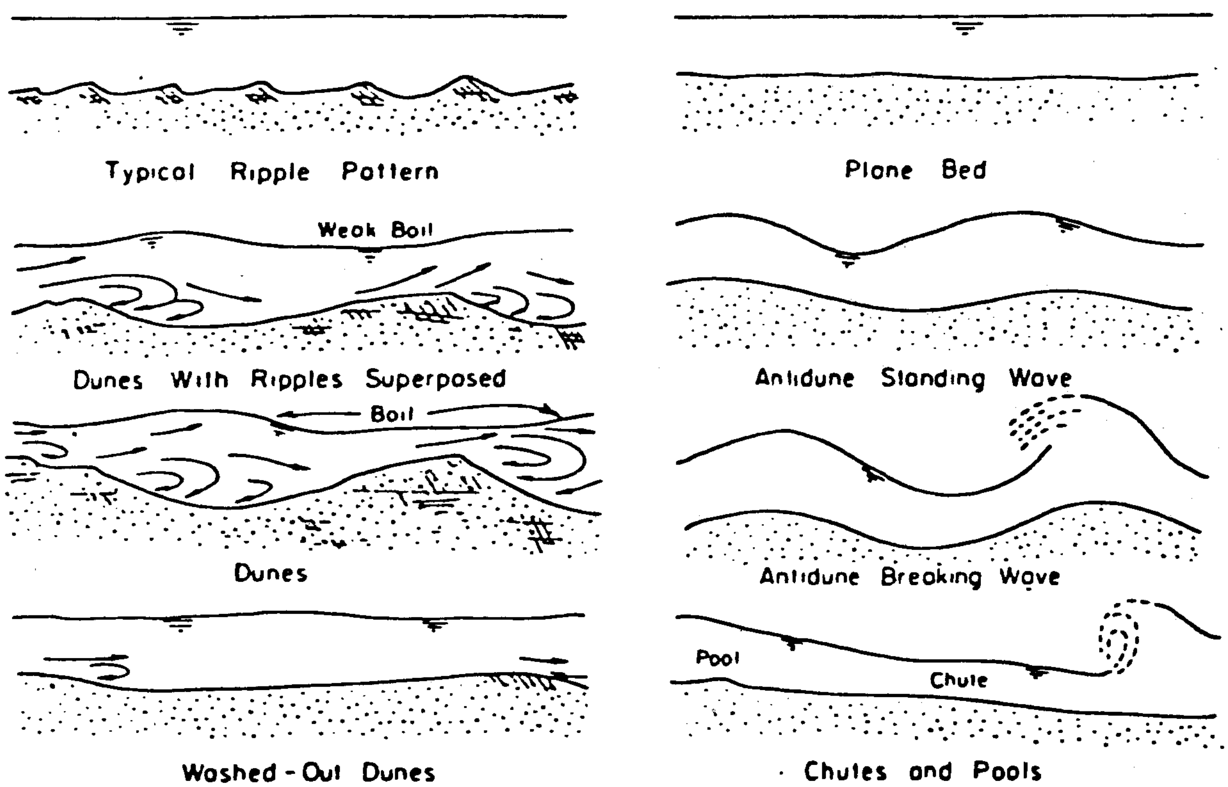


Figure 12.1. Forms of bed roughness in sand channels.

Manning's n varying from 0.018 to 0.05. As the depth of the flow increases, resistance to flow due to bed roughness decreases.

12.4.1.3 Dunes

When the shear stress and the stream power are gradually increased over a bed of ripples, or if the bed material is coarser than 0.6 mm over a plane bed, a new flow condition will be achieved that causes dunes to form. Dunes cause large separation zones in the flow. With dunes, as with tranquil flow over an obstruction, the water surface is always out of phase with the bed surface. The flow accelerates over the crest of the dunes and decelerates over the troughs. The sediment transport rate is relatively small. The concentration of bed materials is approximately 100 to 1200 ppm. Resistance to flow caused by dunes is large, but not as large as that caused by ripples formed of finer sand and at shallow depth. For dunes, Mannings's n varies from 0.018 to 0.035. Resistance to flow increases with an increase in depth for coarser sands ($D_{50} > 0.3$ mm) and decreases with an increase in depth for finer sands ($D_{50} \leq 0.3$ mm). This is because the amplitude of dunes can increase with the increasing depth so that the relative roughness can remain essentially constant or even increase with increasing depth of flow. However, a decrease in angularity of the dunes in finer sands as they increase in size reduces their form roughness and their resistance to flow as well.

12.4.1.4 Plane Bed With Sediment Movement

If the shear stress or the stream power is continuously increased, the size of dunes increases until the dunes reach a maximum height at a certain stream power. Thereafter, a transition regime is reached when the dunes start to diminish in amplitude with the further increase of the stream power. Finally, the dunes completely disappear and a flat bed is formed. In this case, the concentration of bed material ranges from about 1500 to 3000 ppm. Resistance to flow is relatively low, with the Manning's n varying from 0.014 to 0.022.

12.4.1.5 Antidunes

When the shear stress or the stream power is further increased, sand and water waves gradually build up from a plane bed and from a plane water surface. The waves may grow in height until they become unstable and break like the sea surf, or they may gradually subside and subsequently reform. The former have been called breaking antidunes, or antidunes, and the latter standing waves.

With antidune flow, the water and bed surface waves are in phase. This is a positive indication that the local flow is rapid (Froude number $\geq$ 1.0). Resistance to flow with antidunes depends on how often the antidunes form, the area of the reach they occupy, and the violence and frequency of their breaking. If the antidunes do not break, resistance to flow is about the same as for a plane bed with sediment movement. The Manning's n value varies from 0.012 to 0.028.

12.4.1.6 Chutes and Pools

At very steep slopes, sand-bed channel flow changes to chutes and pools. This type of flow consists of a long chute in which the flow accelerates rapidly, a hydraulic jump at the end of the chute, and then a long pool in which the flow is tranquil, but accelerating. Resistance to flow is large. Manning's n varies from 0.015 to 0.031.

12.4.2 Regime of Flow in Alluvial Channels

The flow in sand-bed channels is divided into two flow regimes with a transition zone between. Each of these two flow regimes is characterized by similarities in the shape of the bed configurations, mode of sediment transport, process of energy dissipation, and phase relation between the bed and water surfaces. The two regimes and their associated bed configurations are:

A. Lower flow regimes

   1. Ripples
   2. Dunes

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

C. Upper flow regimes

   1. Plane bed with sediment movement
   2. Antidunes

a. Standing waves
b. Breaking antidunes

3. Chutes and pools

A relationship was developed by Simons and Richardson (1966) that relates stream power (boundary shear stress x velocity, $\tau_o V$), median fall diameter of bed material, and form roughness (Figure 12.2). If the depth, slope, velocity, and fall diameter of bed materials are known, one can predict the form of bed roughness by using this relationship. The fall diameter is the diameter of a sphere that has a specific gravity of 2.65 and has the same terminal uniform settling velocity as the particle of interest. For practical purposes it is adequate to use the actual particle diameter.

12.4.2.1 Lower Flow Regimes

Lower flow regime begins with the beginning of motion. The resistance to flow is large and sediment transport small. The bed form is either ripples or dunes or some combination of the two. The water surface undulations, if they exist, are out of phase with the bed surface, and there is a relatively large separation zone downstream from the crest of each ripple or dune. Resistance to flow is caused mainly by form roughness.

12.4.2.2 Upper Flow Regimes

In the upper flow regime, resistance to flow is relatively small and sediment transport is large. The usual bed forms are plane bed or antidunes. The water surface is in phase with the bed surface except when an antidune breaks, and normally the fluid does not separate from the boundary. A small separation zone may exist downstream from the crest of an antidune prior to its breaking. Resistance to flow is the result of grain roughness with the grains moving, sand waves and their subsidence, and energy dissipation when the antidune breaks.

12.4.2.3 Transitions

The transition zone encompasses the bed forms that occur during the passage from lower regime to upper regime. The bed configuration in the transition zone is erratic. It may range from that typical of the lower flow regime to that typical of the upper flow regime, depending mainly on antece-

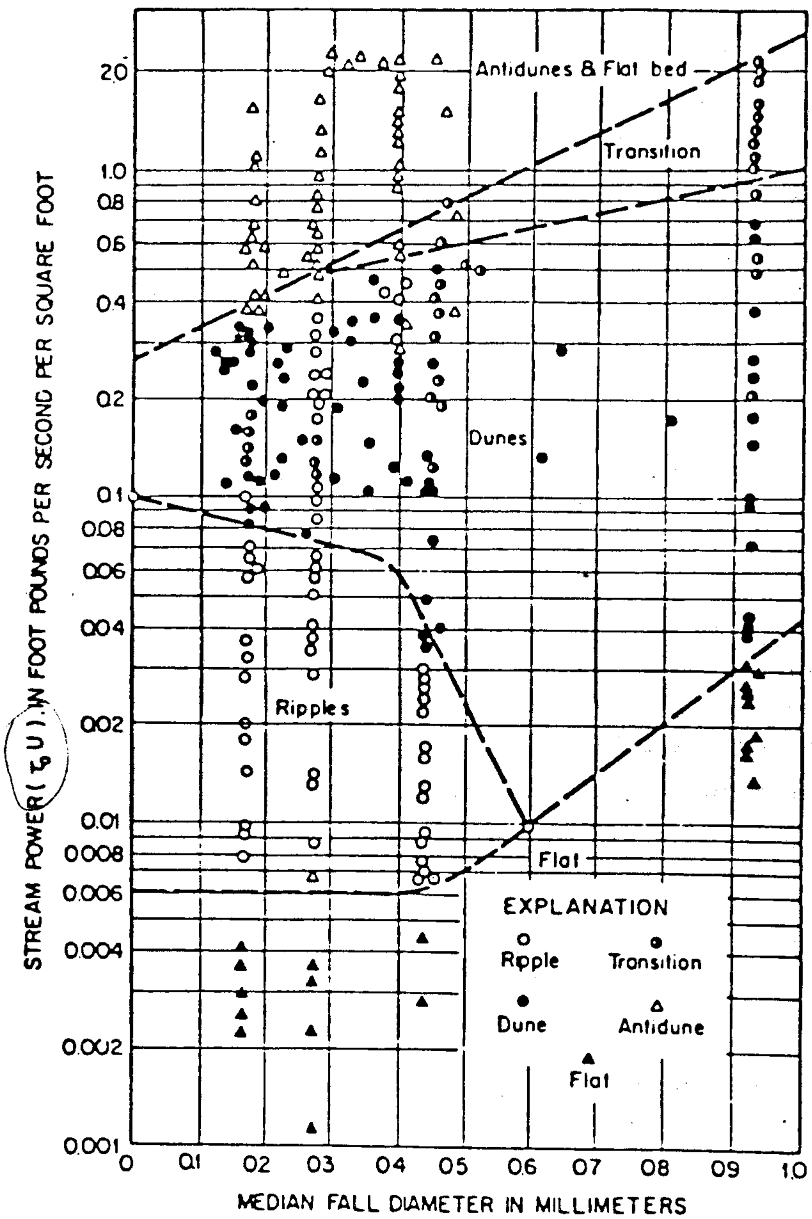


Figure 12.2. Relation of bed forms to stream power and median fall diameter of bed sediment (after Simons and Richardson, 1966).

dent conditions. The bed configuration may also oscillate between dunes and plane bed due to changes in resistance to flow, and consequently the changes in depth and slope as the bed form changes.

12.4.3 Recommended Values of Manning's n

For conservative estimation, it is recommended, within the range of Manning's roughness coefficients specified for each bed form, that lower roughness coefficients should be used for sediment transport analysis. Because of the relatively wide variation in Manning's coefficient considering all possible flow conditions and the full range of sand sizes, recommended values for design are given in Table 12.2.

12.5 Additional Depth Components Due to Bed Forms

In the design of sand-bed channels, the wave height due to an antidune should be considered for upper regime flows. The maximum wave height, due to the antidune before breaking, can be computed as:

$$h_a = 0.14 \frac{2\pi V^2}{g} \tag{12.1}$$

where h_a = the antidune wave height,

V = the mean flow velocity in the channel.

The antidune wave height should be less than or equal to the flow depth.

12.6 Superelevation

See Section 4.7.1.

12.7 Freeboard for Sand Bed Channels

The recommended freeboard for sand-bed diversion channels on a surface mine operation is

$$F.B. = c_{fb}\, d + \frac{1}{2}\Delta Z + \frac{1}{2} h_a \tag{12.2}$$

where c_{fb} is a coefficient defined according to Table 4.4. In all cases, the recommended minimum freeboard is 1.0 foot (Soil Conservation Service minimum) plus one-half superelevation and one-half antidune height (see Section 4.7.2).

Table 12.2. Values of Manning's Coefficient n for Design of Channels with Fine to Medium Sand Beds.

Bed Roughness	Manning's Coefficient n For Sediment Transport and Bank Stability
Ripples	0.018 - 0.022
Dunes	0.025 - 0.030
Transition	0.020 - 0.025
Plane Bed	0.015 - 0.020
Standing Waves	0.015 - 0.020
Antidunes	0.020 - 0.025

12.8 Evaluation of Channel for Reasonable Shape

Stable channel cross sections formed in sandy soils are usually wide and shallow because the fine particles cannot withstand high velocities, turbulence, and tractive forces. Stable channel designs using maximum permissible velocities are frequently utilized. These methods often result in large geometric sections. Regardless of the method for determining stable channel design, this type of cross section is clearly not desirable since the water would probably not flow uniformly across the entire width. Rather, it would concentrate in one area by scouring a new deeper, narrower channel within the limits of the broader channel, particularly for small discharges. Also, the excavation quantities required by a wide channel might be impractical and uneconomical. Consequently, consideration must be given to the computed channel dimensions to ensure they represent a practical design.

Lacey (1929) developed empirical formulas for designing stable sand-bed channels. His formulas for the hydraulic radius, R, and wetted perimeter, P,

$$R = 0.7305 \frac{V^2}{f} \tag{12.3}$$

$$P = 3.8 \frac{V^3}{0.7305f} \tag{12.4}$$

Dividing P by R reveals

$$\frac{P}{R} \cong 7 \times V \tag{12.5}$$

where P and R are expressed in feet and V in ft/sec.

This formula for the wetted perimeter-to-hydraulic radius ratio can provide guidance in assessing the practicality of a channel design. It should be noted, however, that the preceeding empirical formula is simply a guideline. This equation does not pertain to all conceivable flow conditions, nor does it differentiate between practical and impractical channel configurations. Channel designs having P/R ratios within the range 2.5 V to 14 V tend to reinforce the practicality of the design. Those designs falling well outside of this range should be evaluated further. It is often possible to improve the channel design by using a properly designed lining or installing grade control structures. These methods will be discussed in Chapters XV and XVI.

12.9 Examples

12.9.1 Iterative Solution of the Manning Equation

Design a trapezoidal sand-bed channel given the following data.

Design discharge, $Q = 150$ cfs.

$S = 0.001$

Bed and bank material $D_{50} = 0.5$ mm

1. Assume a channel base width of 6 feet and select a side slope of 3 based on information in Section 12.2.

2. Since the given D_{50} is in the range of Equation 4.18

$$n = 0.395\ D_{50}^{1/6}$$

$$n = 0.395\ \left(\frac{0.5\ \text{mm}}{305\ \text{mm/ft}}\right)^{1/6} = 0.013$$

3. From the channel chart in Appendix C with b = 6 ft, $Qn = 1.95$ and $S = 0.001$, the initial estimate for normal depth is

$$d = 2.6$$

With this value, solve the Manning equation by an iterative technique.

$$A = bd + zd^2$$

$$P = b + 2\sqrt{10}\ d$$

$$Q = \frac{1.49}{n} A R^{2/3} S^{1/2}$$

d	A	P	R	$R^{2/3}$	Q
2.6	35.9	22.4	1.6	1.37	178
2.5	33.75	21.8	1.55	1.34	163
2.4	31.7	21.2	1.50	1.31	150.5

Normal depth is equal to 2.4 feet.

12.9.2 Evaluation of Bed Form and Manning n Assumption

For the above channel evaluate the bed form for the design flow conditions.

1. Stream Power: Using the equation

$$\tau_o = \gamma RS$$

where τ_o is the boundary shear stress, γ is the specific weight of the water, R is the hydraulic radius and S is the slope, the stream power is

$$\tau_o V = \gamma RSV$$

$$V = \frac{Q}{A} = 4.75 \text{ ft/sec}$$

$$\tau_0 V = 62.4(1.5)(0.001)(4.75) = 0.44$$

From Figure 12.2, the bed form is dunes.

2. Check Manning n assumption from Table 12.2.

$$n = 0.030 \neq 0.013$$

Recompute normal depth based on n = 0.030, Qn = 4.5

From Appendix C

$$d = 4.0 \text{ ft}$$

d	A	P	R	$R^{2/3}$	Q
4.0	72	31.3	2.3	1.75	198
3.6	60.5	28.8	2.1	1.65	156
3.5	57.8	28.1	2.05	1.62	147

Use normal depth equal to 3.5 feet.

Check bed form.

$$V = \frac{Q}{A} = \frac{150}{57.8} = 2.6 \text{ ft/sec}$$

$$\tau_0 V = \gamma RSV = (62.4)(2.05)(0.001)(2.6) = 0.33$$

From Figure 12.2, the bed form is dunes and the solution is correct.

12.9.3 Evaluation of Superelevation and Freeboard

The channel is assumed to have a bend with a radius of curvature of 75 feet. The top width of the channel is

$$W = b + 2zd = 6 + 6(3.5) = 27 \text{ feet}$$

1. Superelevation is

$$\Delta Z = \frac{V^2 W}{g r_c} = \frac{(2.6)^2\ 27}{(32.2)(75)} = 0.08 \text{ feet} \qquad (4.19)$$

2. Freeboard for the channel is based on the Froude Number. It is

$$Fr = \frac{V}{\sqrt{gd}} = \frac{2.6}{\sqrt{32.2(3.5)}} = 0.24$$

Since the Froude number is less than 1, the flow is subcritical. From Table 4.4, c_{fb} is 0.2 and the freeboard for the straight reaches is

$$F.B. = c_{fb}d + \frac{1}{2}\Delta Z + \frac{1}{2}h_a \qquad (12.2)$$

$$c_{fb}d = 0.2(3.5) = 0.7 < 1.0 \text{ (minimum)}$$

Therefore use 1.0 foot minimum.

F.B. = 1 + 0 + 0 = 1.0 ft.

Freeboard for the channel bend is

$$F.B. = 1.0 + \frac{1}{2}\Delta Z + 0 = 1 + 0.04 = 1.04$$

Use freeboard equal to 1.1 feet.

12.9.4 Evaluation of Channel Shape

Evaluate channel for reasonable shape.

$$\frac{P}{R} = 7 \times V \qquad (12.5)$$

$$\frac{28.1}{2.05} = 7 \times (2.6) = 18.2$$

$$13.7 \cong 18.2$$

The range for the reasonable shape criteria is

$$(2.5V - 14V) = (6.5 - 36.4)$$

Therefore the channel shape is reasonable and need not be evaluated further. The channel design for the straight reach is shown in Figure 12.3.

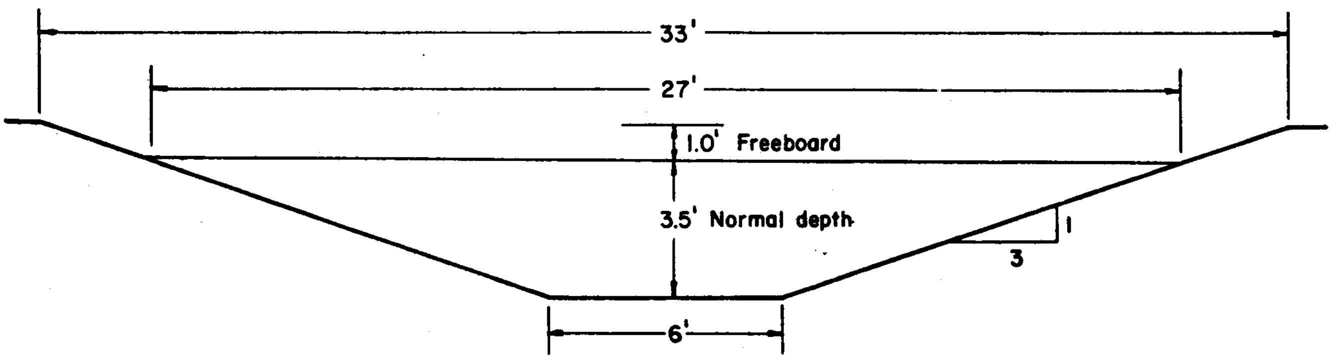


Figure 12.3. Channel design for straight reach.

12.10 References

Lacey, G., 1929, "Stable Channels in Alluvium," Proc. Inst. of Civil Engineers, 229.

Simons, D. B., and E. V. Richardson, 1966, "Resistance of Flow in Alluvial Channels," U.S. Geological Survey Professional Paper 422-J.

Soil Conservation Service, 1977, "Design of Open Channels," Technical Release No. 25.

XIII. SANDY SOIL ALLUVIAL CHANNEL CONCEPTS

13.1 General Sediment Transport Concepts

The amount of material transported or deposited in a channel reach is the result of the interaction of two processes, transport capacity and sediment supply. Transport capacity of the reach is determined by the hydraulic conditions (which are a direct result of water discharge, channel configuration and channel resistance) and the sediment size present. The supply of sediment entering a channel reach is determined by the nature of the channel and watershed above the reach, and any man-induced disturbance, such as surface mining activities, that it may be subject to. Both the supply rate and the transport capacity may limit the actual sediment transport rate in a given reach.

Sediment particles are transported by the flow in one or more of the following ways: (1) surface creep, (2) saltation, and (3) suspension. Surface creep is the rolling or sliding of particles along the bed. Saltation is the cycle of motion above the bed with resting periods on the bed. Suspension involves the sediment particle being supported by the water during its entire motion. Sediments transported by surface creep, sliding, rolling and saltation are referred to as bed load, and those transported by suspension are called suspended load. The suspended load consists of sands, silts, and clays. The bed-material load is the sum of bed load and suspended bed-material load.

The total sediment load in a channel is the sum of bed-material load and wash load. The bed-material load is that part of the total sediment discharge which is composed of grain sizes found in the bed. The wash load is that part composed of particle sizes finer than those found in appreciable quantities in the bed (Simons and Senturk, 1976). The presence of wash load can increase bank stability, reduce seepage and increase bed-material transport. Wash load can be easily transported in large quantities by the stream, but is usually limited by availability from the watershed and banks. The bed-material load is more difficult for the stream to move and is limited in quantity by the transport capacity of the channel.

Sediment transport equations are used to determine the sediment transport capacity for a specific set of flow conditions. Many formulas have been developed since DuBoys first presented his tractive force equation in 1879. The first step in evaluating sediment transport is to select one or more of the available equations for use in solving the given problem. The selection is not straightforward, since the results of different formulas can give drastically different results, and it is usually not possible to positively determine the one providing the best result. Additionally, some of the methods are considerably more complex than others. The initial consideration is to decide what portion of the sediment transport needs to be estimated. If it is desirable to know the contribution of the bed load and the suspended load to the bed-material discharge, formulas for each are available. Other formulas provide direct determination of the bed-material discharge. A common feature of all sediment transport equations is that the wash load is not included since it is governed by upstream supply.

A second consideration in deciding what formula(s) to use is the type of stream or river conditions that exists. It is important to select a formula that was developed under conditions similar to those of the given problem. For example, some formulas were developed from data collected in sand-bed streams where most of the sediment transport was suspended load, while other equations are based on conditions of predominantly bed-load transport.

In addition to the use of purely analytical or empirical formulas, there are methods available for evaluating the bed-material discharge based on measured suspended load and other normal stream flow measurements. By use of observed data these results are usually more accurate and reliable than those given by other formulas. Unfortunately, the measured data are often not available for the desired river location, or the data are not recent enough or of long enough duration to provide sufficient accuracy.

Special consideration must be given to sediment transport equations for sand-bed channels. These equations are based on the assumption that all the sediment sizes present can be moved by the flow. If this is not true, armoring (the development of a layer of coarse sized particles that protect the channel bed from erosion) will take place and the equations are no longer applicable. The sediment transport equations will be discussed in greater detail in Section 13.6.

13.2 Bed Material

Bed material is the sediment mixture of which the streambed is composed. For sand-bed channels, bed material may range from coarse sands and fine gravels to uniform fine to medium sands. The erodibility or stability of a channel largely depends on the size of particles in the bed. In alluvial channels composed of cohesive and some coarse materials, water flowing over the bed carries smaller particles away, while larger particles remain, armoring the bed. The armoring process usually does not occur in sand-bed channels. This is due to the lack of cohesion and uniform size distribution characteristic of the bed material. Therefore, in the absence of armoring the channel's sediment transporting capacity is considered equal to the incoming sediment supply. In this state, there is a constant exchange of material in the channel bed; however, there is no long-term aggradation or degradation of the bed and the channel slope remains unchanged. The slope at which the channel's sediment transporting capacity is equal to the incoming sediment supply is defined as the equilibrium channel slope. The equilibrium channel slope is usually established for the dominant discharge. This concept is discussed in more detail in Chapter XIV.

13.3 Bank Material

Bank material is generally made up of smaller or the same size particles as the bed. Thus, banks with the added gravitational force are often more easily eroded than the bed unless protected by some type of channel lining or bank protection measures.

Sand-bed channels experience varying degrees of flow, both along and through the bank. Forces that cause the movement of water through the bank material may be generated by several factors:

1. On the rising stage of the design hydrograph, an energy gradient develops, sloping from the channel into the bank material. On the falling stage of the design hydrograph, the energy gradient reverses direction and water moves through the bank toward the channel, decreasing the stability of the bank.

2. If the water table is higher than river stage, flow will be from the banks into the channel. The high water table may result from many conditions: (a) a wet period during which water draining from adjacent watersheds saturates the floodplain to a higher level; (b) poor drainage conditions resulting from deterioration or failure of drainage systems; and (c) increased infiltration resulting from changes in land use, causing an increase in water level.

The presence of water in the banks of sand-bed channels and its movement toward or away from the channel affect bank stability and bank erosion in various ways. The related erosion of banks may result as a consequence of seepage forces, piping, and mass wasting.

13.3.1 Seepage Forces

Seepage forces occur whenever there is inflow or outflow through the bed material and banks of a channel formed in permeable alluvium. The inflow or outflow through the interface between water and channel wall depends on the difference in pressure across the interface and the permeability of the bed and bank material. The seepage force acts to reduce or increase the effective weight and stability of the bed and bank materials depending on inflow or outflow. As a direct result of changing the effective weight, seepage forces can influence the form of bed roughness and the resistance to flow for a given channel slope, channel shape, bed material, and discharge.

13.3.2 Piping

Piping is a phenomenon common to the alluvial banks of sand-bed channels. For banks that are stratified with material of varying permeability, flow is induced in the more permeable layers by increase in stage. As the stage drops, the energy gradient is reversed and significant flow occurs toward the channel in the more permeable lenses. Interface between a overlying relatively permeable and underlying impermeable material can develop subsurface channels. Over time these channels enlarge and subsequently without the foundation material supporting the overlying layers, a block of bank material drops down. This also results in the development of tension cracks that may allow surface water to enter, further reducing the stability of the affected block of bank material. Bank erosion may continue on a grain-by-grain basis or the block of bank material may ultimately slide downward and outward into the channel, causing bank failure.

13.3.3 Mass Wasting

An alternate form of bank erosion is caused by local mass wasting. If the bank becomes saturated and possibly undercut by the flowing water, blocks of the bank may slump or slide into the channel. Mass wasting may be further aggravated by operation of equipment on the floodplain adjacent to the banks, added gravitational force resulting from trees, location of roads that may cause unfavorable drainage conditions, and increased infiltration of water into the floodplain as a result of changing land-use practices.

13.4 Lane's Relation

Lane's relation is most useful for a qualitative prediction of channel response to natural or imposed changes in a river system. Good engineering design must be based on a qualitative understanding of stream response to natural responses and man's activities. Considerations should be given not only to the local effects, but also upstream and downstream effects resulting from changes in the river system. Lane (1955) presented his relationship as

$$QS \propto Q_s D_{50} \tag{13.1a}$$

where Q is the water discharge, S is the channel slope, Q_s is the sediment discharge and D_{50} is the median diameter of the bed material. This equation was derived to include the effect of wash load by Simons et al. (1975). This relation states that

$$QS \propto \frac{Q_s D_{50}}{C_w} \tag{13.1b}$$

where C_w is the concentration of wash load. Application of the Lane relation was provided by example in Section 6.9.

13.5 Stream Form and Classification

Streams and rivers can be broadly classified in terms of channel patterns, that is, the configuration of the river as viewed on a map or from the air. Patterns include straight, meandering, and braided systems, or some combination of these patterns. These typical river channel patterns are shown in Figure 13.1. The characteristics of straight, meandering, and braided streams were presented in Section 6.3.2.

13.6 Slope-Discharge Relation

Because of the physical characteristics of straight, braided, and meandering streams, all natural channel patterns intergrade. Although braided and meandering patterns are strikingly different, they actually represent extremes in a continuum of channel patterns.

A number of studies have quantified this concept of a continuum of channel patterns. Lane (1957) investigated the relationship among slope, discharge, and channel pattern in meandering and braided streams and observed that an equation of the form

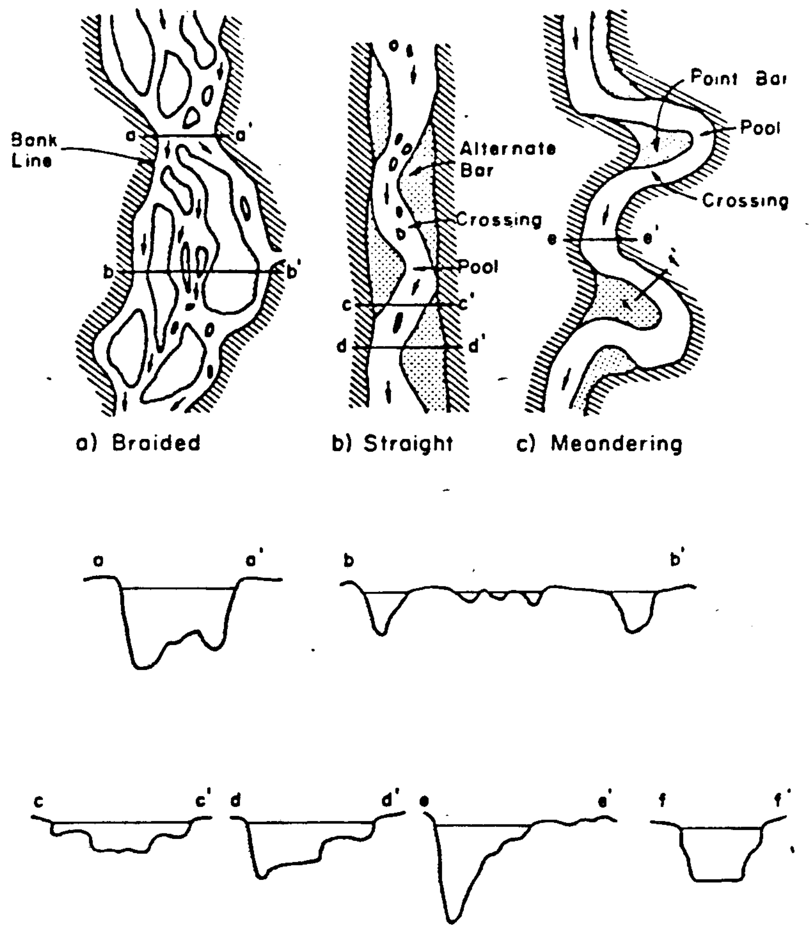


Figure 13.1. Stream and river channel patterns.

$$SQ^{1/4} = k \tag{13.2}$$

fits a large amount of data from meandering sand streams. Here, S is the slope of energy gradient, Q is the mean annual discharge which can be approximated by a two-year peak discharge, and k is a constant. Figure 13.2 summarizes Lane's plots and shows that if

$$SQ^{1/4} \leq 0.0017 \tag{13.3}$$

a sand-bed channel will normally exhibit a meandering pattern. Similarly, when

$$SQ^{1/4} \geq 0.01 \tag{13.4}$$

a river tends toward a braided pattern. The region between these values of $SQ^{1/4}$ can be considered a transitional range where streams are classified as intermediate or transitional.

The slope-dischage relation in Figure 13.2 can be used to analyze large-scale changes in stream form when these parameters are known. This figure illustrates the dependence of stream form on the channel slope and discharge and possible dramatic change in stream form if the SQ relationship borders one of the transitional regions.

The importance of Figure 13.2 is that vertical instability may lead to lateral instability. A diversion channel that increases the channel slope may change an otherwise meandering channel to a braided pattern. Bank protection measures may be needed should this occur. Many other effects can be analyzed by studying the figure.

13.7 Sediment Transport Equations

Many sediment transport formulas are available in the literature. In deciding what formula to use, consideration must be given to the type of stream or river conditions that exist. It is important to select a formula that was developed under conditions similar to those of the given problem. Considering these factors, the method presented below is recommended for application to sand-bed channels located in arid regions.

Power Relationships. This method is based on easy-to-apply power relationships that estimate the sediment transport rate based on the velocity of flow. The power relationships were developed from computer-generated data

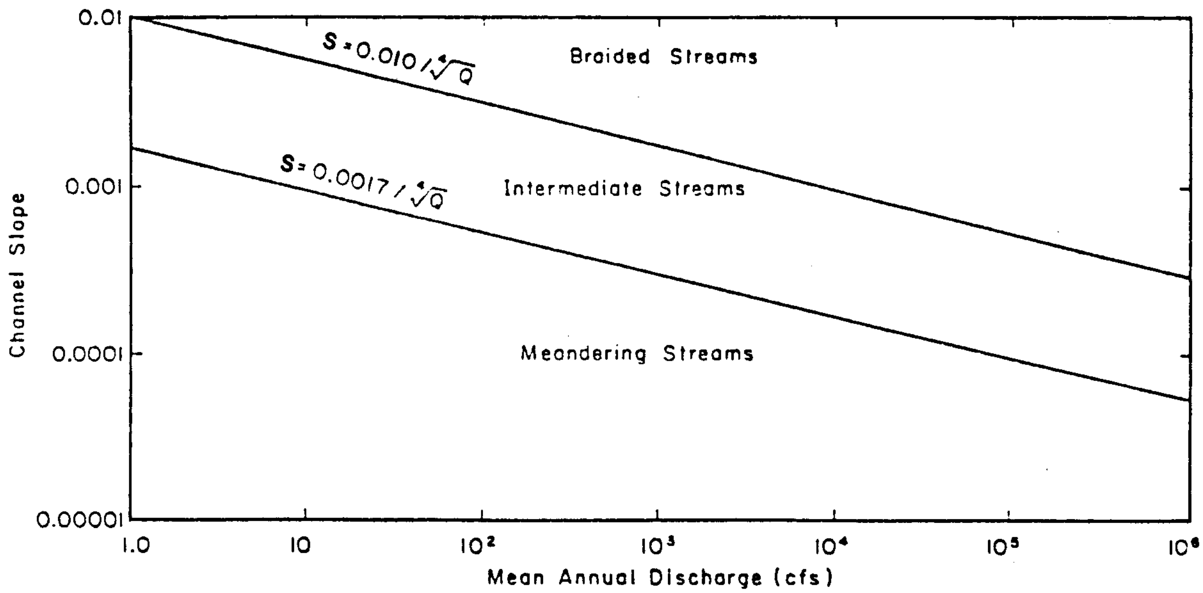


Figure 13.2. Slope-discharge relation for braiding or meandering in sand-bed streams (after Lane, 1957).

obtained from solution of the Meyer-Peter, Muller bed-load transport equation and Einstein's integration of the suspended bed-material discharge (Simons et al., 1981). The results of the sediment transport equations are presented in Figures 13.3 through 13.7. The values for the sediment transport rate obtained from the figures are in terms of cubic feet per second per foot of width. Therefore, these values should be multiplied by the top width of the water surface to obtain the total sediment transport rate.

When applying the information provided in Figures 13.3-13.7, a check should be made in order to insure the equations are applicable to a given problem. The equations are based on the assumption that all the sediment sizes present can be moved by the flow. If this is not true, armoring will take place and the equations are no longer applicable. Evaluation of the potential for armoring can be accomplished using Shields' critical shear stress criteria (see Section 6.3.5). The bed shear stress in uniform flow is given by

$$\tau_o = \gamma R S \tag{13.5}$$

in which γ is the unit weight of water, R is the hydraulic radius and S is the slope of energy gradient. The diameter of the largest particle moving is then

$$D_a = \tau_o/(\gamma_s - \gamma)\ 0.047 \tag{13.6}$$

in which D_a is the diameter of the sediment, γ_s is the unit weight of sediment and 0.047 is the recommended value of Shields' parameter. (All units are in feet, pounds and seconds.) If no sediment of the computed size or larger is present in significant quantities, the equations are applicable. Additionally, since the equations were developed for sand-bed channels, they do not apply to conditions when the bed material has cohesion. The equations would overpredict transport rates in a cohesive channel.

13.8 Design Procedure

1. Determine the channel cross section according to principles discussed in Chapter XII.
2. Knowing the flow velocity, V, and D_{50} of the bed material, obtain value for q_s from Figures 13.3-13.7.

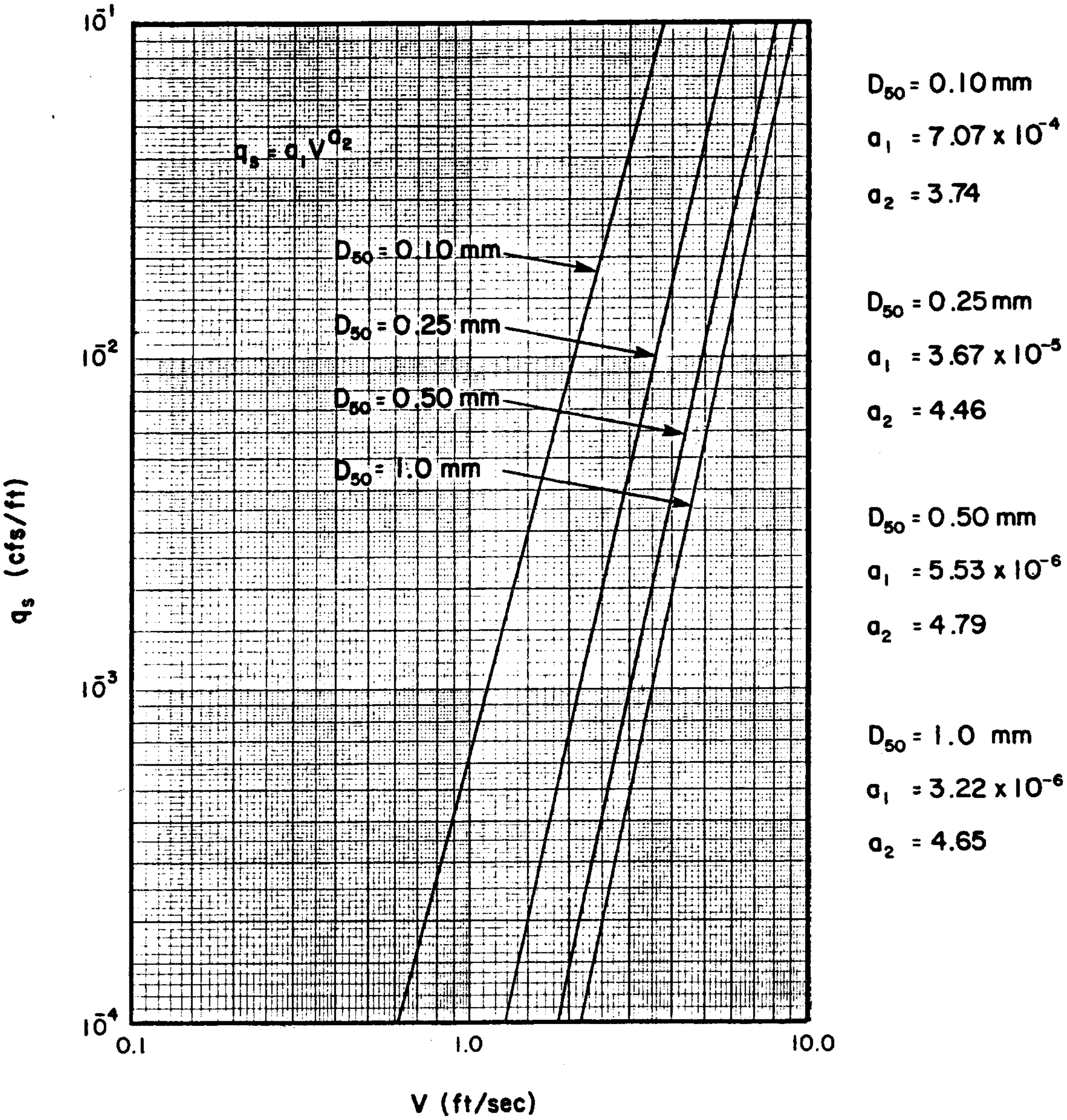


Figure 13.3. Values for sediment transport rate based on power relationships.

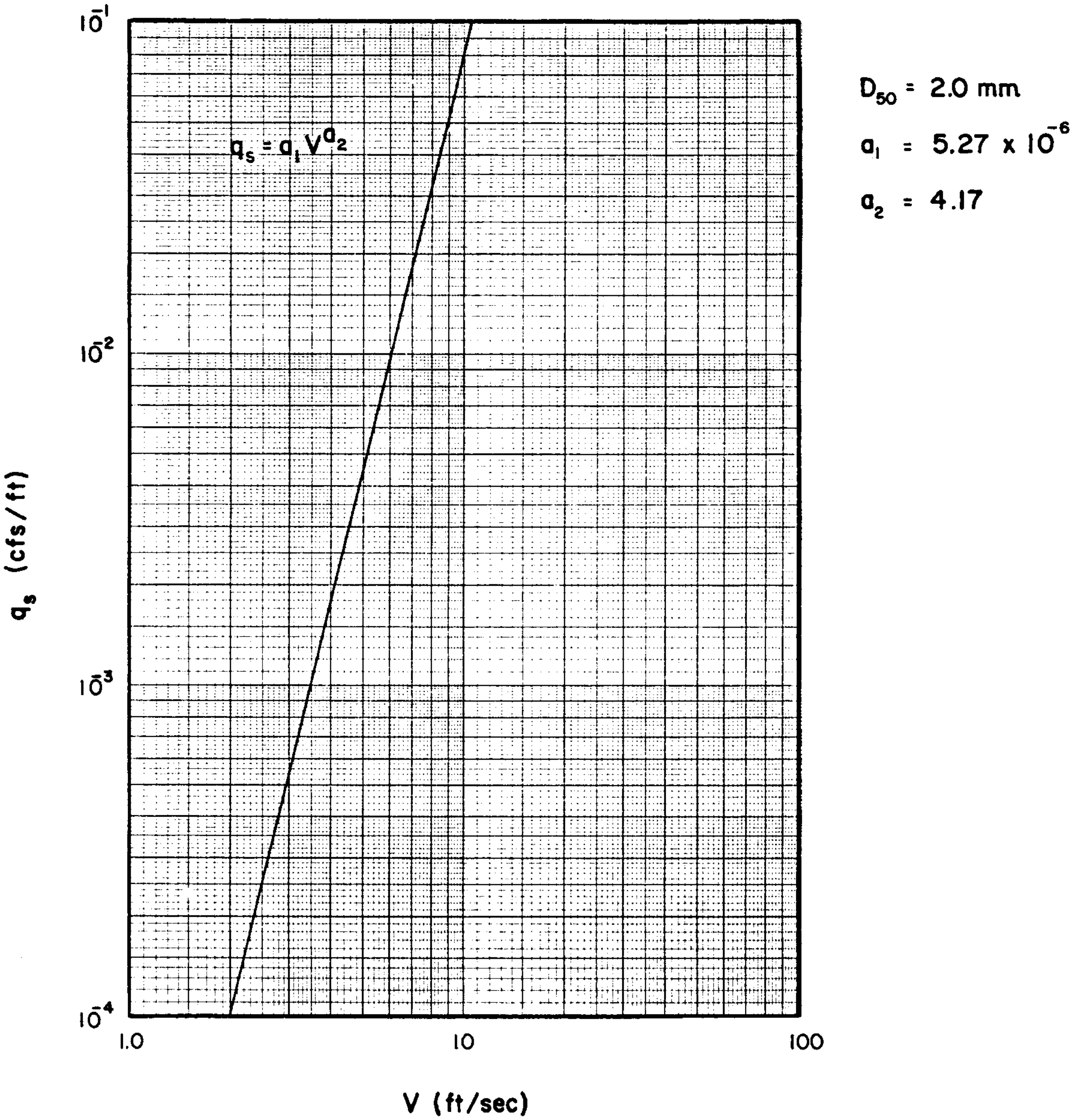


Figure 13.4. Values for sediment transport rate based on power relationships.

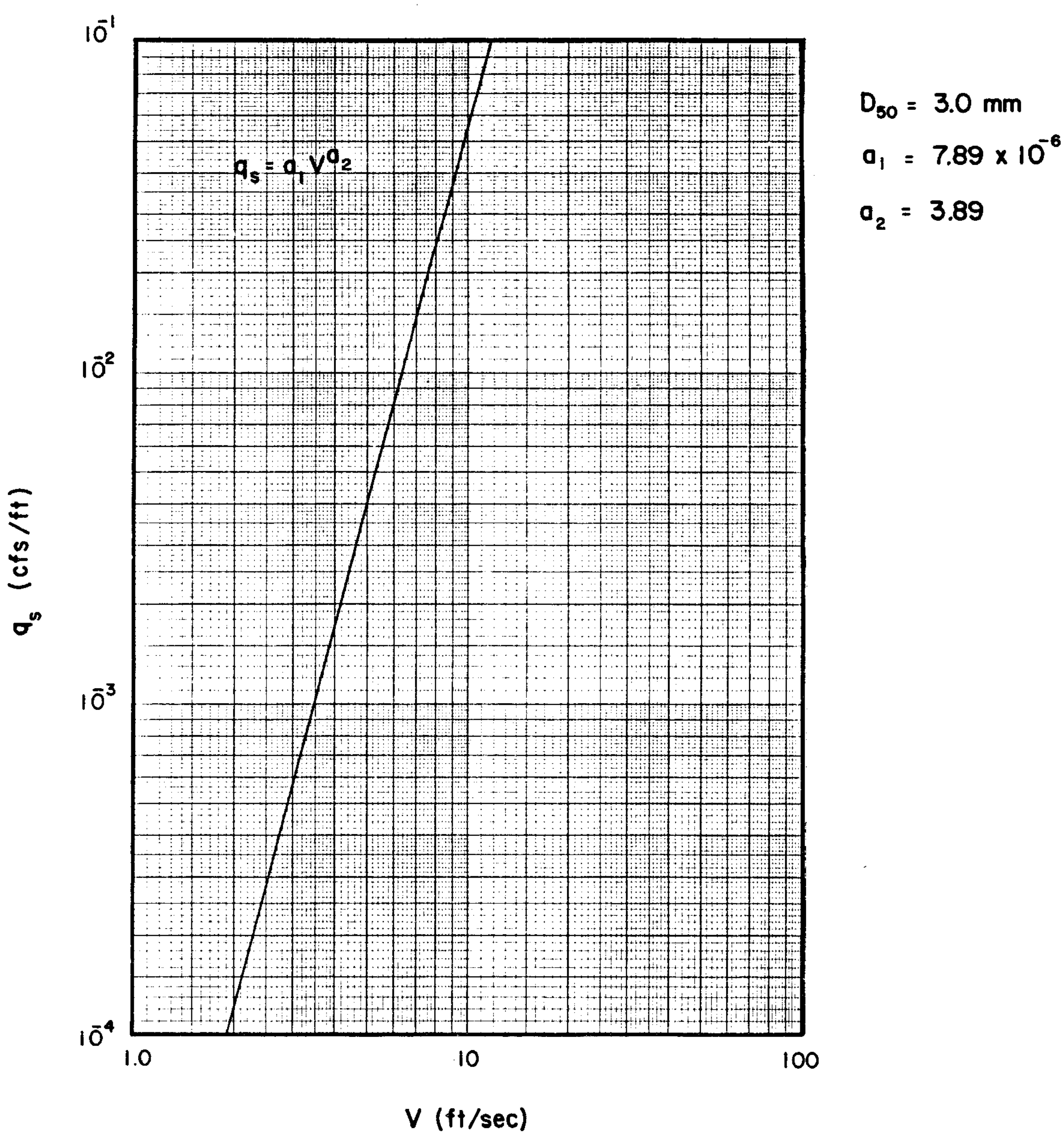


Figure 13.5. Values for sediment transport rate based on power relationships.

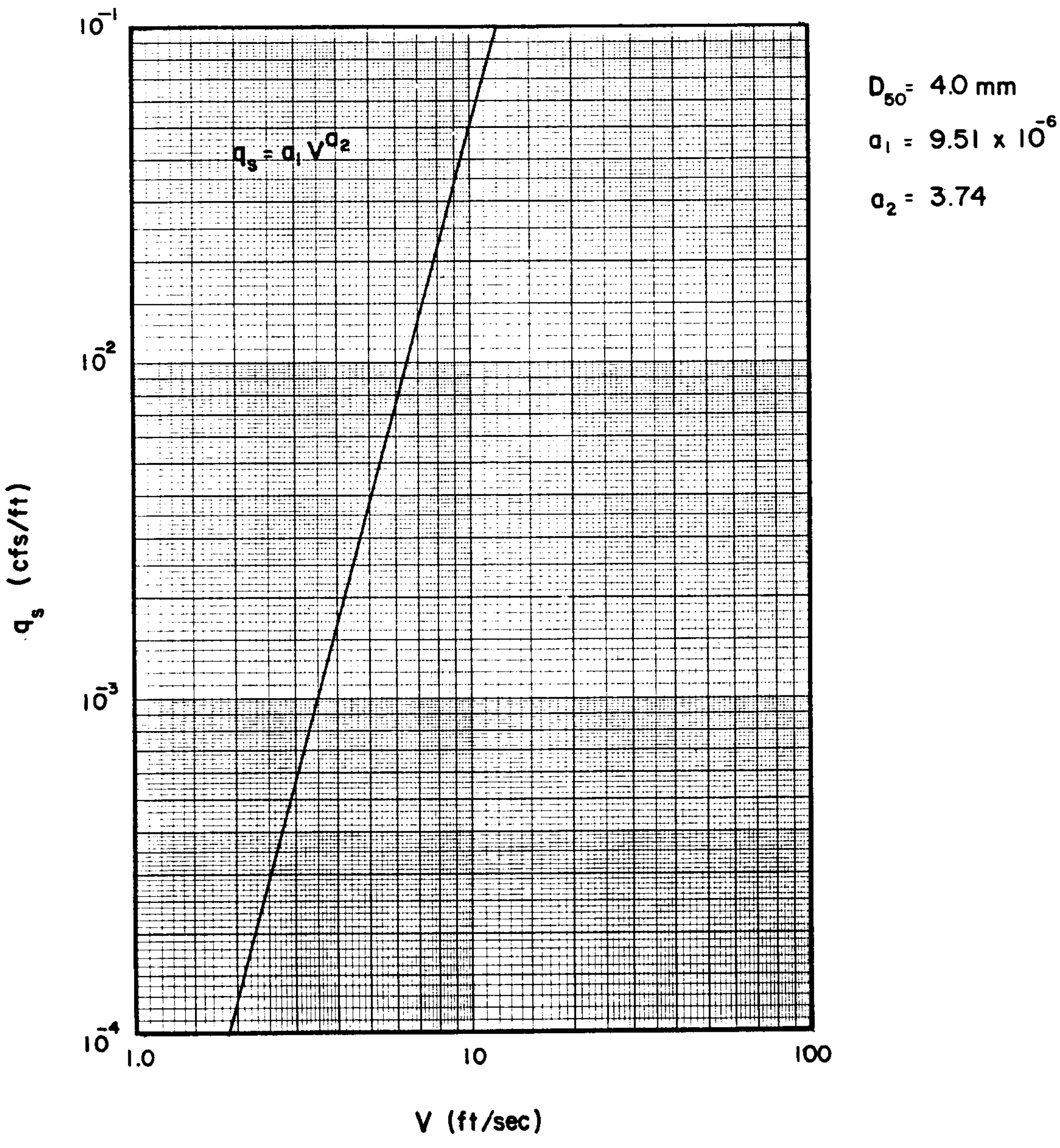


Figure 13.6. Values for sediment transport rate based on power relationships.

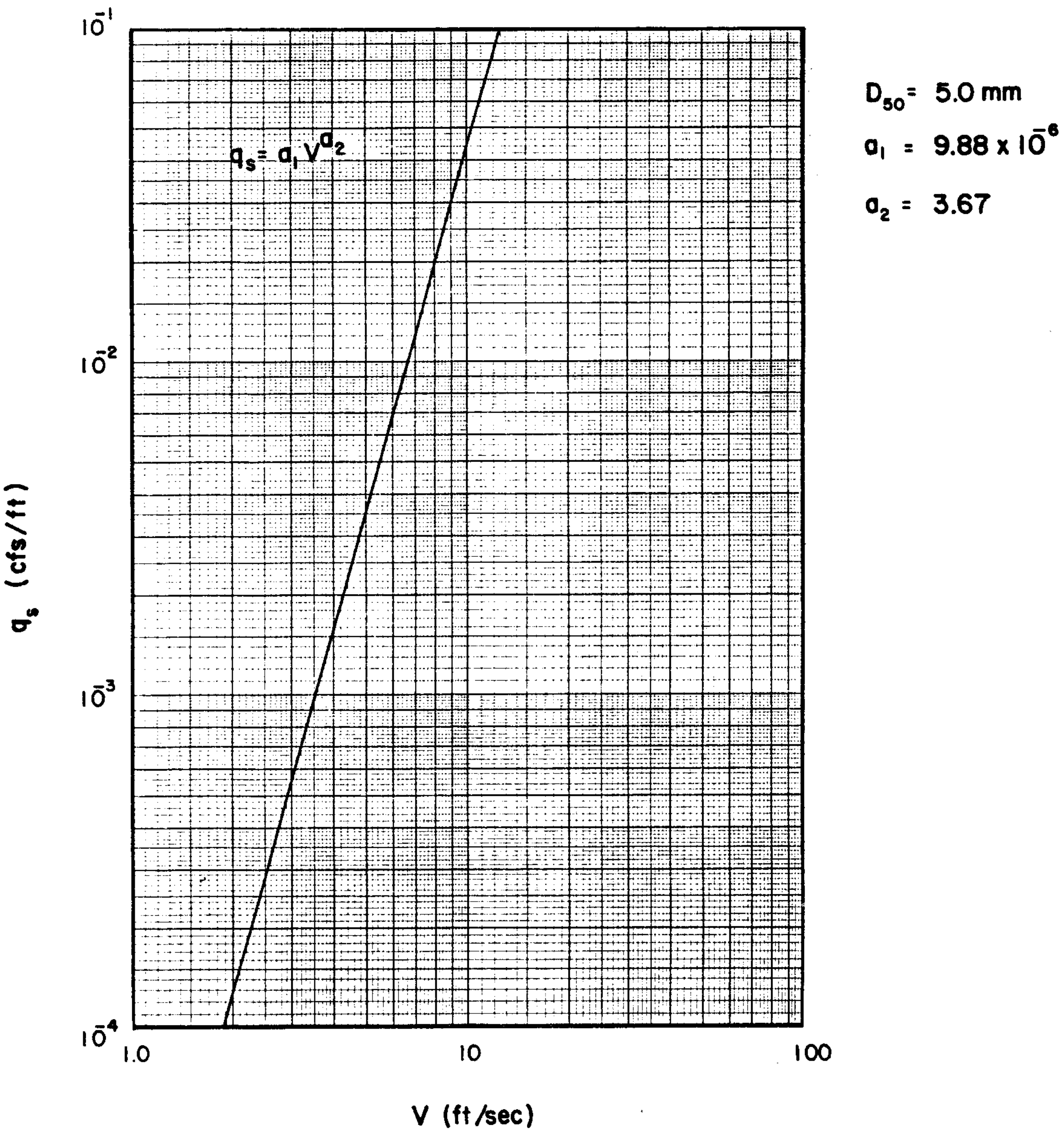


Figure 13.7. Values for sediment transport rate based on power relationships.

3. Determine total transport rate for entire width of channel by multiplying q_s by the top width of water surface.
4. Check the potential for armoring and the applicability of the sediment transport equations (Equations 13.5 and 13.6).

13.9 Design Example

1. Given the necessary information from the channel designed in Section 12.9, compute the sediment transport rate.

$b = 6$ ft	$Z = 3$	$D_{50} = 0.5$ mm
$W = 27$ ft	$V = 2.6$ ft/sec	$D_{85} = 0.75$ mm
$R = 2.01$	$S = 0.001$	$D_{15} = 0.3$ mm

2. Enter Figure 13.3 and obtain

$$q_s = 5.2 \times 10^{-4} \text{ cfs/ft}$$

3. The total transport rate becomes

$$Q_s = q_s\, W = (5.2\text{x}10^{-4})\ (27) = 0.014 \text{ cfs}$$

4. $\tau_o = \gamma_{\sigma} R\, S$ Equation 13.5

$$= (62.4)\ (2.01)\ (0.001)$$

$$= 0.125$$

$$D_a = \frac{\tau_o}{(\gamma_s - \gamma)\ 0.047}$$ Equation 13.6

$$= \frac{0.125}{(62.4)\ (2.65-1)\ 0.047}$$

$$= 0.026 \text{ ft} = 7.9 \text{ mm}$$

Therefore, all sediment sizes will be moving, armoring will not occur, and the sediment transport equations are applicable.

13.10 References

Lane, E. W., 1955, "The Importance of Fluvial Morphology in Hydraulic Engineering," Proc., ASCE, Vol. 21, No. 745, 17 p.

Lane, E. W., 1957, "A study of the shape of channels formed by natural streams flowing in erodible material," Missouri River Division Sediments Series No. 9, U.S. Army Engineer Division, Missouri River, Corps of Engineers, Omaha, Nebraska.

Simons, D. B., R. M. Li and W. T. Fullerton, 1981, "Theoretically Derived Sediment Transport Equations for Pima County, Arizona," for Pima County Department of Transportation and Flood Control District, Tucson, Arizona.

Simons, D. B., and E. V. Richardson, 1966, "Resistance of Flow in Alluvial Channels," U.S. Geological Survey Professional Paper 422-J.

XIV. BASIC DESIGN PROCEDURES FOR SANDY SOIL CHANNELS

14.1 Introduction

The design procedures for water diversion channels in sandy soils involve some of the basic concepts presented in Chapter VI. Of particular importance are the alluvial channel discussions and Section 6.2 on "Determination of Drainage Patterns and Diversion Alignment." However, as presented earlier in the manual, the highly erosive nature of sandy soils also require special considerations when designing water diversion channels. Due to the large sediment yields in sandy soil regions, the natural sediment transport balance must be maintained by the diversion channel. This can be accomplished by establishing the proper hydraulic conditions to create a balanced sediment transport in the channel according to the concepts of dynamic equilibrium. As discussed below this stable alluvial channel design procedure requires establishing the channel slope such that the balanced condition exists. If the design channel slope cannot be obtained feasibly by excavation, grade control structures can be used.

If this design approach is not feasible, a channel lining must be used to prevent excessive erosion. The lining may be a complete lining of the entire channel bed and banks or a partial lining of the channel banks only. For the latter case, the channel bed must then be designed for sediment transport balance according to the concepts of dynamic equilibrium.

Determining the most feasible channel design procedure (stable alluvial channel, partial lining, complete lining) requires an assessment of economic factors, including the costs of excavation, backfilling, grade control structures and riprap. For example, a partial lining requires extending the channel bank riprap into the channel bed. The costs of excavation and backfilling to bury the channel bank riprap to the proper depth should be compared to the costs of riprapping the entire channel to determine if the partial lining is really the most feasible approach.

Information presented below concentrates on stable alluvial channel design using the dynamic equilibrium concept and partial lining design procedures. Design for complete channel linings were presented in adequate detail in Part 1. Since each design situation is unique, it is difficult to provide general guidelines on the most economically feasible channel. The designer should consider the economic factors of a given channel design and its alternatives to ensure the best design is being recommended. Ultimately factors other than economic considerations may govern the final diversion channel design.

14.2 Dynamic Equilibrium Concept

Stable alluvial channel design in sandy soils involves the concept of dynamic equilibrium. Dynamic equilibrium exists when the channel boundary is in motion such that the sediment transporting capacity is equal to the sediment supply rate. According to Lane (1953), "A stable channel is an unlined earth channel (a) which carries water, (b) the banks and bed of which are not scoured objectionably by moving water, and (c) in which objectionable deposits of sediment do not occur. This definition is based on dynamic equilibrium concepts.

Dynamic equilibrium concepts must be applied to sand-bed type systems. The static equilibrium design approach used in Part 1 of the Design Manual is primarily applicable to gravel cobble type channel systems. A major controlling factor when assessing channel response using dynamic equilibrium concepts is the upstream sediment supply. Whether a channel bed aggrades or degrades strongly depends on the balance between the incoming sediment supply and a reach's transporting capacity. This is especially true for sand-bed channels where armoring does not occur. Evaluation of upstream sediment supply is accomplished by applying the appropriate sediment transport relationship. The recommended relationship for sand-bed channels was presented in the previous chapter. These equations are based on the assumption that all sediment sizes present can be moved by the flow. If this is not true, armoring will take place and the equations are not applicable. Therefore, the sediment size is also a major controlling factor when using dynamic equilibrium concepts. Lastly, the slope at which the channel's sediment transporting capacity equals the incoming sediment supply is important to the dynamic equilibrium concept. This slope is called the equilibrium slope, and is discussed in the following section.

14.3 Equilibrium Slope

The equilibrium slope is the bed slope at which the channel's sediment transporting capacity is equal to the incoming sediment supply for the design discharge. That is,

$$(Q_s)_{in} = (Q_s)_{out} \tag{14.1}$$

where

$(Q_s)_{in}$ = supply rate of sediment into the channel reach

$(Q_s)_{out}$ = supply rate of sediment out of the channel reach

Equilibrium slope should be determined for a discharge that will dominate the long-term degradation or aggradation process. The design discharge for surface mining operations must consider the type of stream flow (perennial or ephemeral) and the diversion type (permanent or temporary). Chapter III provides the recommended procedures for determining the design discharge.

14.4 Equilibrium Slope Design Procedures

Designing a channel by the equilibrium slope procedure results in a channel bed that is at dynamic equilibrium with no net aggradation or degradation. The design implicitly assumes the channel banks are in static equilibrium with no movement; otherwise, the channel would be shifting laterally and migrating throughout the floodplain. Under these conditions a long-term stable design would be difficult, if not impossible, to develop. Therefore, the channel banks must be stable for a successful dynamic equilibrium design. However, the equilibrium slope design considers only the condition of the bed, not the banks. Due to the side slope angle and bank material composition, the banks are usually less stable than the bed (see Section 4.6). Therefore, the stability of the side slopes must be checked after designing a channel with the equilibrium slope procedure.

One approach to evaluate the stability of the side slopes is based on maximum permissible velocity. If the mean channel velocity for the equilibrium slope design is greater than the maximum permissible velocity for the given bank material, the channel banks should be lined to insure stability. Use of the mean cross-section velocity for evaluating the bank stability will typically result in a conservative design since the velocity near the banks is usually less than the mean velocity.

In any case, the majority of equilibrium slope designs will require some type of bank lining to insure long-term stability of the channel. On surface mine sites the lining will typically be rock riprap. Rock riprap can usually be designed with a 2:1 side slope due to the greater angle of repose than non-cohesive sandy soil. Therefore, unless other criteria are governing, it is reasonable to use a design side slope of 2:1 for any equilibrium slope design under the assumption that riprap will be used for bank stabilization.

The procedures for designing a sand-bed channel at the equilibrium slope with a bank lining are:

1. Determine design discharge (see Chapter III).
2. Select upstream supply reach and obtain the following pertinent information:
   a. channel geometry (2:1 side slope unless other criteria require smaller angle)
   b. channel slope
   c. sediment size distribution
   d. channel resistance (n)
3. Obtain the same pertinent data as in Step 2 for the channel under consideration.
4. Calculate the hydraulic conditions based on the design discharge (Chapter XII).
5. Calculate the sediment supply from the upstream channel using the sediment transport procedure discussed in Chapter III. The calculated sediment supply is per unit width. The total sediment transport rate is obtained by multiplying the rate per unit width by the top width.
6. Determine the equilibrium slope of the downstream channel with the sediment supply rate determined in Step 5. This requires a trial and error procedure where a given slope is chosen to compute the flow conditions, and from the flow conditions the sediment transport rate is calculated. When the computed rate, $(Q_s)_{out}$, is equal to the supply rate, $(Q_s)_{in}$, the equilibrium slope has been found.
7. Based on the hydraulic conditions at equilibrum slope, estimate the largest particle size moving for armoring control check (Equation 13.5 and 13.6). Also check the design of the channel for reasonable shape and the presence of bedforms.
8. Based on maximum permissible velocity, check the bank lining requirements.

14.5 Evaluation of the Need for Rock Riprap or Drop Structures

If the cross section determined from the equilibrium slope design procedure is not economical or acceptable according to the P/R ratio (Section

12.8), then a more practical cross section may be designed by using a complete channel lining. If it is necessary to adjust the existing channel slope to the equilibrium slope, drop structures should be used. Multiple drop structures can be used effectively to reduce a channel slope which is too steep for the design conditions. The design procedures for channel linings and drop structures are presented in the next two chapters.

14.6 Design Example

The following example illustrates channel design based on the dynamic equilibrium slope concept. The physical layout of the example is given in Figure 14.1. The upstream channel has been in existence for many years and has not significantly changed. It is proposed that the downstream channel bypass a mine spoil area and deliver the diverted water to the main river. Determine the equilibrium slope of the downstream channel assuming a design discharge of 200 cfs. For ease in design and construction, a side slope angle of 3:1 is selected for the diversion to match the existing channel. The step-by-step procedure given in Section 14.5 will be followed.

Pertinent Information:

Steps 1, 2 and 3

	Upstream Channel	Downstream Channel
Design discharge	200 cfs	200 cfs
Channel shape	trapezoidal	trapezoidal
Sediment size distribution	D_{85} = 2.0 mm	D_{85} = 2.0 mm
	D_{50} = 1.0 mm	D_{50} = 1.0 mm
	D_{15} = 0.35 mm	D_{15} = 0.35 mm
Manning's n	0.025	0.025
Side slopes	3:1	3:1
Channel slope	0.0014	0.00135

Step 4. Compute the hydraulic conditions for the upstream channel assuming normal depth at Q = 200 cfs and

$b = 14$

$Qn = 5$

$S = 0.0014$

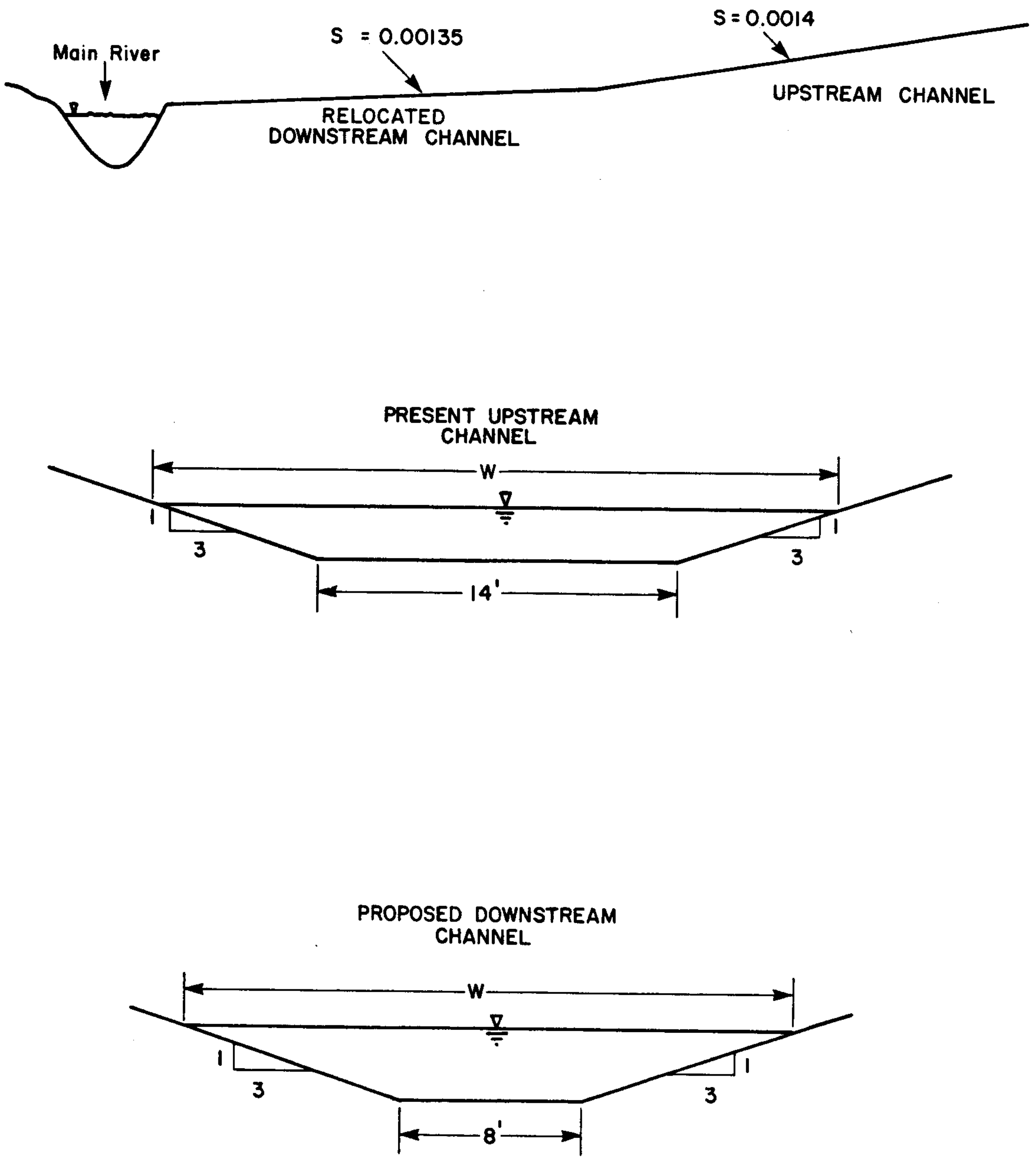


Figure 14.1. Physical layout of design example.

The initial estimate of d according to the channel charts in Appendix C of the Design Manual is

$$d = 2.7 \text{ ft}$$

Solving the Manning equation by an iterative technique:

$$A = 14d + 3d^2$$

$$P = 14 + 2\sqrt{10d}$$

$$Q = \frac{1.49}{n} AR^{2/3}S^{1/2} \qquad \text{Equation 4.14}$$

d	A	P	R	$R^{2/3}$	Q
2.7	59.7	31.1	1.92	1.55	208
2.6	56.7	30.41	1.86	1.52	193
2.65	58.2	30.8	1.89	1.53	200

Therefore, normal depth, d, is equal to 2.65 feet. The hydraulic conditions are

$A = 58.2 \text{ ft}^2$ $\qquad d_h = 1.95$
$d = 2.65 \text{ ft}$ $\qquad V = 3.4 \text{ ft/sec}$
$W = 29.9 \text{ ft}$ $\qquad F_r = 0.43$

Since the sediment transport equations were developed for a unit width channel, the hydraulic depth, $d_h = A/W$, is a better representation of the average channel characteristics.

Step 5. From Figure 13.3, the upstream sediment supply is

$$q_s = 9.0 \times 10^{-4} \text{ cfs/ft}$$

Total sediment supply is then

$$(Q_s)_{in} = Wq_s = 29.9\ (9.0 \times 10^{-4})$$

$$= 0.027 \text{ cfs}$$

Step 6. Determine the equilibrium slope for the downstream channel based on an upstream sediment supply rate of 0.027 cfs. First, assume S = 0.0012 and compute the hydraulic conditions of the downstream channel using

$Q = 200$ cfs

$b = 8$ ft

$Qn = 5$

The initial estimate of the normal depth is obtained from the appropriate channel chart in Appendix C. It is

$d = 3.8$ ft

Solving the Manning equation

$$A = 8d + 3d^2$$

$$P = 8 + 2\sqrt{10}d$$

$$Q = \frac{1.49}{n} AR^{2/3} S^{1/2} \qquad \text{Equation 4.14}$$

d	A	P	R	$R^{2.3}$	Q
3.8	73.7	32	2.30	1.75	266
3.2	56.3	28.2	1.99	1.59	184
3.4	61.9	29.5	2.10	1.64	210
3.3	59.1	28.9	2.05	1.62	200

The normal depth is 3.3 feet and the hydraulic conditions are

$A = 59.1$ ft^2 $\qquad d_n = 2.1$ ft

$d = 3.3$ ft $\qquad V = 3.38$ ft/sec

$W = 27.8$ ft $\qquad F_r = 0.41$

From Figure 13.3, the sediment transport rate is

$$q_s = 9.0 \times 10^{-4} \text{ cfs/ft}$$

$$(Q_s)_{out} = 27.8\ (9.0 \times 10^{-4}) = 0.025 \text{ cfs}$$

Note that $(Q_s)_{in}$ is greater than $(Q_s)_{out}$. Therefore the downstream channel will aggrade if designed at this slope. To increase $(Q_s)_{out}$, the channel slope should be increased. Try S = 0.0013. Using this value for the slope, the hydraulic conditions become

$d = 3.25$ ft $\qquad d_n = 2.1$ ft $\qquad R = 2.02$

$A = 57.7$ ft^2 $\qquad V = 3.46$ ft/sec

$W = 27.5$ fr $\qquad F_r = 0.42$

From Figure 13.3, the sediment transport rate is

$$q_s = 9.8 \times 10^{-4} \text{ cfs/ft}$$

$$(Q_s)_{out} = 27.5(9.8 \times 10^{-4}) = 0.027 \text{ cfs}$$

For this slope, $(Q_s)_{in}$ is equal to $(Q_s)_{out}$ and the channel is stable. If $(Q_s)_{in}$ is not approximately equal to $(Q_s)_{out}$, repeat the trial and error procedure illustrated above. In general, the small difference between $(Q_s)_{in}$ and $(Q_s)_{out}$ in the steps above does not warrant a recalculation of the equilibrium slope. It is done in this example for the purpose of illustrating the design procedure.

Step 7. Check for armoring control. Compute the largest particle transported by

$$\tau = \gamma RS$$

$$= 62.4 \times 2.02 \times 0.0013$$

$$= 0.1 \text{ lb/ft}^2$$

and

$$\tau = 0.047d(\gamma_s - \gamma)$$

$$D_a = \frac{0.164}{0.047(2.65-1)(62.4)}$$

$$= 0.034 \text{ ft} = 10.4 \text{ mm}$$

Comparing this size with the given sediment size distribution indicates that all sediment sizes will be moving. Therefore, armoring will not occur and the equilibrium slope calculations are valid. Checking the bedform reveals

$$\tau = 0.1 \text{ lb/ft}^2 \text{ (Step 5)}$$

Therefore

$$\tau V = 0.1(3.46)$$

$$= 0.346$$

From Figure 12.2, the bedforms present are dunes. Table 12.3 shows that the initial estimate of Manning's n (0.025) for the diversion channel is within the range of values for a dune bed (0.025 - 0.030). Thus, the equilibrium slope calculations based on a Manning's n of 0.025 are correct. If this check reveals that the initial estimate of Manning's n is incorrect, the equilibrium slope must be recomputed based on the value listed in Table 12.4 for the appropriate bedform.

Check channel for reasonable shape:

$$P = \frac{A}{R} = \frac{57.7}{2.02} = 28.6 \text{ ft}$$

Using equation 12.5,

$$\frac{P}{R} \cong 7 \times V = 7 \times 3.46 = 24.22$$

$$28.6 \cong 24.22$$

Therefore the design of the channel is a practical design.

Step 8. In this example it is assumed that the bed and bank are composed of sandy material. From Table 6.1a the maximum permissible velocity is 2.50 ft/sec. Comparing this value with the channel velocity of 3.46 ft/sec indicates that, as expected, a channel lining is needed to stabilize the sideslopes. The design of the sideslope channel lining will be discussed in Chapter XV.

XV. DESIGN OF RIPRAP LININGS IN SANDY SOILS

15.1 Introduction

The equilibrium slope design procedure described in the previous chapter provides a means for determination of a slope at which the sediment transport capacity matches the sediment supply. However, at the equilibrium slope flow velocities may exceed the maximum permissible velocity criteria for the material comprising the channel banks (see Section 14.4). In this instance, rock riprap can be utilized to protect the channel banks. Riprap protection on the bed is not necessary when the channel is designed at the equilibrium slope; however, bank protection must be extended below the bed to prevent undercutting of the banks due to local scour and the troughs of passing sand waves.

Riprap has a larger angle of repose than noncohesive sandy soil (see Section 4.6). Consequently, as previously discussed, if riprap is utilized as bank protection, it is recommended that the channel be constructed with 2:1 side slopes unless other criteria require a smaller side slope. This increase in the side slope angle, from the 3:1 criteria for natural sandy soils, will reduce excavation quantities and associated construction costs. The volume of rock necessary for bank protection will also be reduced, but slightly larger rock will be required.

Adding a channel lining of riprap and changing the channel side slopes will affect the channel roughness and geometry. These changes, in turn, will necessitate reevaluation of the previously computed equilibrium slope. A new roughness coefficient called the effective Manning's n, n_e, will be determined for the partially lined channel. This chapter will present the design procedures based on the effective roughness coefficient.

15.2 General Considations

The general guidelines for riprap design given in Section 5.1 including gradation, thickness and filter layers apply to channels constructed in sandy soils. The mild slope riprap design procedure given in Section 6.6 is the recommended method for sizing the rock required. When a partial lining is required, it is recommended that the bank protection be extended below the channel bed to a depth equal to the normal depth of flow (Figure 15.1).

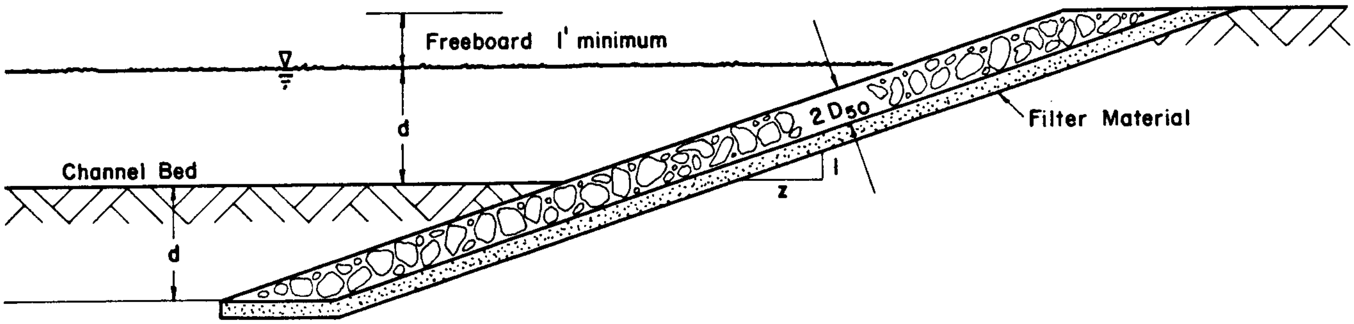


Figure 15.1. Bank protection with rock riprap.

15.3 Channel Roughness Coefficient

When portions of the wetted perimeter in a canal section are composed of different materials having different roughness coefficients, it is usually necessary to determine an effective channel roughness for use in hydraulic computations. Determination of an equivalent Manning roughness coefficient (n_e) is of particular importance when the materials forming a composite lining have significantly different properties of resistance to flow. Riprap typically has a value of Manning's n in a range from 0.03 to 0.045 depending on the riprap size. However, Manning's n for channels with sand bed and banks usually ranges from 0.02 to 0.03 depending on the flow properties and D_{50} of the sand (see Table 12.2).

Lotter (1933) presented an equation for calculation of an equivalent roughness coefficient of the form (Chow, 1959)

$$n_e = \frac{PR^{5/3}}{\sum_{1}^{N} \frac{P_N R_N^{5/3}}{n_N}} = \frac{PR^{5/3}}{\frac{P_1 R_1^{5/3}}{n_1} + \frac{P_2 R_2^{5/3}}{n_2} + \ldots + \frac{P_N R_N^{5/3}}{n_N}} \qquad (15.1)$$

where
- P = wetted perimeter of total section
- R = hydraulic radius of total section
- P_N = wetted perimeter of channel subsection
- R_N = hydraulic radius of channel subsection
- n_N = Manning roughness coefficient of channel subsection

This equation is based on the assumption that the total discharge is equal to the sum of discharges through individual components of a subdivided area.

For a symmetrically shaped trapezoidal channel with riprap on the side slopes and a sand bed, the channel cross section can be subdivided as shown in Figure 15.2. Since subareas 1 and 3 are identical, Equation 15.1 can be written with two terms in the denominator as

$$n_e = \frac{PR^{5/3}}{\frac{P_1 R_1^{5/3}}{n_1} + \frac{P_2 R_2^{5/3}}{n_2}} \qquad (15.2)$$

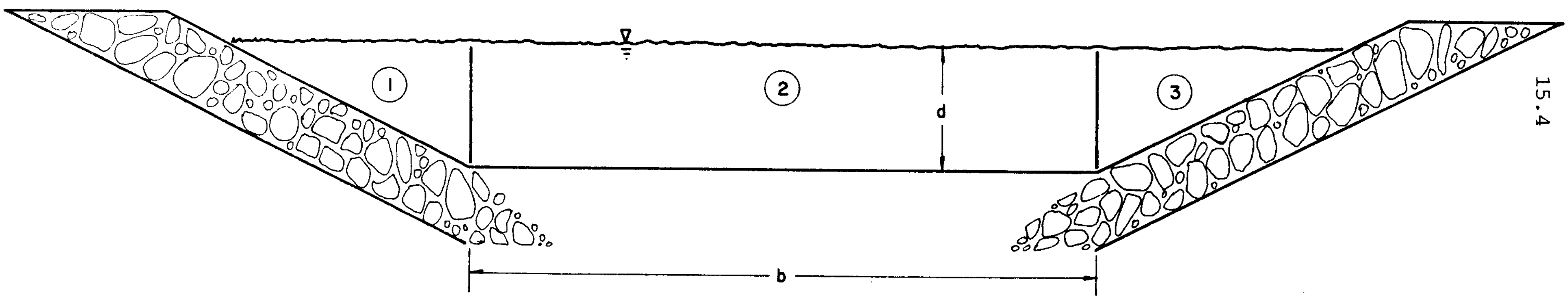


Figure 15.2. Subareas for determination of an equivalent roughness coefficient.

Here P_1, R_1, and n_1, relate to the riprapped portions and P_2, R_2, and n_2 are properties of the subarea. Equations for P and R for each section assuming 2:1 side slopes are

$$P_1 = P_3 = \sqrt{5}\, d \qquad\qquad P_2 = b$$
$$R_1 = R_3 \frac{d}{\sqrt{5}} \qquad\qquad R_2 = d \tag{15.3}$$

Substituting these values into Equation 15.2 yields

$$n_e = \frac{PR^{5/3}}{1.17\, \dfrac{d^{8/3}}{n_1} + \dfrac{bd^{5/3}}{n_2}} \tag{15.4}$$

Finally, rearranging this equation to obtain a ratio of n_1 to n_2 produces

$$\frac{n_e}{n_1} = \frac{PR^{5/3}}{1.17\, d^{8/3} + bd^{5/3}\, \dfrac{n_1}{n_2}} \tag{15.5}$$

This equation has been solved graphically in Figures 15.3 to 15.5 for different ratios of n_1 to n_2 and channel base widths. The design example at the end of the chapter will illustrate the use of these figures.

15.4 Design Procedures Summary

15.4.1 Criteria for Riprap Design

The design of riprap bank protection for channels where maximum permissible velocity criteria are exceeded follows the procedures described below. This design procedure assumes that sediment transport to the channel, equilibrium slope, and the hydraulic properties of the natural sand bed channel have been evaluated previously using methods presented in Chapter XIV.

1. Evaluate equilibrium slope and the hydraulic properties (d, A, P, R, V, etc.) for the natural sand material with 2:1 channel side slopes. Equilibrium slope is evaluated for the sediment supply rate of an upstream supply reach. When the computed transport rate $(Q_s)_{out}$ is equal to the supply rate $(Q_s)_{in}$, the equilibrium slope has been found.

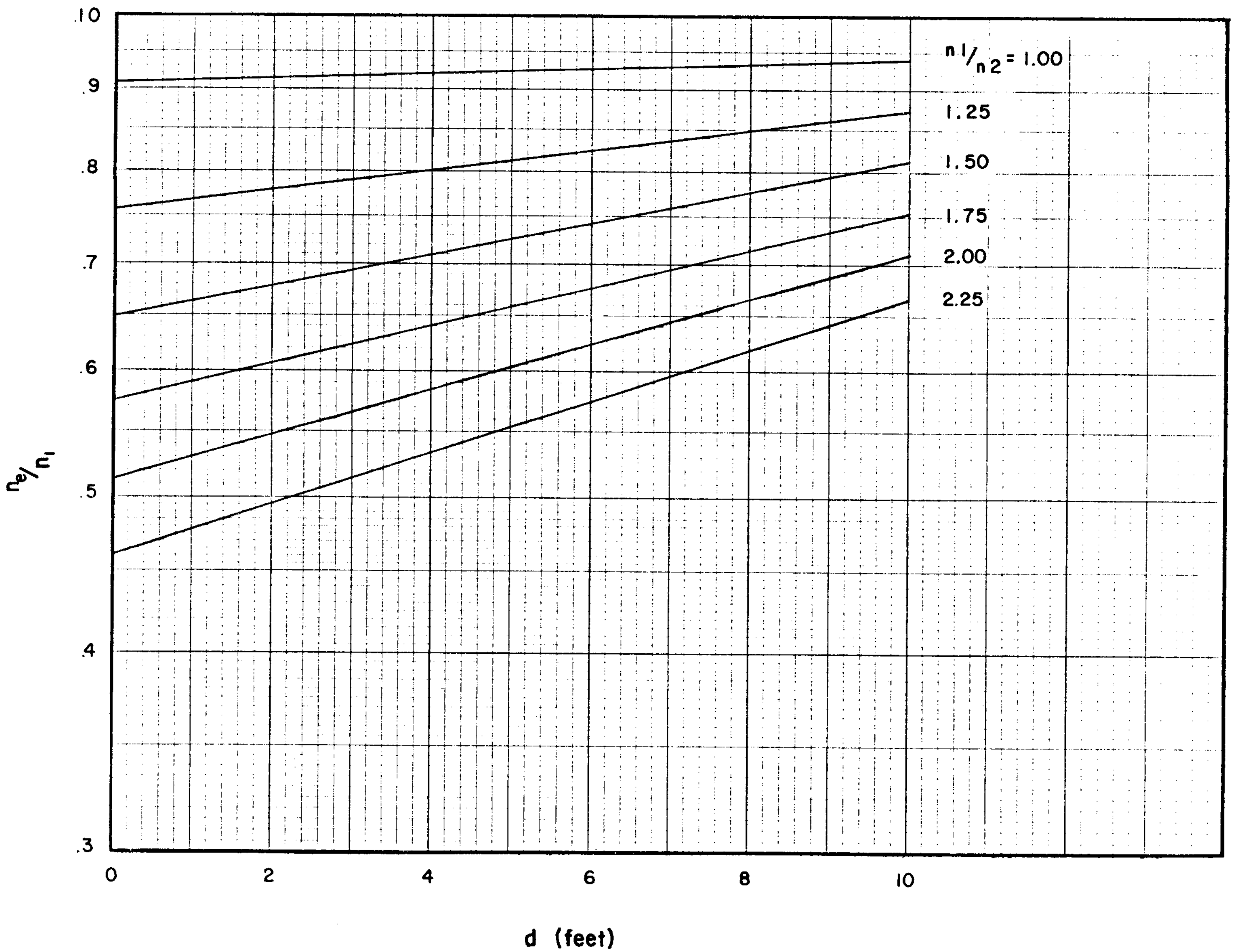


Figure 15.3. Graph for determining n_e, six-foot base width.

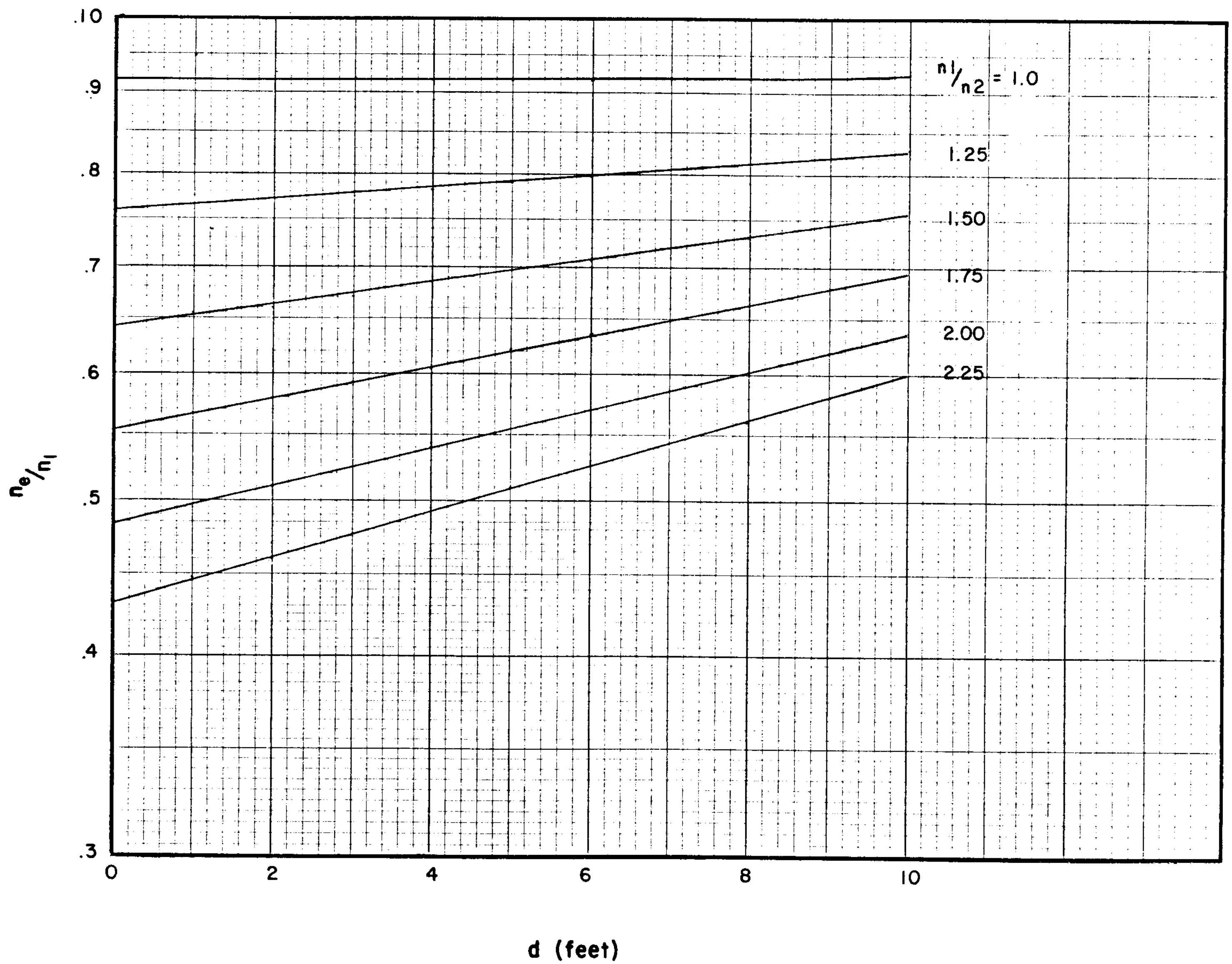


Figure 15.4. Graph for determining n_e, ten-foot base width.

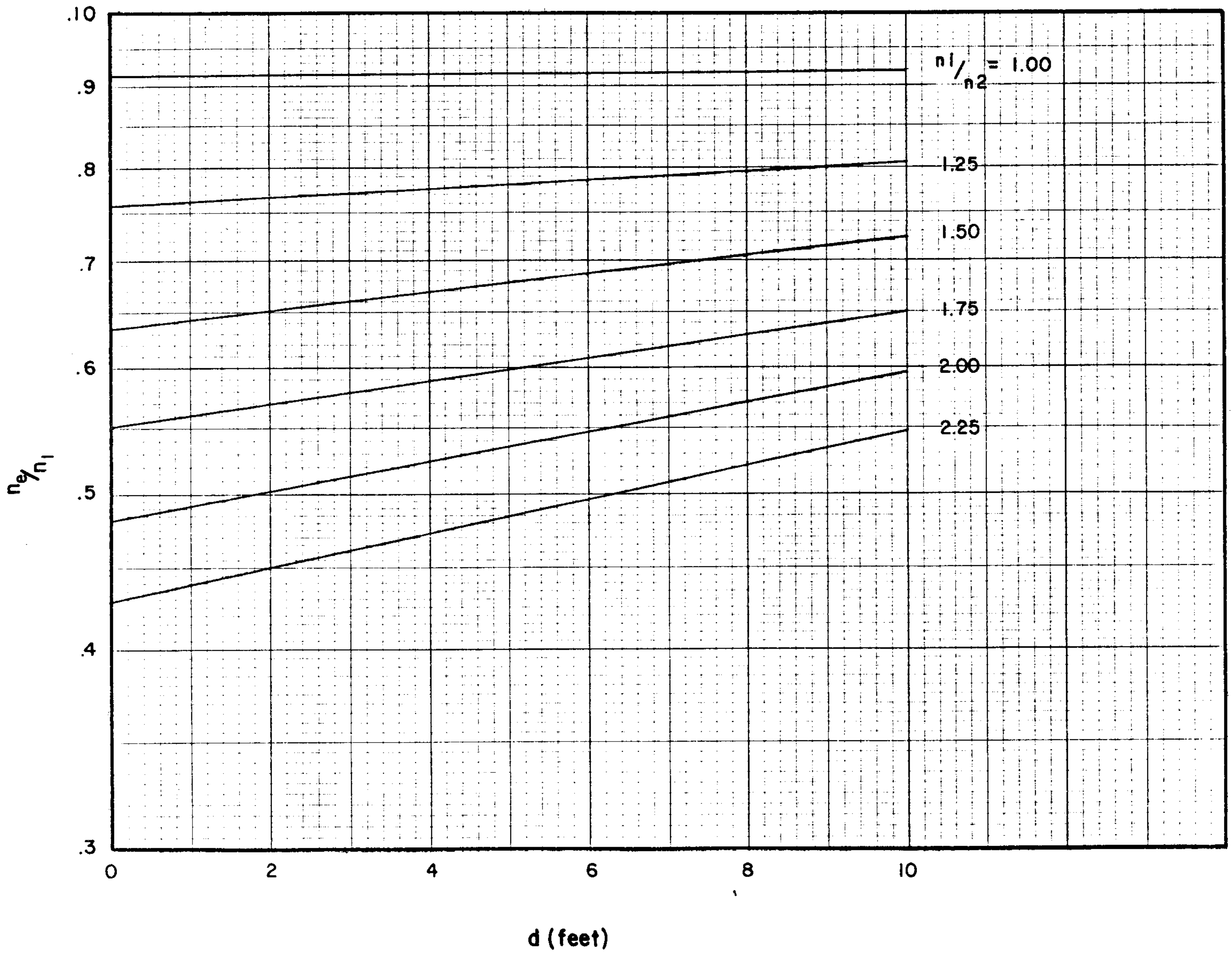


Figure 15.5. Graph for determining n_e, 14-foot base width.

2. Use the values for V and R obtained in Step 1 to compute $V^2/R^{0.33}$. Use this value to obtain an initial estimate for the riprap size required from Tables 6.4 and 6.5.
3. Estimate the Manning roughness coefficient for the riprap

$$n_1 = 0.0395\ (K_m)^{1/6} \qquad \text{Equation 4.18}$$

4. Calculate n_1/n_2, where n_2 equals the Manning roughness coefficient for the assumed bed form. This value was estimated during the design evaluation in Chapter XII.
5. Using Figures 15.3-15.5, recompute the hydraulic properties based on an equivalent roughness coefficient. This procedure involves a trial and error process in which a depth is chosen to compute the design discharge based on the equivalent roughness coefficient.
6. For the flow conditions computed in Step 5, use the velocity, V and the hydraulic radius, R to compute $V^2/R^{0.33}$. Check the riprap required (Table 6.4) with the initial assumption (Step 2). If the assumed value is incorrect, return to Step 2 with a new estimate for K_m.
7. Using the recomputed value for V, check the sediment transport rate $(Q_s)_{out}$ (Figures 13.3-13.7). If $(Q_s)_{in}$ is not equal to $(Q_s)_{out}$, repeat steps 1 to 5 and replace Manning's n with the previously computed value for n_e.
8. Compute stream power, τU, and check bed forms using Figure 12.4. Determine if the assumed value for roughness, n_2, was correct (Table 12.2). If n_2 is significantly different, return to Step 4 with a new value for n_2.
9. Check Froude Number to insure applicability of the method.
10. Determine riprap gradation (Table 6.5), thickness and depth.
11. Evaluate filter requirements (Section 5.2.8)
12. Estimate freeboard requirements from Equation 12.2.

15.4.2 Design Procedure for Riprap Lined Channel Bends

1. Evaluate riprap requirements for straight channel section (Section 15.5.1).
2. Determine channel radius of curvature r_c and top width w and compute r_c/w.
3. Evaluate shear ratio for specific value of r_c/w from Figure 6.8.

4. Multiply value of $V^2/R^{0.33}$ for a straight channel by shear stress ratio.

5. Use adjusted value of $V^2/R^{0.33}$ to size riprap in bend from Table 6.4. If bend riprap size is different from riprap in straight channel reaches, evaluate need for different

   a. gradation of riprap

   b. thickness of riprap

   c. filter requirements

15.5 Design Examples

15.5.1 Riprap Design Example - Use Step by Step Procedure in Section 15.5.1

Step 1. Since riprap will be used for the bank lining, the channel side slope can be assumed to be 2:1, unless other criteria are governing. Prior to design of the riprap lining, the equilibrium slope for a sand bed channel (no riprap) with 2:1 side slopes must be evaluated. Assuming S = 0.0028 and given Q = 250 cfs, D_{50} = 0.5 mm, b = 10 ft and $(Q_s)_{in}$ = 0.35 cfs, hydraulic conditions are:

$$A = bd + zd^2$$

$$P = b + 2\sqrt{5}d$$

$$R = \frac{A}{P}$$

$$Q = \frac{1.49}{n} R^{2/3} S^{1/2} A$$

d	A	P	R	$R^{2/3}$	Q
2.8	43.7	22.5	1.94	1.55	243
2.85	44.7	22.7	1.97	1.57	252

Therefore, normal depth equals 2.85 feet.

$$V = \frac{Q}{A} = \frac{250}{44.7} = 5.6 \text{ fps}$$

From Figure 13.3

$$q_s = 2 \times 10^{-2} \text{ cfs/ft}$$

$$(Q_s)_{out} = q_s w = 2 \times 10^{-2}(21.4) = 0.43 \text{ cfs} > 0.35 \text{ cfs}$$

Since $(Q_s)_{in}$ does not equal $(Q_s)_{out}$, a different value for equilibrium slope must be assumed and checked. Assume a slope of 0.0025:

d (ft)	A (ft^2)	P (ft)	R (ft)	$R^{2/3}$	Q (cfs)
2.9	45.8	23.0	1.99	1.58	246

$$d = 2.9 \text{ ft}$$

$$V = \frac{Q}{A} = \frac{246}{45.8} = 5.4 \text{ fps}$$

$$w = 21.6 \text{ ft}$$

From Figure 13.3

$$q_s = 1.6 \times 10^{-2} \text{ cfs/ft}$$

$$(Q_s)_{out} = q_s w = (1.6 \times 10^{-2})(21.6) = 0.35 \text{ cfs}$$

For this slope (S = 0.0025) the transport capacity of the channel reach is equal to the sediment inflow. Checking the armoring potential reveals

$$\tau = \gamma RS = 62.4(1.99)(0.0025) = 0.31 \text{ lb/ft}^2 \qquad \text{Equation 13.5}$$

$$D_a = \frac{0.311}{0.047(1.65)(62.4)} = 0.06 \text{ ft} = 19.6 \text{ mm} \qquad \text{Equation 13.6}$$

Thus armoring will not occur and the equilibrium slope is 0.0025.

<u>Step 2</u>. Using the values of V and R determined for the sand bed channel, an estimate for the mean riprap size K_m is

$$\frac{V^2}{R^{0.33}} = \frac{5.4^2}{1.99^{0.33}} = 23.2$$

Table 6.4 indicates a Type L riprap with $K_m = 9$ in (Table 6.5).

Step 3. Estimate riprap roughness

$$n_1 = 0.0395(0.75)^{1/6} = 0.038 \qquad \text{Equation 4.18}$$

Step 4. $\dfrac{n_1}{n_2} = \dfrac{0.038}{0.022} = 1.73$

This value for n_1/n_2 is used in Figure 15.4 to determine a n_e/n_1 for a specific depth d.

Step 5. Evaluate the hydraulic properties

$$A = bd + 2d^2$$

$$P = b + 2\sqrt{5}\,d$$

$$R = \frac{A}{P}$$

$$Q = \frac{1.49}{n_e} R^{2/3} S^{1/2} A$$

d (ft)	$\frac{e}{n_1}$	n_e	A (ft^2)	P (ft)	R (ft)	$R^{2/3}$ ($ft^{2/3}$)	Q (cfs)
2.9	0.60	0.023	45.8	23.0	1.99	1.58	237
3.0	0.61	0.023	48.0	23.4	2.05	1.61	253

d = 3.0 ft

$$V = \frac{Q}{A} = \frac{253}{48} = 5.3 \text{ fps}$$

w = 22 ft

Step 6. $\dfrac{V^2}{R^{0.33}} = \dfrac{5.3^2}{2.05^{0.33}} = 22.2$

The initial assumption for the riprap size was correct (Table 6.4).

Step 7. Using Figure 13.3, check sediment transport rate.

qs = 0.015 cfs/ft

$$(Q_s)_{out} = (0.015)(22) = 0.33 \text{ cfs} \approx 0.35 \text{ cfs}$$

The addition of riprap does not significantly alter the flow characteristics. Thus, the equilibrium slope in the channel with riprap protection is the same as the slope computed in Step 1 for the sand bed channel alone.

Step 8. Check channel bedforms

$$\tau V = 0.32(5.3) = 1.7 \frac{\text{lb-ft}}{\text{ft}^2\text{-sec}}$$

From Figure 12.2 with $\tau V = 1.7 \frac{\text{lb-ft}}{\text{ft}^2\text{-sec}}$ and $D_{50} = 0.5$ mm, the bed form is antidunes. Table 12.2 indicates a range of n values for antidunes from 0.020 to 0.025. Thus the values of n = 0.022, used on the design, was a valid assumption.

Step 9. Froude Number

$$F_r = \frac{5.3}{\sqrt{32.2\ (3.0)}} = 0.54$$

Therefore, the use of the mild slope procedure for riprap design is valid.

Step 10. From Table 6.5 the required gradation is

Percent Smaller	Maximum Diameter - inches
100	12
35-55	9
10	2

The thickness of the riprap layer should be $2.0\ D_{50} = 1.5$ ft which is also equal to the maximum recommended riprap size, D_{max}. Riprap should be extended below the channel bed to a depth equal to the depth of flow, d = 3.0 ft.

Step 11. Evaluate filter requirements as discussed below.

Step 12. Freeboard. For a riprap lined channel on a mild slope, Table 4.4 indicates c_{fb} equals 0.25.

$$c_{fb}d = 0.25(3) = 0.75 < 1.0; \text{ therefore use } 1.0 \text{ ft}$$

Antidune height

$$h_a = 2\pi\ (0.14) \frac{V^2}{g} = 0.77 \text{ ft} \qquad \text{Equation 12.1}$$

Superelevation - for an assumed radius of curvature equal to 100 ft

$$\Delta Z = \frac{V^2}{gr_c} (W) \qquad \text{Equation 4.19}$$

$$\Delta Z = \frac{5.3^2}{32.2(100)} (22) = 0.19 \text{ ft}$$

$$F.B. = C_{fb}d + \frac{1}{2} ha + \frac{1}{2} \Delta Z = 1.0 + 0.38 + 0.10 \qquad \text{Equation 12.2}$$

F.B. = 1.48 ft ; use 1.5 ft

The final channel dimensions are shown in Figure 15.6.

15.5.2 Design Example for Granular Filter Layer

This example details procedures for determination and selection of an appropriate filter layer. The U.S. Army Corps of Engineers filter criteria are used because the limits are somewhat less restrictive than the Terzaghi filter criteria. The characteristics of the channel base material are assumed to be:

D_{85} = 1.4 mm
D_{50} = 0.5 mm
D_{15} = 0.16 mm

Riprap properties are determined by plotting the recommended gradation on semi-log paper (Figure 15.7)

D_{85} = 370 mm
D_{50} = 230 mm
D_{15} = 75 mm

1. Evaluate the need for a filter layer.

$$\frac{D_{15}\text{(riprap)}}{D_{85}\text{(base)}} = \frac{75}{1.4} = 54 \; ; \; 54 > 5$$

$$\frac{D_{15}\text{(riprap)}}{D_{15}\text{(base)}} = \frac{75}{0.16} = 469 \; ; \; 469 > 40$$

2. Properties of the filter relative to the base material are:

$$\frac{D_{50}\text{(filter)}}{D_{50}\text{(base)}} < 40, \text{ so } D_{50} \text{ (filter)} < 40(0.5) = 20 \text{ mm}$$

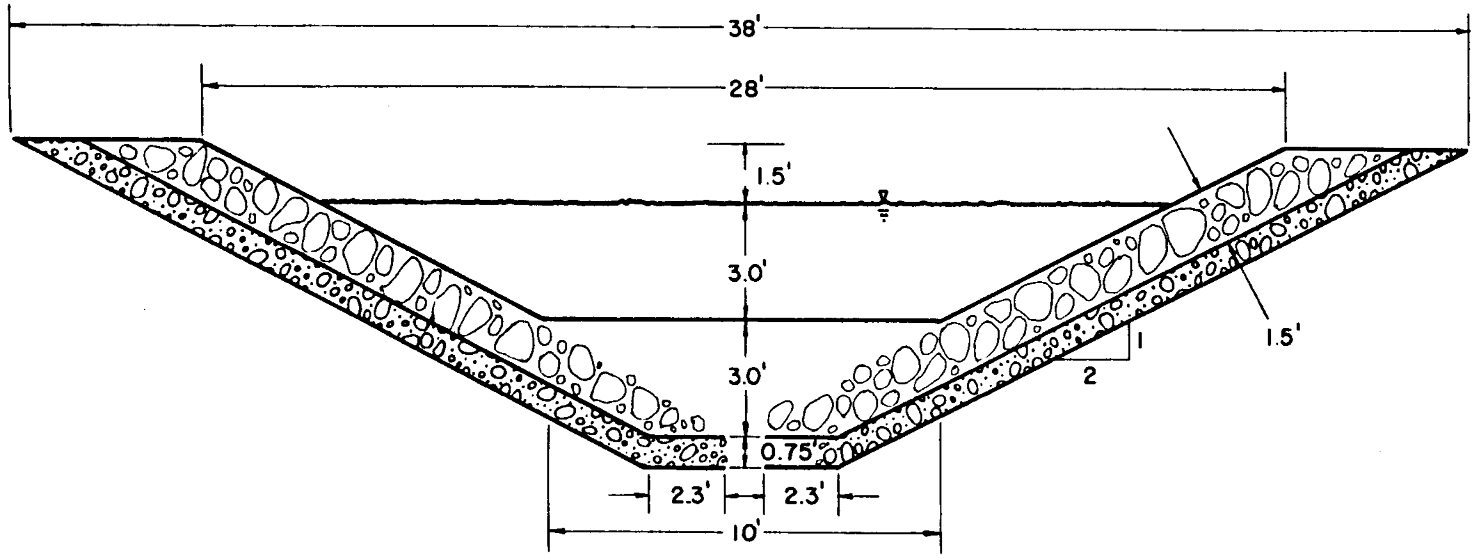


Figure 15.6. Cross section of diversion channel.

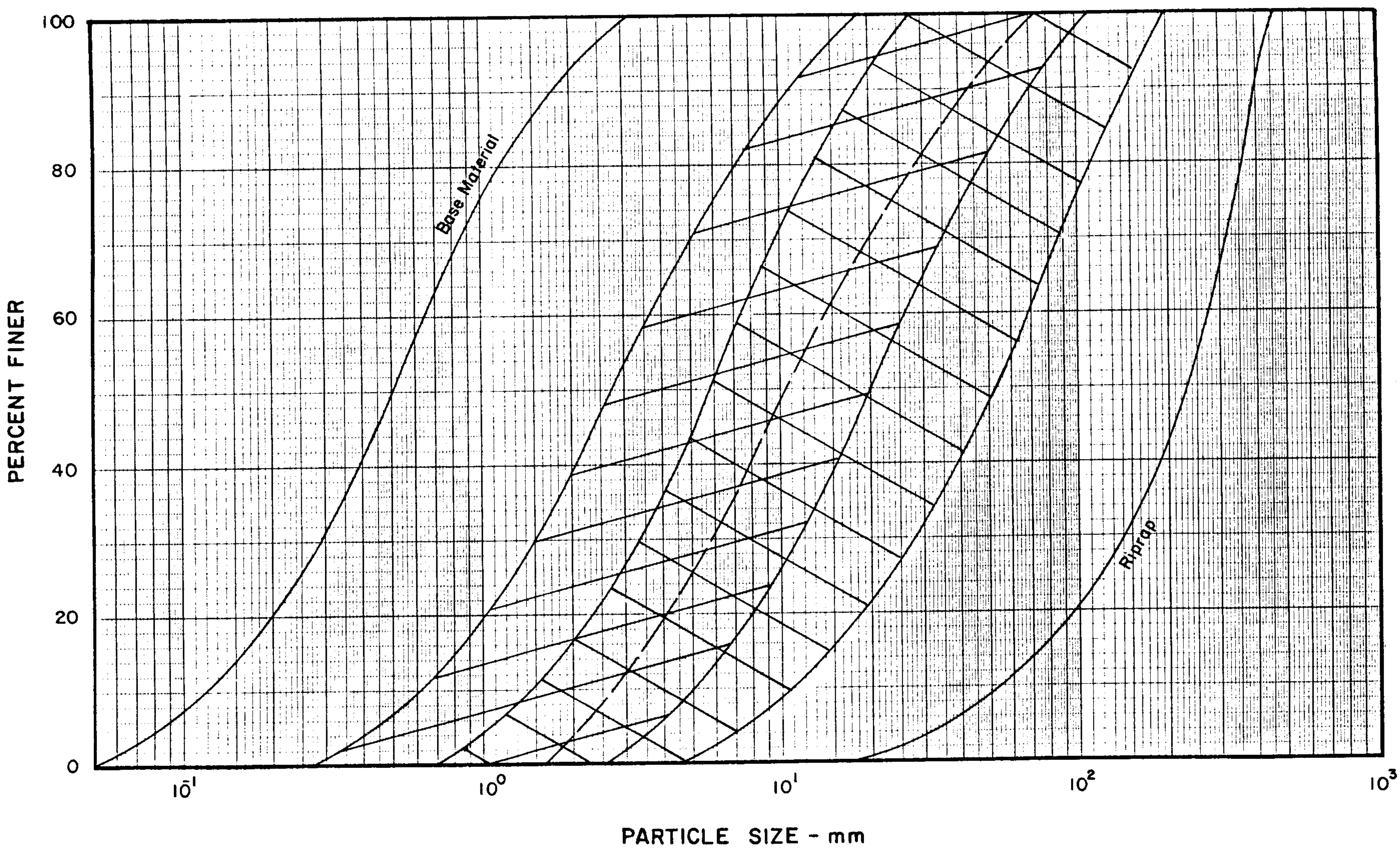


Figure 15.7. Particle size distribution for base material, riprap and filter layer.

$$\frac{D_{15}(\text{filter})}{D_{15}(\text{base})} < 40, \text{ so } D_{15}(\text{filter}) < 40(0.16) = 6.4 \text{ mm}$$

$$\frac{D_{15}(\text{filter})}{D_{85}(\text{base})} < 5, \text{ so } D_{15}(\text{filter}) < 5(1.4) = 7.0 \text{ mm}$$

$$\frac{D_{15}(\text{filter})}{D_{15}(\text{base})} > 5, \text{ so } D_{15}(\text{filter}) > 5(0.16) = 0.8 \text{ mm}$$

With respect to the base material, the filter must satisfy

$$0.8 \text{ mm} < D_{15}(\text{filter}) < 6.4 \text{ mm}$$

$$D_{50}(\text{filter}) < 20 \text{ mm}$$

3. Considering the riprap and filter material

$$\frac{D_{50}(\text{riprap})}{D_{50}(\text{filter})} < 40, \text{ so } D_{50}(\text{filter}) > \frac{230}{40} = 5.8 \text{ mm}$$

$$\frac{D_{15}(\text{riprap})}{D_{15}(\text{filter})} < 40, \text{ so } D_{15}(\text{filter}) > \frac{75}{40} = 1.9 \text{ mm}$$

$$\frac{D_{15}(\text{riprap})}{D_{85}(\text{filter})} < 5, \text{ so } D_{85}(\text{filter}) > \frac{75}{5} = 15 \text{ mm}$$

$$\frac{D_{15}(\text{riprap})}{D_{15}(\text{filter})} < 5, \text{ so } D_{15}(\text{filter}) < \frac{75}{5} = 15 \text{ mm}$$

With respect to the riprap layer, the filter must satisfy

$$1.9 \text{ mm} < D_{15}(\text{filter}) < 15 \text{ mm}$$

$$D_{50}(\text{filter}) > 5.8 \text{ mm}$$

$$D_{85}(\text{filter}) > 15 \text{ mm}$$

4. Figure 15.7 shows the limits of the filter material with respect to both the base and riprap material. The gradation curves for the filter layer have been extrapolated somewhat arbitrarily beyond the computed points. The ranges of suitable filter for both the riprap and the base have been crosshatched; any filter material that falls within the region where the

crosshatching overlaps will meet the criteria for both the riprap and the base material and will thus be suitable for the filter blanket.

5. The thickness of the filter layer can be determined for an assumed value of D_{max} of the filter. The thickness of the filter is equal to D_{max} if more than 9 inches. If filter material was available that has a gradation shown by the dotted line in Figure 15.7 then D_{max} = 75 mm ≅ 3 inches. Therefore use 9 inches.

15.5.3 Plastic Filter Cloth Design Example

It is desired to design a plastic filter cloth suitable for application to base material having the gradation shown in Figure 15.7. Since minimal fines are present in the base material the design criteria are:

$$\frac{\text{85 percent size of material (mm)}}{\text{EOS (mm)}} > 1$$

Open area not to exceed 36 percent.

<u>Solution</u>

1. From Figure 15.7, D_{85} = 1.6 mm
2. A filter cloth should be chosen that has:
   Equivalent Opening Size (EOS) < 1.6 mm
   4 percent $\leq$ Open Area $\leq$ 36 percent
3. A layer of gravel should be placed over the filter cloth to provide protection during riprap placement.

15.5.4 Design Example for Riprap Bend Protection

Evaluate riprap needed in a bend having a radius of curvature, r_c, equal to 100 ft. The bend is part of the channel designed in Section 5.5.1.

From Figure 6.8 for

$$\frac{r_c}{w} = 4.5$$

the ratio of shear stress on the outside of a bend to the mean shear stress is 1.63. Multiplying this factor by the $V^2/R^{0.33}$ values for a straight channel reach yields:

$$1.65 \frac{V^2}{R^{0.33}} = 1.65 \frac{(5.3^2)}{(2.05)^{0.33}} = 36.6$$

Table 6.4 indicates that Type L riprap with $K_m = 9$ in is required. As this riprap gradation is identical to that specified for the straight reaches, no additional protection is required in the bend.

15.6 References

Chow, V. T., 1959, Open-Channel Hydrualics, McGraw-Hill, New York, NY, 680 pp.

Lotler, G. K., 1933, "Considerations on hydraulic design of channels with different roughness of walls," Transactions, All-Union Scientific Research Institute of Hydraulic Engineering, Leningrad, Vol. 9, pp. 238-241.

XVI. DESIGN OF DROP STRUCTURES

The use of drop structures permits adjustment of a channel slope which is too steep for design conditions. In other words, drop structures can be used to achieve the required dynamic equilibrium slope. The structures can be either vertical drops or sloped drops and can range in complexity from simple rock riprap type structures to concrete structures with baffled aprons and stilling basins. For the range of discharges and velocities typically expected on a surface mine site, and considering the construction techniques typically employed, only the design of rock riprap structures is covered in this manual.

16.1 Site Selection

The structure should be located in a reasonably straight section of channel with neither upstream nor downstream curves within 100 to 200 feet of the structure. Where large quantities of excavation are required to reduce the existing profile of the land surface to the equilibrium slope, drop structures should be located to minimize excavation costs.

16.2 Type of Structure

The type of rock riprap drop structure recommended can be classified as a loose rock sloped drop structure. Loose rock drop structures are easily constructed by mechanized equipment and generally result in relatively low cost installations. Sloped drops can also be designed to fit the channel topography needs with little difficulty. The design of this structure will be presented in the remainder of this chapter.

16.3 Height, Number, and Spacing of Structures

The height of drop structures is usually governed by a quantitative assessment of the available construction material, required excavation quantities, and cost. Small drop structures are usually more economical than large structures; however to account for the same overall drop in a given channel reach more structures would be required. This may increase the construction costs depending on the location of available sites and the rock riprap and excavation quantities required at each site. Consequently, a decision on the height of the structures must consider all possible alternatives.

The number of drop structures required to achieve the equilibrium slope is based on analyzing the total drop height required and the height of the individual drop structures. The combined height of all the drop structures must equal the total drop height.

Spacing of the drop structures is a function of site availability. They should be spaced to minimize the excavation and construction costs.

16.4 Local Scour

The velocity of flow on the downstream side of a drop structure can be quite high, creating the potential for local scour at the toe and possible undercutting of the structure. Consequently, protection is required in a transition length between the steep slope of the riprapped drop structure and the mild sloped channel. Protection is also required at the entrance to the drop structure due to the drawdown and increased velocity that results as flow transitions from a mild to a steep slope. For simplicity, the length of protection estimated for the more critical exit section is also specified for entrance protection. Model studies and field observations of sand-bed channels conveying relatively small discharges indicate the depth of local scour is generally no larger than the uniform flow depth computed for the downstream channel section. Therefore, a general rule for the length of protection required is that transition length should be equal to five times the downstream uniform flow depth; however, in no case should this be less than 15 ft. Additional energy dissipation measures are usually not required at the base of rock riprap drop structures since the flow velocity is typically not very large. The size of riprap protection, gradation and thickness will be the same as that discussed for the drop structure in the next section.

16.5 Design of Rock Riprap Drop Structures

A riprap drop structure effectively acts as a steep slope conveyance. Therefore, the design of riprap for drop structures is based on the procedures provided in Chapter V of Part 1. Criteria given there regarding gradation, thickness and filter layers should be followed in designing loose rock, sloped drop structures.

Five sets of design curves (Figures 5.3-5.7) were developed to simplify riprap design for steep conveyance channels. They are also utilized to design the riprap for the drop structures. The design curves were developed for trapezoidal channels with two to one side slopes. However, for drop structure design these curves can also be used for three to one side slopes. The results obtained are reasonably accurate and the design becomes conservative in nature.

Once the height of the drop structure has been determined, the overall design procedure can be initiated. Given the design discharge, the graphs in Figures 5.3-5.7 are entered to determine the median riprap necessary to stabilize a given slope. The slope of the drop structure will be selected based on an evaluation of construction and excavation costs as well as riprap availability. Local scour depths upstream and downstream of the structure will be assumed equal to the normal depth of flow in the downstream channel. Knowing the design slope and the downstream normal flow depth, the length of the structure will be

$$L = L_u + L_s + L_d \tag{16.1}$$

where L_u = 5 x downstream normal flow depth, d (15 ft minimum)

$L_s = h/S$

L_d = 5 x downstream normal flow depth, d (15 ft minimum)

h = the height of the drop.

S = slope of the drop structure (ft/ft)

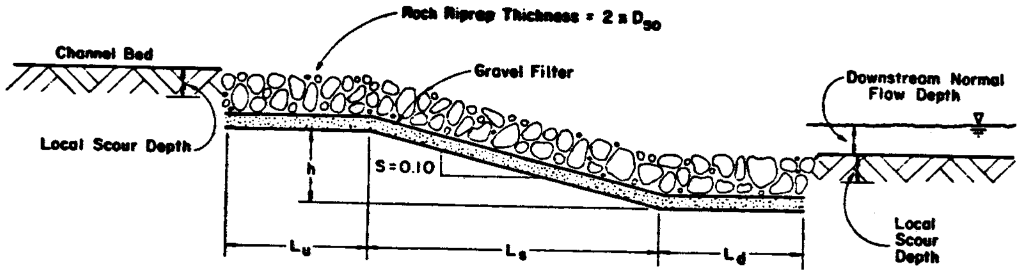
The freeboard requirements will be based on a steep slope design with riprap lining. With that assumption, Table 4.4 gives c_{fb} equal to 1.0 and the equation for freeboard becomes

$$F.B. = d + \frac{1}{2}\Delta Z + \frac{1}{2}h_a \qquad \text{Equation 12.2}$$

The depth of flow, d, in this equation is given in Figures 5.3-5.7. A definition sketch and cross section of the drop structure design is given in Figure 16.1.

16.6 Summary of the Design Procedure for Drop Structure

1. Based on an evaluation of feasible site locations, available construction material, excavation quantities required, and drop height, determine the height, number, and spacing of drop structures.
2. Enter Figures 5.3-5.7 to determine the slope of the drop structure and the D_{50} of the riprap lining. The decision as to which slope and D_{50}



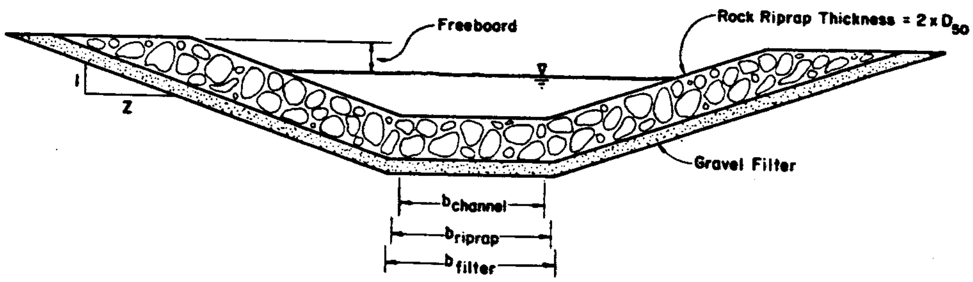


Figure 16.4. Drop structure design.

to use must consider riprap availability and construction and excavation costs.

3. Determine riprap gradation and thickness (Section 5.2.5 and 5.2.6).
4. Evaluate filter requirements (Sections 5.2.7).
5. Evaluate downstream normal flow depth.
6. Determine length of the drop structure (Equation 16.1).
7. Evaluate freeboard requirements (Equation 12.2).

16.7 Design Example

This example details the procedure for designing a rock riprap drop structure. The drop structures will be designed for a trapezoidal channel (3:1 sideslopes) and will account for a ten-foot drop height. The following information is provided for the trapezoidal channel:

$Q = 200$ cfs	$D_{50} = 0.1$ in.
$b = 6$ ft	$D_{85} = 0.27$ in.
$z = 3$	$D_{15} = 0.036$ in.
$n = 0.025$	$S = 0.001$ (bed slope)

1. Two five-foot drop structures spaced according to the criteria of Section 16.3 will account for a drop height of ten feet.
2. From Figure 5.3, the following information is obtained:

$S = 0.10$	$S = 0.30$	$S = 0.50$
$D_{50} = 1.72$ ft	$D_{50} > 3$ ft	$D_{50} > 3$ ft

Choose $S = 0.10$

$D_{50} = 1.72$ ft   Use $D_{50} = 1.75$ ft (Table 5.2)

3. Riprap gradation

$$D_{max} \leq 1.25\, D_{50} = 26 \text{ in.}$$

$$D_{10-20} \cong \frac{D_{50}}{3.5} = 6 \text{ in.}$$

Riprap thickness = $1.25 \times D_{50} = 26$ inches

4. Evaluate filter requirements as previously illustrated
5. Evaluation of downstream flow depth is based on

$Q = 200$ cfs $\qquad b = 6$ ft

$z = 3 \qquad n = 0.025$

$S = 0.001$

Solve Manning's equation

$$Q = \frac{1.49}{0.025} A R^{2/3} S^{1/2} \qquad \text{Equation 4.14}$$

by first entering the appropriate chart in Appendix C and obtaining d = 4.0 feet. Using this value as the initial estimate, the iterative technique illustrated in Section 12.9 provides d = 3.7 feet.

6. Length of drop structure

$$L = L_u + L_s + L_d \qquad \text{Equation 16.1}$$

where $L = L_u = 5d_d = 5\ (3.7) = 18.5$ ft

$L_s = 5/0.10 = 50$

Therefore $L = 18.5 + 50 + 18.5 = 87$ ft

7. Freeboard requirements

$c_{fb} = 1.0$

From Figure 5.4, the depth of flow in the drop structure is 1.1 feet. Therefore

$$F.B. = C_{fb}\, d + \frac{1}{2} \Delta Z + \frac{1}{2} h_a \qquad \text{Equation 12.2}$$

$$= 1.1 + 0 + 0$$

$$= 1.1 \text{ feet}$$

Figures 16.2 and 16.3 present the design of the rock riprap drop structure. The design of a granular filter layer would be accomplished according to the procedures presented in Section 5.2.7.

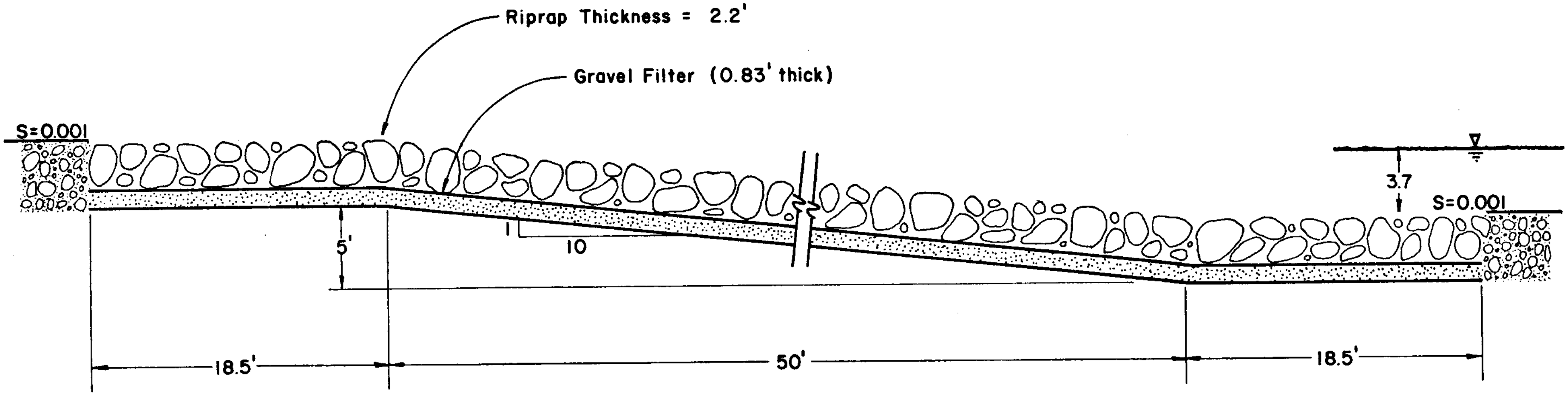


Figure 16.2. Design of Drop Structure.

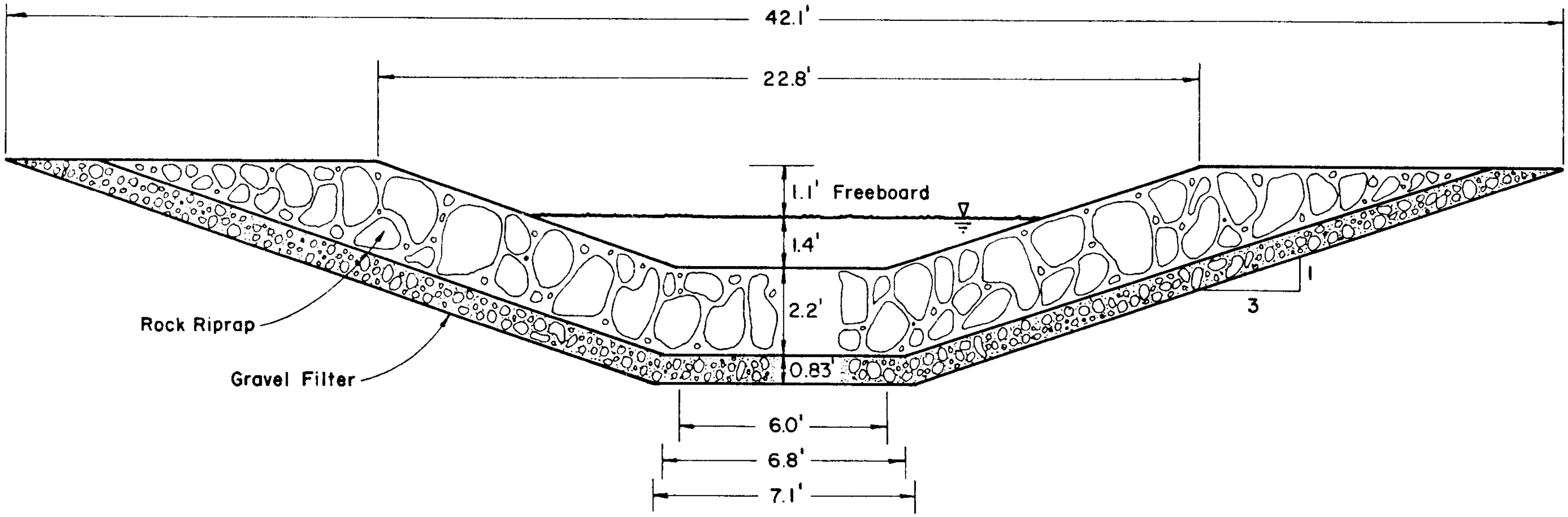


Figure 16.3. Drop structure design cross-sectional view.

XVII. TRANSITION DESIGN

17.1 Basic Considerations

Transition design for diversion channels and relocations in sandy soils will be identical to the procedures given in Chapter VII, Part 1. However, an additional consideration in sandy soil regions is the need for dikes. Dikes are often needed to prevent the flow of a meandering stream from circumventing the inlet transition to the diversion channel. Information presented below details the design of dikes for this purpose.

17.2 Design of Dikes

The first step in the design of a dike is to consider the river pattern. Determination of channel sinuosity and consideration of Figure 13.2 can aid in this evaluation. If it is determined to be a meandering stream, a dike will be needed and should be designed in conjunction with the inlet transition. The design of the dike will include calculations to determine length, height, width, and embankment side slope and protection.

17.2.1 Length

The length of the dike must consider the following factors: meander width of the stream, location of the inlet transition, projection angle of the dike and degree of protection desired. If maximum protection is desired and the inlet transition is located in the center of the meander width, the dike must be extended to allow for one-half the meander width. Should the inlet transition be located near the meander bend, the dike must be extended to allow for the entire meander width. Originating at the upstream end of the inlet transition, the dike should be projected at an angle not to exceed 45 degrees with the centerline of the inlet transition. It may be composed of a series of straight segments or one straight segment where practical. Figure 17.1 illustrates the relationship between dike length, projection angle and meander width.

Although the lengths may vary, dikes are generally placed on both sides of the inlet transition. There are instances, however, when one or both of the dikes may be eliminated. For example, after the point of diversion, the channel may be aligned perpendicularly to the river and eliminate the need for one of the dikes. If this is the case, proper consideration must be given to concentrating the flow and bringing it in at selected locations. Addition-

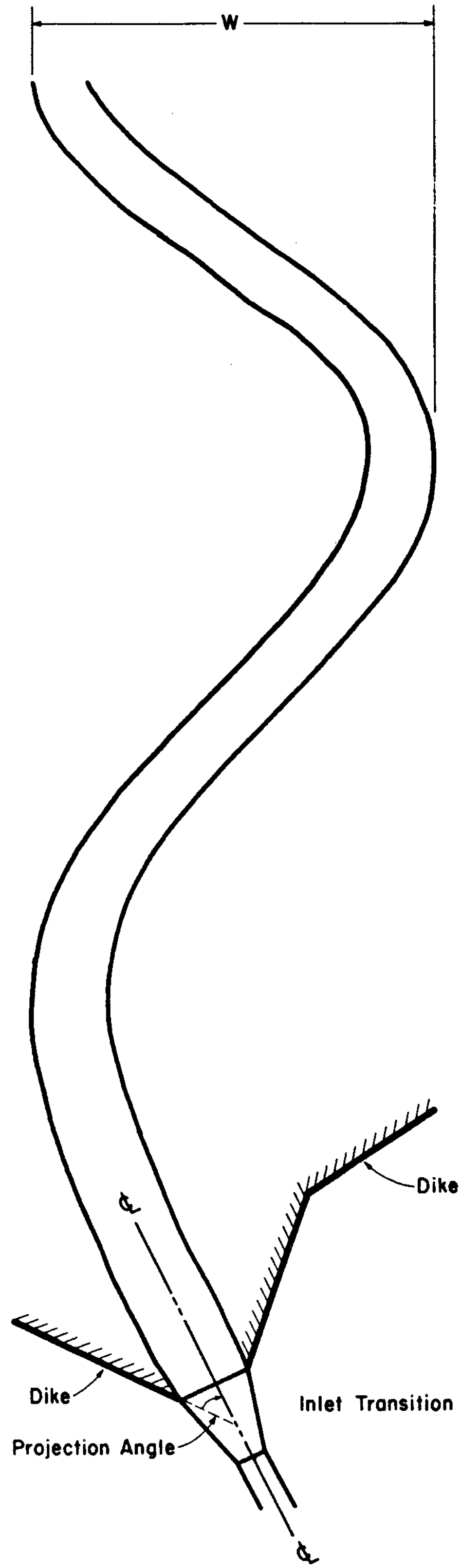


Figure 17.1. Relationship between dike length, projection angle and meander width.

ally, geological controls, such as rock outcroppings, may restrict the meander movement and eliminate the need for one or both of the dikes.

17.2.2 Height

The minimum elevation of the top of the dike is recommended to be three feet above the water surface in the upstream end of the inlet transition with the diversion channel flowing at design depth. At no time should the dike be less than 2.0 feet above design flood stage.

17.2.3 Width

The minimum top width of the dike shall not be less than

$$W = \frac{(h+35)}{5} \tag{17.1}$$

where h is the height, in feet, of the dike as measured from the downstream toe.

17.2.4 Sideslope

The side slopes of the dike shall not be less than 1:5 (vertical to horizontal), with neither slope steeper than 1:2. Refer to Section 12.2 for an evaluation of stable side slopes in sandy soils.

17.2.5 Protection

To stabilize the upsteam side slopes of the dike, it is recommended that riprap protection be provided. The riprap will be designed according to the procedures discussed in Chapter VI. The values for the flow velocity (V) and hydraulic radius (R) will be obtained from the natural river and used in the parameter $V^2/R^{0.33}$. The riprap thickness will be 2 x D_{50} above the ground surface, and doubled for the portion located below the ground surface. Since the dike is constructed on the existing ground surface, the depth of riprap must account for the depth of flow of the stream plus the local scour depth. In this case a maximum scour depth equal to the depth of flow is recommended. The riprap depth becomes

$$d_{riprap} = d_{flow} + d_{scour} = 2 \times d_{flow} \tag{17.2}$$

A schematic diagram of the dike cross section is provided in Figure 17.2.

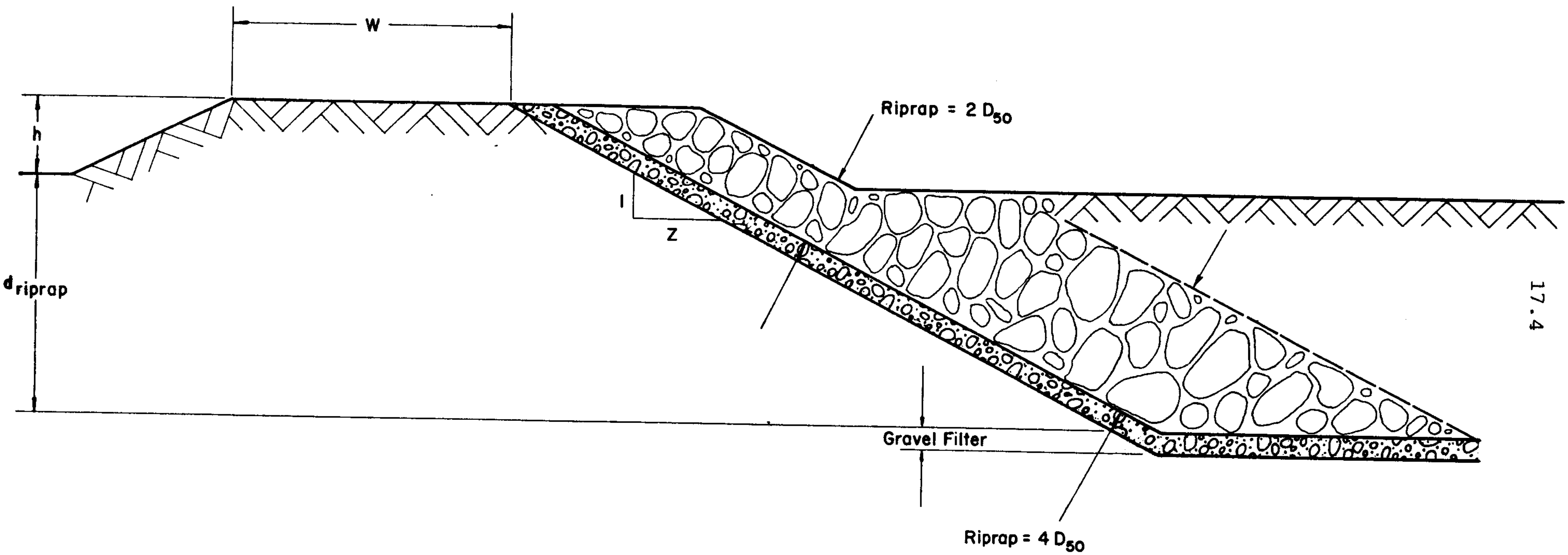


Figure 17.2. Schematic diagram of dike cross section.

17.3 Design Examples

17.3.1 Example of Transition Design

The following example illustrates the transition design procedure. It is required to design a transition between two trapezoidal channels of 3:1 side slope with bottom widths. Given a flow rate of 150 cfs, the characteristics of each channel are:

Upstream Channel Section

Natural smooth earth channel
Base width $b = 20$ ft
$A = 37.5$
$d = 1.5$ ft
$S_o = 0.003$

Downstream Channel Section

Riprap lined
Base width $b = 6$ ft
$A = 30\ \text{ft}^2$
$d = 2.3$ ft
$S_o = 0.008$

1. Compute the flow velocity in each channel

$$V_{upstream} = \frac{Q}{A} = \frac{150}{37.5} = 4 \text{ ft/sec}$$

$$V_{downstream} = \frac{Q}{A} = \frac{150}{30} = 5 \text{ ft/sec}$$

2. Compute change in water surface profile (Equation 7.5a)

$$\Delta W.S. = 1.15\left(\frac{5^2}{2g} - \frac{4^2}{2g}\right) = 0.16 \text{ ft}$$

3. Compute necessary change in bed elevation (ΔB.E.) between transition entrance and exit (Equation 7.6a)

$$\Delta B.E. = d_2 - d_1 + \Delta W.S.$$

$$\Delta B.E. = 2.3 - 1.5 + 0.16 = 0.96 \text{ ft}$$

4. Compute length of transition using maximum included angle of convergence equal to 25° for the water surface.

Upstream water surface width:

$30' + 2(1.5)(3) = 29.0$

Downstream water surface width:

$6 + 2(2.3)(3) = 19.8$

Therefore, from geometry

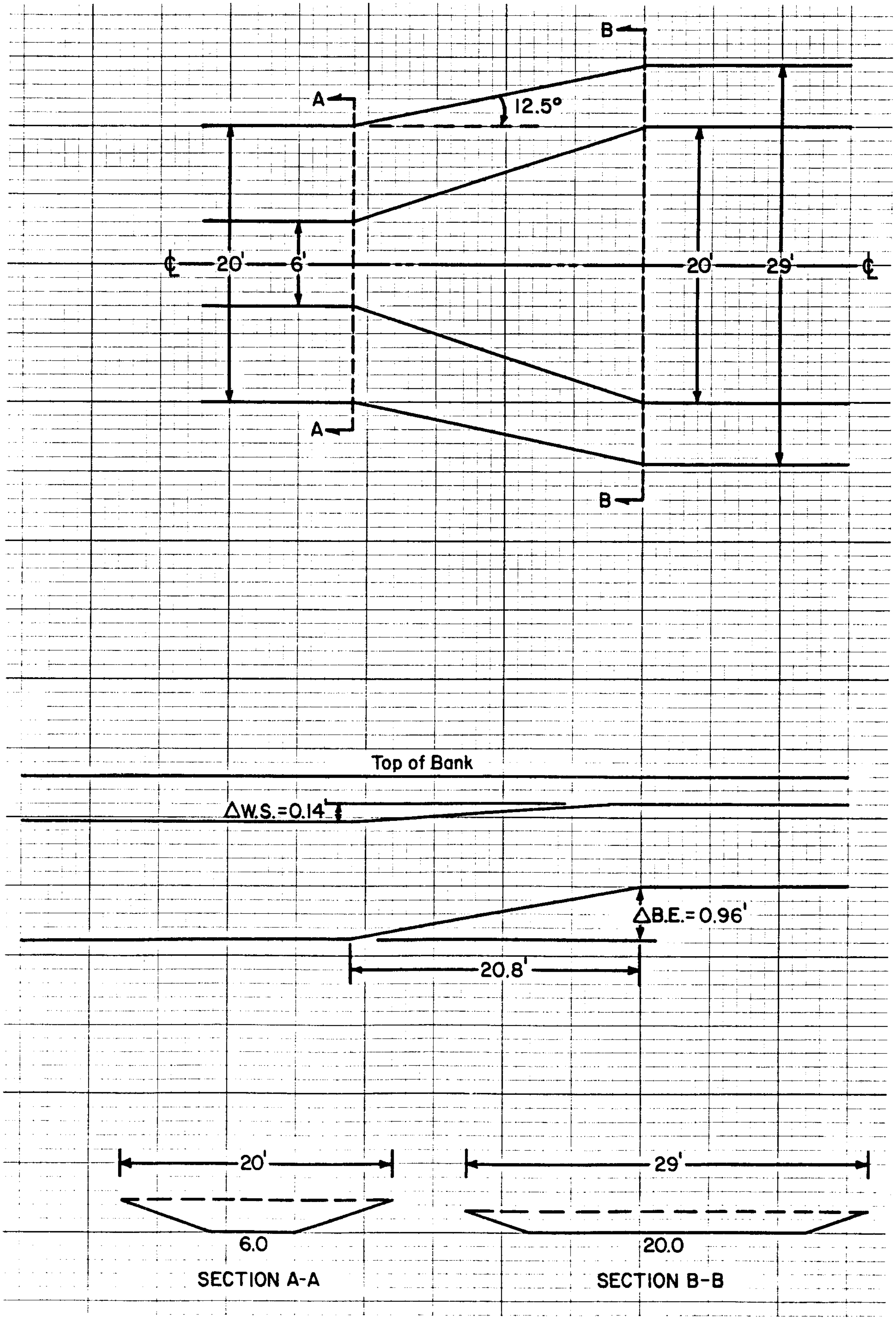


Figure 17.3. Transition design.

$$\tan(12.5) = \frac{(29.0/2 - 19.8/2)}{L}$$

$$L = 20.8 \text{ ft}$$

5. Determine the slope of the transition.

$$\frac{\Delta B.E.}{L} = \frac{0.94}{20.8} = 0.045$$

The Froude number is

$$Fr = \frac{V}{\sqrt{gd}} = \frac{Q/A}{\sqrt{gd}} = \frac{4}{\sqrt{32.2(1.5)}} = 0.57$$

Therefore, design the riprap transition according to the procedures established in Chapter VI. Figure 17.3 illustrates the design.

17.3.2 Example of Dike Design

The following example illustrates the design of a dike at the entrance to an inlet transition. Given a flow rate of 250 cfs, the characteristics of the upstream channel and inlet transition are:

Upstream Channel Section	Inlet Transition
Meander width $w = 200$ ft	Top of Bank = 1.5 ft above water surface at entrance
$A = 37.5$ ft^2	
$d = 1.5$ ft	
$R = 1.27$	Location: center of meander width
$V = 4$ ft/sec	

Assume maximum protection desired.

1. Determine the length of each dike by the scaled drawing in Figure 17.4. The projection angle of the dike is equal to 45°

   East Dike = 120 ft

   West Dike = 147 ft

2. Calculate the height of the dike

   Minimum elevation = 3 ft above water surface

   Height of dike = 3 - 1.5 = 1.5 ft < 2.0 ft

   Therefore, use 2.0 ft

3. Evaluate dike width using Equation 17.1

$$W = \frac{(h + 35)}{5} = \frac{2 + 35}{5} = 7.4 \text{ ft}$$

4. Select a side slope of 2:1

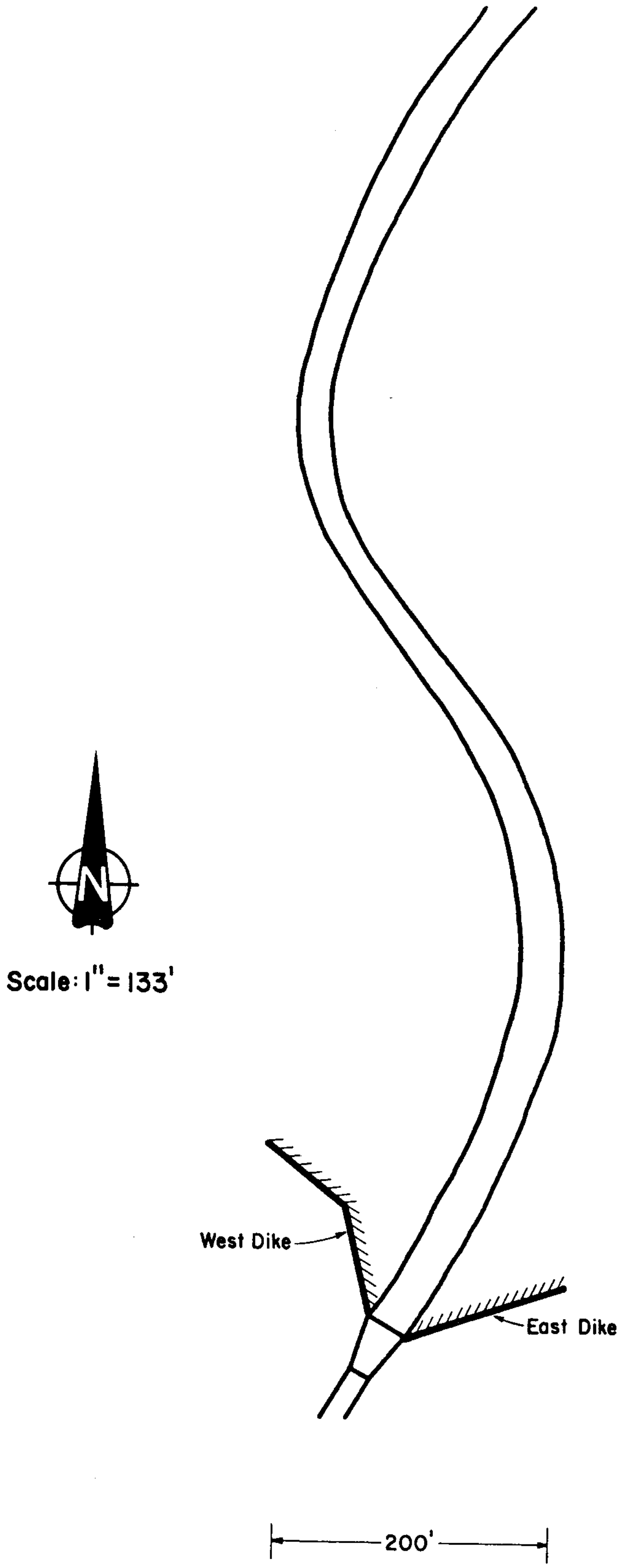


Figure 17.4. Scale drawing of meander width and dike length.

5. Riprap protection is determined using

$$\frac{V^2}{R^{0.33}} = \frac{4^2}{1.27^{0.33}} = 14.8$$

According to Table 6.4, Type VL riprap should be used. From Table 6.5 its gradation is

$$D_{100} = 9 \text{ inches}$$

$$D_{50} = 6 \text{ inches}$$

$$D_{10} = 2 \text{ inches}$$

Thickness of the riprap protection will be

$$2D_{50} = 12 \text{ inches (above ground)}$$

$$4D_{50} = 24 \text{ inches (below ground)}$$

A gravel filter designed according to the procedures of Chapter VI would also be designed and utilized.

The depth of riprap is

$$d_{riprap} = d_{flow} + d_{scour} \; 2 \times d_{flow} \tag{17.2}$$

$$= 2 \times 1.5 = 3.0 \text{ feet}$$

Figure 17.5 illustrates the cross-sectional design of the dike.

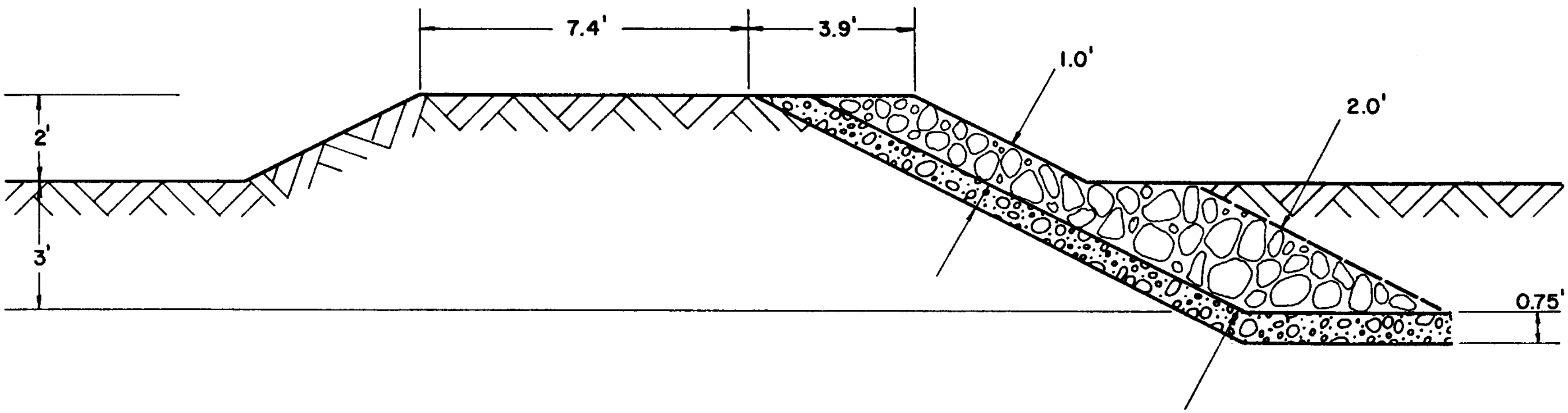


Figure 17.5. Cross-sectional view of dike.

XVIII. ROCK DURABILITY CONSIDERATIONS FOR WESTERN COAL REGIONS

18.1 General

The durability or weatherability of rock is a critical factor whether a diversion is being constructed with a channel in unlined bedrock or lined with riprap. To facilitate proper choice and use of durable riprap material, a rock durability evaluation procedure was described in detail in Section 8.2. It is essentially a threefold procedure based primarily on field observations, field tests, and selected laboratory tests. The recommended procedure is based on conservative values designed for a probable lengthy in-service performance of riprap.

The durability test procedure was designed predominantly for sedimentary rock types which overlie coal seams in the eastern coal region. Because sedimentary rocks are also the associated rock types in western coal regions, the procedure given in Part 1 is a suitable and adaptable method for evaluating rock durability in western U.S.

18.2 Additional Considerations Applicable to Western Coal Regions

The climatic conditions prevailing within the western coal areas must be taken into consideration when evaluating the choice of tests, topographic expression of rock types, and weathering characteristics. For example, freeze-thaw tests may be very relevant to assessing durability in the northern region, while wet-dry or abrasion tests may have more bearing in semi-arid regions. Consultation with mining engineers provide information on tests which have been shown to be useful under certain climatic conditions for a particular mining area.

In certain locales, igneous and metamorphic rocks may be available as a potential riprap source, as well as the sedimentary rock types described in Part 1. Although transportation costs may make the use of intrusive rocks economically infeasible, they are considered here in the event that a local source is available and if sedimentary rocks at a particular site are not suitable as riprap.

Intrusive rock types can be evaluated using the Flow Chart given in Figure 8.1 of Part 1 (e.g. the "Limestone" Flow Chart), but particular attention should be given to fracture patterns observed in the field and in-service performance.

The fractured block dimensions of the rock outcrops must be greater than the riprap design dimensions to be suitable. In other words, for a given channel size and flow, riprap material should not exhibit discontinuities with spacings less than the predetermined riprap dimension. Fracture patterns are more often spatially variable compared to sedimentary rocks, so attention should be given to fracture density and distribution when observing outcrops.

A key for assessing weathering characteristics and fracturing of intrusive rock types is defined by Clayton and Arnold (1972) (Table 18.1). The guideline is a practical tool for classifying significant factors relevant to degrees of weathering. The durability of igneous or metamorphic rocks cannot be defined by this information alone, but the seven classes may be of value when accompanied by the field flow charts to assess in-service performance. Classes 1 and 2 (Table 18.1) are probably suitable rock types, depending on if acceptable results are obtained from the flow chart analysis.

As mentioned previously and stressed in Part 1, in-service performance is an important consideration and is particularly valuable in relating field and laboratory tests to the actual service life of riprap channels. It is important in the western coal regions to consider each rock type and its expressed landform on a site specific basis. For example, the slope and aspect of in situ rock types in the northern montane regions can greatly modify its weatherability; therefore, a conservative approach to assessing potential riprap material is to consider the most weathered rock exposure as representative of the in-service performance of a certain rock type.

A final consideration is that laboratory data, when required, must be used in conjunction with the field data to make a judgement on the quality of the potential riprap material. No one parameter by itself is adequate to qualify a particular rock type but all parameters, qualitative and quantitative, must be considered together (Lienhart and Stransky, 1981).

18.3 Summary

In summary, durability tests of some type are necessary to adequately evaluate rocks suitable for use as riprap. The durability evaluating procedure as described in Part 1 is applicable to the western coal regions. It should be used to methodically evaluate durability using field and laboratory testing during the mining and reclamation stages of a mining operation. Attributes of a rock type to be identified include outcrop massive, cliff-

Table 18.1. Classes of Rock Weathering (from Clayton and Arnold, 1972).

Class 1, Unweathered Rock.--Unweathered rock will ring from a hammer blow; cannot be dug by the point of a rock hammer; joint sets are the only visible fractures; no iron stains emanate from biotites; joint sets are distinct and angular; biotites are black and compact; feldspars appear to be clear and fresh.

Class 2, Very Weakly Weathered Rock.--Very weakly weathered rock is similar to class 1, except for visible iron stains that emanate from biotites; biotites may also appear "expanded" when viewed through a hand lens; feldspars may show some opacity; joint sets are distinct and angular.

Class 3, Weakly Weathered Rock.--Weakly weathered rock gives a full ring from a hammer blow; can be broken into "hand-sized" rocks with moderate difficulty using a hammer; feldspars are opaque and milky; no root penetration; joint sets are subangular.

Class 4, Moderately Weathered Rock.--Moderately weathered rock may be weakly spalling; except for the spall rind, if present, rock cannot be broken by hand; no ring or dull ring from hammer blow; feldspars are opaque and milky; biotites usually have a golden yellow sheen; joint sets indistinct and rounded to subangular.

Classes 5, 6, and 7; Moderately Well Weathered to Very Well Weathered Rock.--Can be broken by hand; feldspars are powdery and weathered to clay minerals; biotite appears silver or white; joints are weakly visible, well-rounded or hard to identify; root penetration within fractures or throughout rock mass.

forming characteristics, grain size, cementation, ease of manual breakdown, and discontinuities. Because climatic conditions are varied over the western coal regions, the effects of climate should be carefully assessed at each mine prior to field investigations and selection of laboratory tests (if required). Also, igneous and metamorphic rocks may be available locally and may prove to be a viable riprap source if sedimentary rocks are not suitable. The durability evaluation flow charts can be used to assess these rock types with special consideration given to fracture density and weathering characteristics.

18.4 References

Clayton, J. L. and J. F. Arnold, 1972, Practical grain-size fracturing density and weathering classification of intrusive rocks of the Idaho Batholith, U.S. Forest Service, Gen. Tech. Report INT-2, 17 pp.

Lienhart, D. A., T. E. Stransky, 1981, Evaluation of potential sources of riprap and armor stone - methods and considerations, Bull. Ass. of Engineering Geologists, Vol. 18, No. 3, pp. 323-332.

This page intentionally left blank.

XIX. COMPREHENSIVE EXAMPLE

Provided in this chapter is an application of the design concepts previously discussed. These concepts will be used to solve a drainage problem associated with a proposed surface mine located in a semi-arid western state.

The scope of the problem involves determination of the surface water and sediment runoff for the appropriate design storm. Additionally, it will involve provision of conceptual designs of new structures to meet requirements regarding water and sediment runoff. A major portion of this project includes the design of a diversion to transfer the flow from North Battle Creek, around the proposed mining operations and back into the original creek (see Figure 19.1). Diversion of North Battle Creek is required because the initial box cut and spoils will block the present drainage systems.

For the purposes of this design example, North Battle Creek is considered an ephemeral creek and the diversion will be permanent. The dominant discharge will be generated from the ten-year 24-hour storm and is determined to be 250 cfs.

The solution to the drainage problem and the diversion of North Battle Creek involves computation and design of the following.

1. Channel alignment
2. Hydraulic conditions of natural and diversion channel
3. Equilibrium slope of diversion channel
4. Channel lining
5. Design depth of diversion channel
6. Drop structures
7. Channel entrance
8. Channel outlet

A step-by-step procedure is presented to aid the user in understanding and using the design concepts. All information pertinent to the diversion design will be provided.

19.1 Design of Diversion Channel Using Equilibrium Slope Concept

The physical layout of the system is given in Figure 19.2. The step-by-step procedure given in Section 14.4 will be followed.

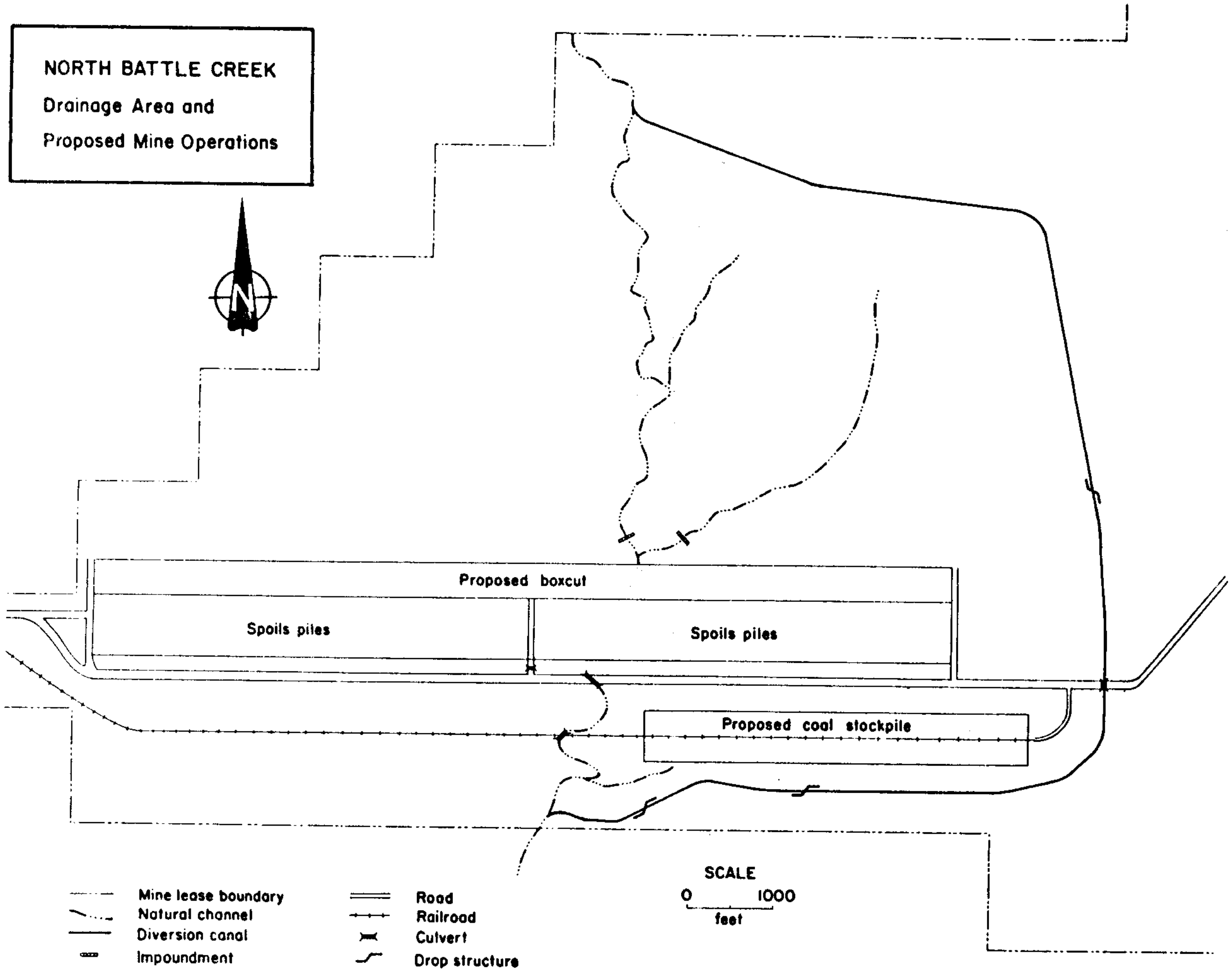


Figure 19.1. Schematic diagram of proposed mining operation.

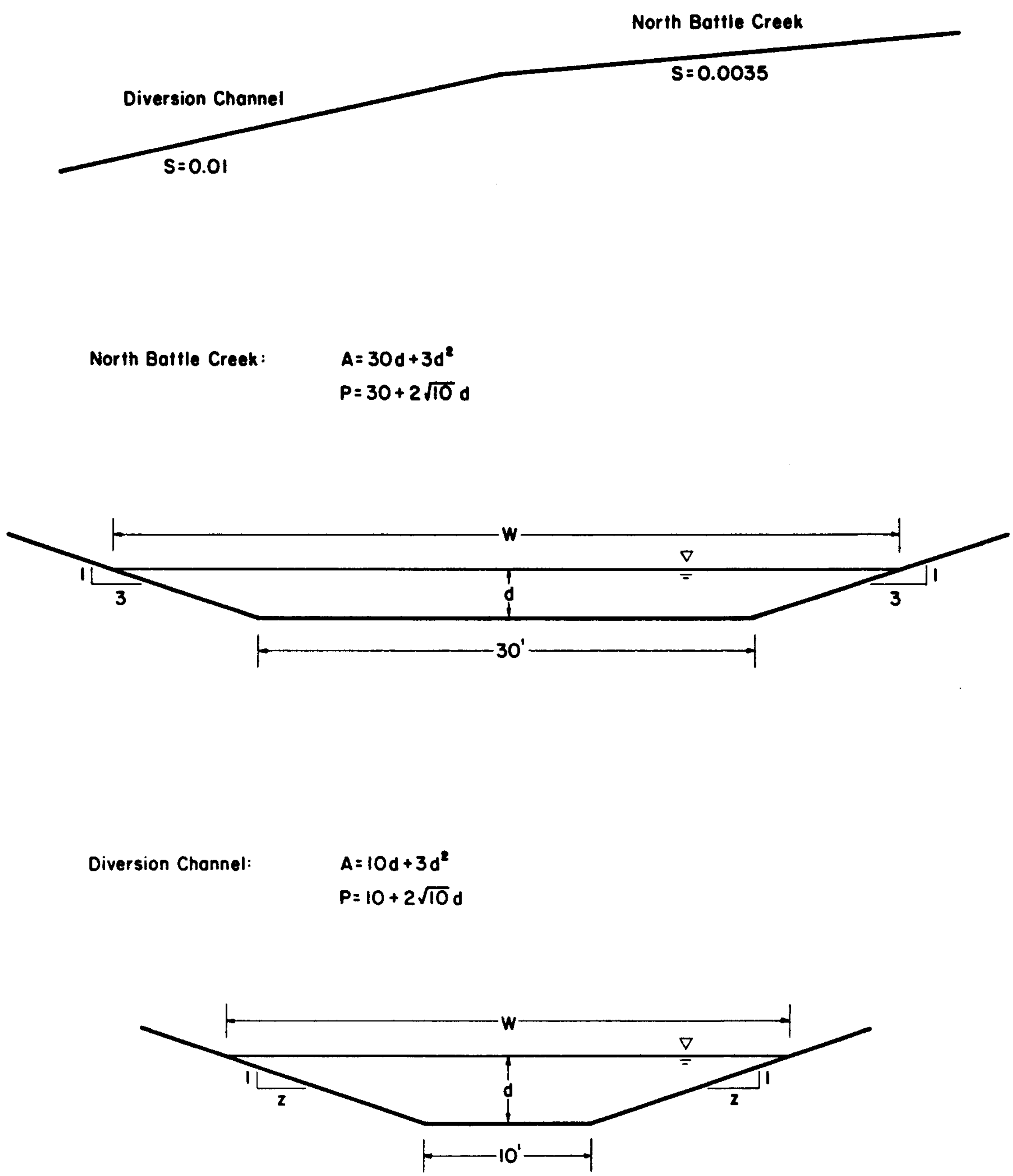


Figure 19.2. Physical layout of comprehensive example.

<u>Steps 1, 2 and 3</u>. Assemble basic information required for design.

	Natural Channel	Diversion Channel
Dominant discharge	250 cfs	250 cfs
Sediment size distribution	D_{85} = 1.0 mm	D_{85} = 1.0 mm
	D_{50} = 0.5 mm	D_{50} = 0.5 mm
	D_{15} = 0.25 mm	D_{15} = 0.25 mm
Channel resistance (Manning's n)	0.022	0.022 (initial estimate)
Side slopes	3:1	2:1
Channel slope	0.0035	0.01

Select alignment of diversion channel. The diversion channel must be within the lease boundary and also circumvent the proposed mining activities. In addition, an alignment must be chosen that will minimize the excavation quantities required for the diversion channel. It is important to avoid any location that would require construction of the channel above the existing land surface as stability problems would result due to the sandy nature of the soil. Based on these considerations, the channel alignment was selected as indicated on Figure 19.1.

<u>Step 4</u>: Compute the hydraulic conditions for the natural channel assuming normal depth at $Q = 250$ cfs. Solving the Manning equation by an iterative technique reveals

$$Q = \frac{1.49}{n} A R^{2/3} S^{1/2} \qquad \text{Equation 4.14}$$

d	A	P	R	$R^{2/3}$	Q
1.3	44.1	37.8	1.17	1.11	196
1.5	51.8	39.5	1.32	1.20	250

The hydraulic conditions are

$d = 1.5$ ft $A = 51.8$ ft^2 $R = 1.32$

$V = 4.8$ ft/sec $W = 39.1$ ft

$d_h = 1.33$ ft $F_r = 0.69$

<u>Step 5</u>: Calculate the upstream sediment supply using Figure 13.3. With $V = 4.80$ ft/sec and $D_{50} = 0.50$ mm,

$q_s = 9 \times 10^{-3}$ cfs/ft

$(Q_s)_{in} = W\, q_s$

$= 39.1 \times (9 \times 10^{-3})$

$= 0.35$ cfs

Step 6: Based on a trapezoidal channel with 2:1 side slopes, determine the equilibrium slope. With $S = 0.0028$, the hydraulic conditions are

d	A	P	R	$R^{2/3}$	Q
2.8	43.7	22.52	1.94	1.56	244
2.85	44.7	22.7	1.97	1.57	252

d = 2.85 ft W = 21.4 ft R = 1.97

V = 5.64 ft/sec d_h = 2.09 ft

A = 44.7 ft^2 F_r = 0.69

From Figure 13.3,

$q_s = 2 \times 10^{-2}$ cfs/ft

$(Q_s)_{out} = q_s\, W = (2 \times 10^{-2})(21.4) = 0.43$ cfs > 0.35 cfs

Note that $(Q_s)_{out}$ is greater than $(Q_s)_{in}$ and the channel will degrade. Decreasing the cnannel slope will decrease $(Q_s)_{out}$. Try $S = 0.0025$.

d	A	P	R	$R^{2/3}$	Q
3.0	48	23.4	2.05	1.62	262
2.9	45.8	23.0	1.99	1.59	246

d = 2.9 ft W = 21.6 R = 1.99

A = 45.8 ft^2 d_h = 2.12

V = 5.4 ft/sec F_r = 0.65

From Figure 13.3,

$$q_s = 1.6 \times 10^{-2} \text{cfs/ft}$$

$$(Q_s)_{out} = q_s\ W = (1.6 \times 10^{-2})(21.6) = 0.35 \text{ cfs}$$

For this slope, $(Q_s)_{in}$ equals $(Q_s)_{out}$.

Step 7. A check of the armoring potential reveals

$$\tau = \gamma RS$$
$$= 62.4 \times 1.99 \times 0.0025$$
$$= 0.31 \text{ lb/ft}^2$$

$$D_a = \frac{0.311}{(0.047(2.65-1)(62.4)}$$

$$= 0.062 \text{ ft} = 19.6 \text{ mm}$$

A comparison of this value and the size distribution of the bed material indicates armoring will not occur.

Check the Manning's n value estimated for the diversion channel. From Step 6,

$$\tau = 0.31 \text{ lbs/ft}^2$$

$$\tau V = 0.31\ (5.40)$$

$$= 1.67$$

From Figure 12.2, the bedforms present are antidunes. Checking Table 12.2 reveals that the Manning's n of 0.022 is within the range of values selected for antidunes. The calculations based on a Manning's n of 0.022 are correct and design of the channel lining will begin with an equilibrium slope of 0.0025.

19.2 Design of Channel Lining

Determination of an appropriate lining for the diversion channel will begin with the following information (Step 1, Section 15.5.1):

Diversion Channel: $V = 5.4$ ft/sec

$R = 1.99$ ft

$d = 2.9$ ft

Side Slope: 2:1

Soil Characteristics:

$D_{50} = 0.5$ mm

$$D_{100} = 1.0 \text{ mm}$$
$$D_{15} = 0.25 \text{ mm}$$
$$F_r = 0.65$$

The size of riprap will be designed according to the mild slope procedure described in Section 6.6.

Step 2: Estimate the riprap size needed to stabilize the sides of the channel.

$$\frac{V^2}{R^{0.33}} = 23$$

According to Table 6.4, Type L riprap is selected. The median diameter is

$$D_{50} = 9 \text{ inches} = 0.75 \text{ ft}$$

Step 3: Determine the Manning's n value for the riprap

$$n_1 = n_{riprap} = 0.0395\, D_{50}^{1/6}$$

$$= 0.0395\, (0.75)^{1/6} = 0.038$$

Steps 4 and 5. Determine equivalent Manning's n and hydraulic properties

$$n_2 = n_{channel} = 0.022$$

$$\frac{n_1}{n_2} = 1.73$$

The equivalent Manning's n value is obtained with the aid of Figure 15.4 as follows:

d (ft)	n_e/n_1	n_e	A (ft^2)	P (ft)	R (ft)	$R^{2/3}$	Q (cfs)
2.9	0.60	0.023	45.8	23.0	1.99	1.59	237
3.0	0.60	0.023	48	23.4	2.05	1.62	254

The equivalent Manning's n_1, n_e, is equal to 0.023 and the hydraulic characteristics are

$d = 3.0$ ft $\quad W = 22$ ft $\quad R = 2.05$ ft

$V = 5.3$ ft/sec $\quad d_h = 2.2$

$A = 48$ ft^2

Step 6: Using n_e, check the riprap size and the total sediment transport rate.

$$V = \frac{Q}{A} = \frac{254}{48} = 5.3 \text{ ft/sec}$$

$$\frac{V^2}{R^{0.33}} = 22$$

From Table 6.4, Type L riprap is selected and the size remains the same.

Step 7. From Figure 13.3, the sediment transport rate is

$$q_s = 1.5 \times 10^{-2} \text{ cfs/ft}$$

$$(Q_s)_{out} = q_s W = (1.5 \times 10^{-2})(22) = 0.33 \text{ cfs} \cong 0.35 \text{ cfs}$$

The computed sediment transport rate is approximately equal to the equilibrium slope sediment transport rate, $(Q_s)_{in}$, computed in Step 2 of Section 19.1. If these values were not equal, a new equilibrium slope would be determined based on an equivalent Manning's n of 0.023.

Step 8. Check streampower and bedforms

$$\tau V = \gamma RSV$$

$$= 62.4\ (2.05)(0.0025)(5.3)$$

$$= 1.7$$

From Figure 12.2 the bedform is antidunes. Checking Table 12.2 shows the assumed $n = 0.022$ is within the range of antidunes.

Step 9. Check Froude number.

$$F_r = \frac{5.3}{\sqrt{32.2\ (3.0)}} = 0.54$$

Therefore, use of the mild slope design procedure is valid.

Step 10: Determine the gradation and thickness of the riprap. From Table 6.6, the values for Type L riprap are

$D_{50} = 0.75$ ft = 230 mm

$D_{100} = 1.0$ ft = 305 mm

$D_{10} = 0.17$ ft = 52 mm

The maximum riprap size should not exceed

$2\ D_{50} = 460$ mm

According to the procedures in Chapter VI, the thickness of riprap is twice the median diameter or

$2 \times D_{50} = 1.5$ ft

<u>Step 11</u>: Determine if a filter is required for stability of the channel reaches. The riprap characteristics are determined graphically by plotting the recommended gradation on semi-log paper (Figure 19.3).

Riprap: $D_{50} = 230$ mm
$D_{85} = 300$ mm
$D_{15} = 75$ mm

The base material gradation is

Sand Base: $D_{50} = 0.5$ mm
$D_{85} = 1.0$ mm
$D_{15} = 0.25$ mm

To assure that a filter is needed,

$$\frac{D_{15}\ \text{(Riprap)}}{D_{85}\ \text{(Base)}} = \frac{75}{1} = 75 > 5$$

$$\frac{D_{15}\ \text{(Riprap)}}{D_{15}\ \text{(Base)}} = \frac{75}{0.25} = 300 > 40$$

Therefore a filter is necessary. The properties of the filter relative to the base material are determined by

$$\frac{D_{50}\ \text{(Filter)}}{D_{50}\ \text{(Base)}} < 40\ ,\ \text{so}\ D_{50}\ \text{(Filter)} < 40 \times 0.5 = 20\ \text{mm}$$

$$\frac{D_{15}\ \text{(Filter)}}{D_{15}\ \text{(Base)}} < 40\ ,\ \text{so}\ D_{15}\ \text{(Filter)} < 40 \times 0.25 = 10\ \text{mm}$$

$$\frac{D_{15}\ \text{(Filter)}}{D_{85}\ \text{(Base)}} < 5\ ,\ \text{so}\ D_{15}\ \text{(Filter)} < 5 \times 1 = 0.5\ \text{mm}$$

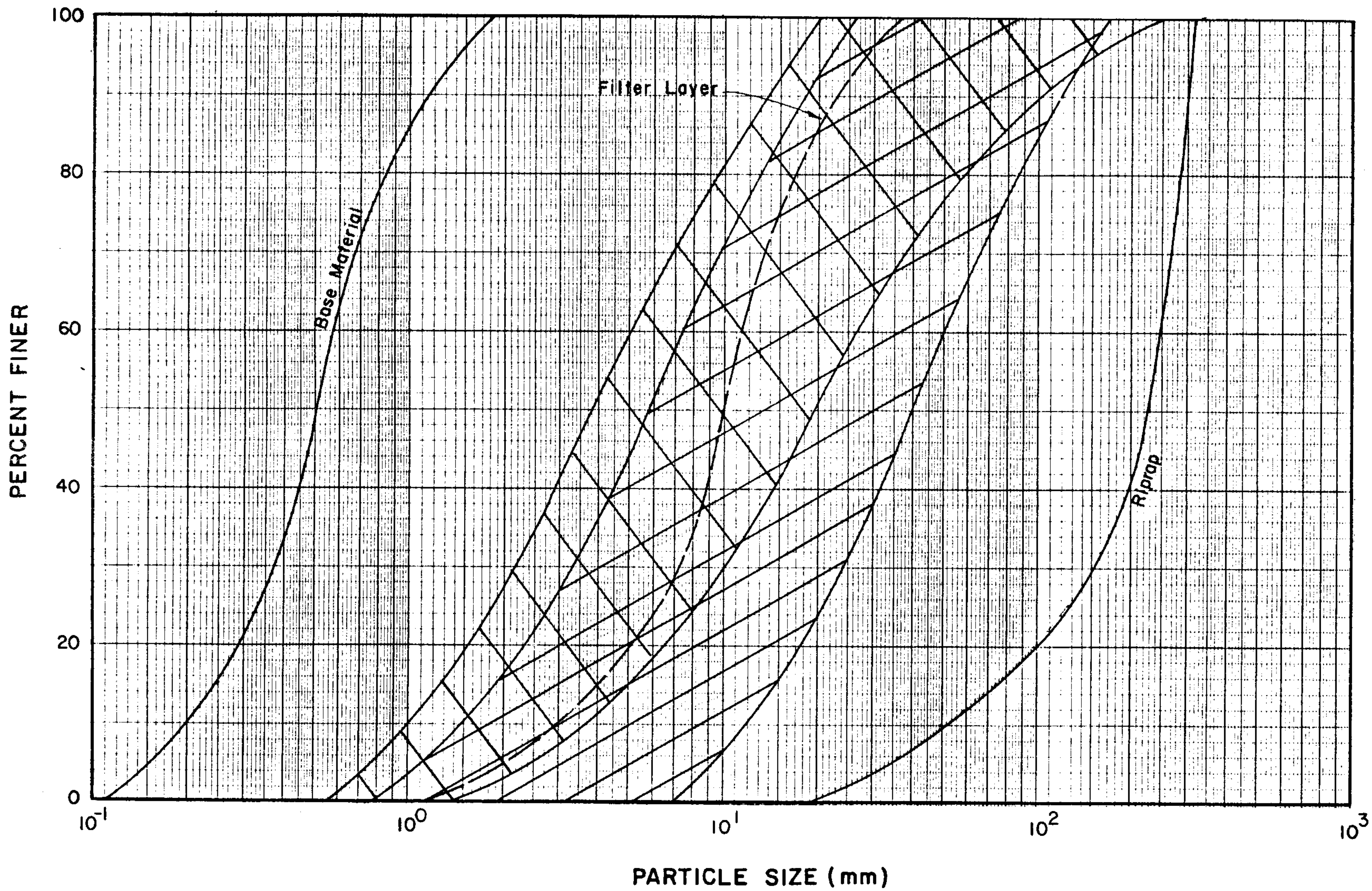


Figure 19.3. Riprap, base material, and filter layer characteristics.

$$\frac{D_{15}\ (\text{Filter})}{D_{15}\ (\text{Base})} > 5\ ,\ \text{so}\ \ D_{15}\ (\text{Filter}) > 5 \times 0.25 = 1.25\ \text{mm}$$

With respect to the base, the filter must satisfy

$$1.25\ \text{mm} < D_{15}\ (\text{Filter}) < 5\ \text{mm}$$

and $$D_{50}\ (\text{Filter}) < 20\ \text{mm}$$

Considering the riprap and filter, the properties must satisfy

$$\frac{D_{50}(\text{Riprap})}{D_{50}(\text{Filter})} < 40\ ,\ \text{so}\ \ D_{50}\ (\text{Filter}) > \frac{230}{40} = 5.8\ \text{mm}$$

$$\frac{D_{15}(\text{Riprap})}{D_{15}(\text{Filter})} < 40\ ,\ \text{so}\ \ D_{15}\ (\text{Filter}) > \frac{75}{40} = 1.9\ \text{mm}$$

$$\frac{D_{15}(\text{Riprap})}{D_{85}(\text{Filter})} < 5\ ,\ \text{so}\ \ D_{85}\ (\text{Filter}) > \frac{75}{5} = 15\ \text{mm}$$

$$\frac{D_{15}(\text{Riprap})}{D_{15}(\text{Filter})} > 5\ ,\ \text{so}\ \ D_{15}\ (\text{Filter}) < \frac{75}{5} = 15\ \text{mm}$$

With respect to the riprap, the filter exhibits the following

$$1.9\ \text{mm} < D_{15}\ (\text{Filter}) < 15\ \text{mm}$$

$$D_{50}\ (\text{Filter}) > 5.8\ \text{mm}$$

and $$D_{85}\ (\text{Filter}) > 15\ \text{mm}$$

The limits of the filter material with respect to the riprap and base material are plotted on Figure 19.3. The results indicate that one gravel filter is adequate for channel stabilty. The dashed line indicates the selected size distribution. Its gradation is

$$D_{85} = 20\ \text{mm} = 0.79\ \text{in.}$$

$$D_{50} = 10\ \text{mm} = 0.39\ \text{in.}$$

$$D_{15} = 4\ \text{mm} = 0.16\ \text{in.}$$

The thickness of the filter layer should be equal to D_{max} (Filter) but not less than 6-9 inches. Therefore, use 9 inches as thickness.

Determine the depth of riprap protection. Riprap should be extended below the channel bed to a distance equal to the depth of flow. Therefore,

$$d_{riprap} = d_{flow} = 3.0 \text{ ft}$$

The width of protection below the channel bed will be

$$3\ D_{50} = 3(0.75) = 2.25 \text{ ft}$$

<u>Step 12</u>: Determine the height of riprap protection. The riprap height will extend to the top of the diversion channel, therefore, the design depth of the diversion channel must be computed. It is composed of the flow depth, one-half the antidune height, one half the superelevation and freeboard.

<u>Antidune height</u>: From Step 5 of Section 19.1, antidunes are present. Their wave height can be computed by:

$$h_a = 2\pi \quad \frac{(0.14)V^2}{g} \qquad \text{Equation 12.1}$$

$$= 2\pi \quad (0.14)\frac{(5.3)^2}{32.2} = 0.77 \text{ ft}$$

In this case, the flow depth, 3.0 feet, is greater than the antidune height; therefore, the accepted antidune height is 0.77 feet.

<u>Superelevation</u>: Superelevation is considered a special problem at the upstream end of the channel and at the intermediate channel bends. A bend is needed that will transition the flow into the diversion channel. It will be constructed with a 100-foot radius of curvature. The intermediate channel bends will have a radius of curvature of 75 feet. The radius of curvature is measured from the centerline of the channel. At the upstream bend, the superelevation is computed to be:

$$\Delta Z = \frac{V^2 W}{g r_c} \qquad \text{Equation 4.18}$$

$$= \frac{(5.30)^2\ (22)}{32.2(100)} = 0.19 \text{ ft}$$

while at the intermediate channel bends,

$$\Delta Z = \frac{(5.30)^2 (22)}{32.2(75)} = 0.26 \text{ ft}$$

Freeboard: According to Step 2, the Froude Number is equal to 0.63 and the flow is subcritical. From Table 4.4, c_{fb} is 0.25 and the freeboard for the straight reaches becomes

$$\text{F. B.} = c_{fb}d + \frac{1}{2}\Delta Z + \frac{1}{2}h_a \qquad \text{Equation 12.2}$$

$$c_{fb}d = 0.25(2.9) = 0.73 < 1.0 \qquad \text{(Use 1.0 foot)}$$

$$\text{F. B.} = 1 + 0 + \frac{0.77}{2}$$

$$\text{F. B.} = 1.39 \text{ ft}$$

For the channel bends, freeboard is

$$\text{F. B.} = 1 + \frac{0.19}{2} + \frac{0.77}{2} = 1.48 \text{ ft} \qquad \text{(Upstream bend)}$$

$$\text{F. B.} = 1 + \frac{0.26}{2} + \frac{0.77}{2} = 1.52 \text{ ft} \qquad \text{(Intermediate bends)}$$

The design depth for the lined diversion channel will be based on the section having the largest design depth. In this case, the largest design depth is at the intermediate bends. The design depth of the diversion channel is indicated in Table 19.1. Riprap protection will be extended to the top of the design depth. Figure 19.4 illustrates a representative cross section of the diversion channel.

Evaluate Riprap Requirements in Bends.

Check the riprap size for stabilization of the channel bends. For the upstream bend, r_c is 100 feet and

$$\frac{r_c}{w} = \frac{100}{22} = 4.5$$

From Figure 6.8

$$\frac{\tau_{bend}}{\tau} = 1.6$$

Table 19.1. Design Depth of Diversion Channel.

Normal Depth (ft)	One-Half Antidune Height (ft)	One-Half Superelevation (ft)	$c_{fb}d$ (ft)	Free Board (ft)	Design Depth (ft)
3.0	0.39	0.13	1.0	1.52	4.52

Note: The design depth will be rounded to 4.5 ft.

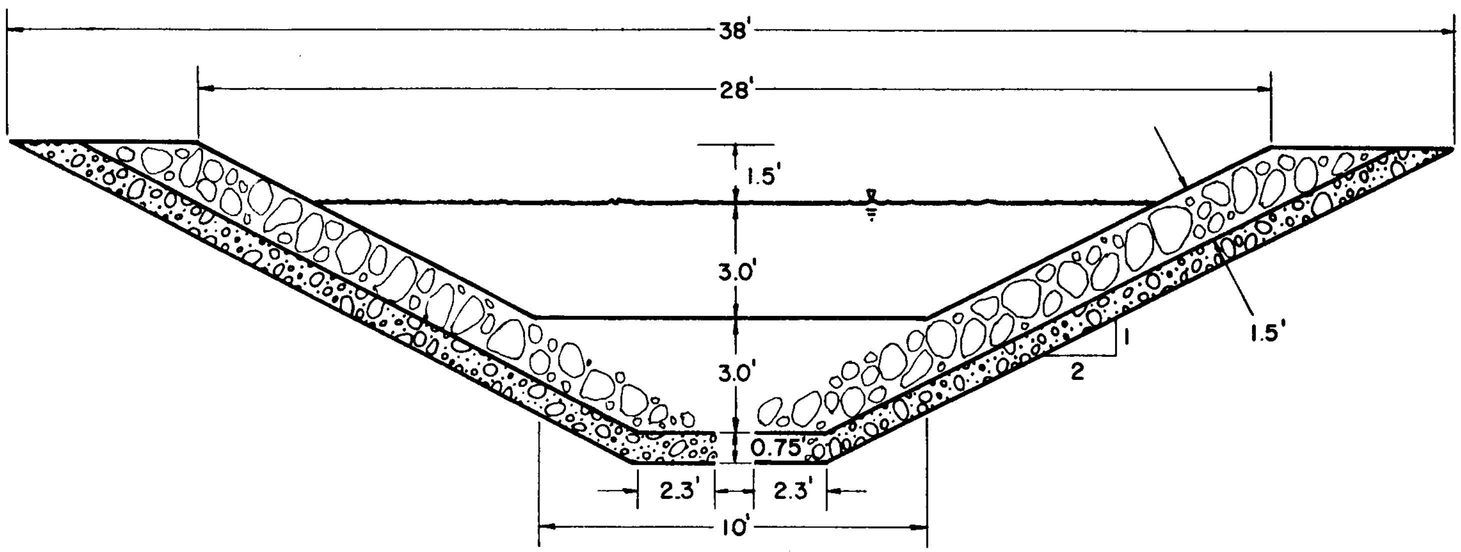


Figure 19.4. Representative cross section of diversion channel.

In Step 3, $\frac{V^2}{R^{0.33}}$ for the straight reaches is 22. The parameter at the bend is

$$\frac{V^2}{R^{0.33}} = 22 \times 1.6 = 35.2$$

According to Table 6.4, Type L riprap should be used. This is the same size as the riprap in the straight reaches. For all other bends, the radius of curvature is 75 feet. Then

$$\frac{r_c}{w} = \frac{75}{22} = 3.4$$

$$\frac{\tau_{bend}}{\tau} = 1.75$$

$$\frac{V^2}{R^{0.33}} = 1.75 \times 22 = 38.5$$

As before, Type L riprap should be used. Thus, for all channel bends and straight reaches, the size of riprap will be the same.

19.3 Design of Drop Structures (using Step-by-Step Procedure Given in Section 16.6)

Step 1. To reduce the existing profile to the proposed channel profile, grade control structures are required. As indicated in Figure 19.5, the grade control structures must account for a total drop of 32 feet. An evaluation of flow rate, available materials, construction costs, and channel topography determined sloped drop structures to be the most feasible. The location of the drop structures indicated in Figure 19.5 is based on minimizing excavation quantities. One 12 foot and two 10 foot drop structures were selected to allow for the 32 foot drop. Lining is necessary due to the nature of the sandy soil, increased flow velocity and change in slope. To minimize cost and blend with the natural landscape, riprap lining will be used. The cross-section of all drop structures will be the same as the channel cross-section.

Step 2: From Figure 5.4, determine the slope of the drop structure and riprap size needed for stabilization. Given the flow rate, 250 cfs, base width, 10 feet, and side slope, 2:1, the following information is assembled

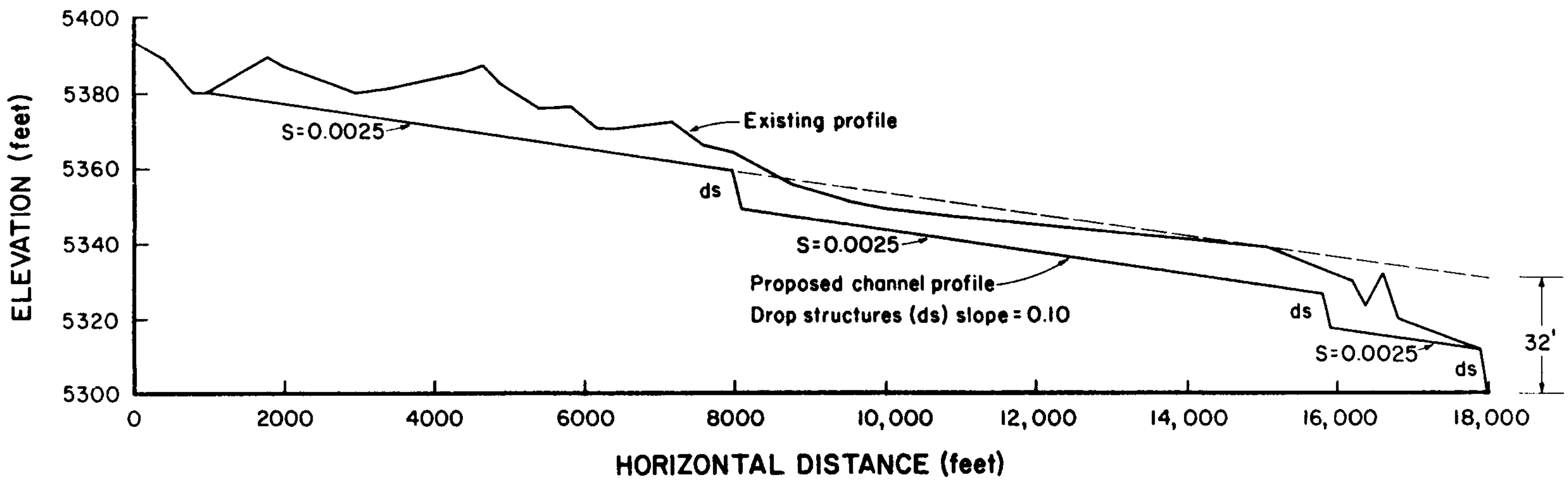


Figure 19.5. Profile view of proposed diversion channel.

For $S = 0.10$

$D_{50} = 1.57$ ft

$d = 1.25$ ft

For $S = 0.25$

$D_{50} = 2.45$ ft

$d = 0.75$ ft

For $S = 0.40$ The velocity of flow is 15 ft/sec and is not practical to design.

A riprap size of 2.45 feet is not available on site and would be difficult and expensive to find. Therefore, select

$D_{50} = 1.75$ ft (Table 6.1)

$S = 0.10$

$d = 1.25$ ft

Step 3: Determine the specifications of the riprap lining. According to Section 5.2.6, the riprap thickness is

$$1.25 \times D_{50} = 2.2 \text{ ft}$$

From Section 5.2.5, the riprap gradation should be

$$D_{max} < 1.25\ D_{50} = 2.2 \text{ ft}$$

$$D_{10-20} \cong \frac{D_{50}}{3.5} = 0.5 \text{ ft}$$

Step 4: Determine if a filter is required for stability of the drop structures. For a sandy base material, the size distribution is

$D_{50} = 0.5$ mm

$D_{85} = 1.0$ mm

$D_{15} = 0.25$ mm

Riprap properties are determined by plotting the gradation given in Step 3 on semi-log paper; see Figure 19.6. The riprap charactersitics are

$D_{50} = 1.75$ ft $= 534$ mm

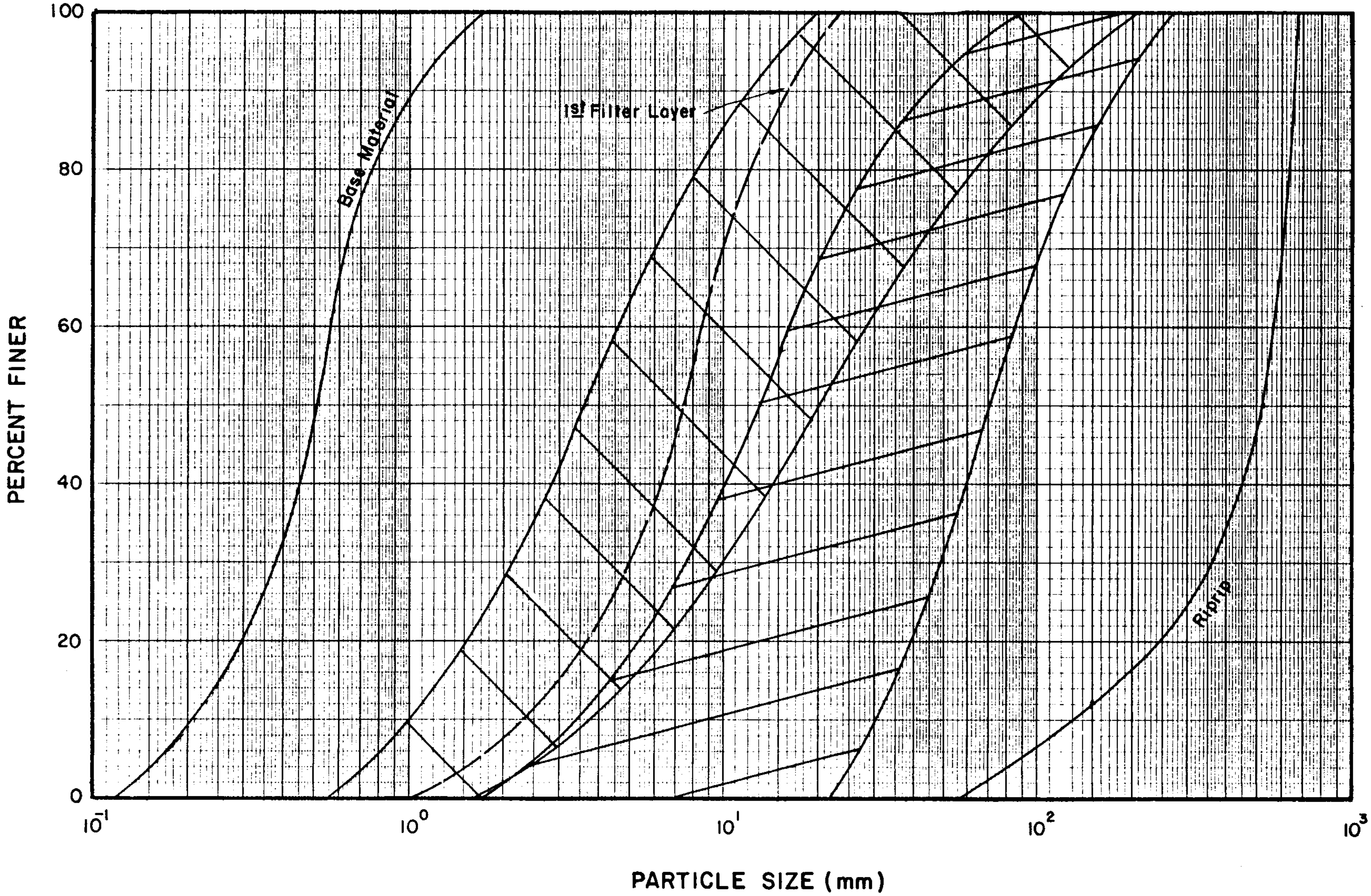


Figure 19.6. Size distribution of base material, riprap, and first filter layer for drop structure.

$D_{85} = 650$ mm

$D_{15} = 180$ mm

To ensure that a filter is needed

$$\frac{D_{15}\ \text{(Riprap)}}{D_{85}\ \text{(Base)}} = \frac{180}{1} = 180 > 5$$

$$\frac{D_{15}\ \text{(Riprap)}}{D_{15}\text{(Base)}} = \frac{180}{0.25} = 720 > 40$$

Therefore, a filter is necessary. The properties of the filter with respect to the sandy base material are the same as determined previously (Step 11, Section 19.2). They are

$$1.25\ \text{mm} < D_{15}\ \text{Filter} < 5\ \text{mm}$$

and $\quad D_{50}$ Filter < 20 mm

Considering the riprap and a filter, the properties of the filter must satisfy

$$\frac{D_{50}\ \text{(Riprap)}}{D_{50}\ \text{(Filter)}} < 40\text{ , so }\ D_{15}\ \text{(Filter)} > \frac{534}{40} = 13.3\ \text{mm}$$

$$\frac{D_{15}\ \text{(Riprap)}}{D_{15}\ \text{(Filter)}} < 40\text{ , so }\ D_{15}\ \text{(Filter)} > \frac{180}{40} = 4.5\ \text{mm}$$

$$\frac{D_{15}\ \text{(Riprap)}}{D_{85}\ \text{(Filter)}} < 5\text{ , so }\ D_{85}\ \text{(Filter)} > \frac{180}{5} = 36\ \text{mm}$$

$$\frac{D_{15}\ \text{(Riprap)}}{D_{15}\ \text{(Filter)}} > 5\text{ , so }\ D_{15}\ \text{(Filter)} < \frac{180}{5} = 36\ \text{mm}$$

With respect to the riprap, the filter must satisfy

$$4.5\ \text{mm} < D_{15}\ \text{(Filter)} < 36\ \text{mm}$$

$$D_{50}\ \text{(Filter)} > 13.3\ \text{mm}$$

and $\quad D_{85}$ (Filter) > 36 mm

The limits of the filter material with respect to the riprap and base material are plotted on Figure 19.6, where the curves have been extrapolated beyond the computed points. The range of suitable filters for both the riprap and base has been cross-hatched. The results of the cross-hatching overlaps indicate that more than one filter is required for channel stability. The first filter layer will be selected from the limits of the filter material with respect to the base material. The dashed line indicates the selected size distribution. Computations for the second filter layer begin with the following information.

Riprap: $D_{50} = 534$ mm

$D_{85} = 650$ mm

$D_{15} = 180$ mm

1st Filter Layer: $D_{50} = 7.5$ mm

$D_{85} = 14$ mm

$D_{15} = 3$ mm

Considering the first filter layer as a base, the properties of the second filter layer must satisfy

$$\frac{D_{50}\ (\text{Filter})}{D_{50}\ (\text{Base})} < 40 \text{ , so } D_{50}\ (\text{Filter}) < 40 \times 7.5 = 300 \text{ mm}$$

$$\frac{D_{15}\ (\text{Filter})}{D_{15}\ (\text{Base})} < 40 \text{ , so } D_{15}\ (\text{Filter}) < 40 \times 3 = 120 \text{ mm}$$

$$\frac{D_{15}\ (\text{Filter})}{D_{85}\ (\text{Base})} < 5 \text{ , so } D_{15}\ (\text{Filter}) < 5 \times 14 = 70 \text{ mm}$$

$$\frac{D_{15}\ (\text{Filter})}{D_{15}\ (\text{Base})} > 5 \text{ , so } D_{15}\ (\text{Filter}) > 5 \times 3 = 15 \text{ mm}$$

Therefore with respect to the first filter layer, the second filter must satisfy

$15 \text{ mm} < D_{15}\ (\text{Filter}) < 70 \text{ mm}$

and $D_{50}\ (\text{Filter}) < 300 \text{ mm}$

with respect to the riprap, the second filter layer must satisfy

$4.5 \text{ mm} < D_{15}\ (\text{Filter}) < 36 \text{ mm}$

$D_{50}\ (\text{Filter}) > 13.3 \text{ mm}$

and $D_{85}\ (\text{Filter}) > 36 \text{ mm}$

The limits of the second filter layer with respect to the riprap and the first filter layer are plotted on Figure 19.7. Any filter material that falls within the region where the cross-hatching overlaps will meet the criteria of both the riprap and the first filter layer. The selected size distribution for the second filter layer is indicated by the dashed line. The characteristics of the second filter layer are

$$D_{50} = 50 \text{ mm}$$

$$D_{85} = 120 \text{ mm}$$

$$D_{15} = 25 \text{ mm}$$

The thickness of each filter layer will be 9 inches.

Although feasible, the design for gravel filters may not be the most practical solution since the additional excavation requirements for only the filters is 1.5 feet. The best alternative is the use of filter cloth. The properties of the filter cloth are determined by

$$\frac{D_{85}}{EOS} > 1$$

$$EOS < D_{85} \text{ (Base)} = 0.9 \text{ mm} = 0.035 \text{ inches}$$

In this case, the Equivalent Opening Size of the filter cloth must be less than 0.9 mm. Furthermore, the open area of the filter cloth should not be in excess of 36 percent. A 6 inch gravel layer will be placed over the filter cloth to protect against puncture during placement of riprap.

Steps 5, 6. Determine the length of the drop structure. The depth of flow downstream of the drop structures is 3.0 feet. From Equation 16.1

$$L = L_u + L_s + L_d$$

$$L_u = L_b = 5 \times d_{flow}$$

$$L_s = zh$$

For the 10 foot drop structures

$$L_u = L_b = 5 \times 3.0 = 15 \text{ ft}$$

$$L_s = 10 \times 10 = 100 \text{ ft}$$

$$L = 15 + 100 + 15 = 130 \text{ ft}$$

For the drop structures with a 12 foot drop

$$L_u = L_b = 5 \times 3 = 15 \text{ ft}$$

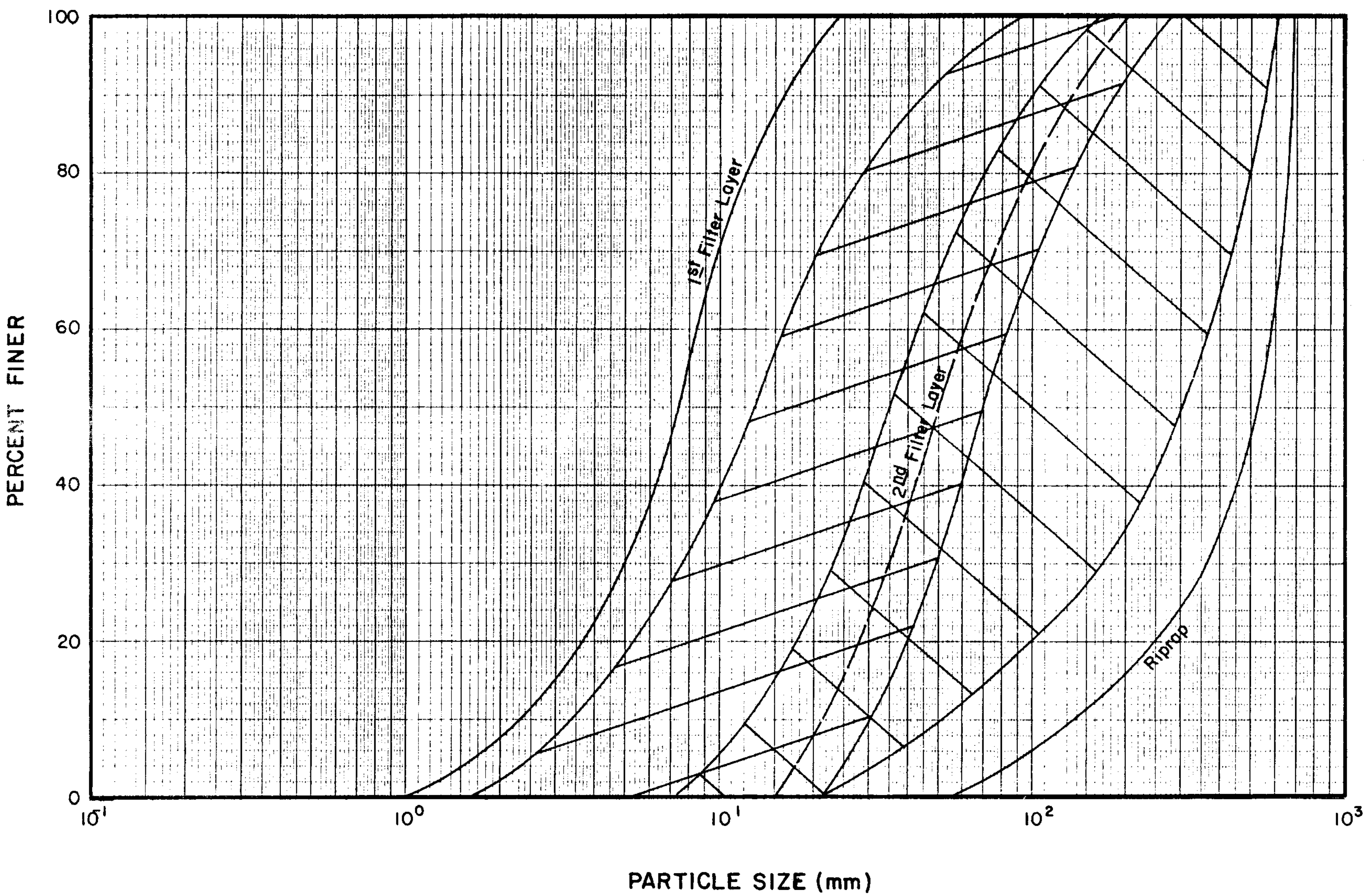


Figure 19.7. Size distribution of second filter layer for drop structure.

$$L_s = 12 \times 10 = 120 \text{ ft}$$

$$L = 15 + 120 + 15 = 150 \text{ ft}$$

Step 7: Determine freeboard requirements. Table 4.4 indicates that c_{fb} is 1.0. Therefore, with flow depth in the drop structures equal to 1.25 ft, the freeboard is

$$F.B. = c_{fb}d + \frac{1}{2}\Delta Z + \frac{1}{2}h_a \qquad \text{Equation 2.7}$$

$$= 1(1.25) + 0 + 0$$

$$= 1.25 \text{ ft}$$

Figures 19.8 and 19.9 present the drop structure design.

19.4 Design of Channel Inlet

Special problems must be considered at the upstream end of the channel, where the flow enters the diversion from North Battle Creek. Without correction, the flow would enter at a right angle, causing significant pressure forces and superelevation on the opposite side of the diversion channel. This problem should be corrected by transitioning the flow with a bend in the channel. A bend with a 100 foot radius of curvature is recommended. The superelevation was calculated to be 0.19 feet (see Step 6, Section 19.2). In addition to the channel bend, a dike will be constructed that will allow for the lateral migration of North Battle Creek. This will prevent the flow from circumventing the diversion channel. It is only necessary to provide a dike to the west of North Battle Creek. Should an eastward progression of the meander be realized, the flow will naturally drain into the diversion channel. A schematic diagram of the channel and North Battle Creek is provided in Figure 19.10.

Step 1: Design the inlet transition to the diversion channel. The characteristics needed for North Battle Creek and the diversion channel are:

North Battle Creek	Diversion Channel
Q = 250 cfs	Q = 250 cfs
b = 30 ft	b = 10 ft
d = 1.5 ft	d = 3.0 ft
V = 4.8 ft/sec	V = 5.3 ft/sec
W = 39 ft	W = 22 ft
R = 1.32 ft	R = 1.99 ft

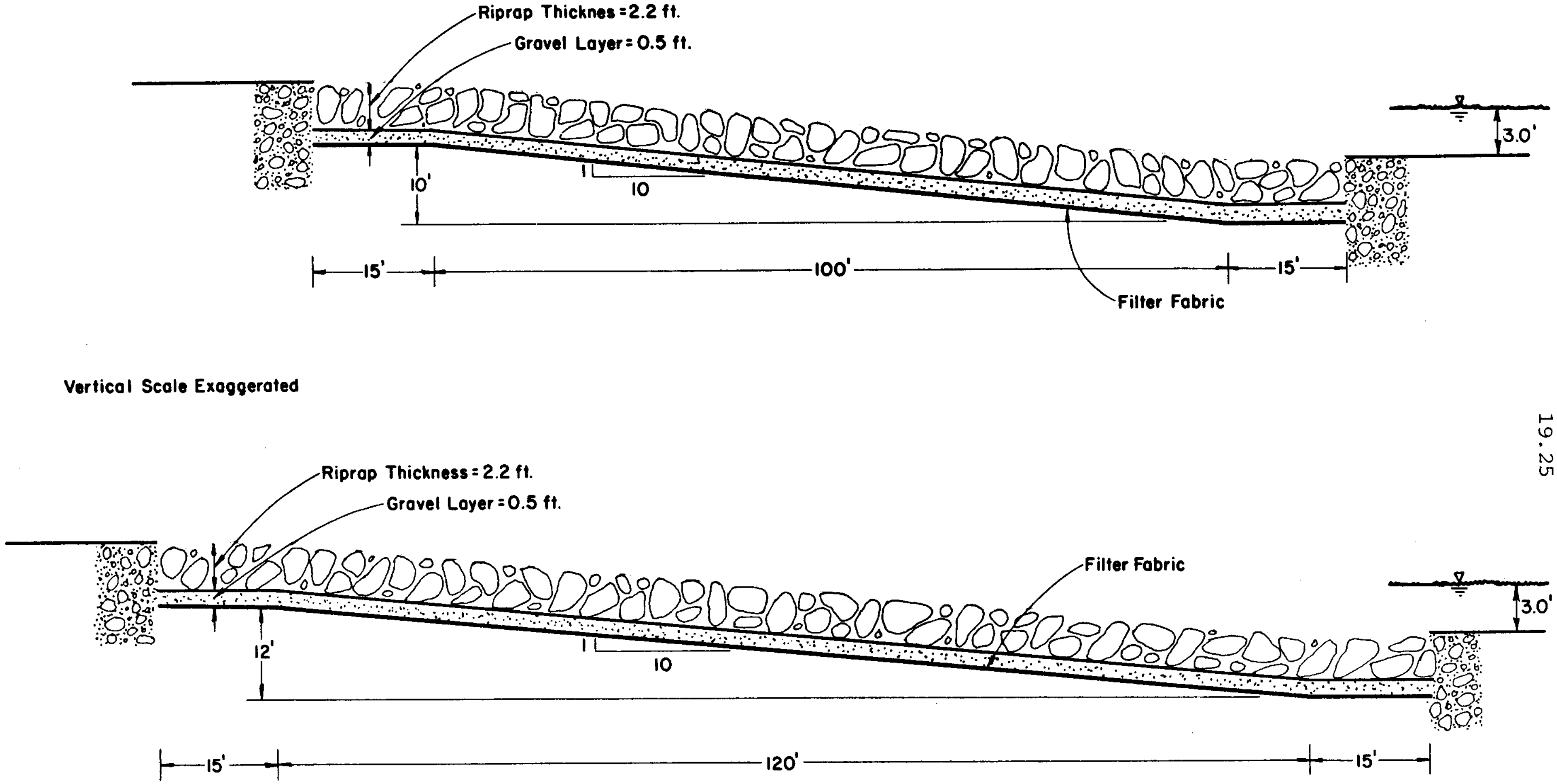


Figure 19.8. Profile view of drop structure.

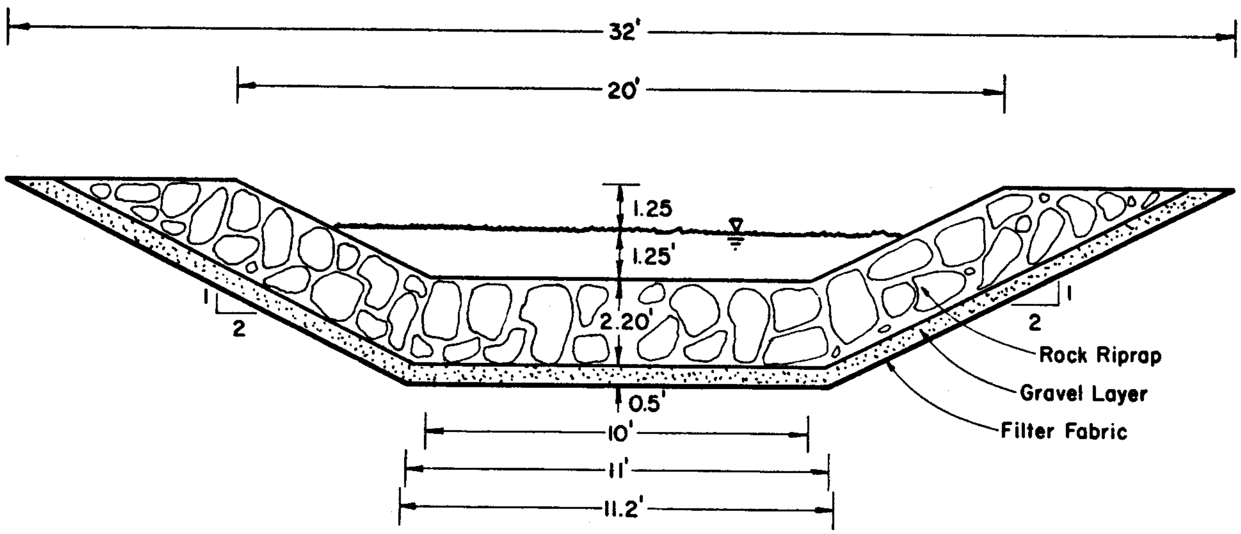


Figure 19.9. Cross section of drop structure.

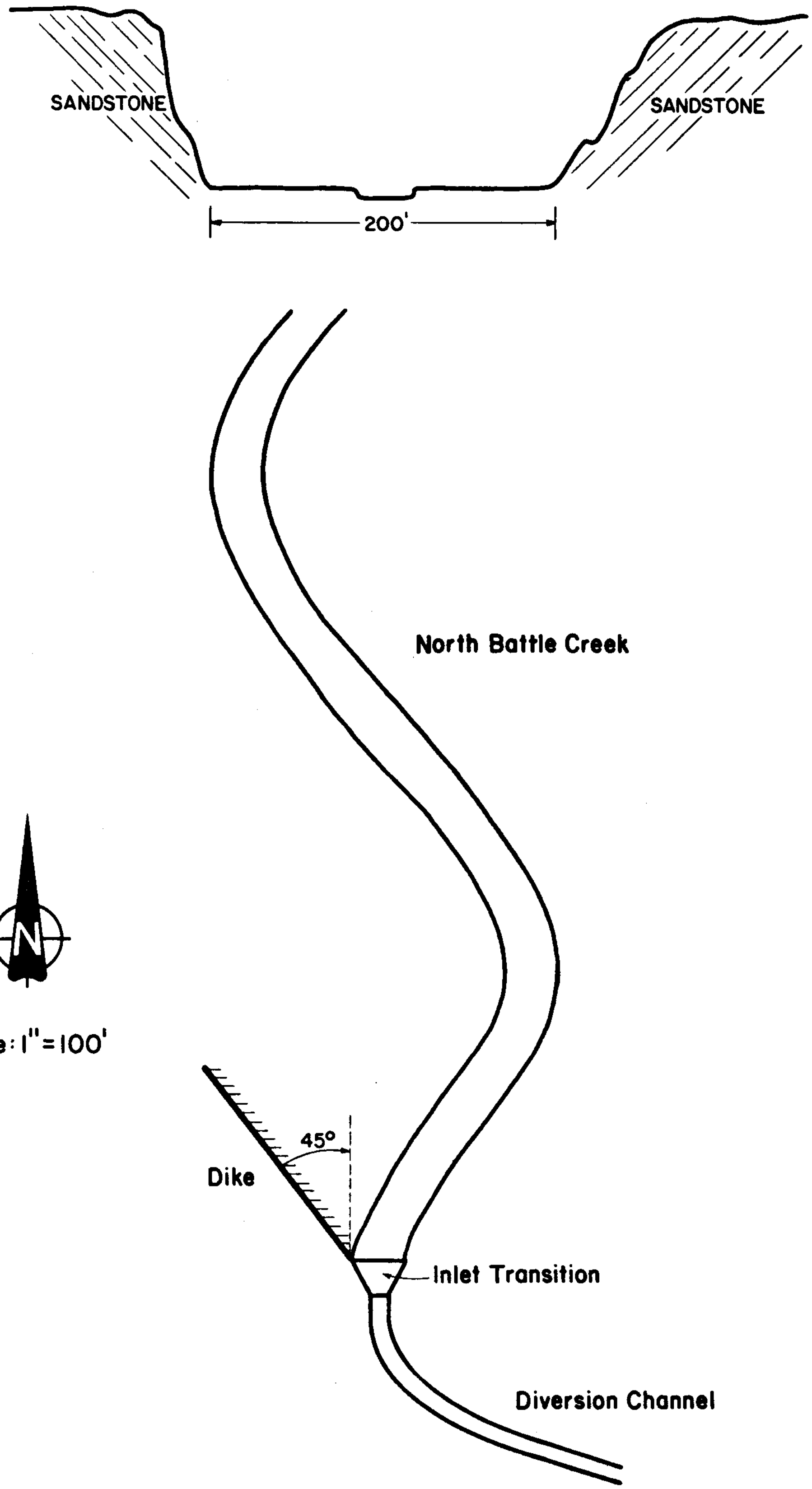


Figure 19.10. Schematic diagram of channel inlet.

The transition will be designed to reduce the backwater effects that could overtop the natural channel banks. Criteria established in Section 7.3 will be followed.

a. Compute the change in water surface profile

$$\Delta W.S. = 1.15 \quad \frac{V_2^{\,2}}{2g} - \frac{V_1^{\,2}}{2g} \qquad \text{Equation 7.5a}$$

$$= 1.15 \times \frac{(5.3)^2 - (4.8)^2}{64.4} = 0.09 \text{ ft}$$

b. Compute the change in bed elevation (ΔB.E.) between transition entrance and exit.

$$\Delta B.E. = d_2 - d_1 + \Delta W.S. \qquad \text{Equation 7.6a}$$

$$\Delta B.E. = 3.0 - 1.5 + 0.09$$

$$= 1.59$$

c. Compute length of transition. As presented in Chapter VII, the length of the transition depends on the included angle between the upstream and downstream cross sections. Using the maximum value of this angle, 25 degrees, the length of the transition becomes (Figure 19.11)

$$\tan(12.5°) = \frac{0.5(39-22)}{L}$$

$$L = 38 \text{ ft (Use 40 ft)}$$

d. Determine slope of inlet transition (Figure 19.12).

$$S = \frac{\Delta B.E.}{L}$$

$$= \frac{1.59}{40} = 0.04$$

e. Determine if riprap protection is needed. The slope of the inlet transition is greater than the equilibrium slope of the diversion channel and degradation of the transition bed will occur. This can also be explained by Lane's relation, $QS \quad Q_s D_{50}$. The discharge, Q, and the median diameter, D_{50}, do not change. With an increase in slope, S, there will be a increase in sediment transport, Q_s, and degradation of the bed occurs. Consequently, a lining of riprap will be required on the bed. The banks of the inlet transi-

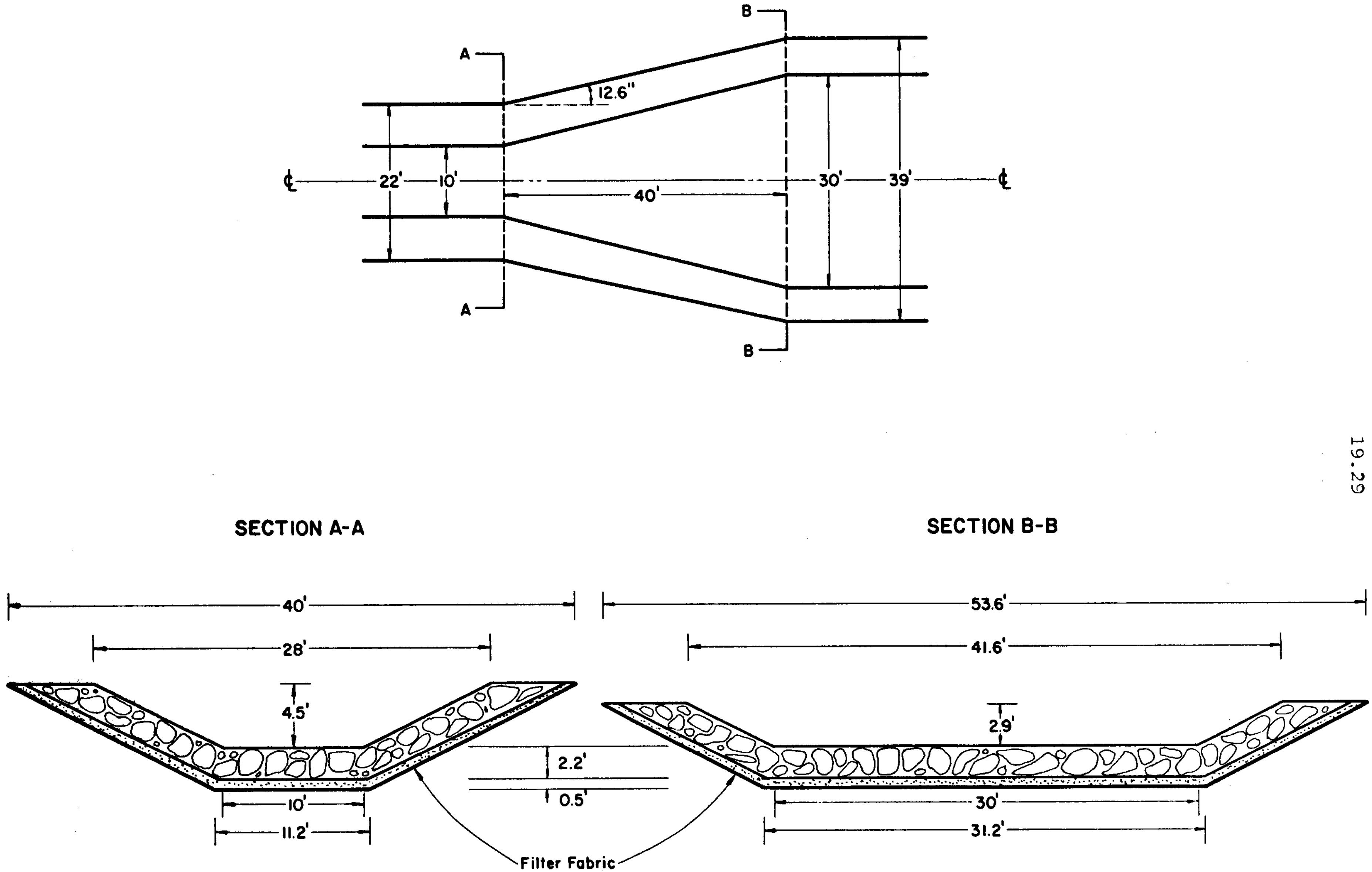


Figure 19.11. Transition design for channel inlet.

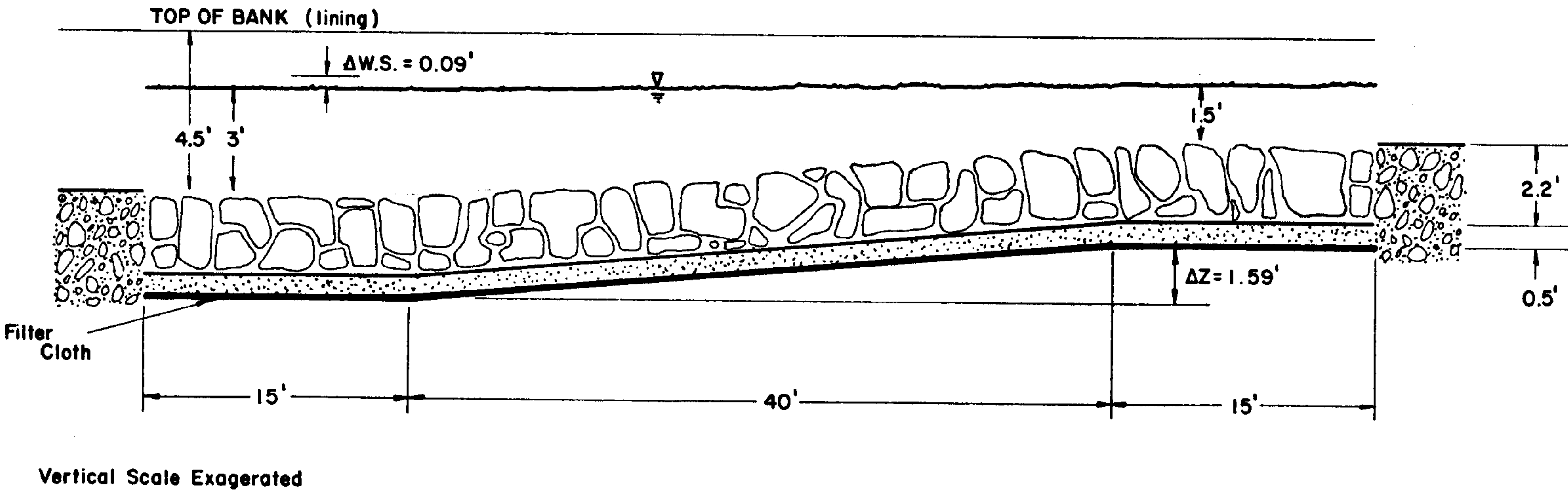


Figure 19.12. Transition design for channel inlet.

tion also need riprap protection because the maximum permissible velocity, 2.5 ft/sec, is less than either 4.8 or 5.3 ft/sec.

f. Determine size of riprap required for stabilization of bed and bank. Based on the steep transition slope, the riprap will be designed in accordance with the procedures presented for drop structures in Chapter XVI. The most critical section for riprap protection will be at the exit of the inlet transition; therefore, a base width of 10 feet will be used for the riprap design. Using the curve for $S = 0.10$, Figure 5.5 provides

$$D_{50} = 1.57 \text{ ft (Use 1.75 ft)}$$

$$d = 1.25$$

The riprap thickness and gradation will be

$$1.25 \times D_{50} = 2.2 \text{ ft} \quad \text{(thickness)}$$

$$D_{max} = 1.25\, D_{50} = 2.2 \text{ ft}$$

$$D_{10-20} \cong \frac{D_{50}}{3.5} = 0.5 \text{ ft}$$

The filter requirements are exactly the same as determined in Step 5 of Section 19.3. Consequently, a plastic filter cloth with an equivalent opening size (EOS) less than 0.9 mm will be used. A six inch gravel layer will be placed over the cloth to prevent puncture. To prevent local scour damage to the bed, protection should extend upstream and downstream of the transition at least equal to five times the downstream flow depth or a minimum of 15 feet. The depth of protection will be equal to the riprap thickness computed for the transition.

g. Check the freeboard at the upstream end of the inlet transition. Keeping the top of the channel at a uniform height from the downstream section to the inlet entrance reveals

$$\text{design depth} - \Delta\text{B.E.} = 4.5 - 1.59 = 2.91 \text{ ft}$$

which is the design depth at the inlet entrance. Subtracting the upstream flow depth, 1.5 ft, from this value gives a freeboard height equal to 1.4 ft. From Equation 12.2, the minimum freeboard height would be

$$c_{fb}d = 1.0(1.25) = 1.25 \text{ (1.0 ft is minimum)}$$

$$F.\ B. = 1.25 + 0 + 0$$

$$= 1.25 \text{ feet}$$

Therefore, the freeboard height of the 1.4 feet is more conservative and the design depth becomes 2.9 feet. Figures 19.11 and 19.12 present the results of the inlet transition design.

Step 2: Design the dike at the channel inlet. As mentioned previously, a dike will be provided only to the west of the inlet. The inlet is located at the center of the meander bend and maximum protection is required. Therefore, the dike will encompass that portion of the meander width to the west of the inlet.

a. Determine the length of the dike. The projection angle is 45 degrees. From Figure 19.10, the dike is measured to be 140 feet.

b. Calculate the height of the dike. The minimum elevation of the dike is three feet above the water surface at the entrance to the inlet. From Figure 19.12, the top of the bank is 1.4 feet above the water surface. Thus, the height of the dike becomes

$$3 - 1.4 = 1.6 \text{ ft}$$

However, the dike will not be less than two feet at any location; therefore, use two feet as the dike height.

c. Calculate width of dike.

$$W = \frac{h+35}{5} = \frac{2+35}{5} = 7.4 \text{ ft} \qquad \text{Equation 17.1}$$

d. Selecting a side slope of 2:1, design riprap for the dike. For the upstream channel, R is 1.32 ft and V is 4.8 ft/sec.

$$\frac{V^2}{R^{0.33}} = 21$$

According to Table 6.4, Type L riprap should be used and its gradation is (Table 6.5)

$$D_{100} = 1.0 \text{ ft}$$

$$D_{50} = 0.75 \text{ ft}$$

$$D_{10} = 0.17 \text{ ft}$$

The thickness of the riprap will be $2D_{50}$ above the ground and $4D_{50}$ below the ground surface. The gravel filter will be identical to that designed in Step 11 of Section 19.2. Its gradation and thickness are

$D_{85} = 20$ mm

$D_{50} = 10$ mm

$D_{15} = 4$ mm

thickness = 0.75 ft

The depth of riprap becomes

$$d_{riprap} = d_{flow} + d_{scour} = 2d_{flow} \qquad \text{Equation 17.2}$$

$$= 2 \times 1.5 = 3.0 \text{ ft}$$

Figure 19.13 provides a cross-sectional view of the dike design.

19.5 Design of Channel Outlet

A major concern in the diversion design is the capacity of North Battle Creek downstream of the diversion. In its natural condition, North Battle Creek drains an area larger than the area drained by the diversion channel. The proposed mining operations would cut off flow to this channel, greatly reducing its discharge (see Figure 19.1). Thus the flow entering North Battle Creek, including that from the diversion, is less than its natural condition, causing no problems with capacity. At the point of possible intersection with the diversion channel, North Battle Creek is bounded on the west by a 30 foot cliff of sandstone. The east bank is composed of the fine sands exhibited in much of the drainage area and most of the diversion channel. A cross-section of North Battle Creek at the point of intersection with the diversion is provided in Figure 19.14. With the proposed mining operations, the maximum discharge expected in North Battle Creek upstream of the diversion outlet is 75 cfs. At the point of intersection, North Battle Creek exhibits the following:

$S = 0.0035$	side slope, $z = 2$ (east bank)
$n = 0.022$	bank height = 2.5 ft (east bank)
$b = 40$ ft	$D_{50} = 0.5$ mm

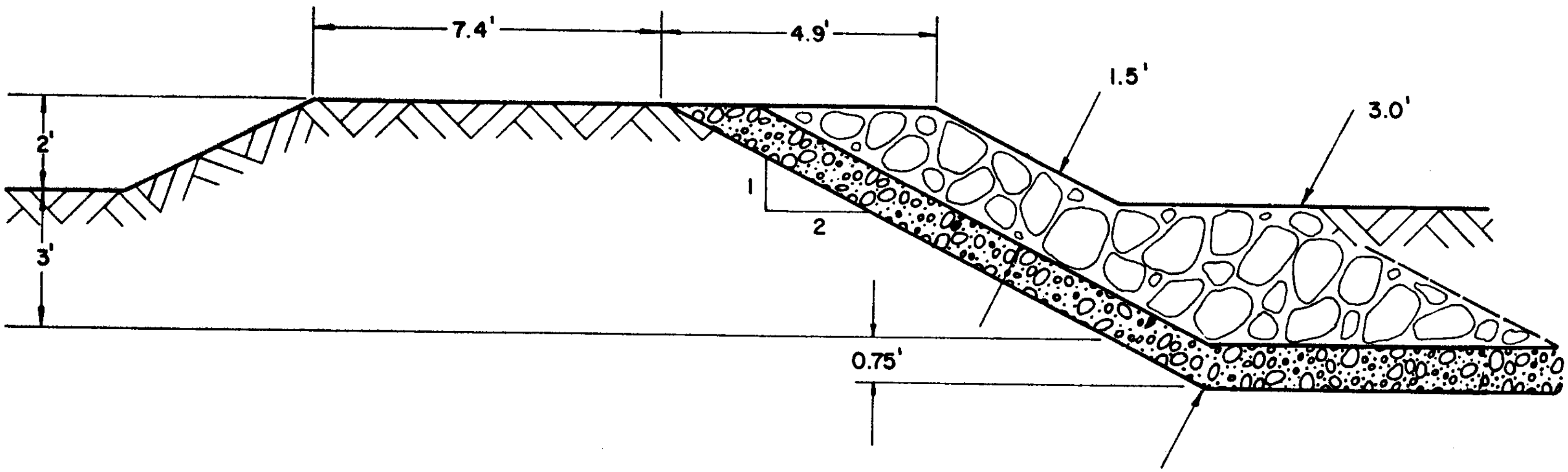


Figure 19.13. Cross-sectional view of dike for channel inlet.

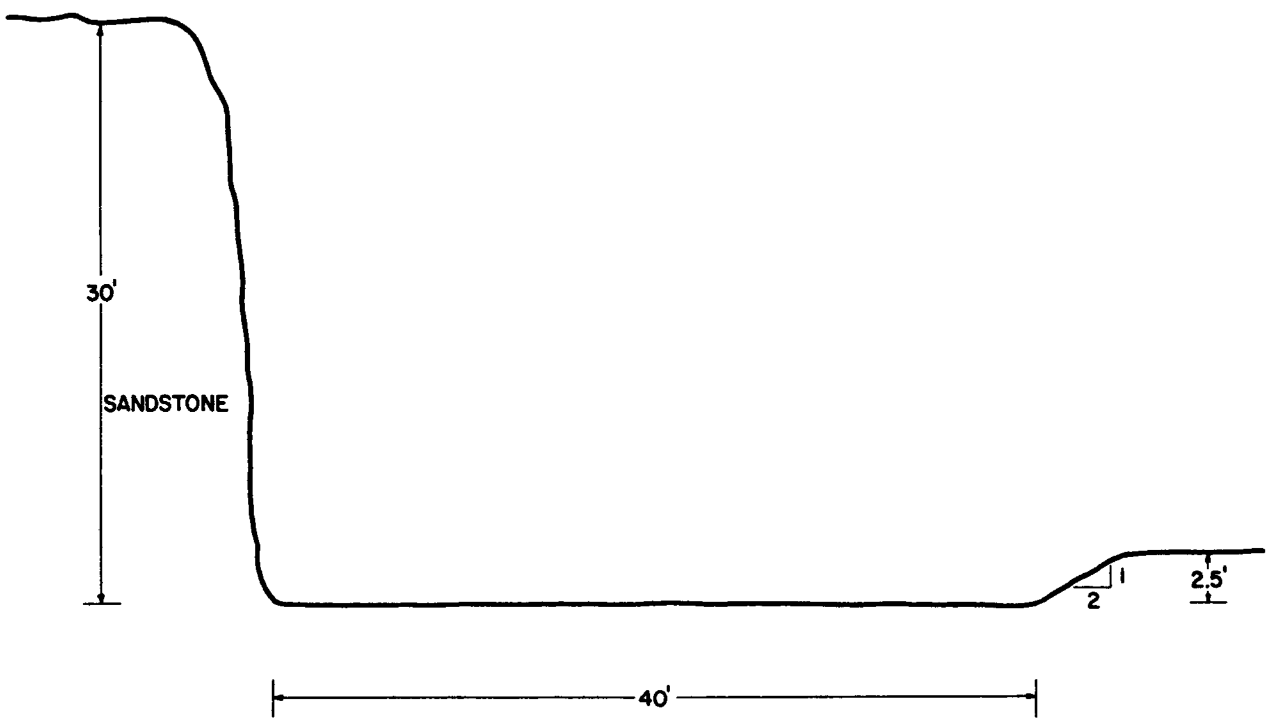


Figure 19.14. Cross section of North Battle Creek at point of intersection with diversion channel.

There are no available natural or existing channels to transition the diverted flow from the diversion channel into North Battle Creek. Consequently, a channel junction will be designed for the diversion channel outlet.

Step 1: Determine angle of intersection with North Battle Creek. The diversion channel should intersect North Battle Creek at channel invert grade. The junction angle should be less than or equal to 55 degrees. Figure 19.15 shows a plan view of the channel junction and a junction angle of 40 degrees.

Step 2: Check the capacity of North Battle Creek. With a diverted discharge of 250 cfs, the total discharge becomes 325 cfs. Solving

$$Q = \frac{1.49}{n} AR^{2/3} S^{1/2} \qquad \text{Equation 4.14}$$

$$A = 40d + d^2$$

$$P = 40 + (1+\sqrt{5})d$$

by trial and error reveals

d	A	P	R	$R^{2/3}$	Q
1.5	62.3	44.9	1.39	1.25	311
1.55	64.4	45	1.43	1.27	328

Therefore

$$d = 1.55 \text{ ft}$$

$$V = 5.1 \text{ ft/sec}$$

$$R = 1.43$$

A check of the flow regime will provide the information needed to determine if antidunes are present.

$$\tau = \gamma RS = 62.4(1.43)(.0035) = 0.31 \text{ lb/ft}^2$$

$$\tau V = 0.31(5.1) = 1.6$$

From Figure 12.2, antidunes are present. Their height can be computed by

$$h_a = 2\pi(0.14)\frac{V^2}{g} \qquad \text{Equation 12.1}$$

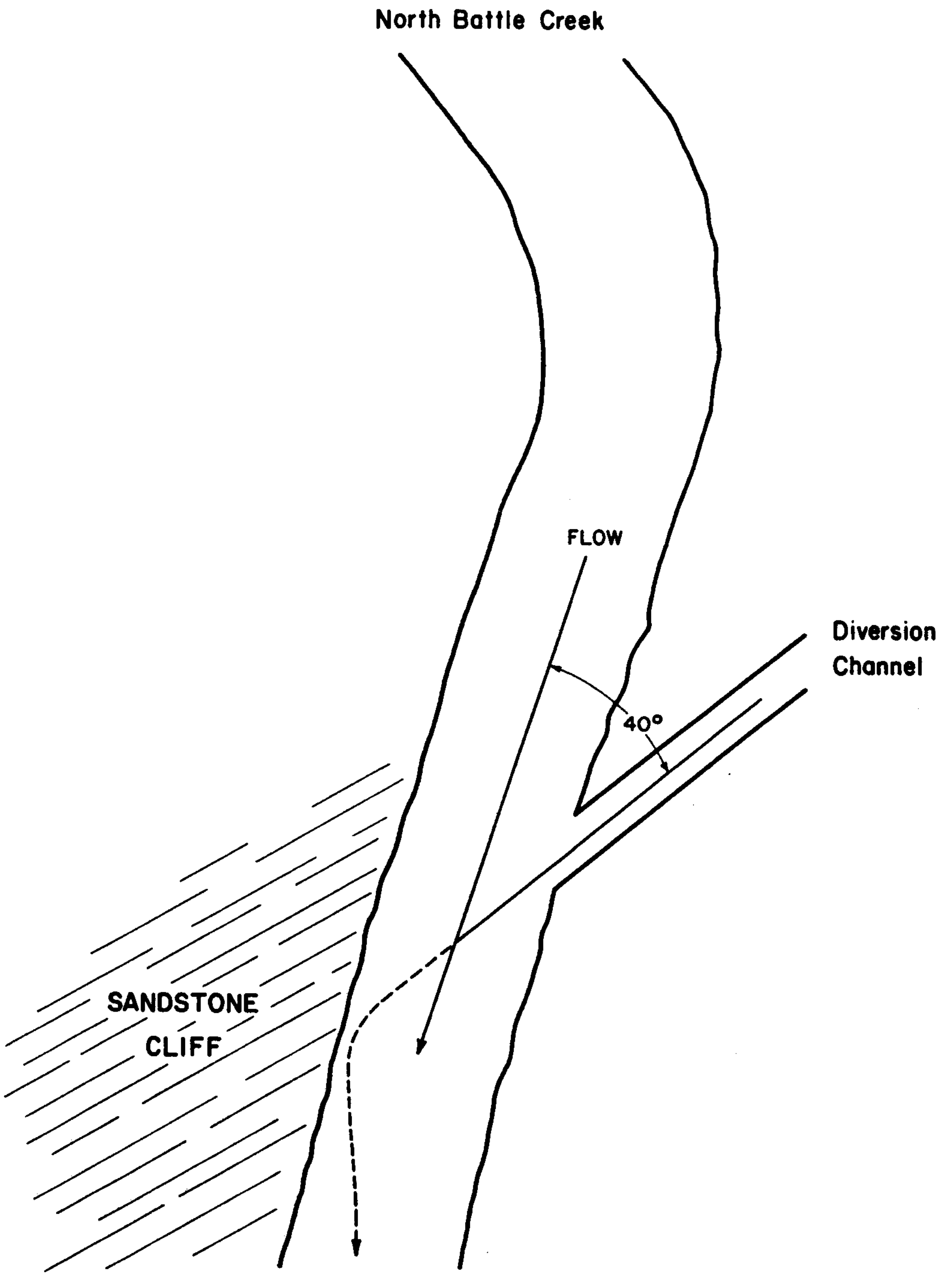


Figure 19.15. Plan view of channel junction.

$$= 2\pi \frac{(0.14)(5.1)^2}{32.2} = 0.71 \text{ ft}$$

For a discharge of 325 cfs, the height of the natural channel (east bank) must account for the depth of flow plus one-half the antidune height. The height of the bank must be

$$1.55 + \frac{0.71}{2} = 1.91 \text{ ft}$$

Comparing this value with the bank height of 2.5 feet indicates that the natural channel will safely pass the combined discharge.

Step 3: Determine the need for bank protection. The primary location for local scour and erosion is the point where the flow from the diversion channel directly impinges on the bank. As can be seen from Figure 19.15, the west bank of North Battle Creek is the most susceptible. Ordinarily, bank stabilization measures would be necessary but in this case, the bank is composed of sandstone, highly resistant to erosion. Therefore, no stabilization measures will be required.

XX. RESEARCH NEEDS

One of the most significant problems to designing effective water diversion channels in the Eastern Coal Province is the lack of steep slope riprap design procedures. A detailed search of the literature revealed that the commonly accepted design methodologies for dumped riprap are limited to moderately sloped conditions. If these methods are indiscriminately applied to the steep slope areas of the Eastern Coal Province, a substantial amount of the riprap will likely fail. Due to the widespread use of riprap for steep channel stabilization, it was considered essential to present an applicable design methodology in order to develop an effective design manual.

Therefore, the time and effort was spent to develop a steep slope riprap design procedure under this contract. Chapter V presented the results of this development as easily applied nomographs. The method is based on sound theoretical principles and limited experimental data, and provides reasonable results; however, physical model testing is strongly recommended to validate the procedure. Until such testing is performed, the results should be carefully applied utilizing engineering judgment and experience to insure satisfactory performance. The basic problem of designing a stable riprap for a slope condition near the angle of repose would suggest that the use of rock riprap may not be physically realistic for long-term application. Additionally, in any steep slope condition, it is extremely difficult to properly place the riprap. There appears to be no simple solution to this problem. The riprap channel can be constructed in stages, for example as each lift of a spoil fill is completed; however, placement will still be difficult. Therefore, even with a proper steep slope riprap design procedure, riprap failure can still occur if adequate placement techniques are not developed and employed.

The alternatives to rock riprap channels for surface mine application are limited. The use of concrete channels or closed conduit structures are not economically feasible on a large scale. Additionally, closed conduit structures are not considered maintenance free. The best alternative to rock riprap channels in spoil fill applications may be the internal rock core drain. However, there are no standardized methods of design currently available. Additionally, their long-term functioning has not been established. Limited field observation has shown some problems of fine sediment deposition that can plug the drain.

Therefore, in summary the research needs are:

1. Physical model testing of the steep slope riprap design procedure to verify its application.

2. Investigation of equipment and placement techniques for construction of riprap channels on steep slopes.

3. Research into the long-term functioning of internal rock core drains.

4. Development of standardized design and construction methods for internal rock core drains.

APPENDIX A

U. S. WEATHER BUREAU

TECHNICAL PAPER 40 CHARTS

This page intentionally left blank.

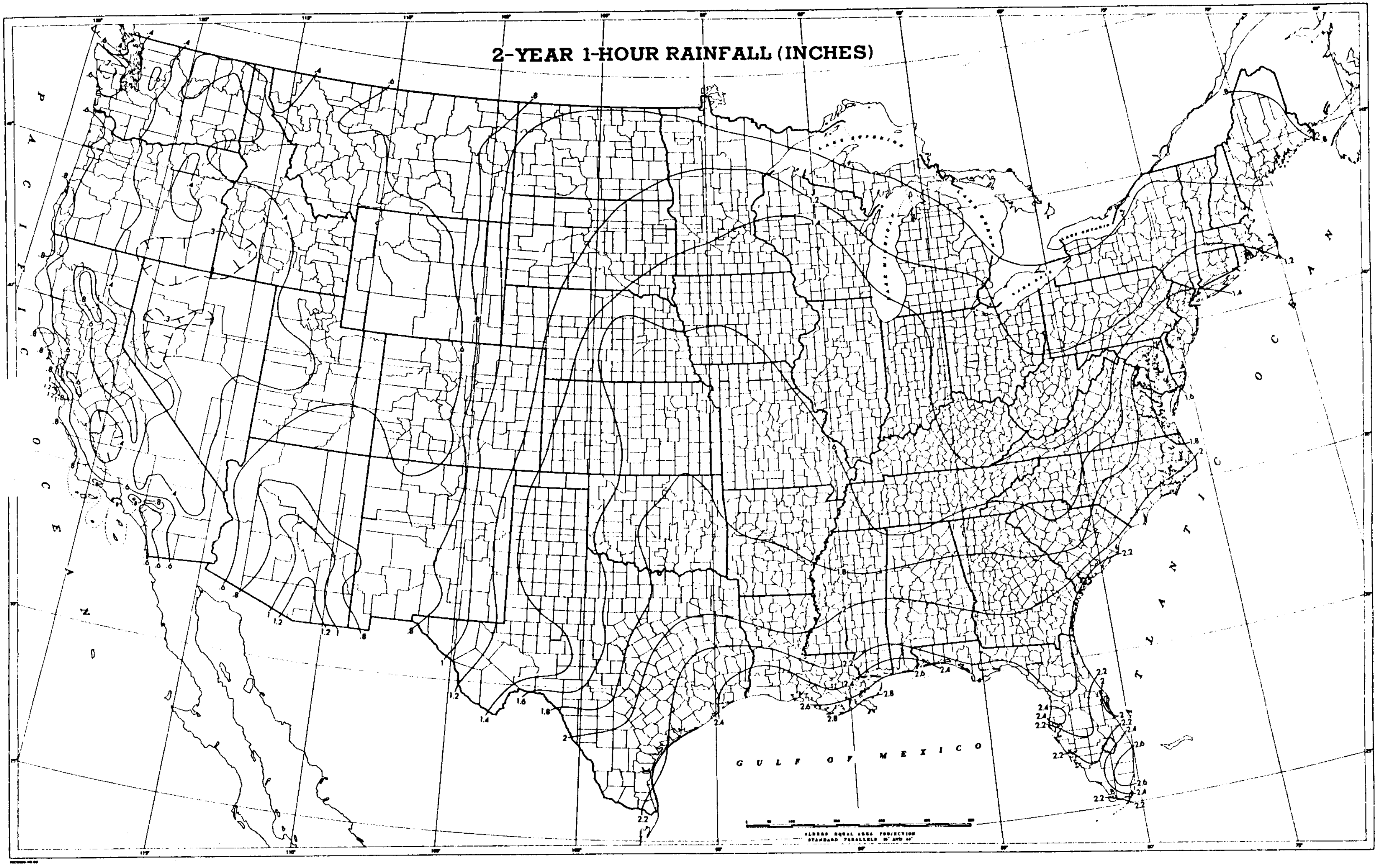


Chart for 2-year 1-hour rainfall (inches).

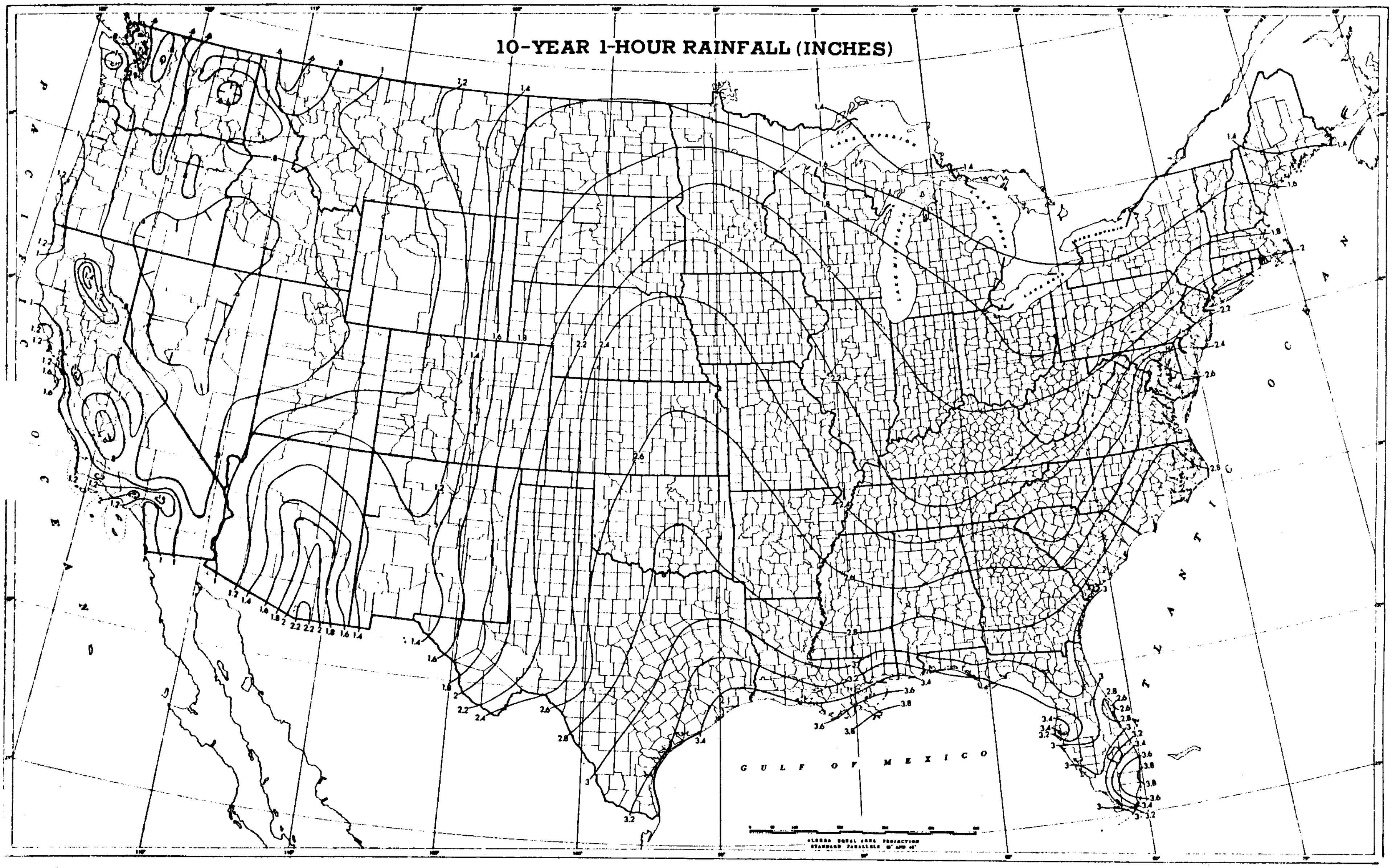


Chart for 10-year 1-hour rainfall (inches).

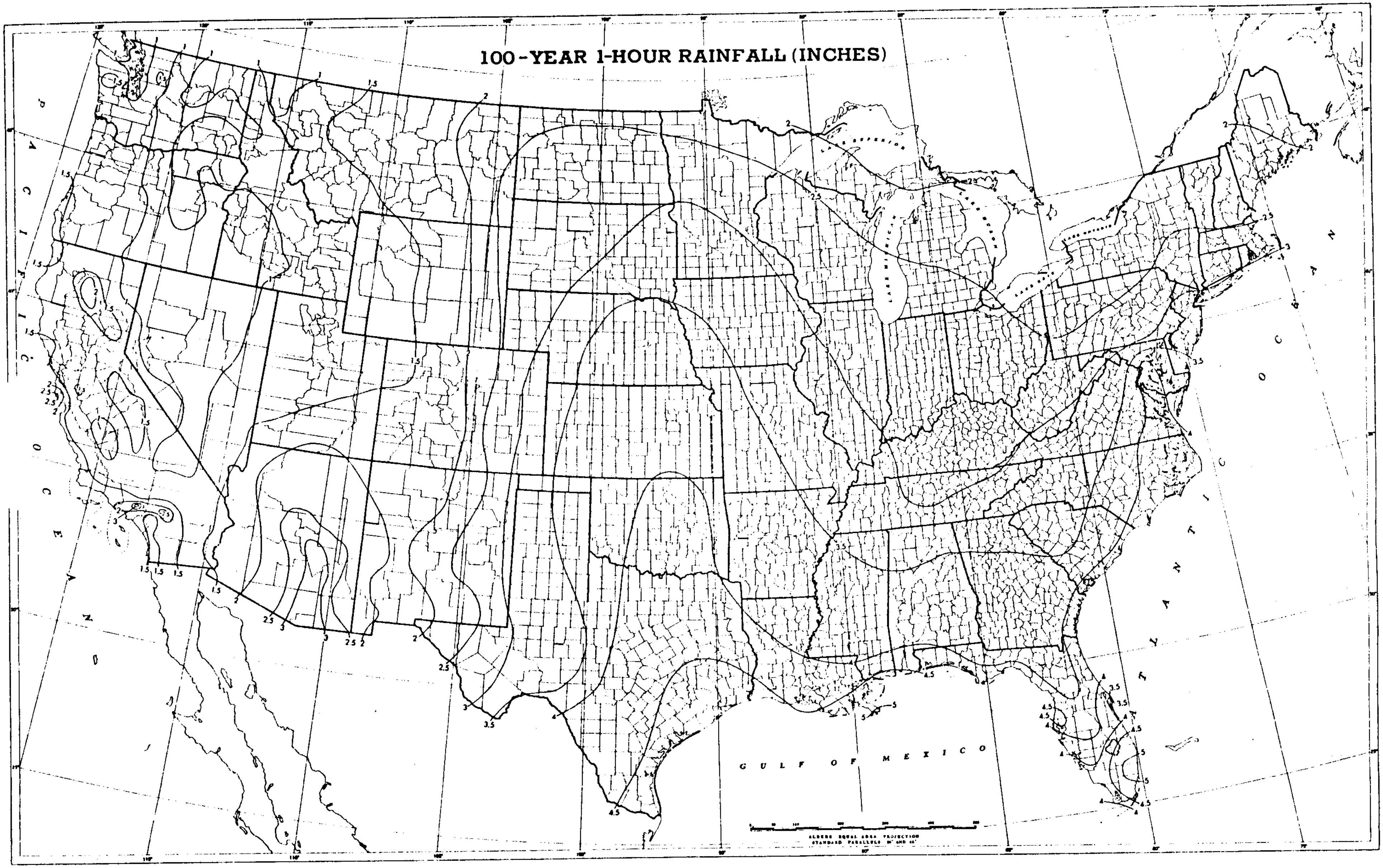


Chart for 100-year 1-hour rainfall (inches).

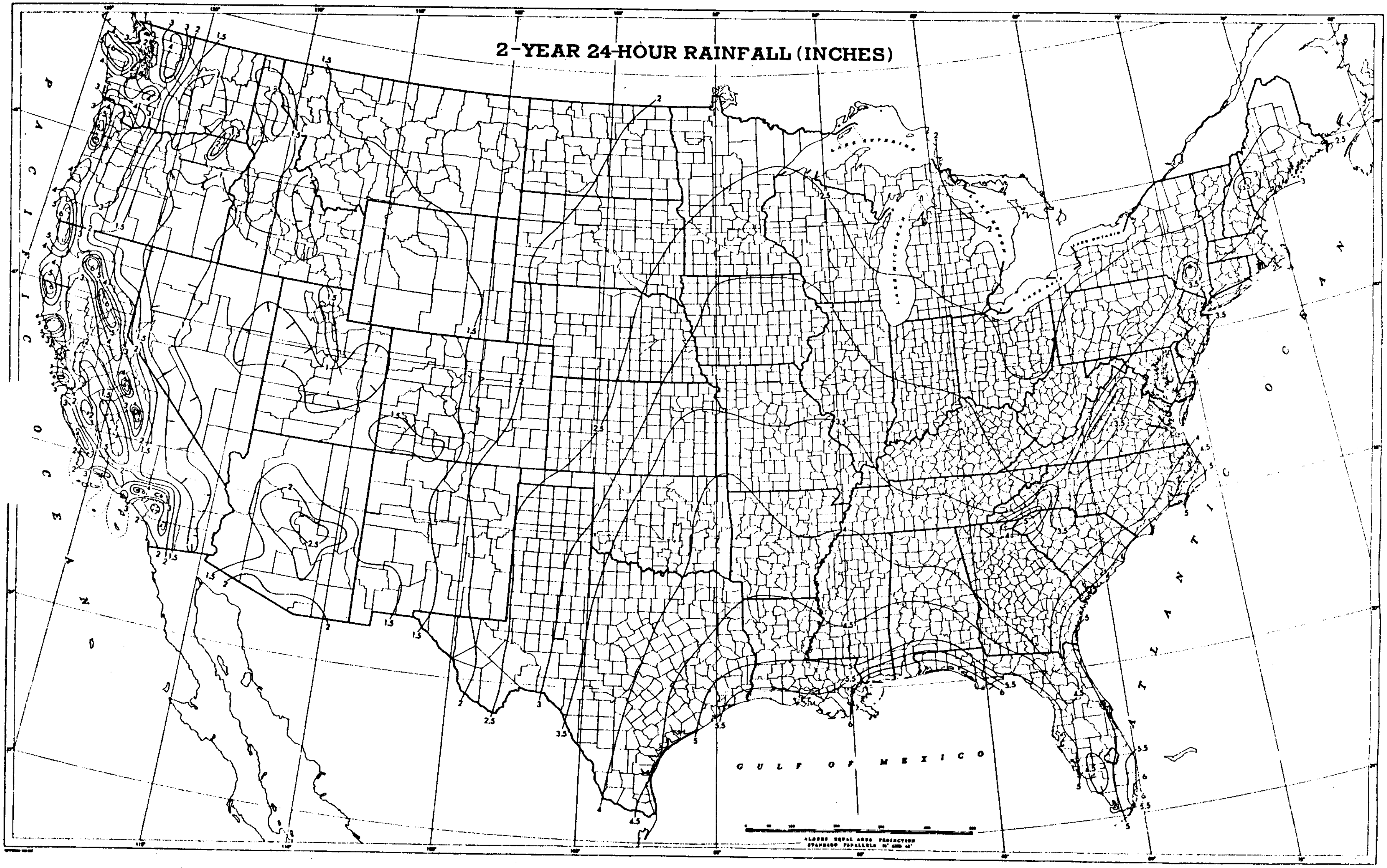


Chart for 2-year 24-hour rainfall (inches).

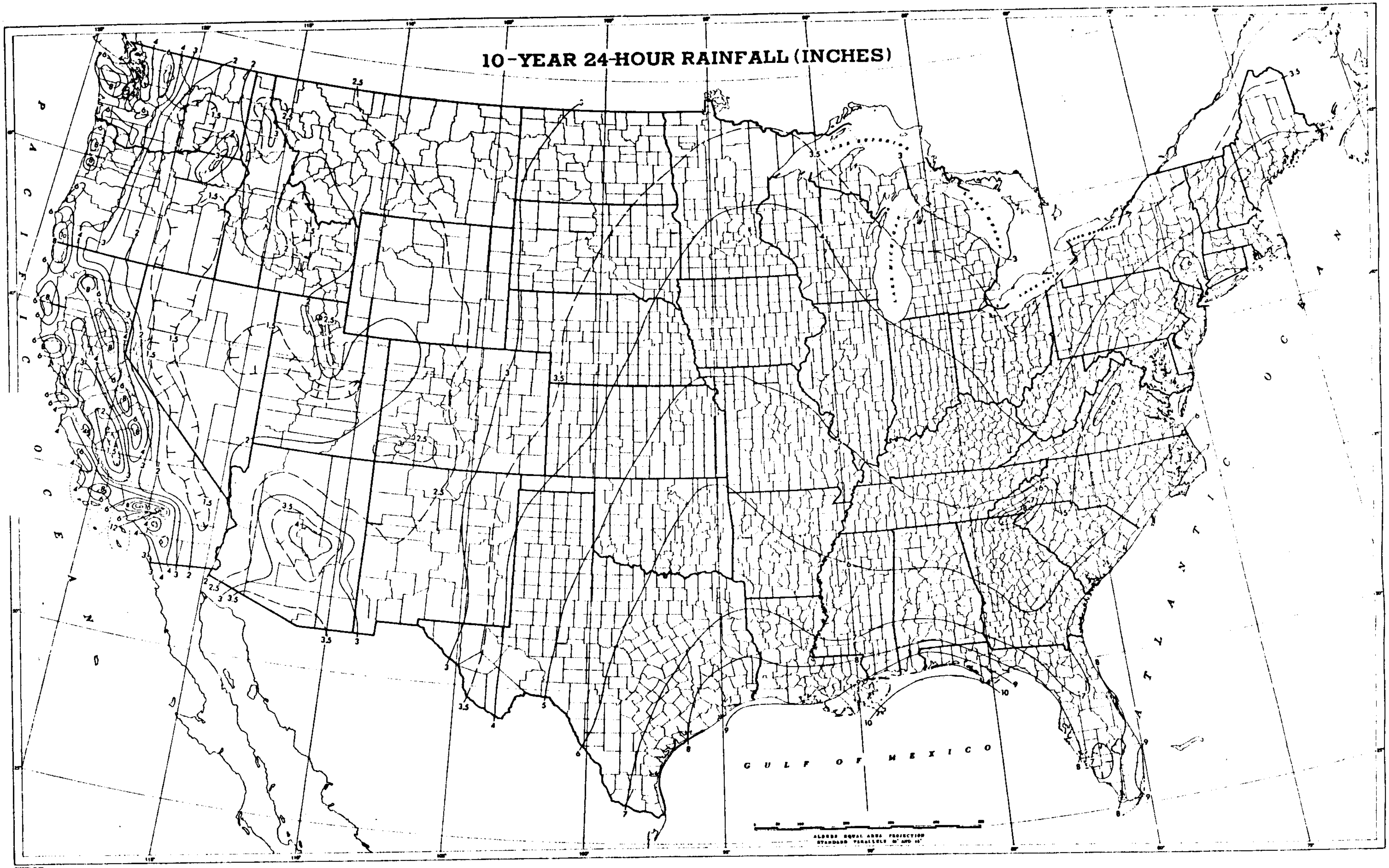


Chart for 10-year 24-hour rainfall (inches).

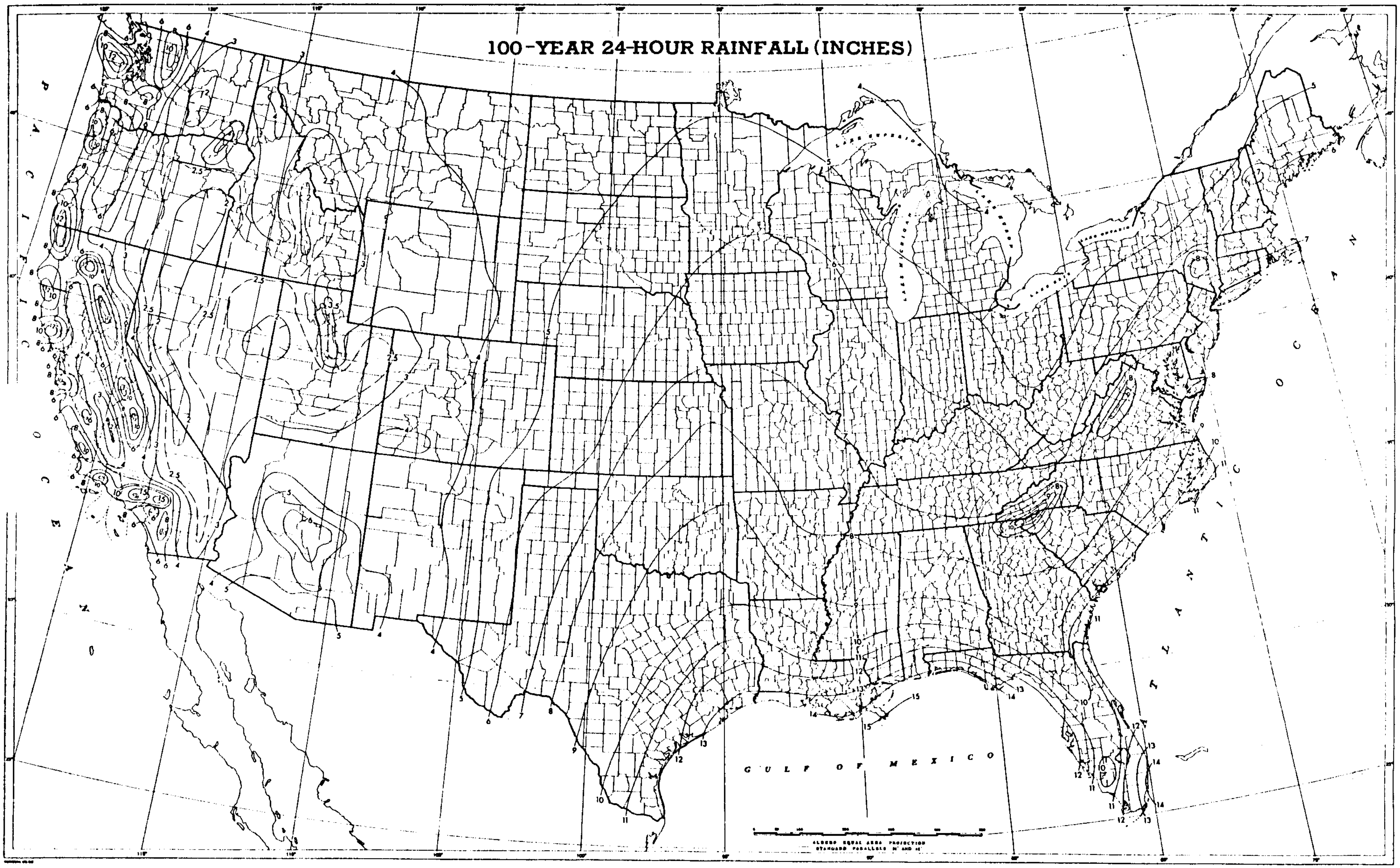


Chart for 100-year 24-hour rainfall (inches).

APPENDIX B

SCS TP-149

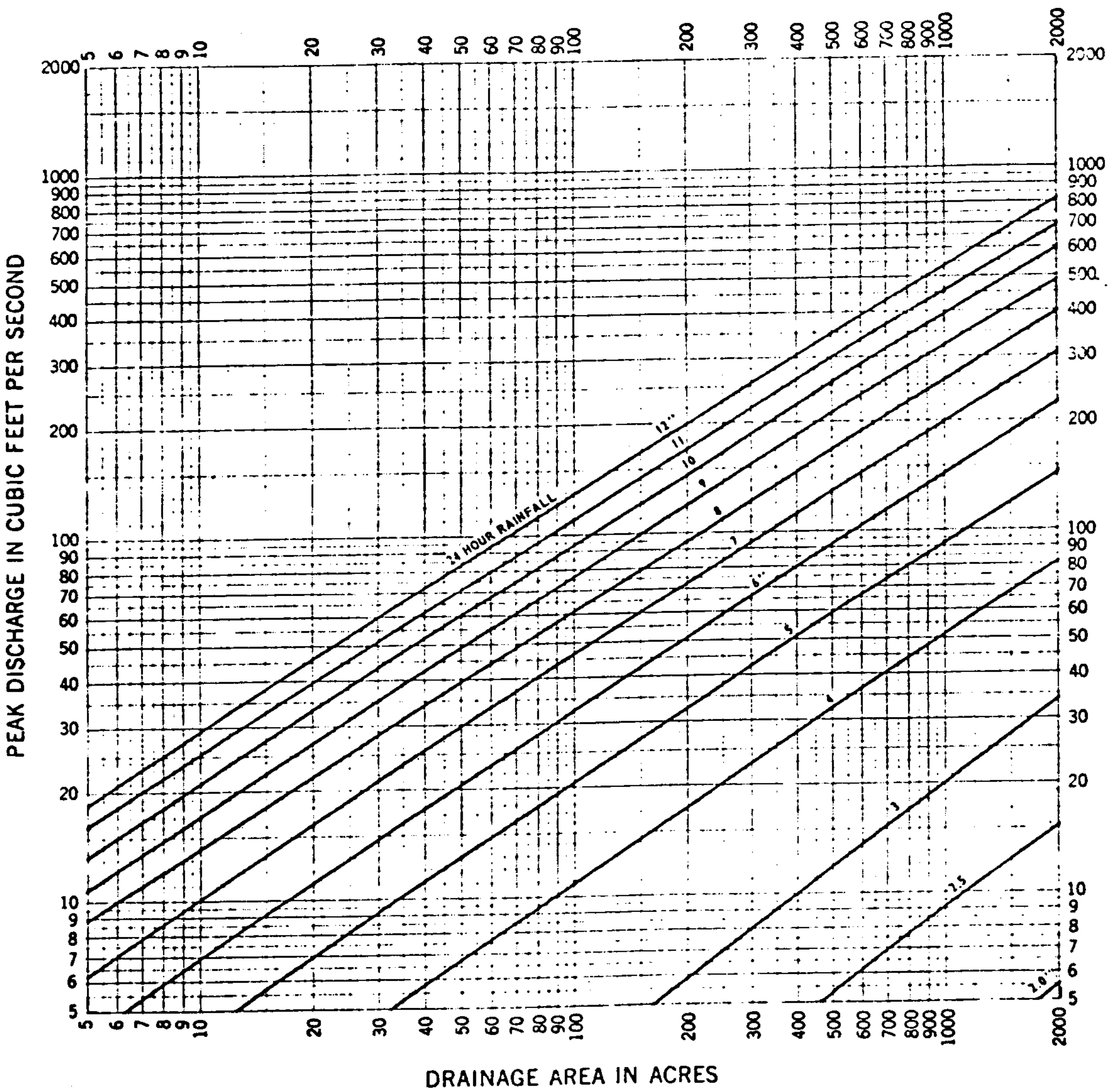
DESIGN CHARTS

This page intentionally left blank.

PEAK RATES OF DISCHARGE FOR SMALL WATERSHEDS
TYPE II STORM DISTRIBUTION

SLOPES - FLAT
CURVE NUMBER - 60

24 HOUR RAINFALL FROM US WB TP-40

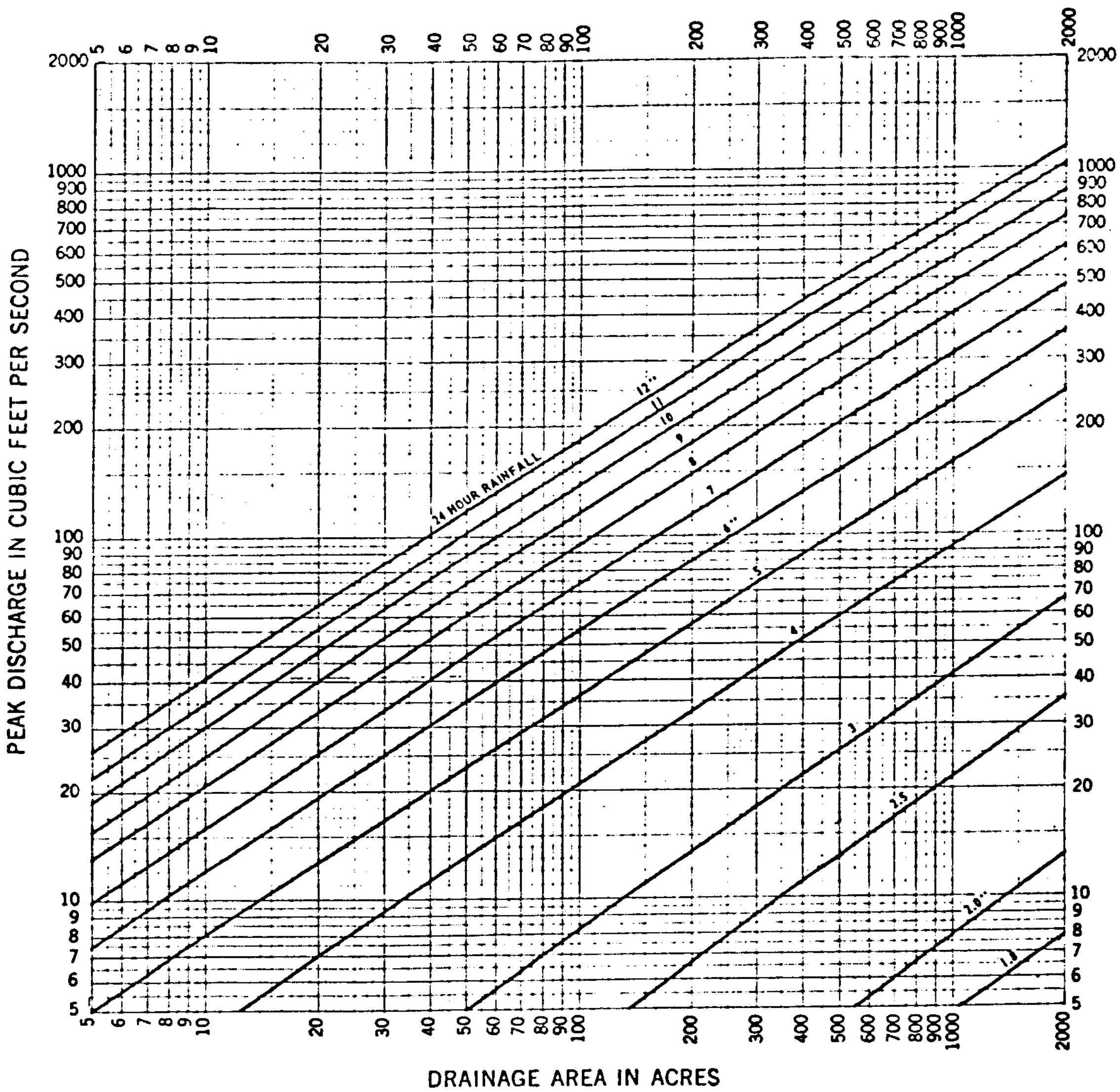


ES- 1027
SHEET 1 OF 21

PEAK RATES OF DISCHARGE FOR SMALL WATERSHEDS
TYPE II STORM DISTRIBUTION

SLOPES - FLAT
CURVE NUMBER 65

24 HOUR RAINFALL FROM US WB TP-40

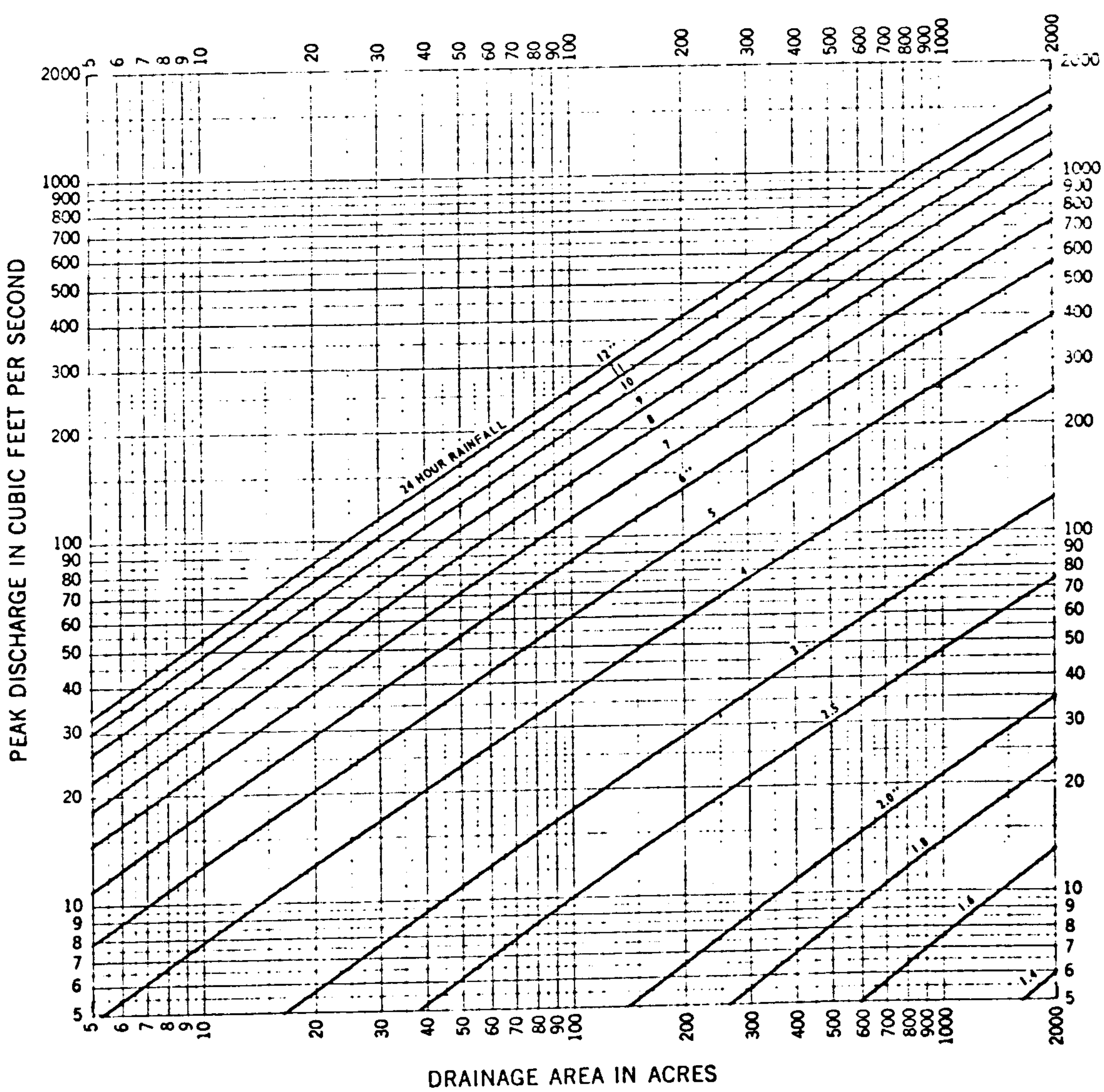


ES-1027
SHEET 2 OF 21

PEAK RATES OF DISCHARGE FOR SMALL WATERSHEDS
TYPE II STORM DISTRIBUTION

SLOPES - **FLAT**
CURVE NUMBER - 70

24 HOUR RAINFALL FROM US WB TP-40

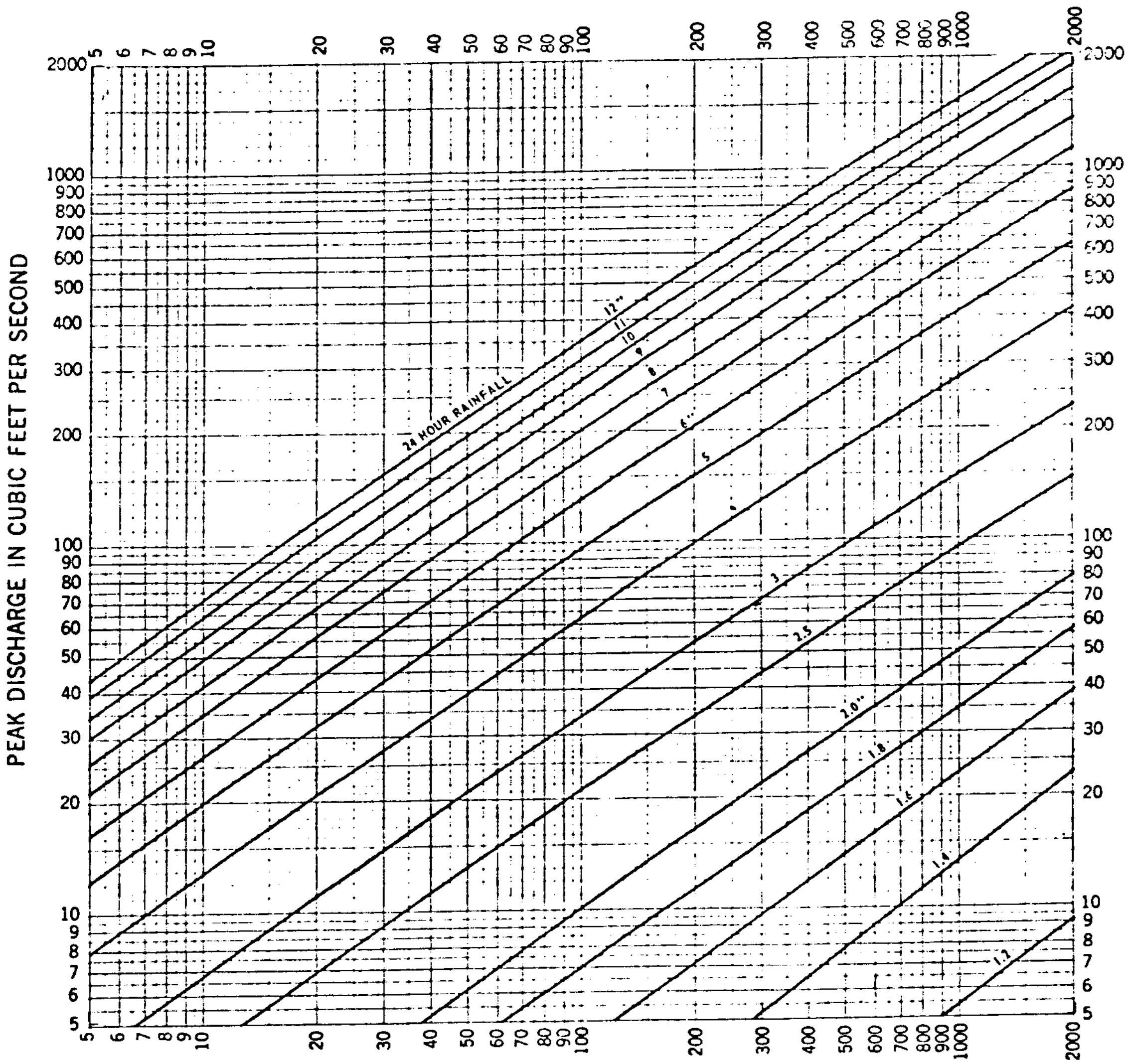


ES- 1027
SHEET 3 OF 21

PEAK RATES OF DISCHARGE FOR SMALL WATERSHEDS
TYPE II STORM DISTRIBUTION

SLOPES - FLAT
CURVE NUMBER - 75

24 HOUR RAINFALL FROM US WB TP-40

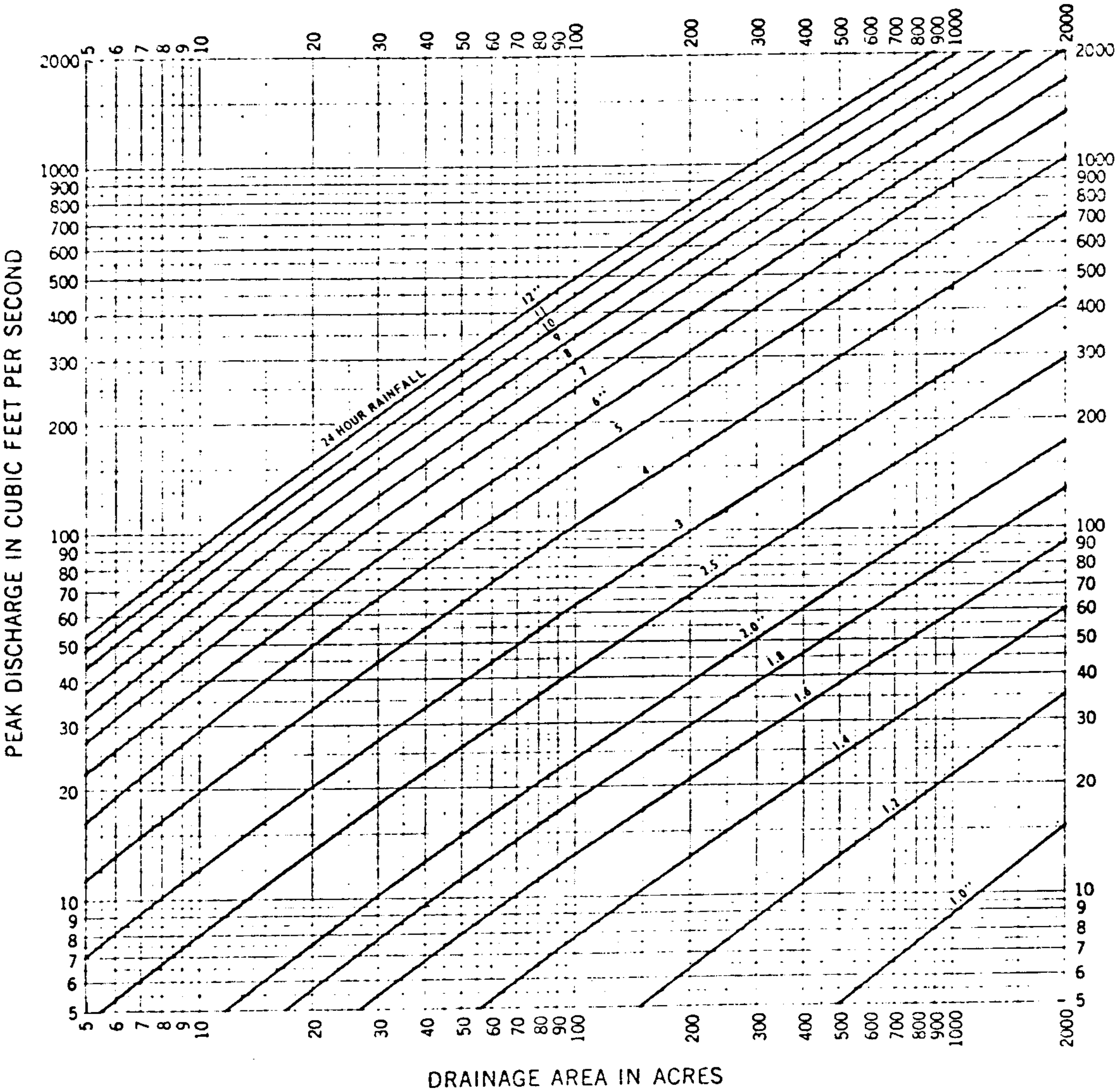


ES- 1027
SHEET 4 OF 21

PEAK RATES OF DISCHARGE FOR SMALL WATERSHEDS
TYPE II STORM DISTRIBUTION

SLOPES - FLAT
CURVE NUMBER - 80

24 HOUR RAINFALL FROM US WB TP-40

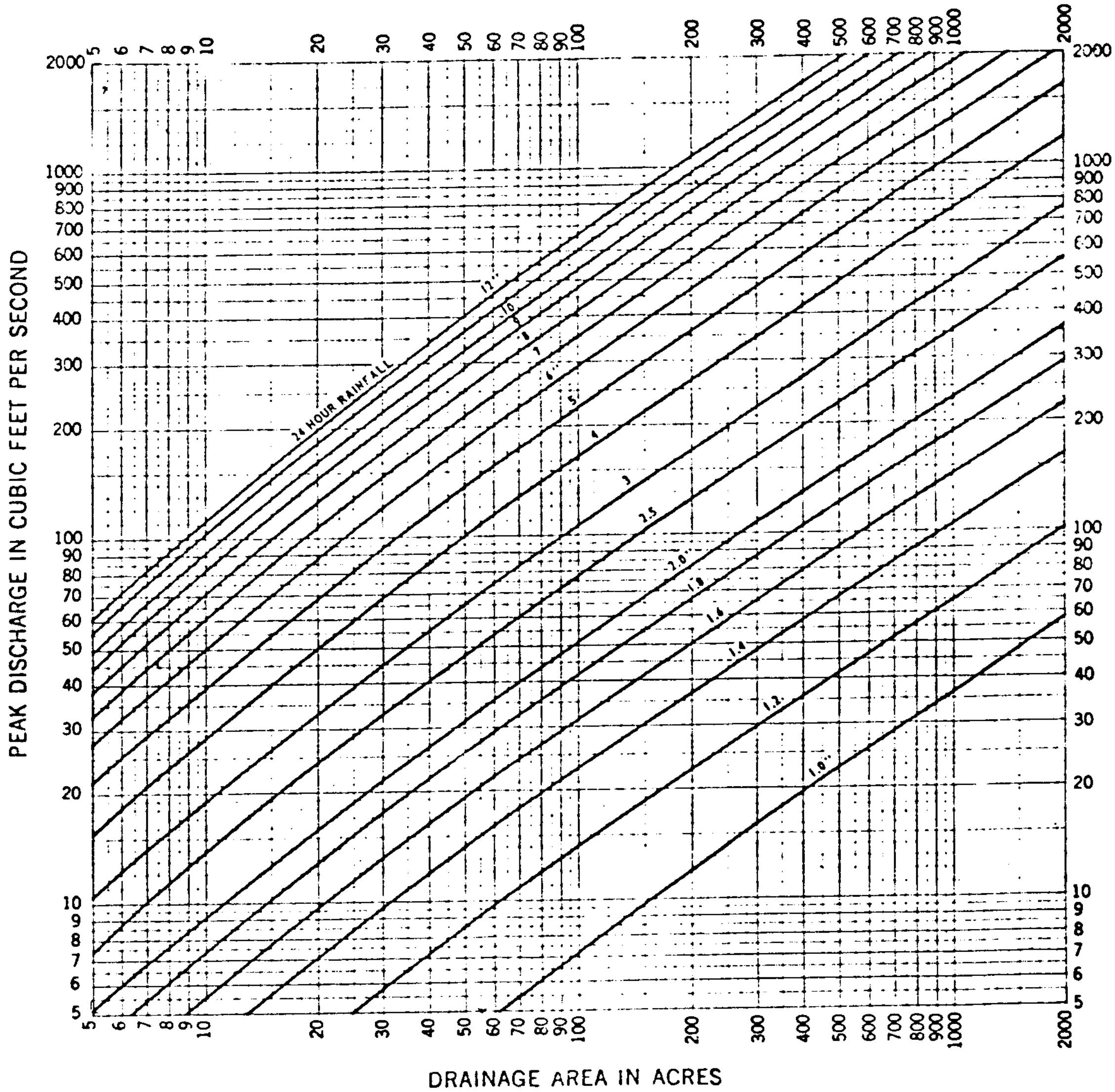


ES- 1027
SHEET 5 OF 21

PEAK RATES OF DISCHARGE FOR SMALL WATERSHEDS
TYPE II STORM DISTRIBUTION

SLOPES - FLAT
CURVE NUMBER - 85

24 HOUR RAINFALL FROM US WB TP-40

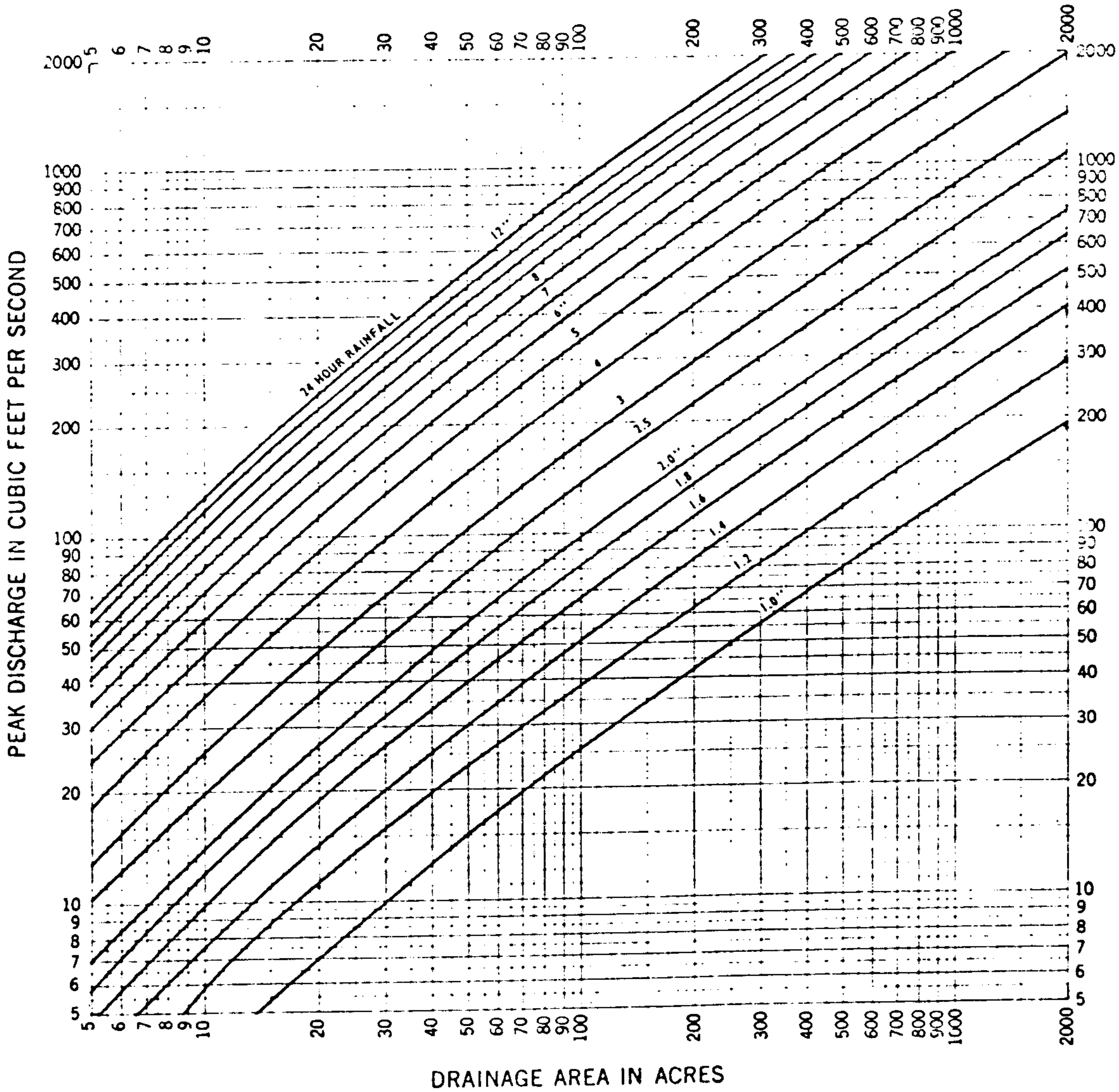


ES- 1027
SHEET 6 OF 21

PEAK RATES OF DISCHARGE FOR SMALL WATERSHEDS
TYPE II STORM DISTRIBUTION

SLOPES - FLAT
CURVE NUMBER - 90

24 HOUR RAINFALL FROM US WB TP-40

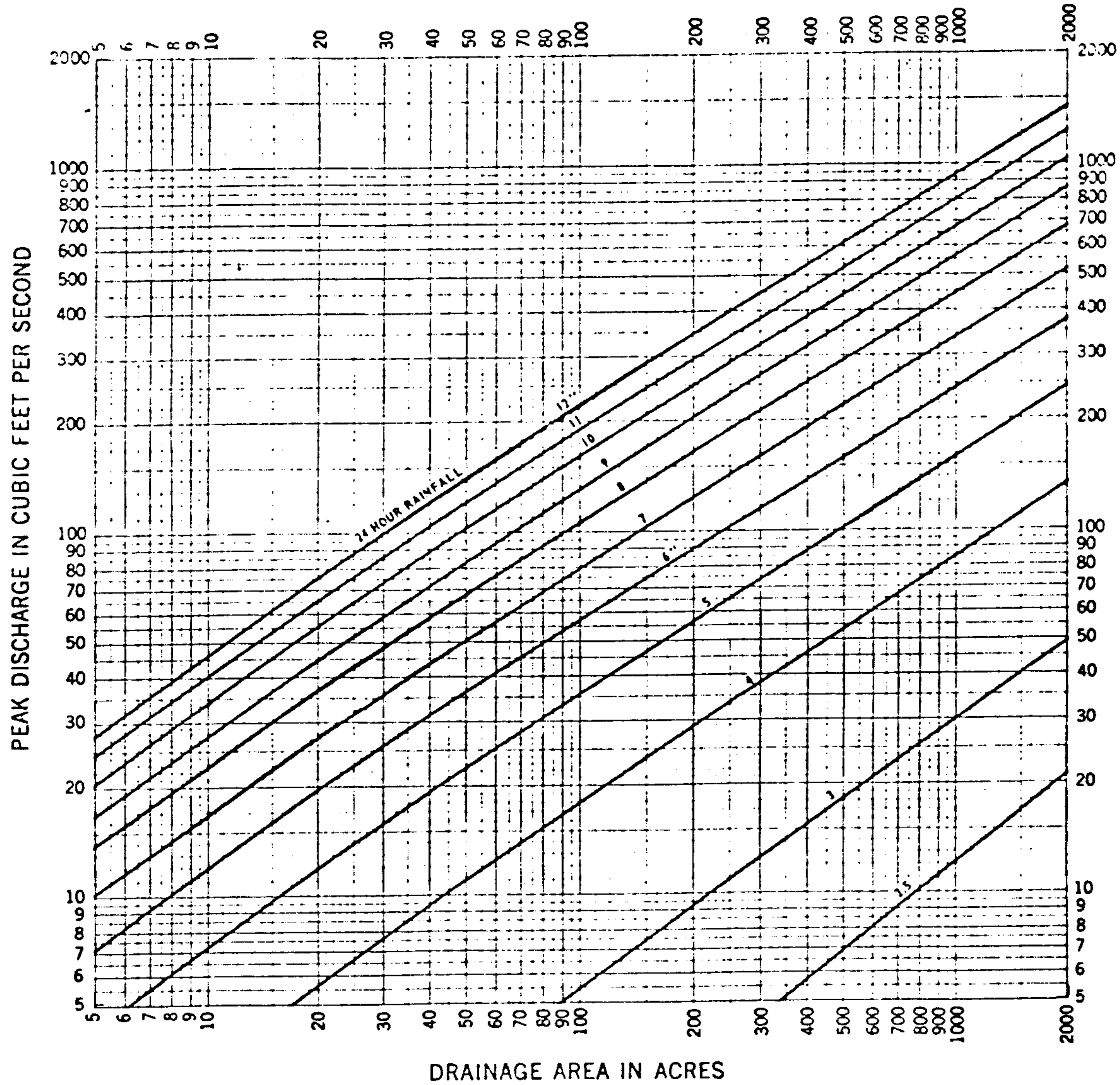


ES- 1027
SHEET 7 OF 21

PEAK RATES OF DISCHARGE FOR SMALL WATERSHEDS
TYPE II STORM DISTRIBUTION

SLOPES - MODERATE
CURVE NUMBER - 60

24 HOUR RAINFALL FROM US WB TP-40

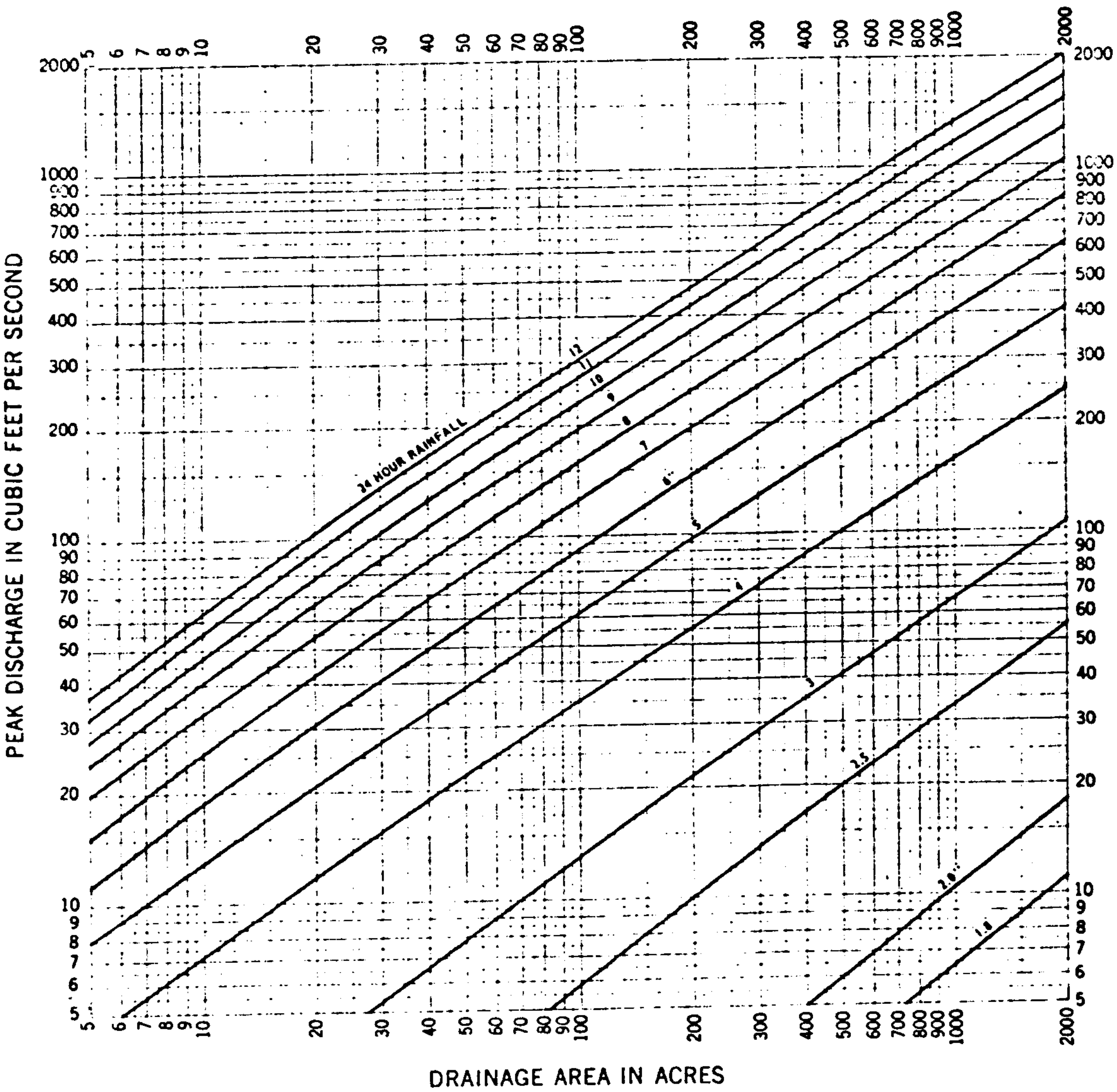


ES- 1027
SHEET 8 OF 21

PEAK RATES OF DISCHARGE FOR SMALL WATERSHEDS
TYPE II STORM DISTRIBUTION

SLOPES - MODERATE
CURVE NUMBER - 65

24 HOUR RAINFALL FROM US WB TP-40

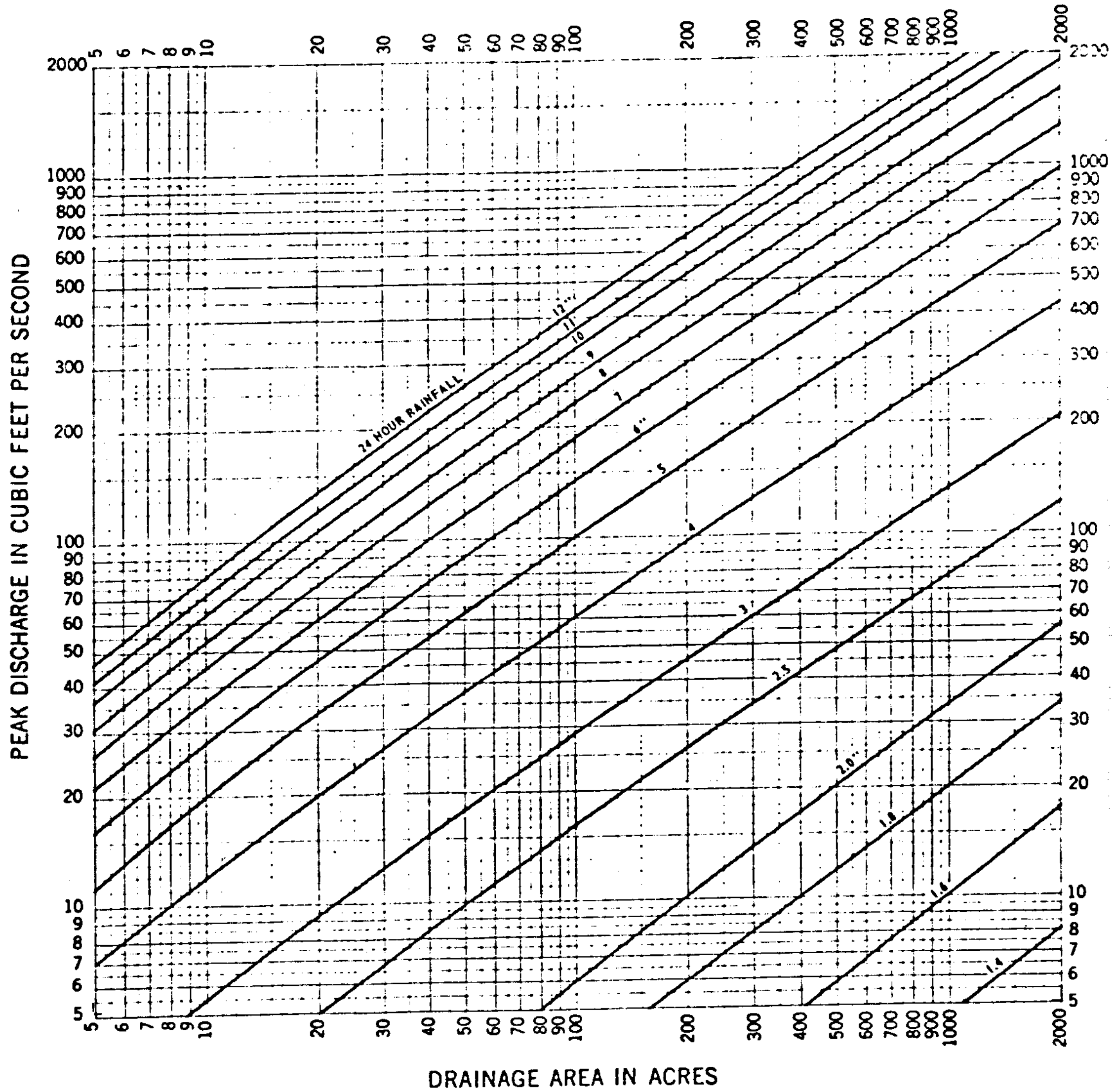


ES- 1027
SHEET 9 OF 21

PEAK RATES OF DISCHARGE FOR SMALL WATERSHEDS
TYPE II STORM DISTRIBUTION

SLOPES - MODERATE
CURVE NUMBER - 70

24 HOUR RAINFALL FROM US WB TP-40

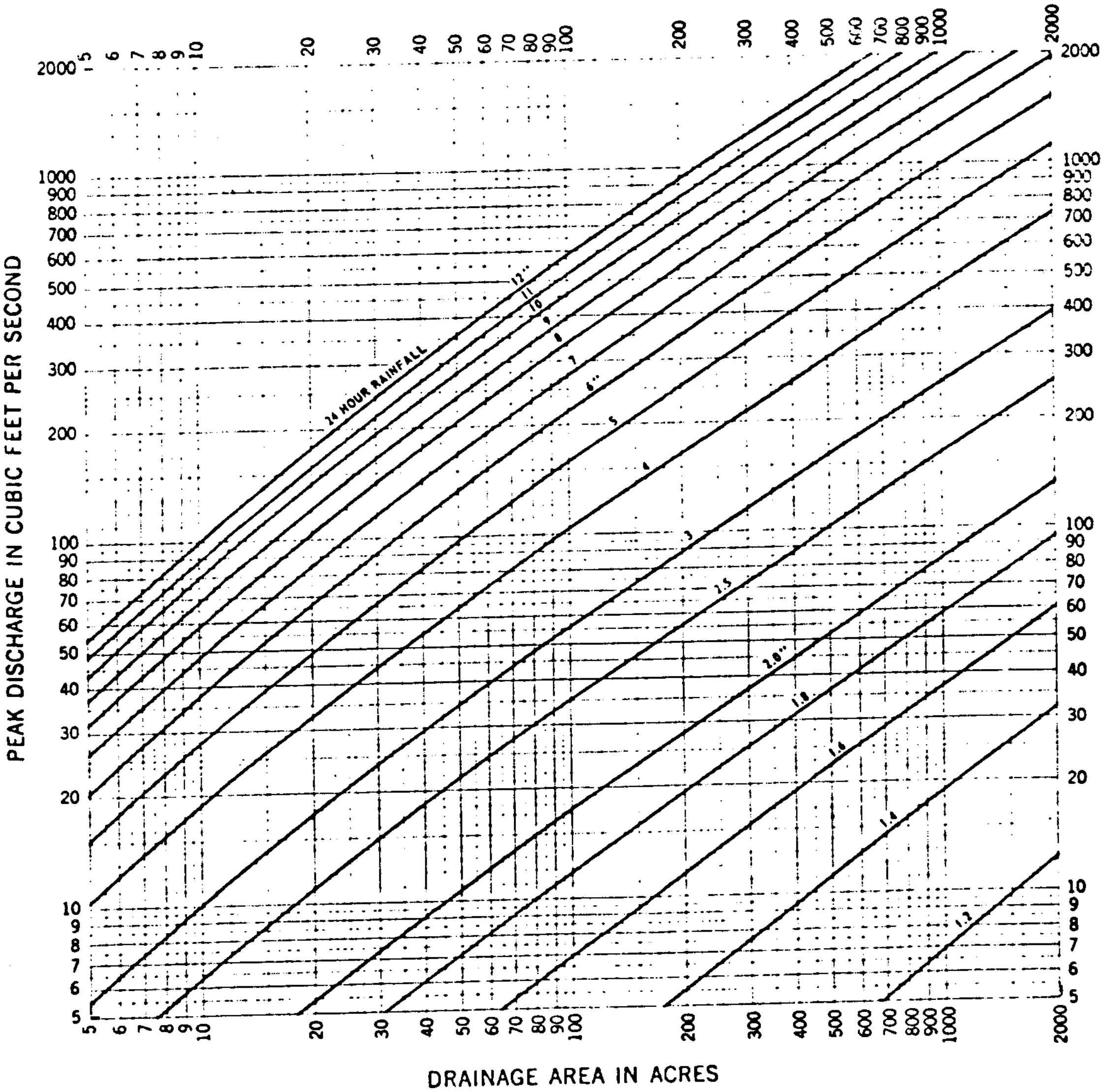


ES- 1027
SHEET 10 OF 21

PEAK RATES OF DISCHARGE FOR SMALL WATERSHEDS
TYPE II STORM DISTRIBUTION

SLOPES - MODERATE
CURVE NUMBER - 75

24 HOUR RAINFALL FROM US WB TP-40

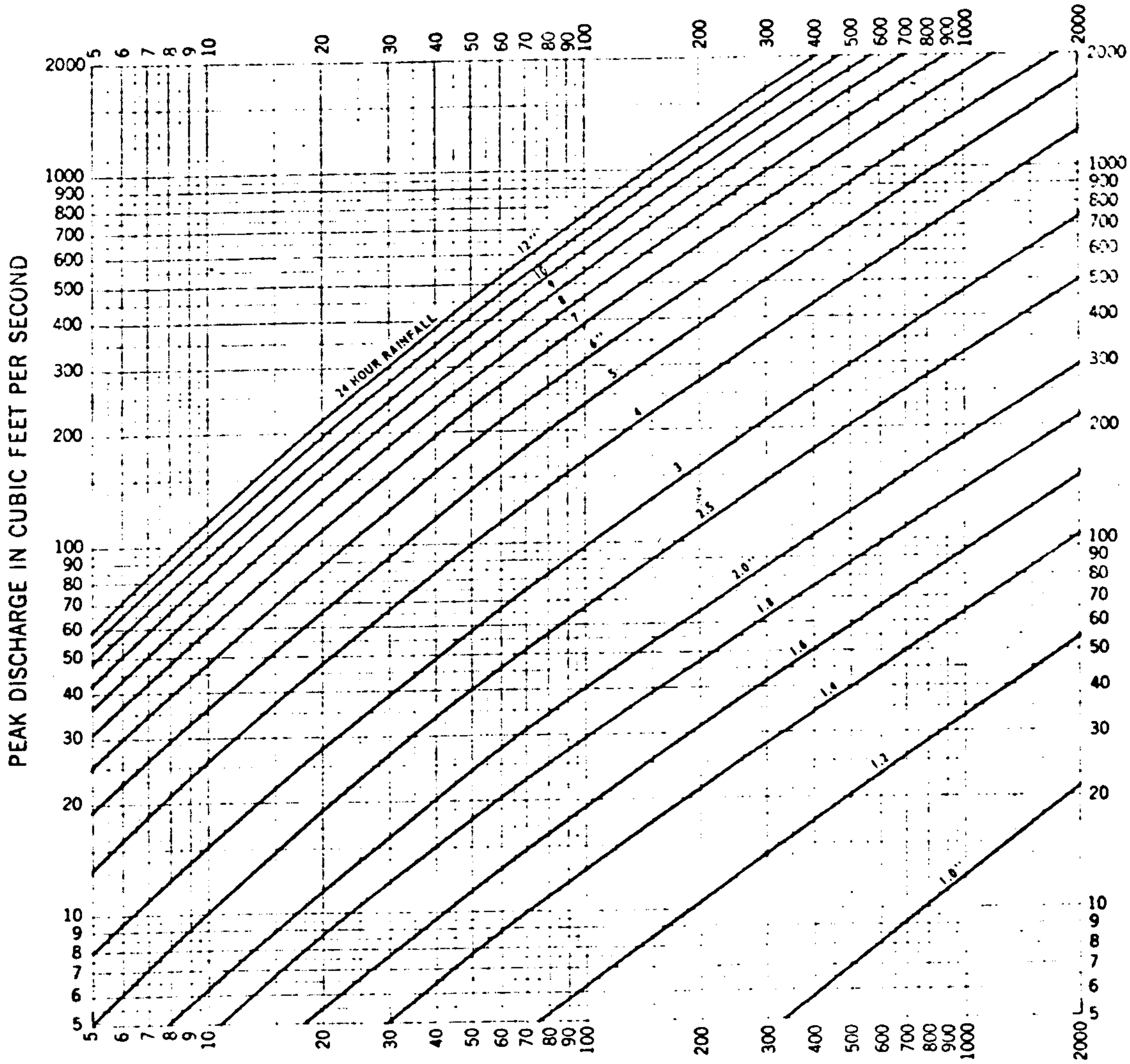


ES- 1027
SHEET 11 OF 21

PEAK RATES OF DISCHARGE FOR SMALL WATERSHEDS
TYPE II STORM DISTRIBUTION

SLOPES - MODERATE
CURVE NUMBER - 80

24 HOUR RAINFALL FROM US WB TP-40

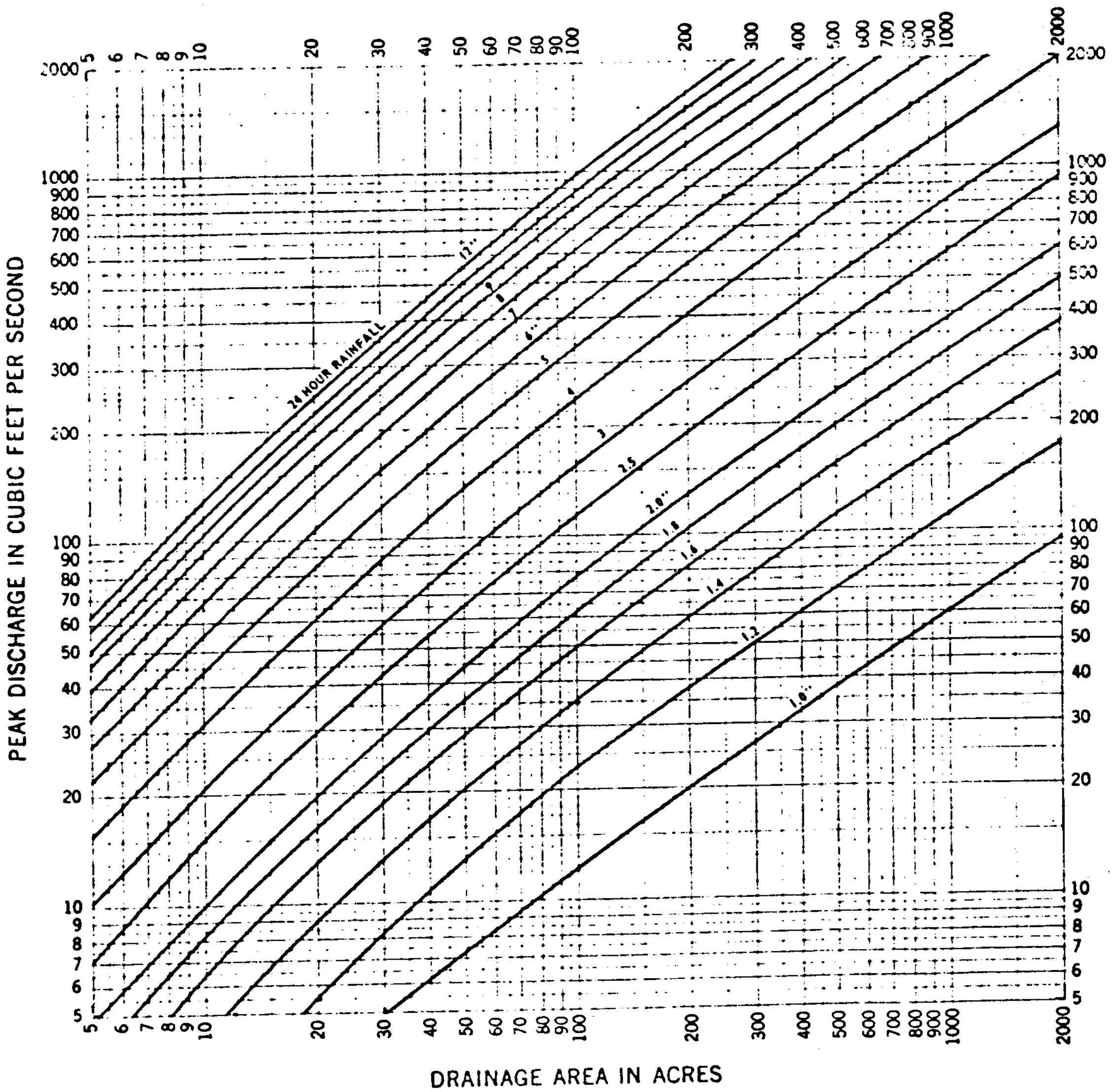


ES- 1027
SHEET 12 OF 21

PEAK RATES OF DISCHARGE FOR SMALL WATERSHEDS
TYPE II STORM DISTRIBUTION

SLOPES - MODERATE
CURVE NUMBER - 85

24 HOUR RAINFALL FROM US WB TP-40

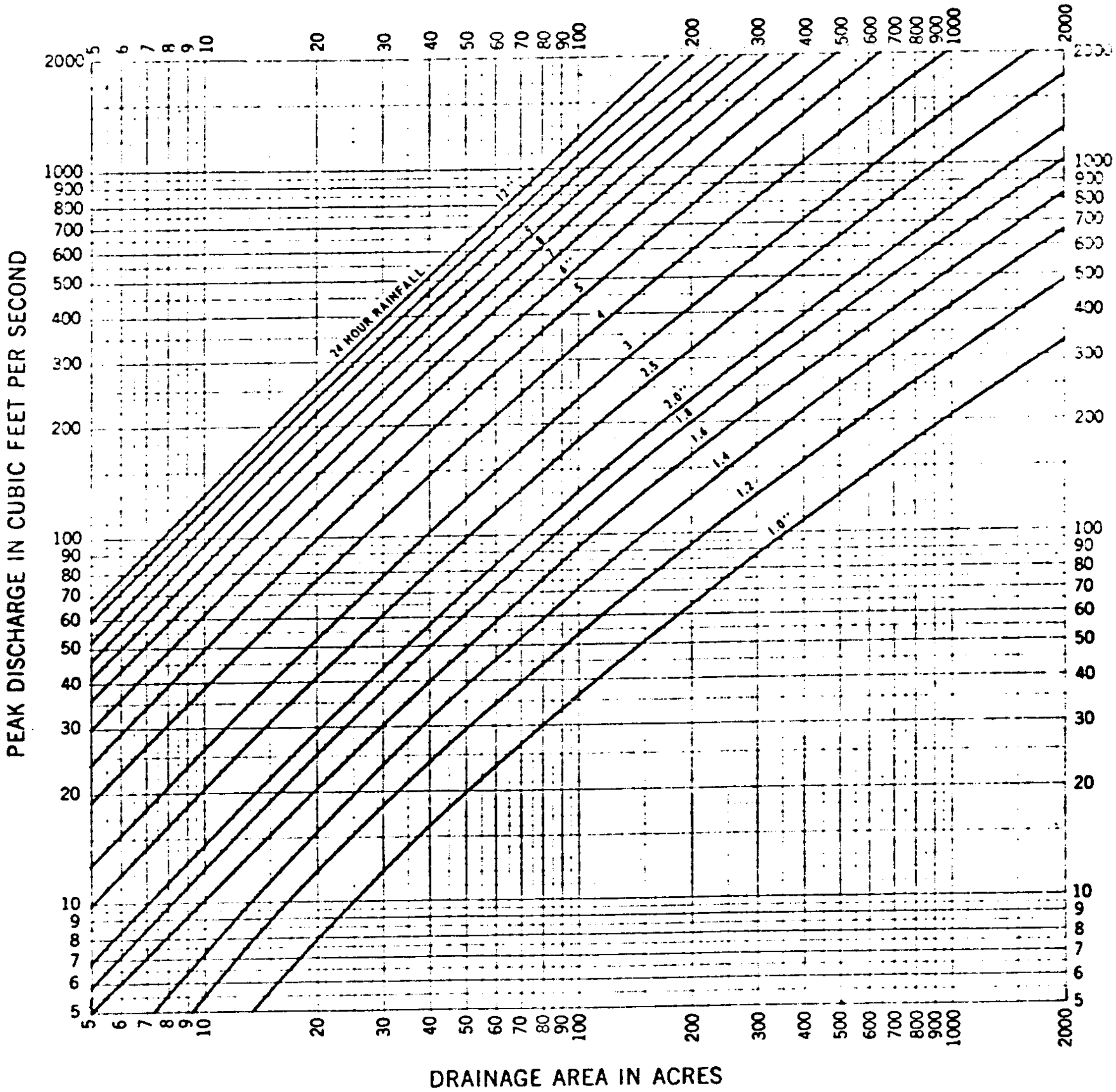


ES- 1027
SHEET 13 OF 21

PEAK RATES OF DISCHARGE FOR SMALL WATERSHEDS
TYPE II STORM DISTRIBUTION

SLOPES – MODERATE
CURVE NUMBER – 90

24 HOUR RAINFALL FROM US WB TP-40

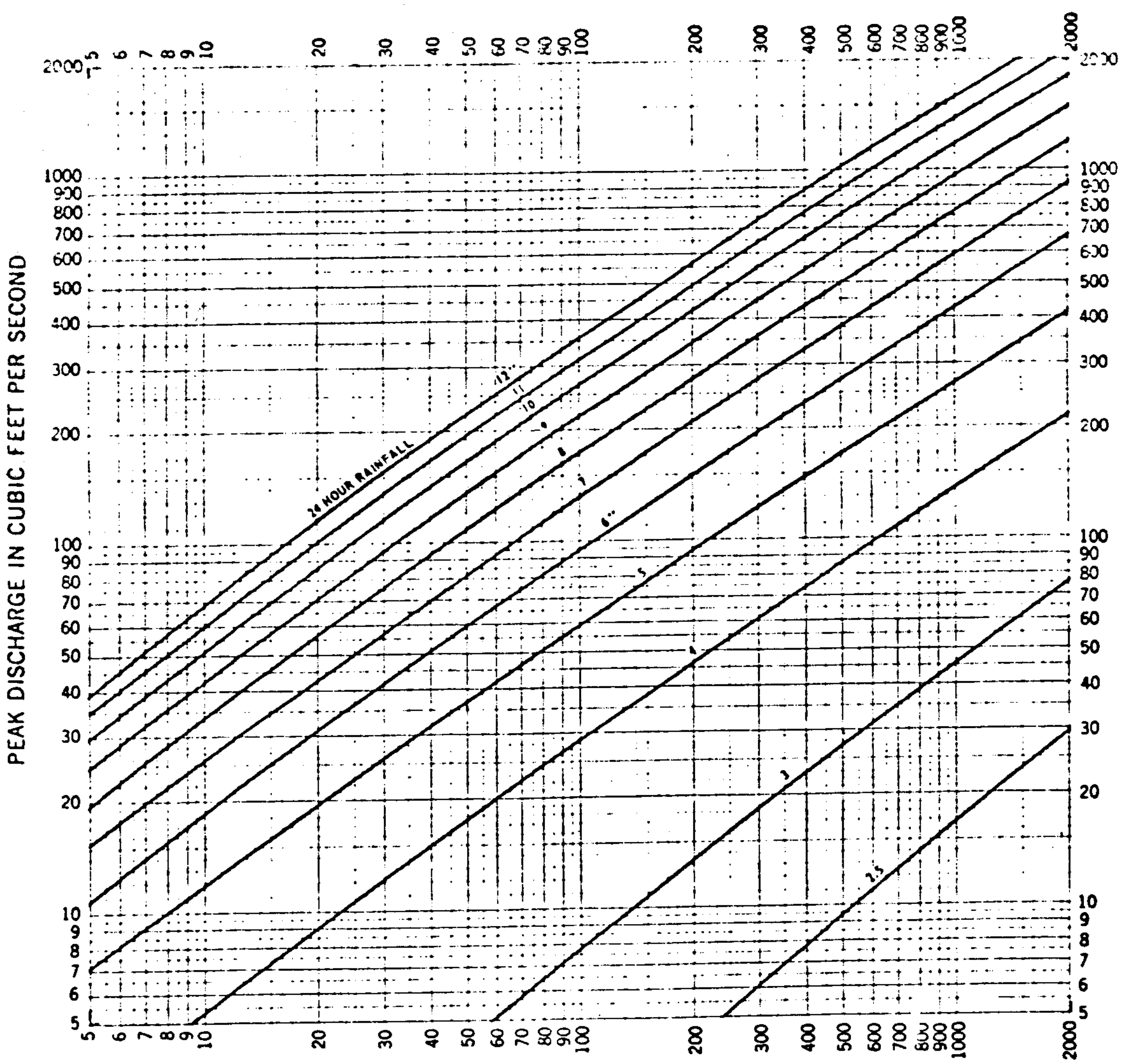


ES- 1027
SHEET 14 OF 21

PEAK RATES OF DISCHARGE FOR SMALL WATERSHEDS
TYPE II STORM DISTRIBUTION

SLOPES - STEEP
CURVE NUMBER - 60

24 HOUR RAINFALL FROM US WB TP-40

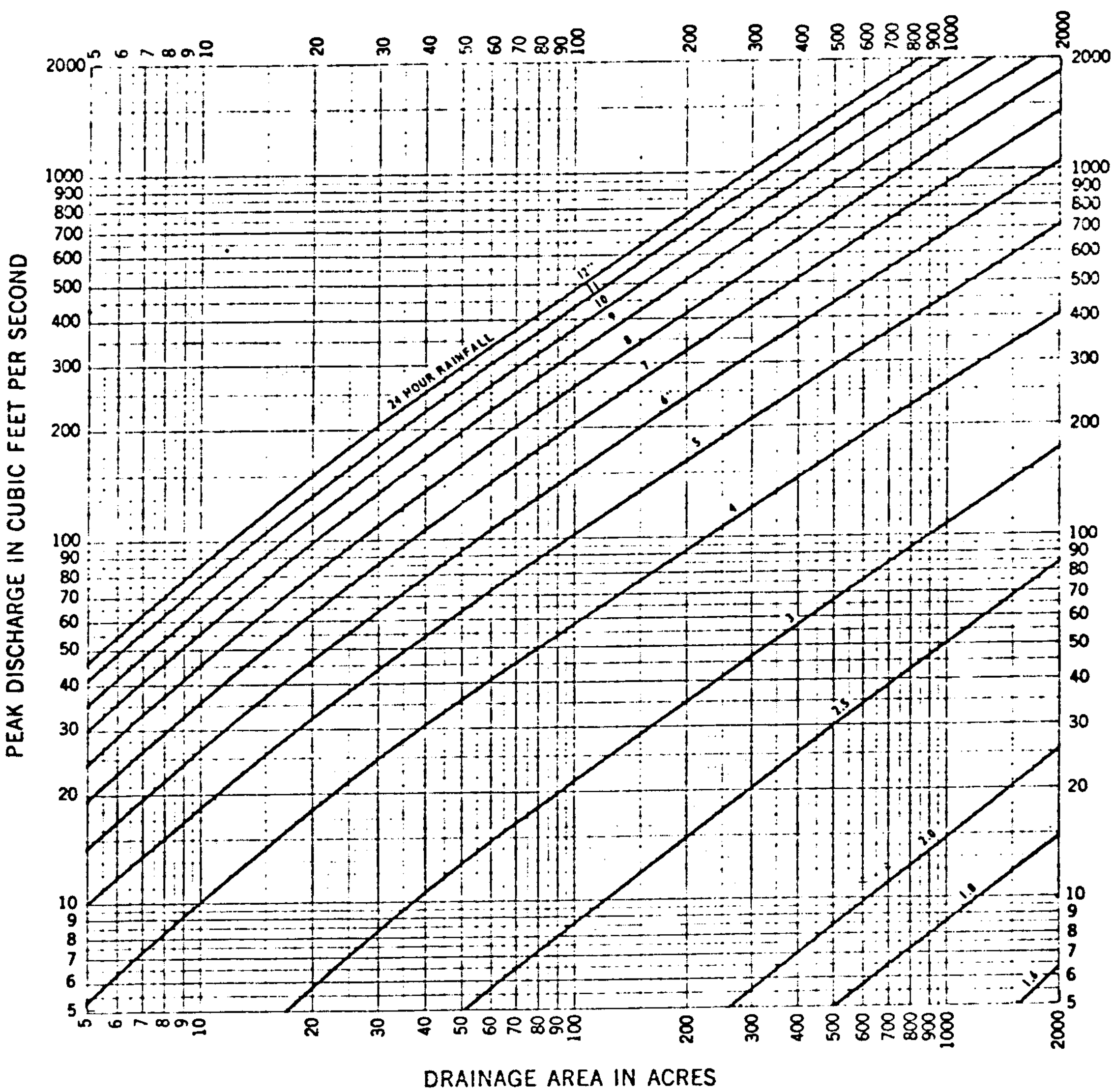


ES- 1027
SHEET 15 OF 21

PEAK RATES OF DISCHARGE FOR SMALL WATERSHEDS
TYPE II STORM DISTRIBUTION

SLOPES - STEEP
CURVE NUMBER - 65

24 HOUR RAINFALL FROM US WB TP-40

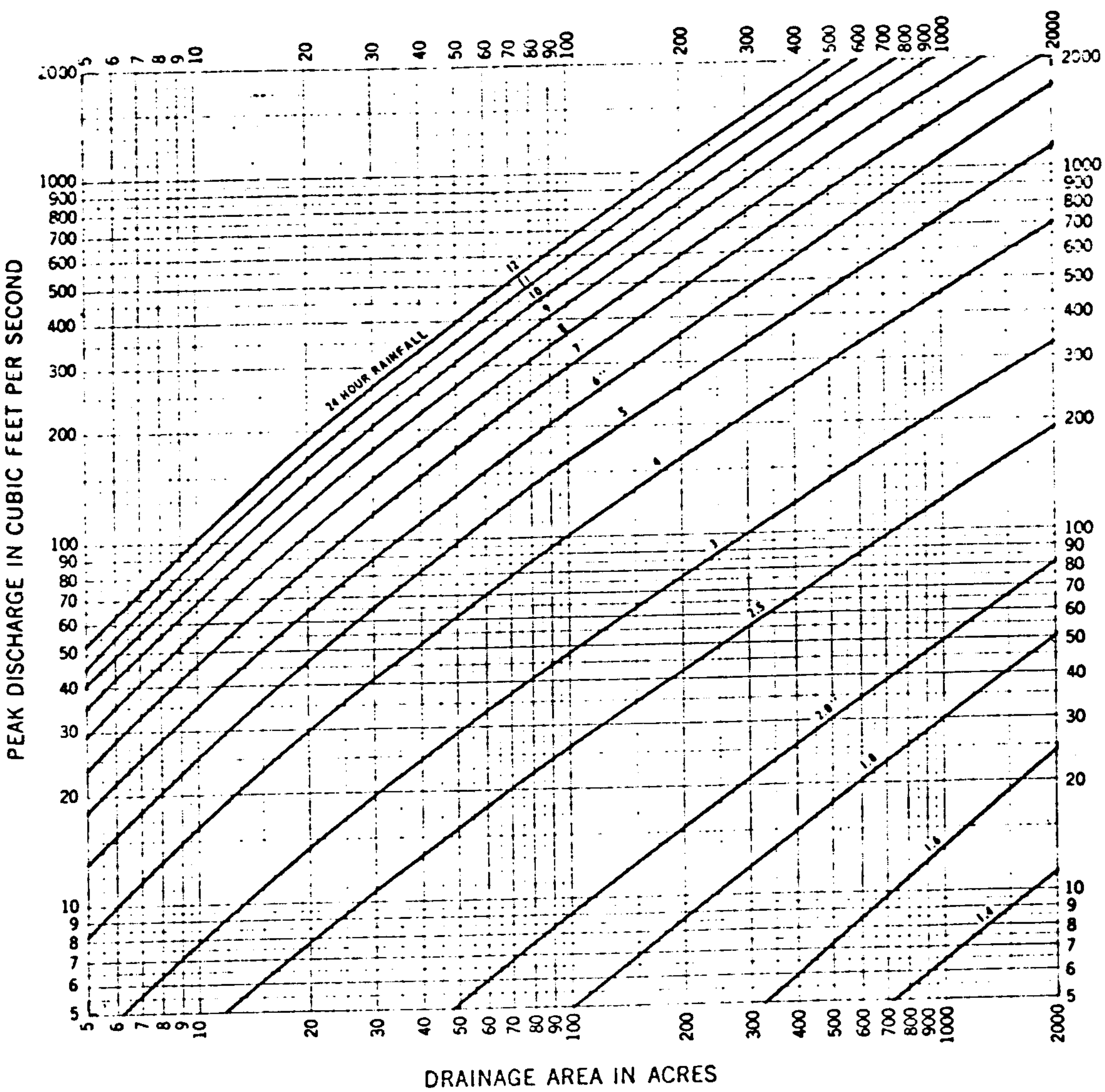


ES- 1027
SHEET 16 OF 21

PEAK RATES OF DISCHARGE FOR SMALL WATERSHEDS
TYPE II STORM DISTRIBUTION

SLOPES - STEEP
CURVE NUMBER - 70

24 HOUR RAINFALL FROM US WB TP-40

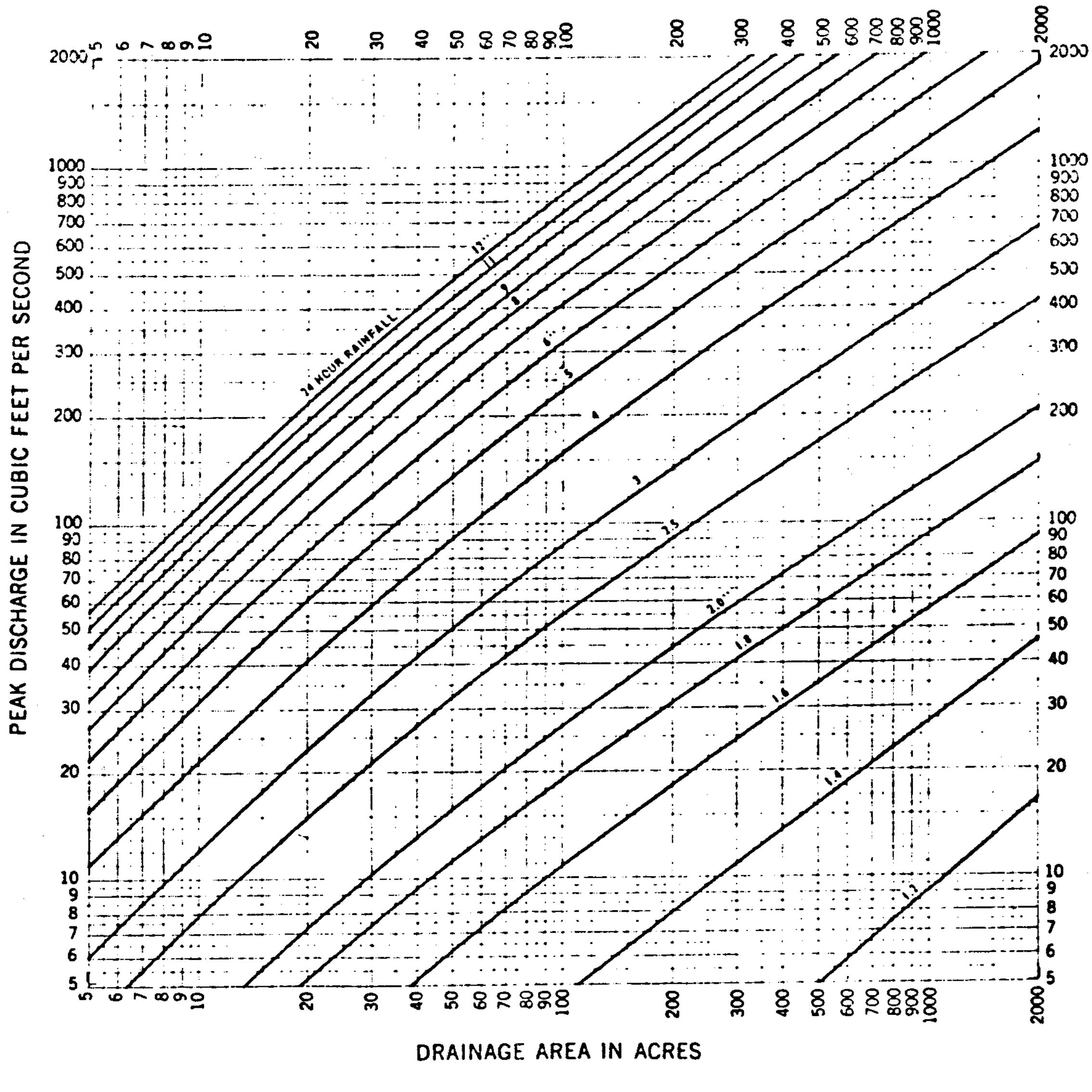


ES- 1027
SHEET 17 OF 21

PEAK RATES OF DISCHARGE FOR SMALL WATERSHEDS
TYPE II STORM DISTRIBUTION

SLOPES - STEEP
CURVE NUMBER - 75

24 HOUR RAINFALL FROM US WB TP-40

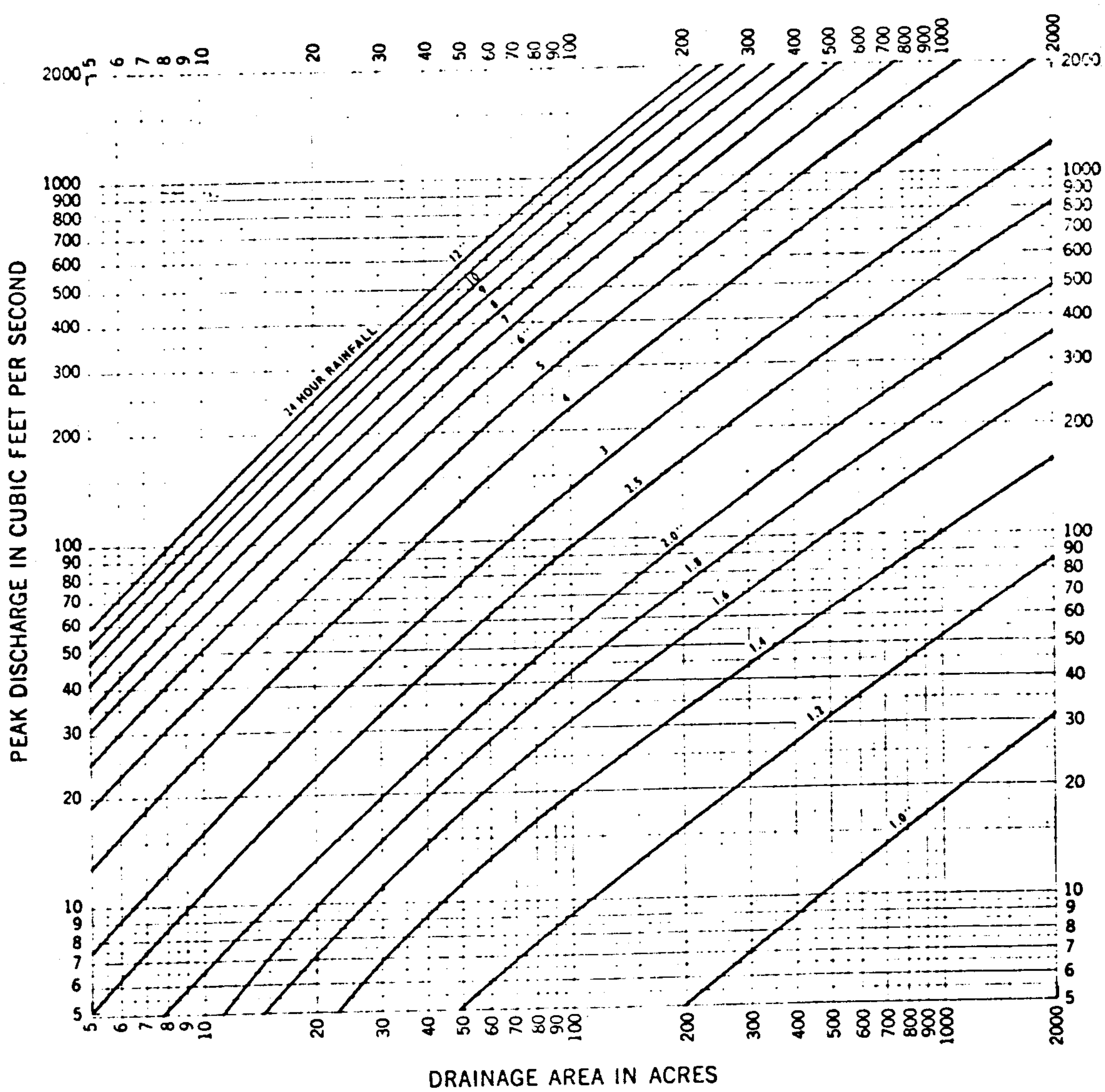


ES- 1027
SHEET 18 OF 21

PEAK RATES OF DISCHARGE FOR SMALL WATERSHEDS
TYPE II STORM DISTRIBUTION

SLOPES - STEEP
CURVE NUMBER - 80

24 HOUR RAINFALL FROM US WB TP-40

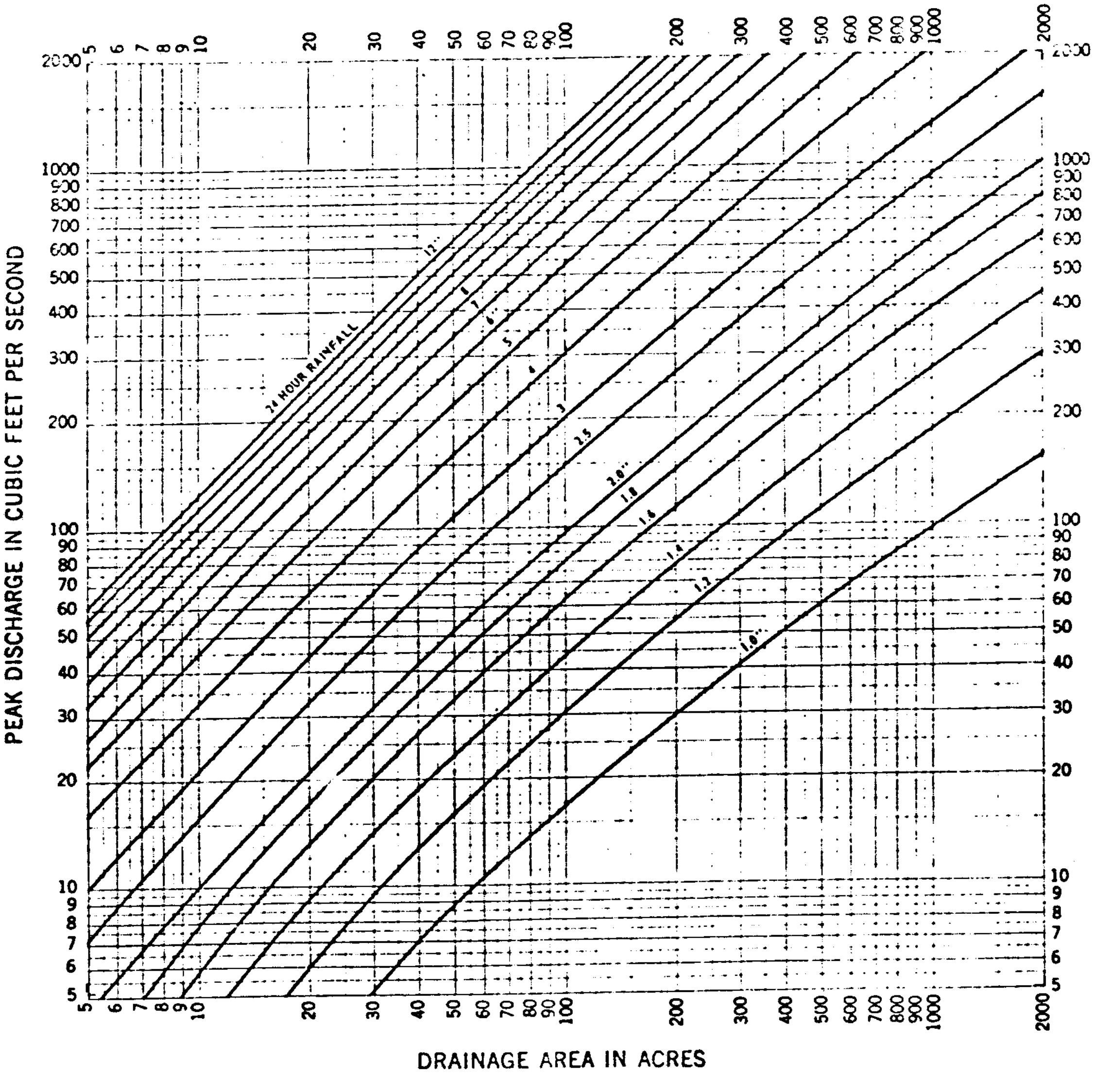


ES- 1027
SHEET 19 OF 21

PEAK RATES OF DISCHARGE FOR SMALL WATERSHEDS
TYPE II STORM DISTRIBUTION

SLOPES - STEEP
CURVE NUMBER - 85

24 HOUR RAINFALL FROM US WB TP-40

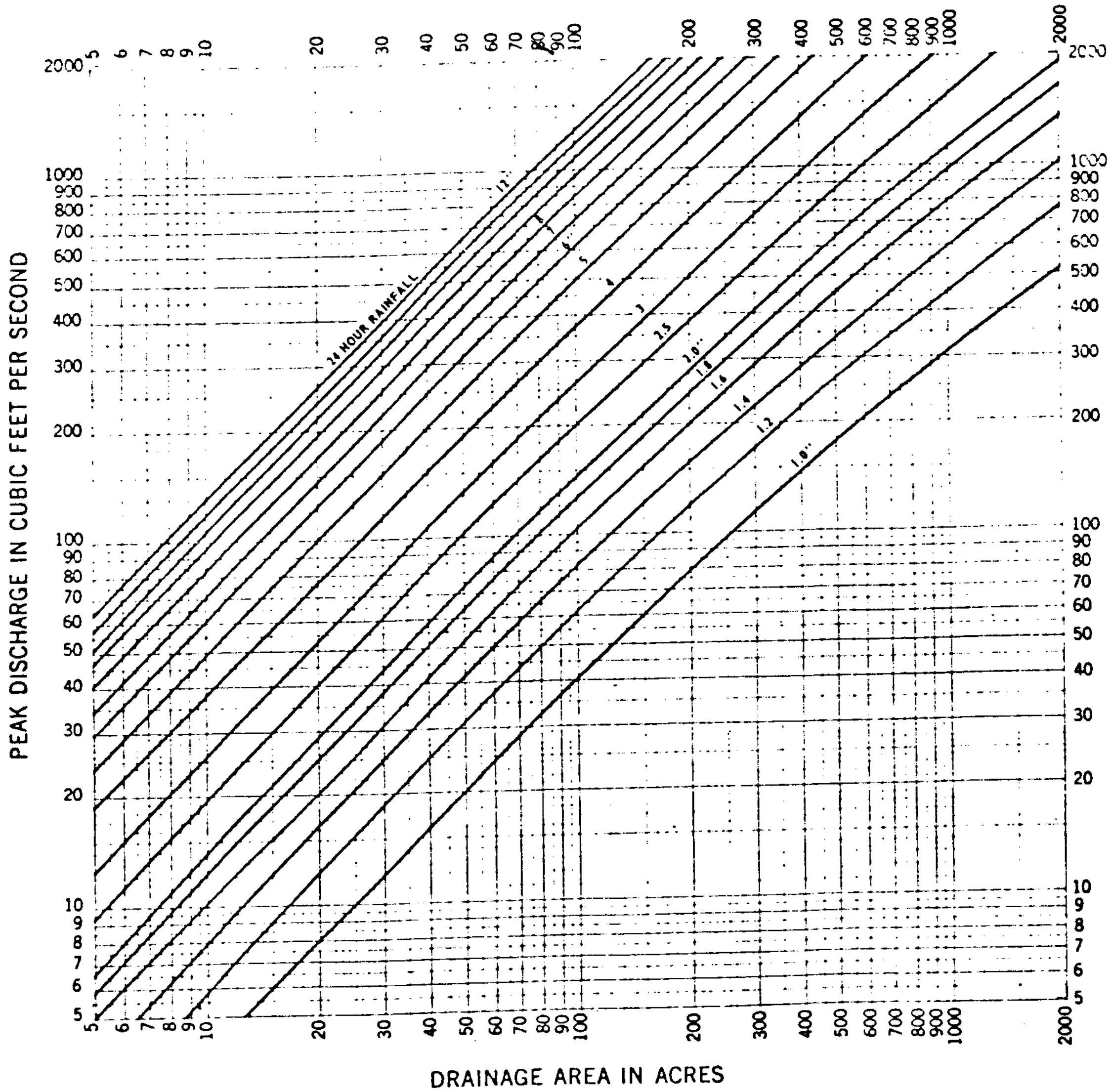


ES- 1027
SHEET 20 OF 21

PEAK RATES OF DISCHARGE FOR SMALL WATERSHEDS
TYPE II STORM DISTRIBUTION

SLOPES - STEEP
CURVE NUMBER - 90

24 HOUR RAINFALL FROM US WB TP-40



ES- 1027
SHEET 21 OF 21

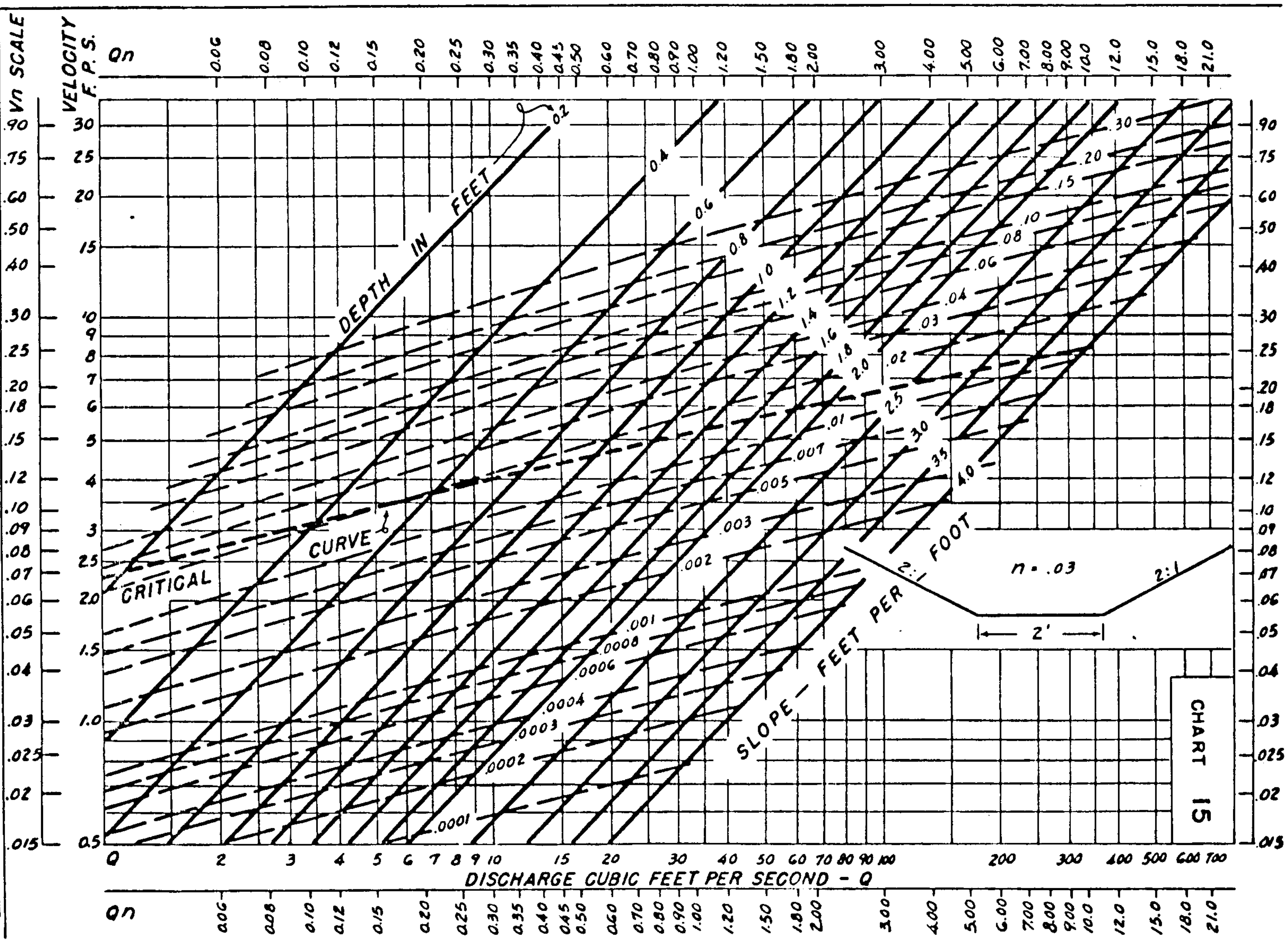
This page intentionally left blank.

APPENDIX C

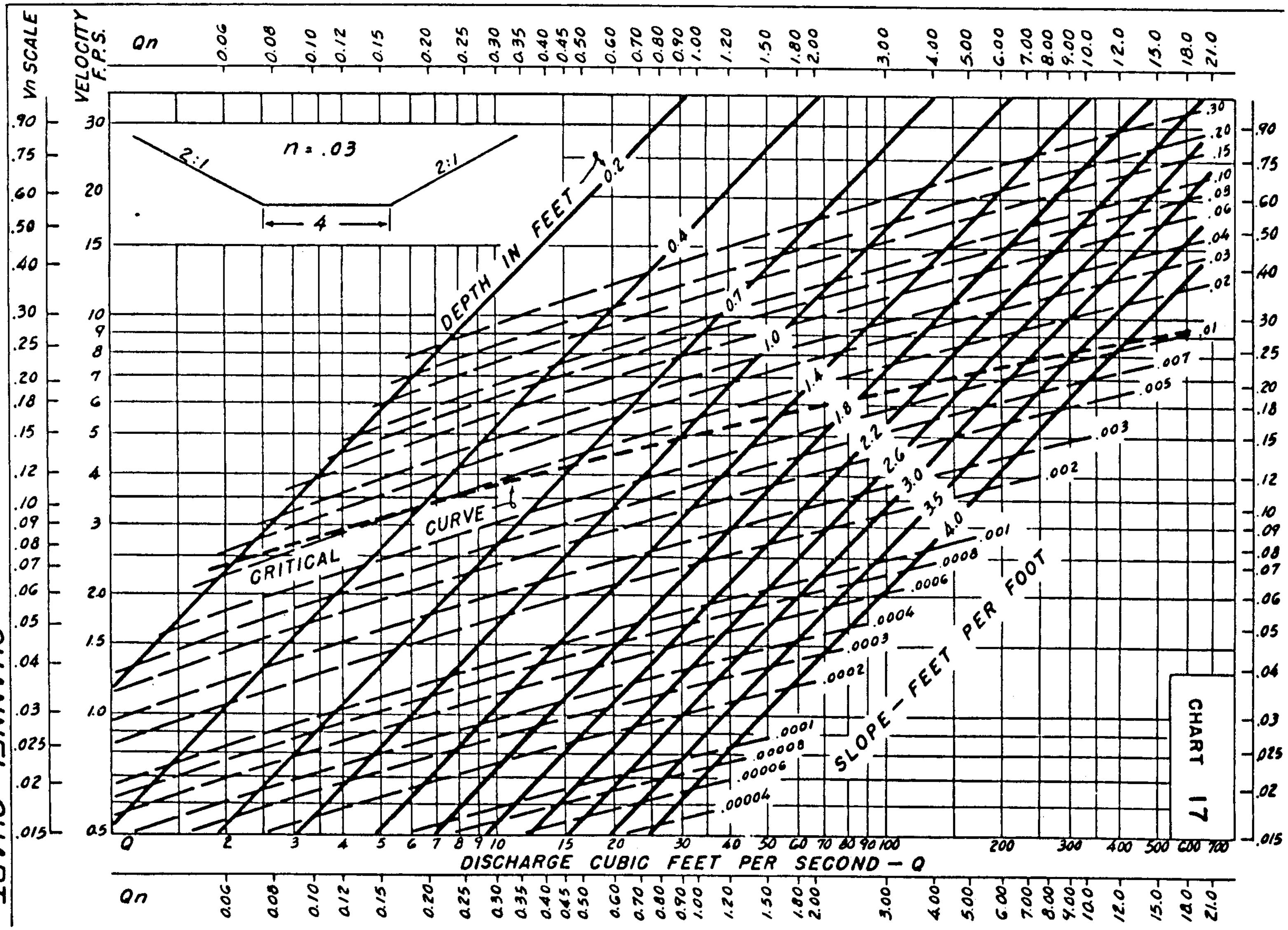
DESIGN CHARTS FOR SOLVING THE MANNING EQUATION AND VALUES FOR THE MANNING n

This page intentionally left blank.

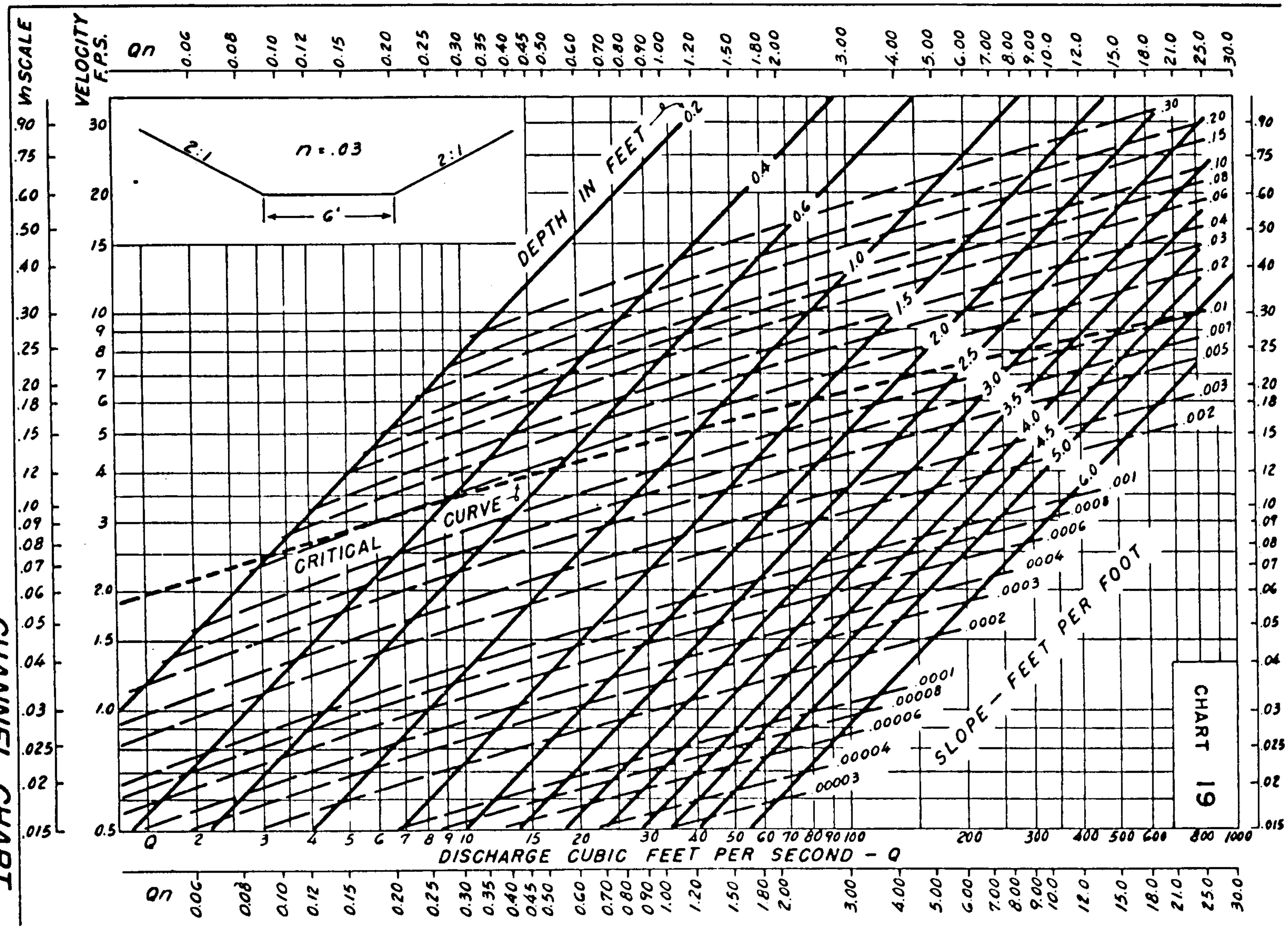
CHANNEL CHART
2:1 b = 2 FT.

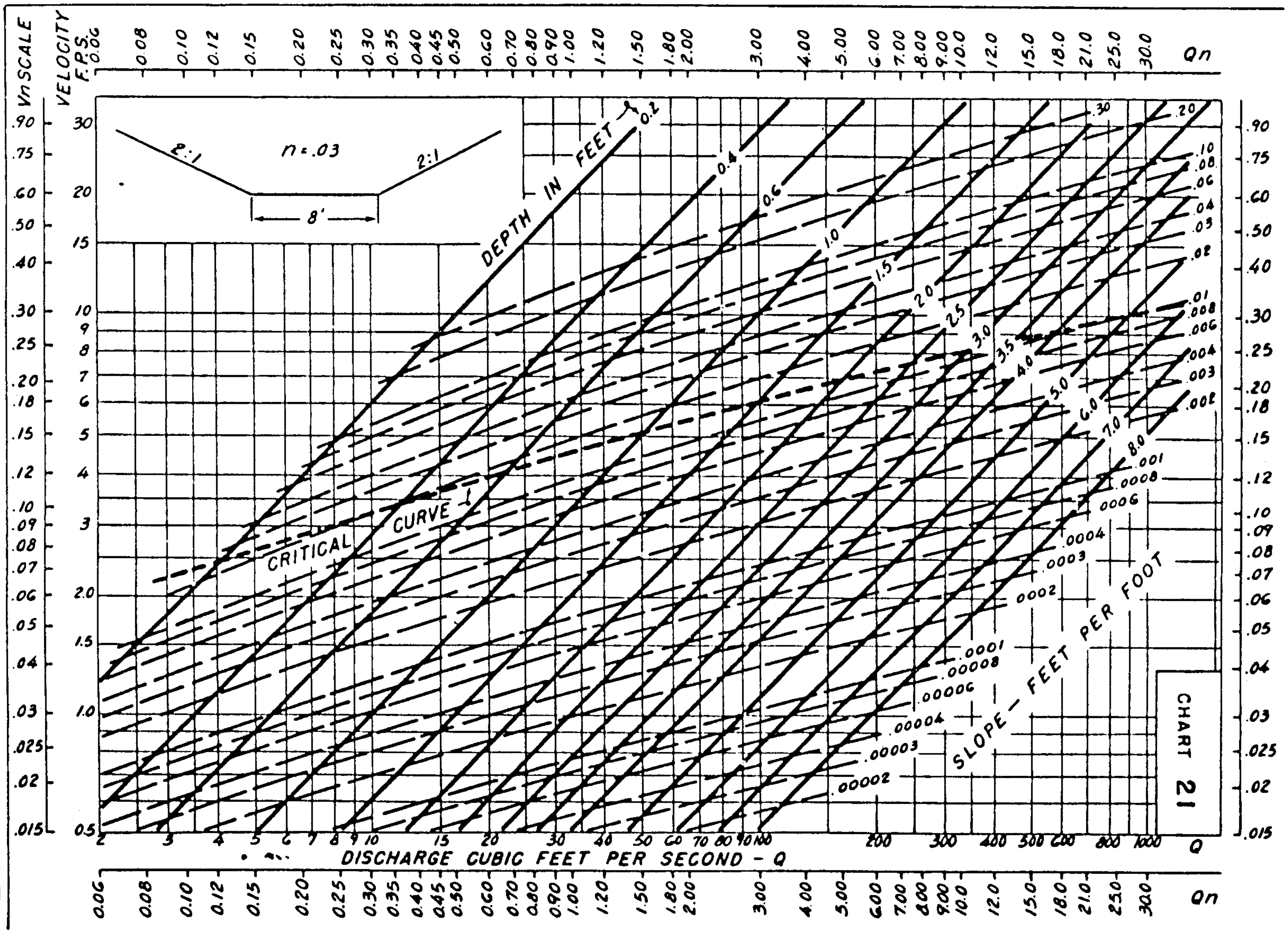


CHANNEL CHART
2:1 b = 4 FT.



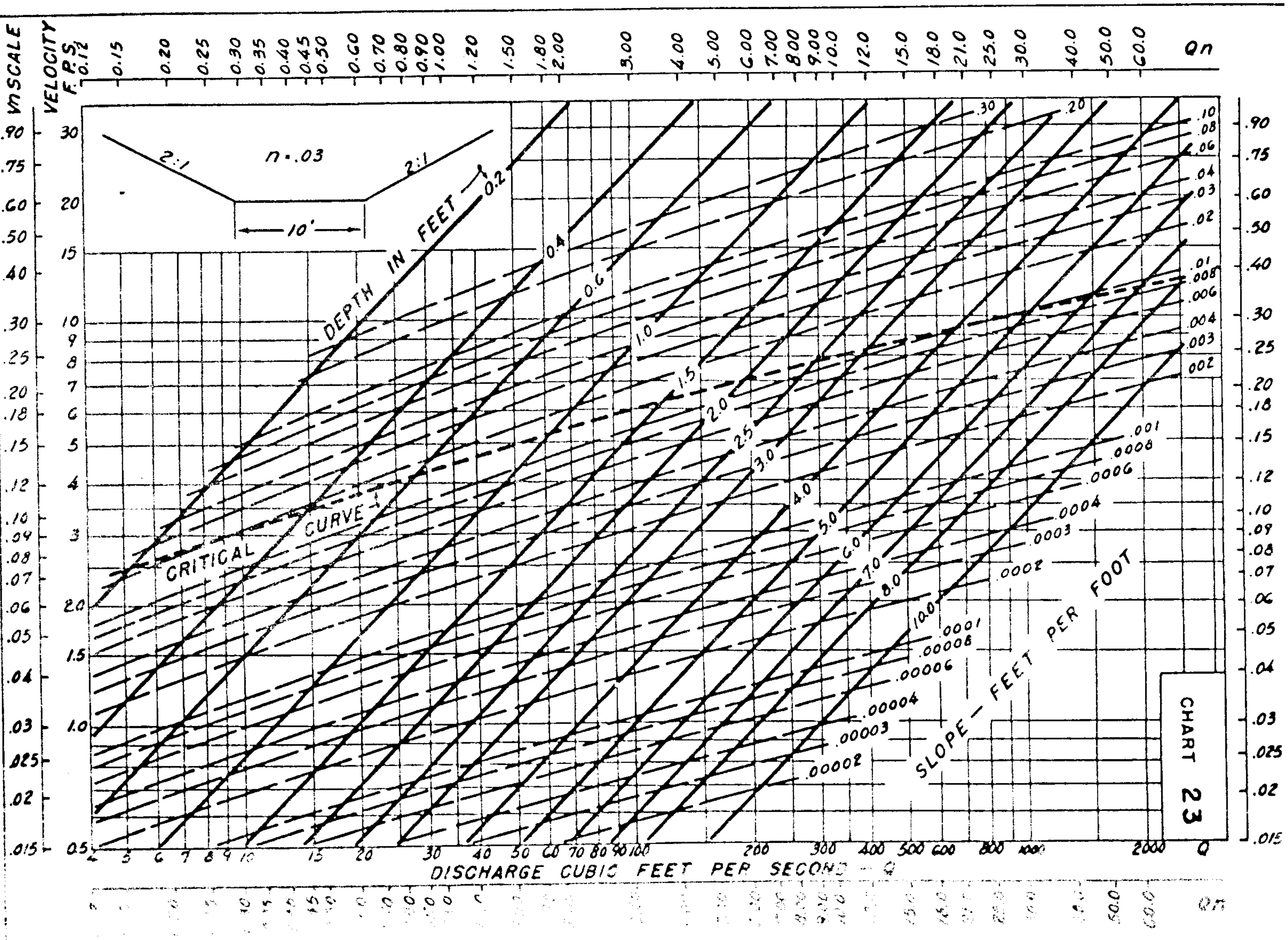
CHANNEL CHART 2:1 b = 6 FT.

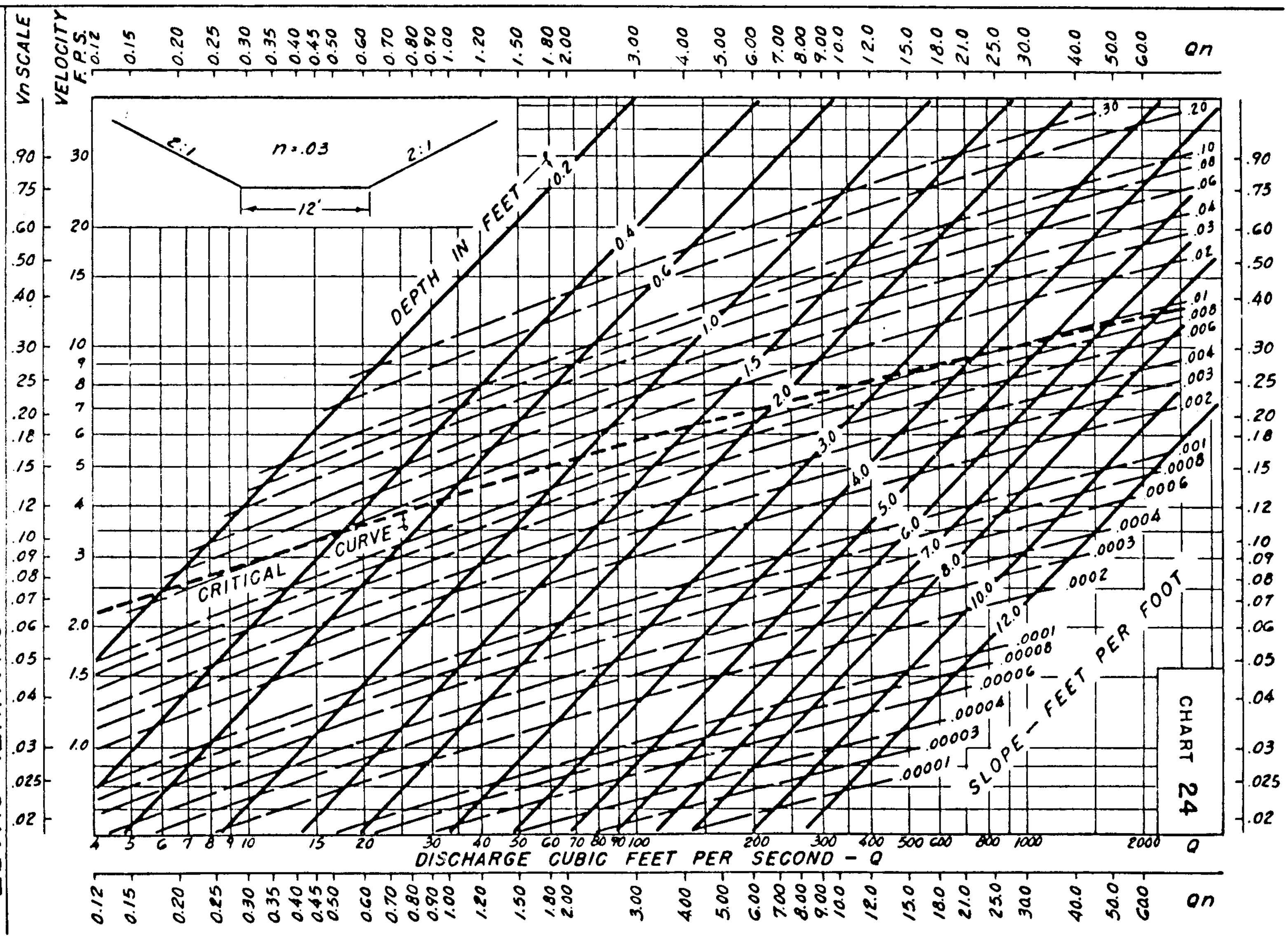




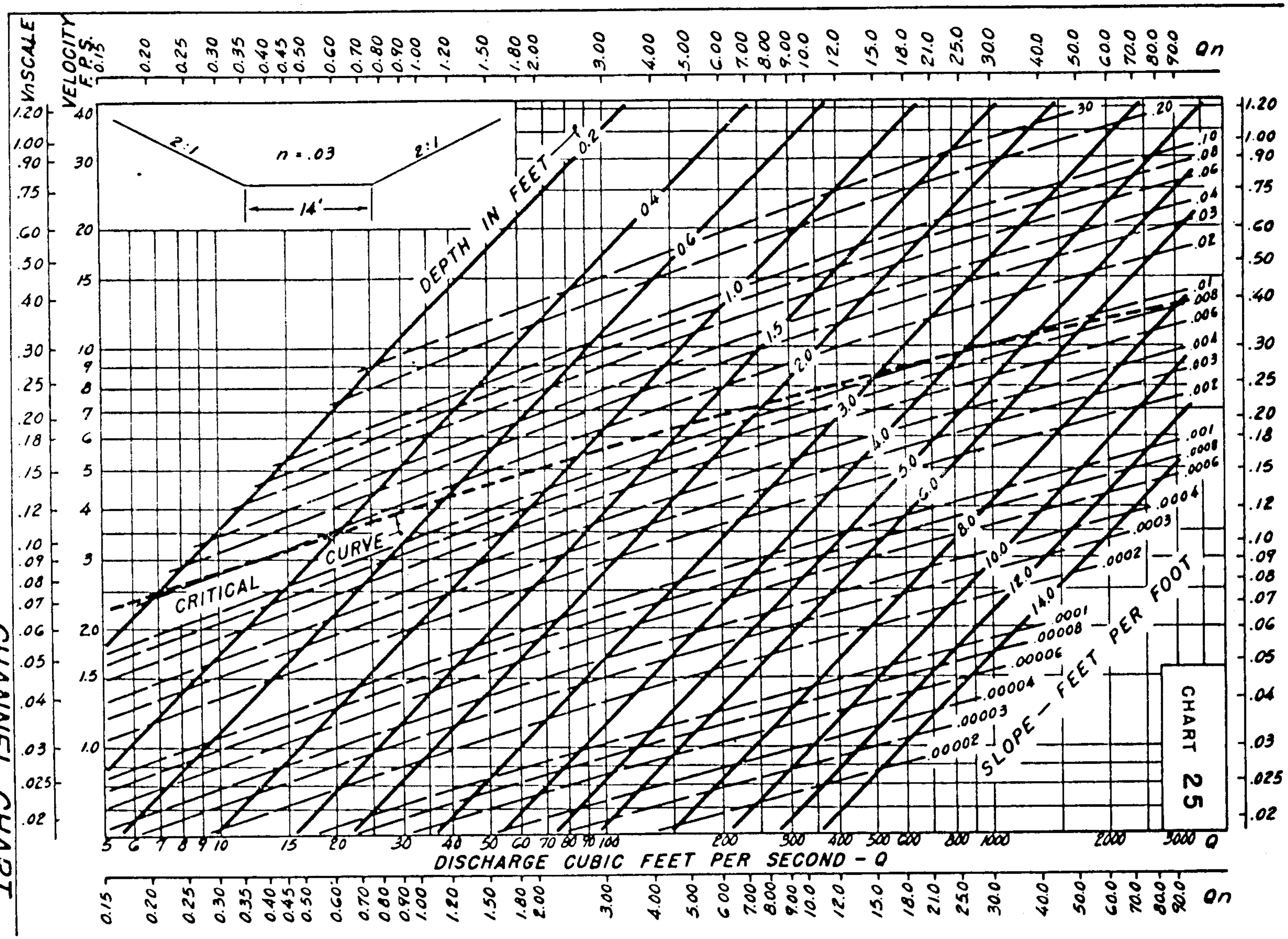
CHANNEL CHART
2:1 b = 8 FT.

CHANNEL CHART
2:1 b = 10 FT





CHANNEL CHART
2:1 b = 12 FT.



CHANNEL CHART
2:1 b = 14 FT.

Table C.1. Manning Roughness Coefficients, n.[1]

	Manning's n range[2]
I. Closed conduits:	
A. Concrete pipe	0.011–0.013
B. Corrugated-metal pipe or pipe-arch:	
1. 2⅔ by ½-in. corrugation (riveted pipe):[3]	
a. Plain or fully coated	0.024
b. Paved invert (range values are for 25 and 50 percent of circumference paved):	
(1) Flow full depth	0.021–0.018
(2) Flow 0.8 depth	0.021–0.016
(3) Flow 0.6 depth	0.019–0.013
2. 6 by 2-in. corrugation (field bolted)	0.03
C. Vitrified clay pipe	0.012–0.014
D. Cast-iron pipe, uncoated	0.013
E. Steel pipe	0.009–0.011
F. Brick	0.014–0.017
G. Monolithic concrete:	
1. Wood forms, rough	0.015–0.017
2. Wood forms, smooth	0.012–0.014
3. Steel forms	0.012–0.013
H. Cemented rubble masonry walls:	
1. Concrete floor and top	0.017–0.022
2. Natural floor	0.019–0.025
I. Laminated treated wood	0.015–0.017
J. Vitrified clay liner plates	0.015
II. Open channels, lined[4] (straight alinement):[5]	
A. Concrete, with surfaces as indicated:	
1. Formed, no finish	0.013–0.017
2. Trowel finish	0.012–0.014
3. Float finish	0.013–0.015
4. Float finish, some gravel on bottom	0.015–0.017
5. Gunite, good section	0.016–0.019
6. Gunite, wavy section	0.018–0.022
B. Concrete, bottom float finished, sides as indicated:	
1. Dressed stone in mortar	0.015–0.017
2. Random stone in mortar	0.017–0.020
3. Cement rubble masonry	0.020–0.025
4. Cement rubble masonry, plastered	0.016–0.020
5. Dry rubble (riprap)	0.020–0.030
C. Gravel bottom, sides as indicated:	
1. Formed concrete	0.017–0.020
2. Random stone in mortar	0.020–0.023
3. Dry rubble (riprap)	0.023–0.033
D. Brick	0.014–0.017
E. Asphalt:	
1. Smooth	0.013
2. Rough	0.016
F. Wood, planed, clean	0.011–0.013
G. Concrete-lined excavated rock:	
1. Good section	0.017–0.020
2. Irregular section	0.022–0.027
III. Open channels, excavated[4] (straight alinement,[5] natural lining):	
A. Earth, uniform section:	
1. Clean, recently completed	0.016–0.018
2. Clean, after weathering	0.018–0.020
3. With short grass, few weeds	0.022–0.027
4. In gravelly soil, uniform section, clean	0.022–0.025
B. Earth, fairly uniform section:	
1. No vegetation	0.022–0.025
2. Grass, some weeds	0.025–0.030
3. Dense weeds or aquatic plants in deep channels	0.030–0.035
4. Sides clean, gravel bottom	0.025–0.030
5. Sides clean, cobble bottom	0.030–0.040
C. Dragline excavated or dredged:	
1. No vegetation	0.028–0.033
2. Light brush on banks	0.035–0.050
D. Rock:	
1. Based on design section	0.035
2. Based on actual mean section:	
a. Smooth and uniform	0.035–0.040
b. Jagged and irregular	0.040–0.045
E. Channels not maintained, weeds and brush uncut:	
1. Dense weeds, high as flow depth	0.08–0.12
2. Clean bottom, brush on sides	0.05–0.08
3. Clean bottom, brush on sides, highest stage of flow	0.07–0.11
4. Dense brush, high stage	0.10–0.14
IV. Highway channels and swales with maintained vegetation[6,7] (values shown are for velocities of 2 and 6 f.p.s.):	
A. Depth of flow up to 0.7 foot:	
1. Bermudagrass, Kentucky bluegrass, buffalograss:	
a. Mowed to 2 inches	0.07–0.045
b. Length 4–6 inches	0.09–0.05
2. Good stand, any grass:	
a. Length about 12 inches	0.18–0.09
b. Length about 24 inches	0.30–0.15
3. Fair stand, any grass:	
a. Length about 12 inches	0.14–0.08
b. Length about 24 inches	0.25–0.13
B. Depth of flow 0.7–1.5 feet:	
1. Bermudagrass, Kentucky bluegrass, buffalograss:	
a. Mowed to 2 inches	0.05–0.035
b. Length 4 to 6 inches	0.06–0.04
2. Good stand, any grass:	
a. Length about 12 inches	0.12–0.07
b. Length about 24 inches	0.20–0.10
3. Fair stand, any grass:	
a. Length about 12 inches	0.10–0.06
b. Length about 24 inches	0.17–0.09
V. Street and expressway gutters:	
A. Concrete gutter, troweled finish	0.012
B. Asphalt pavement:	
1. Smooth texture	0.013
2. Rough texture	0.016
C. Concrete gutter with asphalt pavement:	
1. Smooth	0.013
2. Rough	0.015
D. Concrete pavement:	
1. Float finish	0.014
2. Broom finish	0.016
E. For gutters with small slope, where sediment may accumulate, increase above values of *n* by	*0.002*
VI. Natural stream channels:[8]	
A. Minor streams[9] (surface width at flood stage less than 100 ft.):	
1. Fairly regular section:	
a. Some grass and weeds, little or no brush	0.030–0.035
b. Dense growth of weeds, depth of flow materially greater than weed height	0.035–0.05
c. Some weeds, light brush on banks	0.035–0.05
d. Some weeds, heavy brush on banks	0.05–0.07
e. Some weeds, dense willows on banks	0.06–0.08
f. For trees within channel, with branches submerged at high stage, increase all above values by	*0.01–0.02*
2. Irregular sections, with pools, slight channel meander; increase values given in 1a–e about	*0.01–0.02*
3. Mountain streams, no vegetation in channel, banks usually steep, trees and brush along banks submerged at high stage:	
a. Bottom of gravel, cobbles, and few boulders	0.04–0.05
b. Bottom of cobbles, with large boulders	0.05–0.07
B. Flood plains (adjacent to natural streams):	
1. Pasture, no brush:	
a. Short grass	0.030–0.035
b. High grass	0.035–0.05
2. Cultivated areas:	
a. No crop	0.03–0.04
b. Mature row crops	0.035–0.045
c. Mature field crops	0.04–0.05
3. Heavy weeds, scattered brush	0.05–0.07
4. Light brush and trees:[10]	
a. Winter	0.05–0.06
b. Summer	0.06–0.08
5. Medium to dense brush:[10]	
a. Winter	0.07–0.11
b. Summer	0.10–0.16
6. Dense willows, summer, not bent over by current	0.15–0.20
7. Cleared land with tree stumps, 100–150 per acre:	
a. No sprouts	0.04–0.05
b. With heavy growth of sprouts	0.06–0.08
8. Heavy stand of timber, a few down trees, little undergrowth:	
a. Flood depth below branches	0.10–0.12
b. Flood depth reaches branches	0.12–0.16
C. Major streams (surface width at flood stage more than 100 ft.): Roughness coefficient is usually less than for minor streams of similar description on account of less effective resistance offered by irregular banks or vegetation on banks. Values of *n* may be somewhat reduced. Follow recommendation in publication cited[8] if possible. The value of *n* for larger streams of most regular section, with no boulders or brush, may be in the range of	0.028–0.033

Table C.1 (continued)

[1] Estimates are by Bureau of Public Roads unless otherwise noted.
[2] Ranges indicated for closed conduits and for open channels, lined or excavated, are for good to fair construction (unless otherwise stated). For poor quality construction, use larger values of n.
[3] *Friction Factors in Corrugated Metal Pipe*, by M. J. Webster and L. R. Metcalf, Corps of Engineers, Department of the Army; published in Journal of the Hydraulics Division, Proceedings of the American Society of Civil Engineers, vol. 85, No. HY9, Sept. 1959, Paper No. 2148, pp. 35–67.
[4] For important work and where accurate determination of water profiles is necessary, the designer is urged to consult the following references and to select n by comparison of the specific conditions with the channels tested:
Flow of Water in Irrigation and Similar Channels, by F. C. Scobey, Division of Irrigation, Soil Conservation Service, U.S. Department of Agriculture, Tech. Bull. No. 652, Feb. 1939; and
Flow of Water in Drainage Channels, by C. E. Ramser, Division of Agricultural Engineering, Bureau of Public Roads, U.S. Department of Agriculture, Tech. Bull. No. 129, Nov. 1929.
[5] With channel of an alinement other than straight, loss of head by resistance forces will be increased. A small increase in value of n may be made, to allow for the additional loss of energy.
[6] *Handbook of Channel Design for Soil and Water Conservation*, prepared by the Stillwater Outdoor Hydraulic Laboratory in cooperation with the Oklahoma Agricultural Experiment Station; published by the Soil Conservation Service, U.S. Department of Agriculture, Publ. No. SCS–TP–61, Mar. 1947, rev. June 1954.
[7] *Flow of Water in Channels Protected by Vegetative Linings*, by W. O. Ree and V. J. Palmer, Division of Drainage and Water Control, Research, Soil Conservation Service, U.S. Department of Agriculture, Tech. Bull. No. 967, Feb. 1949.
[8] For calculation of stage or discharge in natural stream channels, it is recommended that the designer consult the local District Office of the Surface Water Branch of the U.S. Geological Survey, to obtain data regarding values of n applicable to streams of any specific locality. Where this procedure is not followed, the table may be used as a guide. The values of n tabulated have been derived from data reported by C. E. Ramser (see footnote 4) and from other incomplete data.
[9] The tentative values of n cited are principally derived from measurements made on fairly short but straight reaches of natural streams. Where slopes calculated from flood elevations along a considerable length of channel, involving meanders and bends, are to be used in velocity calculations by the Manning formula, the value of n must be increased to provide for the additional loss of energy caused by bends. The increase may be in the range of perhaps 3 to 15 percent.
[10] The presence of foliage on trees and brush under flood stage will materially increase the value of n. Therefore, roughness coefficients for vegetation in leaf will be larger than for bare branches. For trees in channel or on banks, and for brush on banks where submergence of branches increases with depth of flow, n will increase with rising stage.

This page intentionally left blank.

APPENDIX D

DEVELOPMENT OF THE DESIGN EQUATIONS FOR STATIC EQUILIBRIUM SLOPE COMPUTATIONS

This page intentionally left blank.

The design and placement of grade control structures on the static equilibrium slope concept involves the Shield's parameter (Equation 6.3) and the Manning equation (Equation 4.13). Shield's parameter can be expressed in equation form as

$$\tau = \gamma RS = 0.047\ (\gamma_s - \gamma)\ d_{50} \tag{D.1}$$

where S is the bed slope, γ and γ_s are the specific weights of water and sediment, respectively, D_{50} is the median diameter of the bed material and R is the hydraulic radius.

This equation represents that combination of channel configuration and bed material size that will result in static equilibrium at the slope S.

For a given design flow Q, and known particle size distribution, it is required that Equation D.1 be solved for slope S. However, the value of hydraulic radius is also unknown in this equation. Rearranging the equation in terms of slope S yields

$$S = \frac{0.047_s(G_s - 1)\ D_{50}}{R} \tag{D.2}$$

where, G_s is the specific gravity of the bed and bank material, often assumed to equal 2.65. Substitution of Equation D.2 for S in the Manning equation produces

$$V = \frac{1.49}{n} R^{2/3} \left(\frac{0.047\ (G_s - 1)}{R} D_{50}\right)^{1/2} \tag{D.3}$$

Since the design discharge (Q) is known (from hydrologic analysis) it is more appropriate to express the preceeding equation in terms of Q

$$Q = \frac{1.49}{n} R^{2/3} \left(\frac{0.047(G_s - 1)\ D_{50}}{R}\right)^{1/2} A \tag{D.4}$$

Equation D.4 in terms of the cross-sectional area A and hydraulic radius R is

$$A\ R^{1/6} = \frac{Qn}{0.323\sqrt{(G_s - 1)\ D_{50}}} \tag{D.5}$$

Since hydraulic radius R equals area A divided by wetted perimeter P, the preceeding equation can be expressed as

$$\frac{A^7}{P} = \left(\frac{Qn}{0.323\sqrt{(G_s - 1)\ D_{50}}}\right)^6 \qquad (D.6)$$

For a trapezoidal channel the geometric equations defining area A and wetted perimeter P are

$$A = bd + zd^2 \qquad (D.7a)$$

$$P = b + zd\sqrt{z^2 + 1} \qquad (D.7b)$$

where b is the channel base width, d is the depth of flow and z is the channel side slope inclination (2:1). Thus equation D.6 becomes

$$\frac{(bd + d^2 z)^7}{b + zd\sqrt{z^2 + 1}} = \left(\frac{Qn}{0.323\sqrt{(G_s - 1)D_{50}}}\right)^6 \qquad (D.8)$$

The right side of this equation is a constant for a given combination of n , Q , and D_{50} or

$$\frac{(bd + d^2\ z)^7}{b + zd\sqrt{z^2 + 1}} = K \quad \text{Where} \quad K = \left[\frac{Qn}{0.323\sqrt{(G_s - 1)D_{50}}}\right]^6 \qquad (D.9)$$

Solution of this equation for d required a trial and error approach. After establishing d , the hydraulic radius R (A/P) can be determined and used to evaluate the armor slope. To aid in solution graphs 6.11a-e were developed for trapezoidal channels with base widths 6, 10 or 14 feet and 2 to 1 side slopes. Using these figures, the computation of a K value for the given conditions allows for solution of the hydraulic radius in a trapezoidal channel. If the computed K value is outside the range given in the figures, Equation D.9 must be solved directly.

APPENDIX E

BIOLOGICAL STREAM INVESTIGATION PROCEDURES

This page intentionally left blank.

SECTION IV

CHAPTER 22

STREAM STUDY PROCEDURES

by D. G. Huggins

Many types of water quality studies may be undertaken, with the objective(s) of the study determining the design. It is impossible to over-emphasize the necessity for a clear statement of objectives at the start of any study. Neglect of this essential preliminary step may result in failure to obtain certain critical information, or conversely, in expenditure of needless and wasteful amounts of time, effort, and money.

The objectives of any stream study should be put in writing (prior to initiating the study) for several reasons: (1) the mere act of putting them on paper requires careful considerations of the actual objectives; (2) the written word is far less likely to be misunderstood by those involved in the operations than is a verbal statement; (3) they fix the responsibility of those charged with supervision of the study; (4) they provide a basis for judging the extent to which the results of the study met the needs that originally justified the undertaking. Additionally, written objectives should define not only the purposes of the study but also the limits, thus discouraging the pursuit of nonessential sidepaths.

Categories of Stream Water Quality Studies

Most studies fall into one of two general categories. One type of study is designed to determine water quality and/or biological quality at a single point or at isolated points. This involves one or more unrelated sampling stations on a stream system. Sampling may be occasional, perhaps at weekly, monthly, or even quarterly intervals, but usually continues over a protracted period. Samples may be of a quantitative or qualitative nature and may include physical, chemical, and biological, data. The following are some objectives of single point investigations:

1. Establishment of a base line record of stream quality.
2. Investigation of suitability for propagation of aquatic life, including fish.

3. Monitoring effects of waste discharge or environmental alterations.
4. Surveillance to detect adherence to or violation of water quality standards.
5. Detection of sudden changes in water caused by slugs of contaminants resulting from spills, deliberate discharges, or treatment plant failures.

The second category of stream studies are those designed to determine changing water quality conditions as the water travels downstream. These types of studies often require a series of related sampling stations. These stations are selected to reflect both instantaneous changes in water quality at waste discharge points (or where major tributaries enter), and slower changes that result from natural purification. Samples are usually collected at frequent intervals, possibly even several times a day, for a limited period. Laboratory determinations are preformed on those water quality parameters that reflect changes in constituents resulting from natural purification and those that reveal effects of wastes discharged into the reach. The following are some objectives of related point investigations.

1. Determinations of patterns of pollution downstream of environmental alterations and waste discharges and effects of water uses.
2. Determination of adherence to or violation of water quality standards.
3. Determination of characteristics and rates of natural water purification in streams.
4. Projection of effects of pollution to conditions of flow and temperature other than those occurring during study.
5. Determination of causes of fish kills or other disasters involving deterioration in water quality.
6. Determination of existing water quality before some change in conditions, such as a new or increased waste discharge.

Site Identification

Identifying the site is an important step in conducting the stream investigation. There are two primary purposes for site identification: (1) clarification of the area of impact; and (2) study site delimitation. Additionally, proper site identification often facilitates the collection of considerable "on the shelf" information prior to actual stream sampling.

A basic tool of the site identification is a recent aerial photograph of the area. These can be obtained at USDA offices of the Soil Conservation Service, (SCS) and Agricultural Stabilization and Conservation Service, (ASCS). Both agencies usually have offices in the county seat. Before contacting these offices, mark the general area of the stream site on a county map. This will assist in locating the same area on the aerial photo.

ASCS photographs are normally the most recent aerial records available. Photo scales are usually 8 inches = 1 mile (or 1 inch = 660 feet). If time permits duplicate photographs can be ordered and purchased from the ASCS office in Salt Lake City, Utah for a small fee. Generally it will take about six weeks to obtain the maps from Salt Lake City. ASCS also has photostatic copies of the photographs available. They are not to scale but will usually provide adequate coverage of the site. One copy can be obtained free, while ASCS charges a modest charge for multiple copies.

Soil Conservation Service offices have several years of flight coverage for their region. Some offices have flights dating back to the 1930s. Map scales are 4 inches = 1 mile or 8 inches = 1 mile. This provides for a good historical picture of past stream impacts such as stream diversions and alterations. Another resource that should be consulted in the SCS office are soil surveys. Valuable interpretations as to type of stream, and locations of springs and wet areas can be made from the soil survey aerial photographs. Several different types of surveys may be available, so look at all of them.

In addition to the above, U.S. Geological Survey quadrangle maps are an excellent tool for conducting stream investigations. Physical features such as stream gradient, springs, and stream classifications can be readily obtained with fair accuracy. County engineers and SCS engineers normally have copies available but it is best to order your own sheet. State geological agencies should be able to provide ordering information or if necessary, further information may be obtained by contacting the United States Geological Survey office in Denver, Colorado.

Once the basic information concerning your site (photo and quad sheet) is collected, a reconnaissance of the area by vehicle and/or foot is advisable. Key access points and impact features such as soil disturbance, sediment discharge points, grazing, "pristine" reaches, etc. should be noted. If time permits, personal interviews with landowners, game wardens, and others concerning the stream can yield a wealth of information.

Survey Design

Survey design should be in response to the investigator's objectives. Basically the investigator's responsibilities will most likely include: (1) the identification of a stream with a biological community; (2) the investigation and/or identification of a possible violation; and (3) a check on the results of reclamation efforts.

Investigations involving the defining of a biologically active stream may necessitate the sampling of a stream section that may containing the three basic stream habitats (riffle, pool, runs), if present. Study sites should be representative of the stream areas most likely to be affected or impacted by mining activity. Stream condition must be considered since unusual stream conditions such as high water can make biological sampling impractical and/or lead to erroneous conclusions based on limited or incomplete samples. Extremely low water stages can also make the results of sampling efforts of limited value. In general, in this area the greatest measurable aquatic invertebrate family diversities will be found in late spring and summer. For a long-term biological survey program it is best that collections be made at least once during each of the annual seasons. Often the choice of location of survey site(s) or station(s) can be reasonably flexible. This may allow the investigator to consider other factors, such as travel time and acessibility, when reaching a decision concerning sampling site selection.

The series approach is used to establish the course of pollution, or to document water quality and biological changes throughout a reach of river or stream. This type of design is generally needed to establish the possibility of a violation or to evaluate the results of reclamation efforts. The pattern of changing quality reflected by the relationship among the several stations is more important than the isolated biological or physiochemical quality at any one station. The assessment of the relationship among the stations therefore depends on the collection of data representative of the stream at each station.

Establishment of sites that are physically similar is desirable. However, when dissimilar sites are to be compared, care should be taken so that data comparisons do not lead to false conclusions concerning the biological communities occurring in these study sites (see Chapter 24). The establishment of one or more control stations can allow for the comparison of water and biological quality above and below the point of pollution or source of alteration. A control station upstream of the source of impact is as important as the stations within or below the impact area and should be chosen with equal care to ensure representative results. The distance between the sampling sites should be sufficient to permit accurate measurement of potential changes. Sampling of a tributary stream may be meaningful, however, if the flow is less than 10 to 20 percent of the main stream, the tributary need not to be sampled. The station or site on a tributary should be as near to the main stream as feasible.

Bridges should be avoided when sampling for bottom organisms. Benthic populations may have been altered or destroyed by bridge construction activities. In addition, the physical environment of the stream near bridges is often altered and maybe unrepresentative of the stream in general. If sampling near a bridge is necessary, then it should be limited to the upstream side. Bridges frequently shade the stream beneath them and reduce light exposure and penetration.

The same considerations concerning sampling time and stream condition should be given to related series stations when investigating possible violations and/or recovery situations.

Stream Survey Field Sheet

The following survey sheet has been developed so that the investigator may accurately and concisely document the physical, chemical, and biological properties observed at each sampling site or locality. Individual survey sheets allow the investigator to record the precise stream conditions at each station or site at the time of survey. The field sheets are used to document, in a comparative manner, the changes that occur from site to site at any given sample date or at one site over an extended time period. The following detailed instructions and suggestions for completing these survey sheets are included so that some consistency might be obtained when data is recorded by the user. In some of the survey sections the methodology is given so that the user can calculate the appropriate answer(s). Often the appropriate aerial or topographic map of the sample site area may be photo-duplicated on the back of the survey sheet.

DETAILED STREAM SURVEY FIELD SHEET INSTRUCTIONS

MINE
Official mine name and/or mining company name.

OSM
Reference #

STATE
Do not abbreviate.

COUNTY
Do not abbreviate.

LOCATION
Legal description (to the 1/4 Section)

DATE
Write out the month. (day-month-year)

TIME
Military or indicate AM or PM.

WEATHER
General conditions (e.g., sunny and clear, cloudy and windy)

OFFICE OF SURFACE MINING
STREAM SURVEY FIELD SHEET

Mine:__________ OSM Reference No.__________

State:_____ County:_____ Location:__________

Date:_____ Time:_____ Weather:_____ Photo_____

Stream Name:__________ Station:__________

Length (ft.) Surveyed:_____ Stream Fall (ft.) per Mile:_____

Stream Flow c.f.s.:__________ Stage:__________

Stream Category - Ephemeral, Intermittent, Perennial (Circle)

RIFFLES (WxLxD)	POOLS (WxLxD)	RUNS (WxLxD)
Avg.	Avg.	Avg.
# Riffles	# Pools	#Runs

Rel. Riffle/Pool/Run Ratio (in tenths)__________

Substrate Composition (in tenths)

RIFFLES Silt/Clay____ Sand____ Gravel____ Pebble/Cobble____ Boulder/Bedrock____

POOLS Silt/Clay____ Sand____ Gravel____ Pebble/Cobble____ Boulder/Bedrock____

RUNS Silt/Clay____ Sand____ Gravel____ Pebble/Cobble____ Boulder/Bedrock____

Predominant bank type__________

Approx. % Canopy_____ Major Floodplain Land Use:_____

Dominant raparian vegetation__________

Water Temperature:____ Air Temperature:____ pH:____ Water Clarity:____

Other:__________

Observed aquatic fauna/flora:

Fish:__________

Benthos:__________

Molluscs:__________

Other:__________

Comments:__________

Survey Performed by:__________

PHOTO #

The section number is usually present on SCS maps but in addition the investigator may wish to give the map a unique number that corresponds to a station number, sample site, etc.

STREAM NAME

County maps or USGS maps are the best sources for official stream names. If not named, write unnamed tributary of ________stream, or, just, "unnamed creek".

LENGTH SURVEYED

Approximate. Length surveyed should be restricted to only that area on the survey sheet used.

STREAM FALL PER MILE

Stream gradient may be available on existing maps or from other studies. Measurement of stream length between contour lines of USGS topographical map will give a rough estimate of gradient.

gradient = contour interval/stream length

STREAM FLOW (C.F.S.)

The velocity of a canal or stream, and hence its discharge, may be roughly determined by the use of surface floats and channel cross sections.

A stretch of the channel, straight and uniform in cross section and grade, with a minimum of surface waves, should be chosen for this method. Surface velocity measurements should be made on a windless day, for even under the best conditions the floats are often diverted from a direct course between measuring stations.

If the width of the stream is more than 3 or 4 feet the canal should be divided into segments, and the average depth determined for each segment. The segments should be narrower in the outer thirds of the canal than in the central third. Float courses should be laid out in the middle of the strips defined by the segments. For regular-shaped channels flowing in a straight course under favorable conditions, the mean velocity of a strip in the channel is approximately 0.85 times its surface velocity. This value is an average of many observations. For any particular channel it may be as low as 0.80 or as high as 0.95. The velocity of the float in each strip, after being adjusted to mean velocity, multiplied by the cross sectional area of the strip, will give the discharge. The sum of the discharge of the strips is the total discharge.

On a small stream rather than dividing the stream into segments, a number of float runs can be made and an average of these used for the surface velocity of the stream. The float method is an approximate method and should be used only with its limitations in mind. Generally, make at least 3 runs with the float along a 100 foot section. Compute the average ft./sec. and multiply by cross sectional area of the stream to obtain the flow in cubic feet per second.

Sometimes stream flow or surface water discharge measurements may be available for the area of study from the Water Resources Division of the U. S. Geological Survey or state geological agencies. Always check to see if an existing gauging station(s) occurs in or near your study area of the stream. When more accurate measurements of stream flow are desirable, and the use of flow meters and more technical flow measurement techniques should be utilized.

STAGE

Obvious stream condition, such as flooding, dry, intermittent, normal.

STREAM CATEGORY

See definitions on Survey Sheet

RIFFLE, POOL AND RUN MEASUREMENTS

Maximum width, length, and depth should be obtained where possible. Depth measurements could be a problem and may have to be estimated. Ten spaces have been provided for writing riffle, pool, and run measurements so an average can be obtained. If many riffles or pools are present, randomly select areas from throughout the study section.

"Riffle areas" are characterized by swift turbulent water and uneven bottom substrates, usually gravel or rubble. "Pool areas" should be considered as those stream reaches where unusually deep water is located, with depth differing drastically and abruptly from adjacent areas. Stream "run areas" are those areas where normal shallow stream water exists, areas characterized by consistent, unbroken flow and fairly even or stable bottom.

SUBSTRATE COMPOSITION

Mineral particles on the beds of rivers and streams exhibit a wide range of sizes, from large boulders through fine silt and clay. While most biologists have been content to be merely descriptive about the nature of the substrate they have worked upon, recently there has been a trend toward more quantitative measurements of stream substrates. Accordingly, the two methods presented here will give "quantitative" estimates of substrate compositions. Method one is conveniently employed in the field without the need for accessory equipment. Method two requires equipment and is more time consuming, but yields more precise estimates than method one. Time and resources should dictate the choice of method to be employed.

Method 1 Five categories for substrate composition are present on the Stream Survey Field Sheet. The percent of the total substrate represented by each of the five classes is estimated and recorded to the nearest tenth to the right of the respective class. The particle size range for the classes is 0.0625 mm and smaller for silt/ clay, 0.0626 to 2.0 mm for sand, 2.1 mm to 16.0 mm for gravel, 16.1 mm to 256.0 mm for pebble/cobble, and greater than 256.0 mm for boulder/bedrock.

Method 2 This method, proposed by Cummins (1962), uses the size categories given in Table 1. Samples of stream substrate are collected and the sand and silt fractions are elutriated. The silt is allowed to settle out of the elutriate and the clay is then centrifuged from aliquats. The remainder of the sample is dried and poured through a series of sieves, each of decreasing aperture size. The sieves are shaken on a mechanical shaker. After shaking, the size classes are weighed and each category is expressed as a percent on the basis of dry weight. The size classes are derived from a modified Wentworth scale of particle sizes and are recorded on the basis of a linear phi-scale ("phi" = the negative logarithm to the base 2 of the smallest diameter in each partical size category).

Table 1

Terminology, categories and methods for particle size analysis. Modified from Cummins (1962) and Hynes (1970).

			Mesh, size, (mm) and methods of measurement		
Name of particle	Range of size of mm.	phi-scale	mm.	U.S. sieve no.	Tyler sieve no.
Boulder	256	-8	Direct measurement	-	-
Cobble	64-256	-6-7	Individual wire square	-	-
Pebble	32-64	-5	Individual wire square	-	-
	16-32	-4	16	-	-
Gravel	8-16	-3	8	-	-
	4-8	-2	4	5	5
	2-4	-1	2	10	9
Very coarse sand	1-2	0	1	18	16
Coarse sand	0.5-1	1	0.5	35	32
Medium sand	0.25-0.5	2	0.25	60	60
Fine sand	0.125-0.25	3	0.125	120	115
Very fine sand	0.0625-0.125	4	0.00625	230	250
Silt	0.0039-0.0625	5,6,7,8	Separated by settling	-	-
Clay	0.0039	9	Separated by centrifuge	-	-

PREDOMINANT BANK TYPE
Type of bank in terms of slope, cover, and composition.

APPROX % CANOPY
Percentage for study section. The user must consider the percent of canopy over the stream along the total stream reach within the study area.

MAJOR FLOODPLAIN LAND USE
Use appropriate terminology (e.g., cropland, pastureland, grazingland, forestry, residential, industrial/commercial).

DOMINANT RIPARIAN VEGETATIAN
If possible, identify dominant trees, shrubs and/or non-woody cover vegetation along stream banks and determine approximate width and relative density of riparian vegetation.

WATER AND AIR TEMPERATURE
Indicate whether centigrade or fahrenheit.

pH
Use most appropriate methodology.

WATER CLARITY
General statement concerning clarity of water (e.g., very clear, clear, turbid, very muddy, etc.).

OTHER
Pertinent physical or chemical data.

OBSERVED AQUATIC FAUNA/FLORA
Minimum information should consist of presence or absence data. Fish may be observed if shallow water or clear water areas are approached quietly and carefully. Benthos, molluscs and aquatic plant samples may be taken to lab for identification and recorded later.

COMMENTS
Pertinent observations concerning the study area and its immediate environments.

SELECTED REFERENCES

Cummins, K. W. 1962. An evaluation of some techniques for the collection and analysis of benthic samples with special emphasis on lotic waters. Amer. Midl. Nat. 67: 477-504.

Hynes, H. B. N. 1970. The Ecology of Running Waters. Univ. of Toronto Press, Toronto. 555 p.

MacKenthum. K. M. 1969. The practice of water pollution biology. Fed. Water Pollut. Control Admin. USDI, Washington, D.C. 281 p.

Mason, W. T., ed. 1978. Methods for the assessment and predction of mineral mining impacts on aquatic communties: a review and analysis. Workshop Proceedings. Publ. No. FW5/OBS078/30. Eastern

Energy and Land Use Team, Office Biol. Serv., Fish Wild. Serv., USDI. Kearneysville.

Weber, C. I., ed. 1973. Biological field and laboratory methods for measuring the quality of surface waters and effluents. Environ. Montoring Series, EPA-670/4-73-001. Environ. Monit. Sup. Lab., Office of Res. and Develop., USEPA, Cincinnati.

CHAPTER 23

BIOLOGICAL SAMPLING TECHNIQUES

by P. M. Liechti

Collection

The simplest method of sample collection is hand picking of the organisms from rocks and debris submerged in the water. A number of aquatic invertebrates are found on these substrates and can be easily picked off the material with forceps after removal from the water.

A D-net with fine mesh (Fig. 23.1) can be used for qualitative sampling of shallow riffles or edges of pools with overhanging vegetation. In riffles or other flowing water the flat surface of the net is placed against the substrate. Upstream from the net, rocks, and debris are disturbed or kicked by foot allowing the dislodged organisms to flow into the net. The material collected can either be hand picked directly into specimen containers, floated in a white pan and specimens picked out, or the entire contents can be placed in a container with a preservative and sorted later. The D-net can be swept through vegetation when sampling along the edges of pools with overhanging and/or emerging vegetation. The area can also be disturbed by foot and backwashed into the net or the vegetation can be pulled and washed into the net. Again sorting can either be done immediately or the sample can be preserved and sorted later.

The kick net (Fig. 23.2) is best suited for sampling shallow flowing water. It can be purchased complete or hand made by attaching a suitable piece of screen with 1 mm square mesh to two dowels or poles. Like the D-net, it is placed in the stream below the area to be sampled (usually a riffle). The area upstream is then disturbed by foot allowing dislodged invertebrates to become entrapped by the downstream net. The net is then simultaneously moved upstream and pulled from the water to prevent loss of the catch. The D-net and the kick net differ in that the former can be handled efficiently by one individual, whereas the kick net usually requires two individuals. In addition, the kick net has the advantage of sampling a larger area than the D-net but has a larger mesh size and therefore does not retain the smaller organisms.

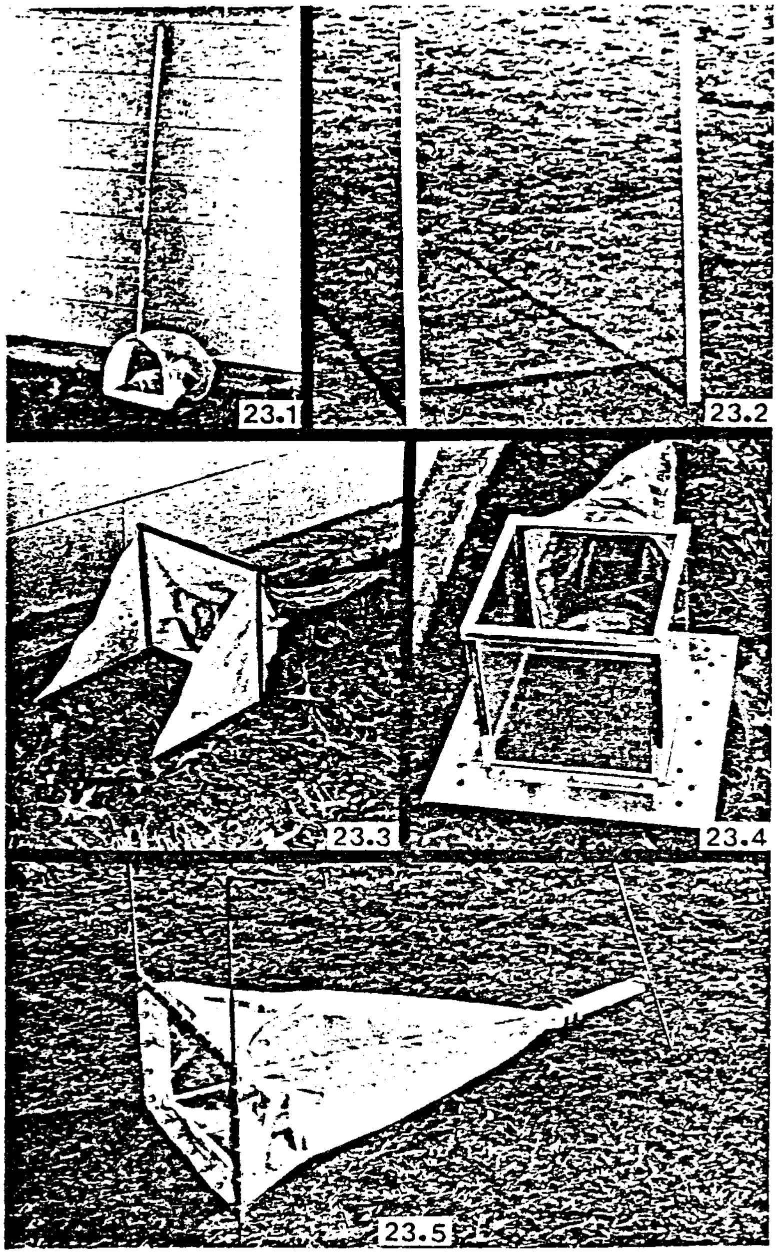
Three devices specifically designed for sampling in flowing water are the Surber sampler (Fig. 23.3), the Portable Invertebrate Box Sampler (PIBS) (Fig. 23.4), and drift net (Fig. 23.5). The first two are quantitative samplers for use in riffles or flowing water less than one foot deep. The Surber and the PIBS have bottom openings of one square foot and one-tenth of a square meter, respectively. The procedure is to rapidly place the sampler against the stream substrate.

Then, by hand or by using a small implement, mix, stir or otherwise disturb the area within the sample area to a depth of about 10 centimeters. This action results in the organisms being washed into a fine mesh net that trails behind the sampler. The entire contents of the net are then preserved for sorting at a later time. Usually, multiple samples are taken randomly through a riffle area to obtain statistically analyzable numbers and diversity of organisms. One difference between the two samplers is that the Surber has no front screen, thus allowing drifting organisms to enter the sample. The PIBS has a sponge along the bottom edge for a tight fit against the substrate. Generally, the Surber is more convenient to carry since it folds easily and is lighter weight. The third flowing water sampling devise is the drift net. Its basic function is to collect those organisms floating downstream. Use of drift nets varies depending on the intended purpose of the study and the type of stream or river being sampled. The net is placed with mouth facing upstream in shallow water and is held in place with metal rods forced into the stream substrate. The drift net is cone-shaped and of fairly small sized mesh with a small detachable bucket, also with fine mesh. The contents of the bucket are normally washed into a sample bottle and preserved for sorting at a later time. Drift nets can be used singly or in a series across the entire width of a stream and samples can be collected from them at any desired time interval. Drift samples are quantifiable if proper stream flow measurements and water volumes passing through the nets are calculated for the sample site.

Collecting samples from still water, pools, and/or ieep water sometimes requires different procedures and equipment. The aforementioned D-net works well for littoral zones in still water yet is less effective on sediments in still water. The most commonly used bottom sampler is one of a variety of grab samplers. They are available in various sizes and differ somewhat in their operating mechanisms but all function similarly. They all have a closing mouth that, when placed on the substrate, can be shut to trap a certain area of bottom material with organisms. Two commonly used grab samplers are illustrated (Fig. 23.6 and 23.7). They can be operated remotely for use from a boat or off bridges. If it is not necessary to retain the substrate from grab samples, the material can be washed through a sieving bucket (Fig. 23.8) or a series of standard sieves which retain any organisms.

Other types of samplers and sampling techniques can be employed such as the stovepipe method (Fig. 23.9). A length of pipe is pushed into the substrate and its contents either taken out and retained or sieved for specimens. The use of a hand dipper (Fig. 23.10) is employed to either take small floating specimens with a quantity of water, or, if attached to the end of a pole, to take a sample of substrate from a pool of water that is otherwise unaccessible.

Another sampling technique requires the use of artificial substrates. Two commonly used devices are the basket sampler (Fig. 23.11) and the multiple-plate sampler (Fig. 23.12). In essence, they function as an area of colonization for invertebrates. Both samplers have a calculatable surface area or volume so that the number and/or kinds of individuals obtained can be determined per unit of measurement, thus making the samplers useful in quantitative sampling. The samplers are



FIGURES: 23.1 D-net; 23.2 kicknet; 23.3 Surber sampler; 23.4 Portable invertebrate box sampler; 23.5 Driftnet.

usually suspended in the water, either in pools or flowing water, and are removed at various time intervals (3 weeks appears best in this region).

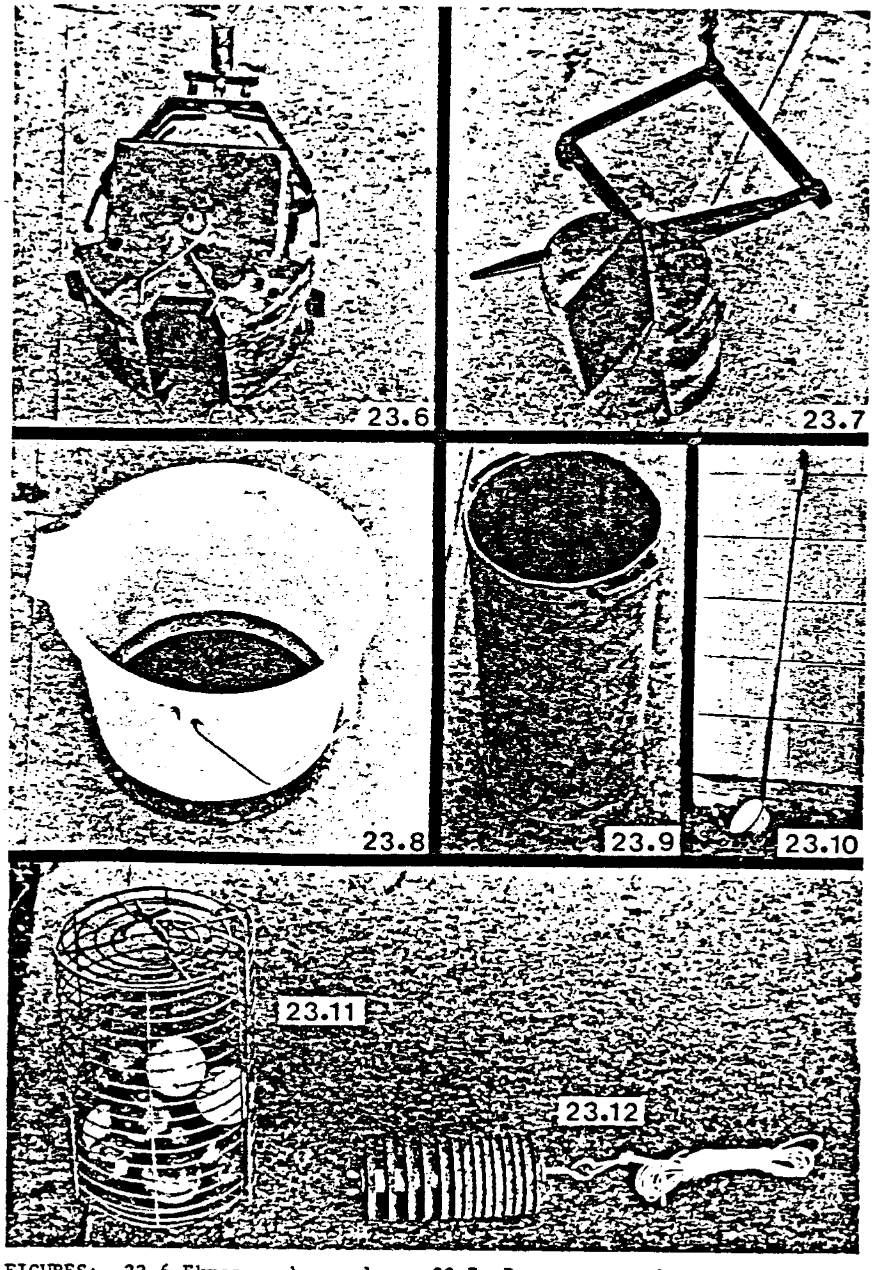
The type of biological sampling one adopts is always planned in accordance with the information desired. When an area or stream is first collected the easiest and quickest methodology is employed (e.g., hand picking rocks and debris and use of the D-net). Once a general knowledge of fauna is established a sampling regime is designed to methodically evaluate the fauna. This may include the use of many sampling techniques with repeated samplings over a designated period of time or may only require the use of one technique, such as Surber sampling, to survey organisms dwelling within a specific area of the stream. Whatever method or methods are decided upon, one should always sample in a consistent manner to derive results that are the best obtainable from the equipment utilized and are repeatable from sample site to sample site.

Sorting

Regardless of the collecting technique, samples invariably have to be sorted in some manner. Hand picking in the field is simple and easy if the specimens being sorted are large. If benthic or quantitative samples are taken, the job is more difficult. Usually field sorting is easiest when samples are placed in a white plastic or enamel pan. Other field techniques can be used to make laboratory sorting less tedious. Samples can be washed through a graded set of sieves to remove large pieces of debris and rocks and to arrange the organisms into approximate size classes. When dealing with large somewhat silty or muddy benthic or quantitative samples it is wise to sort a sample by dividing it into smaller more manageable subsamples. Laboratory sorting is usually carried out by placing the samples or subsamples in a white pan for hand picking. If a systematic approach is used in the subsampling of a quantitative sample, the need for picking an entire sample, may be eliminated. For example, one subsampling method is to place a grid in the bottom of a sorting pan and randomly pick from half of the quadrates, then double the number of specimens. Doing this for all subsamples would in effect cut the sorting time in half yet yield a relatively accurate estimation of the number and kinds of organisms in the entire sample. This method is not foolproof and is meant only as an example of what can be done to obtain the information for the least amount of effort. Regardless of the method utilized, it should be decided upon prior to sorting, and carried out in such a manner as to be consistent and reproducible throughout the study.

Preservation

With few exceptions all aquatic invertebrates should be preserved in a fluid medium. The two most commonly used preservatives are alcohol and formalin, with alcohol being the preferred choice. For field



FIGURES: 23.6 Ekman grab sampler; 23.7 Peterson grab sampler; 23.8 Bucket sieve; 23.9 Stovepipe sampler; 23.10 Dipper 23.11 Basket artificial substrate sampler; 23.12 Multiple-plate artificial substrate sampler.

preservation of small numbers of organisms, 80% ethyl alcohol is usually recommended but its availability is restricted due to federal government licensing. An adequate substitute is iso-propyl or rubbing alcohol which is readily available and relatively inexpensive. It should be used without dilution. Formalin is most frequently used to preserve benthic samples that contain quantities of organic matter. Samples that are initially preserved in formalin should be switched to alcohol within a day or two. Formalin tends to make specimens hard and brittle and if not buffered turns acid which dissolves the shells of molluscs. Formalin is normally diluted to 10% of the commercial formulation for use as a preservative.

Long term storage of specimens should be done with 70 to 80% alcohol as a preservative. It is advisable to add glycerin to the alcohol (1% of total volume) in the event of alcohol evaporation. The glycerin will not evaporate and prevents the specimens from drying out completely. Containers utilized for specimen storage should be sealable (to prevent evaporation), should not have metal caps or lids, and be of adequate size to prevent crowding of specimens. Containers with screw caps require periodic attention and refilling because the lids tend to "back off" due to heating and cooling. Optimum storage is carried out using either glass vials with neoprene rubber or another inert material stoppers or small cotton stoppered vials that are placed into larger glass jars, both of which are filled with alcohol.

Availability usually dictates the type of container used for short term storage of specimens. However, containers for initial preservation in the field should be the snap cap variety (glass or plastic), or small screw cap vials. For larger specimens or entire benthic samples, half-pint or pint jars with plastic screw caps, or glass cap jars with a rubber gasket and metal bail should be used. Other acceptable types of storage containers are baby food jars, commercial glass jars and canning glassware.

One of the most important aspects of collection, storage and preservation of specimens is proper labeling. All collections should be accompanied by a label carrying all necessary information regarding the place and date of collection and the collector's name. The safest method of labeling is to place the label directly into the container with the specimens. Labeling material must be of high quality, high rag content paper. Labels should be written with a permanent ink (e.g., India ink). Various other methods such as stick on labels, tape, or wax pencils and permanent ink markers can also be used to mark containers but should only be considered as short term identifiers since contents of containers can be switched or the labels smeared or washed off through time.

SELECTED REFERENCES

Lattin, J. D. 1956. Equipment and Technique, p. 50-67. In: R. L. Usinger, ed. Aquatic Insects of California. Univ. California Press, Berkeley.

CHAPTER 23 SAMPLING TECHNIQUES

Merritt, R. W., K. W. Cummins and V. H. Resh. 1978. Collecting, Sampling, and Rearing Methods for Aquatic Insects, p. 13-28. In: R. W. Merritt and K. W. Cummins, eds. An Introduction to the Aquatic Insects of North America. Kendall/Hunt Publ. Co., Dubuque.
Weber, C. I., ed. 1973. Biological field and laboratory methods for measuring the quality of surface waters and effluents. Environ. Monit. Series, EPA-670/4-73-001. Environ. Monit. Sup. Lab., Office of Res. and Develop., USEPA, Cincinnati.

CHAPTER 24

DETECTION AND DOCUMENTATION OF IMPACTS

by L. C. Ferrington

Introduction

As indicated in the preface, the basic intent of this manual is to assist the inspector in the recognition and determination of commonly occurring freshwater invertebrates inhabiting various aquatic ecosystems within the Kansas City District (OSM) and to provide additional information that will broaden the investigator's understanding of the impacts of surface mining on aquatic life. This section is concerned specifically with (1) the relationship between water quality deterioration resulting from surface mining operations (i.e., impacts) and the concommitant response by stream invertebrates, and (2) the methodology for the detection and documentation of impacts upon stream invertebrate communities.

It is assumed at this point that the reader is already familiar with the sections describing Stream Study Procedures (Chapter 22) and Biological Sampling Techniques (Chapter 23). Detection and documentation are the last in a series of steps performed when attempting to assess the impact of surface mining activities upon aquatic life. One cannot overemphasize the need for careful planning and execution of all preceeding steps. These steps are covered in detail in the above mentioned chapters and include a concise statement of objectives, accurate site identification, appropriate survey design, correct choice of sampling technique, and careful attention to field data collection. Poor planning and/or execution at any point may result in an inability to detect or sufficiently document the presence or magnitude of the impact upon an aquatic community.

Background Perspective

Extensive research has been performed with regard to the effect of surface mining upon aquatic organisms. Current areas of particular importance include; (1) the effects on water quality, terrestrial wildlife, and aquatic life,, (2) wetland resource management, (3) impact abatement techniques, and (4) mined land reclamation techniques. For an excellent comprehensive coverage of the broad spectrum of problems which have arisen as a result of surface mining in the West Virginia, Ohio, Pennsylvania, Virginia region the reader is referred to a recent publication of the U.S. Department of the Interior, "Surface Mining and Fish/Wildlife Needs in the Eastern United States, Proceedings of a Symposium" (Samuel, et al., 1978). This publication will serve as a starting point for the reader who is interested in acquiring a more technical knowledge of surface mining impacts.

With regard to effects upon aquatic communities, the major influence of surface mining is directly related to deterioration of water quality. Acid mine drainage is perhaps initially the most severe stress placed upon aquatic communities as a result of surface mining operations and is certainly one of the more common problems encountered during reclamation efforts. In addition to acid mine drainage, increased siltation (as a result of increased surface erosion), changes in stream thermal regime, and decreases in food availability also occur.

As indicated, acid mine drainage is one of the most common problems associated with surface mining operations and should be considered in more detail. Acid mine drainage is formed when pyritic material (FeS.) associated with coal bearing strata is exposed to oxygen and water. These reactants undergo a complex series of reactions which result in the formation of ferric hydroxide and sulfuric acid. The net effect upon a stream receiving the products of the reaction are a reduction in the pH and increases in free mineral acidity and non-carbonate hardness. Extreme increases in the concentrations of sulfate, iron, manganese, aluminum, magnesium, and calcium ions also occur. Any of these changes either individually or in concert may quickly affect the aquatic community of the receiving stream.

Concurrent with these initial changes in water chemistry, changes in physical conditions of stream habitat occur rapidly. The low pH of acid mine drainage causes precipitates to form on the stream bottom substrates. In particular, ferric hydroxide tends to precipitate and form a hard yellow to dark red crust on all rock surfaces exposed to the stream water. Some precipitates may be very conspicuous in many streams and are often referred to as "yellow boy" in certain regions. These precipitates kill the dense periphyton cover which is generally present on most stream substrates, thereby reducing food availability for many aquatic invertebrate species. Additionally, many invertebrate species exhibit a marked aversion to colonizing precipitate covered substrates.

Continued surface run-off from mine tailings also increases the deterioration of water quality in receiving streams. Heavy metals such as zinc, copper, aluminum, arsenic, and cadmium may be leached from tailings and can occur in lethal concentrations in stream water. Interaction (synergism) between various metals, specifically zinc with copper, zinc with cadmium, and copper with cadmium, will cause increases in toxicities. The lowered pH of streams affected by acid mine drainage also tends to increase the toxicity of iron, copper, and zinc in solution.

The specific effects that surface mining activities can (may) have upon aquatic communities depend upon various factors and it is not always possible to predict the type of response that aquatic invertebrate communties will exhibit. To fully understand the complexities involved, it must be kept in mind that biotic communities are dynamic entities and as such are highly varible in time and space. Groups of species which form communities tend to be those that are well adapted to the physical and chemical factors which occur in a specific habitat. In so far as habitats differ, for instance a river versus a spring fed stream, or a riffle in a stream versus a pool area in the same stream, the aggregate of species that comprize the "community" will also differ. The specific effects that surface mining activities actually do have upon any given community therefore depend upon the extent to which the physical/chemical make-up of the existing habitat are altered. When alterations are severe, severe stress is placed upon the species and extreme changes can result. When alteration of the physical/chemical make-up of the existing habitat are less severe, subtle or even imperceptable changes may result.

From the standpoint of a single habitat, the following factors are related to the degree of impact upon aquatic organisms; (1) magnitude of acid mine drainage input and associated leacheate, (2) distance of habitat from point of input, and (3) pre-impact physical/chemical conditions.

Factor number one is intuitively obvious. If the amount of input is great, the changes in the habitat will usually be more pronounced. Conversely, if very little acid drainage and leacheate enter the system only small changes in the habitat may occur.

Factor number two is also rather obvious. As one travels downstream from the initial point of input the effects of the impact upon the existing habitats are lessened, at least when point source input is considered. Increases in the amount of stream flow due to springs, confluences with other streams, and surface water run-off dilute the concentrations of acid, other ions and heavy metals. In addition to the dilution effect, many of these contaminants tend to undergo chemical reactions, forming salts and other compounds which precipitate out of the water column, thus decreasing their concentrations.

Factor number three, the pre-impact physical/chemical condition of the habitat, also influences to a great extent the degree to which the aquatic community will be affected. As already indicated, communities tend to be composed of species which are well adapted to their habitat. The pre-impact physical/chemical conditions therefore govern, to a large extent, the species which will be present. Thus, one would expect surface mining activities to have a lesser impact upon an aquatic community present in a stream which naturally has low pH, high turbidity and high concentrations of heavy metals than upon a stream which, in its unimpacted state, has high pH, low turbidity and low concentrations of heavy metals.

CHAPTER 24 **IMPACTS**

Community Responses To Surface Mining Impacts

Biological communities by definition are composed of populations of species occurring in a given area. By this definition then, a community has the following two basic characteristics; (1) a certain number of species, and (2) each species has a certain population size. These two characteristics have been termed "species richness" and "relative abundance", and tend to differ from community to community. With regard to relative abundance, empirically it has been found that in most unimpacted communities a small number of species will be very abundant (i.e., have large population sizes) while a large number of species will be represented by intermediate to small population sizes. Figure 24.1 is a graph of a typical unimpacted community and provides information concerning the species richness (i.e., this community has nine species) and the relationship of the relative abundance of those species.

When exposed to deteriorating water quality conditions the response of a community varies, depending upon factors previously discussed. Typical responses, however, include:

(A) decreases in absolute abundances of all species
(B) decreases in species richness
(C) shifts in relative abundances of species
(D) decreases in species richness combined with decreases in absolute abundance of all remaining tolerant species
(E) decreases in species richness combined with shifts in relative abundances
(F) Complete or partial replacement of pre-impact species by species not present before impact.
(G) death of all organisms and subsequent denuding of the impact area

Response F may also include shifts in relative abundances of the remaining pre-impact species. Responses A through F are illustrated in graphic form in figure 24.2.

In addition to the effects upon communities in the immediate area of the impact, changes in community composition and structure may be detected downstream of the impact area. Figures 24.3 and 24.4 illustrate the relationship of silt pollution and toxic pollution (i.e., toxic concentrations of leached materials) to distance from point of impact. It can be seen from these figures that the effects are not restricted to the immediate impact area, but are manifested in decreasing severity downstream of the impact area. Dilution, precipitation, and decreases in turbidity are three examples of natural changes which occur in streams and act as mitigating factors downstream of the impact point.

Figure 24.3 also approximates the relationship between community response and post-impact recovery. By changing the X axis to an elapsed time axis, it can be seen that species richness and total population size tend to approach pre-impact conditions given sufficient time. The time period for such changes to begin to occur depends upon the persistent effects of the initial impact and upon the type of post-impact reclamation efforts. If no impact abatement or restorative precedures are initiated, increases in the number of species and total population size may still gradually occur, however it is often the case that species other than the pre-impact species are responsible for the observed increases. In essence the community response is of the type illustrated in figure 24.2F.

Detection of Impacts

Of the seven community responses decribed, the first two and the last are rather obvious and easy to detect. The remaining four responses are more subtle changes that occur when communities are stressed by changes originating from surface mining operations. These four responses generally require extensive field sampling effort and personel with expertise in identifying aquatic invertebrates to the genus or even species level before impacts can be detected. Because of this, most of the subsequent discussion will be directed toward detecting the A, B and G types of community responses. It should be kept in mind, however, that the other four responses also may indicate serious habitat degradation as a result of surface mining operations, and, if the situation so dictates, attempts should be made to detect these types of changes. In such instances various state agencies and/or universities could be petitioned for assistance. In Kansas the State Biological Survey is one qualified agency.

There are basically three approaches to detecting impacts upon aquatic invertebrate communities. All approaches require a comparison of the community structure of the suspected impacted area with an unimpacted community standard. The three approaches are:

(1) comparison of pre-impact species composition and relative abundance with post-impact (or current) species composition and relative abundance for the exact same habitat area.
(2) comparison of species composition and relative abundance of areas upstream from the impact zone with the species composition and relative abundance of areas in or just downstream of the impact zone.
(3) Comparison of species composition and relative abundance of an impacted stream with an adjacent stream of similar physiognomy but not impacted.

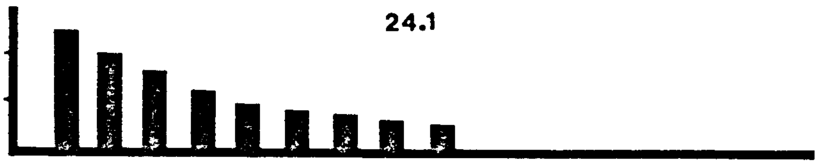


FIGURE 24.1. Species richness and relative abundance relationships in a "typical" unimpacted community setting.

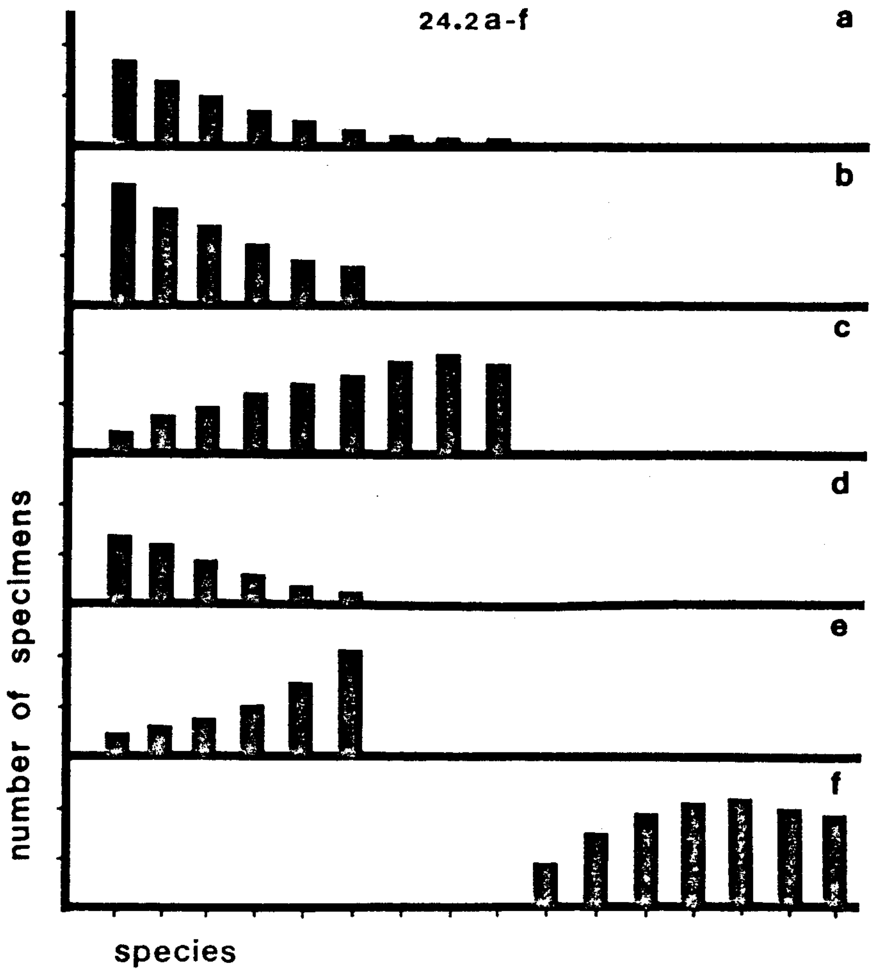
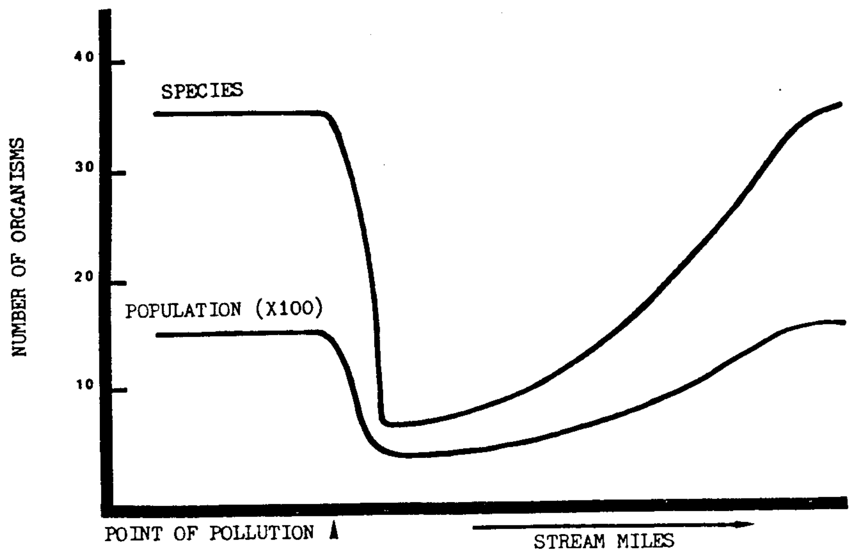
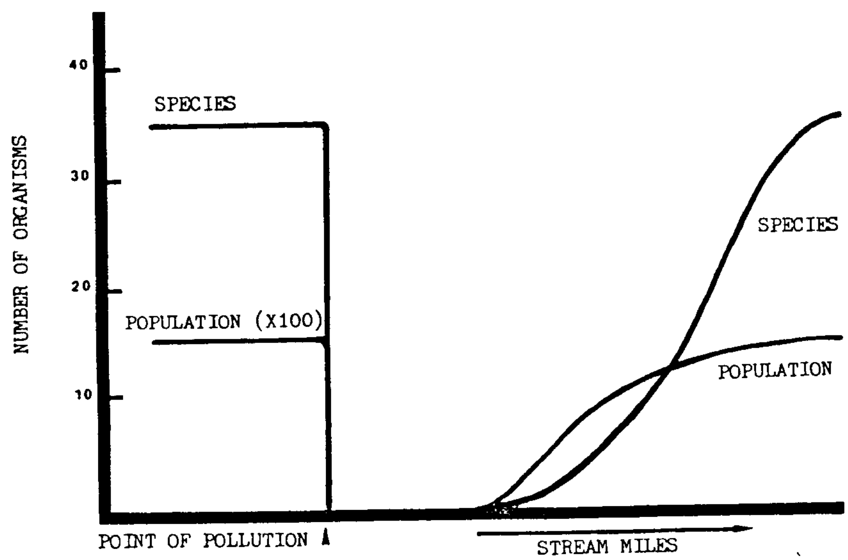


FIGURE 24.2a-f. Various community response modes to impacts: (a) decreases in absolute abundance; (b) decreases in species richness; (c) shifts in relative abundances; (d) decreases in species richness and absolute abundances; (e) decreases in species richness with shifts in relative abundances; (f) replacement of pre-impact species by impact tolerant species.



24.3



24.4

FIGURE 24.3. Community responses to silt (inert) pollution in relation to distance from point of impact in lotic environments.

FIGURE 24.4. Community responses to toxic pollution in relation to distance from point of impact in lotic environments.

Of these three, approach number 1 is preferable. By comparing the pre- and post- conditions at the exact same site, changes observed in community structure are usually directly attributable to the impact. When this method of detection is employed, post-impact samples should be collected as near to the pre-impact sample date as possible, and identical field collecting methods should be employed. A disadvantage of this method is that it requires extensive pre-impact base line information before it can be employed. In many cases, especially when new surface mining operations are anticipated, the pre-impact information may be available in the form of an environmental impact statement. In other situations, usually when impacts are the result of accidental or unanticipated events, base line data will be sparse or entirely lacking and this detection method will be unacceptable.

Approach number 2 is well suited for impacts resulting from accidents or unanticipated events. The basic assumption of this method is that areas of similar habitat in close proximity to each other on the same stream will have aquatic invertebrate communities with equivalent structures. By comparing the upstream community structure, where no impact effects have been manifested, with the downstream (or impacted) community structure, one can detect differences which are attributable to the impact. At both sample sites the collecting methods employed should be identical, and if possible, collections should be made on the same day or within a short period of each other. Sample sites should also be selected such that they are similar in appearance and are not separated from each other by great distances along the stream. However, care should be taken to ensure that one site is entirely out of the impact zone and the other site clearly within the zone.

The third approach to detecting impacts is the least desirable because it requires the comparison of community structures of two different streams, but may in some instances be the only approach which is feasible. When impacts occur at or near the source of streams, suitable upstream habitat may not be available to act as a comparison standard. If no pre-impact data are available the only recourse is to compare the community structure of the impacted area with the community structure of an unimpacted stream. As in the other approaches, care should be taken to ensure that similar habitats are chosen for comparison, similar field methods are employed, and samples are taken within a short time interval of each other, preferrably on the same day. In addition, a stream of equivalent discharge and draining a similar geophysical watershed should be chosen for comparison.

In all three approaches quantitative data should be gathered. Refer to the chapter on Biological Sampling Techniques (Chapter 23) for a detailed discussion of quantitative techniques. Some of the more commonly used methods would include Surber or PIBS samplers, drift nets, stove-pipe samplers, or various grab samplers or coring devices.

If time or lack of resources dictate that only qualitative samples can be gathered, one should employ an equal-effort mode to reduce investigator associated error. Equal effort mode simply means that equal effort is expended, in terms of man hours, in both the unimpacted and impacted areas. The assumption here is that if equal effort is expended in the two areas under consideration, any differences in the observed community structures of the two areas are related to the effect of the impact. It should be emphasized, however, that this approach, while not being a desirable substitute, may be used when strictly quantitative methods are not feasible.

After field sampling and sample sorting are completed, all organisms should be classified to family using the taxonomic keys provided in the initial sections of this manual. The next step in detecting impact related community responses is to determine, based upon the organisms in the samples, if there is a difference in community structure between the unimpacted and impacted areas. If the magnitude of an impact is so great that it has resulted in the death of all organisms in the impact zone and subsequent denuding of sections of the impact area, very little additional effort is needed to detect the impact. If however, organisms are found in the impacted area the problem of detecting community responses becomes more time consuming. In very general terms the following protocol can be used to check for evidence of typical community response patterns.

When quantitative or qualitative collections have been taken the following questions should be asked: (1) Do decreases in species richness (i.e., fewer types of organisms) occur in the impact zone? (2) Are some species which are common in the samples from the unimpacted zone rare or even entirely lacking from the samples gathered from the impacted area? (3) Are some species which are rare in the samples from the unimpacted area extremely abundant in the samples from the impacted area? (4) Are there species present in the samples from the impacted area which are not present in samples from the unimpacted area?

When strictly quantitative samples have been taken, the following question should also be asked. (5) Are large decreases in the absolute abundances of all or many species detectable in the impact area?

If the answer to any one of these questions is yes then you may suspect that there has been some alteration of the aquatic invertebrate community in the impact area. If two or more of these questions receive an affirmative answer then there is strong evidence that the community structure has been altered.

In addition to the above protocol, one should always read the section on biology after identifying an organism to family. Occasionally upon reading about the biology one will find that specific groups are intolerant to certain types of pollution, or conversely, are very tolerant to severe pollution. If samples from a suspected impact area lack intolerant groups but contain many groups regarded as highly tolerant to acid mine drainage, further evidence of the presence of community structure alteration is present. This type of evidence is rather weak by itself and should only be considered from the perspective of additional supportive evidence.

Most often quantitative data is required to detect the more subtle responses of communities that are of the type C through type F. Very sensitive data analysis techniques which are beyond the scope of this manual can be used for detecting statistically significant trends. The following is a list of various data summary techniques that are currently being employed by ecologists to elucidate the underlying factors determining community response modes.

Species Diversity Indices
Analysis of Variance
Cluster Analysis
Factor Analysis
Biotic Index Calculations

Detailed discussions of these techniques are present in the literature (see selected references).

Documentation of Impacts

As in the case of detecting impacts, the ease of documenting an impact is related to the degree of community response. If the impact is severe, documentation is straight forward and usually requires little if any outside assistance. The documentation procedure which follows is suitable for community responses of the types A, B and G. Occasionally, however, when an impact is less severe, statistical analysis of sample data may be required, combined with genus or species level identifications and concise records of individual species tolerance in order to adequately document the magnitude of the impact.

The first step in documenting impacts is to compile separate data sheets listing the organisms found according to sample. Categories of organisms should be listed along the left margin of the data sheet with the corresponding number of organisms for each category indicated to the right. Groups of categories should occur in similar orders on all data sheets, for instance if the first categories for the unimpacted area list are the families of Odonata followed by families of Ephemeroptera, the first categories for the impacted area list should be families of Odonata followed by Ephemeroptera, and so forth down the data sheet.

The next step is to condense the category names of the individual sample lists into one comprehensive list. All categories of organisms found in both sample areas will now be represented on one sheet (or set of sheets). At the top of this list the words "Sample Site" followed by a blank space should be typed. This comprehensive list is then duplicated and labelled with the appropriate sample designation. Transfer the data from the initial data sheets to the corresponding categories on the comprehensive list. A blank should be left or zero should be placed in any category for which specimens have not been collected in the indicated sample. By following this procedure one ends up with a set of data sheets for each sample set such that corresponding categories can be matched and quick comparisons can be made with regard to presence/absence criteria or relative abundance.

The second step is to reread the description of the biology of the groups of organisms present at each site. Once all the background biology is considered, notes should be made indicating any general trends in the data. For instance, are there several pollution tolerant species present in one set of samples, but absent from the other set? Are there large discrepancies in relative abundances? Are some groups of organisms conspicuously absent? Answers to these questions should be carefully formulated and additional notes made regarding any other unusual data trends or personal observations. These notes, in conjunction with detailed field notes will form the justification for your final decision regarding the effects of surface mining activities upon the aquatic community under consideration.

At this point it is also recommended that the field data regarding the physical/chemical characteristics of the two sample sites be reevaluated. If there are large differences in specific data values for the two sites, they should be noted. If no differences exist for several values, this too should be briefly stated. Field characteristics that were not measured, but which upon field observation appeared to differ to a large degree between sample sites should also be noted and, if necessary, background information given.

Before concluding it should be pointed out that documentation is not restricted only to cases where an actual impact has been detected. If it appears, after field investigation and subsequent detection efforts have been completed, that no change in community structure has occurred in the presumed "impact area", the aforementioned documentation procedures should still be followed. In essence, documentation is simply the procedure whereby one assembles and organizes in a logical order all the evidence which has been gathered in the field and upon which a decision has been based regarding the presence or absence of an impact. It is only through careful documentation that one can confidently present to another party the evidence upon which a final decision has been based.

In many instances, even with careful documentation, the investigator may not feel confident in rendering a decision. As indicated, the documentation procedure which has been given is only suitable for detecting gross community responses of the types A, B and G. Many times community responses will be of the more subtle type (i.e., type C through type F) and the inspector may "feel" that a change has occurred but cannot quite find enough evidence in the data which has been gathered to warrant an "impacted" designation. When this type of situation arises outside assistance should be solicited.

SELECTED REFERENCES

Canton, S. P and J. V. Ward. 1978. Environmental effects of western coal surface mining: Part II. The aquatic macroinvertebrates of Trout Creek, Colorado. Publ. No. EPA-600/3-78-095. USEPA. Washington, D.C.

Green, R. H. 1979. Sampling Design and Statistical Methods for Environmental Biologists. John Wiley and Sons Inc., New York. 257 p.

Mason, W. T., Jr., ed. 1978. Methods for the assessment and prediction of mineral mining impacts on aquatic communities: a review and analysis. Workshop Proceedings. Publ. No. FWS/OBS078/30. Eastern Energy and Land Use Team, Office Biol. Serv., Fish Wild. Serv., USDI. Kearneysville. 153 p.

Parsons, J. D. 1968. The effects of acid strip mine effluents on the ecology of a stream. Arch. Hydrobiol. 65(1): 25-50.

Poole, R. W. 1974. An Introduction to Quantitative Ecology. McGraw-Hill, Inc., New York. 532 p.

Samuel, D. E., J. R. Stauffer, and C. H. Hocutt., eds. 1978. Surface mining and fish/wildlife needs in the Eastern United States: proceedings of a symposium. Pub. No. FWS/OBS-78/81 Eastern Energy and Land Use Team, Office Biol. Serv., Fish Wild. Serv., USDI. Kearnysville.

Warner, R. W. 1971. Distribution of biota in a stream polluted by acid mine-drainage. Ohio J. Sci. 71(4): 202-215.

Weber, C. I., ed. 1973. BI.,logI.,al fI.,ld and laboratory methods for measuring the quality of surface waters and effluents. Environ. Monit. Series, EPA-670/4-73-001. Environ. Monit. Sup. Lab., Office Res. Develop., USEPA. Cincinnati.

☆U.S. GOVERNMENT PRINTING OFFICE: 1999 - 573-761