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

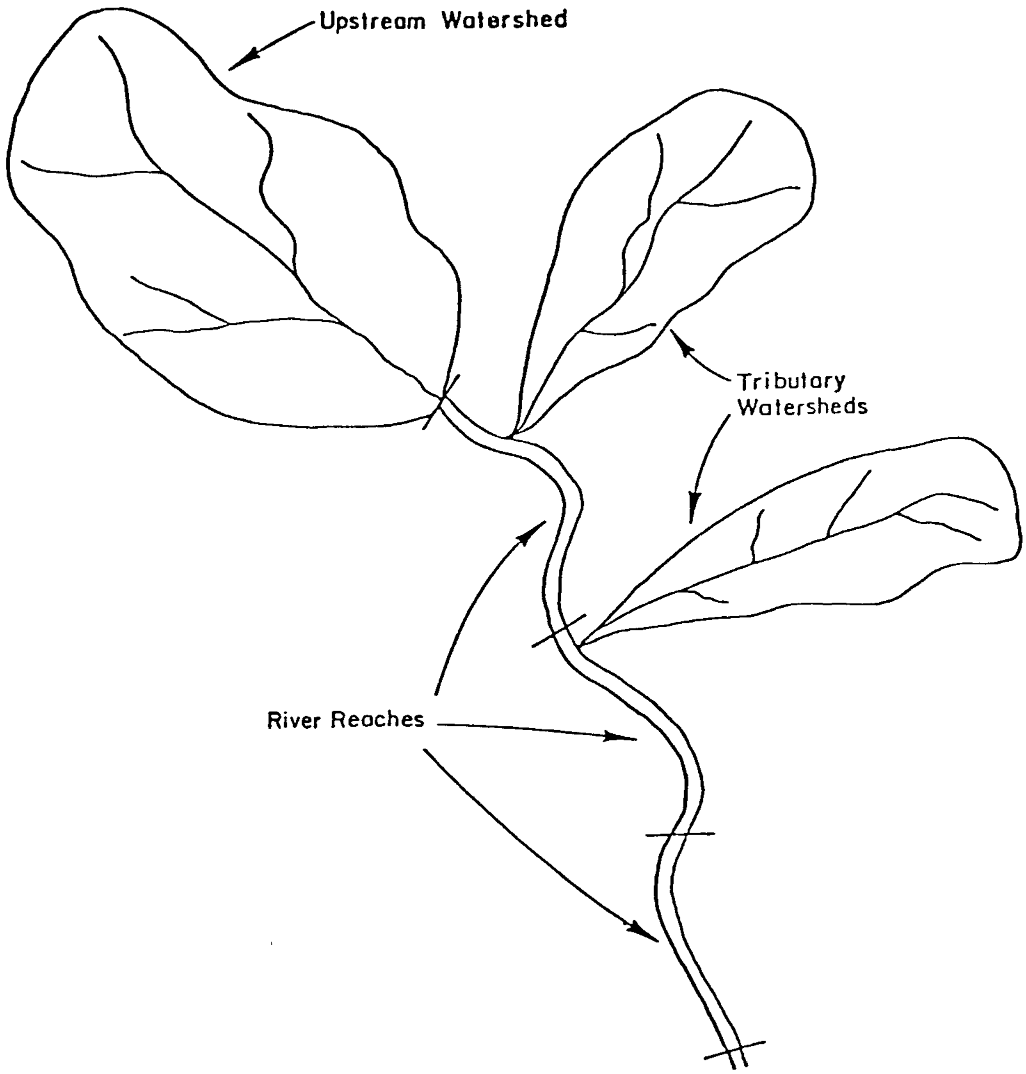

Figure 6.1. Watershed-river system.

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.

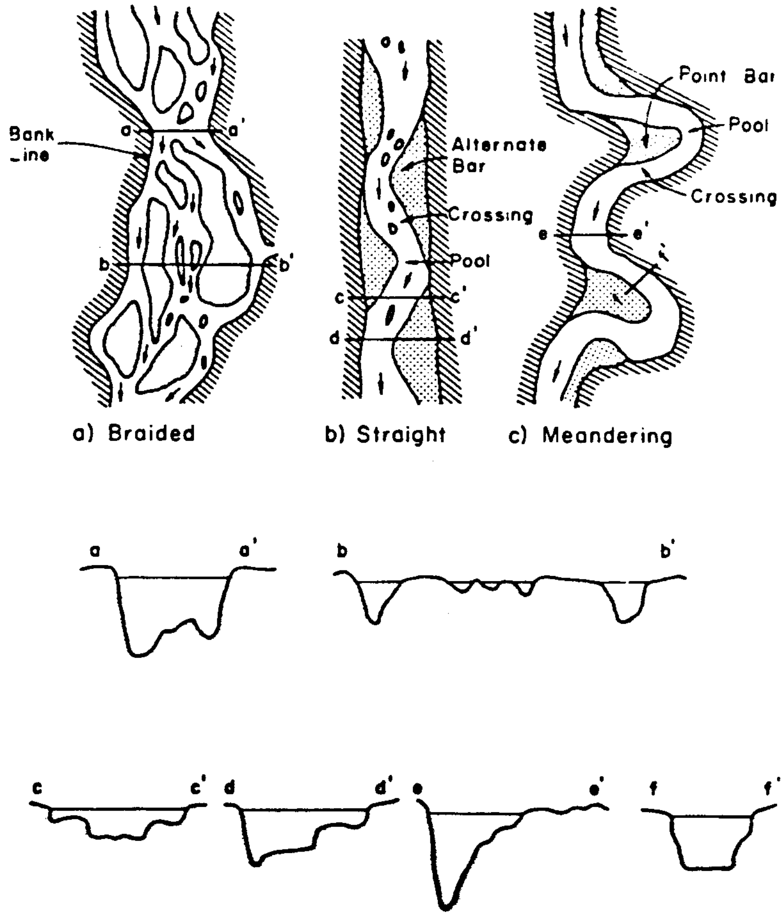

Figure 6.2. River channel patterns.

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 nonmoving 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

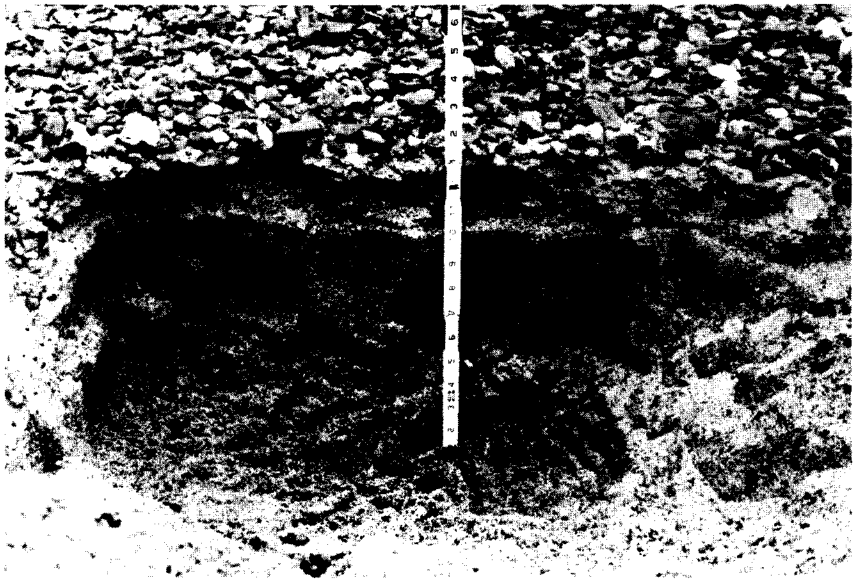

Plate 6.3. Typical armoring situation.

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.

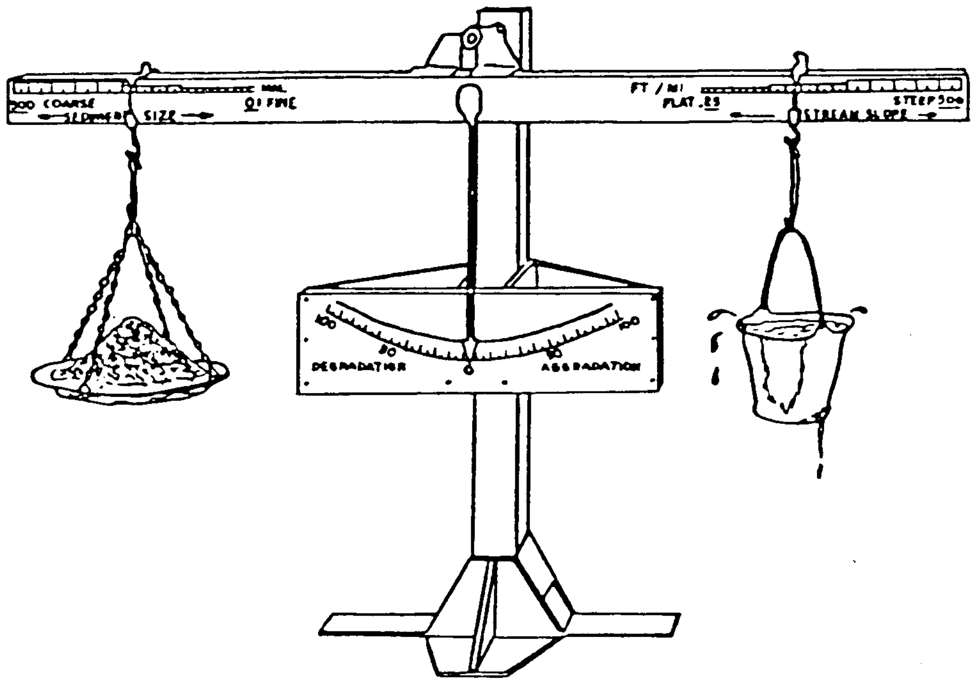

Figure 6.3. Schematic of the Lane relationship for qualitative analysis.

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):

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

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

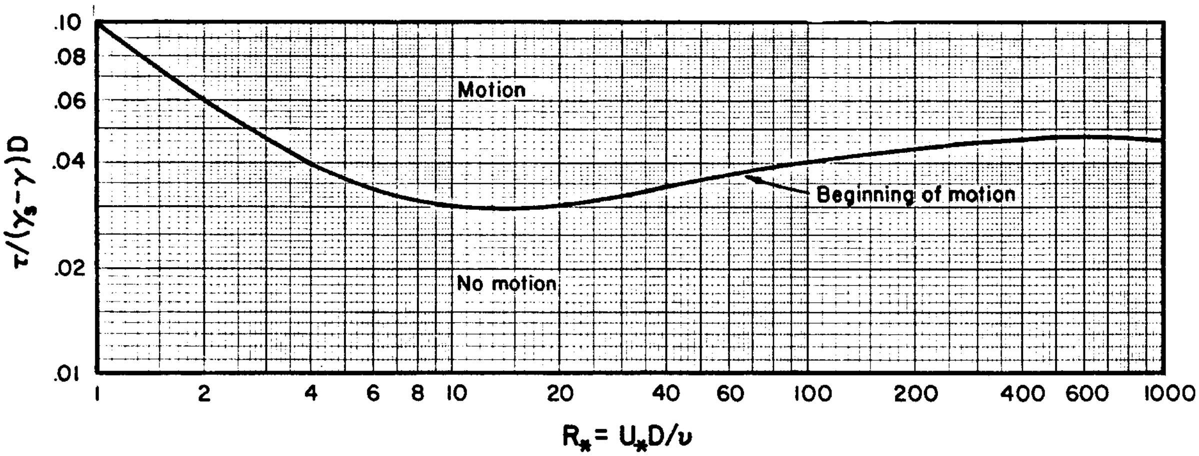

Figure 6.4. Shields' Diagram.

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).

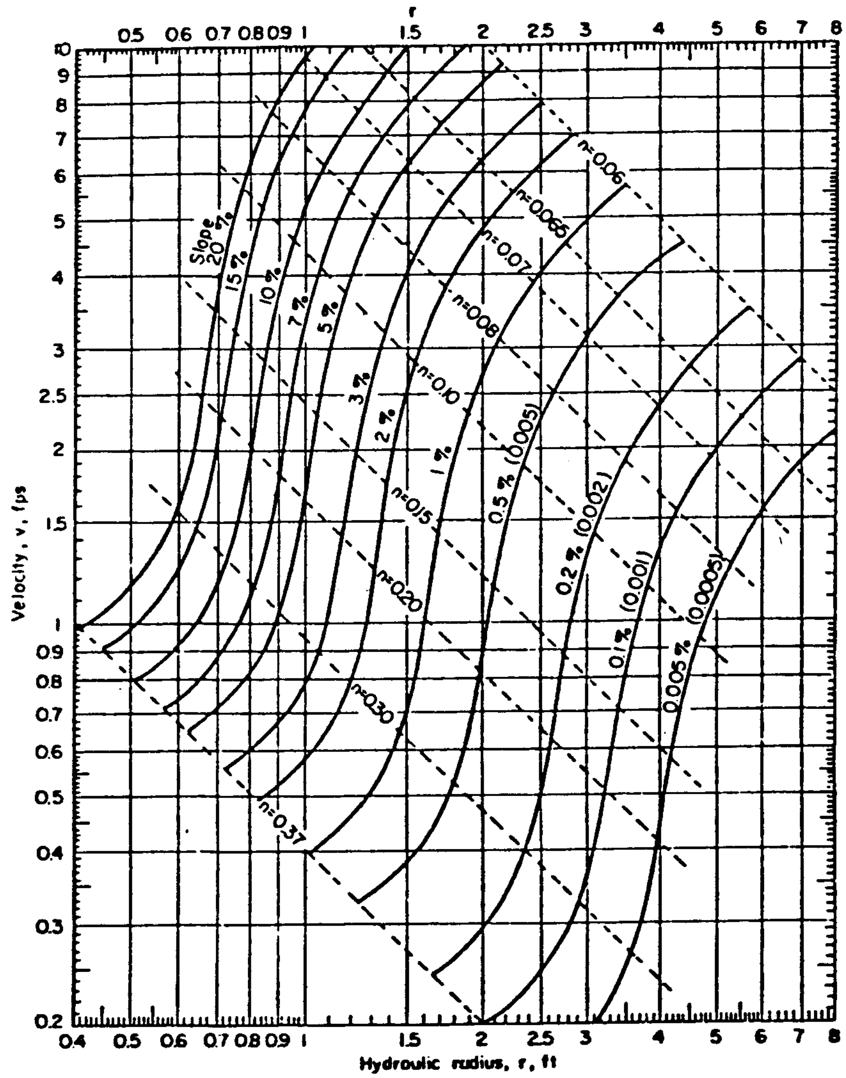

Figure 6.5a. Solution of the Manning equation for retardance A (very high vegetal retardance) (U.S. Soil Conservation Service).

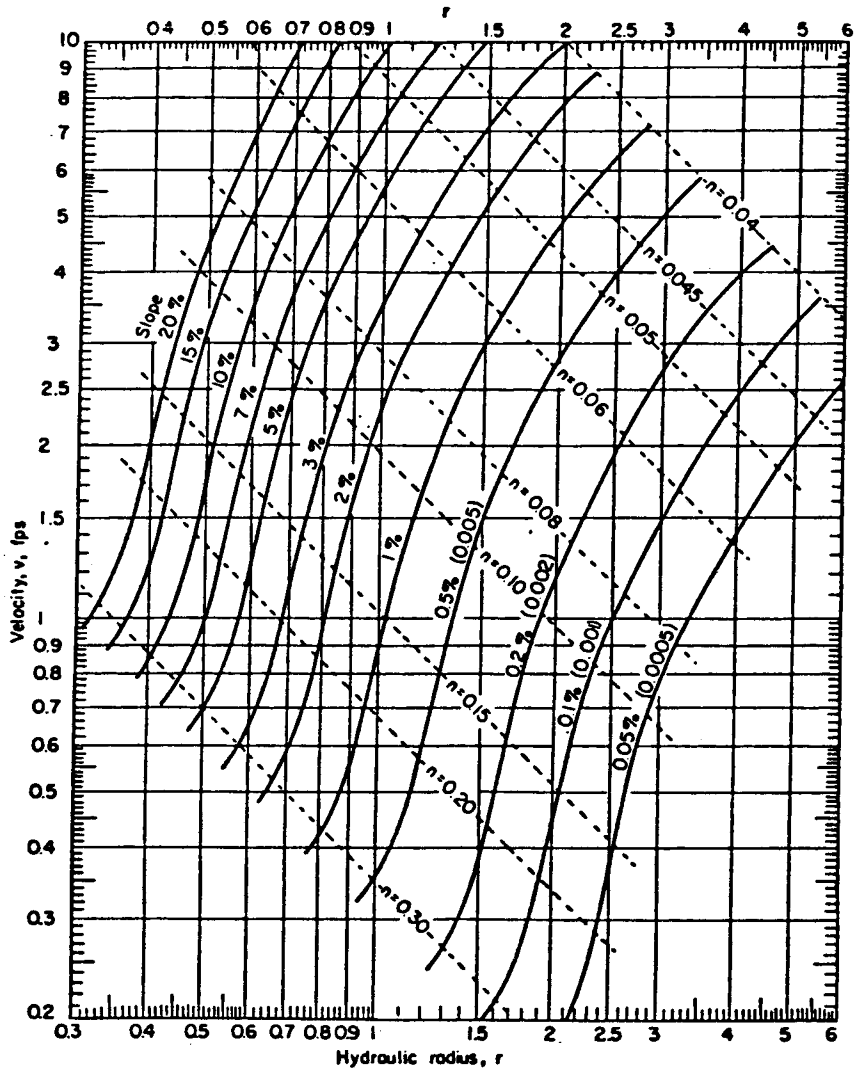

Figure 6.5b. Solution of the Manning equation for retardance B (high vegetal retardance). (U.S. SCS)

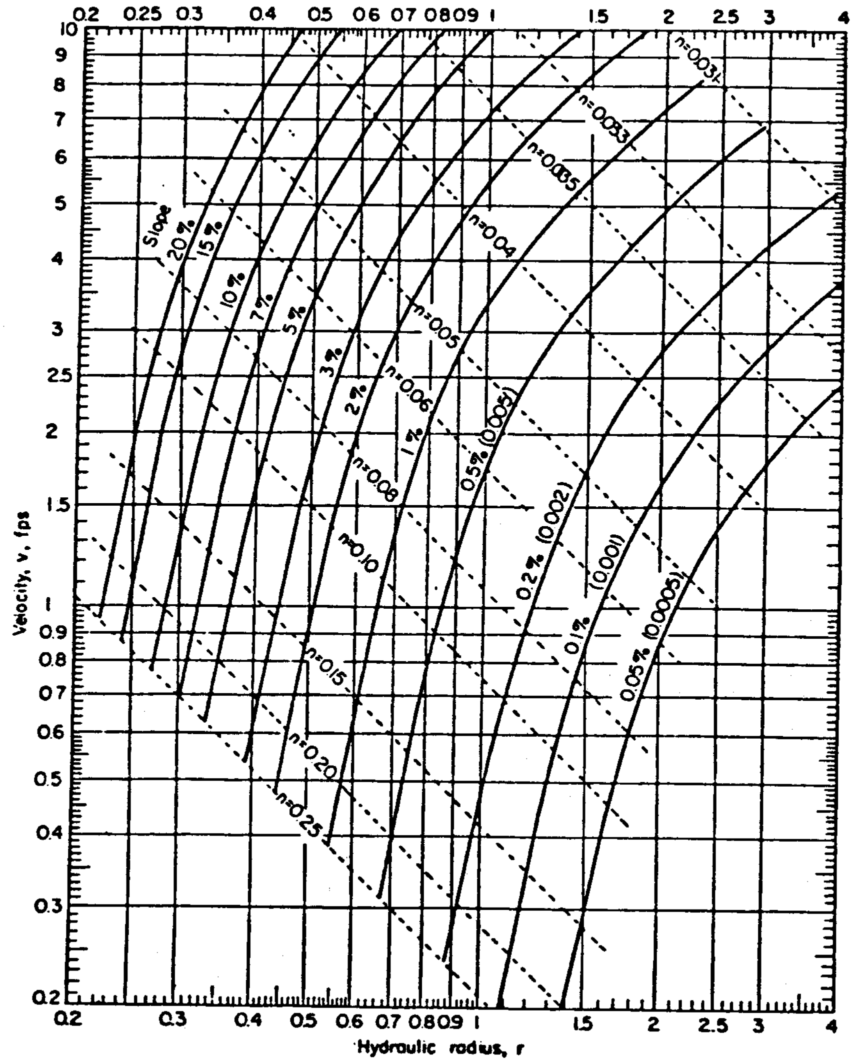

Figure 6.5c. Solution of the Manning equation for retardance C (moderate vegetal retardance). (U.S. SCS)

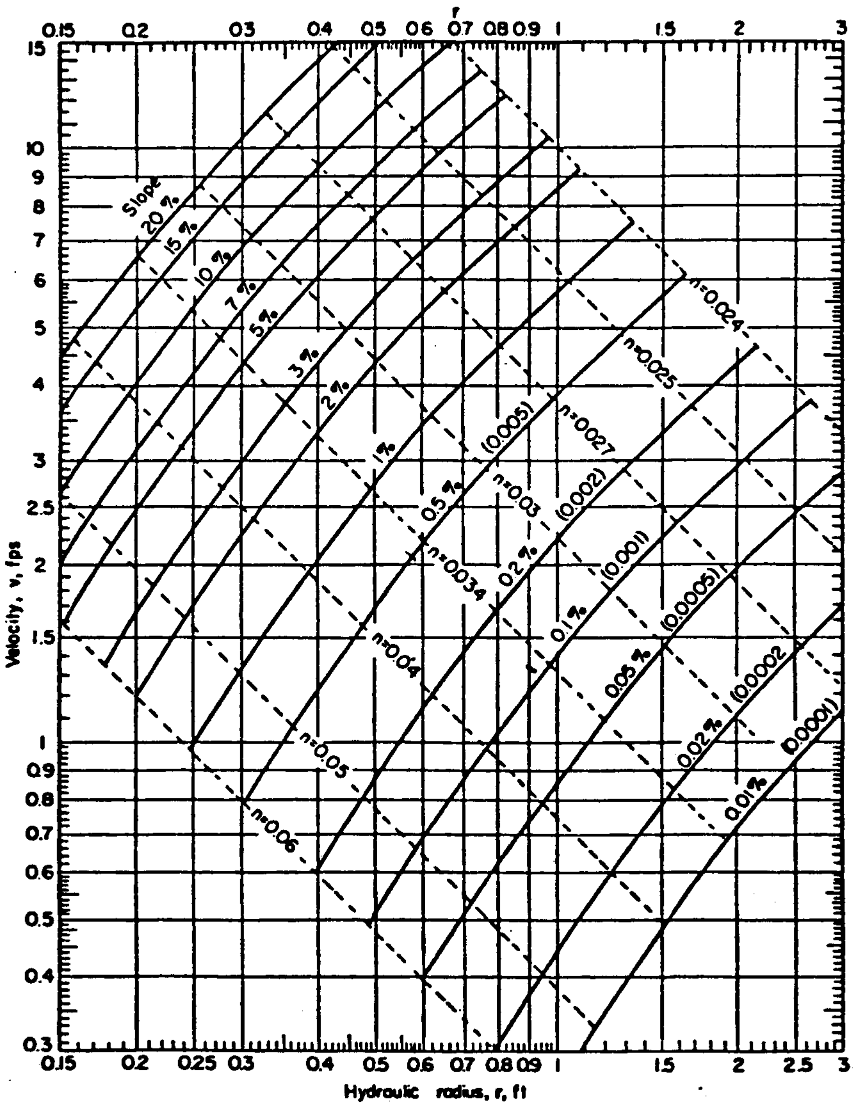

Figure 6.5d. Solution of the Manning equation for retardance D (low vegetal retardance). (U.S. SCS)

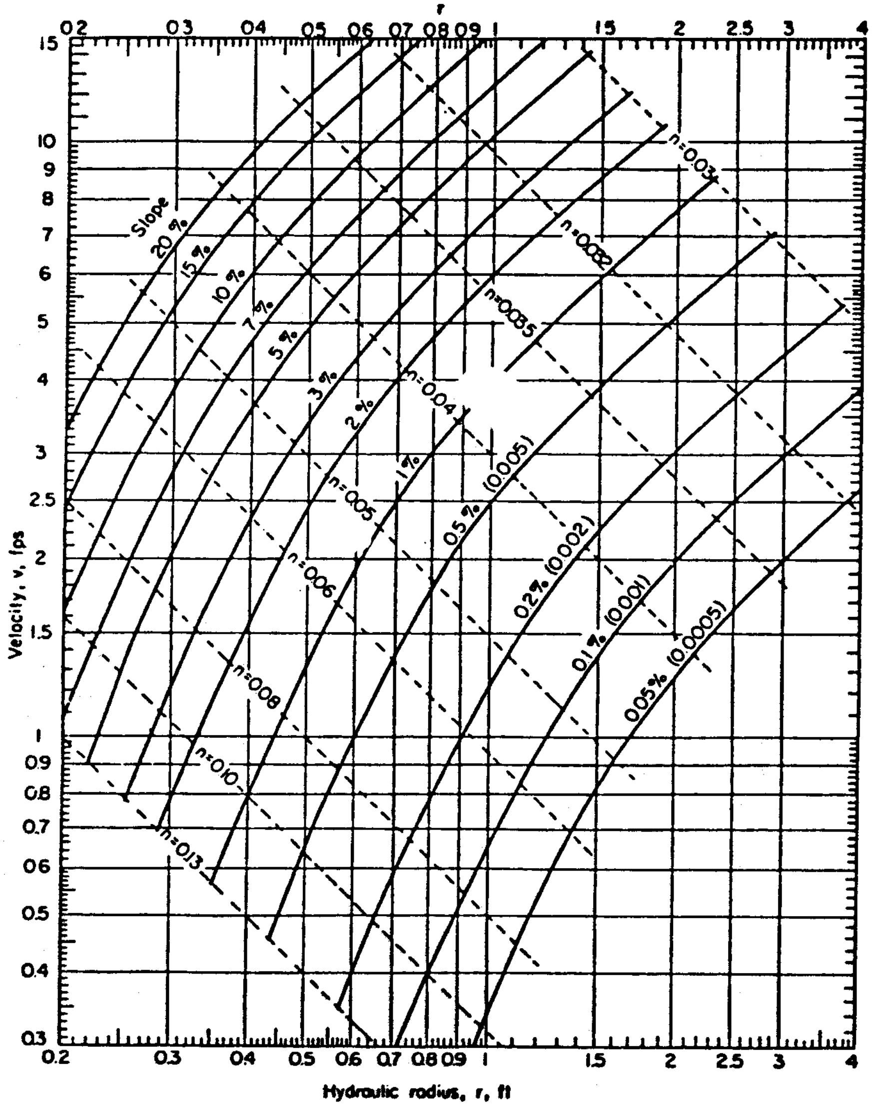

Figure 6.5e. Solution of the Manning equation for retardance E (very low vegetal retardance). (U.S. SCS)

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

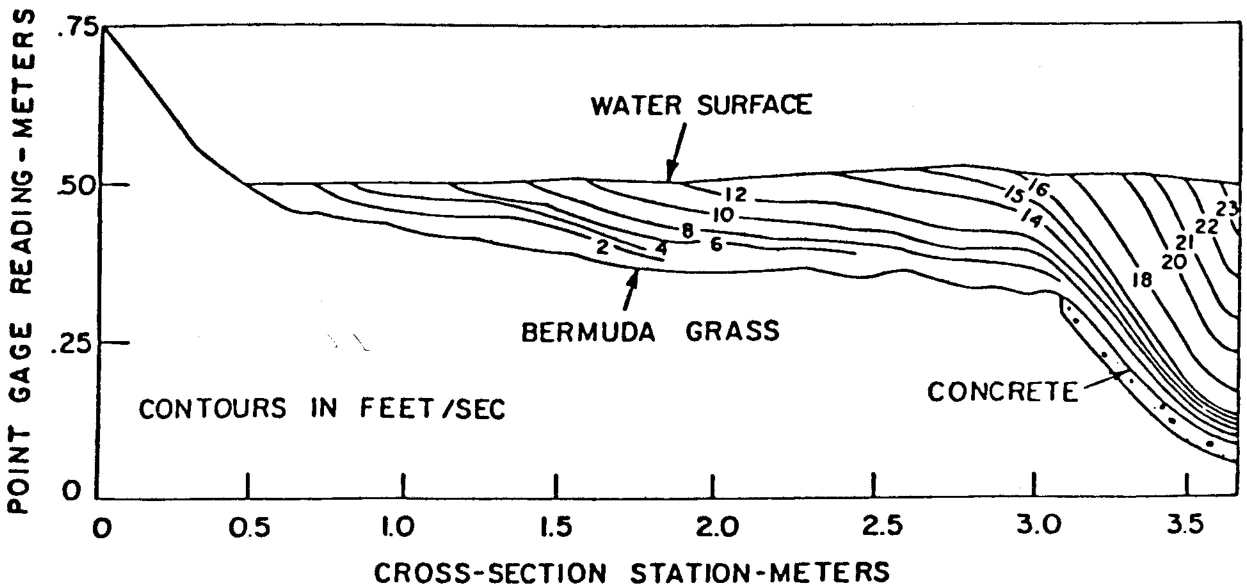

Figure 6.6. Dimensions and velocity distribution, Ree (1951).

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.

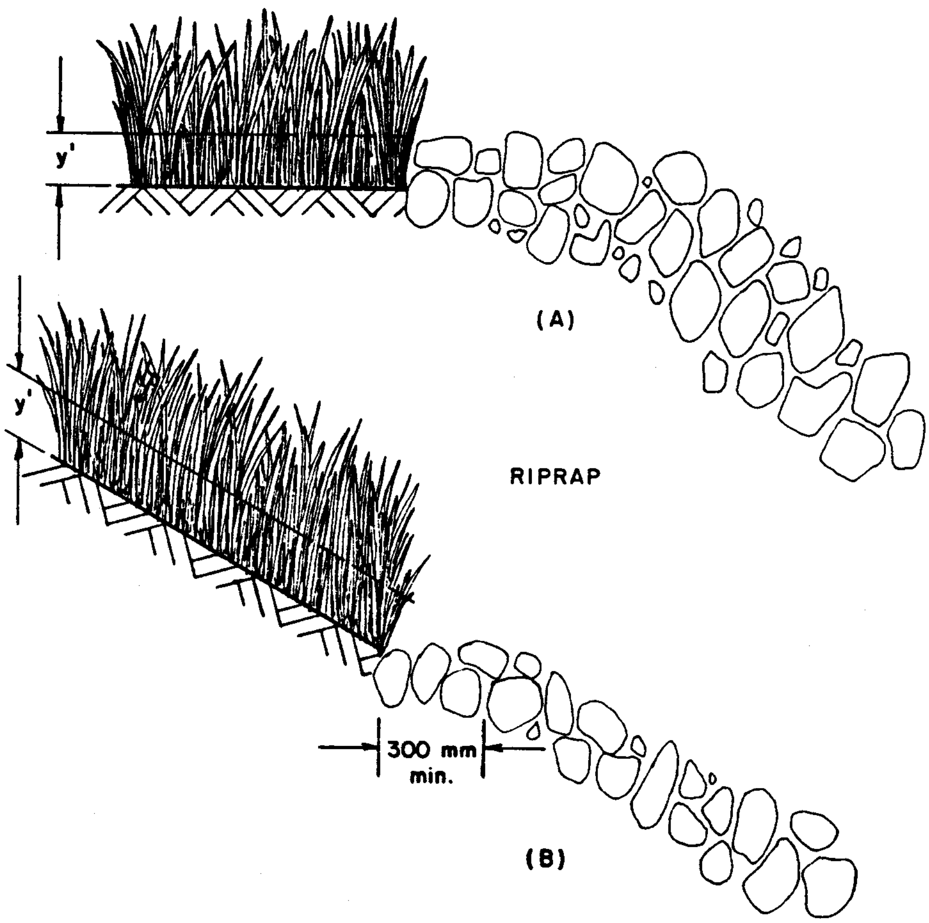

Figure 6.7. Detail of suggested grass to riprap junction.

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.

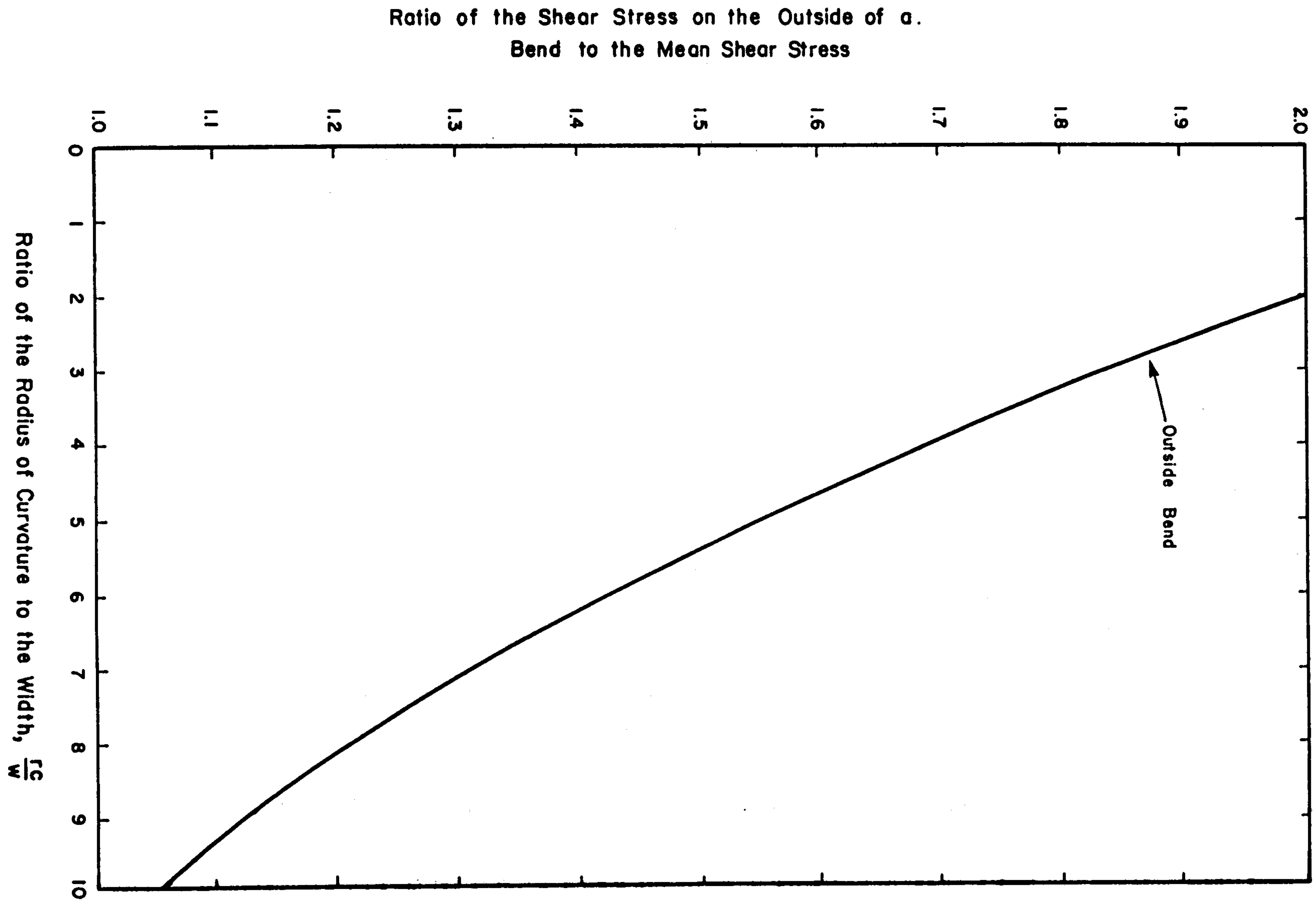

Figure 6.8. Effect of bend on boundary shear stress (after Soil Conservation Service design manual).

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

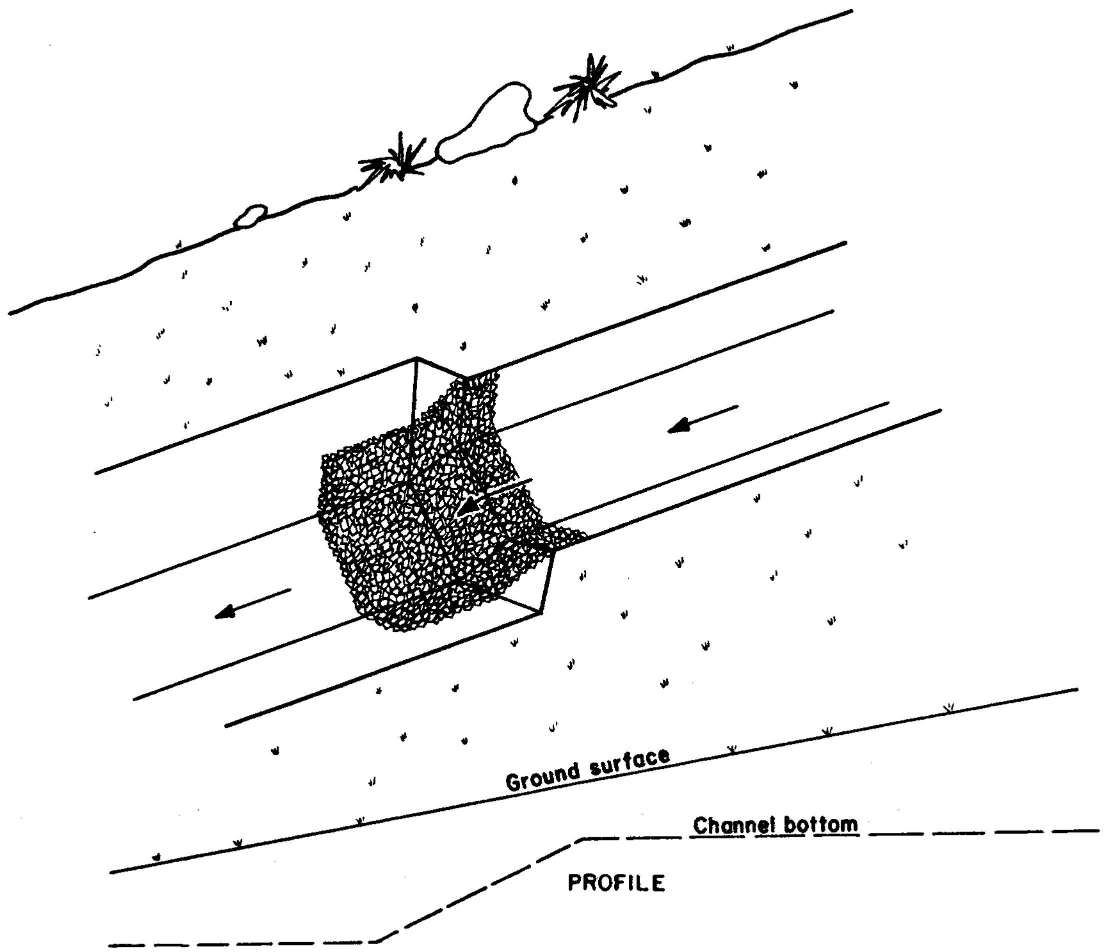

Figure 6.9. Definition sketch of a rock riprap drop structure (protection upstream and downstream according to Section 5.4).

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

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

$$\Delta H = (S_o - S)\ \Delta X \qquad (6.13)$$

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

$$N = \frac{\Delta H}{3} \qquad (6.14)$$

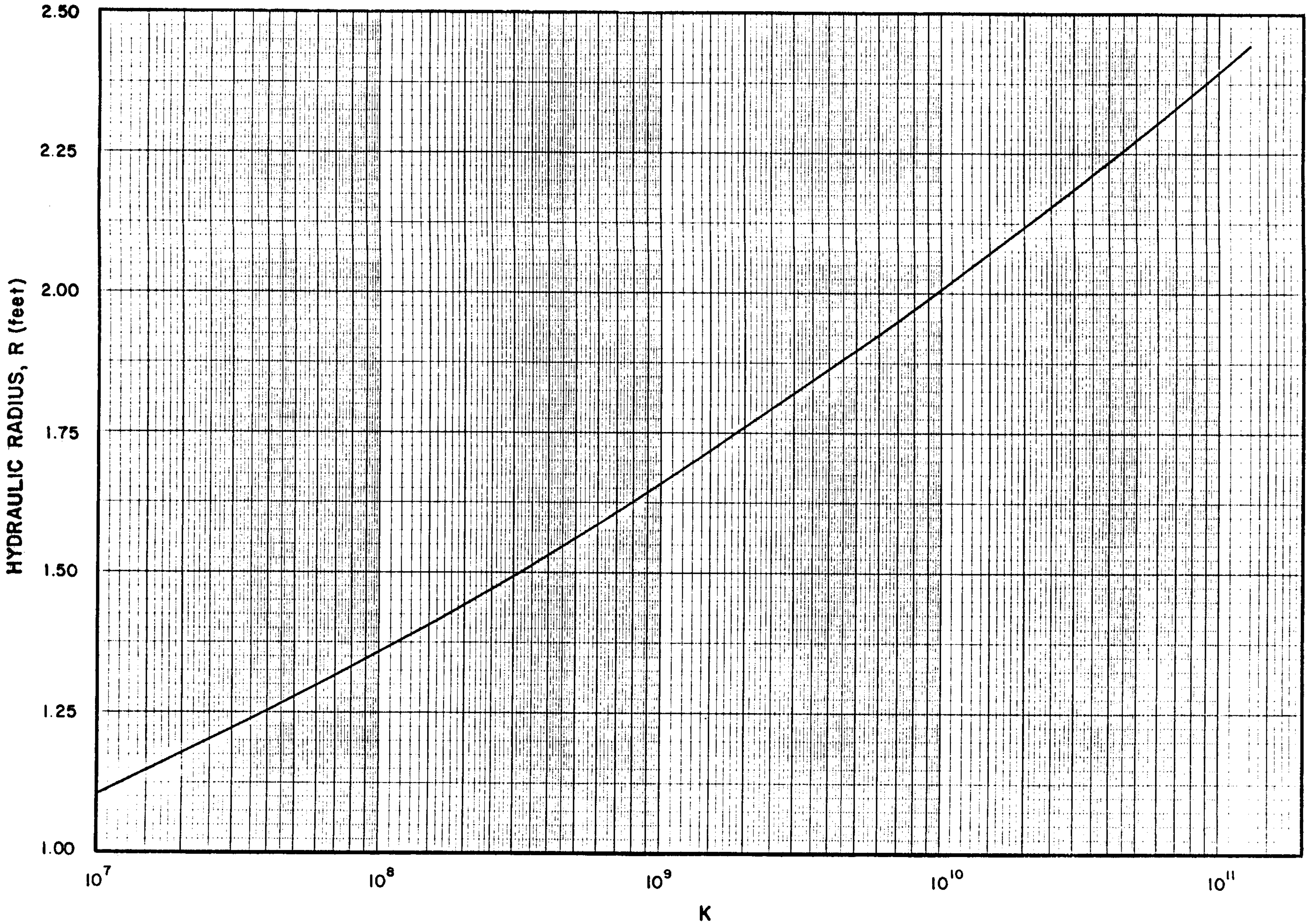

Figure 6.10a. Relationship between hydraulic radius R and K for trapezoidal channel with 2:1 side slopes and 6-foot base width.

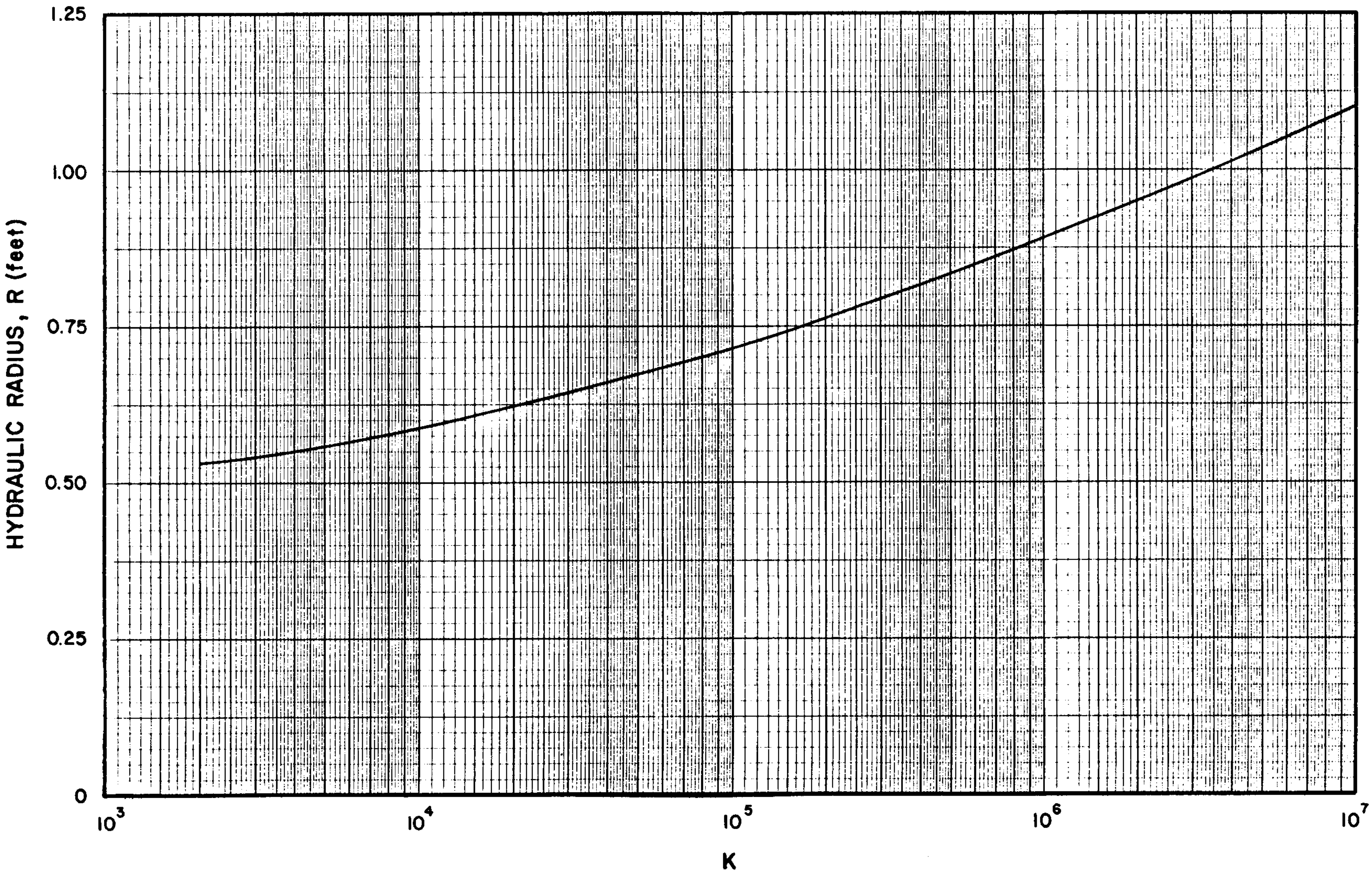

Figure 6.10a. Relationship between hydraulic radius R and K for trapezoidal channel with 2:1 side slopes and 6-foot base width. (continued).

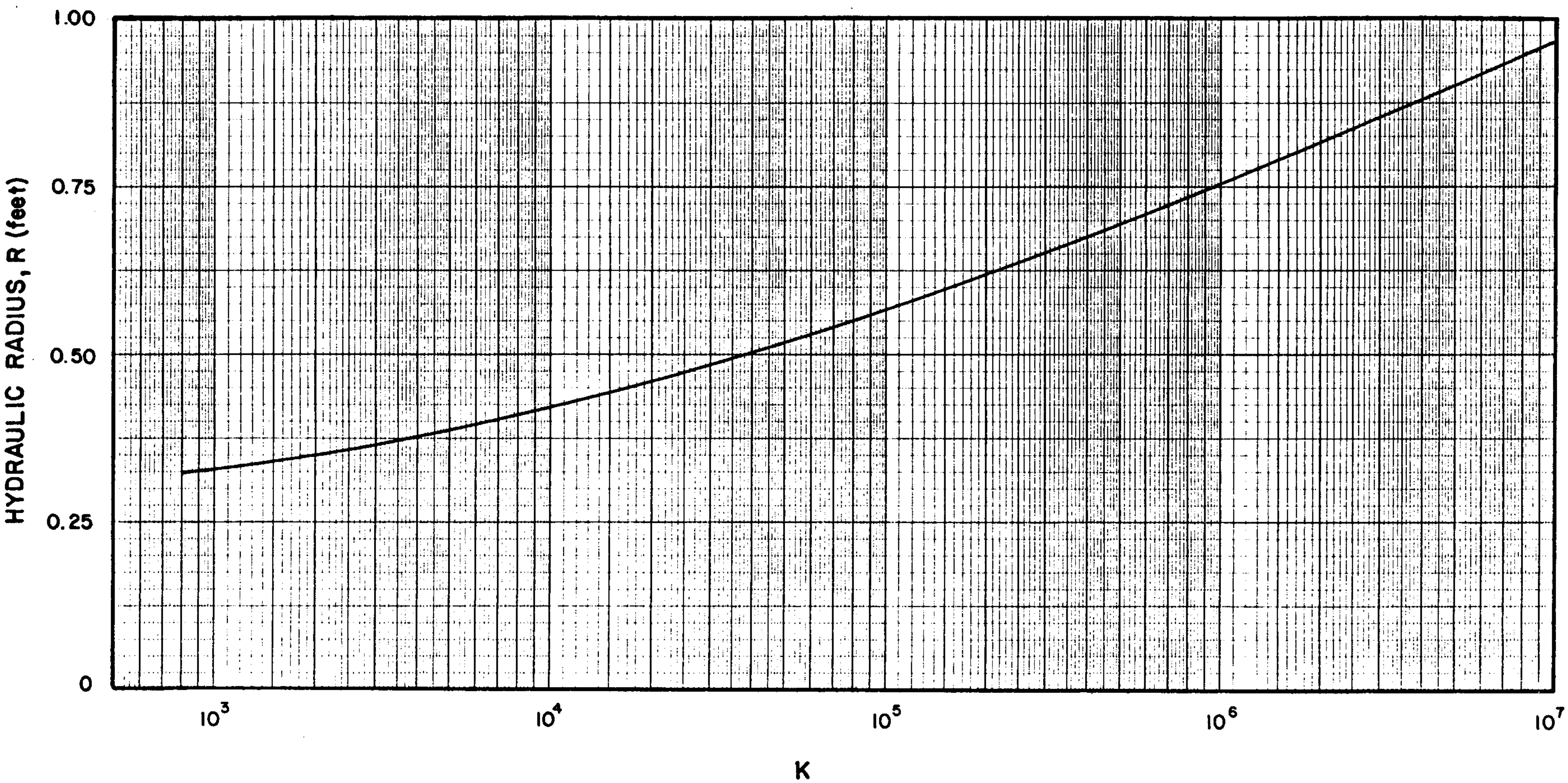

Figure 6.10b. Relationship between hydraulic radius R and K for trapezoidal channel with 2:1 side slopes and 10-foot base width.

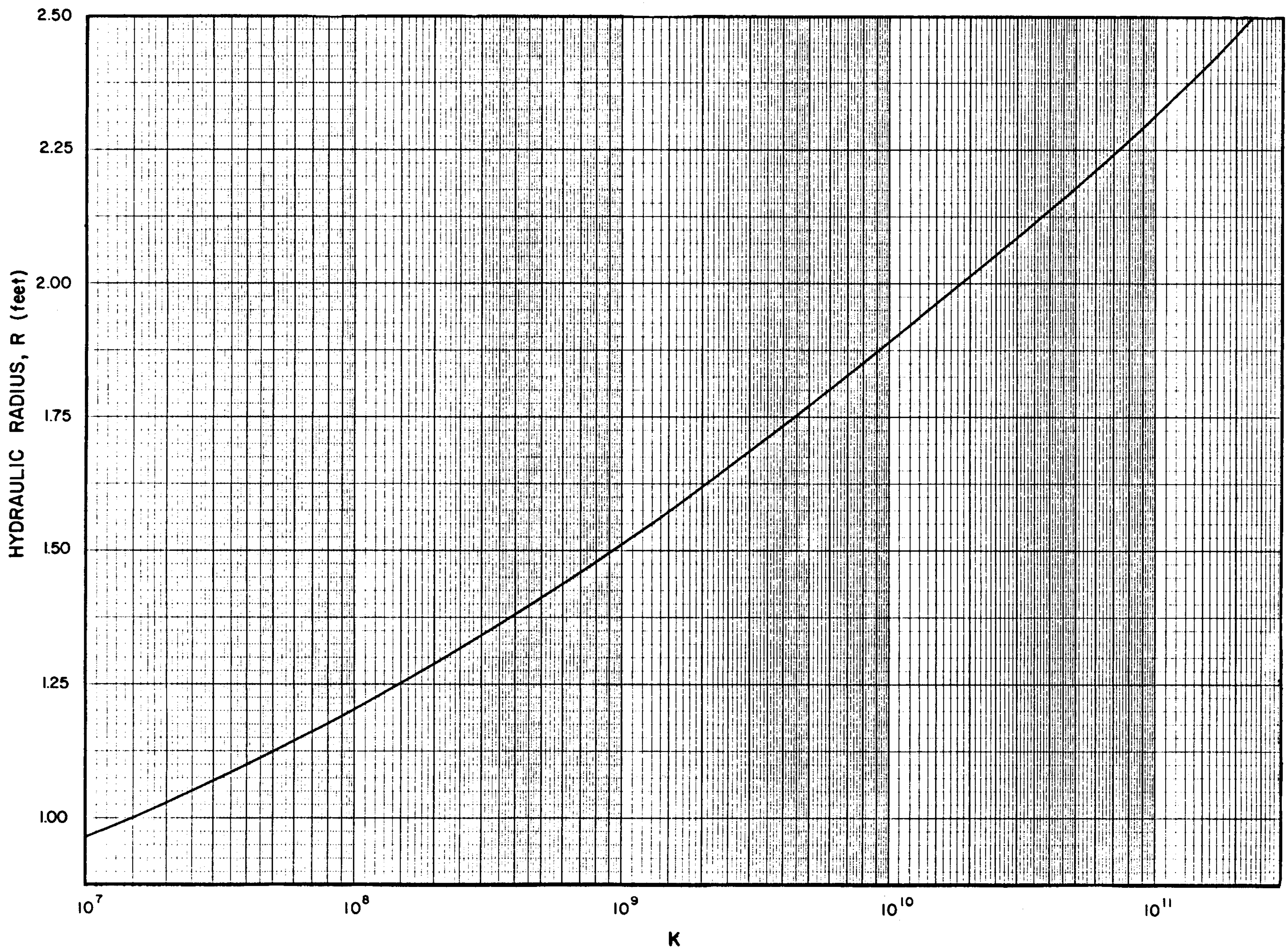

Figure 6.10b. Relationship between hydraulic radius R and K for trapezoidal channel with 2:1 side slopes and 10-foot base width (continued).

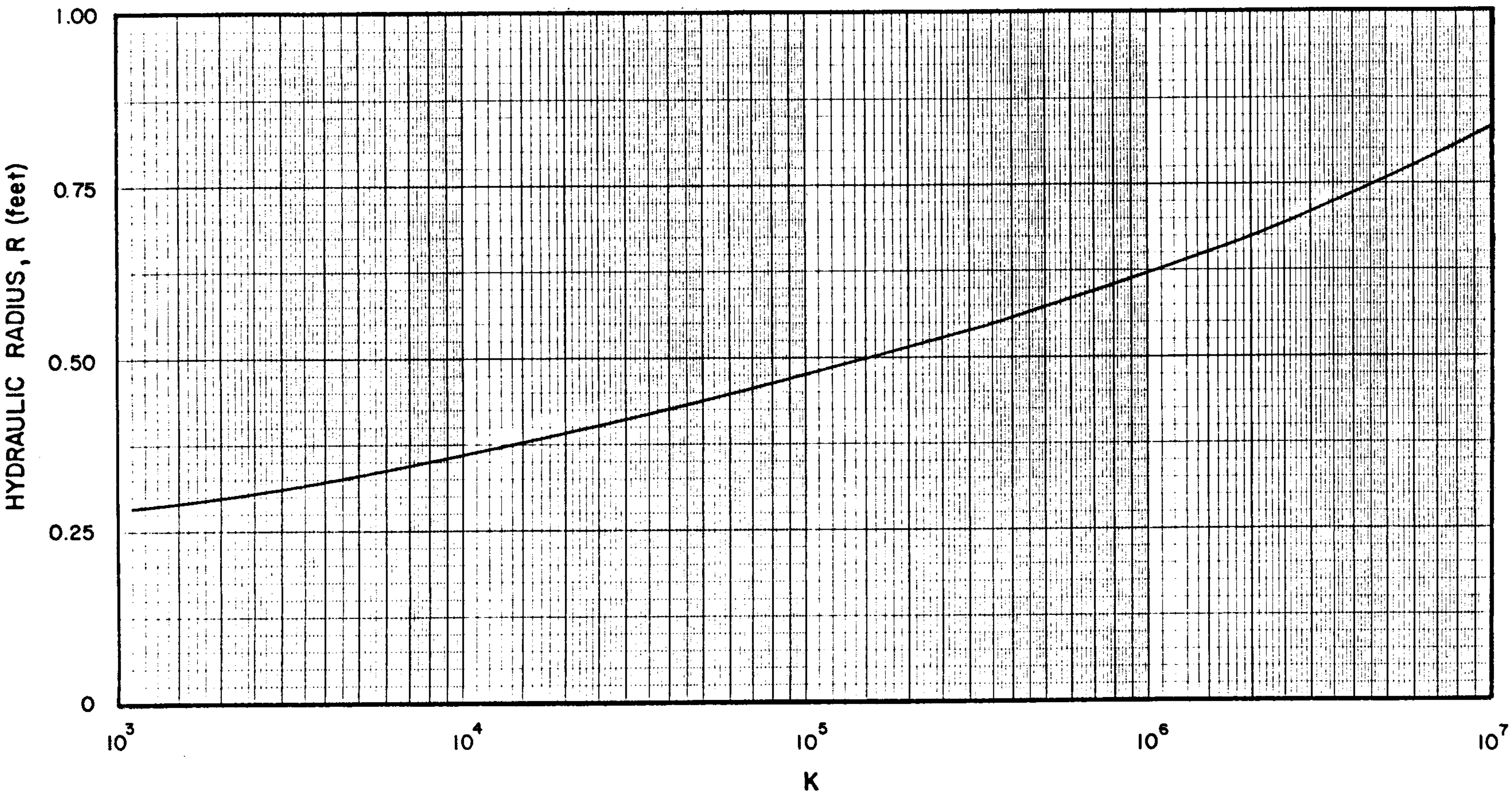

Figure 6.10c. Relationship between hydraulic radius R and K for trapezoidal channels with 2:1 side slopes and 14-foot base width.

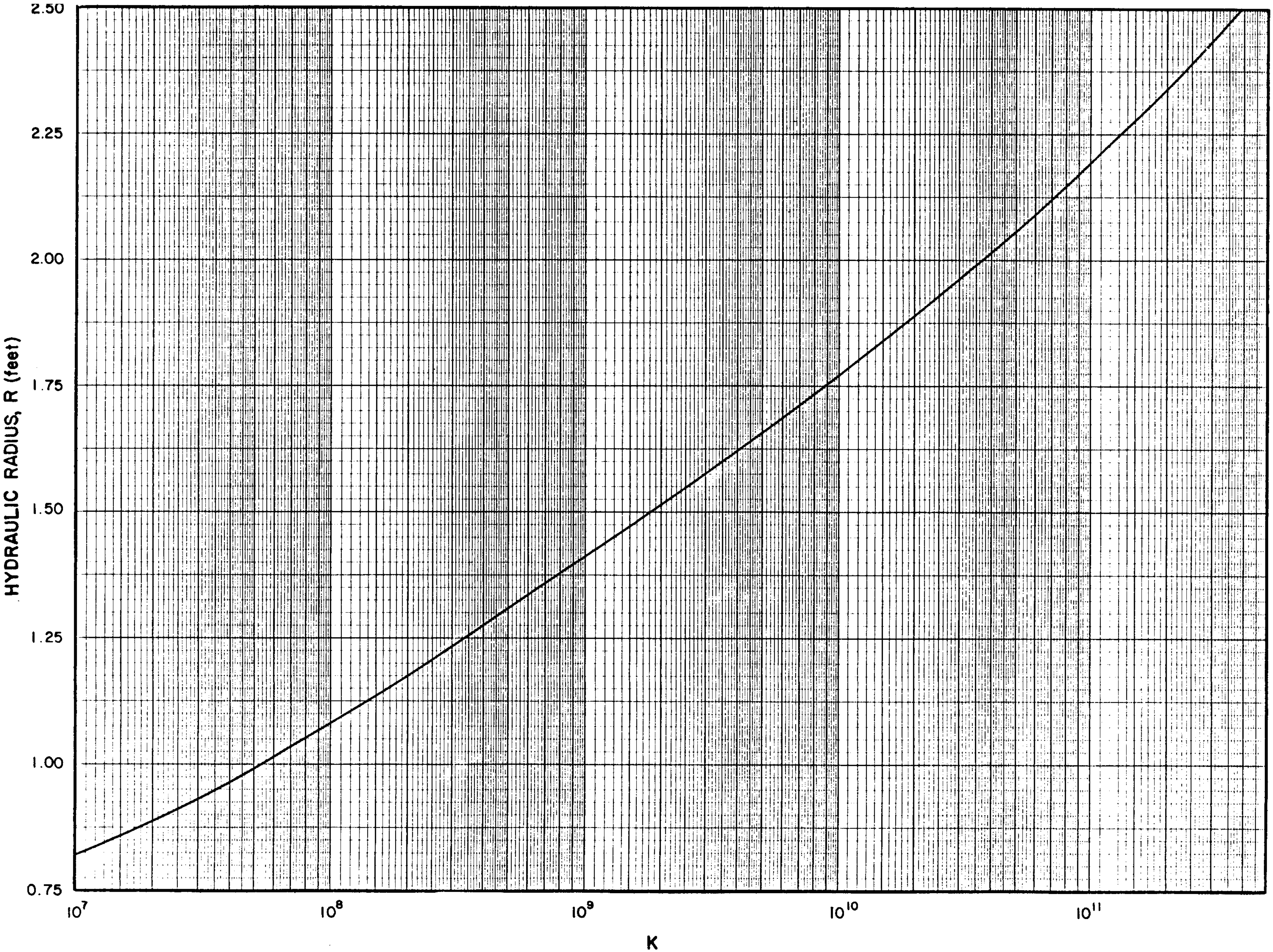

Figure 6.10c. Relationship between hydraulic radius R and K for trapezoidal channels with 2:1 side slopes and 14-foot base width (continued).

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 sedimeı.: 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.

<u>Solution</u>

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

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