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SIZING RIPRAP FOR THE PROTECTION OF APPROACH EMBANKMENTS & SPUR DIKES AND LIMITING THE DEPTH OF SCOUR AT BRIDGE PIERS & ABUTMENTS

Volume II: Design Procedure

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16. Abstract

This report presents methodologies currently available for sizing riprap protection measures. The five riprap performance areas identified in this report are: riprap quality, riprap-layer characteristics, hydraulic requirements, site conditions, and river conditions. Localized design considerations include: the protection of approach embankments, spur dikes, scour at bridge piers and abutments, river bends, and grade control structures. Limitations of the methods and their application to conditions observed in Arizona are evaluated and an interim design procedure is recommended.

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LIST OF SYMBOLS

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LIST OF SYMBOLS

Symbol	Description
A	Velocity-head coefficient
Ab	Angle of particle movement, degrees
Al	Oblique flow angle, degrees
AOS	Apparent opening size associated with filter fabric, mm
Ar	Angle of repose of the riprap, degrees
Az	Angle of the channel side-slope, degrees
B/R	Channel bottom width divided by the hydraulic radius
Ca	Critical shear stress adjustment factor
Cb	Factor for hydraulic forces for the direction of particle motion
Cl	Factor for hydraulic forces due to oblique flow
Cr	Factor for size and shape of riprap on a side-slope
Cs	Shield's parameter
Cw	Factor for stone unit weight on particle stability
Cz	Factor for side-slope particle stability
D50	Stone diameter for which 50 percent of the distribution is finer by weight
da	Average depth, (area/topwidth), ft
Ds	Characteristic stone size, ft
F1	Coefficient for the bridge invert
F2	Coefficient for the abutments and spurs
Fsf	Froude-effect coefficient for the side-slope
G	Gradation coefficient
g	Gravitational acceleration constant, 32.2 ft/sec^2
Hd	Total water-surface drop through the bridge, ft
Ka	Generalized local boundary shear stress adjustment factor
Kas	Boundary shear stress adjustment factor for abutments and spurs
Kbc	Boundary shear stress adjustment factor for channel bed at a bridge
Kbn	Boundary shear stress adjustment factor for channel bed in a straight, non-symmetric reach
Kbs	Boundary shear stress adjustment factor for channel bed in a straight, symmetric reach
Kp	Boundary shear stress adjustment factor for bridge piers
Kr	Boundary shear stress adjustment factor for channel bend
Ksf	Boundary shear stress adjustment factor for Froude effect on channel side-slope
Kss	Boundary shear stress adjustment factor for channel side-slope
L	Bridge abutment width, ft
Lp	Downstream protection length below a bend, ft
nb	Manning's n-value in a bend
Qzb	Base discharge in the zone of main channel flow, cfs
Qzd	Design discharge within the main channel banks, cfs
Rc	Centerline radius of the bend, ft
Se	Slope of the energy grade line, ft/ft
SF	Design safety factor
Ss	Riprap specific weight, Us/Uw
Tb	Boundary shear stress, lb/ft^2
Td	Design shear stress, lb/ft^2

LIST OF SYMBOLS
(continued)

Symbol	Description
To	Mean boundary shear stress, lb/ft^2
Ts	Applied shear stress, lb/ft^2
TW	Topwidth of the main channel (excluding overbank flow) for design conditions, ft
TWz	Topwidth of the zone of main channel flow, ft
Us	Unit weight of the stone, lbs/ft^3
Uw	Unit weight of water, 62.4 lb/ft^3
Va	Average velocity, ft/sec
W50	Particle weight for which 50 percent of the distribution is heavier, lbs
Z	Ratio of the side-slope's horizontal to vertical distance

I. INTRODUCTION

Riprap design involves the evaluation of five performance areas. These areas include the evaluation of: 1) riprap quality; 2) riprap-layer characteristics; 3) hydraulic requirements; 4) site conditions; and, 5) river conditions. In Arizona, site requirements and river conditions are important factors in the protection of bridge structures and flood-control channels. An overview of design considerations in these areas is given; however, the designer is cautioned that attention to these conditions is still largely a matter of engineering judgement. Whenever possible, advice should be sought from experienced engineers familiar with alluvial channel conditions, and their recommendations should be incorporated into the design.

II. QUALITY DETERMINATION

Riprap-quality determination refers to the physical characteristics of the rock particles that make up the bank protection. Qualities determined to be most important include density, durability, and shape. Requirements for each of these properties are summarized in this section.

2.1 Specific Gravity (Density)

The design stone-size for a channel depends on the particle weight, which is a function of the density or specific gravity of the rock material. A typical range of specific gravity in the field is from 2.4 to 2.9, with an average value being 2.65. It is recommended that, unless a test is conducted, the specific gravity be assumed to have a minimum value of 2.4 (150 lb/cf). All stones composing the riprap should have a specific gravity equal to or exceeding 2.4, following the standard test ASTM C127.

2.2 Durability

Durability addresses the in-plcae performance of the individual rock particles, and also the transportation of riprap to the construction site. In-place deterioration of rock particles can occur due to cycles of freezing and thawing, or can occur during transportation to the site. The rock particles must have sufficient strength to withstand abrasive action without reducing the gradation below specified limits. Qualitatively, a stone that is hard, dense, and resistant to weathering and water action should be used. Rock derived from igneous and metamorphic sources provide the most durable riprap. A procedure for field investigation of sedimentary rock sources for use as riprap is given in Charts 1 to 3.

In most cases, laboratory tests are unnecessary when this field investigation procedure identifies a competent rock source. If the procedure indicates the need for further laboratory tests, and other sources of riprap are unavailable, laboratory tests should be conducted. Specified tests that should be used determine durability include: the durability index test (see Appendix A) and absorption test (see ASTM C127). Based on these tests, the durability absorption ratio (DAR) is computed as follows:

$$\text{DAR} = \frac{\text{Durability Index}}{\text{Percent Absorption} + 1} \qquad (1)$$

The following specifications are used to accept or reject material:

1. DAR greater than 23, material is accepted;
2. DAR less than 10, material is rejected;
3. DAR 10 through 23:
   (a) durability index 52 or greater, material is accepted; and,
   (b) durability index 51 or less, material is rejected.

2.3 Shape

There are two basic shape criteria. First, the stones should be angular. Second, not more than 25 percent of the stones should have a length more than 2.5 times the breadth. The length is the longest axis through the stone, and the breadth is the shortest axis perpendicular to the length. Angularity is a qualitative parameter which is assessed by visual inspection. No standard tests are used to evaluate this specification.

CHART 1

Check the appropriate boxes along flow chart lines to define the durability of rock in question.

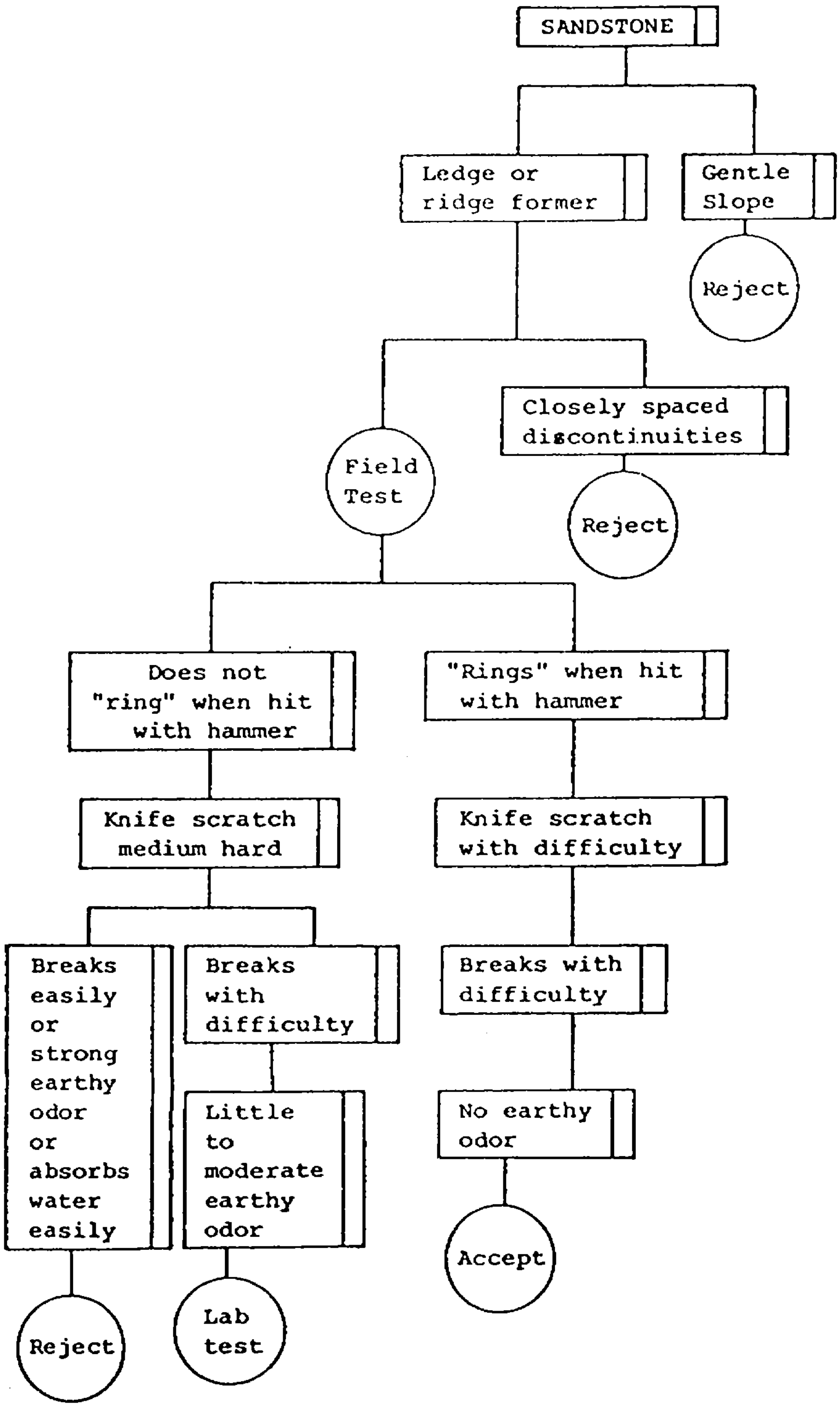


Chart 1. In-field Rock Durability Test - Sandstone (Design Manual for Water Diversions, OSM)

CHART 2

Check the appropriate boxes along flow chart lines to define the durability of rock in question.

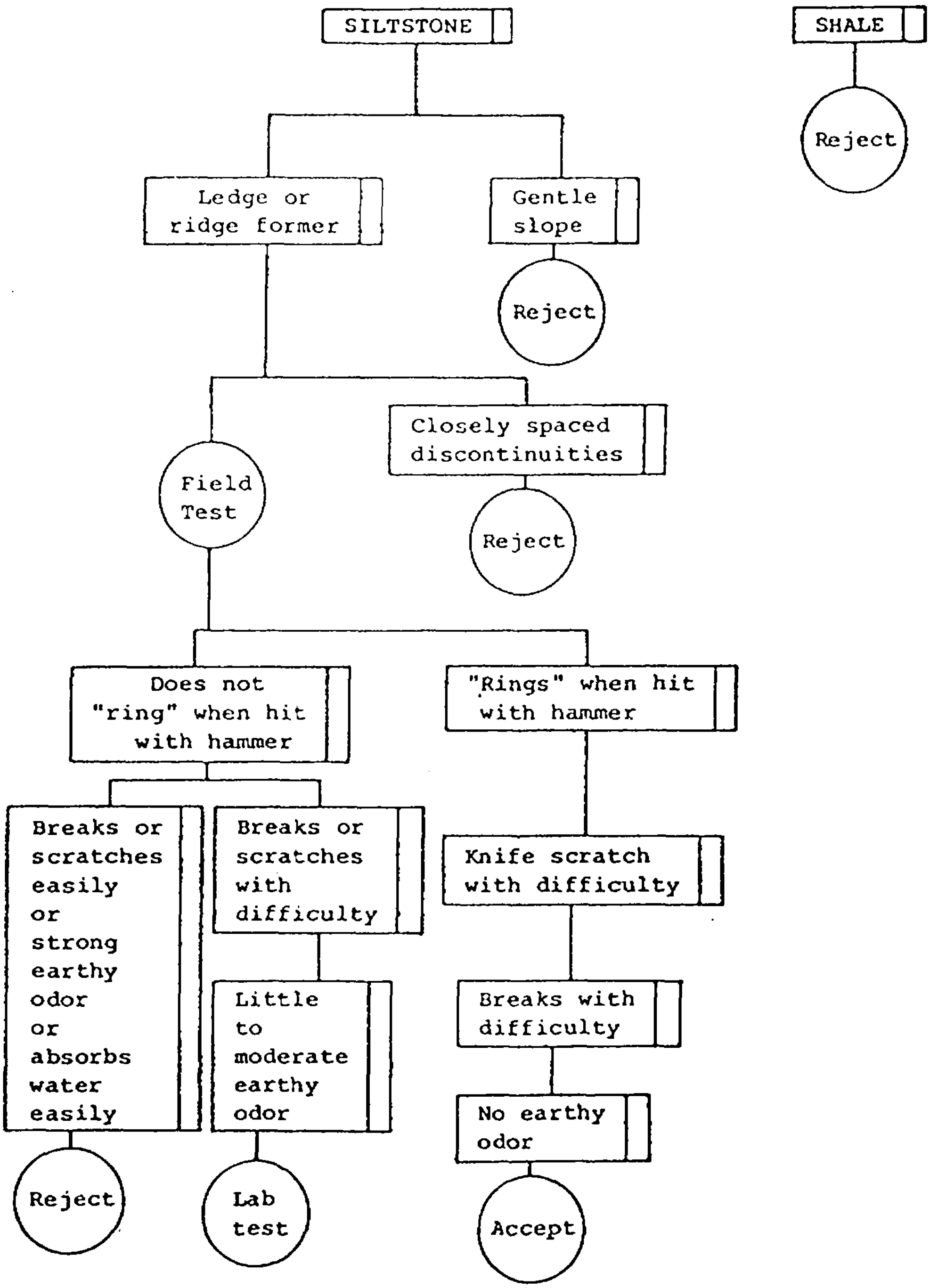


Chart 2. In-field Rock Durability Test - Siltstone

CHART 3

Check the appropriate boxes along flow chart lines to define the durability of rock in question.

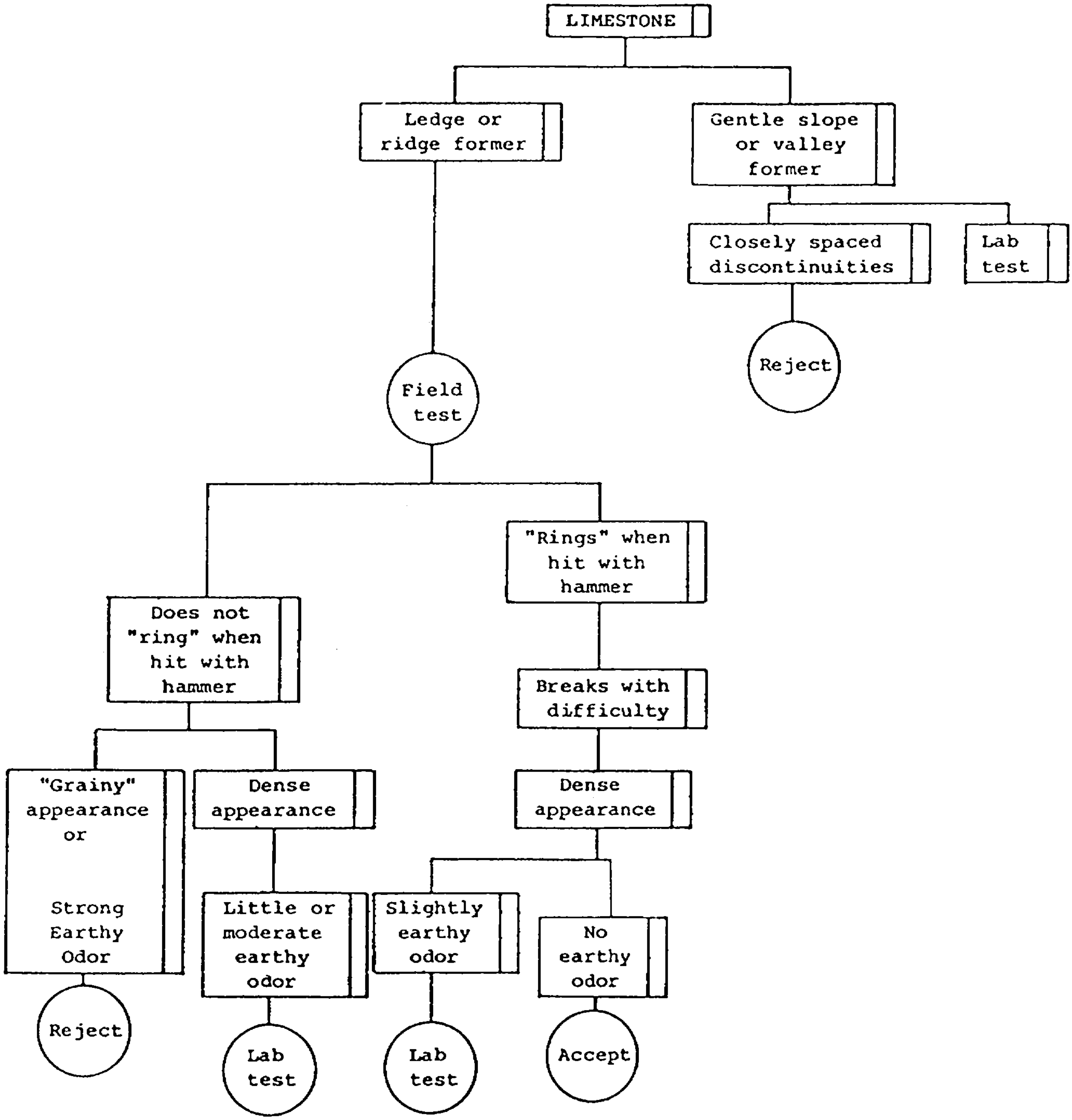


Chart 3. In-field Rock Durability Test - Limestone

III. RIPRAP LAYER CHARACTERISTICS

The major characteristics of the riprap layer include: 1) a characteristic size; 2) gradation; 3) thickness; and, 4) filter-blanket requirements. The following is a discussion of each of these characteristics.

3.1 Characteristic Size

The characteristic size in a riprap gradation is the D_{50}. This size represents the average diameter of a rock particle for which 50 percent of the gradation is finer, by weight. The conversion from particle weight to an average diameter is made by assuming that the particle is spherical, as follows:

$$D50 = \left(\frac{6W_{50}}{\pi U_s}\right)^{1/3}$$

Where

D_{50} = stone diameter for which 50 percent of the distribution is finer by weight;

W_{50} = particle weight for which 50 percent of the distribution is heavier;

π = 3.14159; and,

Us = unit weight of the stone (if untested, assume 150 lb/cf).

The installed riprap must have a characteristic size at least equal to this specified size.

3.2 Gradation

To form an interlocked mass of stones, a range of stone sizes must be specified. A dense, uniform mass of durable, angular stones with no apparent voids or pockets is the objective. The recommended maximum stone size is 2 times the D_{50} and the recommended minimum size is 1/3 the D_{50}. A gradation coefficient, G, of 1.5 is required. Where G is equal to $1/2(D_{84}/D_{50} + D_{50}/D_{16})$, where the Di represents the average diameter of a rock particle for which i percent of the gradation is finer by weight.

The following table provides design gradations for specified classes of riprap.

TABLE 1. Riprap Classifications								
	D50 Class							
% Passing	Size	6"	8"	12"	18"	24"	30"	36"
100-90	2.0 D50	12	16	24	36	48	60	72
85-70	1.5 D50	9	12	18	27	36	45	54
50-30	1.0 D50	6	8	12	18	24	30	36
15- 5	0.67 D50	4	5	8	12	16	20	24
5- 0	0.33 D50	2	3	4	6	8	10	12

As a practical matter, the designer should check with the materials engineer on the classes of riprap available near the site.

3.3 Thickness

The riprap-layer thickness should equal or exceed 1.5 times the D100 value. However, the layer thickness need not exceed twice the D100 value. The thickness is measured perpendicular to the slope upon which the riprap is placed.

3.4 Filter Blanket Requirements

The purpose of granular filter blankets underlying riprap is two fold: 1) they protect the underlying soil from washing out; and 2) they provide a base on which the riprap will rest. The need for a filter blanket is a function of particle-size ratios between the riprap and the underlying soil that comprise the channel bank. The inequalities that must be satisfied are as follows:

$$(D15)_{filter} \le 5 * (D85)_{bank} \qquad (3a)$$

$$4 * (D15)_{bank} \le (D15)_{filter} \le 20 * (D15)_{bank} \qquad (3b)$$

$$(D50)_{filter} \le 25 * (D50)_{bank} \qquad (3c)$$

If the inequalities are satisfied by the riprap itself, then no filter blanket is required. If the difference between the bank material and the riprap gradations are very large, then multiple filter layers may be necessary. To simplify the use of a gravel filter layer, the following standard gradations are recommended:

TABLE 2. Gradation for Gravel Bedding		
U.S. Standard Sieve Size	Type-I (Percent Passing by Weight)	Type-II
3"	-	90-100
1-1/2"	-	-
3/4"	-	20- 90
3/8"	100	-
#4	95-100	0- 20
#16	45- 80	-
#50	10- 30	-
#100	2- 10	-
#200	0- 2	0- 3

The Type-I and Type-II bedding specifications shown in Table 2 were developed using the criteria given in Equation 3, and considering that very fine grained, silty, non-cohesive soils can be protected with the same bedding gradation developed for a mean grain size of 0.045 mm. The Type-I bedding in Table 2 is designed to be the lower layer in a two-layer filter for protecting fine-grained soils. When the channel is excavated in coarse sand and gravel (i.e., 50 percent or more by weight retained on the #40 seive), only the Type-II filter is required. Otherwise, two bedding layers (Type-I topped by Type-II) are required. For the required bedding thickness, see Table 3.

TABLE 3. Thickness Requirements for Gravel Bedding			
	Minimum Bedding Thickness (inches)		
D50	Fine-Grained Soils		Coarse-Grained
Classification	Type-I	Type-II	Type-II
6",8"	4	4	6
12"	4	4	6
18"	4	6	8
24"	4	6	8
30"	4	8	10
36"	4	8	10

The design criteria for filter fabric are a function of the permeability of the fabric and the effective opening size. The permeability of the fabric must exceed the permeability of the underlying soil, and the apparent opening size must be small enough to retain the soil. The criteria for apparent opening size (AOS) are as follows:

1. For soil with less than 50 percent of the particles, by weight, passing a U.S. No. 200 sieve, the AOS should be less than 0.6 mm (a U.S. No. 30 sieve).
2. For soil with more than 50 percent of the particles, by weight, passing a U.S. No. 200 sieve, the AOS should be less than 0.3 mm (a U.S. No. 50 sieve).

IV. HYDRAULIC DESIGN REQUIREMENTS

To achieve stable riprap protection, the hydraulic forces acting to detach a stone must be countered by the weight of the stone. The unit force acting to detach the stone from the riprap layer is referred to as the boundary shear stress. The unit force acting to prevent movement of the stone is the component of stone weight perpendicular to the slope, and is referred to as the permissible design shear stress. To provide a stable riprap layer, the design shear stress must exceed the boundary shear stress, or:

$$Td > Tb \tag{4}$$

Where

Td = design shear stress; and,

Tb = boundary shear stress.

4.1 Determination of Average Boundary Shear Stress

The average boundary shear stress is the tractive force (the force acting parallel to the energy gradient of flow) per unit of wetted area. It is calculated by:

$$To = Uw * da * Se \tag{5}$$

Where

To = mean boundary shear stress, lb/sf;

Uw = unit weight of water, 62.4 lbs/ft^3;

da = average depth, (area/topwidth), feet; and,

Se = slope of the energy grade line, ft/ft.

The average boundary shear stress can be calculated from the following equation, which is derived from the Manning equation and Equation 5.

$$To = \frac{n^2}{2.21} \, Uw \;\; da^{-1/3} Va^2$$

Where

Uw = unit weight of water, 62.4 lb/ft^3;

da = average depth, (area/topwidth), feet; and,

Va = average velocity, ft/sec.

Recommended values for Manning's "n" values are provided in Appendix C, along with methods of calculating "n" values in channels with composite roughness.

4.2 Determination of Critical Shear Stress

The Shield's parameter is the ratio of applied shear stress to particle weight per unit area, and is defined as:

$$Cs = \frac{Ts}{(Us - Uw) * Ds} \qquad (7)$$

Where

Ts = applied shear Stress, lb/sf;

Uw = unit weight of water, 62.4 lb/cf;

Us = unit weight of stone, assume 150 lb/cf; and,

Ds = characteristic stone size, ft.

For a characteristic size based on the D_{50} size, and a minimum specific stone density of 2.4, with a Shield's parameter of Cs = 0.047, the critical shear stress for a given stone size, Tc, can be determined by the following equation:

$$Tc = 4.1 * D_{50} \qquad (8)$$

When the boundary shear stress is equal to the critical shear stress, the stones making up the riprap layer will begin to move. In other words, the riprap layer is stable when the boundary shear stress is less than the critical shear stress.

4.3 Design Safety Factors

The required safety factor, SF, for design depends on the use of the drainage facility, and the degree of risk that can be assumed in the design. Recommended safety factors for highway drainage facilities are:

Minor channels and culvert outlets	1.25
Major channels and roadway embankments	1.40
Bridges and associated protection measures	1.60

4.4 Determination of the Design Shear Stress

The critical shear stress is the maximum force that a riprap particle can sustain on the bed of the channel. Numerous additional factors reduce the critical shear stress (see Appendix B for a derivation). The design shear stress is the maximum shear stress that a stone can safely withstand, given local conditions. The design shear stress is given by the following equation:

$$Td = Ca * Tc \tag{9}$$

Where

Ca = critical shear stress adjustment factor.

Factors that are known to affect the design shear stress include: the rock density, the angle of repose, the channel side-slopes, the flow angle, and the factor of safety. On channel side-slopes, the stone has a reduced weight component with which to resist detachment. The angle of repose and the required safety factor also modify the allowable shear stress. The adjustment factor to the design shear stress is defined as follows:

$$Ca = (1/SF)\ (1/Cl)\ Cz\ Cw\ (1 - Cr\ Cb\ SF) \tag{10}$$
$$Cz = \cos Az \tag{11}$$
$$Cw = (Us - Uw)/(Uw*1.4) \tag{12}$$
$$Cr = \tan(Az)/\tan(Ar) \tag{13}$$
$$Cl = (1/2)\ (1 + \sin(Al + Ab)) \tag{14}$$
$$Cb = \cos(Ab) \tag{15}$$
$$Ab = \tan^{-1}\ (\cos(Al)/(E + \sin(Al))) \tag{16}$$
$$E = 2 * (Tc/To)\ Cz\ Cr \tag{17}$$
$$Az = \tan^{-1}\ (1/z) \tag{18}$$

Where

Az = angle of the channel side-slope;

Ar = angle of repose of the riprap;

Ab = angle of particle movement;

Al = oblique flow angle; and,

SF = design safety factor.

The diagrams in Appendix B illustrate these angles. The factor Cz accounts for the affect of channel side-slope on particle stability. As summarized in Table 4, the factor Cz decreases as the side-slope becomes steeper. The value Z is defined as the ratio of the side-slope's horizontal to vertical distance.

TABLE 4. Values of Factor Cz	
Z	Cz
0.5	0.45
1.0	0.71
1.5	0.83
2.0	0.89
3.0	0.95
4.0	0.97
5.0	0.98
horizontal	1.00

The factor Cw is an adjustment for the affect of stone density on particle stability. As discussed in the previous section, the specific weight of the stone is assumed to be at a minimum value being 2.4. As summarized in Table 5, the factor Cw increases with an increase in the material's specific weight (Ss), where Ss = Us/Uw.

TABLE 5. Values of Factor Cw	
Ss	Cw
2.4	1.00
2.5	1.07
2.6	1.14
2.65	1.18
2.7	1.21
2.8	1.29
2.9	1.36

The factor Cr is an adjustment for the affect of size and shape of riprap on a side-slope. The flatter the side-slope, or the larger the stone size, the more stable the particle becomes. For angular rock of the sizes used for riprap, the factor Cr does not vary with rock size. Table 6 provides values of Cr for angular stone on selected side-slopes.

TABLE 6. Values of Cr (Angular Stone)	
Z	Cr[1]
0.5	2.18
1.0	1.09
1.5	0.73
2.0	0.55
3.0	0.36
4.0	0.27
5.0	0.22
horizontal	0.00

[1] Assumes 42.5° angle of repose (i.e., angular/crushed riprap).

The factors Cl and Cb account for the hydraulic forces due to flow past a stone in the riprap layer. Because the angle of flow impinging on a stone need not be parallel to the stream bank or the angle at which forces on the stone resolve, these angles must be calculated before factors Cl and Cb can be determined. The angle Al is the oblique flow angle. Chart 4 provides values of angle Al as a function of the drop in water surface through a bridge opening relative to the embankment width. This drop in water surface can be estimated using a standard water-surface-profile program which includes routines that account for bridge losses. The embankment width is defined as the distance from the point where the water-surface profile intercepts the upstream face of the embankment, to the point where the water-surface profile intercepts the downstream face of the embankment (Figure 1).

The angle Ab is the angle at which a stone is pushed by the combined forces of drag on the stone and the down-slope weight of the stone. The angle depends on the angle of the side-slope, the stone size and shape, and the ratio of the boundary shear stress to the critical shear stress. Angle Ab is calculated in two steps. First, a coefficient E is determined from Chart 5. Next, using this coefficient and the oblique flow angle, Al, angle Ab is then determined from Chart 6. The mean boundary shear stress is calculated by Equation 5, and the critical shear stress is calculated by Equation 7. Once the angles Al and Ab are determined, the correction factor Cl can be determined using Chart 7. The correction factor Cb is determined from Chart 8.

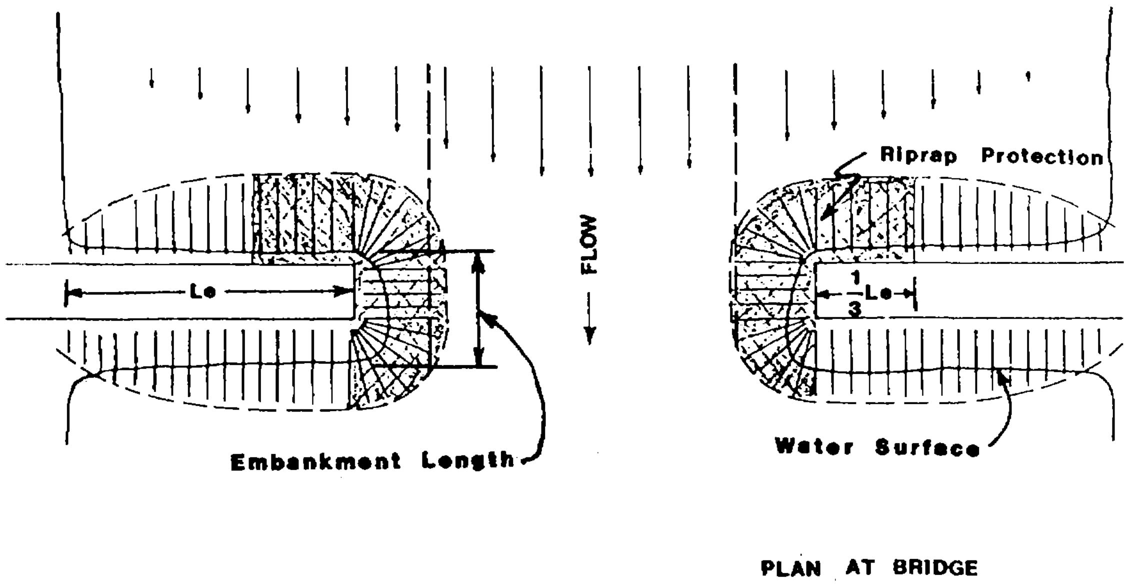


Figure 1. Location of Riprap Protection on a Bridge Approach Embankment

In some special cases, the correction factors in Equation 9 assume values of either zero or one. This can simplify the determination of Ca. Table 7 shows several simplified versions of Equation 9.

TABLE 7. Special Cases of the Critical Shear Stress Adjustment Factor

Case	Cl	Cz	Cr	Cb	Equation Form
Horizontal Bed	1.	1.	0.	1.	(1/SF) Cw
Straight Side-Slope	1.	--	--	1.	(1/SF) Cz Cw (1 - Cr SF)
Sloping Bed (Same as Straight Side-Slope, but Cz and Cr based on bed slope)					

4.5 Determination of the Local Boundary Shear Stress

The boundary shear stress varies within a river reach as a consequence of the non-uniform distribution of velocity in the cross section, or due to other features. For design, it is important to assess the maximum shear stress that can occur at specific locations in the reach. The local boundary shear stress is determined by multiplying the mean boundary shear stress by an associated adjustment factor that accounts for local hydraulic conditions. The maximum local boundary shear stress is calculated from the following equation:

$$Tb = Ka * To \tag{19}$$

Where

Ka = associated local boundary shear stress adjustment factor.

There are eight different local boundary shear stress adjustments factors that are summarized as follows:

Boundary Shear Stress Adjustment Factor	Description
Kbs	Channel bed in a straight, symmetric reach
Kbn	Channel bed in a straight, non-symmetric reach
Kbc	Channel bed at the bridge section
Kss	Channel side-slope
Ksf	Channel side-slope, Froude effect
Kas	Abutments and spurs
Kr	Channel bend
Kp	Bridge piers

The generic adjustment factor Ka in Equation 19 is replaced by the appropriate adjustment factor suited to the local condition in the channel.

4.5.1 Channel Bed Adjustment Factor

Straight, Symmetric Reach. In a straight reach, the shear stress varies across the channel section. The maximum boundary shear stress on the bottom of a trapezoidal channel can be found using the relationship developed for trapezoidal channels. The boundary shear stress adjustment factor for a channel bed in a straight, symmetric channel, Kbs, is given in Chart 9. The adjustment factor, Kbs, is a function of the channel side-slope and the aspect ratio, B/R, where the aspect ratio is defined as the channel bottom width divided by the hydraulic radius. For an aspect ratio greater than five, the hydraulic radius can be approximated by the average flow depth, da.

Straight, Non-Symmetric Reach. In channels with non-symmetric flow, a portion of the channel section will have a higher conveyance relative to the entire channel section (see Figure 2). In this zone, the boundary shear stress will also be higher. Brown and Blodgett (1987) developed an equation from field measurements of non-symmetric channels that was used to account for the increased boundary shear stress in non-symmetric channels. Unlike the other boundary shear stress adjustment factors, the non-symmetric adjustment factor adjusts the mean shear stress of the zone of main channel flow at the bank-full capacity during low flow to determine the shear stress at design discharge. The adjustment factor is:

$$Kbn = ((Qzd/Qzb)*(0.722+2.67(TWz/Tw)))^{0.664} \qquad (20)$$

Where

Qzd = the design discharge (within the main channel banks), in cfs;

Qzb = the base discharge in the zone of main channel flow, in cfs;

TWz = the topwidth of the zone of main channel flow, in feet; and,

TW = the topwidth of the main channel (excluding any overbank flow) for design conditions, in feet.

Chart 10 provides values of Kbn as a function of discharge and topwidth.

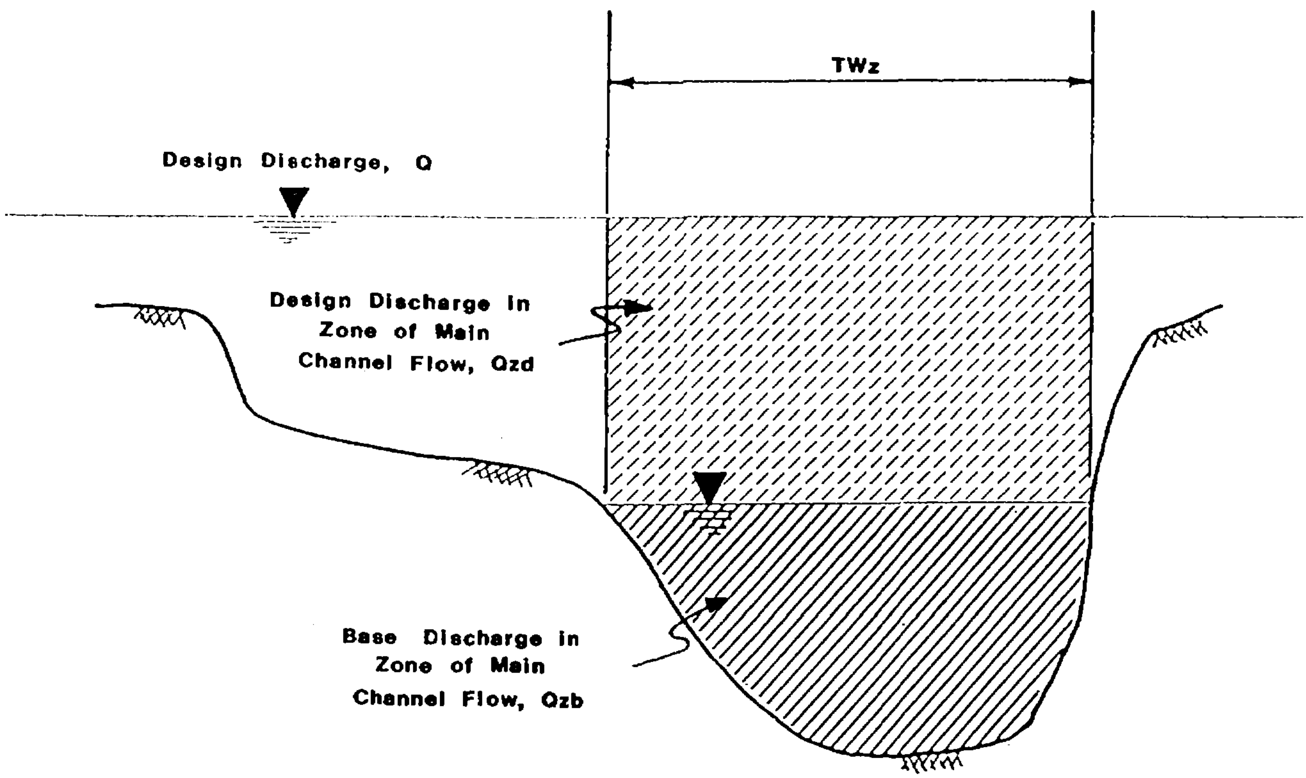


Figure 2. Non-Symmetric Channel Section

If the ratio of the topwidth of the main channel flow to the entire topwidth is greater than 0.75, the channel can be assumed to be symmetric, and the simpler method of computing boundary shear stress, using Chart 12 can be used.

<u>At the Bridge Section.</u> The channel bed beneath the bridge experiences an increase in flow velocity and boundary shear stress due to the contraction of the flow and the blockage caused by the piers. The increase in bed shear stress through the bridge is assumed to be proportional to the square of the ratio of the maximum contraction velocity to the approach velocity. The following equation provides the boundary shear stress adjustment factor for a bridge section:

$$Kbc = Fl*(A + 2g* Hd/Va^2) \qquad (21)$$

Where

Fl = component of the adjustment factor;

A = velocity-head coefficient;

Hd = total water-surface drop through the bridge, in feet;

Va = mean velocity of approach flow, in ft/sec; and,

g = gravitational acceleration constant of 32.2 ft/sec^2.

The velocity-head coefficient, A, ranges from 1.1 to 1.3 for straight channel reaches in the main zone of flow. For symmetric channels, A = 1.1 is recommended. For non-symmetric channels, A = 1.3 is recommended. The water-surface drop and approach velocity can be determined from backwater calculations that include bridge losses. Factor Fl is a function of the drop in water-surface through the bridge embankment. Factor Fl is determined from Chart 11.

4.5.2 Channel Side-Slope Adjustment Factors

The maximum boundary shear stress on the side of a channel can be found using the graph developed by Anderson for trapezoidal channels. The boundary shear stress adjustment factor for a channel side in a straight channel, Kss, is given in Chart 12. The adjustment factor, Kss, is a function of the channel side-slope and the aspect ratio, B/Y, where the aspect ratio is defined as the channel bottom width divided by the hydraulic radius. For an aspect ratio greater than five, the hydraulic radius can be approximated by the average flow depth, da.

Correction for High Froude Number. Many of the rivers in Arizona flow at a high Froude number. This is in contrast to most natural streams in other regions that flow at a Froude number below 0.3, and seldom flow at a Froude number greater than 0.6. For rivers in Arizona, it is not uncommon to have flood flows exceed a Froude number of 0.6 and to extend into the supercritical range, up to a Froude number of 1.5. An increase in Froude number will:

1. Cause the average wall shear to approach the maximum wall shear, thereby decreasing the ratio of maximum wall shear to average wall shear toward unity.

2. Cause the average total cross section shear to approach the average bottom shear, thereby decreasing the ratio of the average floor shear to average total cross section shear toward unity.

3. Cause the average wall shear to approach the average bottom shear, thereby increasing the ratio of average wall shear to average bottom shear toward unity.

The result is a more uniform shear distribution on the boundary.

BSSAF, the boundary shear stress adjustment factor for a channel side-slope due to a high Froude number, should be used for steep channels in Arizona. This boundary shear stress adjustment factor should be determined from Chart 12 and the following equation, with the larger side-slope adjustment factor used to determine the boundary shear stress.

$$Ksf = Fsf * Kb \tag{22}$$

Where

Ksf = BSSAF for the side-slope considering Froude effect;

Fsf = Froude-effect coefficient for the side-slope; and,

Kb = BSSAF for the bed, a previously calculated adjustment factor, symmetric, non-symmetric, or at the bridge section.

Chart 13 gives the coefficient Fsf as a function of Froude number for use with Equation 22.

4.5.3 Abutments and Spurs Adjustment Factor

Bridge abutments and spur protection measures cause a local acceleration in channel velocity and a locally higher shear stress on these embankments. The boundary shear stress adjustment factor due to contractions at abutments and spurs is given as:

$$Kas = F2\ (A + 2g^{*}\ Hd/Va^2) \qquad (23)$$

Where

F2 = component of the adjustment factor;

A = velocity-head correction coefficient;

Hd = total water-surface drop through the bridge, in feet; and,

Va = mean velocity of approach flow, in ft/sec.

Chart 11 provides values of F2, for the abutment and spur boundary shear stress adjustment factor, as a function of the drop in water surface through the bridge embankment.

4.5.4 Pier Adjustment Factor

The shear stress occurring in the vicinity of a bridge pier cannot be calculated directly. Based on potential flow theory, the velocity adjacent to a pier should increase to twice the velocity in the approach section. Therefore, the shear stress on the channel bed adjacent to the pier will be four times the shear on the channel bed in the approach reach. For piers, a constant adjustment factor of Kp = 4.0 is recommended.

4.5.5 Bend Adjustment Factor

A channel bend causes an increase in local boundary shear stress from the beginning of the bend for a significant distance downstream of the bend. The boundary shear stress adjustment factor at a channel bend is calculated by:

$$Kr = 3.16 * (Rc/TW)^{-0.5} \text{ for } Rc/TW < 10.0 \qquad (24a)$$

$$Kr = 1.0 \qquad \text{for } Rc/TW \geq 10.0 \qquad (24b)$$

Where

Rc = centerline radius of the bend, in feet; and,

TW = flow topwidth at upstream end of the bend, in feet.

Chart 14 provides values of the boundary shear stress adjustment factor for channel bend, Kr, as a function of the relative channel radius, Rc/TW.

The length of protection downstream of a bend is calculated using the following equation:

$$Lp = (0.604/nb)*da^{7/6} \tag{25}$$

Where

Lp = protection length, in feet;

da = average flow depth upstream of the bend, in feet; and,

nb = Manning's n-value in the bend.

Chart 15 provides values of the protection length downstream of a channel bend, Lp, as a function of bend roughness and average flow depth.

4.6 Step-by-Step Design Procedure

The objective of the hydraulic design procedure is to compute the design shear stress and the local boundary shear stress, and through a trial-an-error procedure, provide a stable riprap layer where the design shear stress exceeds the local boundary shear stress. The execution of the following steps assists the designer in gathering the necessary data to execute the procedure and in making the required calculations in the correct order.

DESIGN PROCEDURE

HYDRAULIC DESIGN OF RIPRAP

1. Compile the following information:

Average flow depth,	da = ______	ft
Average velocity,	Va = ______	ft/sec
Velocity-head coefficient,	A = ______	
Slope of the energy grade line,	Se = ______	ft/ft
Channel bottom width,	B = ______	ft
Hydraulic radius,	R = ______	ft
Aspect ratio,	B/da = ______	
Froude number	$Va/\sqrt{gda}$ = ______	
Bend radius,	Rc = ______	ft
Topwidth,	Tw = ______	ft
Bend ratio,	Rc/TW = ______	
Drop in water surface through the bridge opening,	Hd = ______	ft
Bridge abutment length,	L = ______	ft
Drop slope through bridge,	Hd/L = ______	ft/ft
Channel side-slope,	Z = ______	ft/ft
Specific gravity of riprap,	Ss = ______	
Unit weight of riprap,	Ss * 62.4 = ______	lb/ft^3
Safety factor,	SF = ______	

2. Select a riprap size, D50 = ______ ft

3. Calculate the mean boundary shear stress and the critical shear stress.

   3.1 Mean boundary shear stress, To = 62.4 * da * Se

   To = ______ lb/ft^2

   3.2 Critical shear stress, Tc = 4.1 * D50

   Tc = ______ lb/ft^2

4. Calculate the critical shear stress adjustment factor, Ca.

   4.1 Determine the side-slope factor, Cz, from Table 4.

   Cz = ______

   4.2 Determine the riprap unit weight factor, Cw, from Table 5.

   Cw = ______

4.3 Determine the angle of repose factor, Cr, from Table 6.

Cr = ________

4.4 Determine the oblique flow factor, Cl, using the following steps.

4.4.1 Calculate the shear stress ratio,

To/Tc = ________

4.4.2 Determine angle Al from Chart 4, Al = ________ degrees

4.4.3 Determine coefficient E from Chart 5,

E = ________

4.4.4 Determine the angle of particle movement, Ab, from Chart 6. Ab = ________ degrees

4.4.5 Determine the oblique flow factor, Cl, from Chart 7.

Cl = ________

4.5 Determine the particle movement factor, Cb, from Chart 8.

Cb = ________

4.6 Calculate the critical shear stress adjustment factor,

Ca = (1/SF)*(1/Cl)*Cz*Cw*(1 - Cr*Cb*SF) Ca = ________

5. Calculate the design shear stress, Td = Ca * Tc

Td = ________ lb/ft^2

6. Determine the boundary shear stress adjustment factor, Ka. Check the applicable local conditions.

____ Channel bed in a straight, symmetric reach.

____ Channel bed in a straight, non-symmetric reach.

____ Channel bed in a straight reach at a bridge section.

____ Channel side-slope in a straight reach.

____ Channel side-slope in a straight reach at high Froude Number.

____ Bridge abutment or spurs in a straight reach.

____ Bridge pier in a straight reach.

____ Channel bend.

Then determine the associated adjustment factor.

6.1 Determine the adjustment factor for the channel bed in a straight, symmetric reach, Kbs, using Chart 9.

Kbs = ________

6.2 Determine the adjustment factor for the channel bed in a straight, non-symmetric reach, Kbn, using Chart 10.

Kbn = ________

6.3 Calculate the adjustment factor for the channel bed at a bridge section in a straight reach, Kbc, where coefficient F1 is found using Chart 11.

$Kbc = F1*(A + 64.4*Hd/Va^2)$

Kbc = ________

6.4 Determine the adjustment factor for the channel side-slope in a straight reach, Kss, using Chart 12.

Kss = ________

6.5 Determine the adjustment factor for the channel side-slope in a straight reach at high Froude Number, Ksf, using Chart 13.

Ksf = ________

6.6 Calculate the adjustment factor for a bridge abutment or spur in a straight reach, Kas, where coefficient F2 is found using Chart 11.

$Kas = F2*(A + 64.4*Hd/Va^2)$

Kas = ________

6.7 The adjustment factor for a bridge pier in a straight reach, is:

Kp = 4.0

6.8 Determine adjustment factor for a channel bend, Kr, using Chart 14.

Kr = ________

Also determine the length of protection downstream of the bend, Lp, using Chart 15.

Lp = ________ ft

To find the boundary shear stress adjustment factor for locations described in Sections 6.1 to 6.7 that occur in a bend, multiply the straight reach adjustment factor by Kr.

7. Calculate the local boundary shear stress, Tb = Ka * To.

Tb = ________ lb/ft^2

Where

Ka = Kbs,Kbn,Kbc,Kss,Ksf,Kas,Kp,Kr or the product of Kr and another adjustment factor.

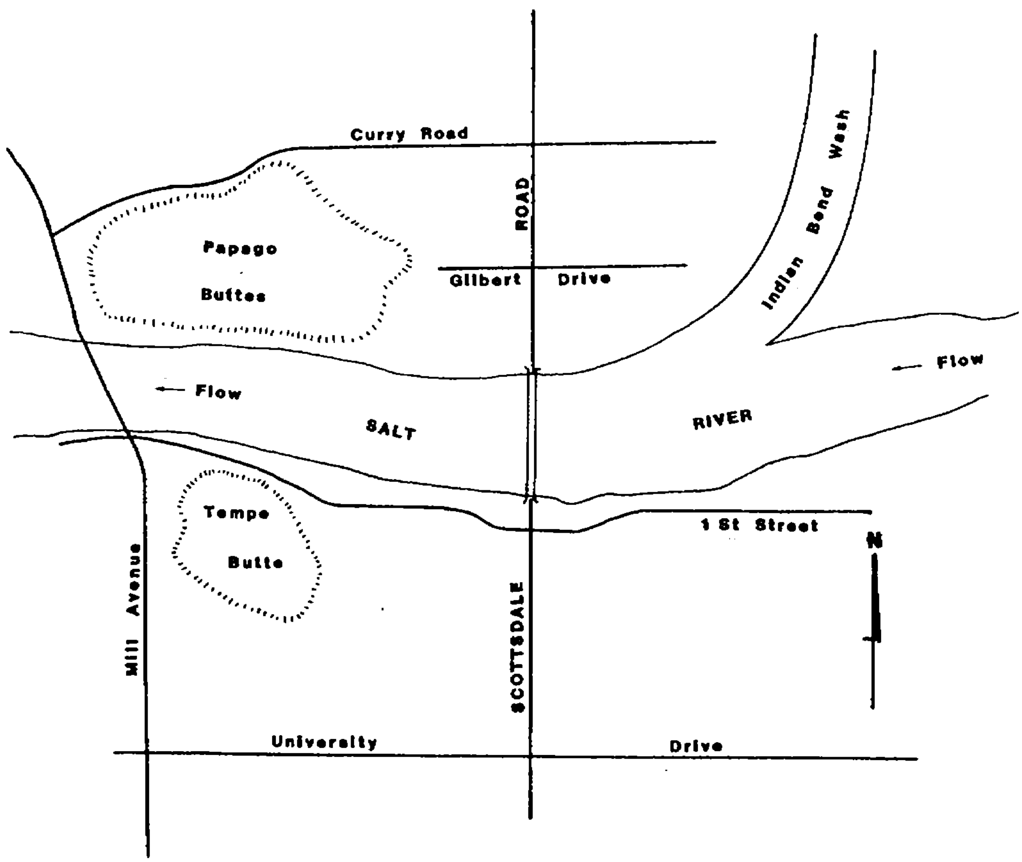
8. Compare the local boundary shear stress to the design shear stress. If Td >= Tb, then the design is acceptable. If not, then repreat Steps 2 through 8 varying the riprap size until an acceptable design is achieved.

4.7 Example Problems

Two comprehensive example problems are presented to illustrate the use of the hydraulic design procedure for riprap. The first example involves a bridge in a straight reach. The second example involves a bridge in a curved reach.

Example 1

This example evaluates a single riprap size for use in bank and bed protection at a bridge site in a straight river reach. The bridge is located on the Salt River at Scottsdale Road in Tempe, Arizona. Hydraulic information about the site is summarized in Step 1. The subsequent calculations show the acceptable locations for an 18" riprap size at this site.



DESIGN PROCEDURE
HYDRAULIC DESIGN OF RIPRAP

1. Compile the following information:

SCOTTSDALE ROAD BRIDGE - SALT RIVER

Average flow depth,	da = 20.6	ft
Average velocity,	Va = 7.65	ft/sec
Velocity-head coefficient,	A = 1.1	
Slope of the energy grade line,	Se = 0.001161	ft/ft
Channel bottom width,	B = 1050	ft
Hydraulic radius,	R = 20	ft
Aspect ratio,	B/da = 51	
Froude number	$Va/\sqrt{gda}$ = 0.297	
Bend radius,	Rc = NA	ft
Topwidth,	Tw = NA	ft
Bend ratio,	Rc/TW = NA	
Drop in water surface through the bridge opening	Hd = 0.14	ft
Bridge abutment length,	L = 115	ft
Drop slope through bridge,	Hd/L = 0.001217	ft/ft
Channel side-slope,	Z = 3	ft/ft
Specific gravity of riprap,	Ss = 2.4	
Unit weight of riprap,	Ss * 62.4 = 150	lb/ft^3
Safety factor,	SF = 1.6	

2. Select a riprap size, D50 = 1.5 ft

3. Calculate the mean boundary shear stress and the critical shear stress.

3.1 Mean boundary shear stress, To = 62.4 * da * Se

To = 1.49 lb/ft^2

3.2 Critical shear stress, Tc = 4.1 * D50

Tc = 6.15 lb/ft^2

4. Calculate the critical shear stress adjustment factor, Ca.

4.1 Determine the side-slope factor, Cz, from Table 4.

Cz = 0.95

4.2 Determine the riprap unit weight factor, Cw, from Table 5.

Cw = 1.0

4.3 Determine the angle of repose factor, Cr, from Table 6.

Cr = 0.36

4.4 Determine the oblique flow factor, Cl, using the following steps.

4.4.1 Calculate the shear stress ratio,

To/Tc = 0.242

4.4.2 Determine angle Al from Chart 4, Al = 2 degrees

4.4.3 Determine coefficient E from Chart 5,

E = 2.825

4.4.4 Determine the angle of particle movement, Ab, from Chart 6. Ab = 19.26 degrees

4.4.5 Determine the oblique flow factor, Cl, from Chart 7.

Cl = 0.68

4.5 Determine the particle movement factor, Cb, from Chart 8.

Cb = 0.94

4.6 Calculate the critical shear stress adjustment factor,

Ca = (1/SF)*(1/Cl)*Cz*Cw*(1 - Cr*Cb*SF) Ca = 0.40

5. Calculate the design shear stress, Td = Ca * Tc

Td = 2.45 lb/ft^2

For the channel bed Ca = 0.63

Td (bed) = 3.84

6. Determine the boundary shear stress adjustment factor, Ka. Check the applicable local conditions.

X Channel bed in a straight, symmetric reach.

____ Channel bed in a straight, non-symmetric reach.

X Channel bed in a straight reach at a bridge section.

X Channel side-slope in a straight reach.

X Channel side-slope in a straight reach at high Froude Number.

X Bridge abutment or spurs in a straight reach.

X Bridge pier in a straight reach.

____ Channel bend.

Then determine the associated adjustment factor.

6.1 Determine the adjustment factor for the channel bed in a straight, symmetric reach, Kbs, using Chart 9.

Kbs = 1.06

6.2 Determine the adjustment factor for the channel bed in a straight, non-symmetric reach, Kbn, using Chart 10.

Kbn = NA

6.3 Calculate the adjustment factor for the channel bed at a bridge section in a straight reach, Kbc, where coefficient F1 is found using Chart 11. $Kbc = F1*(A + 64.4*Hd/Va^2)$

Kbc = 1.25

6.4 Determine the adjustment factor for the channel side-slope in a straight reach, Kss, using Chart 12. Kss = 0.90

6.5 Determine the adjustment factor for the channel side-slope in a straight reach at high Froude Number, Ksf, using Chart 13.

Ksf = 1.0

6.6 Calculate the adjustment factor for a bridge abutment or spur in a straight reach, Kas, where coefficient F2 is found using Chart 11.

$Kas = F2*(A + 64.4*Hd/Va^2)$

Kas = 1.25

6.7 The adjustment factor for a bridge pier in a straight reach, is:

Kp = 4.0

6.8 Determine adjustment factor for a channel bend, Kr, using Chart 14.

Kr = NA

Also determine the length of protection downstream of the bend, Lp, using Chart 15. Lp = NA ft

To find the boundary shear stress adjustment factor for locations described in Sections 6.1 to 6.7 that occur in a bend, multiply the straight reach adjustment factor by Kr.

7. Calculate the local boundary shear stress, Tb = Ka * To.

$Tb = \underline{\text{see table}}\ lb/ft^2$

Where

Ka = Kbs,Kbn,Kbc,Kss,Ksf,Kas,Kp,Kr or the product of Kr and another adjustment factor.

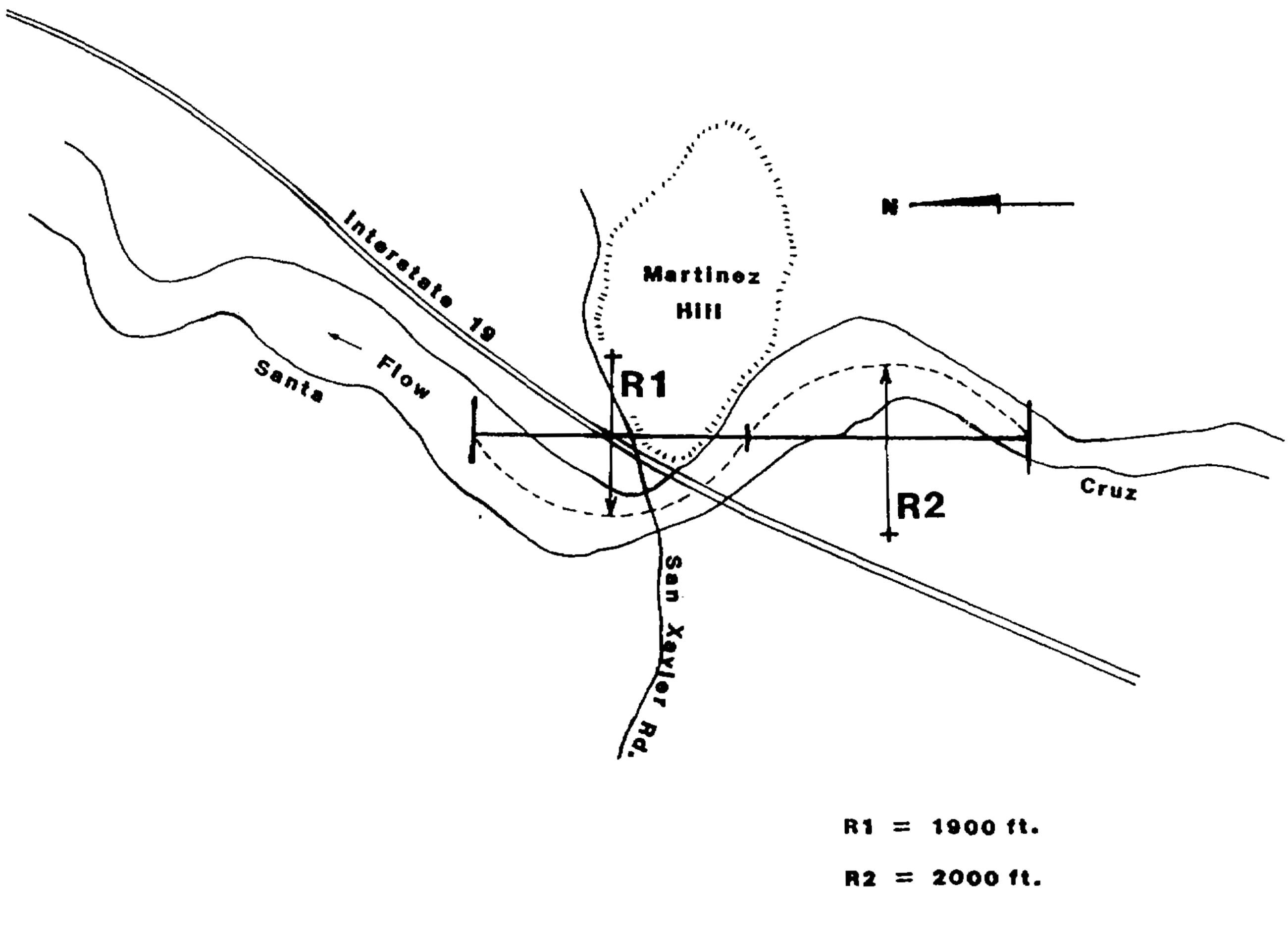
8. Compare the local boundary shear stress to the design shear stress. If Td >= Tb, then the design is acceptable. If not, then repreat Steps 2 through 8 varying the riprap size until an acceptable design is achieved. See below.

Location	τ_b (lb/ft^2)
Channel bed	1.58
Channel bed at bridge section	1.86
Channel side-slope	1.34
Channel side-slope with high Froude No.	1.49
Bridge abutment	1.86
Bridge pier	5.96 ← Not OK

The riprap design size of 18" is acceptable at all locations, except the bridge piers.

Example 2

This example evaluates the performance of a 36" riprap size at various locations at a bridge site on the Santa Cruz River at I-19 near Tucson, Arizona. Hydraulic information about the site is summarized in Step 1. The results of subsequent calculations show some of the difficulties in the use of riprap at bridges in reaches with severe bends.



DESIGN PROCEDURE
HYDRAULIC DESIGN OF RIPRAP

1. Compile the following information:

SANTA CRUZ RIVER AT I-19

Average flow depth,	da = 12.0 ft
Average velocity,	Va = 11.7 ft/sec
Velocity-head coefficient,	A = 1.3
Slope of the energy grade line,	Se = 0.0043 ft/ft
Channel bottom width,	B = 250 ft
Hydraulic radius,	R = 12.0 ft
Aspect ratio,	B/da = 20.8
Froude number	$Va/\sqrt{gda}$ = 0.60
Bend radius,	Rc = 2000 ft
Topwidth,	Tw = 500 ft
Bend ratio,	Rc/TW = 4.0
Drop in water surface through the bridge opening	Hd = 1.5 ft
Bridge abutment length,	L = 190 ft
Drop slope through bridge,	Hd/L = 0.008 ft/ft
Channel side-slope,	Z = 2.5 ft/ft
Specific gravity of riprap,	Ss = 2.4
Unit weight of riprap,	Ss * 62.4 = 150 lb/ft^3
Safety factor,	SF = 1.6

2. Select a riprap size, D50 = 3.0 ft

3. Calculate the mean boundary shear stress and the critical shear stress.

   3.1 Mean boundary shear stress, To = 62.4 * da * Se

   To = 3.22 lb/ft^2

   3.2 Critical shear stress, Tc = 4.1 * D50

   Tc = 12.30 lb/ft^2

4. Calculate the critical shear stress adjustment factor, Ca.

   4.1 Determine the side-slope factor, Cz, from Table 4.

   Cz = 0.92

   4.2 Determine the riprap unit weight factor, Cw, from Table 5.

   Cw = 1.0

4.3 Determine the angle of repose factor, Cr, from Table 6.

Cr = 0.46

4.4 Determine the oblique flow factor, Cl, using the following steps.

4.4.1 Calculate the shear stress ratio,

To/Tc = 0.262

4.4.2 Determine angle Al from Chart 4, Al = 8 degrees

4.4.3 Determine coefficient E from Chart 5,

E = 3.23

4.4.4 Determine the angle of particle movement, Ab, from Chart 6. Ab = 16.4 degrees

4.4.5 Determine the oblique flow factor, Cl, from Chart 7.

Cl = 0.71

4.5 Determine the particle movement factor, Cb, from Chart 8.

Cb = 0.96

4.6 Calculate the critical shear stress adjustment factor,

Ca = (1/SF)*(1/Cl)*Cz*Cw*(1 - Cr*Cb*SF) Ca = 0.238

5. Calculate the design shear stress, Td = Ca * Tc

Td = 2.93 lb/ft^2

For the channel bed Ca = 0.63

Ta (bed) = 7.69 lb/ft^2

6. Determine the boundary shear stress adjustment factor, Ka. Check the applicable local conditions.

X Channel bed in a straight, symmetric reach.

___ Channel bed in a straight, non-symmetric reach.

X Channel bed in a straight reach at a bridge section.

X Channel side-slope in a straight reach.

X Channel side-slope in a straight reach at high Froude Number.

X Bridge abutment or spurs in a straight reach.

X Bridge pier in a straight reach.

X Channel bend.

Then determine the associated adjustment factor.

6.1 Determine the adjustment factor for the channel bed in a straight, symmetric reach, Kbs, using Chart 9.

Kbs = 1.12

6.2 Determine the adjustment factor for the channel bed in a straight, non-symmetric reach, Kbn, using Chart 10.

Kbn = NA

6.3 Calculate the adjustment factor for the channel bed at a bridge section in a straight reach, Kbc, where coefficient F1 is found using Chart 11.

$Kbc = F1*(A + 64.4*Hd/Va^2)$

Kbc = 1.85

6.4 Determine the adjustment factor for the channel side-slope in a straight reach, Kss, using Chart 12.

Kss = 0.92

6.5 Determine the adjustment factor for the channel side-slope in a straight reach at high Froude Number, Ksf, using Chart 13.

Ksf = 1.125

6.6 Calculate the adjustment factor for a bridge abutment or spur in a straight reach, Kas, where coefficient F2 is found using Chart 11.

$Kas = F2*(A + 64.4*Hd/Va^2)$

Kas = 1.60

6.7 The adjustment factor for a bridge pier in a straight reach, is:

Kp = 4.0

6.8 Determine adjustment factor for a channel bend, Kr, using Chart 14.

Kr = 1.58

Also determine the length of protection downstream of the bend, Lp, using Chart 15.

Lp = 315 ft

To find the boundary shear stress adjustment factor for locations described in Sections 6.1 to 6.7 that occur in a bend, multiply the straight reach adjustment factor by Kr.

7. Calculate the local boundary shear stress, Tb = Ka * To.

Tb = see table lb/ft^2

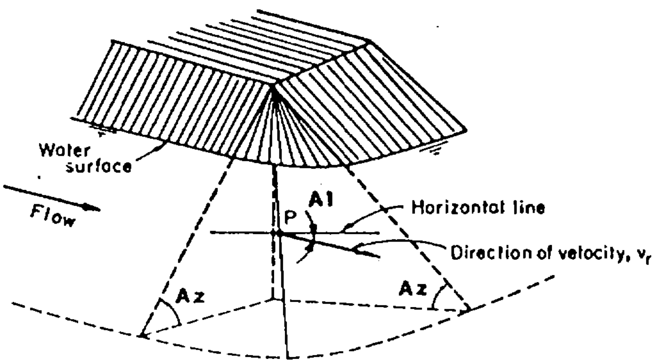
Where

Ka = Kbs,Kbn,Kbc,Kss,Ksf,Kas,Kp,Kr or the product of Kr and another adjustment factor.

8. Compare the local boundary shear stress to the design shear stress. If Td >= Tb, then the design is acceptable. If not, then repreat Steps 2 through 8 varying the riprap size until an acceptable design is achieved. See below.

Location	τ_b (lb/ft^2)
Channel bed in bend	5.70
Channel bed at bridge section in bend	9.4
Channel side-slope in bend	4.70
Channel side-slope, high Froude No., in bend	5.7
Bridge abutment in bend	8.2
Bridge pier in bend	20.4

The 3.0 foot riprap size is acceptable only in the first case. At all other locations, problems occur.



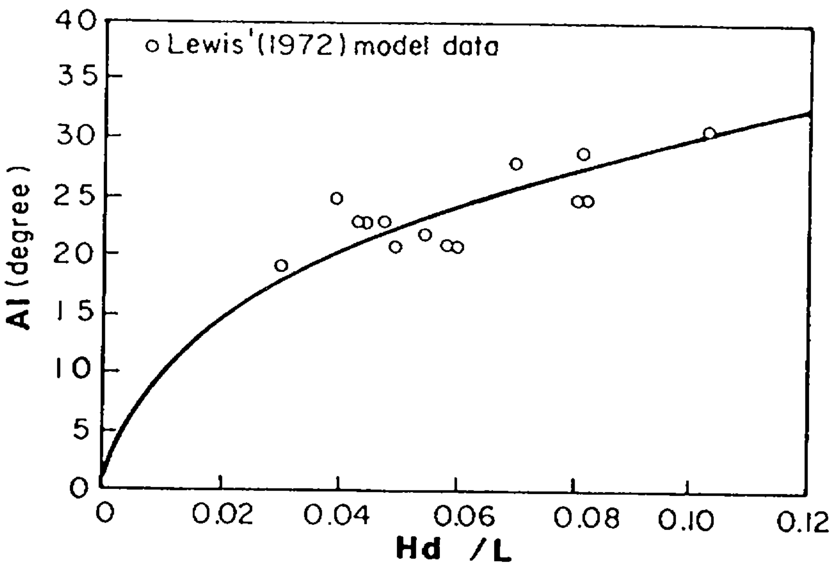
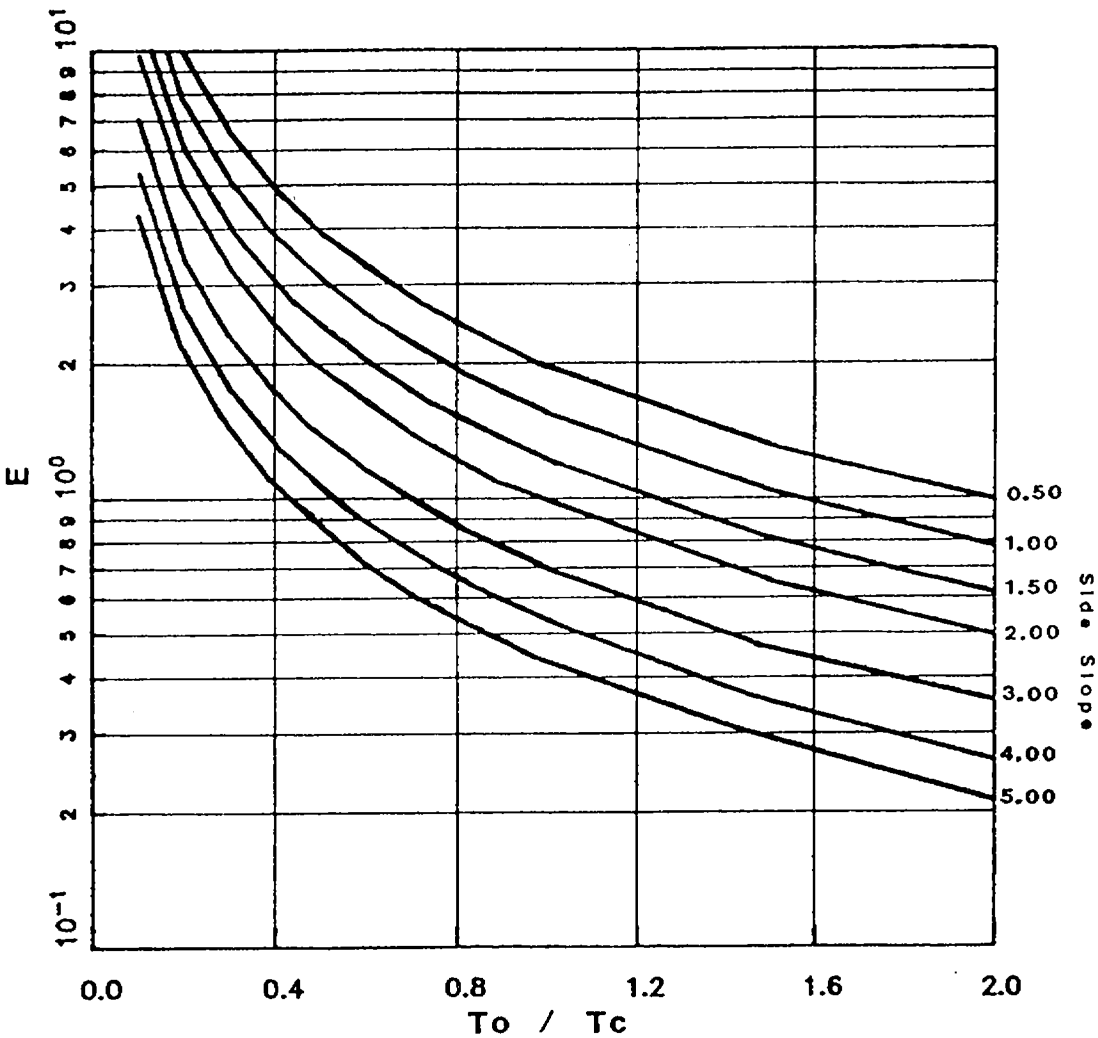


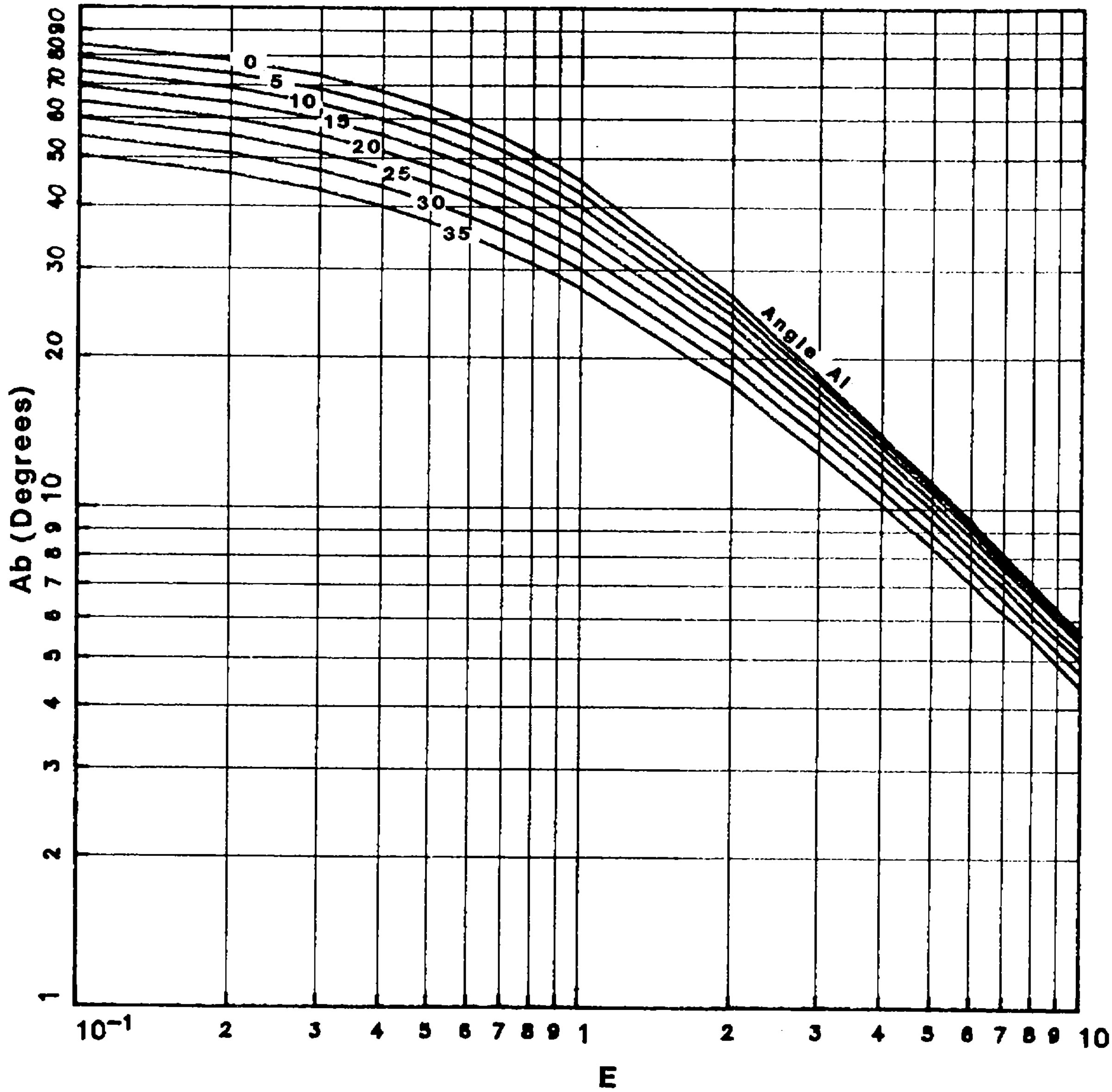
Chart 4. Oblique Flow Angle, Al, at Bridge Embankment
(Highways in the River Environment, Appendix 6)



$$E = 2 * (Tc/To)\ Cz\ Cr$$

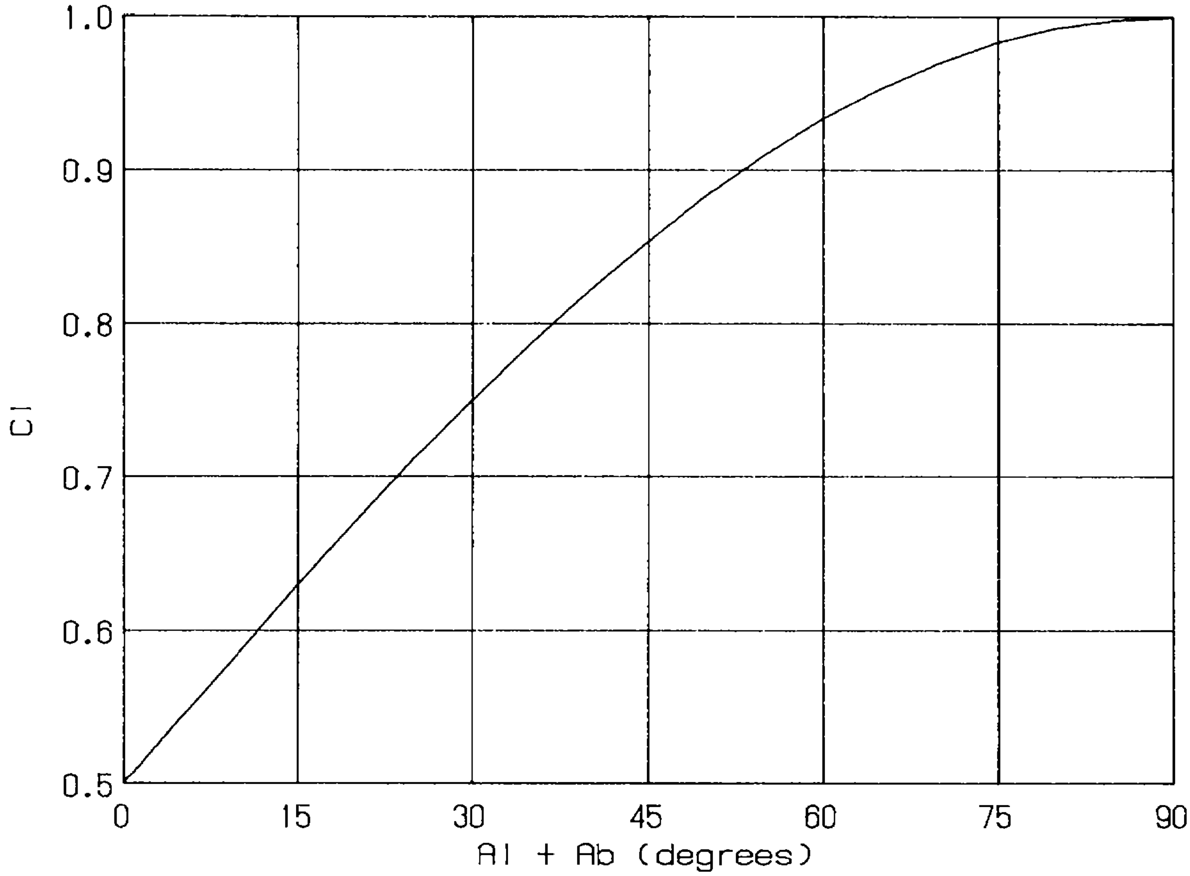
Chart 5. Coefficient E for Equation 14

CHART 6



$$Ab = \tan^{-1} (\cos(Al)/(E + \sin(Al)))$$

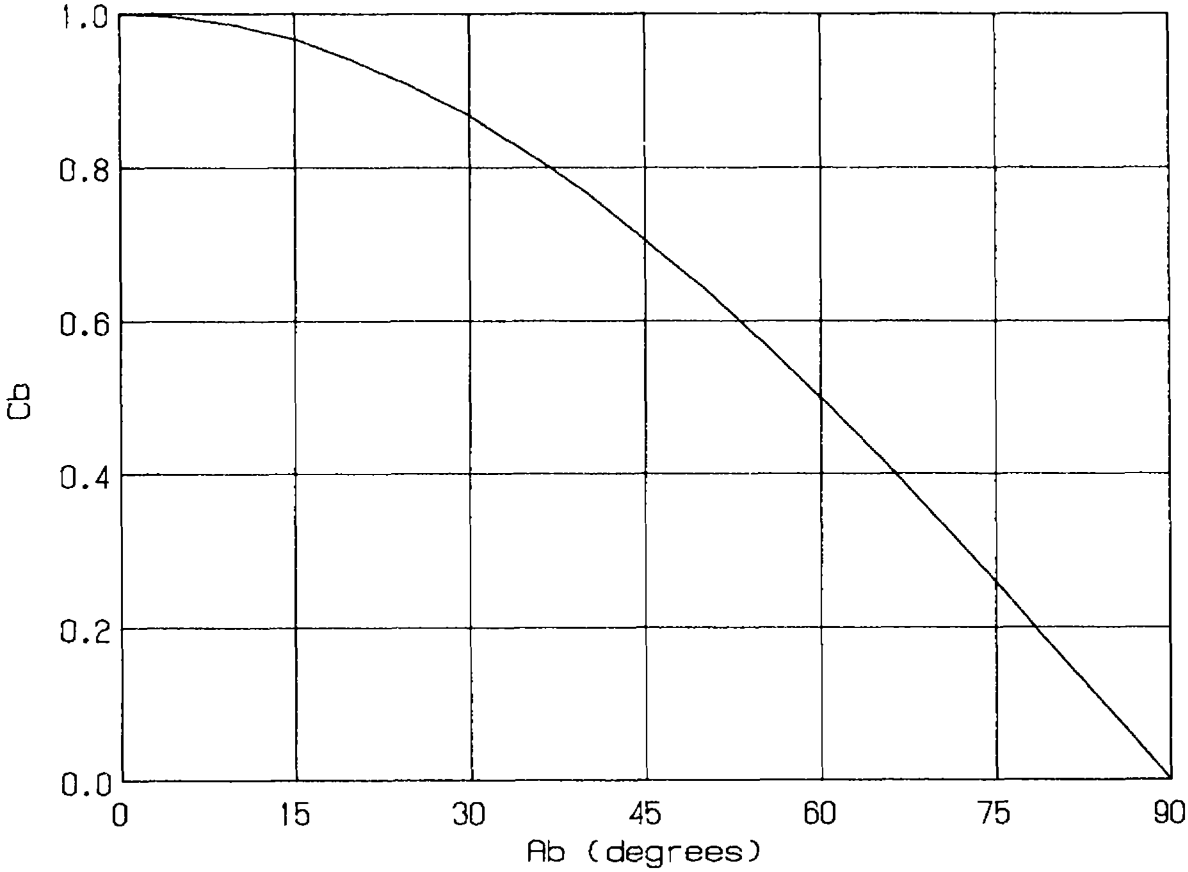
Chart 6. Angle of Particle Movement, Ab



$$Cl = (\tfrac{1}{2})\ (1 + \sin(Al + Ab))$$

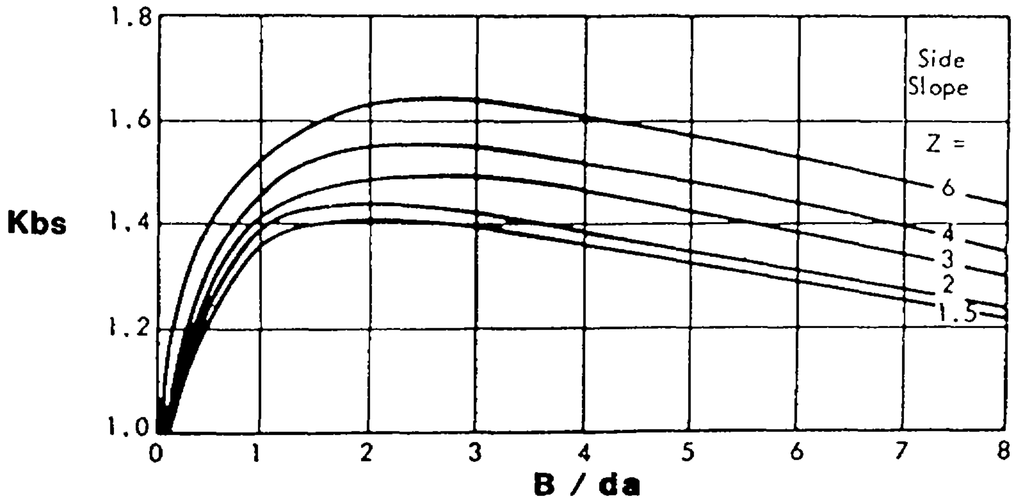
Chart 7. Correction Factor Cl

CHART 8



$$Cb = \cos(Ab)$$

Chart 8. Correction Factor Cb

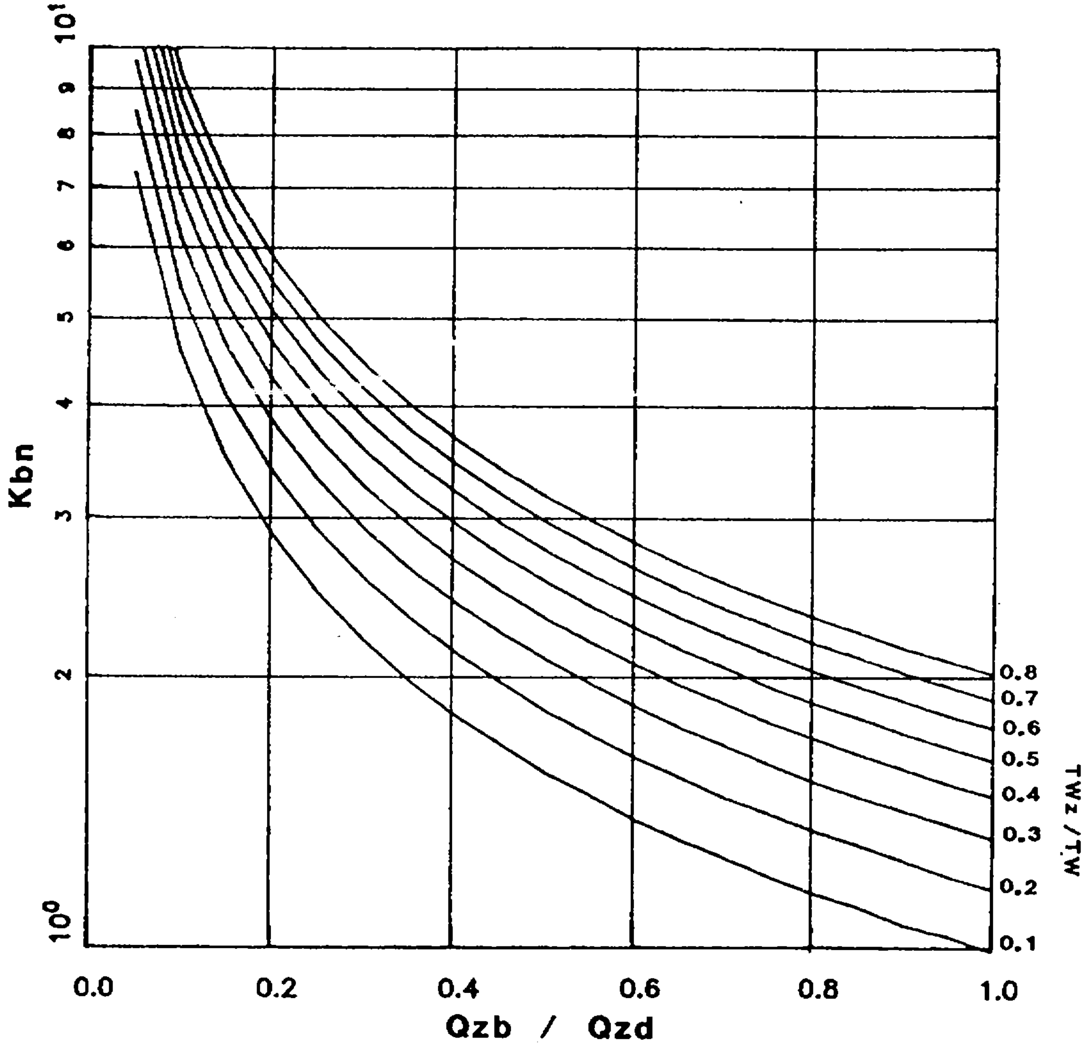


$$K_{bs} = \frac{\frac{B}{da} + 2\sqrt{Z^2 + 1}}{\frac{B}{da} + Z}$$

for $\frac{B}{da} > 8$

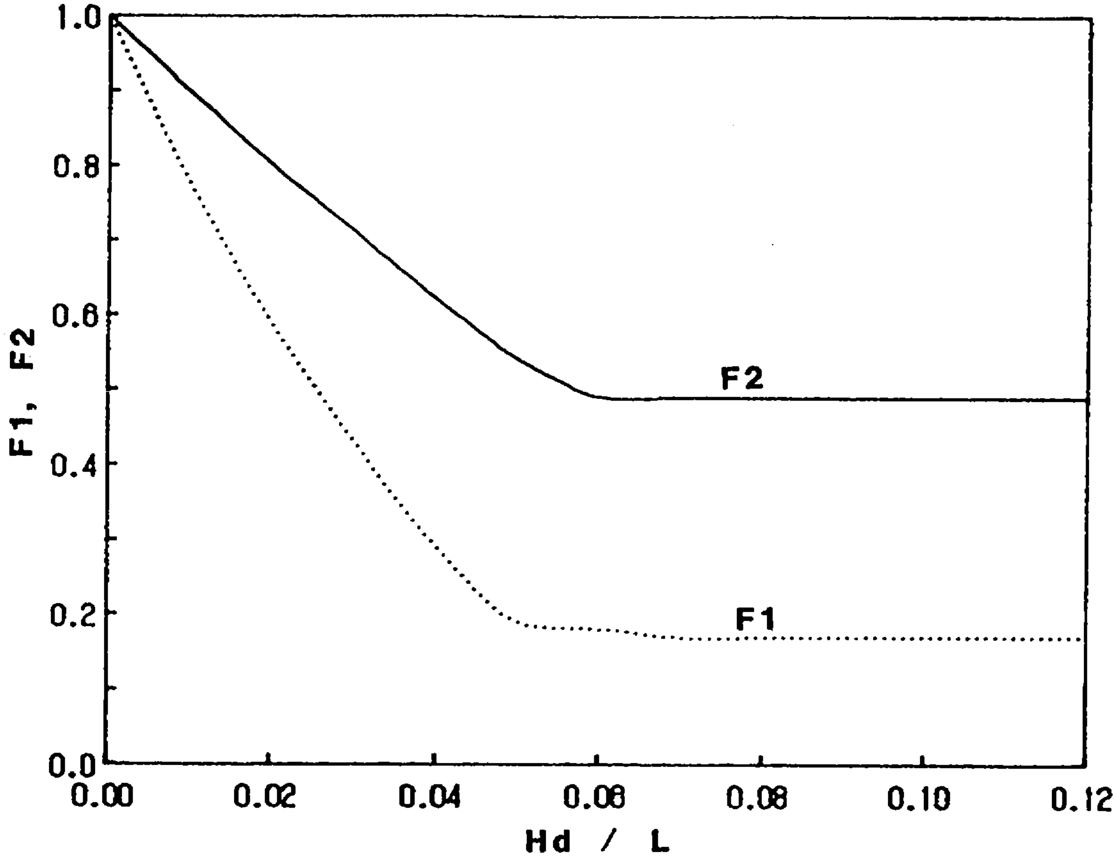
Chart 9. Boundary Shear Stress Adjustment Factor for Channel Bottom for a Symmetric Channel (Report 108, Tentative Design Procedure for Riprap Lined Channels)

CHART 10



$$Kbn = ((Qzd/Qzb)*(0.722+2.67(TWz/Tw)))^{0.664}$$

Chart 10. Boundary Shear Stress Adjustment Factor for the Channel Bottom in the Zone of Main Channel Flow for a Non-Symmetric Reach (Adapted from Brown & Blodgett, 1987)



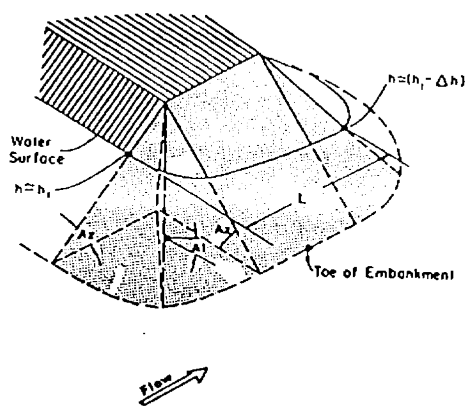
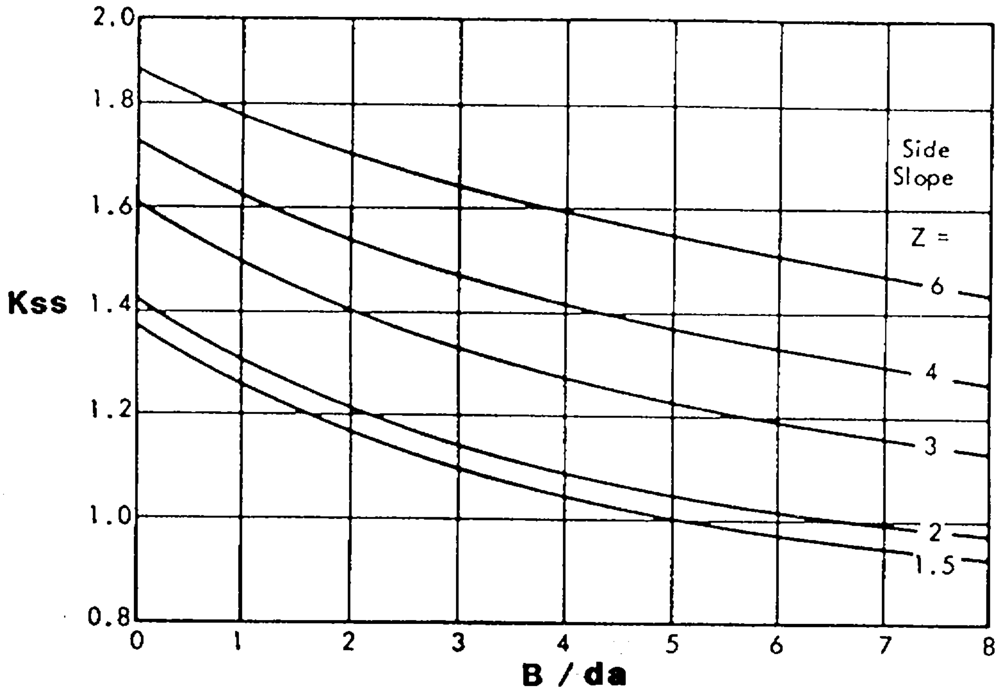


Chart 11. Coefficients F1 and F2 for Channel
Bed and Abutment at a Bridge
(Adapted from Highways in the River Environment,
Appendix 6)

CHART 12



$$K_{ss} = C\left[\frac{\frac{B}{da} + 2\sqrt{Z^2+1}}{\frac{B}{da} + Z}\right]$$

for $\frac{B}{da} > 8$

Where

Z	C
1.5	0.76
2	0.785
3	0.85
4	0.935
6	0.97

Chart 12. Boundary Shear Stress Adjustment Factor for a Channel Side-Slope (Report 108, Tentative Design Procedure for Riprap-Lined Channels)

CHART 13

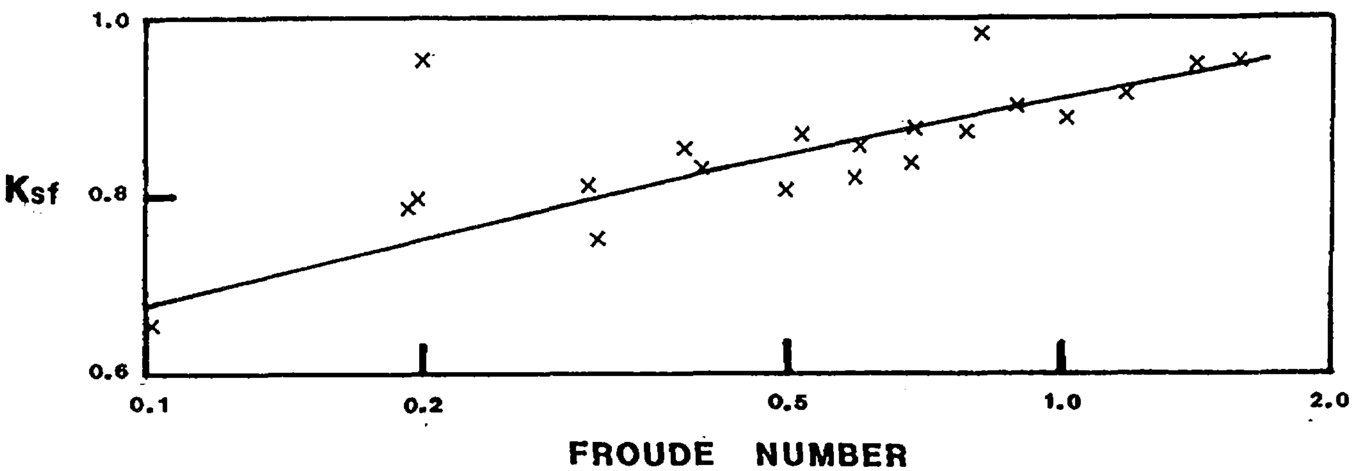
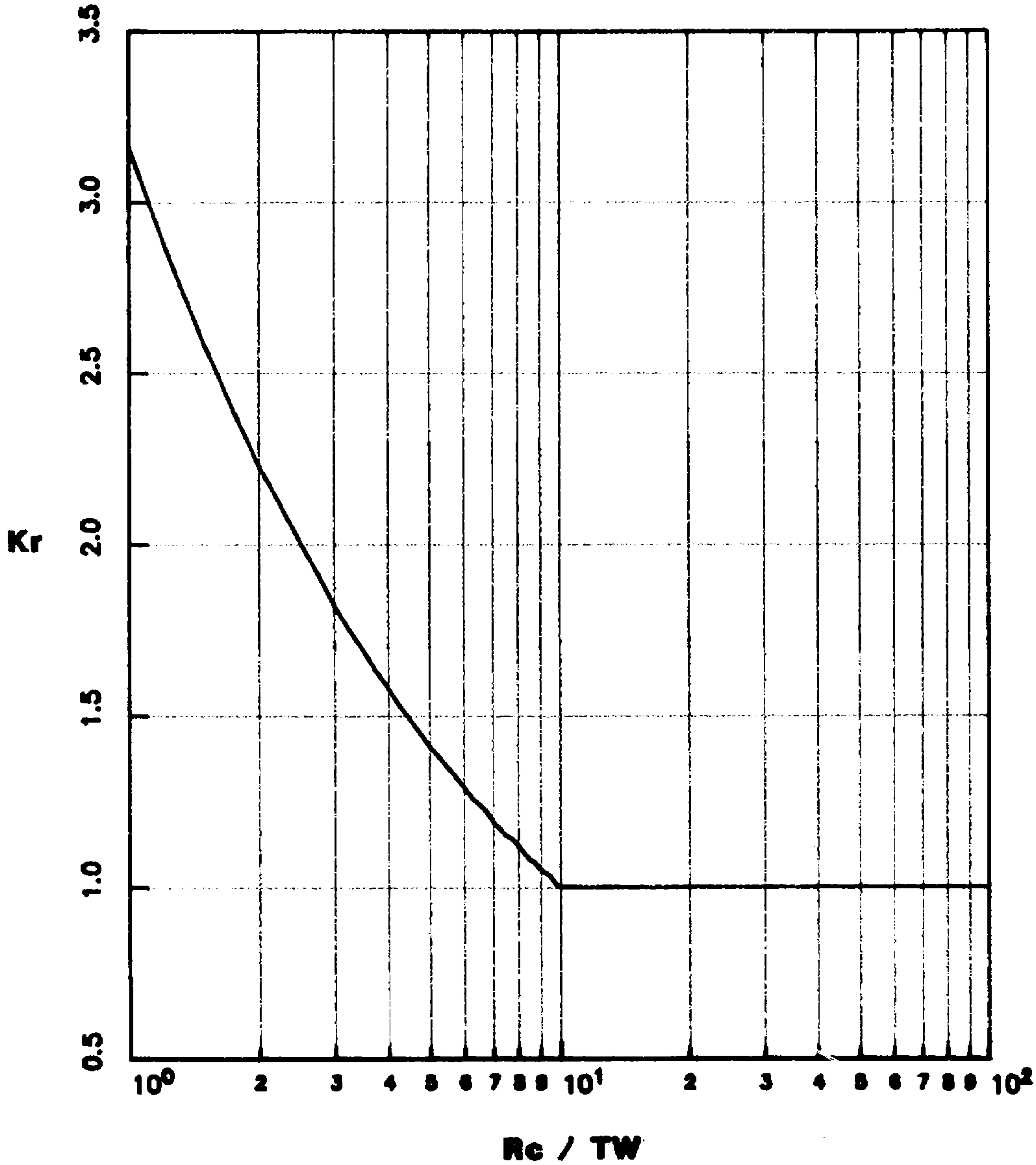


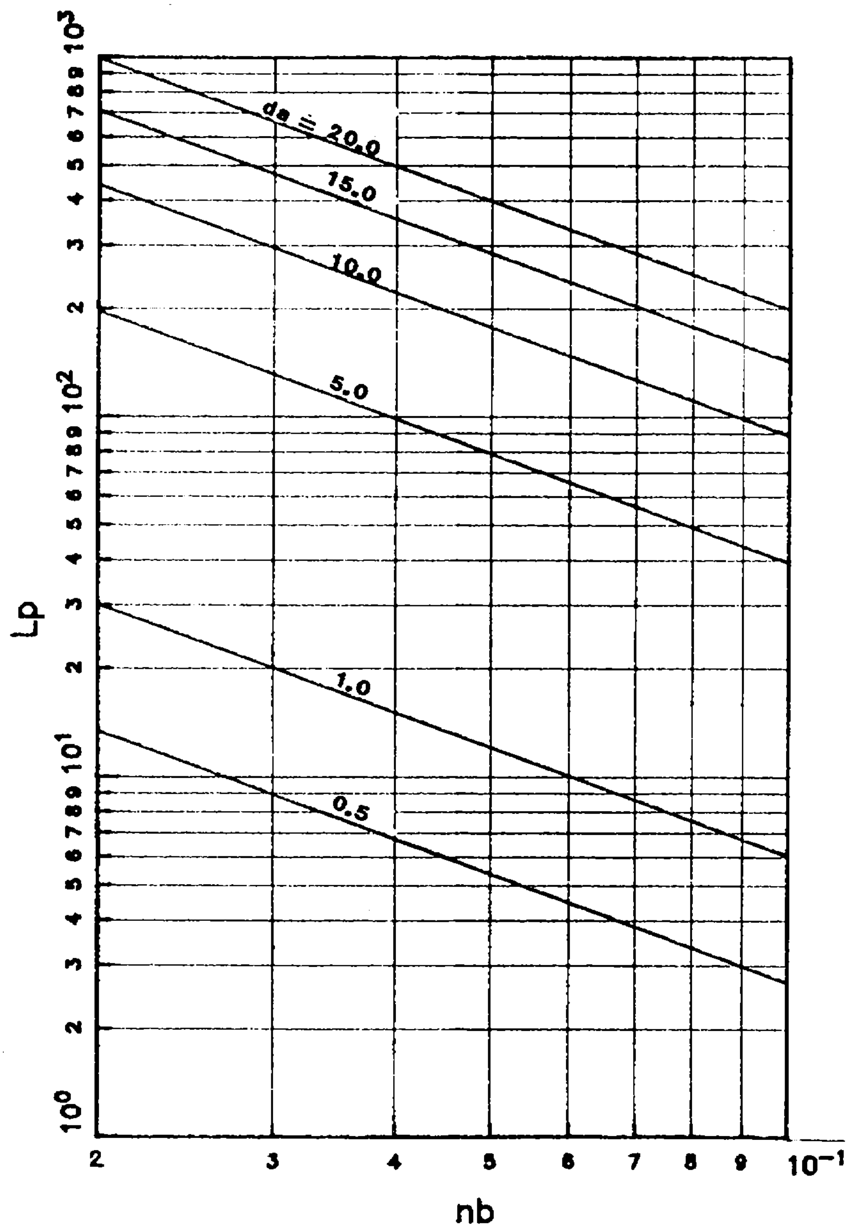
Chart 13. Coefficient for Effect of Froude Number on Boundary Shear Stress Adjustment Factor on Channel Side-Slope (Distribution of Shear in Rectangular Channels, Davidian, Cahal)



$Kr = 3.16 * (Rc/TW)^{-0.5}$ for $Rc/TW < 10.0$
$Kr = 1.0$ for $Rc/TW \geq 10.0$

Chart 14. Boundary Shear Stress Adjustment Factor, Kr, for a Channel Bend (EM1601 - Hydraulic Design of Flood Control Channels)

CHART 15



$$Lp = (0.604/nb)*da^{7/6}$$

Chart 15. Protection Length Downstream of a Channel Bend
(Revised HEC-15)

V. SITE CONDITIONS

The design objective for protection measures at a bridge site on alluvial channels is to provide, over the design life of the bridge, assurance that the desired alignment, cross-section, and profile are maintained. This is a difficult task, since the behavior of an alluvial channel over a 25- to 50-year period is uncertain. To reduce the uncertainty and avoid potential damage to the bridge, protection and control measures are incorporated into the channel reach at the bridge site. The various types of protection and control measures form a controlled reach of channel. Identification of the control reach limits is an important design requirement. Once the control reach is identified, then protection and control measures can be designed.

5.1 The Control Reach

Identification of the control reach involves an evaluation of historic changes in the channel form in the vicinity of the bridge site. A sequence of aerial photographs is one of the best means of observing and assessing changes in channel conditions at a bridge site. As a general rule, the control reach should extend several times the non-constricted channel width upstream and downstream of the bridge site. The control reach should be carefully inspected, and areas of bank erosion and scour documented. An assessment of trends in channel erosion should be made. Consultation with engineers experienced in alluvial channel response is recommended, since the assessments made regarding the channel response depend on engineering judgement.

Within the control reach, various structures are used to maintain efficient flow conditions at the bridge waterway, including: bank protection, approach embankments, spurs, guidebanks, and grade-control sills. Bank protection is used to stabilize existing channel banks or to retard lateral movement or channel widening in the control reach. The primary function of an approach embankment is to reduce the bridge length over a channel. In doing so, however, the embankment redirects the flow. Spurs are used to direct flow away from a channel bank or to direct the flow to the bridge opening. Guidebanks are set to the flow and provide a transition from a wider floodway to the bridge opening. A grade-control sill maintains the channel profile elevation, and is normally used in channels that have a history of degradation.

5.2 Riprap Requirements for Control Structures

Each control structure can incorporate riprap as a means of protecting the embankment surface. In the previous section of the design procedure, the determination of hydraulic conditions and riprap stability requirements at control structures was presented. This section presents methods for determining the location of riprap on a control structure, and the associated depth of riprap below the existing channel invert.

Approach Embankments. The approach embankment creates a zone of increased shear stress near the nead of the embankment as the flow accelerates through the bridge opening. Figure 2 shows the location on the embankment that requires increased protection. This zone extends one-third the length of the encroachment distance. The shear stress reaches a maximum as the flow passes beneath the bridge. The toe of the embankment protection should be placed below the depth of scour anticipated at the embankment. The scour estimate should include the effects of general degradation, degradation due to the bridge contraction, bed form movement, and local scour. Refer to Arizona Department of Water Resources, "Design Manual for Engineering Analysis of Fluvial Systems", by Simons, Li & Associates, Inc., for procedures to be used when estimating scour at a bridge abutment.

Spur Control Structure. Spurs can be used singularly, or as a group to reduce flow near the channel banks. At a bridge site, they are typically used to maintain the alignment of the main flow through the bridge opening (see Figure 3). For a single spur, the zone of maximum shear stress is assumed to be identical to that of an approach embankment. Toe-down requirements for riprap protection on spurs is identical to that for approach embankments.

In general, a spur at 90 degrees to the bank and perpendicular to the flow direction is the most economical for bank protection. Local scour at the spur head is reduced if the spur is angled in the downstream direction.

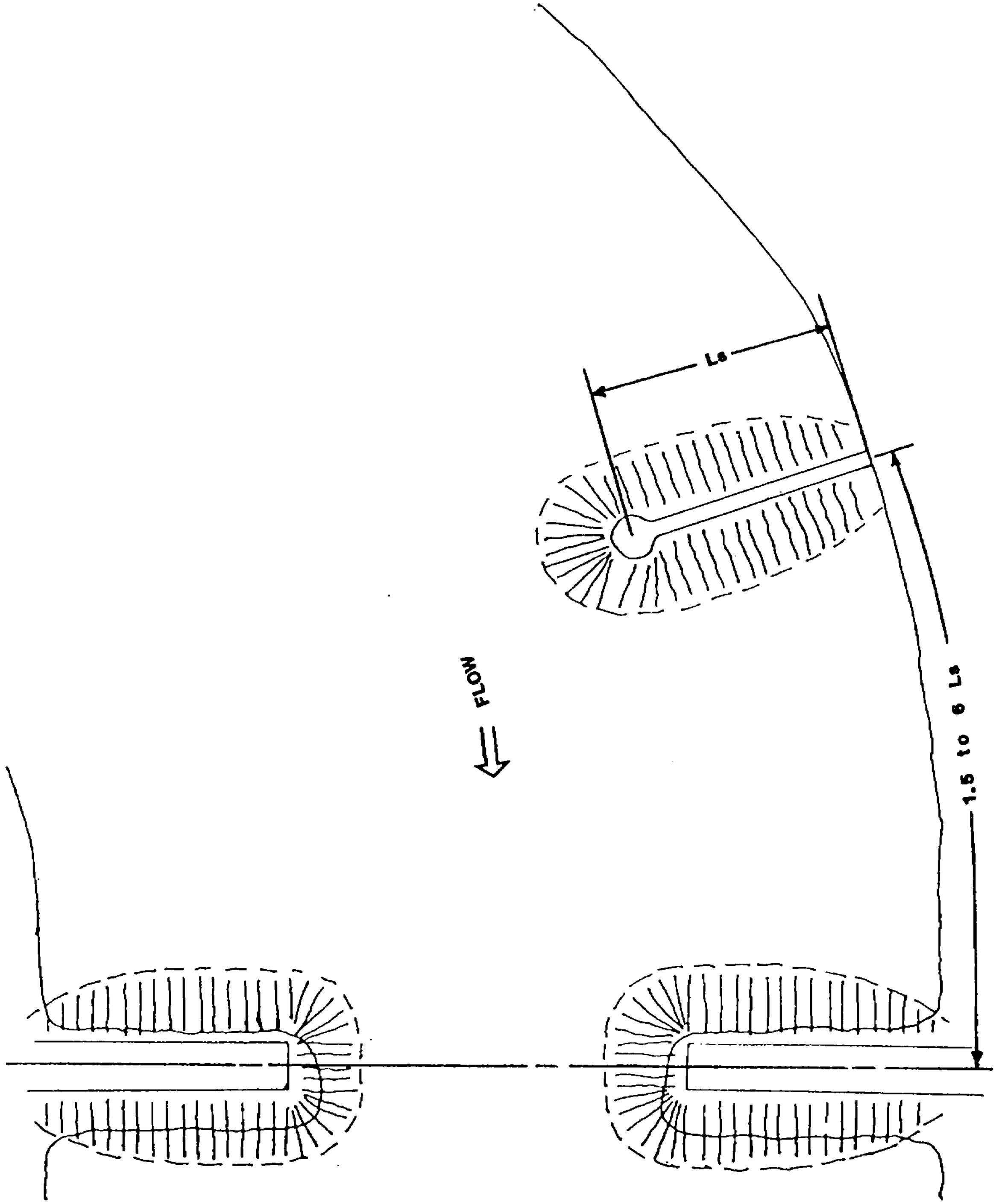


Figure 3. Spur Control Structure

However, the resulting reduction in protection at the spur head is offset by the increase in spur length.

Spacing between spurs is primarily a function of the upstream projected length of the spur into the flow, Ls. Spacing distances of 1.5 to 2.0 Ls is recommended to obtain a well-defined, deeper channel as an approach reach to the bridge. For protecting banks only, a longer spacing is used, 2.0 to 6.0 Ls. In braided channels, straight reaches, or long radius bends, the spacing may be from 4.0 to 6.0 Ls. In meandering channels, the spur spacing should not exceed one-half the meander wave length of the stream, as defined in Figure 4. For high velocity conditions, the spur spacing should be reduced to 3.0 to 3.0 Ls.

<u>Guidebank Control Structure.</u> Guidebanks are placed at the upstream end of the bridge to guide the stream through the bridge opening (see Figure 5). This control structure is provided when flow disturbances such as eddies and cross flow are anticipated at the bridge site. These conditions can cause severe local scour at the bridge abutment, and reduce the efficiency of the bridge opening. Properly constructed guidebanks can eliminate these effects.

The recommended shape of a guidebank is a quarter ellipse, with a major to minor axis ratio of 2.5. The major axis is set parallel to the main flow direction, regardless of the skew of the bridge to the channel. The spur length is a function of both the flow in the floodplain which is diverted by the approach embankment and the width of the bridge opening. Refer to the Federal Highway Administration Publication, Hydraulic Design Series No. 1, "Hydraulics of Bridge Waterways", by J.N. Bradley, for the complete design procedure.

<u>Bank Protection.</u> Bank protection is typically required in the control reach to prevent lateral movement of the channel banks or to stabilize the roadway embankment. Lateral movement of a channel bank is typically associated with a channel bend. Attention should be given to both the increased shear stress on the bank protection and the increased scour that tends to occur in the bend.

Case studies of bridge crossings in Arizona show that channel bends often occur within the control reach, and the design of riprap protection has

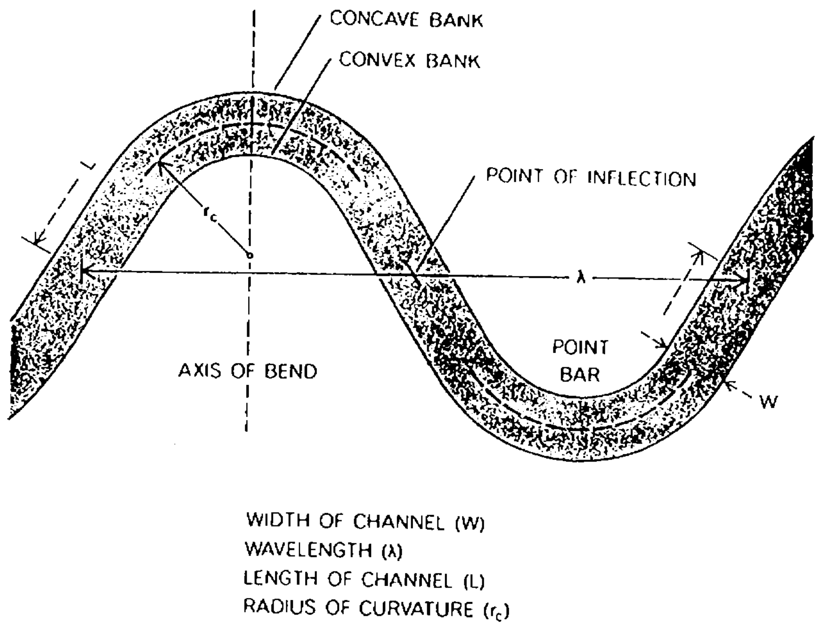


Figure 4. River Meander Parameters

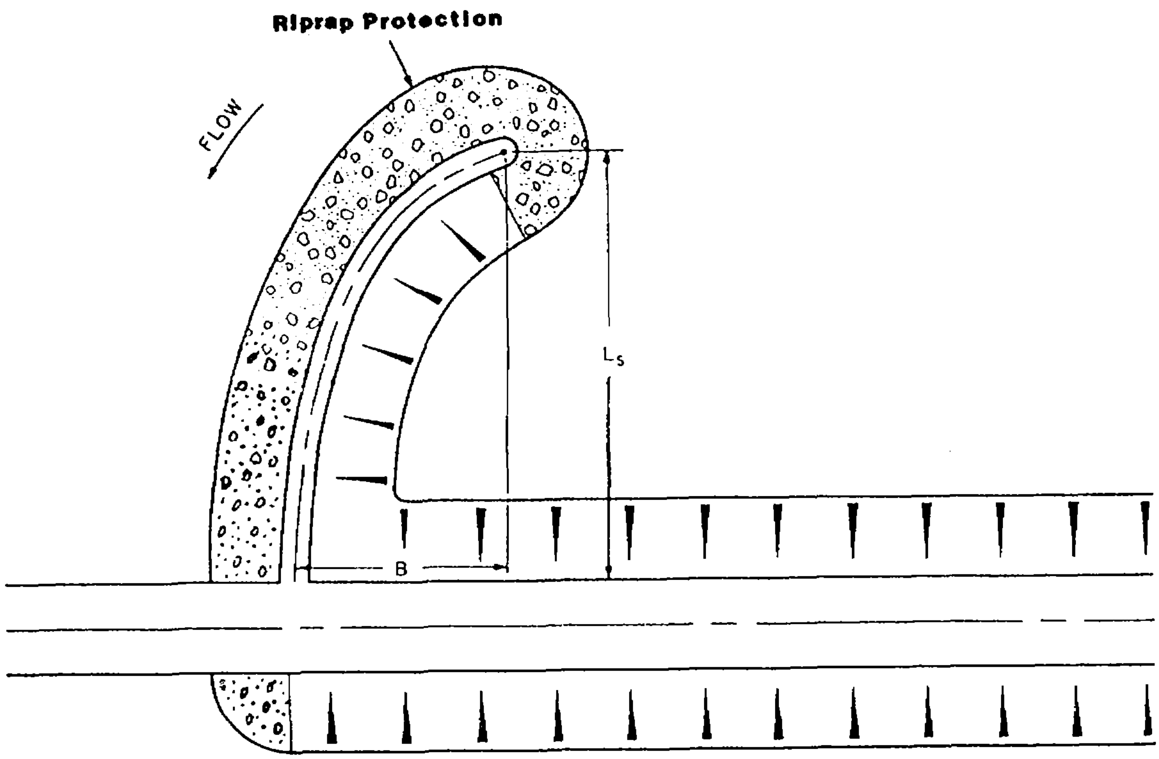


Figure 5. Plan View of Guidebank

not always adequately accounted for their presence. Reasonably conservative estimates of bend formation, based on the history of the channel response, should be made when developing design for bank protection. Refer to Arizona Department of Water Resources, "Design Manual for Engineering Analysis of Fluvial Systems", by Simons, Li & Associates, Inc., for a procedure to estimate scour in a channel bend.

At the termination points, the bank protection should be set back into the existing bank of the channel. The end of the bank protection should extend approximately three to five times the height of the bank protection into the channel bank. Bank protection should not be terminated within a channel bend, unless some other structure is provided to deflect or guide the flow through the bend.

Grade-Control Sills. Channel degradation is a common problem at bridge sites in Arizona. Often the process of degradation extends to the river system, in general, and cannot be specifically associated with activity in the control reach. The size of the drop at individual sills ranges from about three feet to over ten feet in height. Because of the size of drop required at grade-control sills, a structure composed of loose riprap is not feasible. Analysis procedures for design of grade-control sills are limited, particularly in regard to the estimation of scour below a sill. Design of stilling basins for large discharges are covered in the Bureau of Reclamation's Engineering Monograph No. 25, "Hydraulic Design of Stilling Basins and Energy Dissipators", by A.J. Paterka. The design of small stilling basins is covered in the Federal Highway Administration Hydraulic Engineering Circular No. 14, "Hydraulic Design of Energy Dissipators for Culverts and Channels".

Piers. Riprap protection around bridge piers should be considered as a remedial measure only, and never considered as a design feature of a pier. Given the complex flow conditions at a bridge site, coupled with the possibility of significant changes in flow conditions at a bridge site over time, it is not considered prudent to attempt a pier design that plans on preventing local scour at the pier. If conditions have dteriorated at a bridge site and corrective action is required, then riprap protection at the pier can be considered. In most cases, riprap protection is not the only corrective action that will be required. In a degrading channel, the channel bed will often need to be stabilized using a grade-control sill.

Figure 6 shows the installation of riprap pier protection. The riprap is placed on a gravel filter blanket below the depth of combined general scour and bed-form movement. Use of filter fabric is not recommended, since the flow direction and pressure gradients near a pier are not parallel to the channel bed. The riprap is installed in a circle around the pier equal to three times the pier diameter. The thickness of the riprap blanket should equal to 1.5 times the largest size in the riprap gradation. If debris accumulation is anticipated at the pier, then this accumulation should be added to the pier diameter.

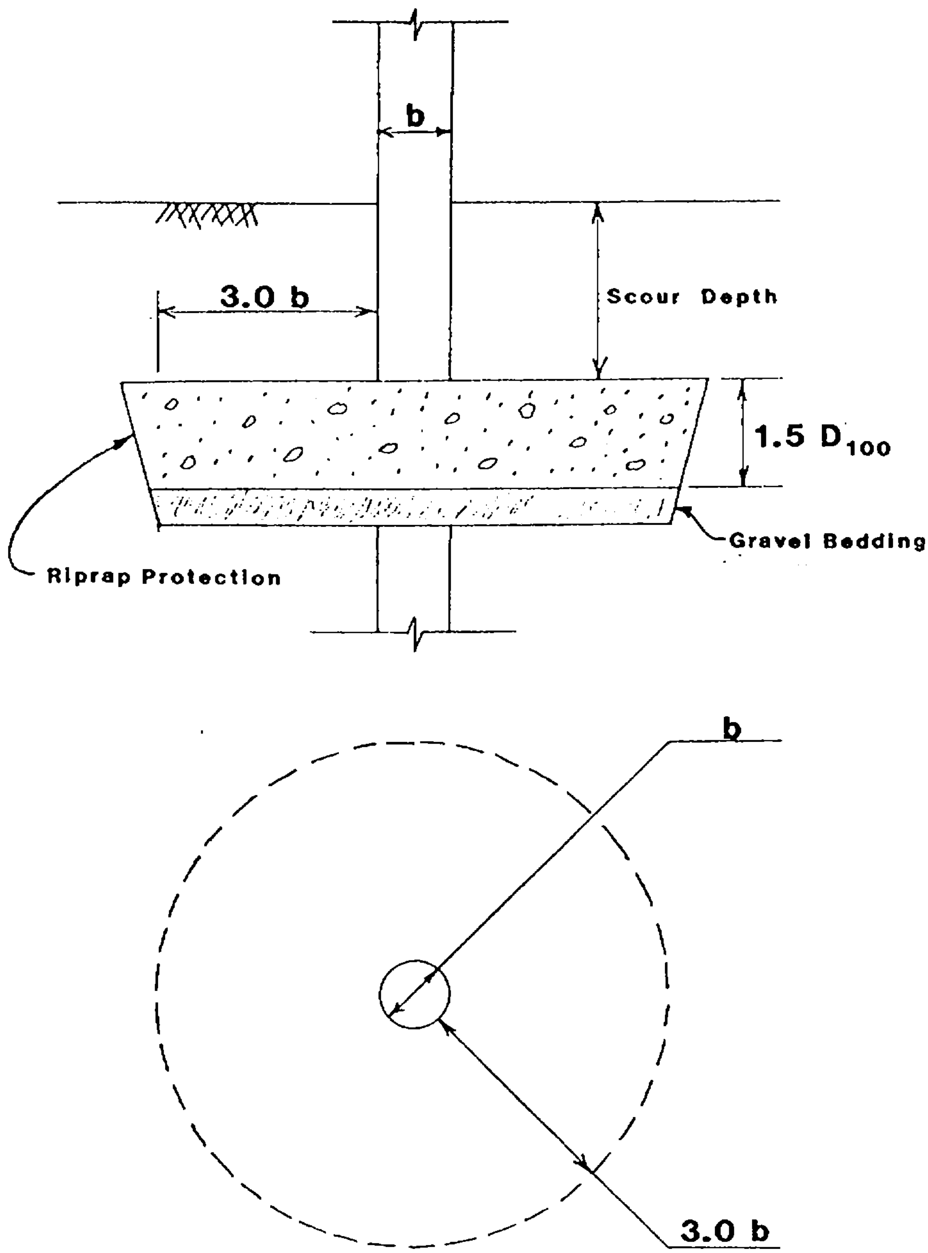


Figure 6. Riprap Pier Protection

VI. RIVER CONDITIONS

The alluvial rivers that characterize the major drainage basins in Arizona are continually changing their local features. Many of these changes are the natural response of the channel to hydrologic conditions in the basin, expecially major floods. Periods of scour and aggradation, channel narrowing and abrupt widening, and changes in stream coarse are documented occurrences for most major Arizona rivers. Man's use of the land has had a profound effect on river response as well. The clearing of floodplains for agriculture, the mining of sand and gravel resources from the stream bed and floodplain, the construction of dams and flood-control works, and general watershed urbanization are among some of the man-made changes that have affected river response. In general, both man-induced changes and natural events affect a large portion of the river basin, and cannot normally be assessed at a single location.

Unfortunately, there is no readily available source of information on regional changes in river conditions. This makes the task of addressing these changes over the design life of a bridge structure difficult. In the absence of good data, the designer is encouraged to be cautious in assessing regional changes. In areas where a more complete understanding of the regional changes in stream conditions has been compiled, the designer can more reliably determine scour conditions at a bridge site.

In evaluating regional conditions, the designer should be aware of all available information on the river system in which he is working. He should check with local flood-control authorities, the Arizona Department of Water Resources, and federal agencies, such as the U.S. Geological Survey, the U.S. Army Corps of Engineers, and the Federal Emergency Management Agency for the reports and data which they have compiled on a particular river system. Maintenance records, bridge inspection reports, and previous design projects in the files of the Arizona Department of Transportation are also a valuable record of river-system conditions. In addition, the following check list can assist the designer in compiling river-response data in conjunction with a bridge design. For structures on smaller streams, this information gathering should still be conducted.

REGIONAL RIVER INFORMATION CHECK LIST

Existing Drainage Structures:

1. Note maintenance problems and repair measures.
2. Note performance problems during floods.

Flood Performance:

1. For the most recent large-magnitude flood (estimate 10-year flood magnitude or greater), note changes in the following channel characteristics:
   a. channel width;
   b. bank height;
   c. sinuosity;
   d. bank vegetation;
   e. number of low-flow channels;
   f. frequency and type of bars and islands;
   g. frequency of cut banks; and,
   h. bed profile.

2. Note the location of the following types of channel scour:
   a. bank erosion;
   b. head cuts; and,
   c. bend erosion.

Existing Stream Development:

1. Note the location and date of construction of dams or water-diversion structures.
2. Note the location and date of construction of flood-control structures.
3. Note the location size, and production rates of in-stream or flood-plain sand and gravel mining operations.

Anticipated Stream Development:

1. Planned flood-control structures.
2. Lease holdings by sand and gravel operators.

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California Test 229
1978

APPENDIX A

METHOD OF TEST FOR DURABILITY INDEX

A. SCOPE

The durability index test provides a measure of the relative resistance of an aggregate to producing clay-sized fines when subjected to prescribed methods of interparticle abrasion in the presence of water. Four procedures are provided for use with materials with various nominal sizes and specific gravities.

Procedure	*Designation*	*Type of Material*	*Section*
A	Dc	Retained No. 4 sieve	E-1
B........................	Dc "modified"	Lightweight or porous, retained No. 4 sieve	E-2
C	Df	Passing No. 4 sieve	E-3
D	Df "modified"	No. 4 x No. 16 sieve (pea gravel, chips)	E-4

B. APPARATUS.

The following equipment is required to perform this test. Detailed descriptions and specifications are included as necessary to assure standardization. Items bearing an Office of Business Management (OBM) catalog number are available to California State Agencies from the Department of Transportation, Office of Business Management. Detailed plans are available for those items bearing a Transportation Laboratory (TL) drawing number.

1. Agitator (Figure 1). A mechanical device designed to hold the wash vessel in an upright position while subjecting it to a lateral reciprocating motion at a rate of 285 ± 10 complete cycles per minute. The reciprocating motion shall be produced by means of an eccentric located in the base of the carrier and the length of the stroke shall be 1.75 ± .025 inches. The clearance between the cam and follower of the eccentric shall be .001 to .004 inches.

The combination sieve shaker-agitator, OBM catalog number 6635-0940-6, meets these requirements when in the agitation mode.

The Tyler portable sieve shaker meets these requirements when modified according to TL drawing number D-536.

2. Mechanical Sand Equivalent Shaker (Figure 2)

a. A mechanical device designed to hold a graduated plastic cylinder in a horizontal position while subjecting it to a reciprocating motion parallel to its length. The motion shall provide a stroke length of 8 ± 0.04 inches. The device shall operate at a speed of 175 ± 2 complete cycles per minute. Prior to use, the shaker shall be fastened securely to a firm and level mount.

b. OBM catalog number 6635-0930-5.

c. TL drawing number D-256.

3. Sand Equivalent Test Apparatus (Figure 3)

a. A graduated plastic cylinder, rubber stopper, irrigator tube, weighted foot assembly and siphon assembly, all conforming to the specifications and dimensions shown in TL drawing number C-218 (Figure 4).

A one gallon minimum size glass or plastic container with cover and fitted with the siphon assembly or a discharge tube near the bottom shall be used to disperse the working calcium chloride solution. The container shall be placed on a shelf or suspended above the work area in such a manner that the level of the solution is maintained between 36 and 46 inches above the work surface.

b. OBM catalog number 6635-0610-7.

4. Measuring Tin. A 3 ounce tin approximately 2¼ inches in diameter having a capacity of 85 ± 5 ml.

5. Wash Vessel. A flat bottomed, straight sided cylindrical vessel equipped with a watertight removable lid and conforming to the dimensions and tolerances shown in Figure 4.

The "Stainless Steel Pot", OBM catalog number 7330-0130-1, meets these requirements.

6. Collection Pot. A round pan or container having vertical or nearly vertical sides and equipped as necessary to hold the wire mesh of an 8-inch diameter sieve at least 3 inches above the bottom. An adaptor which will not allow loss of fines or wash water may be used to nest the sieve with the container, or the sieve may be nested with a blank sieve frame resting in the bottom of the pan.

7. Graduated Cylinder. A graduated cylinder having a capacity of 1000 mls.

8. Rubber Stopper. A stopper to fit the plastic cylinder.

9. Funnels

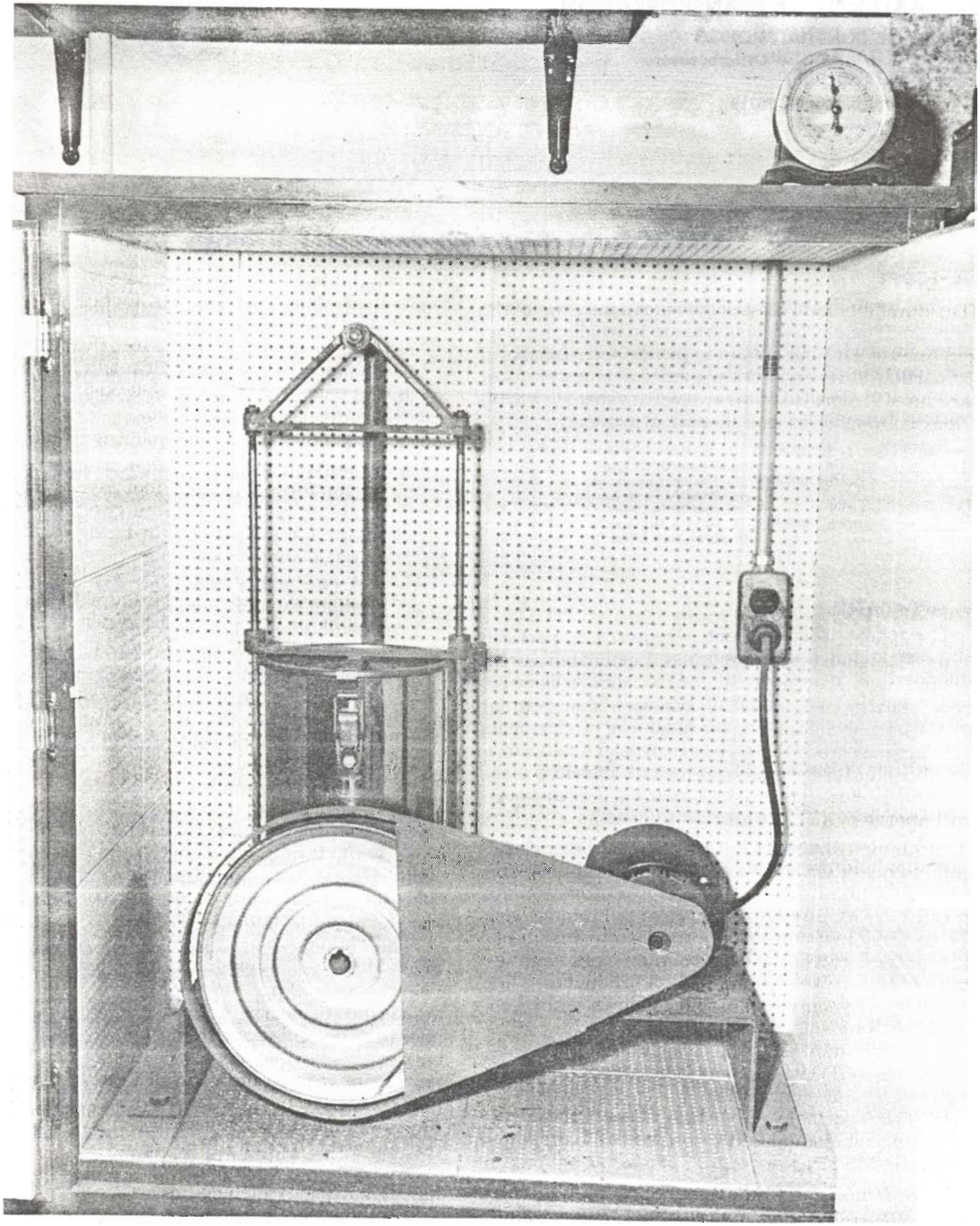

FIGURE 1

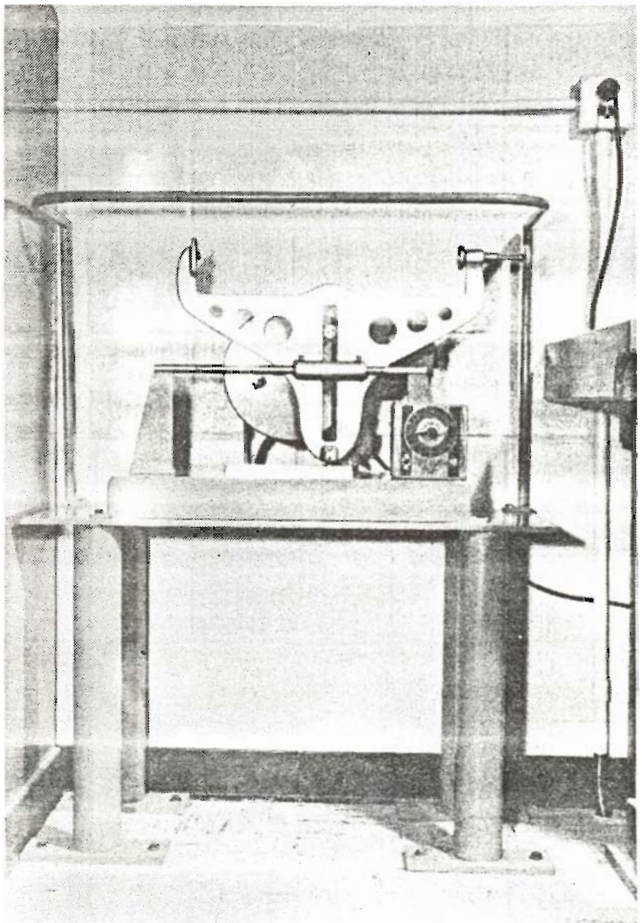

FIGURE 2

a. A wide mouth funnel suitable for directing water or aggregate into the plastic cylinder.

b. A wide mouth funnel large enough to hold an 8-inch diameter sieve while directing water into the plastic cylinder.

10. Balance. A balance or scale accurate to 0.2 percent of the weight of the sample to be tested.

11. Oven. A drying oven set to operate at 230° ± 9°F (110° ± 5°C).

12. Timer. A clock or watch reading in minutes and seconds.

13. Sieves. U.S. Standard Sieves, ¾ inch (19.0 mm), ½ inch (12.5 mm), ⅜ inch (9.5 mm), No. 4 (4.75 mm), No. 8 (2.36 mm) and No. 200 (0.075 mm). The No. 8 and No. 200 sieves shall be in standard 8-inch diameter frames.

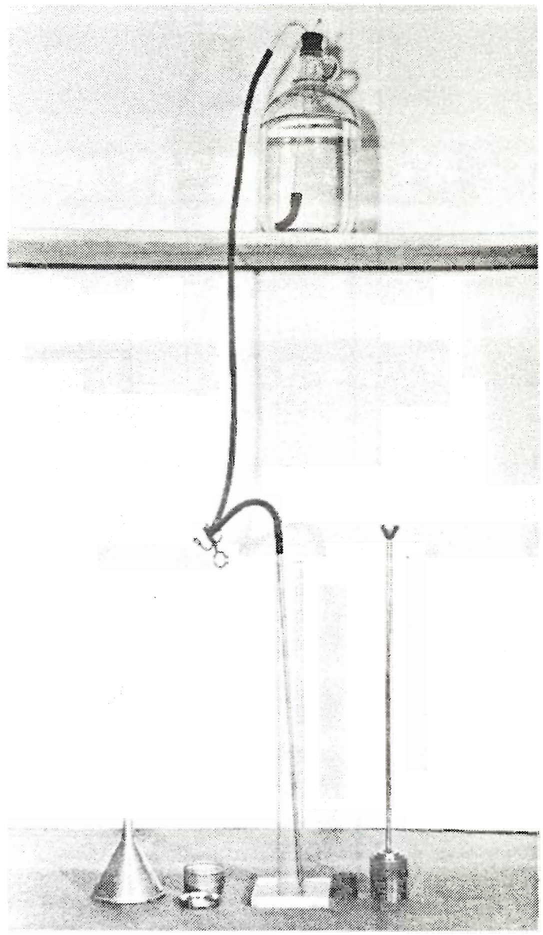

FIGURE 3

14. Flexible Hose

C. MATERIALS

1. Stock Calcium Chloride Solution

a. "Sand Equivalent Stock Solution", OBM catalog number 6810-0100-6.

b. Solution may be prepared from the following:
454 g (1 lb) tech. anhydrous calcium chloride.
2,050 g (1,640 ml) USP glycerine.
47 g (45 ml) formaldehyde (40 percent by volume) solution.

Dissolve the calcium chloride in ½ gal of distilled or demineralized water. Cool the solution to room temperature, then filter it through Whatman No. 12 or equivalent filter paper. Add the glycerine and formaldehyde to the filtered solution, mix well, and dilute to 1 gal with distilled or demineralized water.

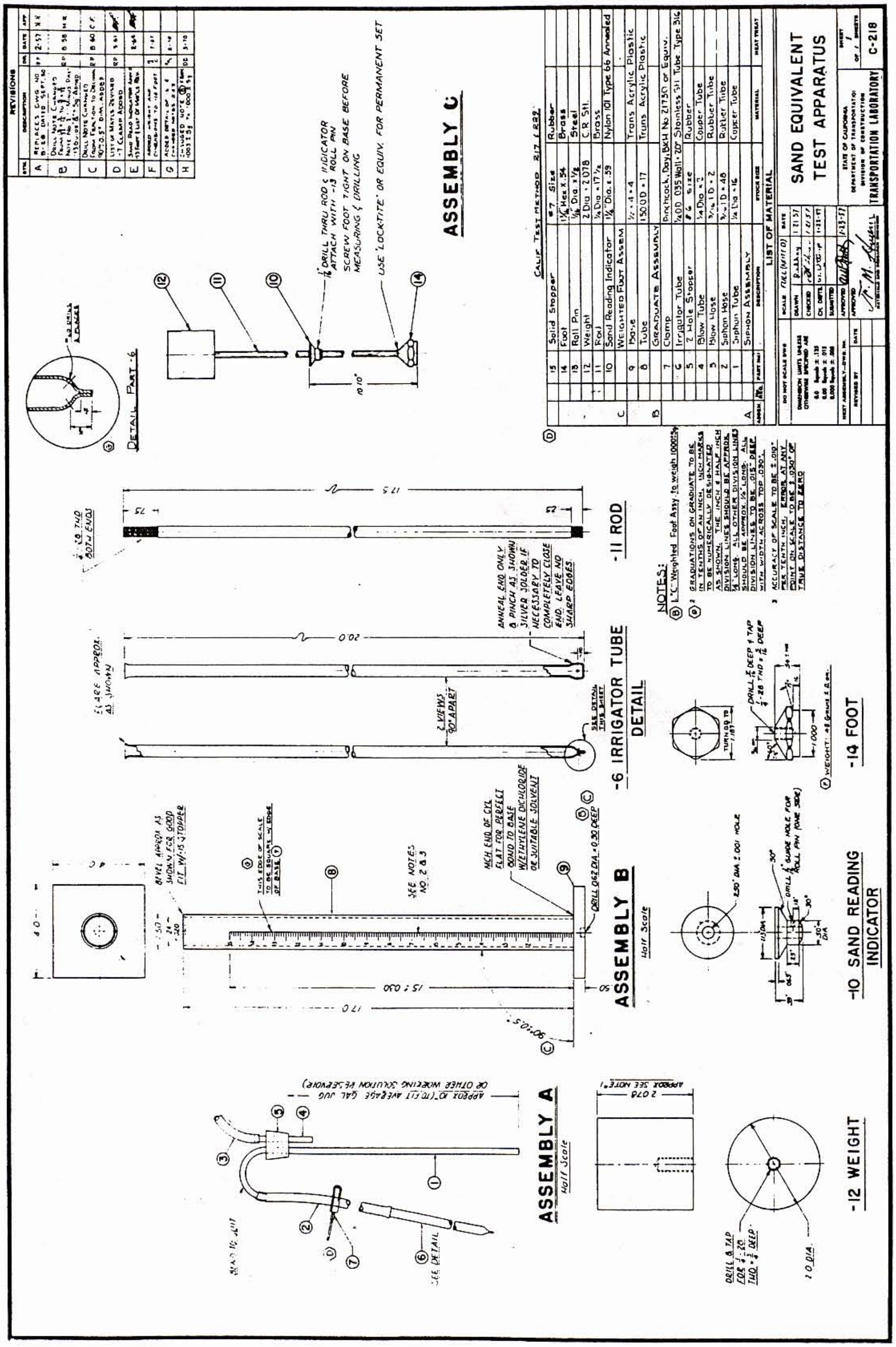
ASSEMBLY A
Half Scale
-12 WEIGHT
ASSEMBLY B
Half Scale
-10 SAND READING INDICATOR
-6 IRRIGATOR TUBE DETAIL
-14 FOOT
-11 ROD
ASSEMBLY C
DETAIL PART -6
REVISIONS
LIST OF MATERIAL
SAND EQUIVALENT TEST APPARATUS
TRANSPORTATION LABORATORY
C-218

FIGURE 4

2. Working Calcium Chloride Solution. Prepare the working calcium chloride solution by diluting 85 ± 5 ml of the stock calcium chloride solution with water to obtain 1 gal of working solution.

3. Water. Use distilled or demineralized water for the normal performance of this test, including the preparation of the working calcium chloride solution. If it is determined, however, that the local tap water is of such quality that it does not affect the test results, it is permissible to use it in lieu of distilled or demineralized water.

D. CONTROL

The temperature of all solutions and water should be maintained at 72° ± 5°F during the performance of this test. Individual test results which meet the minimum durability index value when the temperature is below the recommended range are acceptable.

E. SAMPLE PROCESSING

1. Obtain a representative sample of the material to be tested.

2. Process the sample and separate on the No. 4 sieve according to the procedures in California Test 201. The material passing the No. 4 sieve is tested independently from the material retained on the No. 4 sieve. If either of these primary size portions amounts to less than 15% of the total sample, that portion should not be tested. The durability index of the tested portion will represent the entire sample.

3. Separate the retained No. 4 material on the ¾ inch, ½ inch and ⅜ inch sieves.

4. Calculate the size distribution of the ¾ inch x No. 4 portion of the material. Do not include the material retained on the ¾ inch sieve or the material passing the No. 4 sieve in this calculation.

5. Materials with a minimum nominal size larger than ¾ inch shall be crushed to pass the ¾ inch sieve and then processed as described below. The portion of the crushed material which passes the No. 4 sieve shall not be tested for durability index.

F. TEST PROCEDURES

1. Procedure A, Coarse Durability (D_c) for material retained on a No. 4 sieve.

a. Process the material to be tested as described in Section D "Sample Processing".

b. Prepare a test specimen having an air-dry weight of 2550 ± 25 grams by combining the graded fractions as specified below.

(1) For materials which have a minimum of 10 percent in each of the specified fractions, prepare the test specimen according to the weights listed in Table No. 1.

Table No. 1
Basic Test Specimen Grading

Aggregate Passing	*Sieve Size Retained*	*Air-Dry Weight in grams*
¾ inch	½ inch	1070 ± 10
½ inch	⅜ inch	570 ± 10
⅜ inch	No. 4	910 ± 5
Total Test Specimen Weight		2550 ± 25

(2) For materials with less than 10 percent in any of the fractions specified in Table No. 1, prepare the test specimen using the actual calculated percentage for the deficient fraction and proportionally increase the weights of the remaining fractions to obtain the 2550 gram test specimen.

Example 1—Less than 10% of ¾ in. x ½ in. aggregate.

Aggregate Sieve Size	*Percent Each Size*	*Calculations*	*Air-Dry Weight Grams*
¾ in. x ½ in.	6	.06 x 2550	153 ± 10
½ in. x ⅜ in.	26	$\frac{570\ (2550 - 153)}{570 + 910}$	923 ± 10
⅜ in. x No. 4	68	$\frac{910\ (2550 - 153)}{570 + 910}$	1474 ± 5
Totals	100		2550 ± 25

Example 2—Less than 10% of ¾ in. x ½ in. and ½ in. x ⅜ in. aggregate.

Aggregate Sieve Size	*Percent Each Size*	*Calculations*	*Air-Dry Weight Grams*
¾ in. x ½ in.	4	.04 x 2550	102 ± 10
½ in. x ⅜ in.	7	.07 x 2550	179 ± 10
⅜ in. x No. 4	89	2550 − (102 + 179)	2269 ± 5
Totals	100		2550 ± 25

c. Wash the test specimen using the following procedure.

(1) Place the test specimen in the wash vessel.

(2) Add 1000 ± 5 ml water, clamp the lid in place and secure the vessel in the agitator.

(3) At 1 minute ± 10 seconds after adding the water to the specimen, start the agitator and shake the vessel for 2 minutes ± 5 seconds.

(4) Pour the contents of the vessel into a No. 4 sieve and rinse with fresh water until the water passing through the sieve is clear.

d. Transfer the material to a pan, dry to constant weight at 230° ± 9°F, and cool to room temperature.

e. Abrade the test specimen using the following procedure.

(1) Place the washed and dried test specimen in the wash vessel.

(2) Add 1000 ± 5 ml water, clamp the lid in place and secure the vessel in the agitator.
(3) At 1 minute ± 10 seconds after adding the water to the specimen, start the agitator and shake the vessel for 10 minutes ± 15 seconds.

f. Separate the aggregate and water on the No. 200 sieve.
(1) Remove the lid from the wash vessel and bring the fines into suspension by holding the vessel in an upright position and moving it vigorously in a horizontal circular motion 5 or 6 times causing the contents to swirl inside.
(2) Immediately pour the contents of the vessel into the No. 8 and No. 200 sieves nested over the collection pot.
(3) Tilt the No. 8 sieve to promote drainage, then discard the material retained on the No. 8 sieve.
(4) Collect all of the wash water and minus No. 200 sieve material in the collection pot. To assure that all material finer than the No. 200 sieve is washed through the sieve, use the following procedure:
(a) As the wash water is draining through the No. 200 sieve, apply a jarring action to the sieve by lightly bumping the side of the sieve frame with the heel of the hand.
(b) When a concentration of material is retained on the No. 200 sieve, rerinse this fine material by pouring the wash water through the sieve again, using the following procedure:
(1) Allow the wash water to stand undisturbed in the collection pot for a few moments to permit the heavier particles to settle to the bottom.
(2) Set the No. 200 sieve aside and pour the upper portion of the wash water into a separate container.
(3) Place the No. 200 sieve back on the collection pot and pour the water back through the material on the No. 200 sieve. (If two collection pots are available the specimen may be rinsed by alternately placing the sieve on one and then the other while pouring the wash water through the material on the sieve. Before each rinsing allow the heavier particles to settle to the bottom and pour only the upper portion of the water through the material.)
(4) Repeat this procedure as necessary until all of the minus No. 200 material has been washed through the sieve. When the material has been rinsed sufficiently the material on the sieve will be free of visible streaks of clay and the wash water will flow freely through the sieve and accumulated material.

g. Pour all of the wash water and passing No. 200 sieve material into a graduated cylinder. Use fresh water as necessary to flush all the fines from the collection pot and adjust the volume to 1000 ± 5 mls.

h. Return the wash water to the collection pot taking care to include all water and fines.

i. Fill the graduated plastic cylinder to the 0.3 inch mark with stock calcium chloride solution and place the funnel on the cylinder.

j. Stir the wash water vigorously with one hand to bring all the fines into suspension. Use a circular motion allowing the fingers to rub the sides and bottom of the collection pot.

k. Immediately fill the graduated plastic cylinder to the 15-inch mark with the turbulent wash water.

l. Stopper the cylinder and thoroughly mix the wash water and calcium chloride solution by inverting the cylinder 20 times in approximately 35 seconds. Allow the air bubble to completely transverse the length of the cylinder each time.

m. Immediately place the cylinder on a work bench or table free of vibrations, remove the stopper, and allow the cylinder to stand undisturbed for 20 minutes ± 15 seconds.

n. Immediately read the top of the sediment column to the nearest 0.1 inch.

o. Determine the coarse durability index (d_c) from Table No. 2.

TABLE NO. 2
DURABILITY INDEX OF COARSE AGGREGATE AND CHIPS

Sediment height (inches)	*Durability index*	*Sediment height (inches)*	*Durability index*	*Sediment height (inches)*	*Durability index*	*Sediment height (inches)*	*Durability index*	*Sediment height (inches)*	*Durability index*
0.0	100	3.0	53	6.0	39	9.0	29	12.0	18
0.1	96	3.1	52	6.1	38	9.1	29	12.1	18
0.2	93	3.2	52	6.2	38	9.2	28	12.2	18
0.3	90	3.3	51	6.3	38	9.3	28	12.3	17
0.4	87	3.4	51	6.4	37	9.4	28	12.4	17
0.5	85	3.5	50	6.5	37	9.5	27	12.5	16
0.6	82	3.6	49	6.6	37	9.6	27	12.6	16
0.7	80	3.7	49	6.7	36	9.7	27	12.7	15
0.8	78	3.8	48	6.8	36	9.8	26	12.8	15
0.9	76	3.9	48	6.9	36	9.9	26	12.9	14
1.0	74	4.0	47	7.0	35	10.0	26	13.0	14
1.1	73	4.1	47	7.1	35	10.1	25	13.1	13
1.2	71	4.2	46	7.2	35	10.2	25	13.2	13
1.3	70	4.3	46	7.3	34	10.3	25	13.3	12
1.4	68	4.4	45	7.4	34	10.4	24	13.4	12
1.5	67	4.5	45	7.5	34	10.5	24	13.5	11
1.6	66	4.6	44	7.6	33	10.6	24	13.6	11
1.7	65	4.7	44	7.7	33	10.7	23	13.7	10
1.8	63	4.8	43	7.8	33	10.8	23	13.8	9
1.9	62	4.9	43	7.9	32	10.9	23	13.9	9
2.0	61	5.0	43	8.0	32	11.0	22	14.0	8
2.1	60	5.1	42	8.1	32	11.1	22	14.1	7
2.2	59	5.2	42	8.2	31	11.2	22	14.2	7
2.3	59	5.3	41	8.3	31	11.3	21	14.3	6
2.4	58	5.4	41	8.4	31	11.4	21	14.4	5
2.5	57	5.5	40	8.5	30	11.5	20	14.5	4
2.6	56	5.6	40	8.6	30	11.6	20	14.6	4
2.7	55	5.7	40	8.7	30	11.7	20	14.7	3
2.8	54	5.8	39	8.8	29	11.8	19	14.8	2
2.9	54	5.9	39	8.9	29	11.9	19	14.9	1
								15.0	0

2. Procedure B, Coarse Durability (D_c) "Modified" (for lightweight or porous aggregates)

Because of the low specific gravity and/or high absorption rate of some aggregates, the proportions of aggregate to wash water are too great to permit the intended interparticle abrasion. Testing of these materials will require adjustment of the test specimen weight and volume of test water. All materials which are not completely inundated, when 1000 mls of water are added to a 2500 gram test specimen, shall be tested according to Method A with the following modifications.

a. Determine the bulk, oven-dry specific gravity and the percentage of absorption of the aggregate in accordance with California Test 206.

b. Adjust the total weight of the test specimen specified in E-1-b using the formula:
Adjusted Specimen Wt. (grams) = ((Specific Gravity of Aggregate)/2.65) x 2500

c. Adjust the weight of material in each size fraction proportionally to the weights specified in E-1-b.

d. Adjust the volume of test water specified in E-1-c and E-1-e using the formula except that the volume of water shall always be at least 1000 mls.
Adjust Water = 1000 + (A x W) − 50
Where: A = Absorption of Aggregate (%)
W = Weight of Test Specimen

3. Procedure C, Fine Durability (D_f) for material passing a No. 4 sieve.

a. Process the material to be tested as described in Section D "Sample Processing".

b. Split or quarter 500 ± 25 grams of material from the passing No. 4 portion of the sample.
(1) See step 3-f for optional preparation procedure.

c. Dry to constant weight at 230° ± 9°F and cool to room temperature.

d. Wash the dried material by the following procedure:
(1) Place the material in the wash vessel.
(2) Add 1000 ± 5 mls of water, clamp the lid in place and secure the vessel in the agitator.
(3) At 10 minute ± 30 seconds after adding water to the material start the agitator and shake the vessel for 2 minutes ± 5 seconds.
(4) Pour the contents of the vessel into a No. 200 sieve and rinse with fresh water until the water passing through the sieve is clear. Use a flexible hose attached to a faucet to direct water onto the material.

e. Transfer the material to a pan, dry to constant weight at 230° ± 9°F, and cool to room temperature.
(1) Use water from the flexible hose as necessary to rinse the material from the sieve into the pan.
(2) Free water can be removed by tilting the pan and then, after the fines have settled, carefully pouring off the clear water.

f. A 500 gram fine sieve analysis test specimen which has been tested in accordance with California Test 202, may be utilized in lieu of the material prepared according to steps b. through e. above. If the fine sieve analysis test specimen is used, all of the material separated during sieving including that portion retained in the sieve pan shall be thoroughly recombined before proceeding to step g. below.

g. Split or quarter the washed and dried material to provide a test specimen of sufficient size to fill the measuring tin to level full.
(1) When filling the measuring tin, consolidate the material in the tin by tapping the bottom edge on a hard object such as the work bench.
(2) Fill the measuring tin to slightly rounded above the brim and then strike off to level full using a straightedge.

h. Fill the graduated plastic cylinder to 4 ± 0.1 inches with working calcium chloride solution.

i. Pour the prepared test specimen into the plastic cylinder.
(1) Use the funnel to avoid spillage.
(2) Release air bubbles and promote thorough wetting by bumping the base of the cylinder against a firm object while the test specimen is being poured into the cylinder or by tapping the cylinder sharply on the heel of the hand several times after the test specimen has been poured in.

j. Allow the wetted material to stand undisturbed for 10 ± 1 minutes.

k. Abrade the test specimen by the following procedure:
(1) At the end of the 10 minute soaking period, stopper the cylinder, then loosen the material from the bottom by shaking the cylinder while holding it in a partially inverted position.
(2) Secure the cylinder in the mechanical sand equivalent shaker.
(3) Start the shaker and allow it to operate for 10 minutes ± 15 seconds.

l. Irrigate the test specimen to flush the abraded fines from the sand using the following procedure:
   (1) At the end of the shaking period remove the cylinder from the shaker and set it upright on the work bench. Insert the irrigator tube in the cylinder, start the flow of working calcium chloride solution, and rinse the material from the sides of the cylinder as the irrigator is lowered.
   (2) With the cylinder remaining in an upright position and the solution flowing from the tip, apply a twisting action to the irrigator and force it to the bottom of the cylinder. The flow of solution will flush the clay size particles upward and into suspension. Withdraw the irrigator from the sand as necessary to change position and again force it to the bottom. The most effective technique for penetrating the test sample with the irrigator is to hold the irrigator between the palms of both hands and rotate it by rubbing the hands back and forth while applying a downward pressure.
   (3) Continue twisting and forcing the irrigator to the bottom of the cylinder until the fines have been flushed from all areas of the sample. Rotate the cylinder with each penetration of the irrigator and visually inspect the test specimen for pockets of fine material.
   (4) When the solution reaches the 15-inch mark in the cylinder, slowly withdraw the irrigator without shutting off the flow so that the liquid level is maintained at about 15 inches. Regulate the flow just before the irrigator is entirely withdrawn and adjust the find level to 15 inches.

m. Immediately place the cylinder on a work bench or table free of vibrations and allow the cylinder and contents to stand undisturbed for 20 minutes ± 15 seconds from the time the irrigation is completed.

n. Determine the "clay reading".
   (1) At the end of the 20-minute period read and record the level of the top of the sediment column. This is the clay read.
   (2) When the clay reading falls between 0.1-inch graduations, record the level of the higher graduation.
   (3) If a clearly defined line of demarcation does not form between the sediment and the liquid above it in the specified 20-minute period, allow the cylinder to stand undisturbed until the clear demarcation line does form. Then immediately read and record the time and the height of the column. If tap water was used retest an untested portion of the sample using distilled or demineralized water.
   (4) If the liquid immediately above the line of demarcation is still darkly clouded at the end of 20 minutes, and the demarcation line, although distinct, appears to be in the sediment column itself, read and record the level of this line at the end of the specified 20-minute period. If tap water was used, retest an untested portion of the sample using distilled or demineralized water.

o. Determine the "sand reading".
   (1) After the clay reading has been taken gently lower the weighted foot assembly into the cylinder until it comes to rest on the sand. Do not allow the indicator to hit the mouth of the cylinder as the assembly is being lowered.
   (2) As the weighted foot comes to rest on the sand, tip the assembly toward the graduation on the cylinder so that the position of the indicator is visible. Take care not to press down on the assembly.
   (3) Read the level of the top edge of the indicator.
   (4) Subtract 10 inches from the observed reading. This is the sand reading.
   (5) When the sand reading falls between 0.1-inch graduations, record the level of the higher graduation.

p. Calculate the fine durability index (D_f) using the formula:

$$D_f = (\text{Sand Reading}/\text{Clay Reading}) \times 100$$

   (1) If the calculated durability index is not a whole number, report it as the next higher whole number.

4. Procedure D, Fine Durability (D_f) "Modified", for pea gravel or chips having a nominal minimum size no smaller than a No. 16 sieve.

a. Process the material to be tested as described in Section D "Sample Processing".

b. Split or quarter out 500 ± 25 grams of material from the passing No. 4 portion of the sample.

c. Wash the test specimen by the following procedure.
   (1) Place the material in the wash vessel.
   (2) Add 1000 ± 5 mls of water, clamp the lid in place and secure the vessel in the agitator.
   (3) At 10 minutes ± 30 seconds after adding water to the material, start the agitator and shake the vessel for 2 minutes ± 5 seconds.
   (4) Pour the contents of the vessel into a No.

200 sieve and rinse with fresh water until the water passing through the sieve is clear. Use a flexible hose attached to a faucet to direct water onto the material.

d. Transfer the material to a pan, dry to constant weight at 230° ± 9°F, and cool to room temperature.
   (1) Use water from the flexible hose as necessary to rinse the material from the sieve into the pan.
   (2) Free water can be removed by tilting the pan and then, after the fines have settled, carefully pouring off the clear water.

e. Split or quarter the washed and dried material to provide a test specimen of sufficient size to fill the measuring tin to level full.
   (1) When filling the measuring tin, consolidate the material in the tin by tapping the bottom edge on a hard object such as the work bench.
   (2) Fill the measuring tin to slightly rounded above the brim and then strike off to level full using a straightedge.

f. Fill the graduated plastic cylinder to 4 ± 0.1 inches with water.

g. Pour the prepared test specimen into the plastic cylinder.
   (1) Use the funnel to avoid spillage.
   (2) Release air bubbles and promote thorough wetting by bumping the base of the cylinder against a firm object while the test specimen is being poured into the cylinder or by tapping the cylinder sharply on the heel of the hand several times after the test specimen has been poured.

h. Allow the wetted material to stand undisturbed for 10 ± 1 minutes.

i. Abrade the test specimen by the following procedure:
   (1) At the end of the 10-minute soaking period, stopper the cylinder, then loosen the material from the bottom by shaking the cylinder while holding it in a partially inverted position.
   (2) Secure the cylinder in the mechanical sand equivalent shaker.
   (3) Start the shaker and allow it to operate for 30 ± 1 minutes.

j. Transfer the water and passing No. 200 sieve size material to a second graduated plastic cylinder.
   (1) Fill an empty graduated plastic cylinder to the 0.3 inch mark with stock calcium chloride solution.
   (2) Place a No. 200 sieve into a funnel that empties into the cylinder containing the calcium chloride solution.
   (3) Tip the stoppered cylinder containing the test specimen upside down and shake to loosen the material from the bottom.
   (4) Hold the mouth of the inverted cylinder over the sieve and remove the stopper, allowing the test specimen and water to pour onto the sieve.
   (5) Collect the water and passing No. 200 material in the second cylinder.
   (a) Rinse the remaining fines from the first cylinder onto the sieve with a small amount of fresh water.
   (b) Rinse the material retained on the sieve with additional fresh water to assure that the minus No. 200 portion passes through the sieve. Take care not to fill the cylinder above the 15-inch mark.
   (c) Adjust the level of the liquid to the 15-inch mark with fresh water.

k. Stopper the cylinder and thoroughly mix the wash water and calcium chloride solution by inverting the cylinder 20 times in approximately 35 seconds. Allow the air bubble to completely traverse the length of the cylinder each time.

l. Place the cylinder on a work bench or table free of vibrations, remove the stopper and allow to stand undisturbed for 20 minutes ± 15 seconds.

m. Immediately read the top of the sediment column to the nearest 0.1 inch.

n. Determine the Fine Durability index (D_f) "modified" from Table No. 2.

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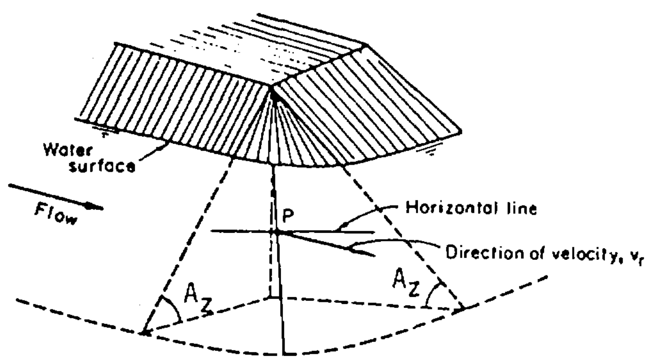
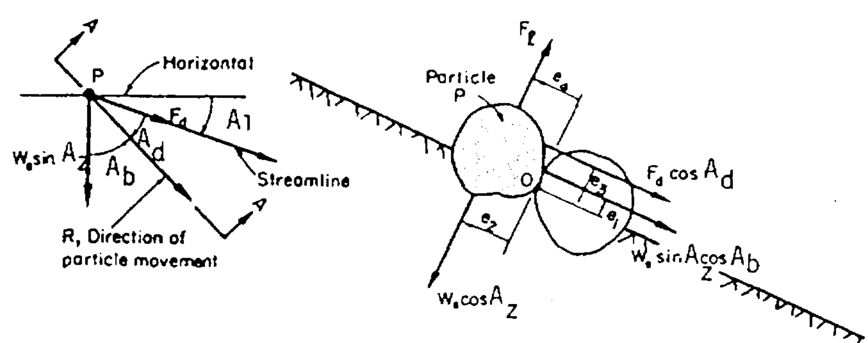
APPENDIX B

Derivation of Critical Shear Stress Correction Factor

The forces on the rock particle are lift force Fl, drag force Fd and weight of the particle Ws. Rock particles on side-slopes tend to roll rather than slide, so it is appropriate to consider stability of rock particles in terms of moments about a contact point O about which rotation must take place. The components of forces relative to the plane of motion are shown in Figure B.1(b).



(a) General view



(b) View of vectors in the plane of the side slope.

(c) Section A-A cross-section of side slope in the direction of particle motion.

Figure B.1 Diagrams for the Riprap Stability Analysis

At incipient motion, there is a balance of moments such that

$$e_2 W_s \cos A_z = e_1 W_s \sin A_z + e_3 F_d \cos A_d + e_4 F_l$$

Where e is the moment arm of each force.

The factor of safety, SF, of particles against rotation is then determined by the ratio of the moments.

$$SF = \frac{e_2 W_n}{e_1 W_t \cos A_b + e_3 F_d \cos A_d + e_4 F_l}$$

Rearranging the above equation can be written as:

$$1/SF = (e_1/e_2)(W_t/W_n)\cos A_b + (e_3/e_2)(F_d/W_n)\cos A_d + (e_4/e_2)(F_l/W_n)$$

Where

$W_n = W_s \cos A_z$ (the weight component normal to the side-slope)
$W_t = W_s \sin A_z$ (the weight component tangent to the side-slope)

It can be shown that

$$e_2/e_1 = \tan Ar$$

Where Ar is the bearing angle of the stone which is assumed to equal the angle of repose.

Let the side-slope stability number be given by

$$SN = (e_3/e_2)(F_d/W_s)\cos A_d + (e_4/e_2)(F_l/W_s)$$

so,

$$(1/SF) = (\tan A_z/\tan A_r)\cos A_b + SN/\cos A_z \qquad \text{(B1)}$$

The lift and drag forces are proportional to the boundary shear where

$F_l = c_1 d^2 To$
$F_d = c_2 d^2 To$

Where

d = the stone diameter; and,
To = the mean boundary shear stress.

The submerged weight of a stone is

$$W_s = c_3 (U_s - U_w) d^3$$

Where U_s and U_w and the unit weight of stone and water respectively.

So,

$$SN = [(e_3/e_2)(c_2/c_3)\cos A_d + (e_4/e_2)(c_1/c_3)] \frac{To}{(U_s-U_w)d}$$

$$\text{let } m = (e_4/e_2)(c_1/c_3)$$
$$n = (e_3/e_2)(c_2/c_3)$$

then

$$SN = (n \cos Ad + m) \frac{To}{(Us-Uw)d}$$

For a stone on a plain bed Ad = 0° (i.e., Ab=90°) and incipient conditions (SF=1.0), then SN=1.0 from equation B1, so

$$1 = (n+m) \frac{Tc}{(Us-Uw)d} \qquad \text{(B2)}$$

Where Tc is the critical boundary shear stress. The term n+m is equivalent to the Shield's parameter. The stability number on a plane horizontal bed is therefore given by

$$SN' = To/Tc$$

In considering the incipient motion of stones in a riprap layer, the ratios of c_1/c_2 and e_4/e_3 vary depending on the turbulent conditions of the flow and the interlocking arrangement of the stones. If it is assumed that the stones are approximately spherical in shape, then

$$e_4/e_3 = 2$$

The ratio of lift to drag was assumed by Stevens and Simons (1971) to be

$$c_1/c_2 = \tfrac{1}{2}$$

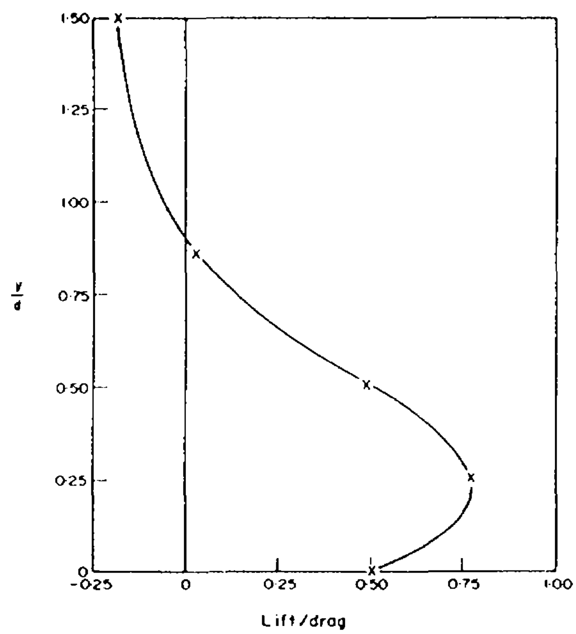
The drag coefficient for a sphere is approximately 0.5. Lift coefficients from various studies on hemispheres are reported by Benedict and Christensen, 1971.

Lift Coefficient, C_L, for Hemispheres

C_L	Study	Comment
0.405	Chepil (1958)	Based on potential flow for widely packed spheres
0.348	Chepil (1958)	Pressure measurements for widely packed spheres

0.359	Benedict (1968)	Closely packed spheres theoretical solution
0.178	El Samni and Einstein	Pressure measurements closely packed spheres

The Stevens and Simons assumption for the lift to drag ratio is a compromise between various measurements for beds composed of spherical particles. Measurements by Apperley (1968) showed the range of lift to drag that can occur as flow depth varies relative to the particle diameter.



Variation of lift/drag ratio with relative depth

The ratio of stability number on a side-slope, SN, to the stability factor on a plane horizontal bed, SN′ is

$$\frac{SN}{SN'} = \frac{m + n \cos Ad}{m + n} = \frac{m/n + \cos Ad}{m/n + 1}$$

Where $\frac{m}{n} = \frac{e4}{e3} \frac{c1}{c2}$

Substituting the Stevens/Simons assumptions gives

$$\frac{m}{n} = 1 \qquad \text{(B3)}$$

and

$$\frac{SN}{SN'} = \tfrac{1}{2}\,(1 + \cos Ad)$$

or

$$SN = \tfrac{1}{2}\,(1 + \cos Ad)\ To/Tc$$

by trigonometric identity, where

$$Ad = 90 - Al - Ab$$

$$\cos Ad = \cos\,(90 - Al - Ab)$$

$$= \sin\,(Al + Ab)$$

Equation B1 becomes

$$\frac{1}{SF} = \frac{\tan Az}{\tan Ar} \cos Ab + \tfrac{1}{2}\,\frac{(1+\sin(Al+Ab))}{\cos Az} \frac{Td}{Tc}$$

where Td is the allowable or design shear stress.

$$\boxed{Td = \frac{1}{SF} \frac{2\cos Az}{(1+\sin(Al+Ab))} \left(1 - SF \frac{\tan Az}{\tan Ar} \cos Ab\right) Tc}$$

The angle Al shown in Figure B.1 is the angle between the velocity vector and the horizontal plane in the plane of the side-slope. The angle Al is a function of hydraulic conditions. The angle Ab can be derived from a balance of moments normal to the direction of stone motion.

$$e_3\ Fd\ \sin Ad = e_1\ W_s\ \sin Az\ \sin Ab$$

now from trigonometric identity

$$\sin Ad = \cos Al\ \cos Ab - \sin Al\ \sin Ab$$

$$\cos Al - \sin Al\ \tan Ab = \frac{e1Ws}{e_3Fd}\ \sin Az\ \tan Ab$$

$$\tan Ab = \frac{\cos A1}{\frac{e1Ws}{e3Fd}\sin Az + \sin A1}$$

The term $\frac{e1Ws}{e3Fd}$ can be written as

$$\frac{e1Ws}{e3Fd} = \frac{e2}{e3}\frac{Ws}{Fd}\frac{e1}{e2}$$

$$= \frac{e2}{e3}\frac{(Us-Uw)d}{To}\frac{1}{\tan Ar}$$

$$= \frac{1}{n}\frac{(Us-Uw)d}{To}\frac{1}{\tan Ar}$$

from equation B2 and B3

$$\frac{e1Ws}{e3Fd} = \frac{1}{\frac{1}{2}To/Tc}\frac{1}{\tan Ar}$$

so

$$\boxed{\tan Ab = \frac{\cos A1}{\frac{2}{To/Tc}\frac{\sin Az}{\tan Ar} + \sin A1}}$$

REFERENCES:

Stevens, M.A. and D.B. Simons, 1971, "Stability Analysis for Coarse Granular Material on Slopes", Chapter 17, in River Mechanics, Hsieh Wen Shen, ed.

Benedict, B.A. and B.A. Christensen, 1972, "Hydrodynamic Lift on a Stream Bed", Chapter 5, in Sedimentation, Hsieh Wen Shen, ed.

Apperley, L.W., 1968, "The Effect of Turbulence on Sediment Entrainment", Ph.D., Thesis Univeristy of Auckland, N.Z., sited from Loose Boundary Hydraulics, A.J. Raudkivi, Second edition, 1976.

APPENDIX C

Resistance Coefficients for Alluvial Channels

The shear stress is proportional to the square of the velocity. In order to estimate the shear stress, a determination of friction losses must be made. In alluvial channels, the flow resistance consists of two components: resistance due to the particles forming the channel, and resistance due to drag caused by bed forms present in the channel which are associated with the sediment transport regime of the flow. The total resistance due to bed forms and particle resistance are summarized in the following table. For alluvial channels, the resistance in the plane bed condition is equivalent to the resistance due to particle roughness alone.

Values of Manning's "n" Resistance Coefficient

Bed Roughness	Typical Range	Recommended for Flood Studies	Recommended for Riprap Design
Ripples	0.018-0.030	0.030	0.022
Dunes	0.020-0.035	0.035	0.030
Transition*	0.014-0.025	0.030	0.025
Plane Bed*	0.012-0.022	0.030	0.020
Standing Wave*	0.014-0.025	0.030	0.020
Antidunes*	0.015-0.030	0.030	0.025

* These bed forms are transient, the resistance coefficient for antidunes is recommended.

Gravel-Bed Channels. The particle roughness for gravel-bed channels or channels lined entirely with riprap ($1.5<da/D_{50}<185$) is given by the following equation:

$$n + C * D_{50}^{(1/6)} \qquad \text{(C1)}$$

Where

$$C = \frac{(da/D50)^{(1/6)}}{[8.58 + 20.0*\log(da/D_{50})]} \qquad \text{(C2)}$$

Values of the coefficient C are given in Chart C.1 as a function of the relative depth, da/D_{50}.

Sand-Bed Channels. For sand-bed channels ($da/D_{50} > 185$), the particle roughness is given by the following equation:

$$n = 0.0185 * da^{(1/6)} \qquad (C3)$$

Composite Roughness. For channels that are formed of two types of material, such as ripraped banks with a sand bed, a composite resistance coefficient should be used. The following equations can be used to determine the composite roughness:

$$n = Fc * n1 \qquad (C4)$$

$$Fc = [P1/P + (1-P1/P)*(n2/n1)^{1.5}]^{(2/3)} \qquad (C5)$$

Where

P1 = wetted perimeter of the channel bed;
P = total wetted perimeter;
n1 = lower resistance value; and,
n2 = larger resistance value.

Values of the coefficient Fc are given in Chart C.2 as a function of the wetted perimeter and channel roughness.

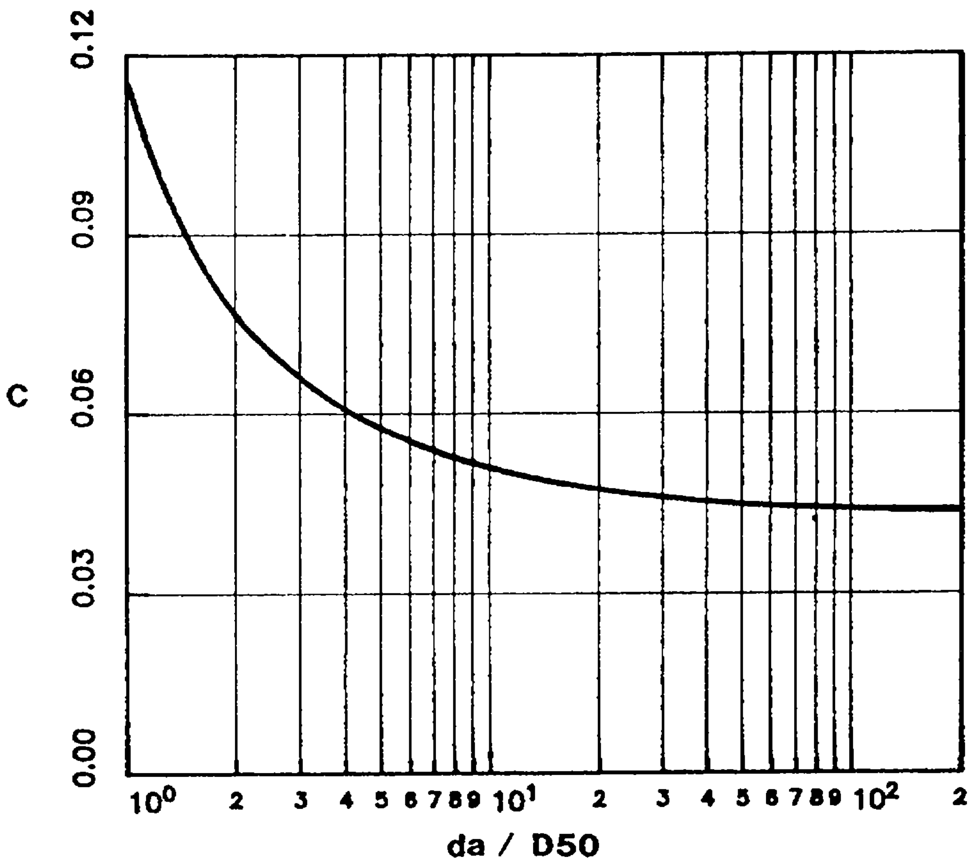


Chart C.1. Flow Resistance Coefficient for Equation C2.

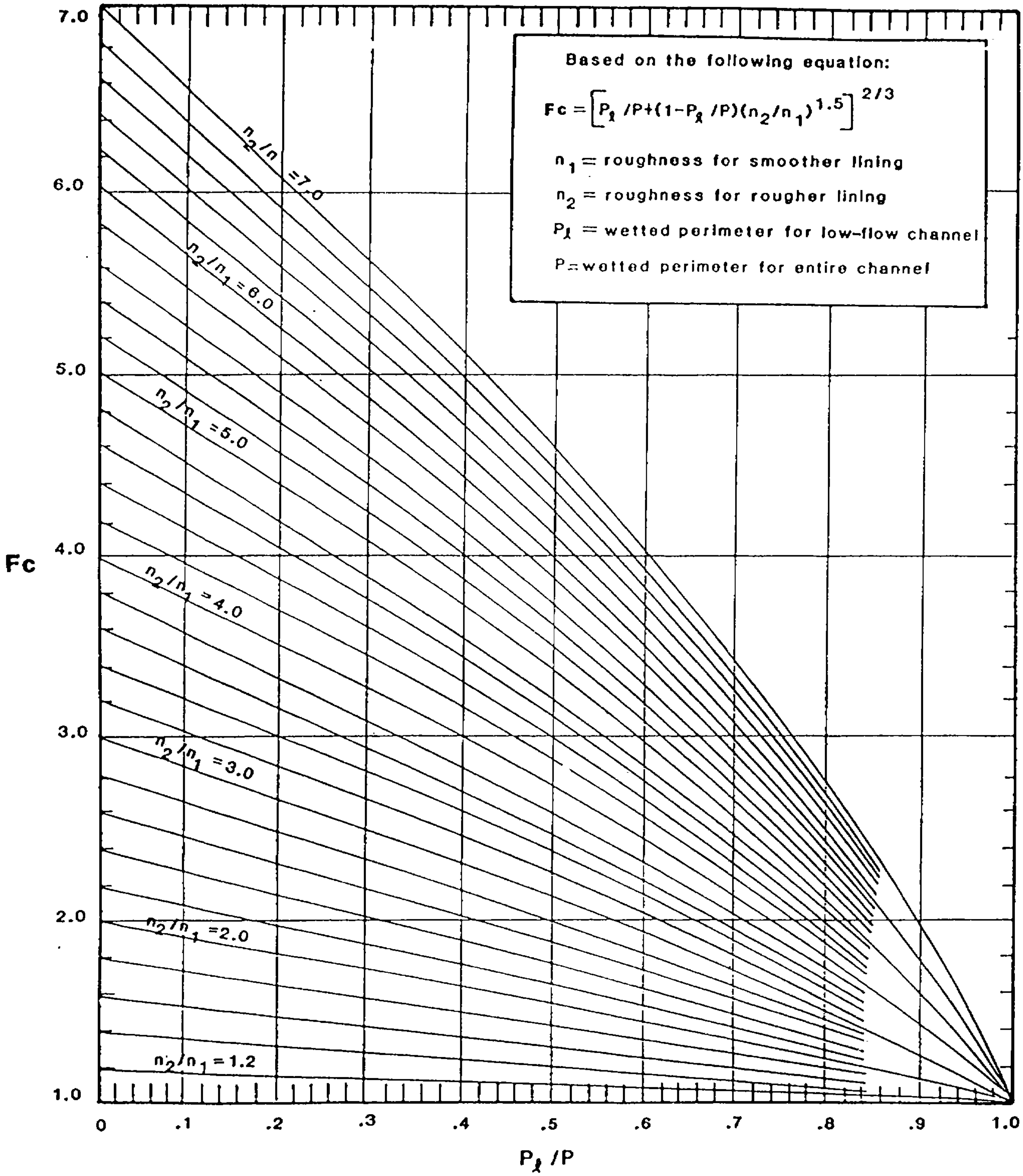


Chart C.2. Composite Roughness Adjustment Factor, Fc
(Revised HEC-15)