

TABLE OF CONTENTS: CHAPTER 5 - CHANNEL LOCATION,  
ALIGNMENT, & HYDRAULIC DESIGN

Location and Alignment . . . . .	5-1
Hydraulic Design . . . . .	5-2
Subcritical Flow . . . . .	5-2
Supercritical Flow . . . . .	5-8
Figure 5-1 -- Hydraulics: Junction Structures. . . . .	5-13

CHAPTER 5 - APPENDIX I - TRANSITIONS

Important Transitions Where it is Necessary to Conserve Head. . . . .	5.1-1
Figure 5.1-1 -- Typical Transition to Rectangular Flume . . . . .	5.1-7
Figure 5.1-2 -- Typical Inlet Transition. . .	5.1-11
Figure 5.1-3 -- Typical Outlet Transition . .	5.1 15
Figure 5.1-4 -- Typical Inlet Transition to Pipe Line. . . . .	5.1-17
Less Important Transitions	5.1-19
Figure 5.1-5 -- Typical Transition with Straight Sides . . . . .	5.1-21

CHAPTER 5 - APPENDIX II - MOMENTUM  
METHOD OF DETERMINING BRIDGE PIER LOSS

Figure 5.2-1 -- Bridge Pier Losses by the Momentum Method . . . . .	5.2-3
Figure 5.2-2 -- Cross-Section of Channel Showing Center Bridge Pier. . . . .	5.2-6
Figure 5.2-3 -- Longitudinal Section Along Centerline of Channel . . . . .	5.2-7
Table 5.2-1 -- Momentum Losses at Bridges . .	5.2-9
Figure 5.2-4 -- Water Depth Curve . . . . .	5.2-11



## CHAPTER 5. CHANNEL LOCATION, ALIGNMENT AND

### HYDRAULIC DESIGN

#### Location & Alignment

Channel alignment is an important feature of channel design. It should be selected with careful consideration given to all factors affecting its location, including an economic comparison of alternate alignments. The economic analysis should include all costs such as channel construction, rights-of-way, bridges, stabilizing measures and maintenance.

Many factors affect the planned alignment of a channel. Topography, the size of the proposed channel, the existing channel, tributary junctions, geologic conditions, channel stability, rights-of-way, existing bridges, required stabilization measures, farm boundaries, land use, and other physical features enter into this decision.

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

Alternate alignment should be considered in areas where geologic conditions present a stability problem. An alternate alignment may locate the channel in more stable soils. In some cases, the alignment of the existing channel may be satisfactory with only minor changes. An alignment resulting in a longer channel may, in a minor degree, help to alleviate stability problems. A longer channel will decrease the energy gradient which, in turn, will decrease the velocities and tractive forces. A meander channel will increase Manning's coefficient "n" which will reduce velocities. A meander channel, however, presents the problem of erosion at the curves in the channel which, in erosive soils, may require structural protection such as jetties, riprap, brush mats, etc. Suggested minimum radii of curvature are cited in NEH, Section 162/, Chapter 6, for channels of indicated size and gradient. Guidance for alignment consideration in design of higher velocity channels is presented later in this chapter under Supercritical Flow. When alternate alignments and designs are not feasible, or do not assure a stable channel, stabilization structures should be included in the design. Such structures are discussed in a later section of this chapter.

Where feasible, the alignment should be planned to make use of existing, adequate bridges and road structures which have many years of remaining life. From a technical standpoint, bridges and road structures in poor condition should not influence the alignment. Care should be exercised to minimize cases of isolating parts of fields from the rest of the farm, but good alignment should not be sacrificed to follow all farm boundaries. Long reaches of channel should be located in the low areas, particularly where drainage is a problem. Long tangents should be used wherever possible. In meander channels, good alignment should not be sacrificed to use the maximum amount of the old channel.

The above discussion on alignment deals with subcritical flow only. Supercritical flow requires consideration of some of the most complex problems in hydraulics and may require model studies for a basis of design criteria. A discussion on supercritical flow may be found in NEH, Section 5<sup>20</sup>/.

### Hydraulic Design

Criteria and procedures for hydraulic design of all types of channels are presented for subcritical and supercritical flow conditions.

#### Subcritical Flow

Design procedures for subcritical flow conditions for a limited range of functional and hydraulic conditions commonly encountered in agricultural drainage work are described in NEH, Section 16<sup>27</sup>/, Chapter 6. The following procedures apply to all conditions of subcritical flow:

Step 1. - - Determine the design discharge for all channel reaches. Use methods given in Chapter 4. Where overbank flow contributes a significant amount of water, it is best to assume that the discharge changes abruptly at arbitrary points within the channel reach and that the discharge remains constant between these points. As a guide to selection of these arbitrary points,  $Q$  should not be decreased more than 10 percent at any such point. As an example, consider a 10,000-foot reach of channel with overbank flow and no significant tributaries.  $Q = 3000$  cfs at the downstream end and 2000 cfs at the upper end. Here it would be logical to reduce  $Q$  by 200 cfs increments at each of four evenly spaced points within the reach. (The last 200 cfs reduction would be at the upstream end of the reach.)

Step 2. - - Determine the water surface elevation at the downstream end of proposed construction. (Assuming that the approximate channel alignment has already been set.) The method for determining this water surface elevation varies with the outlet condition:

1. Water stages in the outlet are independent of channel discharge. For this condition it is necessary to have stage data on the outlet stream or tidal outlet. Two water surface profiles must usually be run on the tributary channel--one with the water surface at the outlet at the highest possible elevation within the design flood frequency to insure capacity, and one with the water surface at the lowest level within the above limitation to insure channel stability with the resulting increased velocity.
2. Outlet is at a control point where critical flow exists. In this case, the control point establishes the water surface elevation at critical depth.
3. Outlet is at a point in the stream where the water surface can be established. In this case, where the downstream channel is prismatic, the water surface elevations at the outlet to the improved reach may be established by the methods outlined in T. R. 15, or by an assumption of uniform flow at that point if the downstream prismatic channel is sufficiently long and is not affected by grade changes.

In the event that the downstream channel is non-prismatic, it will be necessary to begin water surface profile calculations at a point considerably downstream using the methods outlined in T. R. 14 or NEH, Section 5<sup>20</sup>/, Supplement A, to determine the water surface elevation at the starting point for construction.

Step 3. - - Establish the water surface control line by the methods outlined in Chapter 6, NEH, Section 16<sup>2</sup>/. Such a control line is helpful in establishing a desirable hydraulic grade line and subsequently, the design invert grade. In setting the hydraulic control line, consideration must be given to freeboard requirements and, where applicable, to additional depth required for superelevation of the water surface and flow in the unstable range.

Freeboard is defined as the additional channel depth required for safety above the calculated maximum depth of water. Freeboard is exclusive of additional depth computed for superelevation or turbulence in the unstable range. Freeboard for trapezoidal channels at sub-critical flow should be equal to or greater than 20% of the depth at design discharge but not less than one foot. (10% is satisfactory for rectangular lined channels at sub-critical flow.)

In closely controlled irrigation canals or channels where overbank flooding is permissible at frequent intervals, the above freeboard criteria may be disregarded.

Where possible, it is advisable to keep the design water surface below the level of natural ground. There is usually no objection to containing the freeboard in fill. Occasionally, such a great saving can be accomplished by containing part of the design discharge in fill over low areas in the profile that it is economical to do so even at the expense of close construction control on the dike.

Curve radii should be sufficiently great to limit superelevation of the water surface to one foot above computed depth of flow or 10% of water surface width, whichever is the least.

The amount of superelevation may be determined as follows for subcritical flow in trapezoidal channels:

$$s = \frac{V^2 (b + 2 zd)}{2 (gR - 2zV^2)}$$

where the terms are as defined in the glossary.

Channels whose energy gradients at design flow have a slope at or near critical ( $0.7s_c < s_o < 1.3s_c$ ) will require additional depth as follows:

$$H_w = 0.25 d_c \left[ 1 - 11.1 \left( \frac{s_o}{s_c} - 1 \right)^2 \right]$$

where the terms are as defined in the glossary.

The above criterion applies to flows with velocities slightly greater than critical as well as to sub-critical flow in the unstable range. Where possible, grades and/or cross sections should be adjusted to avoid the unstable range.

Step 4. - - Select values of Manning's coefficient "n". The estimation of realistic values of the roughness coefficient "n" is an important factor in channel design. The value "n" indicates the net effect of all factors, except grade and hydraulic radius, causing retardation of flow in the reach of channel under consideration. The estimation of "n" warrants critical study and

judgement in the evaluation of the factors affecting its value. The primary factors are: irregularity of the surfaces of the channel sides and bottom, variations in shape and size of cross sections, obstructions, vegetation, and alignment of the channel.

A systematic procedure for the estimation of "n" values is contained in NEH, Section 520, Supplement B. This procedure should be used and supplemented by any other applicable data.

Table 1 in SCS-TP-61 21/ lists a range of "n" values for various channel linings and conditions, and contains a method of estimating "n" values for various vegetal linings.

The design capacity of a channel should be based on the "n" value anticipated after the channel has aged, giving consideration to the degree of maintenance that can reasonably be expected. When stability is in question, the stability of the channel should be checked with the "n" value anticipated immediately after construction.

Step 5. - - Determine the allowable side slopes by procedures given in Chapter 6 of this guide or NEH, Section 16, Chapter 6, if applicable.

Step 6. - - Determine allowable velocities or tractive forces for the various reaches, depending on which procedure is to be used to check channel stability against flowing water. These values will need to be considered when selecting channel sizes and slopes in step 7. Use procedures given in Chapter 6 of this guide. Since the depth of flow is required for use in both the velocity and tractive force procedures, this step will have to be done concurrently with step 7.

Step 7. - - Determine the size and shape of channel needed. Since the side slopes have been determined by other considerations, the design problem becomes the determination of the required depth and bottom width.

Trial cross sections may be selected by assuming uniform flow conditions and solving for the depth or width by using Manning's equation. Several combinations of depth and width should be evaluated.

Generally, the most economical channel will be one in which the hydraulic grade line approaches the water surface control line determined in step 2. When this is planned, the slope of the water surface control line may be used as the value of  $s$  in Manning's formula. The depth of flow determined for the reach will give the position of the channel bottom. The slope of the channel bottom may be made parallel to the water surface control line. If care is used in selecting the proportions of the cross section, only a

slight change in depth of flow or in bottom width at the ends of each reach will result. For the above condition, the effect of transitions on the water surface profile will be small and for all practical purposes the water surface control line becomes the true water surface profile for the channel. (See Fig. 6-13, NEH, Section 162/)

Step 8. - - Compute the water surface profile for the best apparent cross section. Hydraulic design methods outlined in NEH, Section 162/, Chapter 6 are based on the assumption that Manning's formula defines the slope of the hydraulic gradient. This assumption is valid for channels where changes in velocity head from one reach to the next are insignificant, and will provide a short cut to water surface profile calculations.

The generally accepted method of computing the water surface profile is outlined in T. R. 15. Water surface profiles must begin at the downstream end of the work and proceed upstream. Where reaches are long, the upstream part of the reach will usually approach a condition of uniform flow.

Often, the completed water surface profile will point up the need for reportioning the cross section to provide more capacity or more economy. Usually an adequate and economic cross section may be arrived at within two trial solutions.

Curves in alignment. - - Often it is necessary to re-evaluate curves in earth channels after the hydraulic design is otherwise complete. With velocities known it is possible to determine the minimum curve radius permissible without protection, as outlined under "Channel Stability." The decision between complying with this minimum radius and shortening the radius and providing rock riprap bank protection on the curve then becomes a matter of economics. Cost of right-of-way, severance or structure removal may be greater than the cost of protecting a tight curve. In very flat topography ( $s_0 < 0.001$  ft./ft.) Table 6.1, NEH Section 162/, may be used to determine minimum radius of unprotected curves.

Side drainage. - - Major tributaries on which work is to be done as a part of the project are usually brought in at channel invert grade. Water surface profiles above the junction on both tributaries may be calculated as stated above.

Minor tributaries are usually brought in at an elevation above the channel invert. The channel bank, thus exposed, must be protected by rock riprap, concrete, pipe over-pour or other methods.

Overland flow entering the channel should be concentrated where possible and brought in at selected locations. Where it is not practical to concentrate such flow, the channel bank should be protected or vegetated, unless the soil materials are sufficiently resistant to the erosive forces applied to remain stable.

Where low areas must be drained into the channel, it may be necessary to install conduits through a dike and attach automatic drainage gates to the conduits to prevent outflow from the channel.

Channel entrance. - - Flood channels are usually terminated at the upstream end where:

1. The discharge is sufficiently small that the existing facility has adequate capacity, or
2. The benefits from additional length of channel improvement will not justify the cost thereof.

In the former case, the work may usually be terminated by a simple transition from the constructed channel to the existing channel. Occasionally it will be necessary to install a drop spillway structure to reconcile the grade difference.

In the latter situation, it may be necessary to construct wing dikes to collect the upstream flow and concentrate it into the constructed channel.

Earth channels with grade control structures. - - This type of protection is particularly satisfactory where existing channels have plenty of capacity.

Energy gradient is controlled by proportioning the weir notch so that the head necessary to operate the weir at design discharge corresponds to the uniform flow depth upstream from the structure, thus defining the water surface profile. In this case, energy dissipation is accomplished at the structures by changing part of the horizontal velocity to vertical and dissipating the vertical velocity head in the plunge pool. Drops that are exceptionally low in relation to tailwater (particularly submerged drops) are likely to be inefficient and their basins must be quite long to accomplish energy dissipation.

Three types of drop spillway structures are commonly used in Service work:

1. Type B Drop Spillway
2. Type C Drop Spillway
3. Box Inlet Drop Spillway

Criteria for the design of Type B and Type C drops may be found in NEH Section 11<sup>4/</sup>. Type B drops have shorter basins, are not so deeply buried, and, consequently, are cheaper where hydraulic conditions permit their use. Type B structures are not designed for submergence or high tailwater to fall relationships.

The Type C structure is more tolerant of high tailwater and submergence, but must be longer and more deeply buried below the downstream channel invert.

Criteria for the design of the Box Inlet drop spillway may be found in SCS-TP-106<sup>22/</sup>. This structure is useful when a long weir crest is needed in a relatively narrow channel, when deep excavation would be difficult and expensive, when a sizable pool in the basin would constitute a health hazard, and on high drops where potential sliding constitutes a stability problem.

Lined channels. - - Rock, concrete or other trapezoidal lining is usually provided for channels at subcritical flow where velocities are sufficiently great that the bottom and banks of an unprotected channel would be unstable. When velocities are supercritical, rock lining will usually be uneconomical because of the large size rock and thick section required for stability.

Hydraulic design procedure is similar to that for unprotected earth channels. Additional depth may be required on curves because of superelevation of the water surface.

Because of the relatively high cost of rock and filter blanket or other lining material, it is best to design for maximum hydraulic radius within limits of available depth, bank stability and reasonable excavation equipment width.

Earth channels with bank protection only. - - This type of protection is used when unprotected banks would be unstable, but the bottom is naturally erosion resistant.

Hydraulic design procedure is the same as for unprotected earth or rock-lined channels except for the composite friction coefficient.

### Supercritical Flow

Hydraulic design procedure for lined channels at supercritical flow may differ quite radically from that for subcritical channels. Here, the principal concern is with capacity and, within limits, the greater the velocity the more economical the cross section, as all supercritical channels not excavated in rock will require lining.

Although the procedure for the selection of trial cross sections, alignment, and grade is similar to that for subcritical channels, final design differs in the following ways:

1. Stability against erosion is no longer a factor, having been accomplished by mechanical protection.
2. Side inlets for small discharges can usually be brought in at any level above channel grade without special protection.
3. Junctions for channels with supercritical flow must be carefully designed. Supplemental model studies may be needed if the proposed design differs radically from junctions on which model study information is already available.
4. Superelevation on curves is an important factor in design.
5. Trapezoidal sections should be avoided on curves.
6. For supercritical flow, water surface profiles must be run in a downstream direction.

Design Procedure. - - Trial cross sections and grades may usually be selected on the basis of uniform flow characteristics. In supercritical reaches it is unlikely that changes will be needed when the water surface profile is computed. Curves will nearly always require additional depth. Superelevation of the water surface may be determined using the methods described below:

1. For rectangular channels at subcritical velocity, or at supercritical velocity where a stable transverse slope has been attained by use of an upstream easement curve (spiral easement or compound curve).

$$S = \frac{3V^2b}{4gR} \quad (\text{See glossary})$$

2. Supercritical velocity - simple curve.

$$s = 1.2 \frac{V^2b}{gR}$$

Location of the first point of maximum depth on the outside wall may be determined by the following formula:

$$\theta = \cos^{-1} \left[ \frac{R - \frac{b}{2}}{R + \frac{b}{2}} \cos \beta \right] - \beta$$

where the terms are as defined in the glossary.

Either spiral easements or compound curves may be employed to reduce superelevation in accordance with the following criteria.

### 3. Compound Curve Criteria

The complete curve is to consist of three sections; a central section with radius  $R_c$  and an approach and terminal section each with a radius  $R_t$  equal to twice  $R_c$ . This produces a superelevation in the zone of the first maximum equal to one-half the normal superelevation produced in a simple curve whose radius is equal to that of the central section.

$$R_t = 2 R_c$$

The length of each transition curve in terms of the central angle

$$\theta_t = \tan^{-1} \frac{b}{R_t \tan \beta}$$

Compound curves shall be used under the following conditions:

When necessary to limit superelevation to one foot allowable maximum.

When two successive curves occur with an intervening tangent less than 1000' in length.

### 4. Spiral Easement Curve

Such easement curves other than the constant radius transition may be used provided that a simple disturbance pattern is produced, and the maximum wave height on the outside wall at the beginning of the curvature of the main curve is equal to:

$$S = \frac{V^2 b}{2gR}$$

Trapezoidal lined channels are not recommended for curved alignment at supercritical flow because of the difficulty in predicting wave run-up on the sloped banks.

Freeboard. - - Minimum freeboard of 0.2 times the depth should be provided for rectangular channels at supercritical flow and 0.25 times the depth for trapezoidal supercritical channels.

Junction structures. - - Design of channel junction structures has not been covered in any standard hydraulic reference. Specific studies have resulted in criteria and guidance on analysis and design of elements of the problem. Model study may be necessary to confirm or refine design of important structures.

Model study of confluences has made evident the need to make the junction with the two flows as nearly parallel as possible. This reduces velocity and momentum components (which cause waves normal to the direction of combined flow) to a minimum.

Having accomplished this, application of the momentum principle to the junction will provide a reliable analysis for design. Further guidance for junction design is contained in Fig. 5-1<sub>3</sub> and "Hydraulic Model Studies for Whiting Field Naval Air Station," 23/ prepared by the Soil Conservation Service.

Channel entrance. - - Some type of structure with weir control is needed at the entrance to the lined section. Such a structure may be either the straight weir or folded weir type. Folded weirs require a drop to prevent submergence.

Transitions. - - Simple transitions in bottom width of rectangular channels may usually be handled by limiting the angle between the transition walls to ten degrees or less. Transitions between rectangular and trapezoidal sections, particularly where the trapezoidal section is in earth, are more complex. The attached paper on transitions (see Appendix I) has been used in the design of several transitions that function satisfactorily.

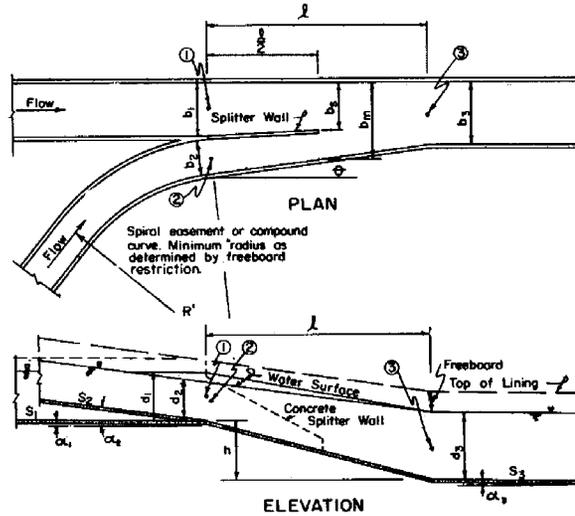
Channel outlet. - - The SAF Basin is the most satisfactory outlet structure where rectangular R/C channels at supercritical flow discharge into an earth section. It is also permissible to use a R/C transition to a trapezoidal rock-lined section. The rock lining should extend a sufficient distance downstream at zero or very low gradient that velocities are reduced to those permissible in the earth materials in the bed and banks of the channel.

Bridges and Culverts. - - Crossings over relatively narrow, rectangular R/C channels are easily accomplished with R/C single span box culverts. These, when bottom slab and sidewalls match those of the channel, have no hydraulic effect. This is also true for clear span bridges over rectangular or trapezoidal channels. Losses caused by bridge piers or interior walls of multiple cell box culverts may be determined by the momentum method. (See Appendix II)



**NOMENCLATURE**

- A = cross-sectional area of water prism ft.<sup>2</sup>
  - b = base width of channel section ft.
  - c = constant - see Chart A --
  - d = vertical depth of water ft.
  - g = acceleration due to gravity (32.2) fps.<sup>2</sup>
  - h = difference in invert elevation between any two sections ft.
  - l = length of channel reach ft.
  - n = coefficient of roughness in Manning's formula --
  - P<sub>m</sub> = average wetted perimeter between two sections ft.
  - P<sub>m</sub> = wetted perimeter at middle section cfs.
  - Q = rate of discharge ft.<sup>3</sup>
  - R<sub>m</sub> = average hydraulic radius between two sections ft.
  - R<sub>m</sub> = hydraulic radius at middle section ft.
  - R = radius of curve to centerline ft.
  - S = critical slope of channel invert --
  - S<sub>c</sub> = channel invert slope --
  - V<sub>a</sub> = average velocity between two sections fps.
  - V<sub>m</sub> = velocity at middle section fps.
  - θ = side slope of trapezoidal section (horizontal to vertical) --
  - θ = angle of merging channels deg.
  - α = invert slope angle deg.
  - F = Froude number --
- Numerical subscripts refer to appropriate channel members.



**1. TRAPEZOIDAL CHANNELS**

Constant Width Main Channel  $\frac{Q_1^2}{gA_1^3} \cos^2 \alpha_1 + \left(\frac{b_2}{2} + \frac{Sd_2}{3}\right) \left(d_2^2\right) \cos^2 \alpha_2 = \frac{Q_1^2}{gA_1} \cos \alpha_1 + \frac{Q_2^2}{gA_2} \cos \alpha_2 \cos \theta + \left(\frac{b_1}{2} + \frac{Sd_1}{3}\right) \left(d_1^2\right) \cos^2 \alpha_1 + A_m h - \frac{P_m l n^2 V_m^4}{2.21 R_m^{1/3}}$

Unequal Width Main Channel  $\frac{Q_1^2}{gA_1} \cos \alpha_1 + \left(\frac{b_2}{2} + \frac{Sd_2}{3}\right) \left(d_2^2\right) \cos^2 \alpha_2 = \frac{Q_1^2}{gA_1} \cos \alpha_1 + \frac{Q_2^2}{gA_2} \cos \alpha_2 \cos \theta + \left(\frac{b_1}{2} + \frac{Sd_1}{3}\right) \left(d_1^2\right) \cos^2 \alpha_1 + (b_2 - b_1) d_m h - \frac{P_m l n^2 V_m^4}{2.21 R_m^{1/3}}$

Note: For flows at supercritical velocities these equations may be used only for preliminary design. Final design must be based on results of supplemental hydraulic model studies.

**2. RECTANGULAR CHANNELS**

Constant Width Main Channel  $\frac{Q_1^2}{gA_1^3} \cos^2 \alpha_1 + \frac{b_2 d_2^2}{2} \cos^2 \alpha_2 - \frac{b_2 d_2 h}{2} = \frac{Q_1^2}{gA_1} \cos \alpha_1 + \frac{Q_2^2}{gA_2} \cos \alpha_2 \cos \theta + \frac{b_1 d_1 h}{2} + \frac{b_2 d_2^2}{2} \cos^2 \alpha_1 - \frac{P_m l n^2 V_m^4}{2.21 R_m^{1/3}}$

Unequal Width Main Channel  $\frac{Q_1^2}{gA_1} \cos \alpha_1 + \frac{b_2 d_2^2}{2} \cos^2 \alpha_2 - \frac{b_2 d_2 h}{2} = \frac{Q_1^2}{gA_1} \cos \alpha_1 + \frac{Q_2^2}{gA_2} \cos \alpha_2 \cos \theta + \frac{b_1 d_1 h}{2} + \frac{b_2 d_2^2}{2} \cos^2 \alpha_1 + \frac{(b_2 - b_1) d_m^2}{2} \cos^2 \alpha_2 - \frac{P_m l n^2 V_m^4}{2.21 R_m^{1/3}}$

**SPECIAL CASE**

Limiting Criteria - Rectangular Channels  
Supercritical Velocity Flow in All Branches.

Constant Width Main Channel  $\frac{Q_1^2}{gA_1} + \frac{b_2 d_2^2}{2} - \frac{b_2 d_2 h}{2} = \frac{Q_1^2}{gA_1} + \frac{Q_2^2}{gA_2} + \frac{b_1 d_1 h}{2} + \frac{b_2 d_2^2}{2} - \frac{P_m l n^2 V_m^4}{2.21 R_m^{1/3}}$

Unequal Width Main Channel  $\frac{Q_1^2}{gA_1} + \frac{b_2 d_2^2}{2} - \frac{b_2 d_2 h}{2} = \frac{Q_1^2}{gA_1} + \frac{Q_2^2}{gA_2} + \frac{b_1 d_1 h}{2} + \frac{(b_2 - b_1) d_m^2}{2} - \frac{P_m l n^2 V_m^4}{2.21 R_m^{1/3}}$

**Criteria**

$\tan \alpha < 0.1$

Convergence angle θ less than 10°

$F = \frac{V}{\sqrt{gd}} \geq 1.2$

$\theta \leq 10^\circ$

$1.2 \geq \frac{d_1}{d_2} \geq 0.8$

Height of sidewall at ① =  $1.2 d_3 + 0.25 d_{c3} \left[ 1 - 11.1 \left( \frac{S_3}{S_{c3}} - 1 \right)^2 \right] + 1.2 \frac{V_3^2 b_2}{gR_2}$

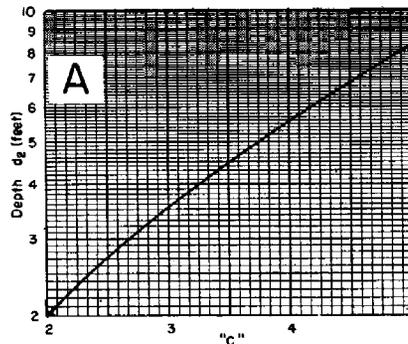
Height of splitter wall  
upstream =  $d_{1,2}$  max.  
downstream = 6" min.

$b_3 = \frac{b_1 (b_1 + b_2 + b_3)}{2(b_1 + b_2)}$  when  $b_1 > b_2$

$l = cV_2 [b_1 + b_2 - b_3]^{1/3}$

$b_3 = \frac{b_2 (b_1 + b_2 + b_3)}{2(b_1 + b_2)}$  when  $b_1 < b_2$

$b_3 \geq 0.8 (b_1 + b_2)$



**FIGURE 5-1  
HYDRAULICS JUNCTION STRUCTURES**

REFERENCE: FLOW ANALYSIS AT OPEN CHANNEL JUNCTIONS; U.S.C.E. - L.A. 1947



APPENDIX I TO CHAPTER 5  
ON  
CHANNEL PLANNING AND DESIGN  
TRANSITIONS

Important Transitions Where it is Necessary to Conserve Head

Circumstances requiring a change in channel section occur frequently. The crossing of roads, the changing of grades, and many topographic conditions provide situations where acceleration or deceleration of flow is necessary to meet required changes of cross section.

Adequate design of transition structures to provide for gradual changes of the flow section is important because at such places the capacity of the whole system is frequently determined. Poor transition design not only vitiates good channel construction but may cause undesirable backwater effects.

Although transition design is based on the Bernoulli and Continuity equations, experience plays a very significant part. Model studies and observations of many actual structures have indicated several rules to be followed. They are:

1. The water surface should be smoothly transitioned to meet end conditions.
2. The water surface edges should not at any section converge at an angle greater than  $28^{\circ}$  with the center line, nor diverge at an angle greater than  $25^{\circ}$ .
3. In well designed transitions, losses in addition to friction should not exceed  $.10 h_v$  for convergence and  $.20 h_v$  for divergence.
4. In general it is desirable to have bottom grades and side slopes meet end conditions tangentially. (Usually this rule must be violated when critical depths are approached going from an earth channel to a lined ditch.)

To outline the specific steps in designing a transition, it is desirable to reduce Bernoulli's equation to a more convenient form. Equation (5.1-1) reads

$$\frac{v_1^2}{2g} + \frac{P_1}{W} + z_1 = \frac{v_2^2}{2g} + \frac{P_2}{W} + z_2 + h_f \quad (\text{Eq. 5.1-1})$$

If the flow takes place on sufficiently flat slopes so that  $P$  equals  $WY$ , then the terms  $\frac{P}{W} + z$  are exactly equal to the elevation of the water surface for all elements. If a fall of surface downstream is taken as positive, Bernoulli's equation becomes:

$$\Delta W.S. = \Delta h_v + h_f + \text{impact} \quad (\text{Eq. 5.1-2})$$

For simplicity, the term "impact" is used as a measure of losses due to change in direction of stream lines in both converging and diverging transitions. In a converging transition, these losses are truly due to impact as stream lines impinge against the converging walls. In a diverging transition, losses are caused primarily by eddy currents resulting from negative pressures along the diverging walls.

In relatively short transition structures the ordinary friction losses given by the Manning formula are small compared with the impact losses and the head loss in the transition is very nearly  $.10\Delta h_v$  for inlets and  $.20\Delta h_v$  for outlets. Equation (5.1-2) reduces finally to  $\Delta W.S.$  equals  $1.10\Delta h_v$  for inlets and  $\Delta W.S.$  equals  $.80\Delta h_v$  for outlets. Friction loss must be considered for long transitions and for velocities in excess of 20 fps.

These simple relationships assisted by the continuity equation

$$Q = A_1 V_1 = A_2 V_2 \quad (\text{Eq. 5.1-3})$$

form the basis of all transition design. Detailed steps to be followed in the computation follow.

1. From the quantity of flow compute the velocities and velocity heads at the end sections.
2. Compute the overall change in water surface from

$$\Delta W.S. \text{ equals } 1.10 \Delta h_v \text{ (inlets)}$$

$$\Delta W.S. \text{ equals } .80 \Delta h_v \text{ (outlets)}$$

(neglecting ordinary friction loss)

3. Construct a smooth curve to represent the water surface having the computed change in elevation and tangent to the surfaces at the ends. Two reverse parabolic curves are good. However, any smooth curve can be used.

4. Mark this surface curve at 6 to 10 stations (depending on the size of the structure), and tabulate the total  $\Delta$  W.S. from the beginning of the transition to each station. The distance between these stations is assumed at first and then adjusted until the desired conditions are reached.
5. Compute  $\Delta h_v$  from  $\frac{\Delta \text{ W.S.}}{1.1.0}$  or  $\frac{\Delta \text{ W.S.}}{0.80}$  for each station and evaluate each  $h_v$  and  $V$ .
6. From  $Q$  equals  $AV$ , obtain the cross sectional area required at each section.
7. Assuming a bottom grade line to meet end conditions, list depths at each station.
8. Evaluating the average width from  $\frac{A}{d}$ , select side slopes to make the water surface converge smoothly according to the requirements of rules 2 and 4. If this cannot be done by adjustment of side slopes along, the transition may be lengthened, the bottom grade line changed or the water surface may be varied. A juggling of these controls will finally produce the required results.

Care should be taken in designing the transition when the velocity goes through critical. On inlet transitions the bottom should be raised gradually until the critical depth is reached and just beyond it should drop as fast as possible. In this way the critical depth is very unstable and unless this is done the critical depth may not come at the computed location and cause improper loading in the channel below.

It is possible to design an outlet transition from a subcritical to a super critical depth without going through a hydraulic jump, but it is better to avoid this condition if possible.

9. To allow for the friction loss, compute  $P$  and  $R$  for each section and compute the rate of friction loss,  $f$ , for flow at each section from

$$f = \frac{n^2 v^2}{2.208R}^{4/3} \quad (\text{Eq. 5.1-4})$$

10. Then the friction loss between any two sections equals the average value of  $f$  for the two sections times the length between sections.

5.1-4

11. Then the water surface and bottom at each section must be dropped an amount equal to the summation of the values found in step 10. (From the beginning to the point in question.)

These steps are illustrated in the following examples.

Design an inlet transition from an earth channel, with a discharge of 314 C.F.S., bottom width 18.0 feet, depth 4.30 feet, side slopes 2:1, area 114.40 square feet and a velocity of 2.75 feet per second, to a rectangular concrete lined channel, with a width of 12.5 feet, depth 4.220 feet, area 52.70 square feet and a velocity of 5.97 feet per second.\*

Critical depth is not involved in this transition.

Line	Item											
		**0 + 00	0 + 05	0 + 10	0 + 15	0 + 20	0 + 25	0 + 30	0 + 35	0 + 40	0 + 45	0 + 50
1	$\Delta$ WS = Drop in WS		0.010	0.038	0.086	0.154	0.240	0.326	0.394	0.442	0.470	0.480
2	$\Delta$ $h_v = WS \div 1.1$		0.009	0.035	0.079	0.140	0.218	0.296	0.357	0.401	0.427	0.436
3	$h_v = 0.117 + \Delta h_v$		0.126	0.152	0.196	0.257	0.335	0.413	0.474	0.518	0.544	0.553
4	V	2.75	2.85	3.13	3.55	4.07	4.64	5.15	5.52	5.77	5.91	5.97
5	Area = $Q \div V$	114.40	110.50	100.60	88.75	77.40	67.88	61.20	57.10	54.60	53.30	52.70
6	0.5T=Half width at WS	17.600	17.000	15.427	13.460	11.228	9.139	7.717	6.847	6.458	6.315	6.25
7	0.5B=Half bottom width	9.000	8.625	7.917	7.250	6.958	6.771	6.667	6.563	6.458	6.315	6.25
8	0.5T+0.5B=Average width	26.600	25.625	23.344	20.710	18.186	15.910	14.384	13.410	12.916	12.630	12.500
9	d = Area $\div$ Ave. width	4.30	4.309	4.310	4.280	4.260	4.264	4.252	4.253	4.225	4.220	4.220
10	f = Friction slope	0.00015	0.00017	0.00020	0.00026	0.00034	0.00046	0.00061	0.00076	0.00083	0.00087	0.00090
11	$h_f = 5f$ (Use ave. f)		0.00080	0.00090	0.00115	0.00150	0.00200	0.00270	0.00345	0.00400	0.00425	0.00445
12	$\sum h_f$		0.00080	0.00170	0.00285	0.00435	0.00635	0.00905	0.01250	0.01650	0.02075	0.02520
13	WS Elev = 57.41 - $\Delta$ WS - $\sum h_f$	57.410	57.399	57.370	57.321	57.252	57.164	57.075	57.003	56.951	56.919	56.905
14	Grade = WS Elev - d	53.110	53.090	53.060	53.041	52.992	52.900	52.823	52.750	52.726	52.699	52.685
15	0.5 T - 0.5 B	8.600	8.375	7.510	6.210	4.270	2.368	1.050	0.284			
16	Side slopes	2.000	1.945	1.744	1.447	1.000	0.554	0.247	0.067			
17	Height of lining = H	5.330	5.295	5.270	5.234	5.228	5.265	5.287	5.305	5.274	5.236	5.205
18	0.5W-0.5B=Side slope xH	10.660	10.310	9.210	7.575	5.228	2.920	1.305	0.354			
19	0.5 W = 0.5 top width	19.660	18.935	17.127	14.825	12.186	9.691	7.972	6.917	6.458	6.315	6.250
20	0.5 W to nearest 1/2 in.	19' 8"	18' 11"	17' 11/2"	14' 10"	12' 2"	9' 8 1/2"	7' 11 1/2"	6' 11"	6' 5 1/2"	6' 4"	6' 3"

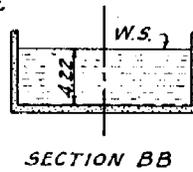
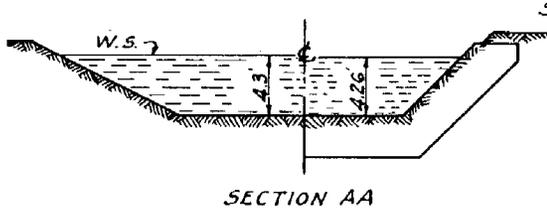
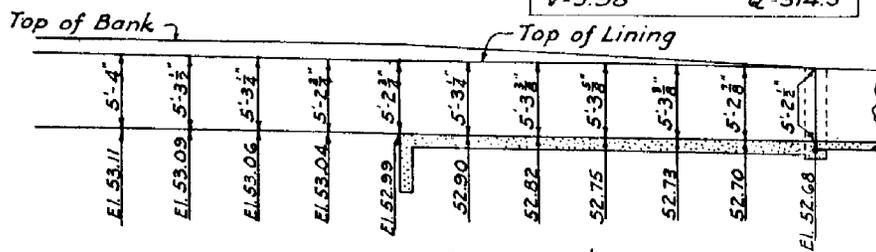
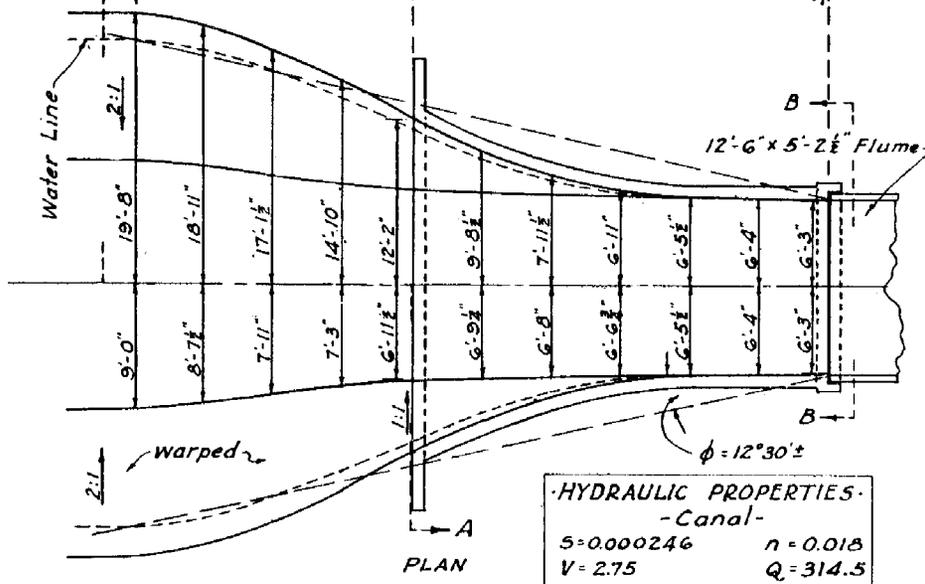
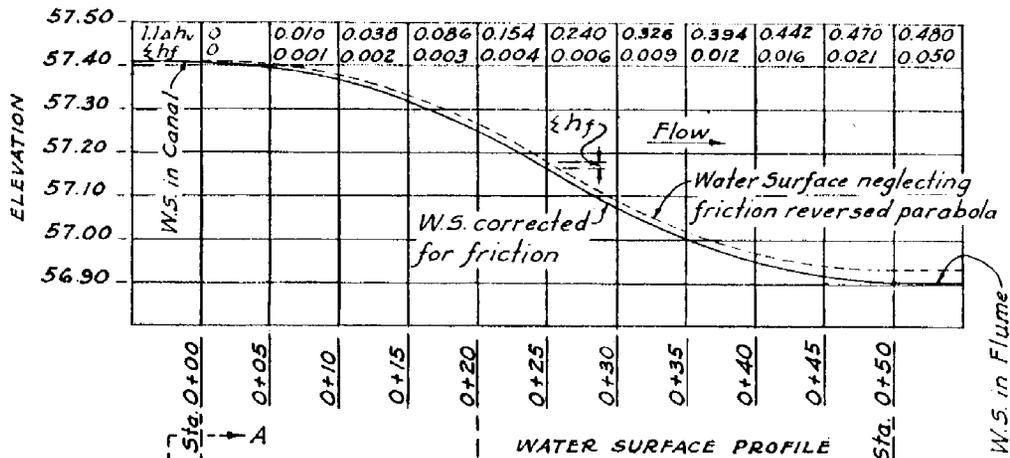
\* This transition was taken from "Design of Important Transitions, not Involving Critical Flow or the Hydraulic Jump" by Julian Hinds, from "Transactions of the American Society of C.E." Vol. 92, Page 1430 et seq.

\*\* In this case the stations are given in distances direct.



FIGURE 5.1-1

TYPICAL TRANSITION TO RECTANGULAR FLUME





Design an inlet transition from an earth channel with a discharge of 200 C.F.S., bottom width 10.0 feet, depth 3.68 feet, side slopes 1-1/2:1, area 57.11 square feet and a velocity of 3.5 feet per second to a rectangular concrete lined channel with a width of 5.17 feet, depth 2.58 feet, area 13.34 square feet and a velocity of 15.0 feet per second.

In this example the depth is greater than the critical in the earth channel and less than critical in the concrete lined channel. In such a transition the bottom should have a slope less than the critical up to the point of critical depth, at this point it should break sharply to a slope greater than critical. In this example the critical depth is at sta. 6 or 20 feet from the earth channel.

Line	Item	Station									
		*1	2	3	4	5	6	7	8	9	10
1	$\Delta W.S. = 1.1 h_v$	0.00	0.030	0.080	0.21	0.45	0.96	1.98	3.20	3.58	3.64
2	$\Delta h_v = \Delta W.S. \div 1.1$	0.00	0.027	0.073	0.191	0.409	0.873	1.800	2.910	3.255	3.310
3	$h_v = 0.190 + \Delta h_v$	0.190	0.217	0.263	0.381	0.600	1.063	1.999	3.100	3.445	3.500
4	$V = \sqrt{2 gh_v}$	3.50	3.74	4.11	4.95	6.21	8.27	11.31	14.12	14.89	15.00
5	$A = Q/V$	57.14	53.48	48.66	40.40	32.21	24.18	17.68	14.16	13.43	13.33
6	0.5 T (Measured)	10.52	10.18	9.52	8.32	7.00	5.755	4.40	3.48	2.87	2.585
7	0.5 b (Measured)	5.00	4.80	4.44	3.94	3.40	2.975	2.73	2.60	2.58	2.585
8	Average Width = (.5 T + .5b)	15.52	14.98	13.96	12.26	10.40	8.73	7.13	6.08	5.45	5.17
9	d = A/Av. Width	3.68	3.57	3.48	3.30	3.10	2.77	2.48	2.33	2.46	2.58
10	**f=Friction Slope= $n^2 v^2 \div 2208R^{4/3}$	.00028	.00034	.00043	.00068	.00119	.00249	.00541	.00985	.01206	.0123
11	$h_f = 4$ (Average f)		.00124	.00154	.00220	.00372	.00736	.01580	.03052	.04384	.04872
12	$\xi h_f$		.00124	.00278	.00498	.00870	.01606	.03186	.06238	.10622	.15494
13	W.S. Elev.=103.68- $\Delta W.S. - \xi h_f$	103.68	103.649	103.597	103.465	103.221	102.704	101.668	100.418	99.994	99.835
14	Grade = W.S. Elev. - d	100.00	100.079	100.117	100.165	100.121	99.934	99.188	98.088	97.534	97.305
15	Side Slopes	1.5:1	1.51:1	1.46:1	1.33:1	1.16:1	1.01:1	0.66:1	0.38:1	0.12:1	Vertical
16	Height of Lining H	4.18	4.07	3.98	3.80	3.60	3.27	2.98	2.83	2.96	3.03
17	.5W - .5b = side slopes x H	6.27	6.12	5.81	5.05	4.18	3.28	1.96	1.10	0.35	0.0
18	.5W = .5 top width of lining	11.27	10.92	10.25	8.99	7.58	6.26	4.69	3.70	2.93	2.585

\* By plotting it was found a distance of 4 ft. between stations fit the desired conditions.

\*\* n equals 0.013







Design an outlet transition from a concrete lined channel with a discharge of 200 C.F.S., bottom width 2.00 feet, depth 3.00 feet, side slopes 1 to 1, area 15.00 square feet and a velocity of 13.33 feet per second to another concrete lined channel, bottom width 6.00 feet, depth 2.25 feet, side slopes 1 to 1, area 18.57 square feet and a velocity of 10.77 feet per second, both depths being subcritical.

Due to the high velocities involved, the transition should not diverge greater than about 1 to 10 for each side. If in proportioning the side and bottom it is necessary to change the surface curve, it should be borne in mind that the velocity head changes rapidly with a small change in velocity for high velocities.

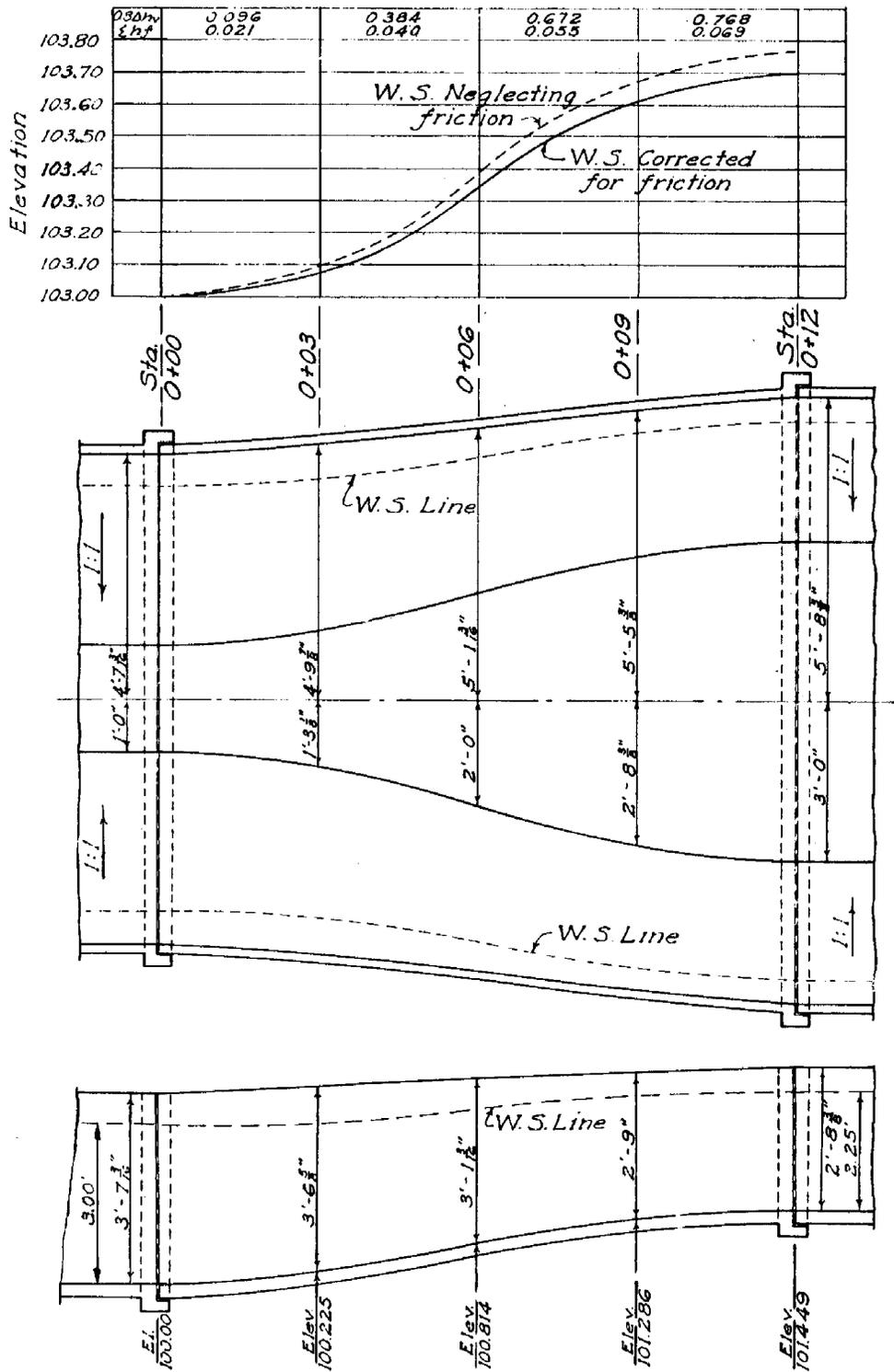
Line	Item	Station				
		*1	2	3	4	5
1	$\Delta$ W.S. = Rise in W.S.		0.096	0.384	0.672	0.768
2	$\Delta h_v = \Delta$ W.S. $\div$ 0.80		0.120	0.480	0.840	0.960
3	$h_v = 2.76 - \Delta h_v$	2.760	2.640	2.280	1.920	1.800
4	$V = \sqrt{2gh}$	13.33	13.09	12.11	11.11	10.77
5	Area = $Q \div V$	15.00	15.34	16.52	18.00	18.57
6	d scaled	3.00	2.85	2.53	2.33	2.25
7	Average width = $A \div d$	5.00	5.38	6.53	7.73	8.25
8	.5 T = Half top width	4.00	4.11	4.53	5.03	5.25
9	.5 b = Half bottom width	1.00	1.26	2.00	2.70	3.00
10	**f = Friction slope	.0072	.0068	.0057	.0047	.0044
11	$h_f = 3$ (Average f)		.0210	.0188	.0156	.0137
12	$\sum h_f =$		.021	.040	.055	.069
13	W.S. El. = $103 + \Delta$ W.S. - $\sum h_f$	103.00	103.075	103.344	103.616	103.699
14	Grade = W.S. El. - d	100.00	100.225	100.814	101.286	101.449
15	Side Slopes	1.0	1.0	1.0	1.0	1.0
16	Height of lining = H	3.60	3.55	3.10	2.75	2.70
17	.5W - .5B = side slopes x H	3.60	3.55	3.10	2.75	2.70
18	.5W = .5 top width of lining	4.60	4.82	5.10	5.45	5.70

\* By plotting it was found that a distance of 3 ft. between stations fit the desired conditions.

\*\* n equals 0.012



FIGURE 5.1-3  
TYPICAL OUTLET TRANSITION



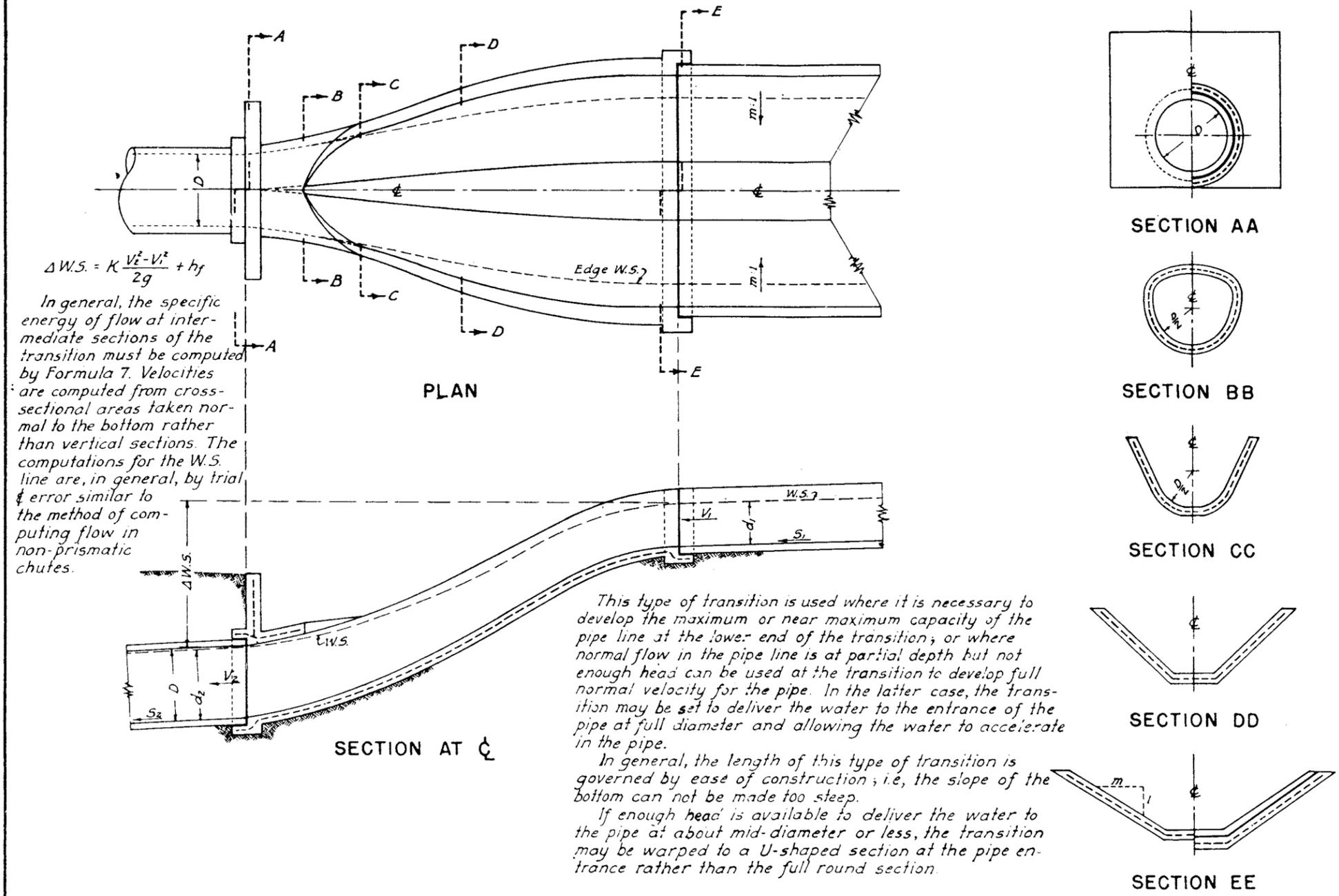
HYDRAULIC PROPERTIES

Q = 200 c.f.s.

Upstream		Downstream
s = 0.0072 %	n = 0.012	s = 0.0044 %
V = 13.33'/sec.	h <sub>v</sub> = 2.760'	V = 10.77'/sec.
		h <sub>v</sub> = 1.800'



FIGURE 5.1-4  
TYPICAL INLET TRANSITION TO PIPE LINE



O - 78 (Face blank p. 128)

Less Important Transitions

This type of design is used when head is not at a premium.

The elevation of the water surface at each end is known. No attempt is made to trace out the water surface curve at intermediate points. The sides are straight lines and can be made vertical when going from an earth channel to a rectangular or circular section and vice versa. If the side slopes of the two sections are different, they should be gradually warped to meet the end conditions. The bottom should be laid in tangent to the grade at each end.

In the absence of more specific knowledge the length of the transition should be such that a straight line joining the flow line at the two ends of the transition will make an angle of about  $12\ 1/2^\circ$  with the axis of the structure.

Neglecting friction the losses can be taken as  $0.15 \Delta h_v$  for inlet, and  $0.25 \Delta h_v$  for outlet transitions.

In transitioning from an earth channel to a lined channel with a velocity greater than the critical, the earth channel should be contracted at the entrance to the transition sufficient to develop critical depth, and not develop scouring velocities above. The bottom of the transition should drop rapidly from the entrance and connect tangent to the grade on the channel below.

The procedure in designing such a transition is:

1. Compute the length.
2. Compute the change in water surface from:

$$\text{W.S. equals } 1.15 \Delta h_v \text{ (inlets)}$$

$$\text{W.S. equals } 0.75 \Delta h_v \text{ (outlets)}$$

(neglecting ordinary friction loss)

To illustrate this procedure let it be required to design a transition from an earth channel carrying 100 second feet, bottom width 12.6 feet, depth of water 2.1 feet, total depth 2.6 feet, side slopes 1-1/2 to 1 and an average velocity of 3.0 feet per second to a concrete lined channel with a bottom width of 3.0 feet, depth of water 1.72 feet, total depth 2.0 feet, side slopes 1-1/2 to 1 and an average velocity of 10.43 feet per second.

The normal depth in the concrete channel of 1.72 feet is less than the critical, therefore it is necessary to develop a velocity greater than the critical. The earth channel should be contracted to develop critical depth, without an excess drawdown effect in the earth channel above. In the earth channel under consideration it is necessary to contract the bottom to a width of 8.5 feet, the side slopes being 1-1/2 to 1.

$$d_c \text{ equals } 1.44 \text{ feet}$$

$$V_c \text{ equals } 6.5 \text{ feet per second}$$

1. Length of transition

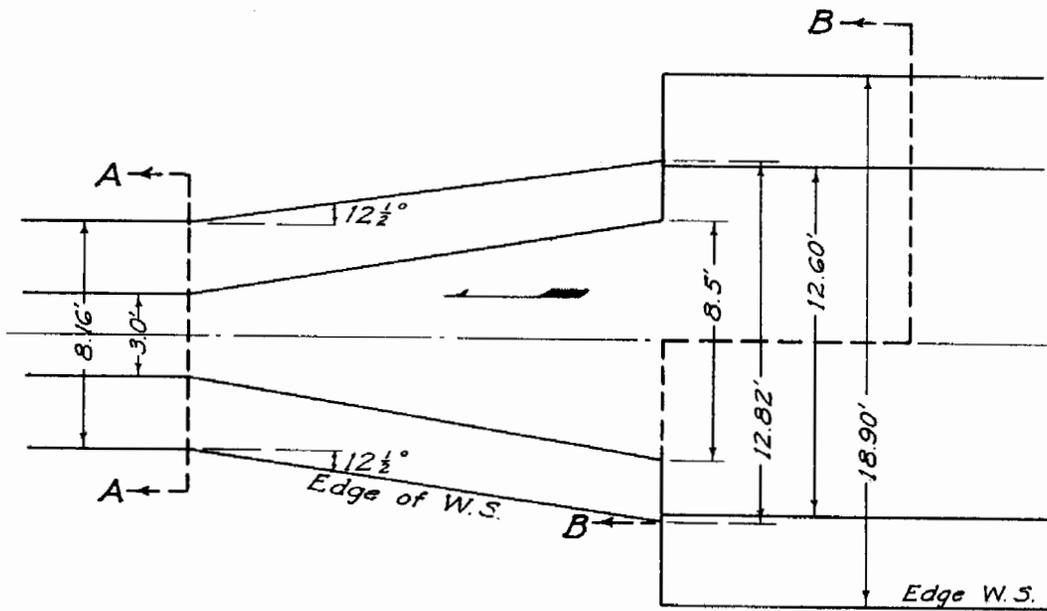
$$2.33 \times \cot 12-1/2^\circ \text{ equals } 2.33 \times 4.51 \text{ equals } 10.5 \text{ feet say } 10.0 \text{ ft.}$$

2. W.S. equals 1.15 ( $\Delta h_v$ )

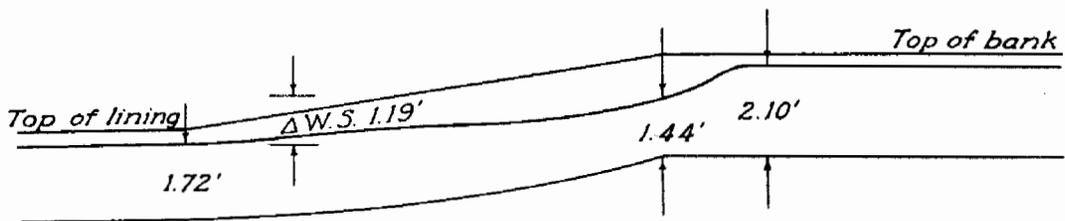
$$\text{equals } 1.15 (1.035) \text{ equals } 1.19 \text{ feet (See Fig. 5.1-5)}$$

FIGURE 5.1-5

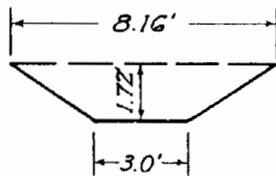
TYPICAL TRANSITION WITH STRAIGHT SIDES



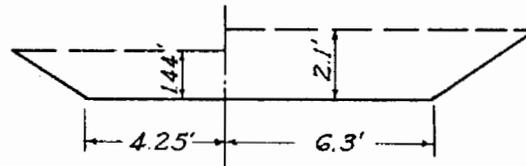
Plan Showing Bottom & Lines at W.S.



Side Elevation Showing Bottom & Approx. W.S.



Section A-A



Section B-B



## APPENDIX II TO CHAPTER 5

### MOMENTUM METHOD OF DETERMINING BRIDGE PIER LOSS \*

Flow past an obstruction has been divided into three types which follow roughly "Class A and B" flow as defined by Yarnell, and "Class C" flow as indicated by Yarnell and defined herein. The definitions as given by Koch and Carstanjen for the three flow conditions follow:

"Class A" flow is defined as a flow condition whereby critical flow within the constricted bridge section is insufficient to produce the momentum required downstream. It is apparent that for this type of flow, the bridge section is not a "control point" and, therefore, the upstream water depth is controlled by the downstream water depth plus the total losses incurred in passing the bridge section.

"Class B" flow is defined as a flow condition whereby critical flow within the constricted bridge section produces or exceeds the momentum required downstream. When this condition exists, the upstream water depth is independent of the downstream water depth, being controlled directly by the critical momentum required within the constricted bridge section and the entrance losses.

"Class C" A special form of "Class B" flow occurs when the upstream water is flowing at a subcritical depth and containing sufficient momentum to overcome the entrance losses and produce a super-critical velocity within the constricted bridge section.

The drawing on the following page, entitled "Bridge Pier Losses by the Momentum Method" shows the water surface profiles and momentum curves for the three classes of flow.

Momentum, as referred to above, is defined as total momentum or the total of static and kinetic momentum, and may be written as  $m + \frac{Q^2}{gA}$

where m = total static pressure of the water at a given section in pounds

Q = discharge in cubic feet per second

g = acceleration of gravity in feet per second per second

A = channel cross-sectional area in square feet.

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\* Data derived from "Report of Engineering Aspects, Flood of March 1938, Los Angeles, California," - Appendix I, Theoretical and Observed Bridge Pier Losses - U. S. Engineer's Office, Los Angeles, California, - May 1949 and from "Approximate Method Determines Bridge Pier Loss," by G. M. Allen, Jr., in March 1953, Civil Engineering.

The unit weight of water ( $w$ ) should appear in each term, but since it would cancel in the final equations, it has been assumed equal to unity, dimensions being pounds per cubic foot.

Based on experiments under all conditions of open channel flow where the channel was constricted by short flat surfaces perpendicular to flow, such as bridge pier, Koch and Carstanjen found that

the total kinetic loss was equal to  $\frac{A_0 Q^2}{A_1 g A_1}$  where  $A_0$  is the area of

the obstruction on the upstream surface and  $A_1$  is the water area in the upstream unobstructed channel. For circular nose piers, Koch

and Carstanjen show that  $2/3$  of  $\frac{(A_0 Q^2)}{(A_1 g A_1)}$  should be used. It is

apparent that the static pressure  $m_0$  against the upstream obstructed area is not effective downstream, whereas the static pressure against the downstream obstructed area is effective downstream. Therefore, if we let the subscripts 1, 2, and 3 represent conditions upstream, within and downstream of the constricted section, respectively, we may write the general momentum relationship as follows:

Total upstream momentum minus the momentum loss at entrance must equal the total momentum within the constricted section, or

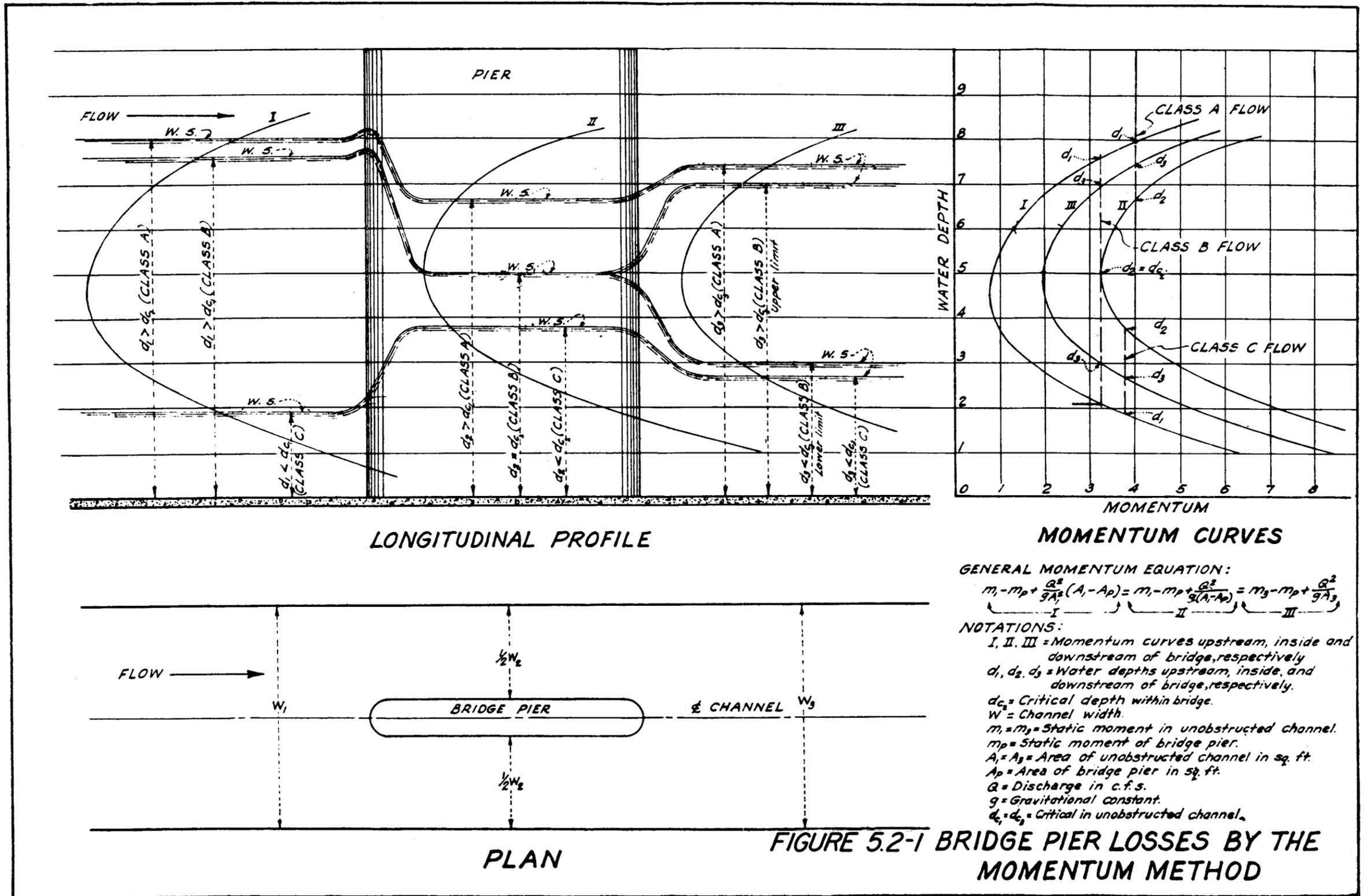
$$m_1 - m_0 + \frac{Q^2}{g A_1} - \frac{A_0}{A_1} \frac{Q^2}{g A_1} = m_2 + \frac{Q^2}{g A_2}, \text{ or}$$

$$m_1 - m_0 + \frac{Q^2}{g A_1^2} (A_1 - A_0) = m_2 + \frac{Q^2}{g A_2}$$

Total momentum within the constricted section plus static pressure on the downstream obstructed area must equal the total momentum in the downstream channel, or

$$m_2 + \frac{Q^2}{g A_2} + m_0 = m_3 + \frac{Q^2}{g A_3}, \text{ or}$$

$$m_2 + \frac{Q^2}{g A_2} = m_3 - m_0 + \frac{Q^2}{g A_3}$$



**GENERAL MOMENTUM EQUATION:**

$$m_1 - m_p + \frac{Q^2}{gA_1^3}(A_1 - A_p) = m_2 - m_p + \frac{Q^2}{g(A_1 - A_p)} = m_3 - m_p + \frac{Q^2}{gA_3}$$

**NOTATIONS:**

- I, II, III = Momentum curves upstream, inside and downstream of bridge, respectively
- $d_1, d_2, d_3$  = Water depths upstream, inside, and downstream of bridge, respectively.
- $d_c$  = Critical depth within bridge.
- $W$  = Channel width.
- $m_1, m_3$  = Static moment in unobstructed channel.
- $m_p$  = Static moment of bridge pier.
- $A_1, A_3$  = Area of unobstructed channel in sq. ft.
- $A_p$  = Area of bridge pier in sq. ft.
- $Q$  = Discharge in c.f.s.
- $g$  = Gravitational constant.
- $d_c, d_c$  = Critical in unobstructed channel.

The general momentum equation follows:

$$m_1 - m_0 + \frac{Q^2}{gA_1^2} (A_1 - A_0) = m_2 + \frac{Q^2}{gA_2} = m_3 - m_0 + \frac{Q^2}{gA_3}$$

The above equations cannot be solved as presented, and it is necessary that a simpler method be used. The total sum of momentum and hydrostatic pressure for each section (Fig. 5.2-3) for equal depths of flow past each section should first be determined. Using equal depths,  $A_1 - A_0 = A_2 = A_3 - A_0$  (or  $A_1 = A_3$ ) and  $M_1 - M_0 = M_2 = M_3 - M_0$  (or  $M_1 = M_3$ ).

Also, for equal depths, the sum of momentum and hydrostatic pressure for each section I, II, and III is:

$$I = M_1 - M_0 + \frac{Q^2 (A_1 - A_0)}{gA_1^2}$$

$$II = M_1 - M_0 + \frac{Q^2}{g (A_1 - A_0)}$$

$$III = M_1 - M_0 + \frac{Q^2}{gA_3} = M_1 - M_0 + \frac{Q^2}{gA_1}$$

where, for equal depths,  $A_1 = A_3$

The values for equations I, II, and III are determined for various depths, both subcritical and supercritical. A curve for each section is plotted using the depth as the ordinate and the values from columns I, II, and III as the abscissa (see Fig. 5.2-4). A vertical line passed through the three curves gives a graphic solution of the equations, as it gives, for equal momentum, the corresponding depths of flow.

This vertical line must intersect a minimum of five depth values, and preferably six. Drwg. No. 7-N-Eng. 248, page 3, shows the depth of flow and its indicated class. If only one value is intersected on curve II, the flow is critical at Section II.

Values of "d" on the lower portions of Curves I and III are used for supercritical flow, and on the upper portions for subcritical flow.

Backwater computations will determine either a flow depth at Section I or III, depending upon type of flow conditions, and the curves give a direct solution, as the vertical line must pass through the known depth on Curve I or III, and must also pass through Curve II. If this vertical line does not pass through Curve II, it is possible that the momentum of the given depth is not great enough for the flow to pass the obstruction, and a change in the computed depth must be made. The flow would then be critical at Section II, as the critical depth is the depth at which the momentum and pressure is the minimum.

Example

Given a trapezoidal channel, base-width 16 feet, side-slopes 1-3/4:1 and capacity  $Q = 5000$  c.f.s.

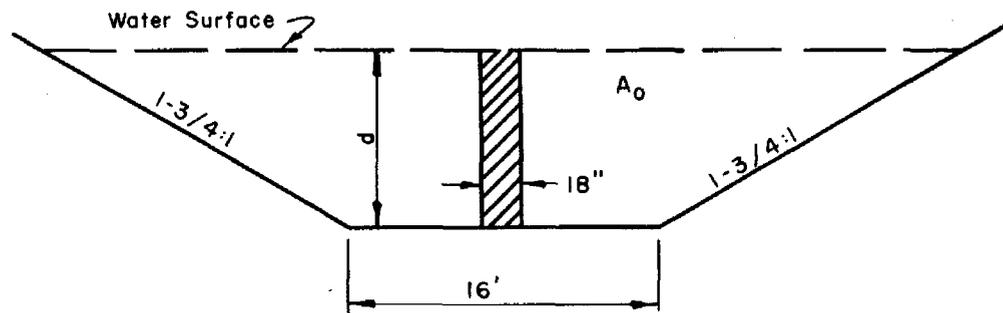


Fig. 5.2-2. Cross-section of channel showing center bridge pier.

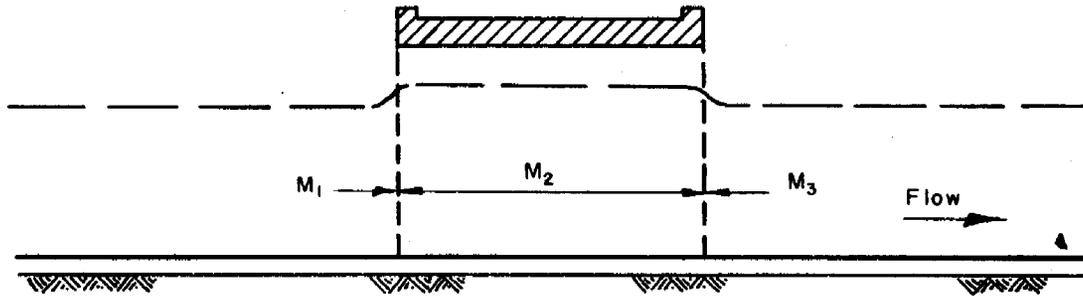


Fig. 5.2-3. Longitudinal section along centerline of channel indicates three locations: I - immediately upstream, II - Under bridge, III - Immediately downstream.

Compute  $M_1 - M_3$  by solving for the distance  $\bar{y}$  from the water surface to the center of gravity of the trapezoidal section where  $\bar{y} = \frac{d(T + 2b)}{3(T + b)}$ .

( $T =$  top width and  $b =$  base width) and multiplying  $\bar{y}$  by the area  $A$ , or ( $M_1 = \bar{y} A_1$ ). The  $\bar{y}$  distance for the obstruction, which is of rectangular area  $A_0$  is  $\frac{d}{2}$ ; this multiplied by  $A_0$ , will give  $M_0$ .

Quantities for the remaining columns can be easily computed by use of the formulas given on pages 5.2-2 and 5.2-5 and in the column headings in Table 5.2-1.

Momentum values given in columns I, II, and III, Table 5.2-1, are shown plotted on the graph, Fig. 5.2-4, giving curves I, II, and III. From a point on Curve I, which represents the upstream depth  $d_1$ , draw a vertical line through Curves II and III which will give the depth values for the sections under the bridge and immediately downstream. For the example given, refer to the curves on Fig. 5.2-4. For an upstream depth of 8.0 feet; the depth under the bridge is 9.3 feet and the depth immediately downstream is 8.7 feet. As the flow upstream is at subcritical depth,  $d_1$  is less than  $d_c$  and "Class C" flow applies (see Fig. 5.2-1).

It is shown in Table 5.2-1 that the bridge section includes an 18-inch wide pier. As debris piles up on the center pier its net

effect is to widen the center pier increasing  $A_0$  and  $M_0$ . The effect of such debris accumulations can be estimated by computing flow conditions for the wider pier that would result when debris had accumulated. In critical cases the effect of debris lodging against piers can be minimized by constructing a 2:1 incline on the upstream edge of the piers. This causes debris to rise toward the surface and widen only a portion of the pier height. It should be noted, however, that the top eight (8) feet are normally considered to be affected by such debris so an inclined leading edge on piers in shallow streams would not be too effective.

The momentum method of computing the approximate change of water surface is not dependent upon coefficients "K" as are necessary in the formulas derived by Nagler, Weisbach, Rehbock and others (see USDA Technical Bulletin No. 429, "Pile Trestles as Channel Obstructions" and USDA Technical Bulletin No. 442, "Bridge Piers as Channel Obstructions").

A check computation for the raise in water surface due to the bridge obstruction was made using the Nagler formula. The coefficient K varies from .87 to .94 with the channel contraction approximately five percent. The difference in water surface elevation between the depth in the unobstructed channel and the depth caused by the obstruction was from 0.7 to 0.8 foot. Difference in depths indicated by the Momentum Method was greater, shown by the curves to be 1.3 feet, and is on the conservative side.

The water surface profile, above and below bridges, can be computed by the standard step method, as described in a report "Technical Memorandum - Water Surface Computation in Open Channels" by R. F. Wong, Los Angeles District Corps of Engineers, and also given in King's Handbook of Hydraulics.

Table 5.2-1 MOMENTUM LOSSES AT BRIDGES										Section			
Project Bull Creek Channel					Locality Bridges over channel								
Q = 5,000 c.f.s.		Sheet 1 of 1			By R.M.J.		Date February 1953						
d	A <sub>1</sub> =A <sub>3</sub>	M <sub>1</sub> =M <sub>3</sub>	A <sub>0</sub>	M <sub>0</sub>	A <sub>1</sub> -A <sub>0</sub>	M <sub>1</sub> -M <sub>0</sub> (1)	$\frac{Q^2}{g(A_1-A_0)}$ (2)	$\frac{Q^2}{g A_1^2}$	$\frac{Q^2(A_1-A_0)}{g A_1^2}$ (3)	$\frac{Q^2}{g A_3}$ (4)	I (1) + (3)	II (1) + (2)	III (1) + (4)
4	92	165	6.0	12.0	86	154	9028	92	7889	8440	8043	9182	8594
5	124	274	7.5	18.8	116	255	6664	50	5908	6261	6163	6919	6516
6	159	413	9.0	27.0	150	386	5176	31	4607	4883	4993	5562	5269
7	198	592	10.5	36.8	187	555	4141	20	3723	3921	4278	4696	4476
8	240	811	12.0	48.0	228	763	3405	13.5	3073	3235	3836	4168	3998
9	286	1075	13.5	60.8	272	1014	2849	9.5	2591	2715	3605	3863	3729
10	335	1384	15.0	75.0	320	1309	2426	6.9	2214	2318	3523	3735	3627
11	388	1746	16.5	90.8	371	1655	2090	5.2	1924	2001	3556	3745	3656
12	444	2158	18.0	108	426	2050	1825	3.9	1680	1749	3730	3875	3799
13	504	2636	19.5	126.8	484	2509	1602	3.1	1482	1540	3991	4111	4049

M<sub>1</sub> = Static moment in unobstructed channel

M<sub>0</sub> = Static moment of obstruction

M<sub>1</sub> - M<sub>0</sub> = Static moment in obstructed channel

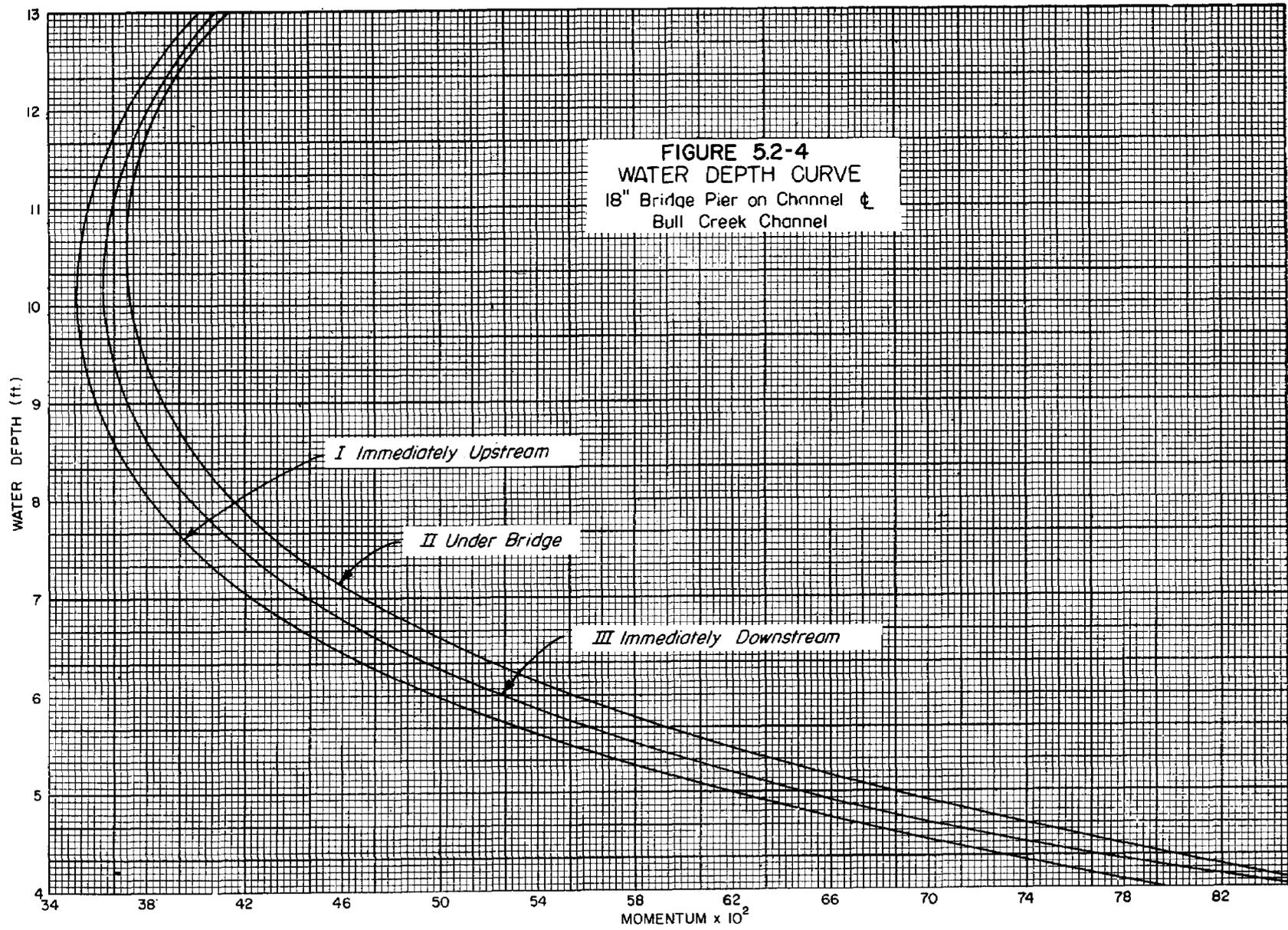
$$M_1 - M_0 + \frac{Q^2}{g A_3} = M_1 - M_0 + \frac{Q^2}{g (A_1 - A_0)} = M_1 - M_0 + \frac{Q^2 (A_1 - A_0)}{g A_1^2}$$

$$\frac{Q^2}{g (A_1 - A_0)} = \text{Kinetic momentum in obstructed channel}$$

$$\frac{Q^2}{g A_1^2} = \text{Kinetic momentum in unobstructed channel}$$

$$\frac{A_0 Q^2}{g A_1^2} = \text{Kinetic momentum lost}$$







CHAPTER 6 - STABILITY DESIGN

Introduction . . . . .	6-1
Stability Design . . . . .	6-1
Essentially Rigid Boundaries . . . . .	6-1
Figure 6-1 -- Channel Evaluation Procedural Guide . . . . .	6-2
Mobile Boundaries . . . . .	6-3
Procedure for Determining Sediment Concentration . . . . .	6-3
Allowable Velocity Approach . . . . .	6-3
General . . . . .	6-3
Figure 6-2 -- Allowable Velocities for Unprotected Earth Channels . . . . .	6-4
Procedures- Allowable Velocity Approach . . . . .	6-6
Examples of Allowable Velocity Approach . . . . .	6-6
Example 6-1 . . . . .	6-6
Example 6-2 . . . . .	6-8
Example 6-3 . . . . .	6-9
Example 6-4 . . . . .	6-10
Tractive Stress Approach . . . . .	6-12
General . . . . .	6-12
Coarse Grained Discrete Particle Soils . . . . .	6-12
Actual Tractive Stress . . . . .	6-12
Figure 6-3 -- Hydraulics: Channel Stability; Actual Maximum Tractive Stress $\tau_b$ , on Bed of Straight Trapezoidal Channels . . . . .	6-14
Figure 6-4 -- Hydraulics: Channel Stability; Actual Maximum Tractive Stress $\tau_s$ , on Sides of Straight Trapezoidal Channels . . . . .	6-15

Figure 6-5 -- Hydraulics: Channel Stability; Actual Maximum Tractive Forces, $\tau_{bc}$ and $\tau_{sc}$ , on Bed and Sides of Trapezoidal Channels within a Curved Reach . . . . .	6-16
Figure 6-6 -- Hydraulics: Channel Stability; Actual Maximum Tractive Stress, $\tau_{bt}$ and $\tau_{st}$ , on Bed and Sides of Trapezoidal Channels in Straight Reaches Immediately Downstream from Curved Reaches . . . . .	6-17
Allowable Tractive Stress . . . . .	6-18
Fine Grained Soils . . . . .	6-18
Actual Tractive Stress . . . . .	6-18
Figure 6-7 -- Hydraulics: Channel Stability; Angle of Repose, $\phi_R$ , for Non-Cohesive Materials . . . . .	6-19
Figure 6-8 -- Hydraulics: Channel Stability; Limiting Tractive Force $\tau_{Ls}$ for sides of Trapezoidal Channels Having Non-Cohesive Materials . . . . .	6-20
Figure 6-9 -- Graphic Solution of Equation 6-10 . . . . .	6-22
Figure 6-10 -- Graphic Solution of Equation 6-10 . . . . .	6-23
Figure 6-11 -- Values of $\nu$ and $\rho$ for Various Water Temperatures . . . . .	6-24
Figure 6-12 -- Applied Maximum Tractive Stresses, $\tau_s$ , On Sides of Straight Trapezoidal Channels . . . . .	6-25
Figure 6-13 -- Applied Maximum Tractive Stresses, $\tau_b$ , on Bed of Straight Trapezoidal Channels . . . . .	6-25
Figure 6-14 -- Allowable Tractive Stresses Non-Cohesive Soils, $D_{75} < 0.25''$ . . . . .	6-26
Allowable Tractive Stress . . . . .	6-27
Procedures - Tractive Stress Approach . . . . .	6-27
Examples of Tractive Stress Approach . . . . .	6-28
Example 6-5 . . . . .	6-28
Example 6-6 . . . . .	6-29
Formation of Bed Armor in Coarse Material . . . . .	6-30

	<u>Page</u>
Tractive Power Approach . . . . .	6-31
Figure 6-15 -- Unconfined Compressive Strength and Tractive Power as Related to Channel Stability . . . . .	6-32
Procedures - Tractive Power Approach . . . . .	6-33
Example 6-7 . . . . .	6-33
The Modified Regime Approach . . . . .	6-35
Procedures - Modified Regime Approach . . . . .	6-37
Example 6-8 . . . . .	6-38
Channel Stability With Respect to Sediment Transport . . . . .	6-40
Application of Bedload Transport Equations . . . . .	6-40
Sediment Transport in Sand Bed Streams Not in Equilibrium . . . . .	6-41
Figure 6-16 -- Relationship Between Mean Velocity and Sediment Transport on and Near Stream Bed . . . . .	6-42
Figure 6-17 -- Relation of Mean Velocity to Product of Slope Hydraulic Radius and Unit Weight of Water . . . . .	6-43
Example 6-9 . . . . .	6-44
Table 6-1 -- Mean Velocity Computations - Two Channel Sections . . . . .	6-46
Table 6-2 -- Calculation of "n" adjusted for 2-1/2 to 1 side slopes (riprapped) . . . . .	6-47
Figure 6-18 -- Reservoir Release Hydrograph Principal Spillway . . . . .	6-48
Figure 6-19 -- Velocity-Area Curve . . . . .	6-49
Figure 6-20 -- Velocity-Discharge Curve . . . . .	6-50
Table 6-3 -- Bedload Sediment Transport . . . . .	6-52
Example 6-10 . . . . .	6-53
Figure 6-21 -- Discharge-Bedload Sediment Transport . . . . .	6-54

	<u>Page</u>
Figure 6-22 -- Synthetic Storm Hydrograph . . . . .	6-55
Figure 6-23 -- Velocity-Area Curve . . . . .	6-56
Figure 6-24 -- Velocity-Discharge Curve . . . . .	6-57
Table 6-4 -- Mean Velocity Computations - Three Channel Sections . . . . .	6-58
Slope (Bank) Stability Analysis . . . . .	6-59
General . . . . .	6-59
Table 6-5 -- Bedload Sediment Transport - Three Stream Sections . . . . .	6-60
Figure 6-25 -- Discharge-Bedload Sediment Transport . . .	6-61
Types of Slides and Methods of Analysis . . . . .	6-62
Boundary Conditions and Parameters Affecting Slope Stability . . . . .	6-64
Factors of Safety Against Sliding . . . . .	6-68
Piping . . . . .	6-68
Stabilizing Measures . . . . .	6-69
General . . . . .	6-69
Bank Protection . . . . .	6-69
Channel Linings . . . . .	6-77
Grade Control Structures . . . . .	6-78
Other Structures . . . . .	6-82
General . . . . .	6-82
Channel Crossings . . . . .	6-82
Channel Junction Structures . . . . .	6-82
Side Inlet Structures . . . . .	6-83
Water Level Control Structures . . . . .	6-84

	<u>Page</u>
Design Features Related to Maintenance . . . . .	6-84
Added Depth or Capacity for Deposition . . . . .	6-84
Relationship of Side Slopes to Maintenance Methods . . . . .	6-85
Berms . . . . .	6-85
Maintenance Roadways . . . . .	6-85
Spoil . . . . .	6-85
Entrance of Side Surface Water to Channel . . . . .	6-85
Seeding . . . . .	6-86
Pilot Channels . . . . .	6-86
Glossary of Symbols . . . . .	6-87
References . . . . .	6-90



Introduction

The analysis of earth channels with acceptable limits of stability is of primary importance to Soil Conservation Service activities. The evaluation or design of any water conveyance system that includes earth channels requires knowledge of the relationships between flowing water and the earth materials forming the boundary of the channel, as well as an understanding of the expected stream response when structures, lining, vegetation, or other features are imposed. These relationships may be the controlling factors in determining channel alignment, grade, dimensioning of cross section and selection of design features to assure the operational requirements of the system.

The methods included herein to evaluate channel stability against the flow forces are for bare earth. When evaluations indicate the ability of the soil is insufficient to resist or tolerate the forces applied by the flow under consideration it may be necessary to consider that the channel has mobile boundaries. The magnitude of the channel instability needs to be determined in order to evaluate whether or not vegetative practices and/or structural measures are needed. Where such practices or measures are required, methods of analysis that appropriately evaluate the stream's response should be used.

Figure 6-1 provides general guidance in selecting evaluation procedures that apply to various site conditions.

All terms used in this chapter are defined in the glossary on page 6-87.

Stability Evaluation

Methods presently used by the SCS in the evaluation of the stability of earth channels are based on the following fundamental physical concepts.

1. Essentially rigid boundaries. Stability is attained when the interaction between flow and the material forming the channel boundary is such that the soil boundary effectively resists the erosive efforts of the flow.

Where properly evaluated and designed the bed and banks in this class of channels remains essentially unchanged during all stages of flow. The principles of hydraulics based on rigid boundaries are applicable in analyzing such channels.

The procedures described in this chapter that are based on this definition of stability are:

- a. Allowable velocity approach.
- b. Tractive stress approach.
- c. Tractive power approach.

# CHANNEL EVALUATION PROCEDURAL GUIDE

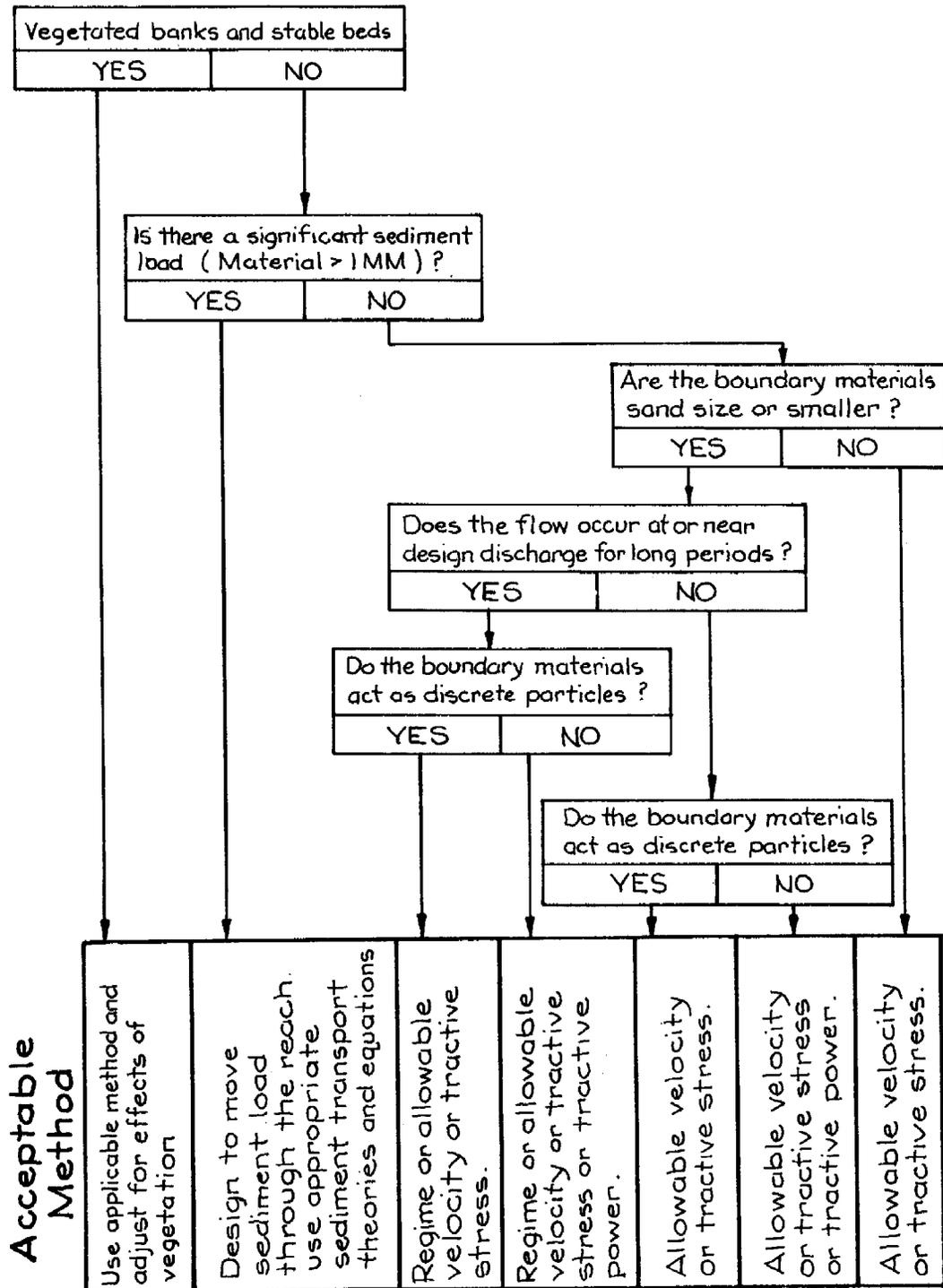


Figure 6-1

2. Mobile Boundaries. Stability is attained when the rate at which sediment enters the channel from upstream is equal to the capacity of the channel to carry material having the same composition as the incoming sediment. The bed and the banks of the channel are mobile and may vary somewhat from designed position. Stability in such channels may be determined by methods that use the principles of flow in channels with movable boundaries.

The procedures described in this chapter that are based on this definition of stability are:

- a. Sediment Transport Approach.
- b. Modified Regime Approach.

#### Procedure for Determining Sediment Concentration

The stability of a channel is influenced by the concentration and physical characteristics of the sediment entering the channel and available for transport as bedload and in suspension. Procedures for computing sediment transport are described in NEH-3, Chapter 4.<sup>52/</sup> If clear water is not used, stream gage data when available and representing a wide range of flows are useful in predicting sediment loads. When the clear water procedure is not chosen and suitable data are not available, there is a method of making rough estimates of sediment loads presented in Geologic Note 2.

#### Allowable Velocity Approach

##### General

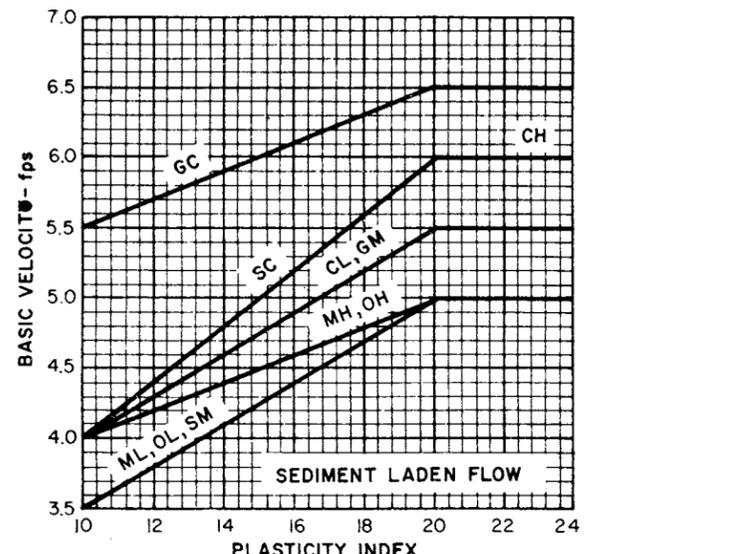
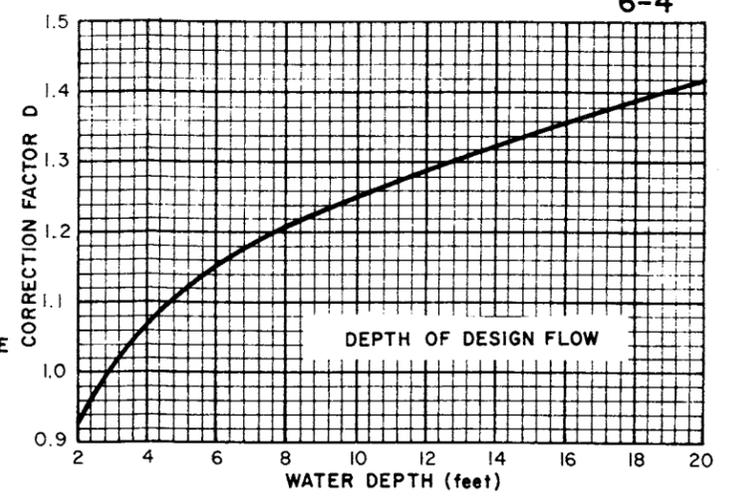
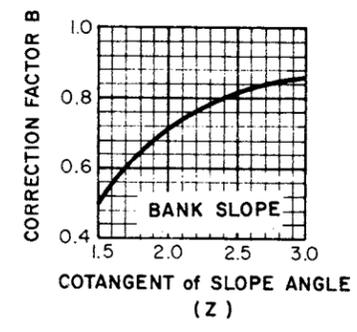
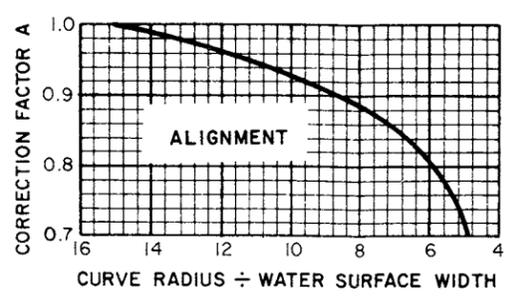
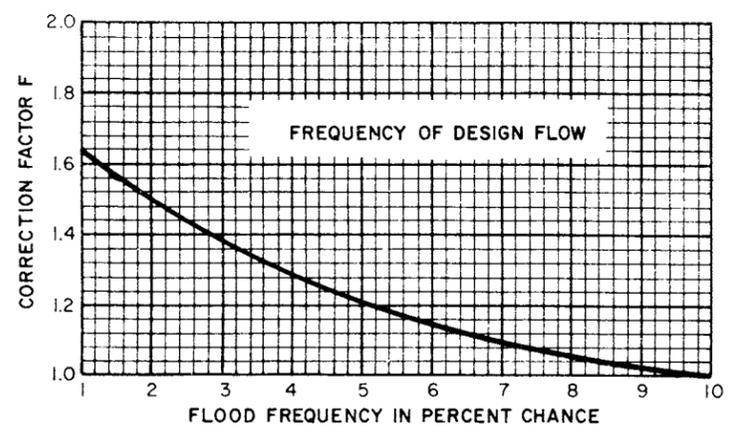
This method of testing the erosion resistance of earth channels is based on data collected by several investigators.

Figure 6-2 shows "Allowable Velocities for Unprotected Earth Channels" developed chiefly from data by Fortier and Scobey<sup>24/</sup>, Lane<sup>25/</sup>, by investigators in the U.S.S.R.<sup>26/</sup> and others. The allowable velocities determined from Figure 6-2 refer to channels formed in earth with no vegetative or structural protection. The Fortier and Scobey data shown on Figure 6-2 were collected by the authors from engineers experienced in irrigation systems. The canals were well-seasoned, were on low gradients, and had flow depths of less than 3 feet.

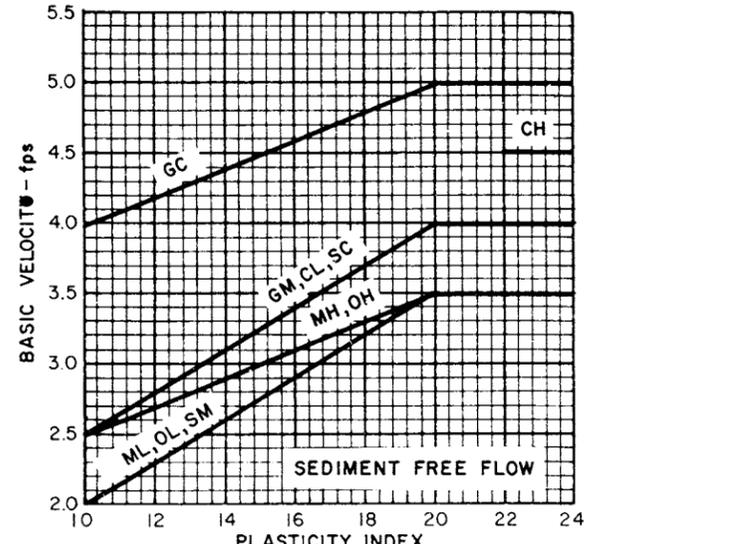
Stability is influenced by the concentration of fine material carried by the flow in suspension. There are two distinct types of flow depending on concentration of material in suspension.

1. Sediment free flow is defined as the condition in which fine material is carried in suspension by the flow at concentrations so low that it

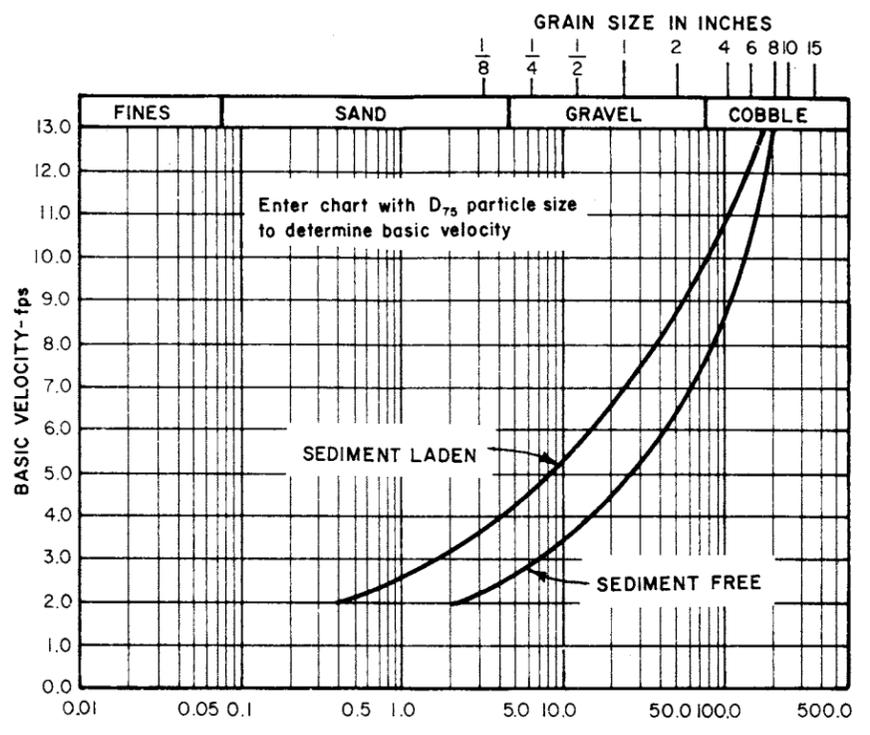




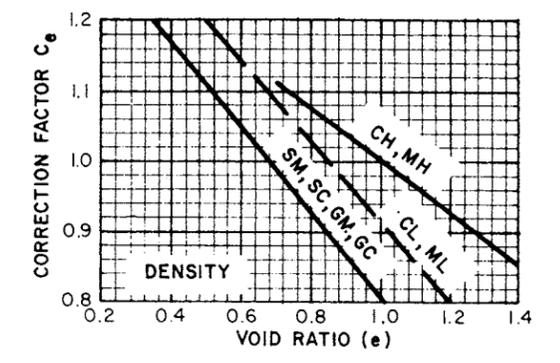
BASIC VELOCITIES FOR COHERENT EARTH MATERIALS ( $v_b$ )



BASIC VELOCITIES FOR COHERENT EARTH MATERIALS ( $v_b$ )



BASIC VELOCITY FOR DISCRETE PARTICLES OF EARTH MATERIALS ( $v_b$ )



NOTES:

1. In no case should the allowable velocity be exceeded when the 10% chance discharge occurs, regardless of the design flow frequency.

ALLOWABLE VELOCITIES FOR UNPROTECTED EARTH CHANNELS	
CHANNEL BOUNDARY MATERIALS	ALLOWABLE VELOCITY
DISCRETE PARTICLES	
Sediment Laden Flow	Basic velocity chart value x D x A x B
$D_{75} > 0.4$ mm	2.0 fps
$D_{75} < 0.4$ mm	
Sediment Free Flow	Basic velocity chart value x D x A x B
$D_{75} > 2.0$ mm	2.0 fps
$D_{75} < 2.0$ mm	
COHERENT EARTH MATERIALS	
$PI > 10$	Basic velocity chart value x D x A x F x $C_e$
$PI < 10$	2.0 fps

FIGURE 6-2  
ALLOWABLE VELOCITIES  
FOR UNPROTECTED EARTH CHANNELS

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has no effect on channel stability. Flows with concentrations lower than 1,000 ppm by weight are treated as sediment free flows.

2. Sediment laden flow is the condition in which the flow carries fine material in suspension at moderate to high concentrations so that stability is enhanced either through replacement of dislodged particles or through formation of a protective cover as the result of settling. Flows in this class carry sediment in suspension at concentrations equal or larger than 20,000 ppm by weight.

Estimation of the concentration of sediment in suspension is best made by sampling. See NEH, Section 3 for methods of sampling. If the concentration is not known from measurement, it can be estimated by the methods in Geologic Note 2.

Sediment transport rates are usually expressed in tons per day. To convert them into concentration use the equation:

$$C = 370 \frac{Q_s}{Q} \quad \text{Eq. 6-1}$$

See page 4-41 of NEH 3, Chapter 4, for conversion from concentration in parts per million to milligrams per liter.

Depending on the type of soil, the effect of concentration of fine sediment (material smaller than 0.074 mm) in suspension on the allowable velocity is obtained from the curves on Figure 6-2.

If the suspended sediment concentration equals or exceeds 20,000 ppm by weight, use the sediment laden curve on Figure 6-2. If the suspended sediment concentration is 1,000 ppm or less by weight, use the sediment free curve on Figure 6-2. A linear interpolation may be made between these curves for suspended sediment concentrations between 1,000 ppm and 20,000 ppm.

Adjustment in the basic velocity to reflect the modifying effects of frequency of runoff, curvature in alignment, bank slopes, density of bed and bank materials, and depth of flow are made using the adjustment curves on Figure 6-2.

The alignment factor, A, and the depth factor, D, apply to all soil conditions. The bank slope factor, B, applies only to channels in soils that behave as discrete particles. The frequency correction, F, applies only to channels in soils that resist erosion as a coherent mass. The density correction factor,  $C_e$ , applies to all soil materials except clean sands and gravels (containing less than 5 percent material passing size #200).

Figure 6-2 gives the correction factors (F) for frequencies of occurrence lower than 10 percent. Channels designed for less frequent flows using this correction factor should be designed to be stable at the 10 percent chance frequency discharge as well as at the design discharge.

If the soils along the channel boundary behave as discrete particles with  $D_{75}$  larger than 0.4 mm for sediment laden flow or larger than 2.0 mm for sediment free flow, the allowable velocity is determined by adjusting the basic velocity read from the curves on Figure 6-2 for the effects of alignment, bank slope, and depth. If the soils behave as discrete particles and  $D_{75}$  is smaller than 0.4 mm for sediment laden flow or 2.0 mm for sediment free flow the allowable velocity is 2.0 fps. For channels in these soils no adjustments are to be made to the basic velocity of 2.0 fps.

In cases where the soils in the channel boundary resist erosion as a coherent mass, the allowable velocity is determined by adjusting the basic velocity from Figure 6-2 for the effects of depth, alignment, bank slope, frequency of occurrence of design flow, and for the density of the boundary soil materials.

#### Design Procedure for Allowable Velocity Approach

The use of the allowable velocity approach in checking the stability of earth channels involves the following steps:

1. Determine the hydraulics of the system. This includes hydrologic determinations as well as the stage-discharge relationships for the channel considered. The procedures to be used in this step are included in Chapter 4 and Chapter 5 of this Technical Release.
2. Determine the properties of the earth materials forming the banks and bed of the design reach and of the channel upstream.
3. Determine sediment yield to reach and calculate sediment concentration for design flow.
4. Check to see if the allowable velocity procedure is applicable. Use Figure 6-1.
5. Compare the design velocities with the allowable velocities from Figure 6-2 for the materials forming the channel boundary.
6. If the allowable velocities are less than design velocities, it may be necessary to consider a mobile boundary condition and evaluate the channel using appropriate sediment transport theory.

#### Examples of Allowable Velocity Approach

##### Example 6-1

Given: A channel is to be constructed to convey the flow from a 2 percent chance flood through an intensively cultivated area. The hydraulics of the

system indicate that a trapezoidal channel with 2:1 side slopes and a 40 foot bottom width will carry the design flow at a depth of 8.7 feet and a velocity of 5.45 fps. Soil investigations reveal that the channel will be excavated in a moderately rounded clean sandy gravel with a  $D_{75}$  size of 2.25 inches. Sampling of soils in the drainage area and estimate of erosion and sediment yield indicate that on an average annual basis approximately 1000 tons of sediment finer than 1.0 mm. and 20 tons of material coarser than 1.0 mm are available for transport in channel. The amount of abrasion resulting from the transporting of this small amount of sediment coarser than 1.0 mm. is considered insignificant. Sediment transport computations indicate all of the sediment supplied to the channel will be transported through the reach. The sediment transport and hydrologic evaluations indicate the design flow will transport the available sediment at a concentration of about 500 ppm. The channel is straight except for one curve with a radius of 600 feet.

Determine:

1. The allowable velocity,  $V_a$
2. The stability of the reach.

Solution: Determine basic velocity from Figure 6-2, sediment free curve because sediment concentration of 500 ppm is less than 1,000 ppm.

$$V_b = 6.7 \text{ fps}$$

Depth correction factor,  $D = 1.22$  (from Figure 6-2)

Bank slope correction,  $B = 0.72$  (from Figure 6-2)

Alignment correction A

$$\frac{\text{curve radius}}{\text{water surface width}} = \frac{600}{74.8} = 8.02$$

$A = 0.89$  (from Figure 6-2)

Density correction,  $C_e$ , does not apply

Frequency correction,  $F$ , does not apply

$$\begin{aligned} V_a &= V_b DB = (6.7)(1.22)(0.72) && \text{straight reaches} \\ &= 5.88 \text{ fps} \end{aligned}$$

$$\begin{aligned} V_a &= V_b DBA = (6.7)(1.22)(0.72)(0.89) \text{ curved reach} \\ &= 5.24 \text{ fps} \end{aligned}$$

The proposed design velocity of 5.45 fps is less than  $V_a = 5.88$  fps in the straight reaches but greater than  $V_a = 5.24$  fps in the curved reaches. Either the channel alignment or geometry needs to be altered or the curve needs structural protection.

Example 6-2

Given: A channel is to be constructed to convey the flow from a 2 percent chance flood through an intensively cultivated area. The hydraulics of the system indicate that a trapezoidal channel with 2:1 side slopes and a 40 foot bottom width will carry the design flow at a depth of 8.7 feet and a velocity of 5.45 fps. The channel is to be excavated into a silty clay CL soil with a Plasticity Index of 18, a dry density of 92 pcf, and a specific gravity of 2.71. Sediment transport evaluations indicate the design flow will have a fairly stable sediment concentration of about 500 ppm. with essentially no bed material load larger than 1.0 mm. The channel is straight except for one curve with a radius of 600 feet. The 10 percent chance flood results in a depth of flow of 7.4 feet and a velocity of 4.93 fps.

Determine:

1. The allowable velocity,  $V_a$
2. The stability of the reach.

Solution: Sediment concentration of 500 ppm is less than 1,000 ppm therefore it is classed as sediment free flow.

$$V_b = 3.7 \text{ fps (from Figure 6-2)}$$

for the 2 percent chance flood

$$\text{Depth correction, } D = 1.22 \text{ (from Figure 6-2)}$$

Density correction, compute e.

$$e = C \frac{\gamma_w}{\gamma_d} - 1 = \frac{(2.71)(62.4)}{92} - 1 = 0.83$$

$$C_e = 1.0 \text{ (from Figure 6-2)}$$

$$\text{Frequency correction, } F = 1.5 \text{ (from Figure 6-2)}$$

Alignment correction A

$$\frac{\text{Curve radius}}{\text{water surface width}} = \frac{600}{74.8} = 8.02$$

$$A = 0.89 \text{ (from Figure 6-2)}$$

$$V_a = V_b DC_e F \quad \text{Straight reach}$$

$$V_a = (3.7)(1.22)(1.0)(1.5) = 6.77 \text{ fps}$$

$$V_a = V_b DC_e FA \quad \text{Curved reach}$$

$$V_a = (3.7)(1.22)(1.0)(1.5)(0.89) = 6.03 \text{ fps}$$

The design velocity is less than the allowable velocity for the 2 percent chance flow. Check the 10 percent chance flow velocity with no frequency correction against the allowable velocity for the 10 percent chance flow.

$$V_a = V_b DC_e \quad \text{Straight reaches}$$

$$V_a = (3.7)(1.19)(1.0) = 4.40 \text{ fps}$$

$$V_a = V_b DC_e A \quad \text{Curved reaches}$$

$$V_a = (3.7)(1.19)(1.0)(0.90) = 3.96 \text{ fps}$$

The allowable velocity with no frequency correction is exceeded by the 10 percent chance flow velocity. An evaluation should be made to estimate the magnitude of scour or possible depth of scour before an armor is formed (See page 6-30). Using this procedure in conjunction with the appropriate sediment transport equations, the magnitude of instability can be evaluated. Channel alignment, slope, or geometry must be altered or the channel must be protected.

### Example 6-3

Given: The same conditions as in Example 6-1 except that the suspended sediment concentration is 30,000 ppm.

Determine:

1. The allowable velocity,  $V_a$
2. The stability of the reach.

Solution: The suspended sediment concentration of 30,000 ppm is greater than 20,000 ppm.

Therefore it is classed as sediment laden flow.

$$V_b = 9.0 \text{ fps} \quad (\text{from Figure 6-2})$$

Depth correction factor  $D = 1.22$  (From Figure 6-2)

Bank Slope correction factor  $B = 0.72$  (From Figure 6-2)

Alignment correction factor  $A = 0.89$  (From Figure 6-2)

Density correction factor,  $C_e$  does not apply

Frequency correction factor  $F$ , does not apply

$$V_a = V_b DB = (9.0)(1.22)(0.72) = 7.91 \text{ fps for straight reaches}$$

$$V_a = V_b DBA = (9.0)(1.22)(0.72)(0.89) = 7.04 \text{ fps for the curved reach}$$

The proposed design velocity of 5.45 fps is less than  $V_a = 7.91$  fps, straight reaches and  $V_a = 7.04$  fps in the curved reaches. This reach of channel is considered to be stable relative to scour. Use sediment transport equations to determine the possibility of channel aggradation.

#### Example 6-4

Given:

1. Trapezoidal channel to convey the 50 percent chance flood at bank full flow.
2. The 10-year peak discharge exceeds the 2-year peak by 30 percent; the 10-year flow for as-built conditions exceeds bank full capacity.
3. From design hydraulic calculations:

Bottom Width = 30 ft.

Flow Depth = 9.0 ft. (bank full)

Side Slopes = 2:1

$n$  (aged condition) = 0.030

$n$  (as-built condition) = 0.025

Velocity (aged condition) = 3.3 fps (bank full)

4. Sharpest Curve = 350 ft. radius.
5. Estimated sediment concentration at design discharge = 2000 ppm.
6. Two layers of soil material are to be evaluated for stability. The upper layer is classified CL; the plasticity index is 15 and the void ratio is 0.9. The lower layer is classified as a GM with a  $D_{75}$  particle size of 10 mm.

Determine:

1. The allowable velocity for the CL and GM materials.
2. The stability of the channel.

Solution:

#### 1. CL Layer

From Figure 6-2 for coherent particles, the allowable velocity = (basic velocity)  $DAFC_e$

Basic velocity for sediment laden flow (20,000 ppm) = 4.75 fps.

Basic velocity for sediment free flow (1000 ppm) = 3.25 fps.

Basic velocity for 2000 ppm sediment concentration (by linear interpolation) = 3.33 fps.

Computations for correction factor A for sharpest curve:

$$\frac{\text{Curve radius}}{\text{Water surface width}} = \frac{350}{66} = 5.3$$

Correction factor F = 1.0  
 Correction factor A = 0.75  
 Correction factor D = 1.23  
 Correction factor C<sub>e</sub> = 0.97

Allowable velocity for straight channel = V<sub>a</sub> = V<sub>b</sub>DAFC<sub>e</sub>  
 V<sub>a</sub> = (3.33)(1.23)(1.0)(1.0)(0.97) = 3.97 fps

This is greater than the 3.3 fps design value and the channel is stable for this condition.

Allowable velocity for sharpest curve = V<sub>a</sub> = V<sub>b</sub>DAFC<sub>e</sub>  
 V<sub>a</sub> = (3.33)(1.23)(0.75)(1.0)(0.97) = 2.98 fps

This is less than the 3.3 fps design value and the channel is not stable for the sharpest curve.

## 2. GM Layer

From Figure 6-2, the allowable velocity is V<sub>a</sub> = (V<sub>b</sub>) DAB

V<sub>b</sub> for sediment laden flow (20,000 ppm) = 5.3 ft/sec

V<sub>b</sub> for sediment free flow (1,000 ppm) = 3.4 ft/sec

V<sub>b</sub> for 2000 ppm sediment concentration (by linear interpolation) = 3.5 ft/sec

Correction factor A = 0.75 (R = 350')  
 Correction factor D = 1.23  
 Correction factor B = 0.71

Allowable velocity for straight channel

$$V_a = V_b DAB = (3.5)(1.23)(1.0)(0.71) = 3.06 \text{ fps}$$

Allowable velocity for sharpest curve

$$V_a = V_b \text{ DAB} = (3.5)(1.25)(0.75)(0.71) = 2.29 \text{ fps}$$

**Summary:**

The upper layer (CL) is stable for the straight sections and unstable for curve with a radius of 350 ft.

The lower layer (GM) is unstable for both the straight and curved sections.

This condition may need additional evaluation using the appropriate sediment transport equations.

### Tractive Stress Approach

#### General

The tractive force is the tangential pull of flowing water on the wetted channel boundary; it is equal to the total friction force that resists flow but acts in the opposite direction. Tractive stress is the tractive force per unit area of the boundary. The tractive force is expressed in units of pounds, while tractive stress is expressed in units of pounds per square foot. The tractive force in a prismatic channel reach is equal to the weight of the fluid prism multiplied by the energy gradient.

The tractive stress approach to channel stability analysis provides a method to evaluate the stress at the interface between flowing water and the materials in the channel boundary.

The method for obtaining the design or actual tractive stress acting on the bed or sides of a channel and the allowable tractive stress depends on the  $D_{75}$  size of the materials involved. When coarse grained discrete particle soils are involved Lane's<sup>25/</sup> method is used. When fine grained soils are involved, a method derived from the work of Keulegan and modified by Einstein<sup>27/</sup>, and Vanoni and Brooks<sup>48/</sup> is used. The separation size for this determination is  $D_{75} = 1/4$  inch.

#### Coarse-grained Discrete Particle Soils - $D_{75} > 1/4$ inch - Lane's Method

##### A. Determination of Actual Tractive Stress

###### 1. Actual tractive stress in an infinitely wide channel.

Generally, Manning's roughness coefficient  $n$  reflects the overall impedance to flow including grain roughness, form roughness, vegetation, curved alignment, etc. Lane's<sup>25/</sup> work showed that for soils with a  $D_{75}$  size between 0.25" (6.35mm) and 5.0" (127mm) the value of Manning's coefficient  $n$  resulting from the roughness of the soil particles is determined by:

$$n_t = \frac{D_{75}^{1/6}}{39} \quad \text{with } D_{75} \text{ expressed in inches (Eq. 6-2)}$$

The value of  $n_t$  determined by equation 6-2 represents the retardance to flow caused by roughness of the soil grains.

Use  $n_t$  from equation 6-2 as a first step in the procedure in NEH, Section 5, Supplement B to determine the total value of Manning's coefficient. The value of  $n_t$  from equation 6-2 can be used next in equation 6-3 to compute  $s_t$ , the friction gradient associated with the particular boundary material being considered.

$$s_t = \left( \frac{n_t}{n} \right)^2 s_e \quad (\text{Eq. 6-3})$$

The tractive stress acting on the soil grains in an infinitely wide channel is found by:

$$\tau_\infty = \gamma_w ds_t \quad (\text{Eq. 6-4})$$

where the terms are as defined in the glossary.

## 2. Distribution of the tractive stress along the channel perimeter:

In open channels the tractive stresses are not distributed uniformly along the perimeter. Laboratory experiments and field observations have indicated that in trapezoidal channels the stresses are very small near the water surface and near the corners of the channel and assume their maximum value near the center of the bed. The maximum value on the banks occurs near the lower third point.

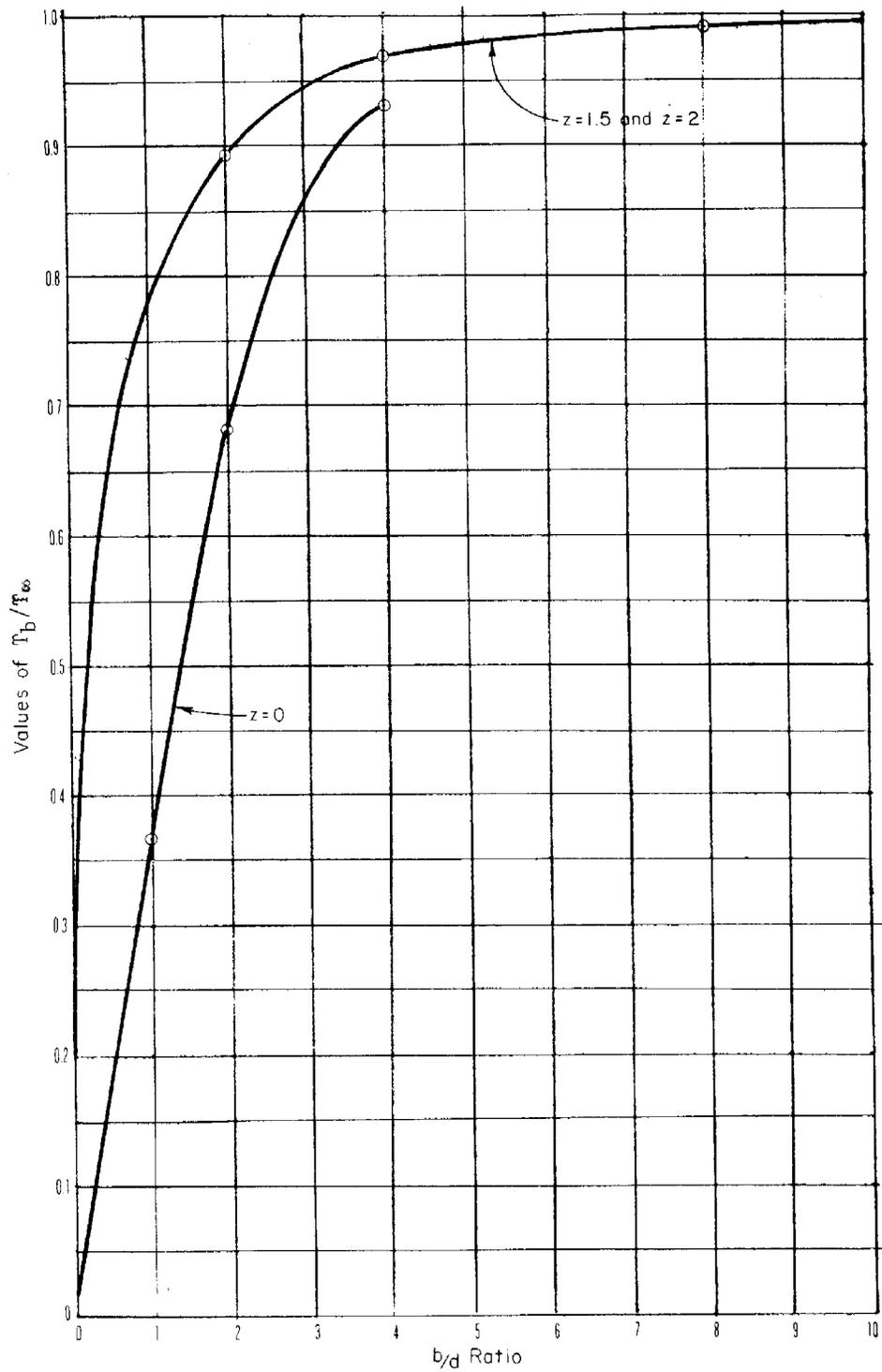
Figure 6-3 and 6-4 give the maximum tractive stresses in a trapezoidal channel in relation to the tractive stress in an infinitely wide channel having the same depth of flow and value of  $s_t$ .

## 3. Tractive stresses on curved reaches:

Curves in channels cause the maximum tractive stresses to increase above those in straight channels. The maximum tractive stresses in a channel with a single curve occur on the inside bank in the upstream portion of the curve and near the outer bank downstream from the curve. Compounding of curves in a channel complicates the flow pattern and causes a compounding of the maximum tractive stresses.

Figure 6-5 gives values of maximum tractive stresses based on judgment coupled with very limited experimental data. It does not show the effect of depth of flow and length of curve and its use is only justified until more accurate information is obtained. Figure 6-6 with a similar degree of accuracy, gives

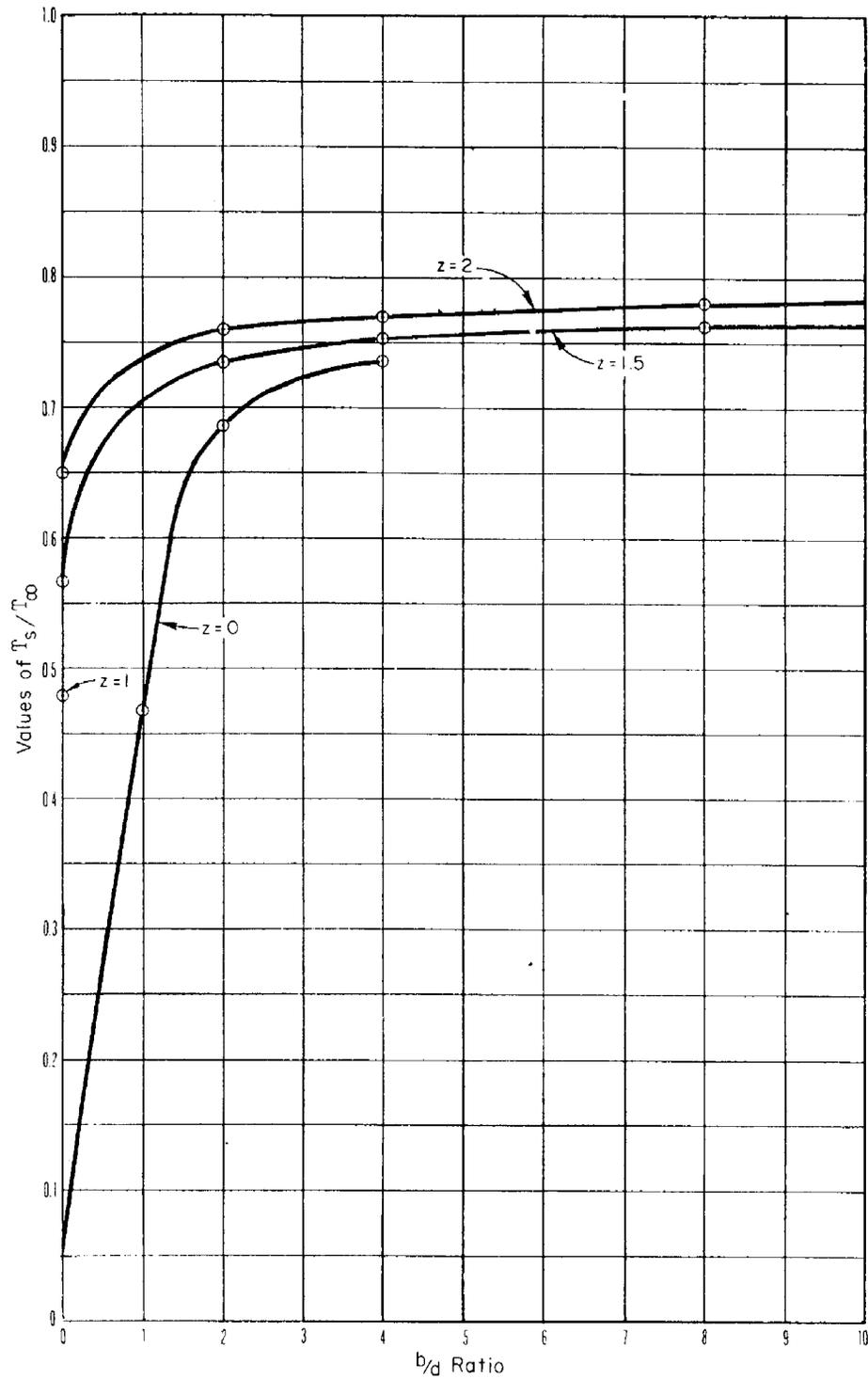
HYDRAULICS: Channel stability; actual maximum tractive stress,  $T_b$ , on bed of straight trapezoidal channels



REFERENCE  
Bureau Of Reclamation "Progress  
Report On Results Of Studies On  
Design Of Stable Channels" Hyd-352

FIGURE 6-3

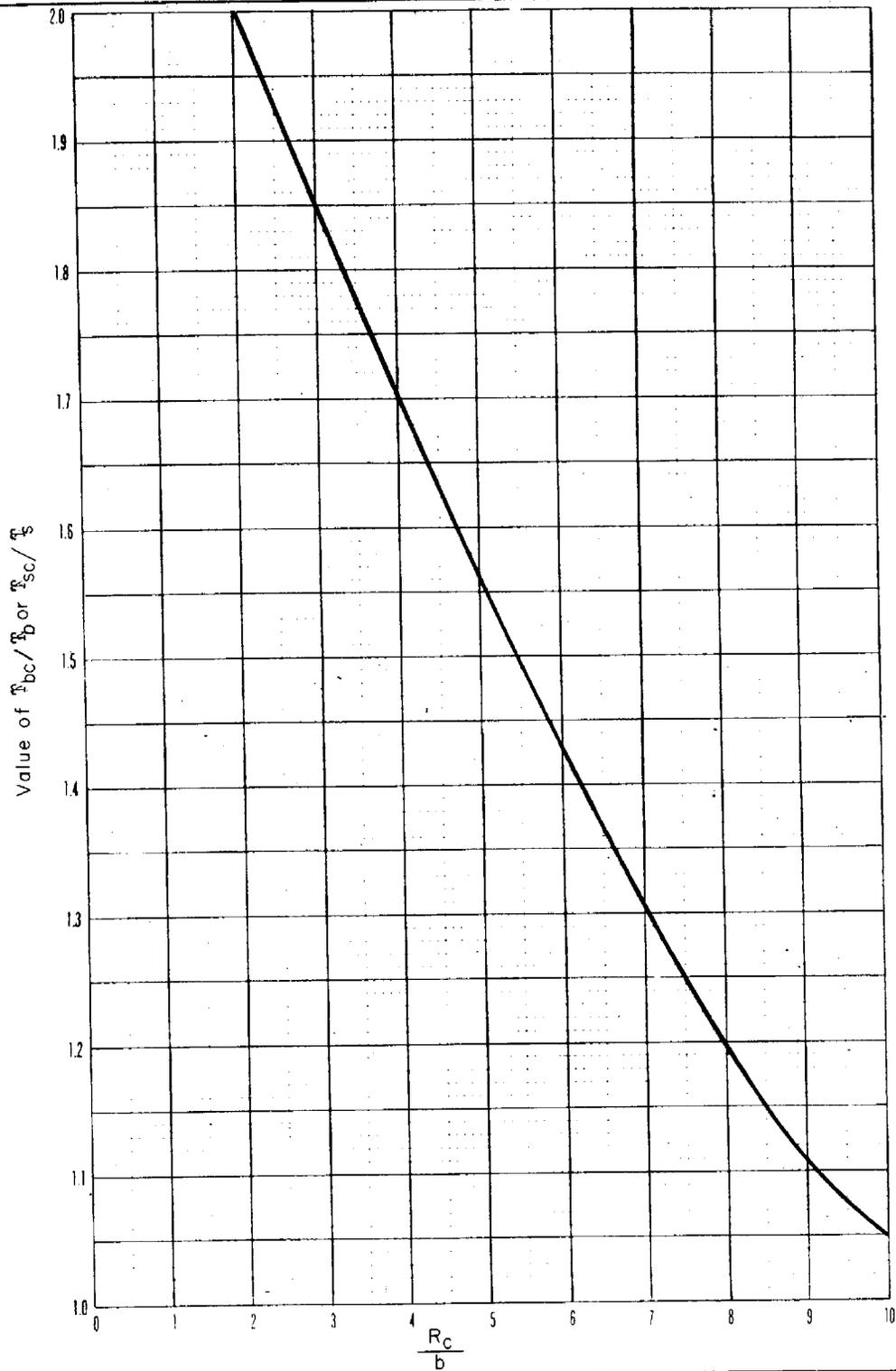
HYDRAULICS: Channel stability; actual maximum tractive stress,  $T_s$ , on sides of straight trapezoidal channels



REFERENCE  
 Bureau Of Reclamation "Progress  
 Report Of Results Of Studies On  
 Design Of Stable Channels" Hyd-352

FIGURE 6-4

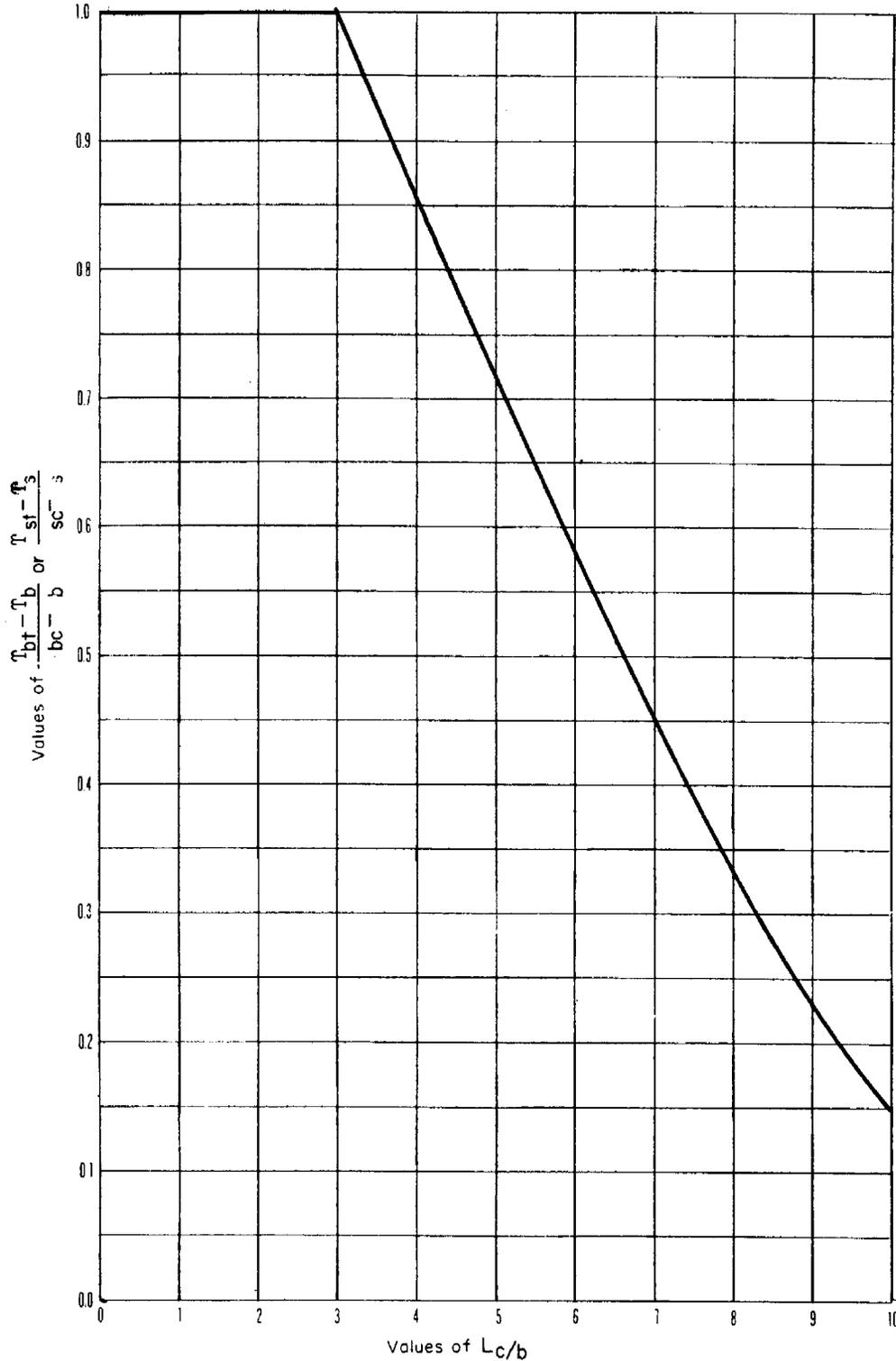
HYDRAULICS: Channel stability; actual maximum tractive stress,  $\tau_{bc}$  and  $\tau_{sc}$ , on bed and sides of trapezoidal channels within a curved reach



REFERENCE  
Lane, Emory W., Design of Stable Channels Trans-  
action, A S C E , vol 120, 1955  
Nace, R. E., Givler, C. A., and Drinker, P. A., Measure-  
ment of Boundary Shear Stress in an Open Channel. Curve  
with a Surface Pitot Tube. MIT Tech note (no 6),  
August 1959

FIGURE 6-5

HYDRAULICS: Channel stability; actual maximum tractive stresses  $\tau_{bt}$  and  $\tau_{st}$ , on bed and sides of trapezoidal channels in straight reaches immediately downstream from curved reaches



REFERENCE  
 Nece, R.E., Givier, G.A., and Drinker, P.A.,  
 Measurement of Boundary Shear Stress  
 in an Open Curve Channel with a Surface  
 Pitot Tube: M.I.T. Tech note (no 6), Aug.  
 1959

FIGURE 6-6

the maximum tractive stresses at various distances downstream from the curve.

## B. Allowable Tractive Stress

The allowable tractive stress for channel beds,  $\tau_{Lb}$ , composed of soil particles with discrete, single grain behavior with a given  $D_{75}$  is:

$$\tau_{Lb} = 0.4 D_{75}$$

When 0.25 in. <  $D_{75}$  < 5.0 in. (Eq. 6-5)

The allowable tractive stress for channel sides  $\tau_{Ls}$  is less than that of the same material in the bed of the channel because the gravity force aids the tractive stress in moving the materials. The allowable tractive stress for channel sides composed of soil particles behaving as discrete single grain materials, considering the effect of the side slope  $z$  and the angle of repose  $\phi_R$  with the horizontal is

$$\tau_{Ls} = 0.4 K D_{75} \dots 0.25 \text{ in.} < D_{75} < 5.0 \text{ in.} \quad (\text{Eq. 6-6})$$

$$K = \sqrt{\frac{z^2 - \cot^2 \phi_R}{1 + z^2}} \dots \quad (\text{Eq. 6-7})$$

Figure 6-7 gives an evaluation of the angles of repose corresponding to the degree of angularity of the material. Figure 6-8 gives values of  $K$  from equation 6-7.

When the unit weight  $\gamma_s$  of the constituents of the material having a grain size larger than the  $D_{75}$  size is significantly different than 160 lb/ft<sup>3</sup>, the limiting tractive stress  $\tau_{Lb}$  and  $\tau_{Ls}$  as given by equations (6-5) and (6-6) should be multiplied by the factor.

$$T = \frac{\gamma_s - \gamma_w}{97.6} \quad (\text{Eq. 6-8})$$

### Fine Grained Soils - $D_{75} < 1/4$ inch

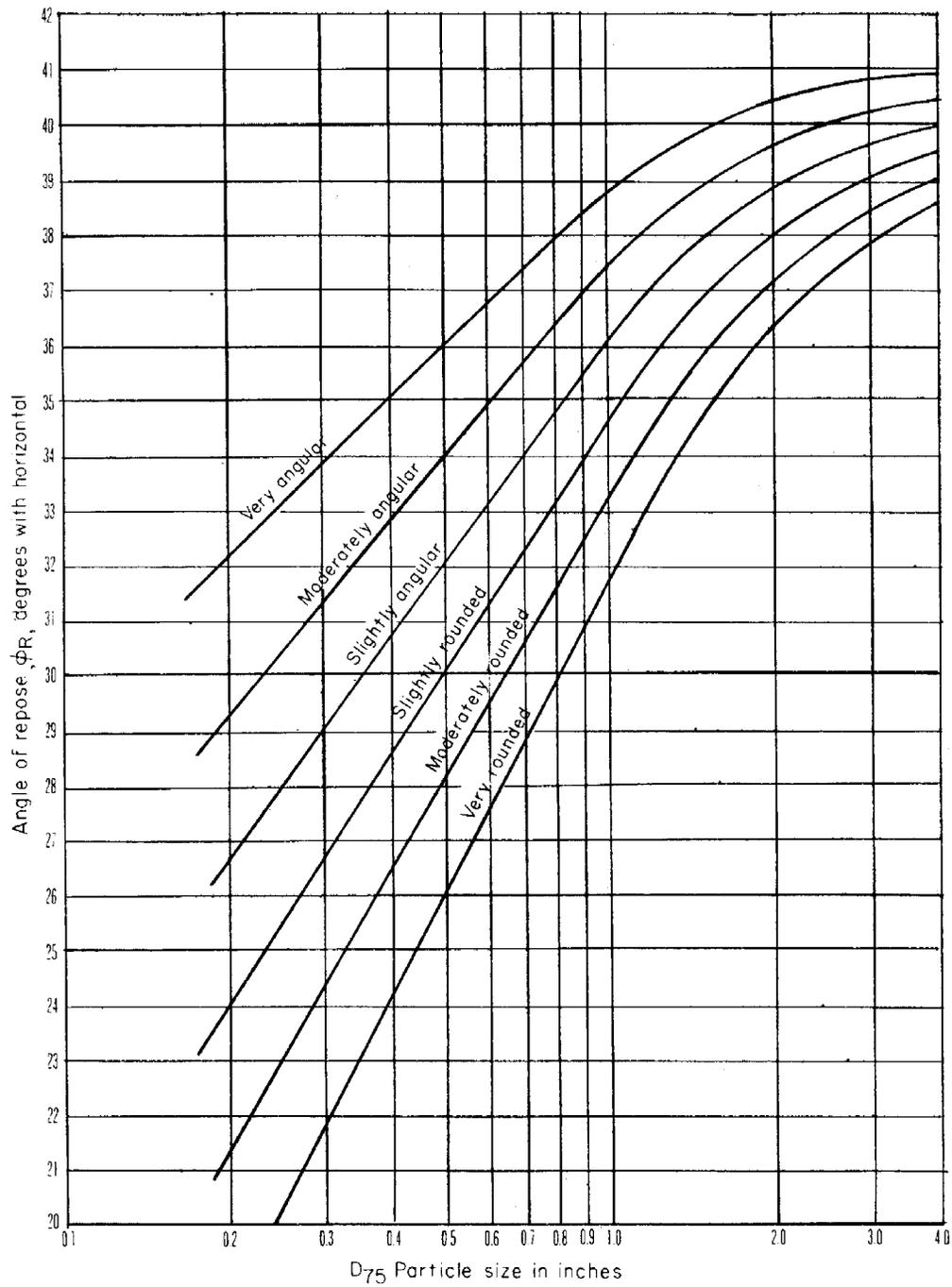
#### A. Determination of Actual Tractive Stress

##### 1. Reference tractive stress

The expression for the reference tractive stress is:

$$\tau = \gamma_w R_t s_e \quad (\text{Eq. 6-9})$$

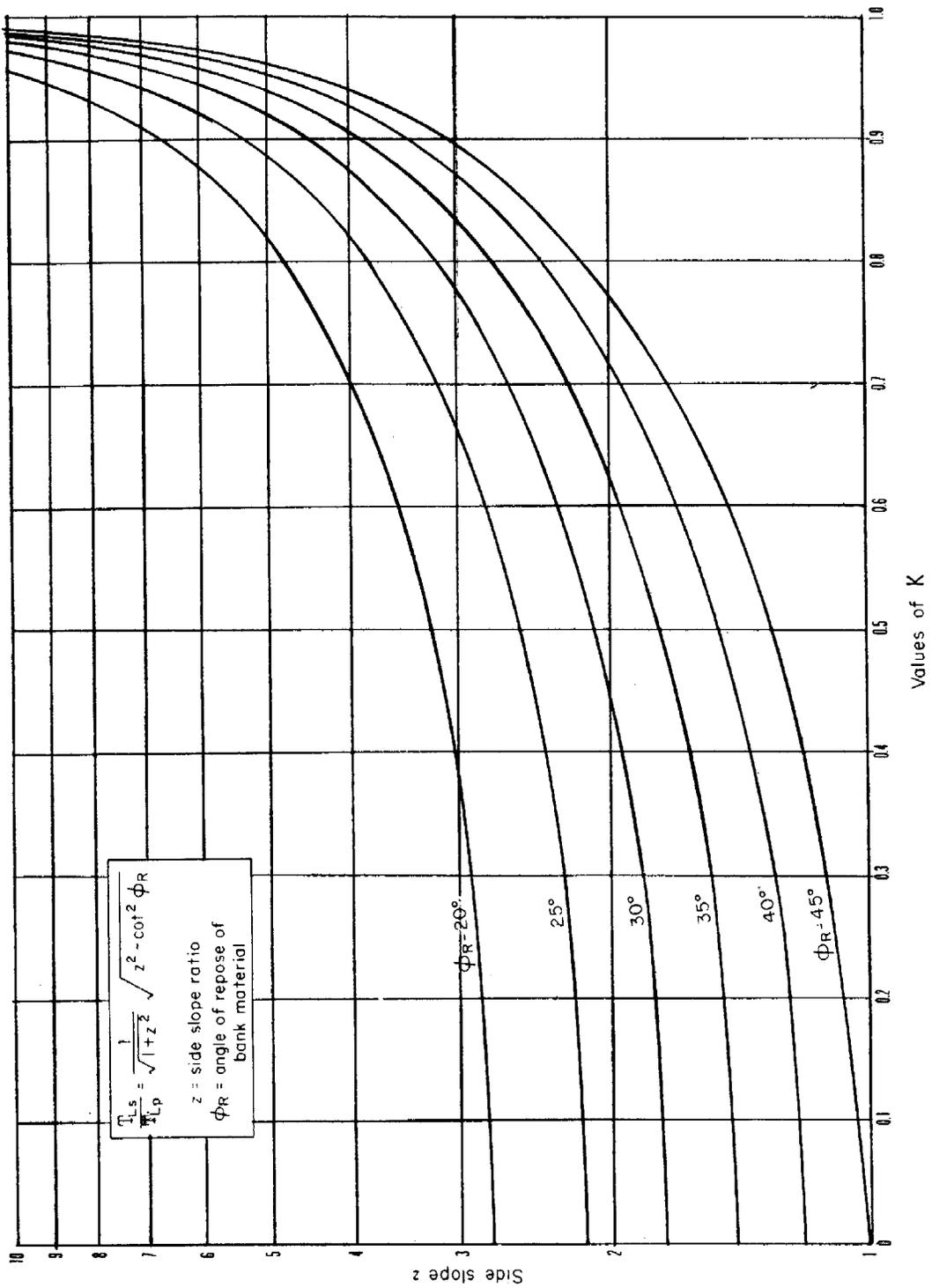
**HYDRAULICS:** Channel stability; angle of repose,  $\phi_R$ , for non-cohesive materials



REFERENCE  
 Bureau Of Reclamation "Progress  
 Report Of Results Of Studies On  
 Design Of Stable Channels" Hy-352

FIGURE 6-7

HYDRAULICS: Channel stability; limiting tractive stress  $\tau_{Ls}$  for sides of trapezoidal channels having non-cohesive materials



$$\frac{\tau_{Ls}}{\gamma L_p} = \frac{1}{\sqrt{1+z^2}} \sqrt{z^2 - \cot^2 \phi_R}$$

$z$  = side slope ratio  
 $\phi_R$  = angle of repose of bank material

REFERENCE  
 Bureau of Reclamation "Progress Report On Results Of Studies On Design Of Stable Channels" Hyd-352

FIGURE 6-8

In a given situation  $\gamma$  and  $s_e$  are known so that the only unknown is  $R_t$ . The value of  $R_t$  can be determined from the logarithmic frictional formula developed by Keulegan and modified by Einstein.<sup>27/</sup>

$$\frac{V}{\sqrt{g R_t s_e}} = 5.75 \log \left( 12.27 \frac{R_t x}{k_s} \right) \quad (\text{Eq. 6-10})$$

$k_s$  is the  $D_{65}$  size in ft.

The factor  $x$  in equation 6-10 describes the effect on the frictional resistance of the ratio of the characteristic roughness length  $k_s$  to the thickness of the laminar sublayer  $\delta$ . This thickness is determined from the equation

$$\delta = \frac{11.6 \nu}{\sqrt{g R_t s_e}} \quad (\text{Eq. 6-11})$$

A relationship between  $x$  and  $k_s/\delta$  has been developed empirically by Einstein<sup>27/</sup> and represented by a curve. With the help of this curve and equations 6-10 and 6-11 the value of  $R_t$  can be determined provided that  $V$ ,  $s_e$ ,  $k_s$  and the temperature of the water are known. The computational solution for  $R_t$  follows an iterative procedure which is rather involved. A simpler graphical solution has been developed by Vanoni and Brooks<sup>48/</sup> and the basic family of curves that constitute it, is shown in Figure 6-9. Figure 6-10 shows the extension of the curves outside the region covered in the original publication.

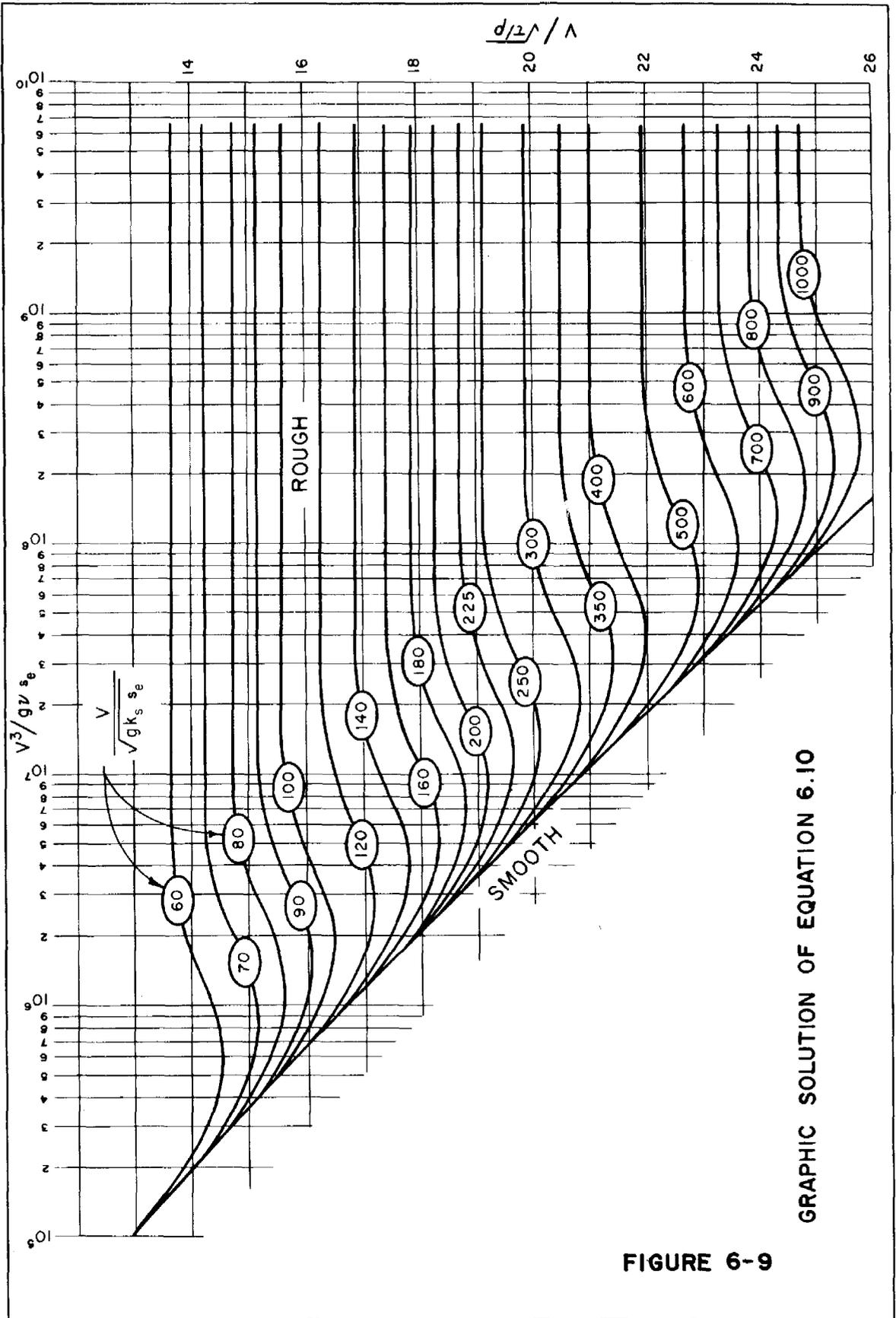
Figure 6-11 gives curves from which values of density  $\rho$  and kinematic viscosity of the water  $\nu$  can be obtained.

The computation of reference tractive stress ( $\tau$ ) is facilitated by following the procedure on page 6-28.

## 2. Distribution of the tractive stress along the channel perimeter:

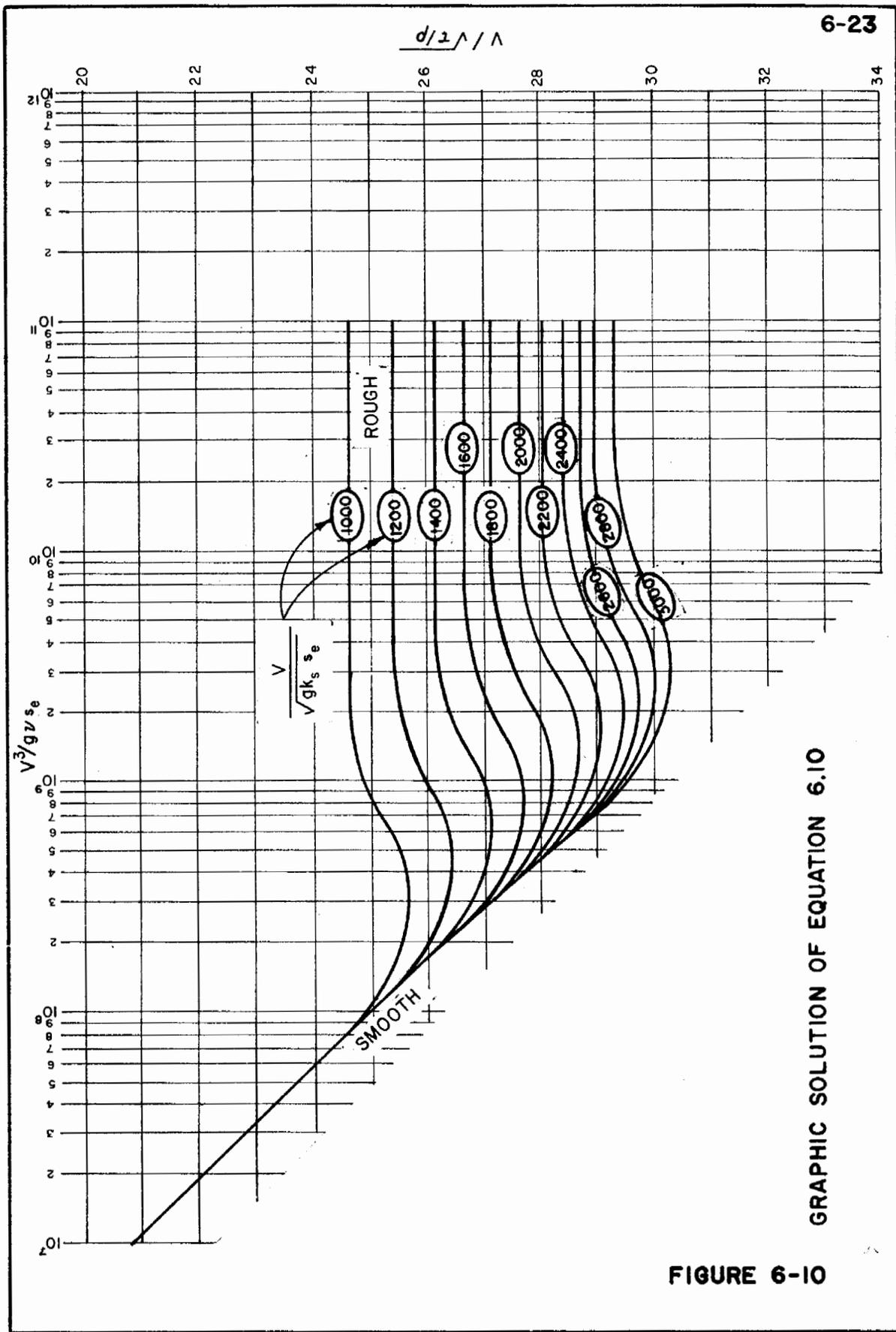
In open channels the tractive stresses are not distributed uniformly along the perimeter. Laboratory experiments and field observations have indicated that in trapezoidal channels the stresses are very small near the water surface and near the corners of the channel and assume their maximum value near the center of the bed. The maximum value on the banks occurs near the lower third point.

The graphs in Figures 6-12 and 6-13 may be used to evaluate maximum stress values on the banks and the bed respectively. These figures are to be used along with  $\tau$ , the reference tractive stress, to obtain values for the maximum tractive stress on the sides and bed of trapezoidal channels in fine grained soils.



GRAPHIC SOLUTION OF EQUATION 6.10

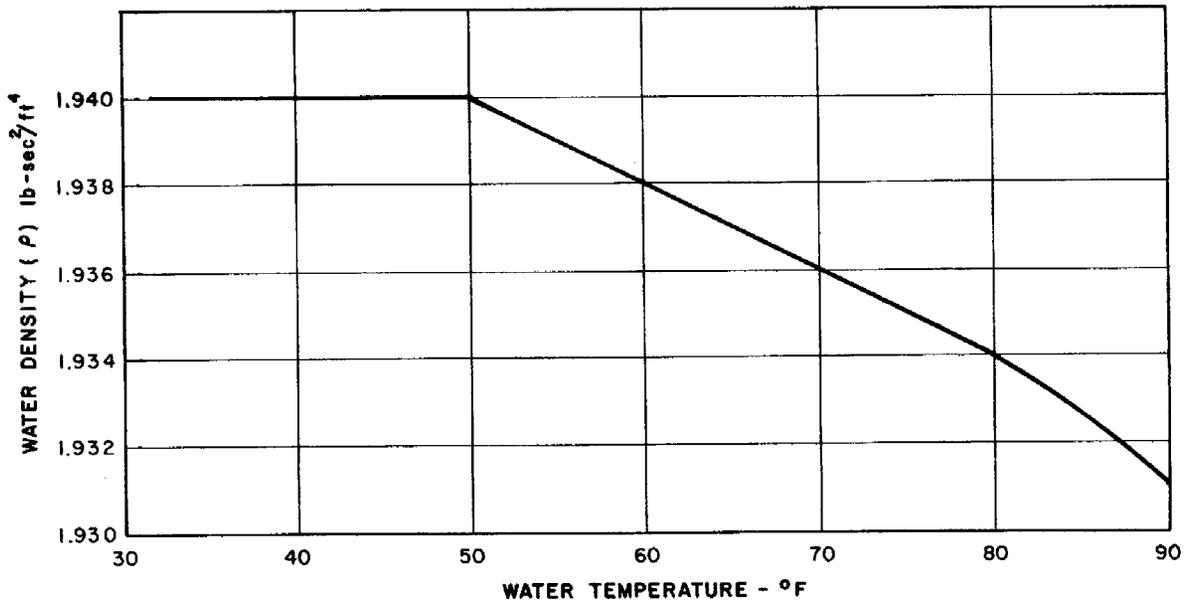
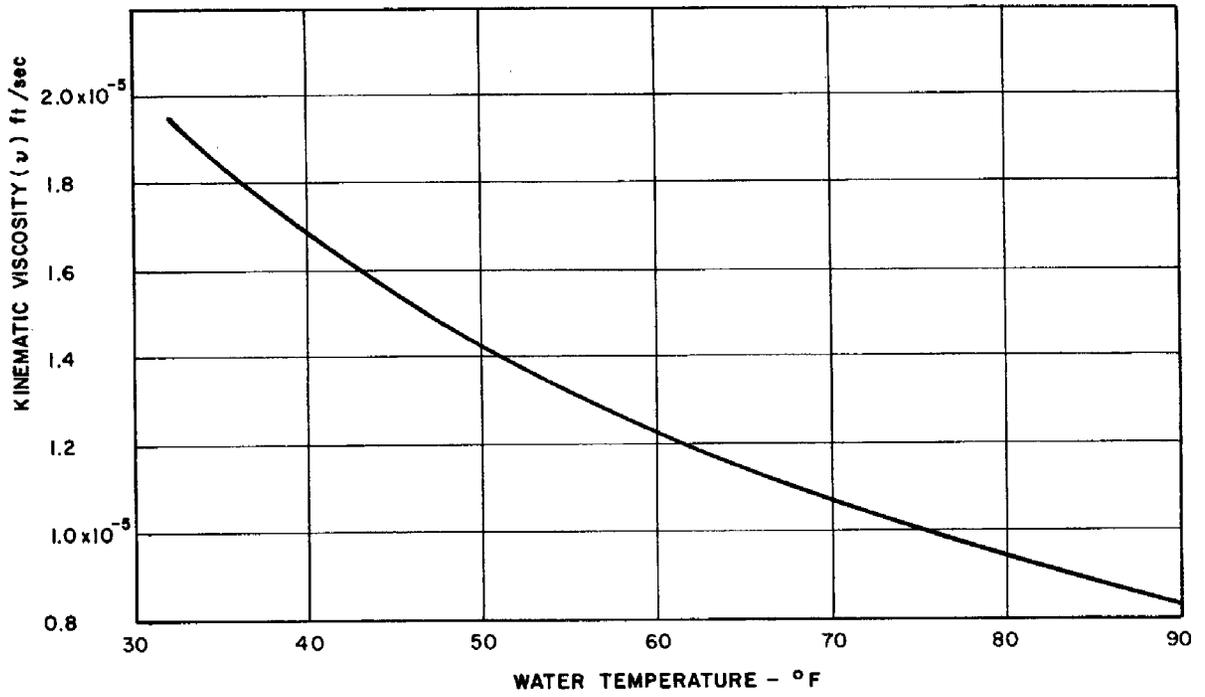
FIGURE 6-9



GRAPHIC SOLUTION OF EQUATION 6.10

FIGURE 6-10

6-24



Values of  $\rho$  and  $\nu$  for various water temperatures

FIGURE 6-11

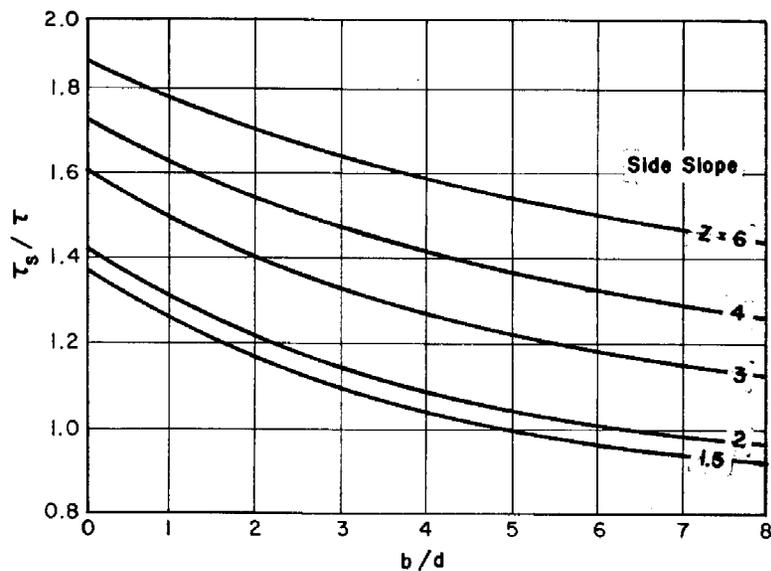


FIGURE 6-12 : Applied Maximum Tractive Stresses,  $\tau_s$ ,  
On Sides Of Straight Trapezoidal Channels.

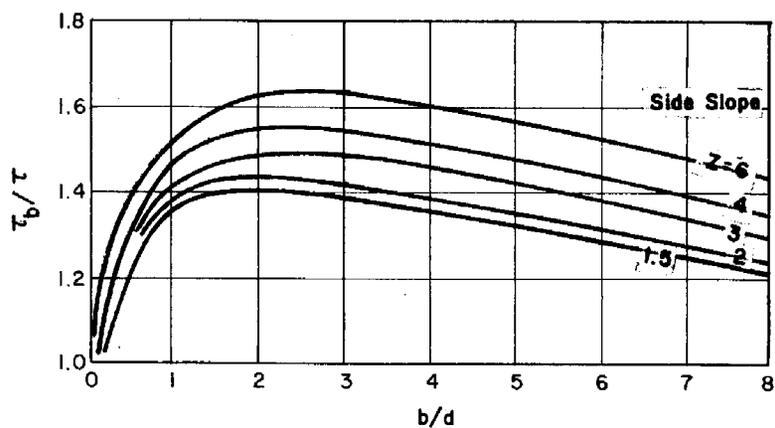


FIGURE 6-13 : Applied Maximum Tractive Stresses,  $\tau_b$ ,  
On Bed Of Straight Trapezoidal Channels.

Curves reproduced from " Tentative Design Procedure for Riprap - Lined Channels " National Cooperative Highway Research Program .Report No.108.

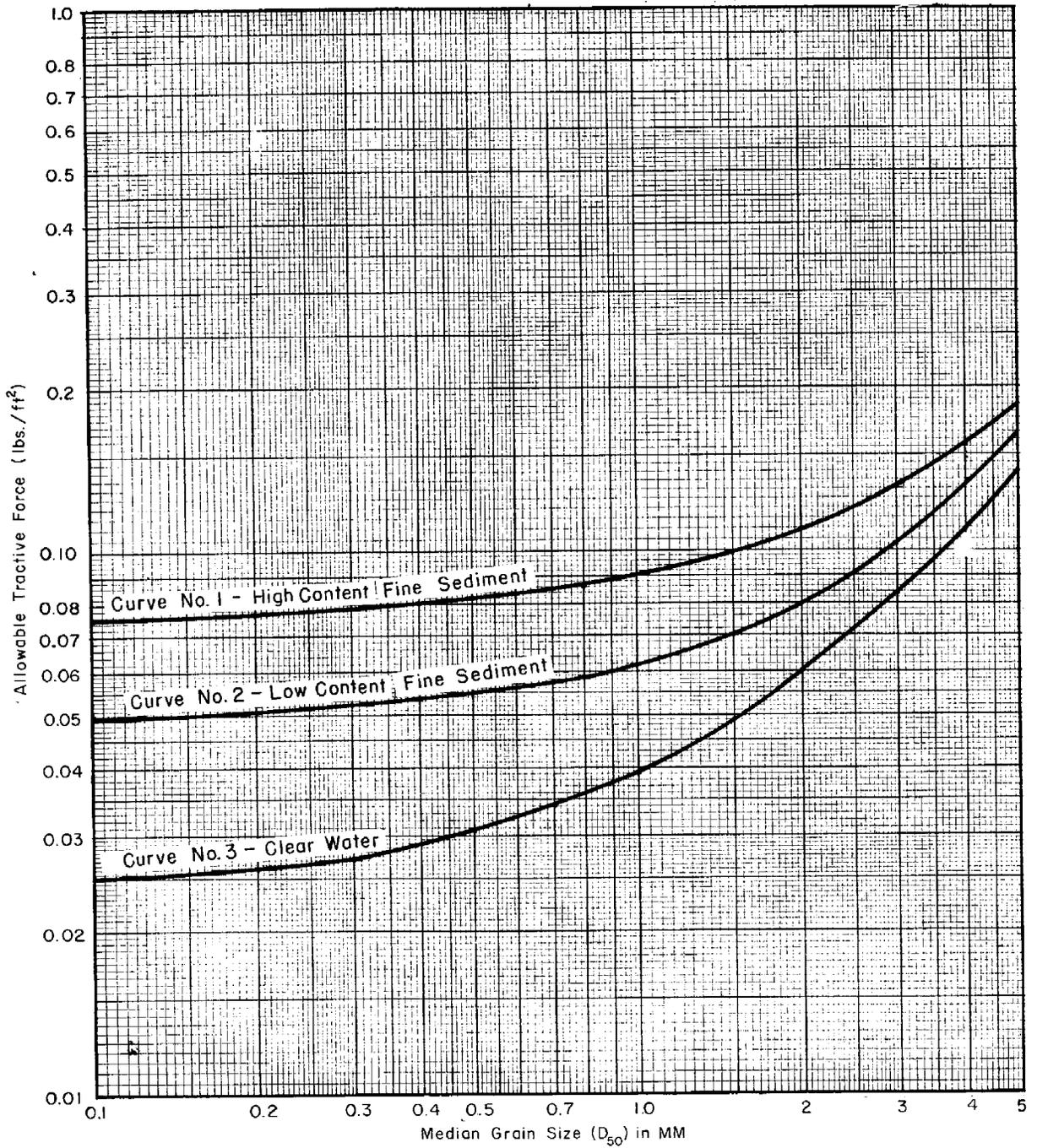


FIGURE 6-14

ALLOWABLE TRACTIVE STRESS  
NON-COHESIVE SOILS,  $D_{75} \leq 0.25''$

REFERENCE: LANE, E. W., "DESIGN OF STABLE CHANNELS", TRANSACTIONS  
A.S.C.E., VOLUME 120

### 3. Tractive stresses in curved reaches:

Figures 6-5 and 6-6 used to determine the maximum tractive stresses in curved reaches for coarse grained soils may also be used to obtain these values for fine grained soils. The values for the maximum tractive stresses on the beds and sides as determined above are used in conjunction with these charts to obtain values for curved reaches.

#### B. Allowable Tractive Stresses - Fine grained soils

The stability of channels in fine grained soils ( $D_{75} < 0.25''$ ) may be checked using the curves in Figure 6-14. These curves were developed by Lane<sup>25/</sup>. The curves relate the median grain size of the soils to the allowable tractive stress. Curve 1 is to be used when the stream under consideration carries a load of 20,000 ppm by weight or more of fine suspended sediment. Curve 2 is to be used for streams carrying up to 2,000 ppm by weight of fine suspended sediment. Curve 3 is for sediment free flows (less than 1,000 ppm).

When the value of  $D_{50}$  for fine grained soils is greater than 5 mm use the allowable tractive stress values shown on the chart for 5 mm.

For values of  $D_{50}$  less than those shown on the chart (0.1mm) use the allowable tractive stress values for 0.1 mm. However, if this is done 0.1 mm should be used as the  $D_{65}$  size in obtaining the reference tractive stress.

#### Procedures - Tractive Stress Approach

The use of tractive stress to check the ability of earth channels to resist erosive stresses involves the following steps:

1. Determine the hydraulics of the channel. This includes hydrologic determinations as well as the stage-discharge relationships for the channel being considered. The procedures to be used in making these determinations are included in Chapter 4 and Chapter 5 of this Technical Release.
2. Determine sediment yield to reach and calculate sediment concentration for design flow.
3. Determine the character of the earth materials in the boundary of the channel.
4. Check to see if the tractive stress approach is applicable. Use Figure 6-1.

5. Compute the tractive stresses exerted by the flowing water on the boundary of the channel being studied. Use the proper procedure as established by the  $D_{75}$  size of the materials.
6. Check the ability of the soil materials forming the channel to resist the computed tractive stresses.

The computation for the reference tractive stress for fine grained soils is facilitated by using the following procedure:

1. Determine  $s_e$  and  $V$ : Evaluate Manning's  $n$  by the method described in NEH-5, Supplement B.
2. Enter the graphs in Figure 6-11 with the value of temperature in  $^{\circ}\text{F}$  and read the density  $\rho$  and the kinematic viscosity of the water  $\nu$ .
3. Compute  $\frac{V^3}{g\nu s_e}$ .
4. Compute  $\frac{V}{\sqrt{gk_s s_e}}$ .
5. Enter the graph in Figure 6-9 (or Figure 6-10) with the computed values in steps 2 and 3 above and read the value of  $\frac{V}{\sqrt{\tau/\rho}}$ .
6. Compute  $\tau$  from  $\frac{V}{\sqrt{\tau/\rho}}$ ,  $V$  and  $\rho$

$$\tau = \frac{V^2 \rho}{(V/\sqrt{\tau/\rho})^2}$$

where the terms are defined in the glossary.

#### Examples - Tractive Stress Approach

##### Example 6-5

Given: A channel is to be constructed through an area of intense cultivation. The bottom width of the trapezoidal channel is 18 feet with side slopes of 1 1/2:1. The design flow is 262 cfs at a depth of 3.5 feet and a velocity of 3.23 fps. The slope of the energy grade line is 0.0026. There is one curve in the reach, with a radius of 150 feet. The aged  $n$  value is estimated to be 0.045. The channel will be excavated in a GM soil that is nonplastic, with  $D_{75} = 0.90$  inches (22.9 mm). The gravel is very angular.

Determine: The actual and allowable tractive stress.

Solution: Since  $D_{75} > 1/4$  inch use the Lane method.

$$n_t = (0.90)^{1/6}/39 = 0.0252 \text{ (from Eq. 6-2)}$$

$$\text{From equation 6-3: } s_t = (n_t/n)^2 s_e = (0.0252/0.045)^2 0.0026 = 0.00082$$

$$\text{actual } \tau_\infty = \gamma_w ds_t = (62.4)(3.5)(0.00082) = 0.179 \text{ psf}$$

$$b/d \text{ (ratio of bottom width to depth)} = 18/3.5 = 5.14$$

$$\text{from Figure 6-3 and 6-4 } \tau_s/\tau_\infty = 0.76; \tau_b/\tau_\infty = 0.98$$

$$R_c/b \text{ (radius of curve/bottom width)} = 150/18 = 8.33$$

$$\tau_{bc}/\tau_b = \tau_{sc}/\tau_s = 1.17 \text{ (Figure 6-5)}$$

$$\text{Actual } \tau_b = (0.179)(0.98) = 0.175 \text{ psf;}$$

$$\text{actual } \tau_s = (0.179)(0.76) = 0.136 \text{ psf}$$

$$\text{Actual } \tau_{bc} = (0.175)(1.17) = 0.205 \text{ psf;}$$

$$\text{actual } \tau_{sc} = (0.136)(1.17) = 0.159 \text{ psf}$$

Solving for allowable tractive stress -

$$\phi_R = 38.4^\circ \text{ (From Figure 6-7)} \quad K = 0.45 \text{ (From Figure 6-8)}$$

$$\text{allowable: } \tau_{Lb} = (0.4)(D_{75}) = (0.4)(0.90) = 0.36$$

$$\text{allowable: } \tau_{Ls} = 0.4 KD_{75} = (0.4)(0.45)(0.90) = 0.162$$

Comparing actual with allowable, the channel will be stable in straight and curved sections.

#### Example 6-6

Given: A channel is to be constructed through an area of intense cultivation. Bottom width of the trapezoidal section is 18 feet, side slopes are 1-1/2:1. Design flow is 262 cfs, with a depth of 3.5 feet at a velocity of 3.23 fps. Slope of the hydraulic grade line is 0.0026. The design temperature is 50° F. The channel will be cut in nonplastic SM soil, with a  $D_{75}$  size of 0.035 inches, a  $D_{65}$  size of 0.01075 inches (0.273 mm) and a  $D_{50}$  of 0.127 mm. The  $n$  value for the channel is 0.045. There are no curves in the reach. Sediment load is quite light in this locality, in the range of clear water criteria.

Determine: The actual tractive stress and the allowable tractive stress.

Solution: Since the  $D_{75}$  size is less than 1/4 inch use the reference tractive stress method.

$$v = 1.42 \times 10^{-5} \text{ ft}^2/\text{sec.}, \rho = 1.940 \text{ lb sec}^2/\text{ft}^4 \quad (\text{Figure 6-11})$$

$$v^3/gv_s e = 3.23^3/((32.2)(1.42 \times 10^{-5})(0.0026)) = 2.83 \times 10^7$$

$$v/\sqrt{gk_s s_e} = 3.23/\sqrt{(32.2)(0.01075/12)(0.0026)} = 373$$

$$v/\sqrt{\tau/\rho} = 21.6 \quad (\text{From Figure 6-9})$$

$$\tau = v^2 \rho / (v/\sqrt{\tau/\rho})^2 = (3.23^2) 1.94 / (21.6)^2 = 0.0434 \text{ psf}$$

$$b/d \text{ (ratio of bottom width to depth)} = 18/3.5 = 5.14$$

$$\tau_s/\tau \approx 1.0; \tau_b/\tau = 1.31 \text{ (from Figure 6-12 and 6-13)}$$

Actual Tractive Stresses:

$$\tau_s = (0.0434)(1.0) = 0.0434 \text{ psf}; \tau_b = (0.0434)(1.31) = 0.0569 \text{ psf}$$

Allowable Tractive Stresses:

$D_{50} = 0.127 \text{ mm}$ ; from Figure 6-14 and assuming clear water flow (curve No. 3) the allowable tractive force is 0.025 psf. Both the bed and the banks of the channel are unstable. An evaluation should be made to estimate the magnitude of scour or possible depth of scour before an armor is formed (Refer to next section). Using this procedure in conjunction with the appropriate sediment transport equations, the magnitude of instability can be evaluated.

#### Formation of Bed Armor in Coarse Material

In material where the coarsest fraction consists of gravel or cobbles an armoring of the bed commonly develops if the allowable tractive stress is exceeded and scour occurs. The depth at which this armor will form may be evaluated if it is determined that some deterioration of the channel can be permitted before stability is reached. The  $D_{90} - D_{95}$  size of a representative sample of bed material is frequently found to be the size paving channels when scouring stops. Finer sizes, such as the  $D_{75}$  may form the armor, once the finer material is eroded. On the other hand, the coarsest particles may not be sufficiently large to prevent scour. The  $D_{95}$  size is considered to be about the maximum for pavement formation within practical limits of planning and design.

The following procedure may be used for determining depth of scour to armor formation.

The actual tractive stress under design hydraulic conditions is computed in accord with equation 6-5. By rearranging this equation

$$D = \frac{\tau_b}{0.4}, \text{ where } D \text{ is the limiting size}$$

For example

$$\tau_b = 0.6 \text{ psf}$$

$$\text{Then } D = \frac{0.6}{0.4} = 1.5 \text{ inches.}$$

Reading from the size distribution curve of a representative bed sample, it is determined that 1.5 inches is the  $D_{90}$  size. With armor customarily forming as a single layer, the depth of scour to formation of a  $D_{90}$  size armor is equal to the  $D_{90}$  size in inches divided by the percentage of material equal to or larger than the armor size. For this case  $1.5 \div 0.10 = 15$  inches (1.25 ft.) depth to armor formation.

Armoring of the bed will not usually develop initially as a flat bed across the channel. After forming an armor along the thalweg, bars of finer material will next be removed, followed by an increasing attack on the banks.

### Tractive Power Approach

#### General

In general the observations, assumptions, and computational methods used in the development and use of the allowable velocities and the tractive stress methods of analysis are based on correlating a soils erosion resistance with simple index properties determined on disturbed samples. These methods at their present state of development do not assess the effects of cementation, partial lithification, dispersion, and related geologic processes on the erosional resistance of earth materials.

This limitation has been recognized for many years. In the early 1960's, efforts were made by SCS in the Western states to evaluate the stability of channels in cemented and partially lithified soils. The procedures resulting from this effort have come to be known as the Tractive Power Approach.

In this approach the aggregate stability of saturated soils is assessed by use of the unconfined compression test. Field observations of several channels were evaluated against the unconfined compressive strength of soil samples taken from the same channels. The results are shown on Figure 6-15. Soils in channels with unconfined compressive strength versus tractive power that plot above and to the left of the S-line on Figure 6-15 have questionable resistance to erosion. Soils in channels with unconfined compression strength versus tractive power that plot below and to the right of the S-line can be expected to effectively resist the erosive efforts of the stream flow.

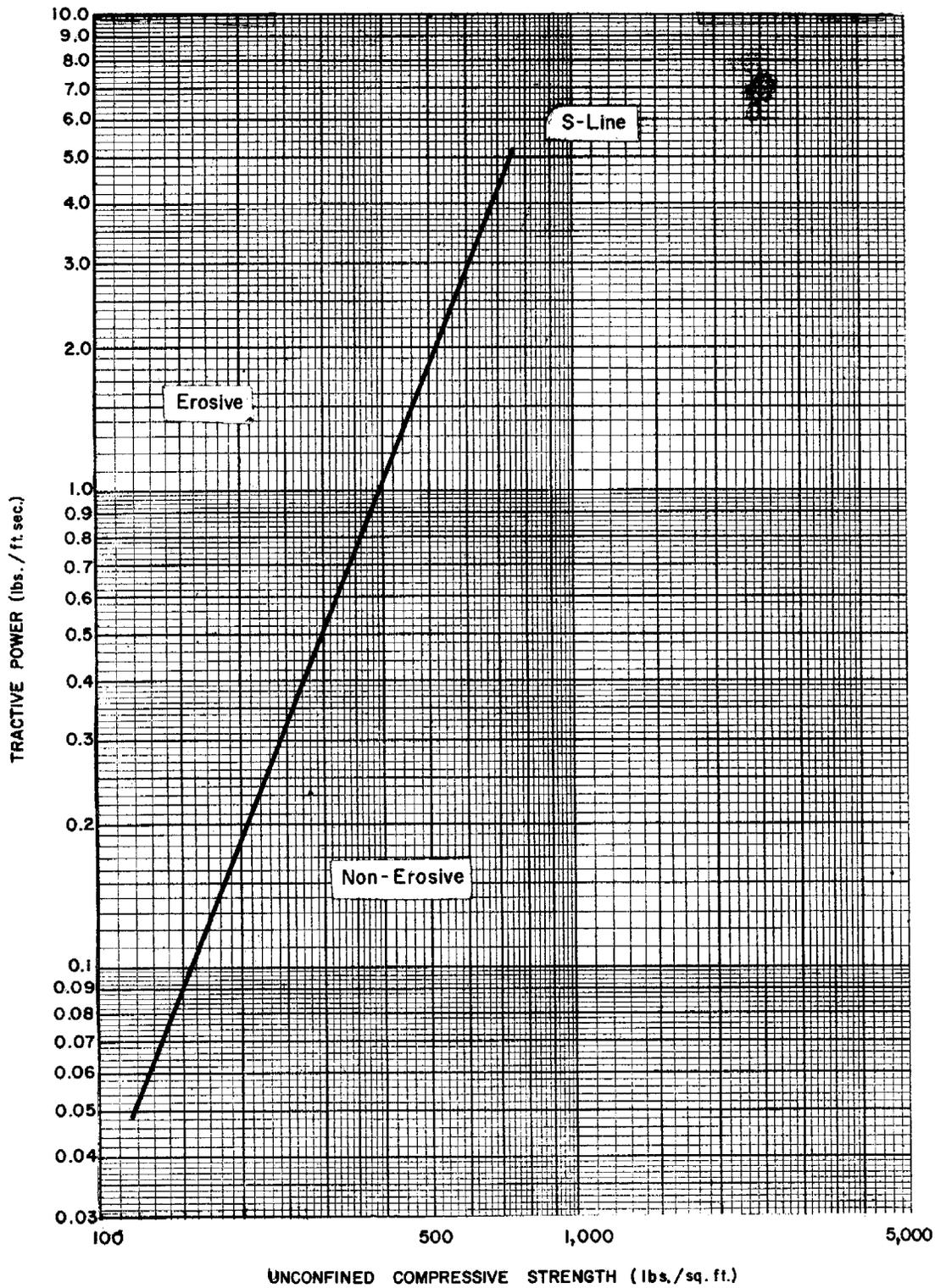


Figure 6-15  
 Unconfined Compressive Strength And Tractive  
 Power As Related To Channel Stability

Tractive power is defined as the product of mean velocity and tractive stress. Use the appropriate method based on soil characteristics as described in the tractive force procedure to calculate the tractive stress.

#### Procedure - Tractive Power Approach

The use of tractive power to evaluate earth channel stability involves the following steps:

1. Determine the hydraulics of the channel. This includes hydrologic determinations as well as the stage-discharge relationships for the channel being considered. The procedures to be used in making these determinations are included in chapters 4 and 5 of this Technical Release.
2. Evaluate the sediment transport carrying capacity in the design reach to (a) determine the sediment concentration and (b) test the possibility for aggradation.
3. Determine the physical characteristics, including the saturated unconfined compressive strength of the earth materials in the boundary of the channel. The procedures for making this determination are included in chapter 3 of this Technical Release.
4. Check to see if the Tractive Power Approach is applicable. Use Figure 6-1.
5. Compute the tractive power of the flows being evaluated. Use the mean velocity determined in step one and the procedures in this chapter to determine the tractive stress. Use the method that is appropriate for the grain size of the channel materials.
6. Determine the erosion resistance of the materials in the channel boundary from Figure 6-15.

The following example illustrates the use of the Tractive Power Approach to evaluate channel stability:

#### Example 6-7 - Tractive Power Approach

Given: A channel is to be constructed for the drainage of an area of moderate cultivation. Its bottom width is to be 46 feet, side slopes 2-1/2:1, design flow depth 15.0 feet, and estimated  $n$  value of 0.03. The channel will be excavated in clayey silt (ML) having a Plasticity Index of 3 and a  $D_{75}$  size of 0.15 mm, a  $D_{65}$  size of 0.00256 inches (0.065 mm), and an unconfined compressive strength of 790 psf. The hydraulic gradient is 0.00042, as determined by water surface profile calculations. There are no curves in this reach of channel. The water temperature for the period under consideration is taken as 50°F.

Design flow is 4750 cfs at a depth of 13.5 feet and a velocity of 3.77 fps.

Determine: The actual tractive power and evaluate the stability of the channel.

Solution: Since the  $D_{75} < 1/4$  inch use the reference tractive stress method.

$$v = 1.42 \times 10^{-5} \text{ ft}^2/\text{sec}; \rho = 1.940 \text{ lb sec}^2/\text{ft}^4 \text{ (Figure 6-11)}$$

$$V^3/gv s_e = 3.77^3/((32.2)(1.42 \times 10^{-5})(0.00042)) = 2.79 \times 10^8$$

$$V/\sqrt{gk_s s_e} = 3.77/\sqrt{(32.2)(0.00256/12)(0.00042)} = 2220$$

$$\text{From Figure 6-10 } V/\sqrt{\tau/\rho} = 27$$

$$\tau = V^2 \rho / (V/\sqrt{\tau/\rho})^2 = (3.77^2) 1.940 / (27)^2 = 0.0378$$

$$b/d = 46/15 = 3.07; \tau_s / \tau = 1.21 \text{ (Figure 6-12)}$$

$$\tau_b / \tau = 1.45 \text{ Figure 6-13}$$

$$\tau_s = (0.0378)(1.21) = 0.0458$$

$$\tau_b = (0.0378)(1.45) = 0.0548$$

Use the larger of  $\tau_s$  or  $\tau_b$  to compute tractive power.

$$\tau_b V = (0.0548)(3.77) = 0.207 \text{ - Actual Tractive power}$$

The tractive power versus unconfined compressive strength plots well into the non-erosive zone (Figure 6-15). The channel should be stable for the design conditions.