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Design of Open Channels

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Technical Release No. 25

U.S. Department of Agriculture
Soil Conservation Service
Engineering Division
Washington, D.C. 20250

UNITED STATES DEPARTMENT OF AGRICULTURE
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TECHNICAL RELEASE NOTICE 25-5

This technical release notice transmits a new edition of Technical Release No. 25 titled "Design of Open Channels." It supersedes the version of Technical Release No. 25 dated December 15, 1964, and all subsequent revisions. Technical Release Notices 25-1 through 25-4 are canceled.

New material on the use of landscape architecture has been added to Chapters 2 and 7. Chapter 6 has been revised to include current state of the art concepts for channel stability analysis.

Geology Note 2 is being prepared and should be ready for issuance in the near future. The Note is referred to in Chapter 6. A new Chapter 8 on "Landscape Architecture Design" is also being developed and will be added to Technical Release No. 25 when available.

NEIL F. BOGNER
Director
Engineering Division

Enclosure

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DESIGN OF OPEN CHANNELS

Technical Release No. 25

PREFACE

This technical release was prepared by Soil Conservation Service specialists for the use of field personnel. It supersedes Technical Release No. 25 dated December 15, 1964, and all revisions to this document. Previous editions should be discarded. This version incorporates visual resource and environmental considerations and updates stability analysis information.

The technical release covers procedures for design of open channels and related measures such as floodways. Cirteria and standards applicable for each situation should be used in conjunction with these procedures.

Designers of open channels should find the technical release useful in considering the numerous technical aspects that are important to sound channel modifications. Close coordination of many technical fields is important if channels with minimum environmental impacts are to be developed.

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DESIGN OF OPEN CHANNELS

CHAPTER 1. GENERAL CONSIDERATIONS

This guide defines the technical aspects of open channel planning and design. Channel design involves the fields of hydrology, geology, hydraulics, drainage, irrigation, erosion control, soil mechanics, landscape architecture* and structural design. Adequate planning and design require close coordination of these technical fields.

Open channels are used to serve a variety of functions. These include flood protection, drainage, irrigation, diversion of water to control erosion and sedimentation, and for recreational and other purposes, either singly or in multiple purpose combinations.

The contents of this technical release are primarily directed at the design and planning of floodways and open channels where channel erosion is of primary concern. The planning and design of low gradient channels such as irrigation canals, drainage ditches, grassed waterways and so on are covered in other service handbooks.

Channel Planning

The objectives of the project first must be clearly defined. The level of development or adequacy of improvements for each purpose and the general location and layout also needs to be resolved in early stages of planning. Economic and other factors which are outside the scope of this guide should be considered with the engineering alternatives, based on the procedures presented herein.

After the functional requirements of the channel have been determined in planning channel work, the following steps in preparing open channel plans and design are necessary:

1. Field surveys to provide topographic information and other physical data pertinent to location and hydraulic and structural design.

2. Geologic and soil investigation and testing of soil materials for planning and design requirements.

3. Analysis of hydrologic or other conditions to determine channel capacity needed to meet project objectives.

* A Glossary of technical terms for Landscape Architecture may be found in Chapter 2, Appendix A.

4. Analysis of geomorphology viewsheds, visual resource values and landscape use values to determine visual resource planning and design requirements.

5. Use of the above data in developing the engineering aspects of project formulation.

In the formulation stage, physical conditions should be determined and an appraisal should be made of other limitations imposed on channel work that might limit functional performance in meeting objectives. These include:

1. Adequacy of outlets to handle required flow without excessive scour or deposition, and without damaging downstream flooding.

2. Protection of water rights.

3. Availability of rights-of-way.

4. Satisfying minimum flow requirements.

5. Environmental considerations including landscape resources that must be conserved.

Adequacy of Outlets

Basic requirements. - - In determining adequacy of outlets, the following basic requirements should be met:

1. The capacity of the outlet should be such that the design flow from its watershed can be discharged into it at an elevation equal to or less than that of the hydraulic gradeline that is used for design of the project. The storm used for this analysis should have the same chance of occurrence as the storm used for design of the improvements.

2. Where subsurface drainage is needed, the depth of the outlets should be such that subsurface drains may be discharged into it above normal low water flow.

3. The capacity of the outlet should be such that the discharge from the project watershed, after proposed improvements, will not result in stage increases that will cause significant damages below the termination of the project channel.

4. Flow conditions in the outlet should be capable of maintaining equilibrium with the sediment transport of the project channel. There should not be excessive scour or deposition of sediment in the outlet.

Evaluation of project effects. - - Many items must be considered in evaluating project benefits, among these are:

1. The stage-discharge relationship of the project channel, including overbank-flow, should be determined for "before" and "after" project conditions.

2. Stage-discharge curves for the outlet should be developed by computing the water surface profile through two or more cross sections below the outlet using the existing roughness coefficient.

3. The effect of project improvements on stages in the outlet should be analyzed for storms of at least two recurrence frequencies. Storm frequencies selected for the analysis should be that used for project design and one other significantly different from the design discharge.

   Procedures outlined in National Engineering Handbook, Section 4, Hydrology, Part 1, Watershed Planning, shall be used for the analysis.

4. Where downstream effects of channel improvement are significant (stage increases will cause damages below the termination of the project channel) an analysis to determine effects should be carried downstream to the point where effects have been dissipated.

5. Geologic and soil investigations to determine the effect of project improvements on the stability of the outlet should be made as outlined in Chapter 3, "Site Investigations."

Special outlet conditions. - -

1. Tidal Influence. Where channel improvements discharge into rivers, estuaries, bays, and sounds, which are subject to tidal influence, the effect of the tides on discharge from the channel should be determined. This is true if the outlet is a tidegate, which opens and closes according to the relative elevations of the tide and the hydraulic grade of the channel; or if the channel discharges directly into tidewater, without a tidegate.

   The characteristics and types of tides are discussed fully in "Tidal Datum Planes."1/ Annual editions of "Tide Tables, High and Low Water Predictions" are available from the Coast and Geodetic Survey, U.S. Department of Commerce.

2. Pumping. Where the project is provided with pumps to discharge the runoff from the watershed, the area may be protected by levees.

1/ Numbers refer to numbered references at end of this technical release.

In this case the levees should meet the National Engineering Standards for Dikes and Levees, the pumping capacity should be adequate for the design runoff, and the outlet into which the pumps discharge should meet the basic requirements for outlets.

Legal Requirements

Applicable provisions of state water laws must be met in all channel work. Existing water rights may require protection or rebuilding of diversion works, control structures for regulating flows, etc.

Preliminary investigations should determine the existence of any water rights on channels to be improved and the limitations that they may impose on the improvements.

Rights-of-Way

Acquisition of rights-of-way is an essential element of channel improvement. Careful consideration of this element from the preliminary investigation or reconnaissance stage through the construction stage will greatly expedite the job and reduce its cost. Certain modifications of location and alignment, within the limits of good design, may be made to ease rights-of-way problems.

Environmental Considerations

Landscape resources shall be given full consideration from the preliminary investigation through final inspection. These resources include visual, fish, wildlife and recreational and may include areas outside of the right-of-way that are affected environmentally as a result of the project. A guide for planning and design considerations related to wildlife, fish and recreation resources may be found in Chapter 7.

A guide to visual resource data needed in planning and design may be found in Chapter 2, Appendix A. Visual changes in the landscape that result from channel work should be examined. The before and after appearance of the site should be documented and discussed with the local community and other interested individuals or groups. Procedures discussed in Chapter 8 shall be used for designing the visual resource.

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CHAPTER 2. FIELD SURVEYS & PLAN LAYOUT

General

Objectives of the Engineering Survey

The surveys for channel improvement work should be in sufficient detail to:

1. Determine the location, hydraulic properties, visual characteristics and condition of existing channels and associated structures.

2. Determine the needed improvements on existing channels and the required additional channels and appurtenant structures.

3. Determine the cost of the needed improvements.

4. Prepare suitable landrights work maps for easements and working permit requirements.

These objectives apply to both planning surveys and final design surveys and differ only in the amount of detail and precision needed for the respective stages of project development.

Preliminary Surveys

Information Needed

To accomplish the objectives stated above, the following information is needed for planning channel work:

1. Drainage area at junctions of tributaries and all flow control points. Drainage areas also should be delineated for valley sections used for hydrologic and economic evaluations where these are needed at locations other than at junctions of tributaries and structural control points.

   The drainage area determinations, including those needed for drainage purposes, should be made carefully for use in both the planning and the final design stages of the project.

   Where topographic quadrangle maps are available they are usually sufficiently accurate for delineating and planimetering the required drainage areas, except on extremely flat land. Field surveys may be required to determine watershed divides on flat land. Where feasible, the delineation of drainage areas should be checked by means of a stereoscopic study of 4-inch or 8-inch = 1 mile aerial photos.

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All maps, especially in flat topography, should be field checked. This step should be done preferably in the preplanning stages to avoid the need for revisions as the planning work progresses.

2. Approximate profiles of the existing channel showing the elevation of the existing channel bottom, low bank, points of natural low ground away from, but subject to, drainage into the channel, and elevation and dimensions of all structures in or over the channel. In flat areas, occasional topography or perimeter and spot elevations may be needed to determine the drainage pattern. Existing tributaries should be located and sufficient bottom and ground elevations obtained to permit correlation of hydraulic gradelines and design of any grade control structures that may be needed.

3. Representative channel and valley cross sections for each hydraulic or economic reach. Additional channel cross sections should be taken as needed for reliable estimates of quantities of excavation and clearing, to determine easement requirements.

4. Mannings coefficient "n" for each channel and valley cross section. The "n" value should be representative of the hydraulic reach to which the section applies, except that where segments of a cross section differ significantly in flow retardance factors either within the channel, between the channel and the flood plain, or between segments of the flood plain, separate "n" values should be recorded for each segment.

5. The location and elevation of all soil investigation sites along the proposed channel.

6. The location of viewpoints and viewsheds along the proposed alignment.

7. The landscape character and use patterns along the alignment.

8. Stationing and delineation of apparent ownership boundary lines in the vicinity of probable channel improvement work.

9. Data including-dimensions, elevations, kinds of material, and condition of existing structures such as bridges, culverts, drops, and dams.

10. Data including acreage and density of brush, trees and debris on clearing required.

11. Other significant features affected such as roads, pipelines, power and telephone lines, buildings, wells, cemeteries, and fences. Such features should be located on aerial photographs or base maps and the elevations of strategic points recorded.

Survey Procedures

The survey procedure for "Preliminary Surveys" outlined in the National Engineering Handbook Section 16,2/Chapter 2, is applicable. This procedure should be augmented by "Landscape Architecture Survey and Analysis" as outlined in Chapter 2, Appendix A. Technical Release-62, (TR-62), contains guidance for notekeeping and stationing.

Horizontal Control

Where suitable 8-inch aerial photographs or photo mosaics are available which show sufficient detail to locate and identify an existing channel that can be used as a base line, it will not be necessary to run a transit traverse in the planning stage to establish a base line. Other existing maps or plans of equivalent accuracy sometimes are acceptable for this purpose.

Horizontal control for the planning and design of stream channel work varies with the survey method selected. In most cases, sufficiently accurate horizontal control can be obtained from semicontrolled photo mosaics for both planning and preliminary design of channel work. The photo mosaic also may be used to show drainage areas, flood plain area, control elevations, channel locations, land ownership, etc.

The following steps are involved in obtaining semicontrolled photo mosaics:

1. Using the latest available aerial photos of the flood plain (preferably the 4" = 1 mi. scale), select two points near the center of each aerial photo, which can be identified on both the aerial photo and on the ground. These points should be at least 500 feet apart and, preferably parallel with the flood plain. Some additional accuracy may be obtained by selecting a second line, approximately perpendicular to the first, on each photo. To facilitate identification and measurement, these lines should be selected along established lines, such as roads or fences. Also, delineate the approximate area of the flood plain on the aerial photos.

2. Measure the distance to the nearest foot between the selected points in the field. Stadia distances may be used for lines up to about 1000 feet long. Identify the points on the photos by a small pin prick at each point. Circle the pin points on both the front and back of the photo. Also, number or letter each point for identification and mark the distance between the points on the back of each photo. For a permanent reference, record each measurement in a field notebook as follows:

Photo No.	Date	Point	Line	Approx. Brg.	Length	Description
DGB-2E 110	27 Apr. 1949	A				¢ rd. at bridge Roanoke Chan.
			A-B	NE	548'	
		B				N.W.cor.fld.N. of rd. and E. of Chan.
DGB-2E 112	27 Apr. 1949	C				S.W.cor.fld.with N-S lines, N. of Roanoke Chan.
			C-D	ENE	942'	
		D				S.E.cor.fld.with N-S lines, N. of Roanoke Chan.
DGB-2E-4S	27 Apr. 1949	E				¢ rd.in line with N-S fence W. of barn about 300' W. of rd. junc.
			E-F	SSW	736'	
		F				¢ rd.in line with N-S fence on W. side of L-shaped fld. S. of rd.

3. Have prepared from these photographs a semicontrolled photo mosaic of the flood plain to a scale not smaller than 1" = 400'. A film positive should be prepared of the aerial mosaic on 22" x 30" planprofile sheets with the aerial mosaic as the plan portion of the sheet. At least two copies of each film positive are suggested- one for use in preparing a problem location map and profile of the channel and flood plain, and the other for use in preparing the location plan and profile of channel improvement. Also, ozalid prints of each sheet are suggested for use, as working copies by all planning specialists in planning and evaluating project works of improvement.

4. In areas for which USGS topographic maps (scale 1" = 2000') have been completed recently by photogrammetric methods, the aerial mosaic may be semicontrolled to these USGS quadrangle sheets. In this case, the steps outlined in items 1 and 2 above would not be needed.

Where semicontrolled photo mosaics are used for horizontal control, the location of stream channel improvement may be shown on these mosaics. Other existing maps or plans of equivalent accuracy sometimes are acceptable for this purpose.

The following steps are needed:

1. Draw a preliminary centerline in pencil on the mosaic, showing curves, intersecting angles, etc. Plot and measure intersection angles by the tangent method for accuracy. To locate the center of curvature accurately, erect perpendiculars - do not use triangles.

2. Walk the full length of the flood plain, noting on an ozalid print of the aerial mosaic:

   Probable channel realignment,
   Points of significant breaks in grade,
   Location of all channel cross sections obtained with a hand level,
   Location of all rock outcrops or critical soil conditions,
   Approximate locations where valley cross sections may or should be obtained,
   Location of significant tributary junctions and places where side inlets may be needed,
   Location of all homes, institutions, roads, parks and recreation areas that maybe effected by the channel work. These include but are not limited to facilities that are within 500 feet of the alignment,
   Location of tree areas and large individual trees adjacent to the alignment,
   Location of utility crossings, such as: powerlines, telephone lines (aerial or buried) and pipelines. Obtain hand-level cross section of the channel at all visible pipeline crossings to show general relation of the pipeline elevation with the present channel bottom and bank elevations, and
   Location of fence lines and/or apparent property lines, foot bridges, etc., if not already visible on aerial photo.

3. Make a photo record of the landscape surrounding the project and record the viewsheds of the project from view points.

4. Following the field check, accurately establish the revised centerline on the photo mosaic. The final selection of alignment should be based on full use of all cross section, geologic and environmental data.

5. Show the actual location of all surveyed cross sections on the photo mosaic.

If suitable aerial photographs are not available it will be necessary to establish a base line in the field by means of an instrument survey. Where feasible, the base line should be located close to the improvement area but outside of brushy or wooded areas and any probable construction areas. This will facilitate survey work and avoid the necessity of establishing a new baseline at the time of construction. This is usually done by staking a series of tangent lines offset a convenient distance from the centerline of the existing channel. It is recommended that all points of intersection (PI's) of the tangent lines be staked with metal pins, when practicable, for later use in final design and construction layout surveys. A closed traverse survey should be made on this baseline. For closed traverse surveys, the error of angular closure should not be greater than $1.5\sqrt{N}$ minutes, N being the number of angles in the traverse. Horizontal closure of chained distances should not exceed 1.0 foot per 1000 feet of traverse length. All traverse angles should be doubled and checked by comparing computed bearings with observed magnetic bearings.

The use of a coordinate system will often facilitate the drafting and layout of complex horizontal controls. The coordinates should be related to existing systems in the area, where established, or to an assumed origin located so that all traverse points will plot in the same quadrant.

Vertical control

Vertical datum. - - Mean sea level datum is recommended for all channel improvement work.

Bench marks. - - Reasonably permanent bench marks should be established in the work plan stage at tributary junctions, at or near bridges or culverts where channels intersect roads, and at the beginning and end of channels. In addition, bench marks are suggested at approximately one-half mile intervals along the valley to facilitate the survey of valley cross sections and the preparation of an adequate profile.

All bench marks should be set by making turns through each bench mark; they should never be set by side shots from the level circuit. Level circuit surveys should be closed within the required degree of accuracy. The bench mark may be a plate or cap set in concrete, a plate on a concrete post similar to those used by USGS, or other similar means that will give the same degree of permanency. Manufactured survey markers, consisting of metal stakes with bronze caps are sometimes used because they are relatively inexpensive and can be set quickly and easily. Where manufactured markers are not used, it is advantageous to tag each bench mark with an aluminum tag showing the bench mark number and elevation.

Permissible error of closure. - - For most channel work, vertical control is of paramount importance. Channel designs are dependent upon the amount of fall between hydraulic control points. Therefore, "third order" leveling (i.e., .05 times the square root of the length of level circuit in miles) should be maintained for all bench level circuits, including subcircuits within larger circuits. These bench marks should be established to the specified degree of accuracy in the planning stage for use in both the planning and final design surveys.

Stationing

Main channel. - - The base line should be stationed in conformance to Engineering Memorandum SCS-39.

Where semicontrolled aerial mosaics are used to show the location of channels, mark the stationing on the centerline with a short dash at each 200-foot point.

Determine the stations at all cross sections and the pertinent features noted during the field reconnaissance by scaling the distances between points on the aerial mosaic. Although the points and the cross sections may be numbered both on the mosaic and in the notebook, these points also should be stationed in the field notebook as a permanent record. Thus, complete profile notes are obtained without actually measuring the distances in the field.

Where preliminary location surveys are obtained by means of an instrument survey, stationing is accomplished by measuring with a chain or stadia in the field.

Tributary channels. - - Base lines on tributary channels should be tied into the main channel base line.

Reaches

The length of design reaches normally is governed by the distance between points where a change of elements of the channel occurs, such as the entry of a side tributary or a change in gradient, depth, width, etc. Where the elements of a channel are constant for relatively long distances, the reach should be subdivided so that no design reach will exceed about 1/2 mile in length.

Cross Sections

Valley flood plain. - - The survey of valley cross sections used to develop water surface profiles for damage evaluation should be extended on each side of the valley to be above the maximum high water mark to permit computation of flood plain storage and to establish the vertical relationship between viewers and the channel work. Normally, this work is done in sufficient detail in the planning stage to serve for both the planning and final design requirements.

Sufficient points should be surveyed to represent adequately the hydraulic and visual characteristics of the cross section. Elevations should be established at all significant changes in slope. The distance between points on the cross section should not exceed 300 feet where the slopes are relatively uniform.

Channels. - - Channel sections are required to make reliable estimates of quantities and to determine easement requirements. They should extend far enough to permit improved channel alignment without additional surveys or at least 50 feet beyond the expected right-of-way, whichever is greater. The allowable distance between cross sections will vary with channel and valley conditions. Normally, 300 to 1000 feet are sufficient for work plan estimates. These sections should be tied into the same basic datum as the other sections in the watershed.

Structures

The condition and serviceability of all structures should be recorded. Adequate survey data are needed for all bridges and culverts in order to compute the carrying capacity for each. A minimum of three cross sections is needed at each structure; i.e., an inlet or approach section, a section perpendicular to the direction of flow through the structure, and an exit section. The entrance and exit sections usually are taken approximately 50 feet from the respective ends of the structure. The section through the structure should include the size of the opening, size of bridge piers, abutment footing elevations, and elevations of the bottom of bridge girders and the road surface. This section should be extended along the centerline of the road on either side of the structure beyond any probable overflow elevations. The grade and length and invert elevations of all bridges and culverts also should be obtained. See National Technical Release 14.

The three sections described for bridges and culverts also should be taken for all grade control structures along the channel. Similarly, cross sections are needed for all rock ledges which act as grade or hydraulic control sections.

Visual Resources

The data necessary to identify, plan, and design the visual resources will vary considerably among projects and may vary among parts of the same project. For example, given a flat terrain covered solely by a crop pattern with only stream way trees, the needed data may include only recording the possible road viewsheds and noting how the treetop edges contribute visual variety or a visual sense of space. However, in an urban landscape it may be necessary to pinpoint the locations of landscape elements such as large trees, accoustical and visual screen shrub masses, and landforms. Landscape architecture survey outline including a visual resource survey appears in Chapter 2, Appendix A. A more detailed discussion of visual resources and landscape architectural design appears in Chapter 8.

Plotting Data

General. - - Plan, profile, and cross section data obtained during the planning stage should be neatly plotted for use in preparing preliminary drawings. The scales used for plotting the data should be selected so that the information can be clearly shown on as few sheets as practical

Plan data. - - The plan should show the location of the existing channel; base line; cross sections; apparent property lines; general land use of adjacent areas; bench marks; existing vegetation, noting general size and specie, especially individual trees with greater than 6-inch diameter and trees with less than 6-inch diameter that are unique or not common for the watershed; roads, bridges and culverts; houses and institutional buildings that are within or adjacent to right-of-ways; and any utilities that may fall within the planned right-of-way. The base line should show either azimuths, bearings, or deflection angles. They may be shown as traced from aerial photographs. The direction and stationing of the cross sections should be shown on the plan. The plan may be made from an overlay of corrected 8-inch aerial photographs, plane table sheets, or a plot made from transit survey field notes. If coordinates are used, they should be shown on the plan.

Profile. - - Data obtained from the survey of valley cross sections and cross sections surveyed at hydraulic structures, supplemented if necessary with information from available USGS quadrangle sheets, should be used to plot a profile of the existing channel and valley. Standard sized sheets of profile paper or plan-profile paper should be used for this purpose, to facilitate preparation of preliminary drawings. (See Figure 2-1.)

The profile should show the water surface of the design flood for present conditions; the present ground line; the existing bottom grade; cross section locations; soil investigation sites; the elevation and location of any exposed or underground utilities, roads and existing or proposed bridges and culverts, junctions of tributaries; etc.

It may be desirable to plot separate profiles for use in considering existing and planned conditions if it is expected that existing alignment will be changed significantly.

Cross sections. - - Cross sections should show any existing channels; the intersection with the base line; the elevation of the design flood for existing conditions; the elevation of any exposed or underground utilities; and any existing or proposed bridges.

Preparation of Preliminary Drawings

Preliminary drawings may include plan, profile and cross sections of all channel work proposed for the project. In the case of large complex channel systems, it may be expedient to show only representative channels in this degree of detail in order to reduce the bulk of the preliminary document.

In either case the following preliminary design data should be shown in addition to the survey data collected as shown above:

1. Proposed channel alignment with bearings or azimuth should be shown for each tangent. Curve data is usually not needed at this stage or development.

2. Proposed grade and water surface profile should be shown on the profile including pertinent elevations and invert gradient.

3. Typical cross sections showing existing and proposed sections one for each type or size of proposed section should be included on each sheet of plan-profile.

4. If structures are proposed for the project, a drawing of a typical structure showing general dimensions should be included.

5. Drawings, sketches, cross sections or photo montages should be included to illustrate how the proposed project will appear in the surrounding landscape.

Profile and cross sections may be omitted in reaches to be cleared and snagged only.

A horizontal scale of 1 inch = 200 ft. to 1 inch = 1000 ft. for plan-profile is usually adequate for preliminary drawings. If the original plot of survey data can be made to the same scale as that needed for drawings, considerable time and effort will be saved.

Documentation

All data developed during the planning phase should be properly documented and filed. This will insure that such data will be of maximum use for future reference. All notes, computations, and drawings should be properly identified and be complete so that they can be used in preparing detailed plans for design and construction. The data should be dated, since Service criteria are modified from time to time. All papers should show the initials of the person preparing the data sheets or making the computations. A reference should be made to indicate the methods or source of data used.

Survey for Final Design

General Requirements

The section covering "Design Surveys - Surface Drainage" in the National Engineering Handbook Section 16, 2/ Chapter 2, will serve as a useful reference for final design surveys, where applicable. As indicated in the NEH, a staked base line is required. An exception to this requirement may be made where clearing and snagging is the only work needed to provide the desired level of protection. For this condition, no additional surveys over those recommended for planning may be needed for design purposes. However, sufficient data must be obtained to permit the determination that this treatment will result in a stable channel which will meet the desired project objectives.

Field Check of Design Layout

For most projects it is highly desirable to check the final design on the ground. Verify the following:

1. Alignment and stationing.

2. Rights-of-way boundaries.

3. Construction easement limits.

4. Structure locations.

5. Stationing and description of bench marks.

6. Utility locations.

7. Apparent ownership boundaries.

8. Woody vegetation locations, including individual large and/or unique trees.

9. Home and institutional locations when potentially visible from the project site.

Land Rights Work Maps

The land rights work map may be prepared by making a full scale or photographically reduced overlay of the appropriate portions of the strip map. Judgment must be exercised to insure that adequate land rights are provided for the channel, spoil bank area, visual design areas and the maintenance and construction operations.

Except where specified otherwise in existing state laws, the land rights work map should show the existing channel, utilities, apparent ownership boundaries, roads, base lines showing key stations, and proposed works of improvement. If requested by the sponsors or required by state law the apparent acreage of right-of-way for each landowner should be indicated on the land rights work map.

A typical cross section should be shown on the land rights work map to illustrate the various elements of the proposed works of improvement within the right-of-way area. When utilities are involved, the actual cross section including the elevation of the design flow should be shown at the intersection with the utility.

Supplementing Planning Surveys

Field survey data obtained in the planning phase must be supplemented to provide basic design data. In fact, budgetary considerations will often preclude much of the detail described above for the planning phase so that the survey data used in preparing preliminary drawings may be of little value as design data.

Supplemental data needed to establish the visual resource design may be determined by the outlines shown in Chapter 2, Appendix A.

Bench levels must be checked and supplemented. Damaged BM's should be reestablished and additional bench marks should be set and tied in as needed.

Coordinate control should be established by traverse and closed within third order accuracy. Triangulation may be used to establish control on large, complex projects, if desired.

Topographic strip maps of the area will be needed for layout and design. Data for these maps may be secured either by plane-table or transit topography tied to the traverse. Strip map data must be secured to provide accurate location of contours, fences, buildings, bridges, culverts, roads, utilities, the edges of woody vegetation areas, large trees, orchards, existing drainage or irrigation ditches and structures, and all other physical or man-made features within the probable limits of work.

Test pits and borings should be accurately located and the elevation should be established on a reference hub adjacent to the site.

Preparation of Strip Maps and Profile

Strip maps should be plotted to the same scale as that intended for final drawings - usually 1 inch = 40 ft. to 1 inch = 200 ft., depending on the detail needed. For convenience, strip maps should be plotted on transparent paper or cloth sheets not exceeding 42 inches in width and 20 ft. in length. This provides sufficient length for proper selection of long tangents and curves for the "paper location."

Trial locations complete with stationing and curve data are then superimposed on prints of the strip maps and profiles are plotted on equally long profile sheets to permit selection of long tangent grades.

As the hydraulic design progresses and design cross sections are selected one of the alternate locations will usually emerge as apparently superior. The best apparent alternate including right-of-way requirements should then be drawn on the strip maps. Design grades and cross sections should be drawn on the strip profile. Prints should then be made of both the maps and profiles for field checking, prior to tracing final drawings on standard sheets.

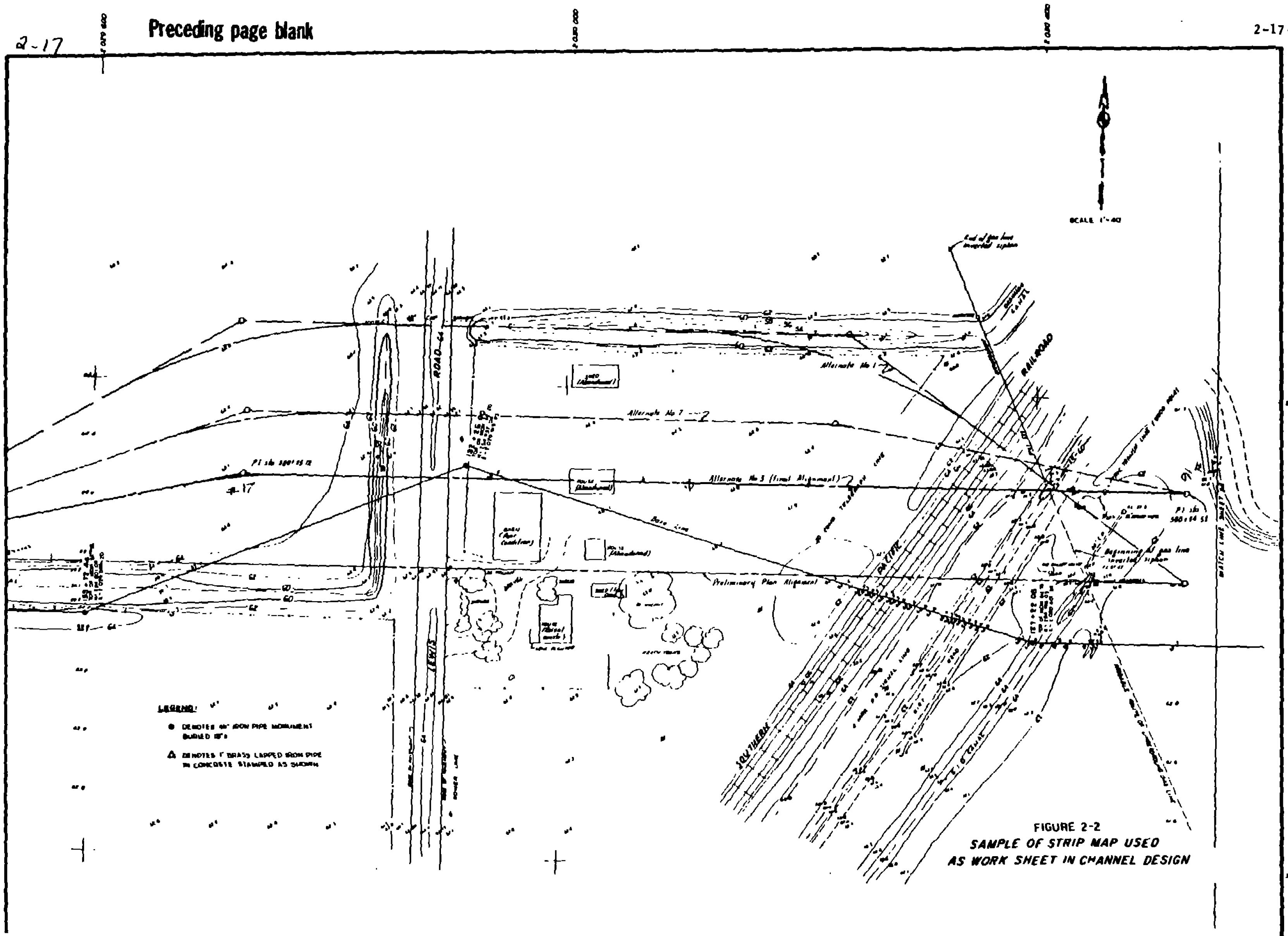

FIGURE 2-2
SAMPLE OF STRIP MAP USED AS WORK SHEET IN CHANNEL DESIGN

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FIGURE 2-3.

SAMPLE FROM STRIP PROFILE
(DESIGN WORK SHEET)

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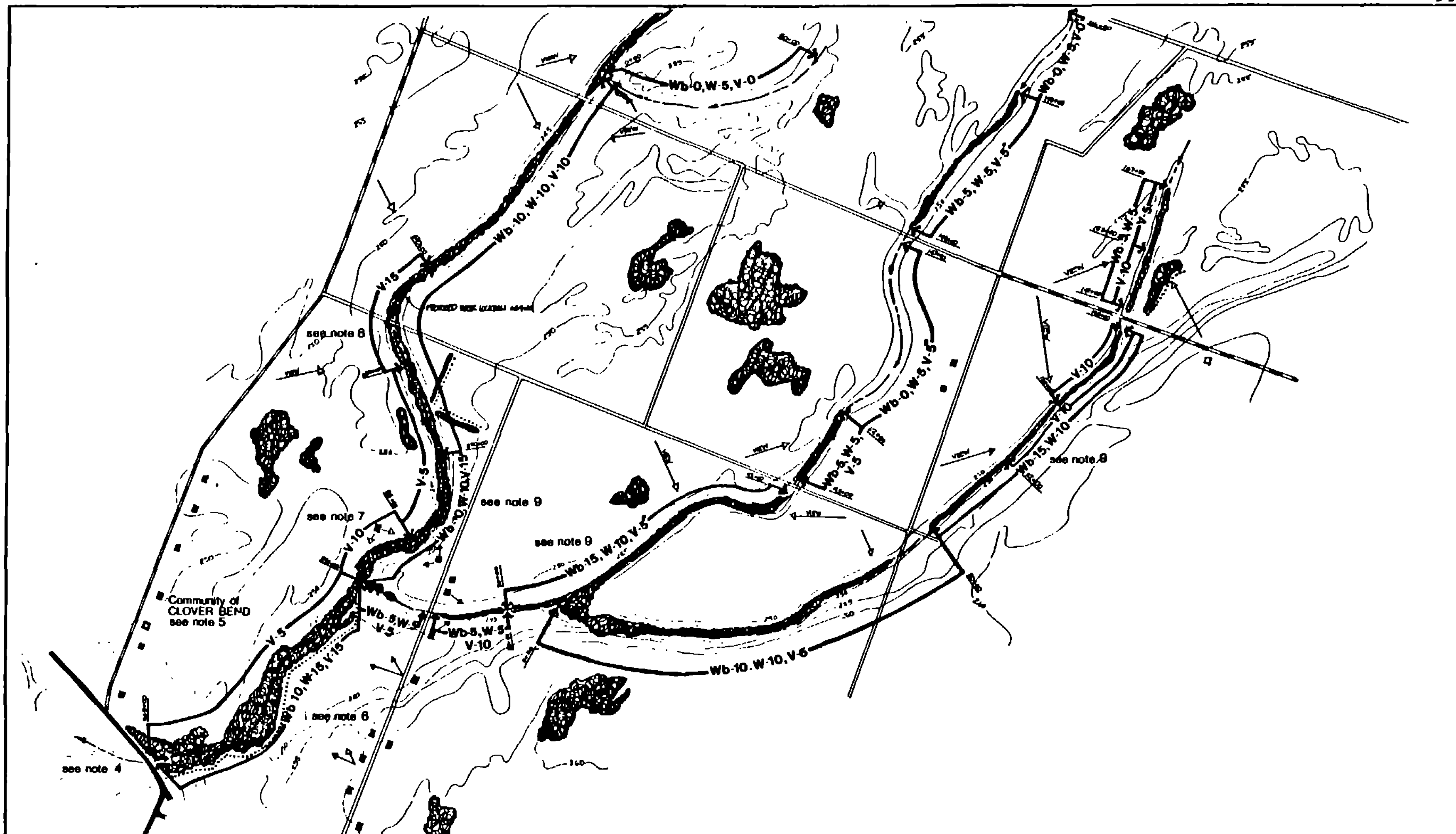


NOTES

1 The purpose of this sheet is to identify graphically the resource value of landscape elements occurring within or adjacent to the R/W's of channels proposed for modification under this contract.

2 Three major resources were studied. (1) windbrake resource, (2) wildlife habitat resources, and (3) visual resources.

3 The relative value of each resource can be identified at any point on the channel to be modified. (See legend for identification of symbols.)

4 State highway #228 is elevated above the channelway approx. 20 ft so that observers will be looking down on channel. (Time of viewing @ 50 mph is 5 sec.) Channel alignment is such that lengthy down channel views will be created. Selective trees could be left within the clearing limit to break up sight line down channelway. Spoil placement and shaping should be performed to simulate more natural land form characteristics.

5 The channelway, including woodland edges is not significantly viewable from paved county roadway or residences west of proposed channel modification. Thus the low visual value rating from station 545+00 to 599+00.

6 The channelway (particularly the woodland edge) is in close proximity to a gravelled county roadway and a number of residences. Sight lines are primarily perpendicular into woodland edge. Visual quality is high because of the wide section of woodland area (200-800 ft.). Channel modification will be screened from view and visual quality would not be altered. Clearing should be limited to the minimum needed for excavation and spoil spreading on both sides.

7 Visual value of woodland edge rated proximity moderate because of residence. Need to retain tree line to screen channel work.

8 Woodland edge highly viewable from station 650-00 to 670-00, thus visual rating of 15. Should attempt to maintain tree line edge on one side.

9 On reaches of the channel where any resource value is rated a 15 i.e., Wb-15, W-10, V-5, construction should be limited to one side for retention of woods on opposite side. Retention of woodland edge for one purpose or another would normally benefit other resources.

KEY

- State hwy. moderate to heavy duty
- Paved county rd. moderate to low duty
- Gravelled county rd. low duty
- Contour line (5 ft. interval)
- Residence
- Woodline
- Screening effect of woods outside channel R/W
- Sight lines of significance (VIEW)
- Limits of value ratings

- **Wb** Windbrake
- **W** Wildlife habitat
- **V** Visual resource
- **V*** Asteric denotes visual value rating as viewed from either side of channel

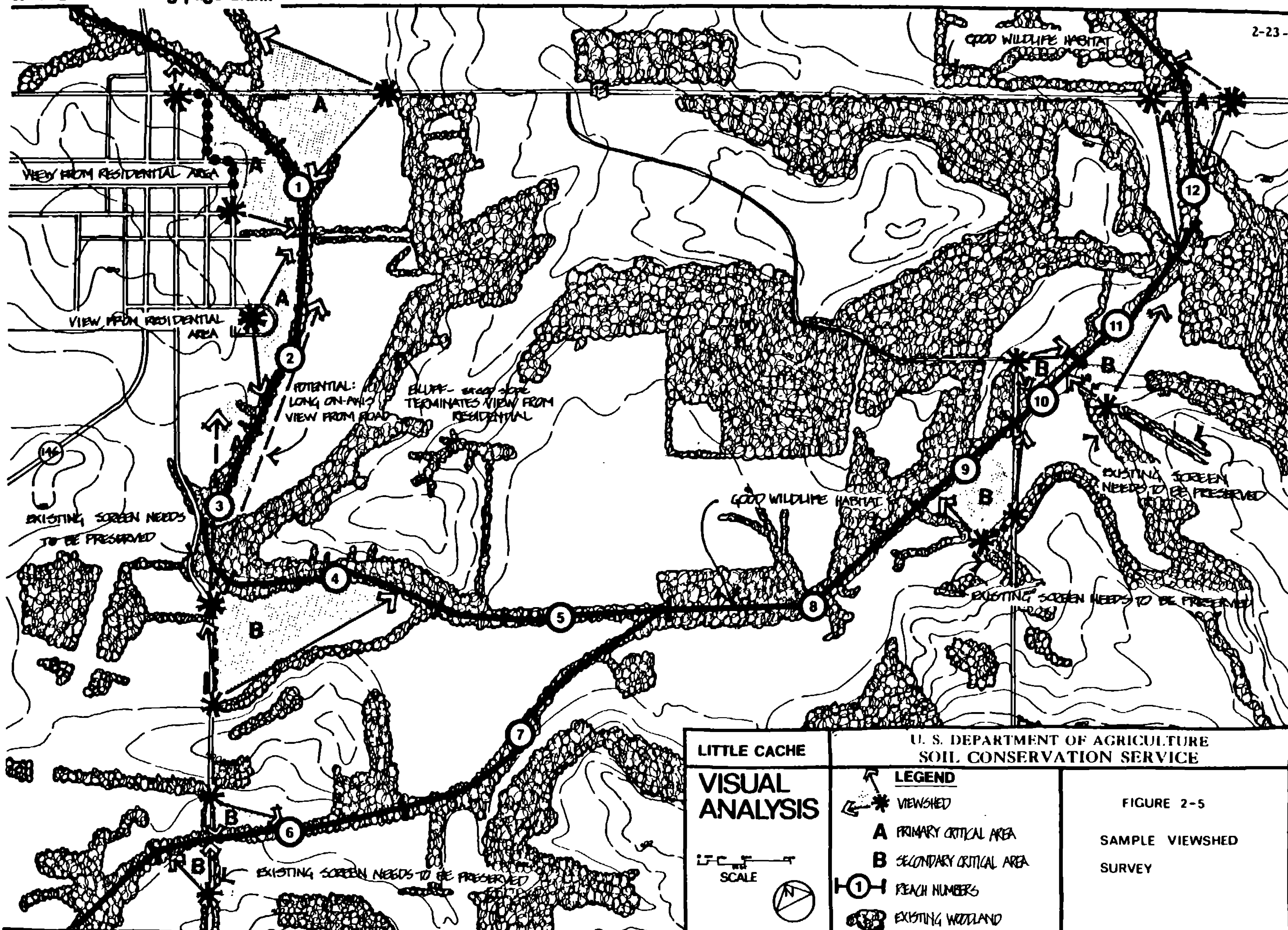
RATING:

- **15** High value
- **10** Moderate value
- **5** Insignificant
- **0** Low value

FIGURE 2-4

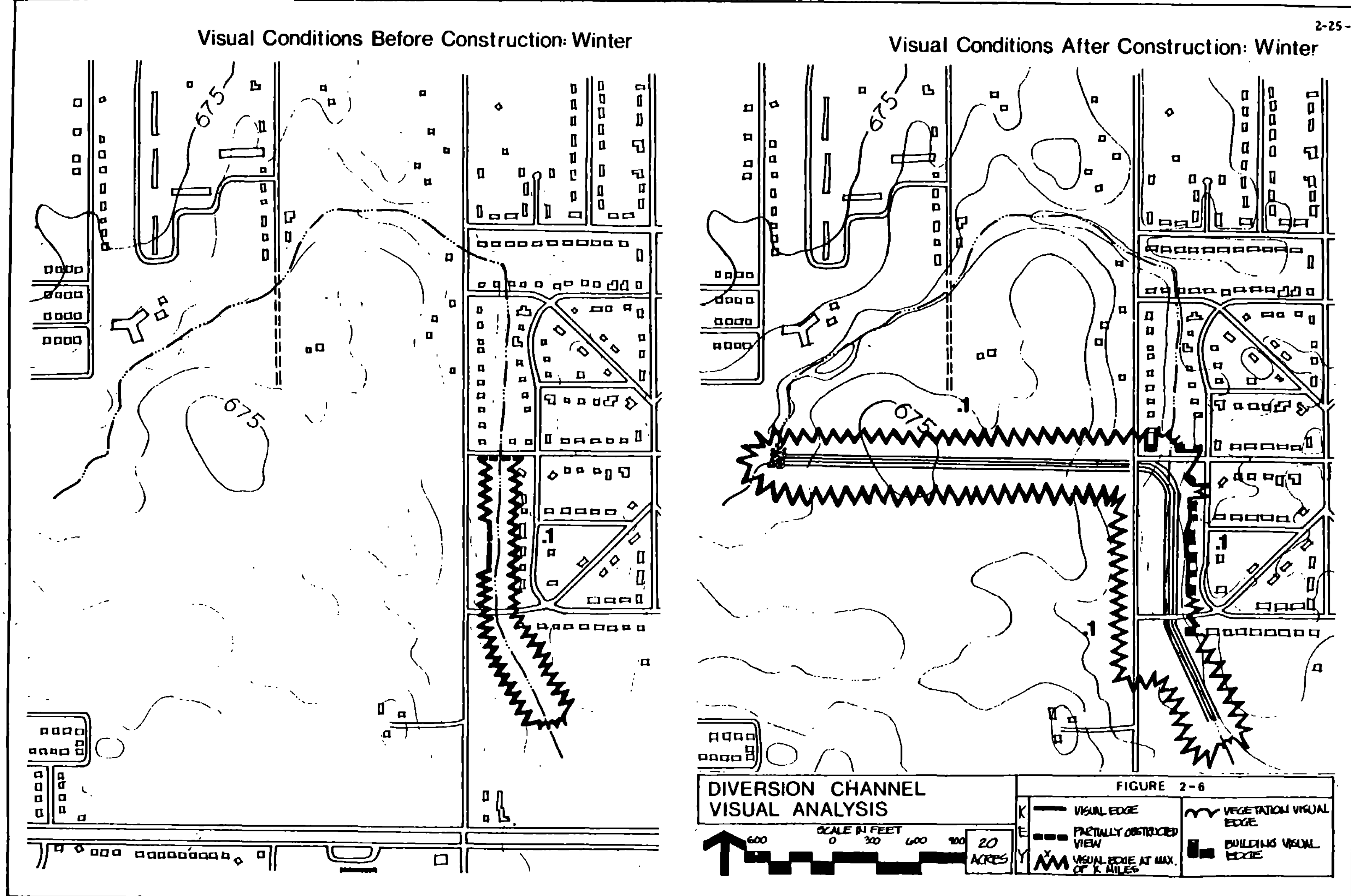
SAMPLE LANDSCAPE VALUE SURVEY

U.S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE


2-23-a
GOOD WILDLIFE HABITAT
A
VIEW FROM RESIDENTIAL AREA
A
VIEW FROM RESIDENTIAL AREA
POTENTIAL: LONG ON-AXIS VIEW FROM ROAD
BLUFF- STEEP SLOPE TERMINATES VIEW FROM RESIDENTIAL
146
EXISTING SCREEN NEEDS TO BE PRESERVED
B
GOOD WILDLIFE HABITAT
EXISTING SCREEN NEEDS TO BE PRESERVED
EXISTING SCREEN NEEDS TO BE PRESERVED
B
B
B
B
EXISTING SCREEN NEEDS TO BE PRESERVED
1
2
3
4
5
6
7
8
9
10
11
12
LITTLE CACHE
VISUAL ANALYSIS
SCALE
N
U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
LEGEND
VIEWSHED
A PRIMARY CRITICAL AREA
B SECONDARY CRITICAL AREA
REACH NUMBERS
EXISTING WOODLAND
FIGURE 2-5
SAMPLE VIEWSHED SURVEY

A

B


Visual Conditions Before Construction: Winter
675
675
Visual Conditions After Construction: Winter
675
675
.1
.1
.1
DIVERSION CHANNEL
VISUAL ANALYSIS
FIGURE 2-6
KEY
VISUAL EDGE
PARTIALLY OBSTRUCTED VIEW
VISUAL EDGE AT MAX. OF X MILES
VEGETATION VISUAL EDGE
BUILDING VISUAL EDGE
SCALE IN FEET
600 0 300 600 900
20 ACRES

A

B

CHAPTER 2. APPENDIX A.

LANDSCAPE ARCHITECTURE SITE SURVEY AND ANALYSIS

INTRODUCTION

The goals of landscape architecture are to identify, analyze, plan and design so as to retain, replace, or improve as many landscape resource values as possible. Material in this Appendix will cover the identification and analysis procedures. Planning and design will be discussed in Chapter 8. Material presented in this technical release assumes prior landscape architectural inputs have occurred in the development of project alternatives. Some of the material discussed herein could be used in the initial project assessment phase.

Meeting the goals of landscape architecture in channel work does not always mean a channel should be built to appear as it did before the project or necessarily to appear as a "natural landscape." The project site appearance can change drastically, as long as the landscape resource values have been retained, replaced, or improved. To achieve these goals the following four step procedure should be followed:

1. Landscape Architecture Site Survey to identify and map the landscape use and visual resource values* and to modify these values as indicated by project visibility.

2. Landscape Architecture Site Analysis to identify opportunities and problems and to define objectives so as to utilize opportunities and overcome problems.

3. Planning to identify alternates that achieve the objectives. In this phase the landscape architectural objectives merge with other engineering objectives to find the best possible alternatives.

4. Design to detail the chosen alternative so as to achieve the best structural and environmental solution.

LANDSCAPE ARCHITECTURE SITE SURVEY

Any landscape architecture (LA) site survey includes two basic factors the landscape (its physical characteristics) and people (how they see and use the landscape.) Judgment should be used to determine what data are needed. Obviously the scope and detail of the data will vary depending upon the size and location of the channel. Both the explict value of the landscape and some sense of the landscape's implicit value as perceived by its users and viewers must be considered in channel

* Note: definition may be found in Glossary at the end of Appendix A.

work. For example, a channel may have a straight alignment and may be a previously dug ditch; but it may be perceived as valuable because it is used by the public as an open space. A LA site survey for channel work should cover the following factors:

1. Landscape Use Value
2. Visual Resource
3. Project Visibility

1. Landscape Use Value

The LA site survey should note how the existing project area is used by the public. In most cases, channel work can be designed so the landscape will retain its utility for human use, while improving its hydrologic characteristics. Data to be noted and/or mapped for various reaches include but are not limited to:

A. Identifying places where existing streamway vegetation function as a:

(1) Shelterbelt to help control wind erosion.

(2) Privacy screen between homes.

(3) Buffer between incompatible land uses such as industrial and residential areas.

(4) Noise barrier such as between homes and a busy highway.

(5) Safety barrier controlling pedestrian traffic such as between schools and roads.

(6) Climate shelterbelt such as a wind/sun screen providing energy conservation to homes and institutions.

B. Identifying areas along the streamway that function as or contain:

(1) Pedestrian paths between homes, schools and commercial areas.

(2) Recreation areas contiguous to existing playgrounds, schools and parks.

(3) An open space or environmental corridor within a developed area.

2. Visual Resource (VR) Value

Investigating the visual resource values should not involve a personal judgment as to beauty. For example, a previously dug ditch might be judged as "ugly" and yet have high VR value because it provides variety in an otherwise monotonous landscape. A concrete drop structure may be judged "ugly" and yet have high VR value because it relates architecturally to the surrounding landscape. Visual resource value involves an assessment of several measurable factors. The factors cited in Rating Charts A-C below were developed from research and preference studies cited at the end of this Appendix. These charts must be used within a local frame of reference and must be verified by citizen participation. For example, water carrying sediment may appear relatively muddy or clear depending on the appearance of other streams within the area.

The Rating Charts A-C below rank VR values into only two groups, high or low. One or more middle groups do exist between high and low. These charts should be expanded into at least three groups and the criteria shifted to fit the locale. For example, crystal clear water (Chart B, item 1) may not exist in a locale. In that case, the clearest water for the locale should be rated as a high value visual resource and all other water clarity rated in comparison to it. Investigation of VR values include, but is not limited, to identifying and/or mapping the existing landscape as to the following.

RATING CHART A: Visual Resources of Streamway and Surrounding Landscape

Rural Areas

High Value Visual Resource	Low Value Visual Resource
(1) Adjacent landscape has no linear patterns, no agricultural activity, few or no manmade structures.	(1) Adjacent landscape contains many linear forms and patterns, (other channels, transmission lines, field patterns).
(2) Streamway has the major trees in the landscape providing the only variety and spatial definition to surrounding landscape.	(2) Streamway provides no spatial definition or variety to surrounding landscape.
(3) Surrounding landscape has no visual detractors (strip mining, derelect lands, erosion, etc.) and/or is highly maintained with obvious care for its appearance.	(3) Surrounding landscape has obvious visual detractors.

Urban Areas

(1) Streamway area is highly maintained (lawn, shrubs and trees) and/or is a visual part of residential lots, school grounds and parks.	(1) Streamway area is not a visual part of any maintained area.
(2) Adjacent landscape is highly maintained with obvious care for its appearance (lawns, residential gardens, industrial parks).	(2) Adjacent landscape is derelict, poorly maintained with no obvious concern of its appearance.
(3) Large, mature and/or flowering trees along streamway.	(3) No significant woody plants along streamways.

RATING CHART B: Visual Characteristics of the Water

High Value Visual Resource	Low Value Visual Resource
(1) Clear, generally transparent water.	(1) Visible color or turbidity, and/or pollution and/or algae.
(2) Desirable drift material such as leaves.	(2) Floating debris and trash.
(3) Variety of bottom material (bedrock boulders, gravel or unique bottom material for area).	(3) Homogenous bottom material commonly found in all area streamways.
(4) Conspicuous water movement, rapids, riffles or slow movement with high reflecting ability.	(4) No open water, weed choked or imperceptible water movement with no reflecting ability.
(5) Variety of movement from fast reaches to still pools.	(5) Homogenous movement with no visible variety.

RATING CHART C: Visual Resources within the Streamway

High Value Visual Resource	Low Value Visual Resource
(1) Streamway is obvious, meandering visual edge in landscape.	(1) Previously dug straight ditch.
(2) Variety of streambank slopes.	(2) Uniform streambank slopes.
(3) Variety in size and shape of cross section.	(3) Uniform cross section.
(4) Unique streamway vegetation compared to surrounding landscape (cypress, beech, birch).	(4) Vegetation in streamway is weedy or commonly seen in surrounding lands scape.

3. Project Visibility

Investigate the types of viewers and their significant viewsheds* to determine how visible the project will be within a locale. Determining the public's opportunities to see the project will modify the use and visual resource values determined previously. The charts below are guides to identifying project visibility and evaluating its significance. The guides should be modified to fit a local context. For example, viewers who may see the project one time only may or may not need to be considered depending on the local frame of reference. Visibility factors to be noted and/or mapped include but are not limited to:

CHART D: Viewers

	Critical	Important	Normal
1. Purpose of seeing or being on project site	·Homeowner, including farmer ·Recreationist ·Tourist ·Local resident	·Resident of region ·Adjacent businessman ·Students at school	·Farmer at work
2. Frequency of view	·Daily $>$1 min	·Daily $<$1 min ·Infrequent but for longer time periods $>$1 hour	·Infrequently for short time periods
3. Speed of viewer	Slow such as ·Pedestrian ·Bicyclist ·Canoe or slow moving boat	·Moderate 15-30 MPH	·Fast 30 MPH

CHART E: Viewshed and Viewpoints

		Critical	Important	Normal
1.	Viewpoint location relative to project	·Elevated as from bridge, road, or two story bldg. (> 20')	·Elevated <20'	· Ground level
		·Scenic hwy or overlook	·Interstate, state or busy country roads	· County or farm road
		·From residential, institutional or recreational areas	·From isolated home or farmstead	· From commercial areas
2.	Location of project within viewshed	·Foreground* only ·Foreground to background* or a long vista on channel	·Middleground* only ·Middleground and background	· Background only

See Figures 2-5 and 2-6 for examples of viewshed maps.

LANDSCAPE ARCHITECTURE SITE ANALYSIS

The intensity of a site analysis will vary between projects but the following steps remain the same:

1. Locate opportunity and problem sites in the project area.

2. List in priority the landscape architecture objectives for both problem and opportunity areas.

3. Document the analysis in a supporting data file.

1. Opportunity and Problem Sites

Opportunity sites are defined as areas where the channel design may retain or rehabilitate existing landscape values. Problem sites are defined as areas where the channel design may reduce or eliminate existing landscape values. Opportunities and problems are located by superimposing landscape value areas (use and visual) and project visibility data. For example, an area of low visual resource value, moderate project visibility and high use may be an opportunity site; whereas an area of high visual resource value, high use, and moderate project visibility may be a problem area. The relative importance of the visual resource value, use value and visibility within the decision process can shift depending on the project. For example, retaining the use value of the landscape may be most important in some locations; while in other areas retaining the visual resource values may come first.

2. Landscape Architecture, (LA), Objectives

The LA objectives should be the logical outcome of the LA analysis, and a LA design, (Chapter 8) should be the logical product of the objectives. The success of the final design will depend on how well the site analysis has been done and how clearly the landscape architectural objectives have been stated. These objectives should be prioritized and stated as objectives, not design solutions. For example, an objective might be "to retain the climate shelterbelt value of urban trees." The various options for achieving this might include; moving the channel alignment, changing the side slopes to retain trees, selective clearing to retain the most functional trees or replanting with mature trees. The decision to choose among these design options should be correlated with other engineering factors in final design.

3. Documentation

Documentation of the site survey, analysis and objectives can be done on several formats: reports, maps with notes, photo record with notes, computer graphics or a combination of any of these. Often the analysis and documentation can be done in a single process by mapping with notes (See Figure 2-4). A photographic record of use and visual resource value areas and critical viewsheds should be done for all channels in urban areas or areas of projected future urban development.

GLOSSARY

1. BACKGROUND is the viewshed zone most distant from the viewer. Details aré not seen in this zone. The horizon line is prominent in this zone as are general form, colors, and textures.

2. FOREGROUND is the viewshed zone nearest the viewer. It is the zone in which details, such as construction joints, movement of water and the finish of earth grading, are visible.

3. MIDDLEGROUND is the viewshed zone between the back and foreground. Some details can be seen in this zone but only those which are in sharp contrast visually.

4. VIEWSHED is the zone of view or volume in a given direction as seen from a specific viewpoint.

5. VISUAL RESOURCE is the appearance of the landscape as described by the measurable visual elements; topography, vegetation, water, and human structures and patterns, and by the measurable patterns of interaction among these elements.

6. VISUAL RESOURCE VALUE is the relative desirability of a visual resource unit as evaluated by rational criteria.

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CHAPTER 3. SITE INVESTIGATIONS

Introduction

Methods of investigation and testing of soil conditions along a channel system are discussed in this chapter. The purposes of these investigations and testing are to evaluate the resistance of the soils in the bed and banks of the channel to erosion forces, to evaluate the sediment transport relationships, to determine slope stability against sloughing and sliding, to estimate earth loads that may act on structural members, and to determine the rate of water movement through the soils.

The procedures include identification, sampling, and testing or evaluation of stratigraphic units encountered in the channel system to be modified and an evaluation of the sediment transport characteristics of the system.

Investigation Requirements

There are conditions specific to investigations for channel improvements that cause them to differ from investigations at other types of construction sites. One usually significant difference is that channels may extend for many miles through a variety of materials. This fact demands that data obtained from any one test hole must be correlated on as knowledgeable a basis as possible with data obtained at the next test hole, both upstream and downstream. A distinguishing feature of investigations for channel improvements is that almost always they are located in alluvial material. The stability of channels is affected by watershed conditions upstream that control the availability of bed material, and by ground-water conditions at the site. A similarity with foundation investigations for other structure sites exists in that the in-place characteristics of the soil materials are very important if these characteristics that pertain to erodibility differ from those that exist when the material is disturbed.

The following sequence of work is needed to appropriately recognize the conditions specific to investigations for channel improvements while ensuring that an adequate amount and type of data are obtained.

1. Determine the geomorphology of the deposits in which the channel is to be located to the limit necessary for this purpose.

2. Identify stratigraphic units along the proposed channel route and to a depth of at least three feet below the invert or channel bottom elevation as proposed. A stratigraphic unit is defined in this instance as an identifiable stratum of alluvium or other soil material whose susceptibility to erosion is a reflection of the original materials, mode of accumulation and the changes that have occurred since deposition or soil formation.

3. Log soils within stratigraphic units and classify and sample for evaluation or determination of erodibility characteristics.

Geomorphology of Deposits

Significance of the Stratigraphic Unit

There are a number of factors which influence the erodibility or stability of soil materials. Erodibility and stability are explained in this context as responses of the soils to the environment of channel boundaries. Erosion results from the flow of water against the boundary as expressed by mean velocity, tractive force, or a combination of these parameters such as tractive power, the product of mean velocity and tractive force. Instability of stream banks often results from internal seepage forces. The soil's ability to resist erosion is affected by the kinds, amount, and character (dispersive or aggregated) of clay, the amount and size distribution of coarse particles, and the nature and amount of cementing agents. The term coherent is used to describe soils that resist erosion as a mass because of bonding action of clay, cementing agents or other causes. The climate and age of the deposit since its accumulation have a strong bearing on these erodibility characteristics. There is an almost infinite variation that can occur in each of these factors and in their influence on the others. Deposits change with geologic time. Each change produces an expression of the dominant factor and interfactor relationships persisting at that time. Thus, a combination of forces acting on a deposit produce a layer of material identifiable as a stratigraphic unit. Relative uniformity of location in an alluvial profile, uniformity of appearance, and erodibility are a result of a similarity in source of materials, the history of deposition, and the weathering characteristics associated with the areas. In time, with certain exceptions, weathering, consolidation, cementation, accumulation of humus or other influences increases the significance of structure on erodibility. By the same token the significance of local variations in texture is diminished.

The delineation of stratigraphic units simplifies the field investigations in that the units may extend for distances that can include the entire route of channels to be improved. Their thickness and depth below the surface may be relatively uniform and consistent, providing a means for correlation and representative sampling. At the other extreme, alluvial fan deposition can result in a heterogeneous mixture of sand, silt and gravel where correlation of individual beds may be not only impossible but not particularly useful.

Identification of Stratigraphic Units

The origin of the defined stratigraphic units as alluvium dictates that certain relationships existed between the deposits and the carrying capacity of the stream at the time. Further, the relationships that exist today are not necessarily the same as at the time the sediment was deposited. For example, during portions of the Pleistocene, runoff and sediment transport were on an appreciably larger scale than at present.

The primary identifying characteristics of a stratigraphic unit are: color, thickness, grain size distribution, texture, structure, plasticity, consistency, density, dispersive characteristics and cementation. Slight variations that may occur do not necessarily signify the presence of a new stratigraphic unit, particularly if the deposits are conformable with units above and below the one being described.

The location of stratigraphic units areally within the alluvial valley is as important as their position and identification within the vertical profile. Because most alluvial channels meander about within a relatively short geologic time period, it is probable that stratigraphic units in a valley confined by side slopes will be at about the same elevation across the valley. This requires field checking before assuming a uniform valley-wide distribution of stratigraphic units exists.

Geomorphic History as a Determinant of Erosion Resistance

The history of the stratigraphic unit from the moment of deposition determines its erosion resistance. The grain size distribution and texture of the deposit is initially the most important characteristic and it may remain so if the environment following deposition is not conducive to modifications tending to change the arrangement or bonding of particles. These modifications can be produced by one or a number of post depositional changes such as the movement downward of particles from the overlying deposits, by weathering, by consolidation or chemical action that increases or decreases the bond between particles.

The result of this geomorphic history is that the stratigraphic unit responds in characteristic manner to the hydraulic forces that may be exerted against its exposed profile. The unit may remain as individual particles free of any bond with adjacent ones, or it may resist as a mass due to coherence between particles. The bond between particles that may be reflective of a long history in a particular environment may be destroyed when the profile is disturbed.

The following is an example of the depositional and environmental influences that can affect the erosion resistance of a stratigraphic unit. A sequence of stratigraphic units may indicate that in this instance they are alluvial deposits consisting of (1) light colored, finely stratified noncoherent silt and fine sand about two feet thick. Below this unit there is (2) a compact dark gray clayey silt horizon several feet in thickness. Then there is (3) a layer of gravel in a matrix of compact brown silty clay that starts a foot above the stream bed in the exposed banks and extends below the streambed and the proposed invert grade to unknown depths.

The reasons for a uniqueness in character of stratigraphic units within a profile are several fold. Taking the profile cited above as an example, the number 1 stratigraphic unit of this hypothetical profile is an accumulation of recent sediment. Soil forming processes have not had time to modify the sediment's characteristics over that contributed by the texture and grain size distribution of the transported sediment and its mode of deposition. The bulk of unit 2 accumulated either as a mixture of, or dominantly of, lateral accumulation as the stream meandered or vertical accumulation during overbank flooding. The developed soil in this unit indicates a following period relatively free of deposition or erosion. Time was available for limited weathering of the sediment, and for humus to accumulate. The gravel layer identified as unit 3 was deposited at a time of relatively high velocity flow. The high discharges that brought down the gravel could have formed a braided wash that extended from one side of the valley to the other. During flows of lower velocities fine sediment was deposited and migrated into the gravels. With age and weathering of clay forming materials this stratigraphic unit, with its characteristic erosion resistance properties, evolved.

The sediment source, texture, and mode of deposition of units 1 and 2 could be very similar, yet their appearance including color, degree of consolidation, and erosion characteristics can be quite different. These differences created by changes occurring since deposition can vary widely depending upon the interaction of the average annual climatic environment: humid or arid, cold or hot. The climate following deposition can create a wide change in erosion characteristics for different reasons. In a humid hot climate, textural changes can occur by weathering of clay forming minerals, leaching of soluble salts, carbonates, etc. or by movement of fine clay-size particles from the upper to the lower profiles. Structural changes can be caused by alternate wetting and drying, and consequent swelling and shrinking, as well as by growth and decay of vegetation.

In an area affected by a permanently high water table, little or no post-depositional changes in the deposits may occur. This is especially the case when there is little oxygen in water in which the stratigraphic unit is submerged. The reason for this is that oxidation essential to the weathering process has been prevented or retarded. Reduction of iron, resulting in grayish, greenish, or bluish coloration, however, can occur. Thus sediment which may give the appearance of being recent in origin could have been deposited at about the same time as a nearby unit with a profile containing morphological evidence suggestive that it has been in place for a long period of time.

Accumulations of or cementation by silica or carbonates, or both, can change the structure and erosion resistance of the sediments without changing the engineering classification materially because the cementing material may not noticeably add to the finer soil constituents.

The junction with tributaries can change the uniformity of main stream stratigraphic units, depending upon whether the tributary has been or is a significant contributor of sediment. The type of sediment deposition may be so similar to the main valley deposits that interfingering or mixing results in an accumulation that is not separately distingishable. Tributary deposits could also be quite different either in age, color, texture or all three.

Figure 3-1 illustrates a geomorphic setting and related stratigraphic units that would be pertinent to proposed channel improvements on a stream and some of its tributaries. Unit 1 consists of recent main stream deposits and unit 2 of recent deposits from the identified tributaries. These deposits constitute a replacement of older alluvium that was eroded during a lowering of base level in times past. Unit 3 consists of old alluvium which has undergone aging and the development of a profile. Unit 4 is a still older buried profile and unit 5 is an old stream terrace remnant.

Discontinuities in Alluvial Stratigraphic Units

Discontinuities are here defined as breaks in stratigraphic units that were originally continuous. Such breaks, narrow or wide, can have a bearing on the stability of an improved channel since the deposit may be replaced by more or less erodible material. Discontinuities can occur in a number of ways. One example is illustrated by Figure 3-1 where fine grained coherent soils developed at a higher base level than exists today. A lowering of this base level, created by a lowering of the nearby sea level elevation, caused downcutting of the main stream and then its tributaries. A subsequent raise in base level induced replacement by more recent sediments that are erodible.

Discontinuities similar to the above described type can occur without changes in base level. For example, a tributary producing a different type of sediment than the main stream can determine the characteristics of the deposit on the main floodplain in the vicinity of that tributary.

Occasionally the character of certain stratigraphic units differs widely from the type normally expected for a particular environment. For example, the occurrence of a very dark, highly coherent unit in a semiarid environment near the sea coast. In known instances where these disparities occurred, this coherent unit was interpreted to have accumulated during a rise in sea level. Fine sediment deposition in an embayment followed by the growth of marsh vegetation was probably the origin of this stratigraphic unit.

Another example of seeming disparity is the presence of an old fan of gravelly deposits encroaching on a valley but lacking a watershed of a size and geology capable of furnishing these materials. Inspection of geologic maps indicates that the gravelly fan deposits accumulated prior to stream piracy which transferred most of the watershed and also the source of coarse sediment to another drainage system.

Stratigraphic Units Without Internal Continuity

In certain geomorphic environments, the sediment deposited varies widely in texture within a short distance. If there is any unit identification, it is to the effect that there is a range in particle sizes from silt to gravel, all noncoherent, except for possibly some lens cementation. Such deposits are extensive in arid, semiarid and glaciated regions where in a network of ill-defined channels, the deposits were reworked during flood flows. Variability on a smaller scale may occur in humid climates where deposits accumulated during braided flows, but weathering and other factors contributing to development of coherent soil may be inhibited by such environmental factors as a high water table. In arid or semiarid climates, caliche by cementation can provide a resistant stratigraphic unit in an otherwise noncoherent alluvial deposit.

In the instances cited above, the absence of continuity, as well as the lack of coherence, should be established in the investigations stage preliminary to sampling. Then procedures to establish continuity can be modified.

Conducting the Site Investigation

Data Requirements and Observations Preliminary to Site Investigations

A longitudinal profile of the channel reaches to be improved, with survey stationing and profiles of the existing and proposed channel invert and top of bank is needed. A profile of the top of one bank is usually sufficient in alluvial valleys where the flood-plain surface is about the same on both sides of the channel. In addition, a flood-plain map is needed that shows the existing channel, the proposed channel, and the survey base line. The location of test sites and other pertinent geologic features are to be shown on the map.

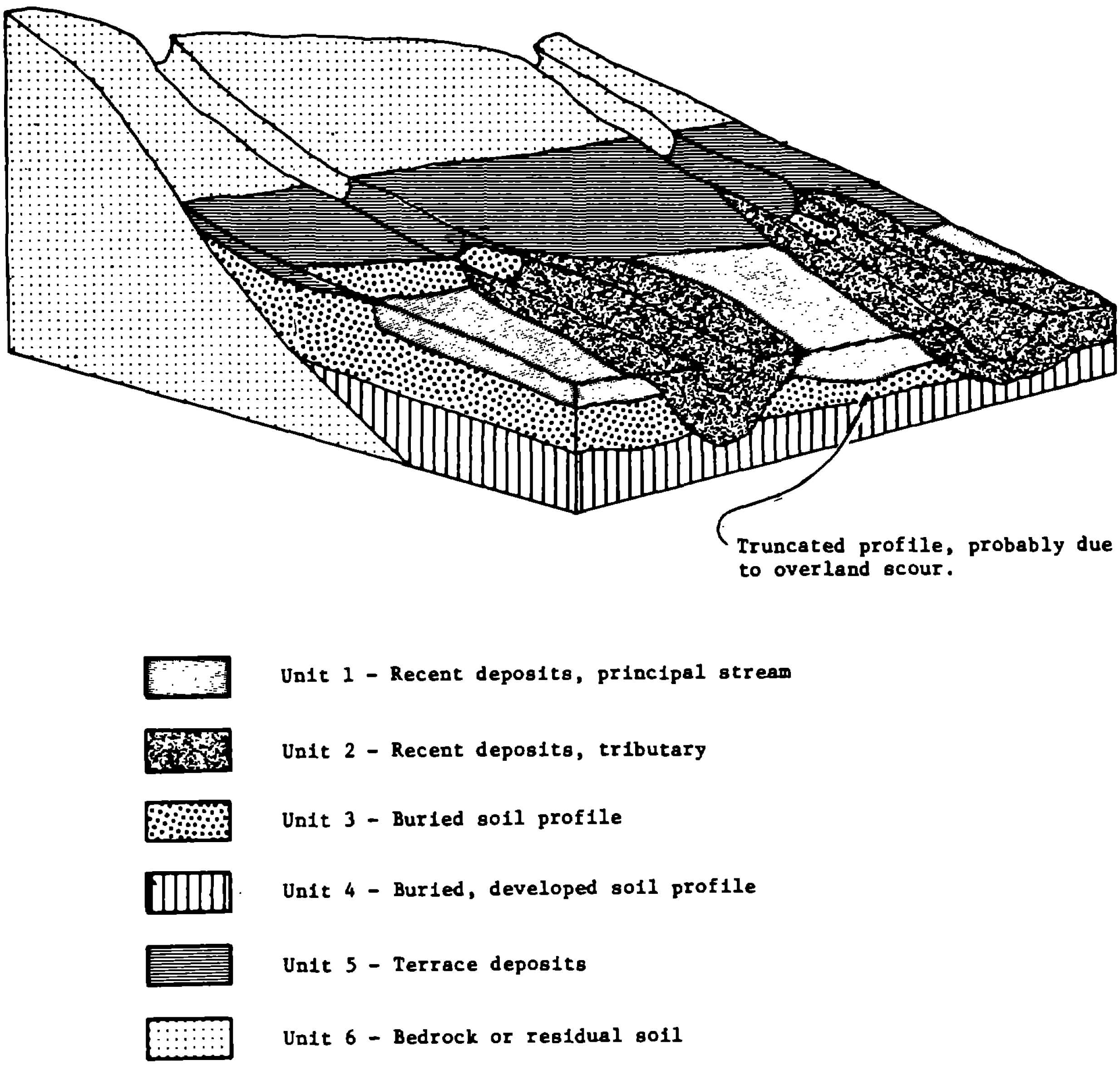

Figure 3-1 - STRATIGRAPHIC UNITS RELATED TO CHANNEL IMPROVEMENTS ALONG A STREAM AND ITS TRIBUTARIES (SIMPLIFIED)

Guidance on the erodibility of soils in a proposed channel improvement may be obtained by observation of existing channel performance. However, there should be a good general understanding of how flows will differ after project installation from what is characteristic under present conditions, that is, the change in grade, if any, the relative change in peak flows for frequent as well as infrequent events, and the relative change in duration of flows as well as frequency of bankfull flows. Observations on present stability are relevant if minor changes in flow characteristics are to be made. Such observations are substantially less relevant if major alterations in channelized flow are to result. These alterations could consist of on-site changes such as straightening and enlargement, or off-site changes such as upstream reservoirs that reduce the peak but substantially increase the duration of flows. For one example, an existing meandering channel is so small that overbank flooding may occur annually or more frequently. A proposed realigned and enlarged channel to be built to carry the ten percent event will cause a substantial increase in hydraulic stresses. The fact that existing stream banks are stable is not in itself supporting evidence that bank stability can be expected following construction. Then too, a major change in the rate of bed material load transport, if such load is present, can occur as a result of increased channel flow. While a general knowledge of changes in flow characteristics with project installed will suffice early in the field investigations, specific information will be needed in evaluating the erosion resistance of soils following sampling and laboratory analysis.

Bank sloughing or sliding may be a problem independent of changing discharge characteristics and needs to be identified. Fine sands, silts and some clays are susceptible to sloughing or slips under a range of moisture conditions. Sliding or sloughing may result from a rapid decline in water stage leaving a steep seepage gradient within the bank. The bank may slide or slough because of being undermined by erosion at the toe or by excavation below the water table. Underground flow from tributaries or springs, excessive rainfall, or over-irrigation can provide a source of ground-water flow which may result in bank sliding or sloughing. The location of any seeps or springs with reference to such problems should be identified.

Reconnaissance of Site for Channel Improvements

The initial field studies phase of site investigations should include inspection of the soils to and below the proposed channel invert level to interpret the geomorphology and to identify stratigraphic units. It is of primary importance to determine what stratigraphic units exist and their relationship one to another. This knowledge should be used to determine whether the same units occur at other randomly picked locations

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based on their appearance and lithology. Decisions on sampling sites, units to be sampled, and types of samples to be obtained must await their approximate delineation through the reach to be improved. Soil surveys along a proposed channel alignment can be useful in organizing the investigation and identifying stratigraphic units. It should be borne in mind that soil survey information is limited to a depth of about 5 feet and is frequently not reflective of stratigrahpic units at greater depth.

The inspection of the alluvial soil profile may be achieved by one or a combination of several means. Exposures along the banks of streams offer a frequently available means of study. This has the advantage of enabling observation of in-place characteristics as well as relating these characteristics to erosion from stream flow or bank stability from seepage forces. Bank stability may be due to protection by vegetation, or an irregular alignment which increases the roughness and thereby reduces the velocity or other reasons.

From the standpoint of bank stability from seepage forces, the existing channel could have sufficient vegetation, a protecting coating of fine sediment, or a ground-water level which has adapted to the channel depth over time. New excavation could cause problems which do not presently exist. Information gained from inspection of stream banks and bottom must be supplemented by borings or test pits when exposures along the banks do not provide sufficient information on the in-place characteristics of the deposits.

Exposed banks are often disturbed, desiccated or cracked and exhibit a different set of properties than the protected materials along the proposed cut and gradelines.

The place of observation along the channel is identified on the profile. Observation points in addition to those along the channel alignment will usually be required in order to interpret the stratigraphy and continuity of the valley deposits. The tentative upper and lower surfaces of stratigraphic units are determined and recorded on the profile. Each unit is to be numbered and geomorphically classified. For example, unit 1 is a recent sediment deposit overlying unit 2, a buried soil profile. Field notes and logs should describe the appearance and condition, including color, grain size, texture, structure, moisture, etc. of each unit in accordance with ASTM D-2488. Geomorphic terms are to be used in correlating between observation points. Identification as a stratigraphic unit is required in addition to the specific characteristics that can be described.

The tentative position of stratigraphic units within the profile and their extent along the channel alignment should form the basis for selecting soils investigation sites. The unit delineation must be informative enough to indicate the location of sites required to provide adequate data in the detailed investigation.

Determining the Intensity of Investigations

Four factors control the intensity of needed site investigations: (1) design requirements that may be independent or partially independent of the soil materials in the study reach; (2) hazard to life or property in the event of failure or partial failure; (3) complexity of the geomorphology in the site area; and (4) possible effects on quality of environment.

Concerning design requirements, for example, restrictions in availability of rights-of-way and capacity for each earth channel may dictate that the channel be a lined one, irrespective of the kind of materials in the reach involved. In this instance the investigations need only obtain the information that may affect construction such as classification of materials to invert grade, location of the water table and of seeps or springs. The presence of rock outcrops, layers of hard caliche, etc. should be carefully identified by location and description.

Where the water table is above proposed invert grade, its elevation should be identified in the logs of test holes or pits. If the water table fluctuates from season to season so that in one part of the year its elevation is below invert grade, the average time period this occurs and the dates of occurrence should be estimated. A high water table may interfere substantially with construction or affect the stability of an earth channel after excavation. If drop structures or other installations requiring data on bearing strength are being considered, investigations should be made in accord with recommendations in NEH, Section 8, Engineering Geology.

Channels located in urban or intensively developed agricultural areas differ widely in damage potential if a failure occurs than do channels in rural areas. In an urban area, the channel is usually designed to retain the one percent event with even minor erosion generally not tolerable. In both instances, the erosion characteristics of the bed and bank materials must be identified by procedures which include stratigraphic unit delineation and representative sampling and testing. Stratigraphic units must be more thoroughly delineated and variations within the units more closely identified by sampling and testing in intensively developed areas than in agricultural areas.

Complexity in the geomorphology of the site has a strong bearing on all phases of the investigation. Contrasting levels of complexity may include on the one hand a geomorphology as simple as one having a single stratigraphic unit uninterrupted by discontinuities, or maybe thin beds of sand or gravel that lack continuity and can be treated as one highly variable unit, to another that consists of a number of overlapping units frequently broken by discontinuities or disappearances from one reach to the next. In all instances, including those with the most complex array of stratigraphic units, an understanding is needed of the sedimentation processes and geologic history facilitates correlation and delineation of units, locations of discontinuities, etc.

The presence of dispersive clay soils along proposed channel alignment will have an effect on the intensity of investigation.

Dispersive clay soils often occur in lenses, spots and discontinuous strata. In order to tentatively identify dispersion problems, it will be necessary to check each stratigraphic unit at each test hole site for dispersive clays by means of the field "crumb" test performed on material at natural moisture. If field tests indicate possible dispersion problems, sufficient holes and samples should be established to delineate the problem areas or depths.

Location of Sites for Sampling and Logging

The location of pit or drill hole sites is based on the tentative delineation of stratigraphic units. There is a two-fold purpose in selection of these sites. One purpose is to obtain representative samples, the other is to "prove out" the tentatively identified stratigraphic units shown on the profile. If the logs at one test site fail to correlate with those at the site next upstream or downstream, then another site should be chosen between them, at least for the purpose of correcting the profile if not for additional sampling.

For long reaches where continuity of stratigraphic units has been established, it is best to sample widely from any one stratigraphic unit rather than concentrate in a small part of the total reach. This will improve the possibilities for discerning variations within the unit.

The usual location for test hole sites is on top of the bank near to but not necessarily immediately adjacent to the stream. The preliminary geomorphic determinations should indicate whether sites on both sides of the channel must be investigated.

In the process of investigation, the geologist should bear in mind that one part of a stream bank may be more subject to erosion than another due to differences in the hydraulic stresses that are exerted by the flow. These stresses are greatest on the bed, on the outside of bends, and along the toe of the banks. Greater emphasis in logging, sampling and testing should be used in searching for and identifying by thickness and location any easily erodible stratigraphic units from midbank to the invert level of the proposed channel. A bank is only as resistant as its weakest segment, particularly if this segment is low in the bank.

Determining the Sample Types to be Obtained

Prior to the detailed investigations, the geologist and engineer have made a tentative decision as to whether data from undisturbed and disturbed samples are to be obtained and the best means for obtaining them. Each stratigraphic unit observed in an undisturbed condition is evaluated before the decision is made as to the type of sampling to be done. It is a good rule to obtain undisturbed samples when in doubt as to whether the unit is characteristically composed of discrete particles or of aggregates that would resist erosion as a mass. The effects of the environment on sediment, once it has accumulated, is discussed in the section on the geomorphology of alluvial deposits. The in-place behavior of materials that resist erosion as a coherent mass cannot be determined by testing disturbed samples. Hence, undisturbed sample analysis is critically important if the in-place deposits differ in erodibility from that of the individual particles. The Unified Soil Classification System and similar systems do not identify erosion characteristics of an undisturbed coherent soil mass except that soils with a high plasticity index (P.I.) are usually more resistant to erosion than noncoherent soils. Some undisturbed soils with a low P.I. are about as resistant to erosion as those with high P.I., and under certain conditions, material with a high P.I. can be as erodible as one with a low P.I. such as soils that are highly dispersive. Another such condition exists when the soil mass is weakened by a fine network of fractures such as that which may be caused by expansive clays. The fractures may or may not be filled with dissimilar material. The above would have to be determined by appropriate tests and observations.

Careful selection of investigation tools and procedures for investigations are necessary to obtain samples and logs of the natural, undisturbed in-place materials. Augers, excavation equipment and many laboratory procedures disturb the soils and provide data which result in erroneous interpretations of in-place properties.

Selection of Equipment for Logging and Sampling

The frequently limited depth required in field investigations for channel improvements and the advantages afforded by in-place examination of the alluvial profile indicates that at least a framework of pit excavations is desirable. Safety precautions must be rigidly maintained. The backhoe is the preferred equipment for such excavations. Once in-place profiles can be examined and sampled above water tables, the pits can be supplemented by drill holes below water tables. Limitations of pits include the possibilities that the depth of sampling required exceeds the capabilities of available equipment, that a high water table would restrict or prevent investigations below that level, and that access is sometimes limited for physical reasons or property owner objections. The advantages of drilling equipment are no depth or ground-water limitations, in-place testing with less site disturbance, and greater safety. For guidance in maintaining safe working conditions, refer to the safety manual for geologic investigations. Light weight drilling equipment mounted on track vehicles may provide a means of collecting disturbed and undisturbed samples from areas that are not accessible to heavy drill rigs.

Sampling equipment is described in NEH, Section 8, Chapter 2. In instances where drilling equipment is used, push tube sampling of relatively soft fine-grained soils or Denison core barrel sampling of hard fine-grained soils are recommended. Special equipment has been devised for undisturbed sampling of soils in investigations for channel improvements where pits or stream banks are the sampling sites. Figure 3-2 is a drawing of the sampling tool which is designed for placement on a shelf dug in the side or bottom of a pit or bank. It is built to obtain more than one sample from the same setup. The thin-walled brass tubes used should be at least two inches and preferably larger in diameter by five or six inches in length. This size is usually sufficient for shear strength tests. Duplicates from the same stratum must be obtained in the event one is damaged in shipping or handling. Following tests in an undisturbed condition, the same material is used for classification, or an additional sample of the disturbed soil can be obtained. The sample or samples are judged to be undisturbed if the soil column on the inside of the tube and on the outside are both level with the upper edge. A tendency for compaction to occur in a tough, dry soil because of the frictional resistance can be reduced by pre-wetting the inside of the tube with water or a small amount of a light lubricant. On completion of sampling, the tubes are sealed. It is essential that the undisturbed samples do not become desiccated.

The cutting of cubes or other block shapes for laboratory testing is another means of obtaining undisturbed samples. It may, in fact, be necessary to obtain such samples if other equipment compacts or otherwise

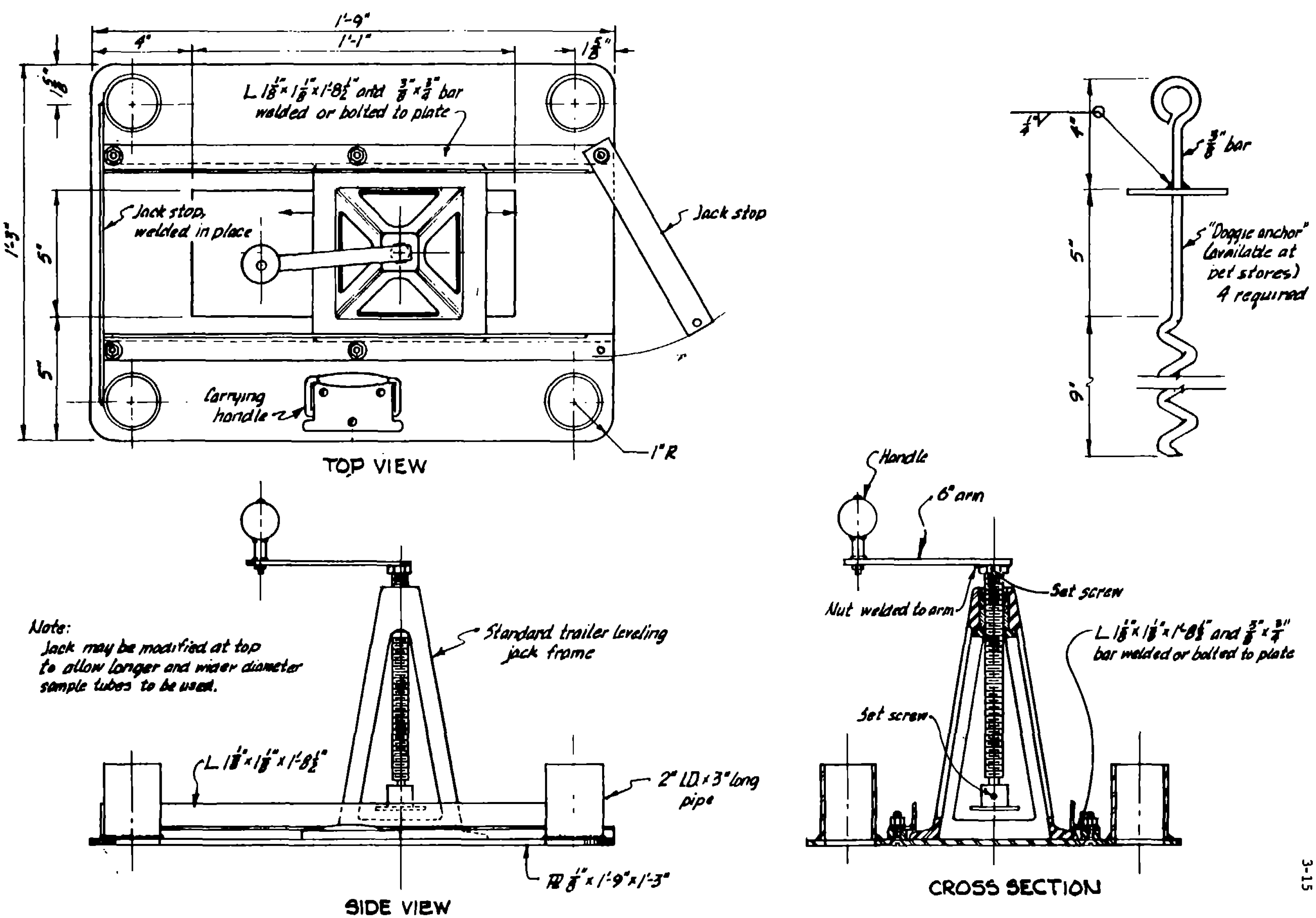

Fig 3-2 - UNDISTURBED-SAMPLE CYLINDER PRESS

SCALE: 3" = 1'-0"

disturbs the soil. While this sampling procedure is usually much more time consuming than push tube methods, it generally results in a much better sample. The presence of coarse sand or gravel in the soil can render the push tube or drill hole sampling equipment unsuitable for use in obtaining undisturbed samples. Exposures in stream banks or pits sometimes exhibit pockets of fine-grained matrix free of the coarse particles. These pockets frequently contain the same material and the same erosion resistance properties as the matrix in an intimate mixture of coarse and fine particles. Sampling and testing of this fine-grained matrix can provide information on the erosion resistance of the mass. Careful evaluation is necessary to insure the applicability of such data.

Channel improvements may include the construction of dikes or embankments to protect adjacent lands from overflow or to increase the resistance of natural materials by compaction. The sample needs for these construction purposes as well as for foundations for all structures should be discussed with the engineer prior to the detailed site investigations. The amounts of disturbed samples needed for various tests and sizes of material are described in Tables 3-3 and 3-4. Packaging and shipping instructions are given in NEH, Section 8, Chapter 3, pages 10 and 11.

Test Hole Logs

Logs of test holes should contain necessary information to ensure that the stratigraphic units are identified and their characteristics, depth of occurrence in the hole and thickness are recorded. If a stratigraphic unit identified in the adjacent test hole fails to appear in this one, this information should be noted. Figure 3-3 is a suggested field worksheet for logging and sampling at test hole sites.

Generally bedrock will not be drilled during investigations. Descriptions of bedrock characteristics are to be provided for the engineer's use. These descriptions must be thorough, complete and accurate.

Determination of the Availability of Movable Bed Material

The availability of movable bed material, specifically sand and gravel, must be determined in order to evaluate its impact on the stability of the improved channel. The most apparent source of supply is deposits that may have accumulated in the bed of the stream within and above the reach to be improved. Whether or not the bed-material supply is a threat to the operation or reasonable maintenance of the improved channel depends on several factors. These include the amount and ease of transport of the material and the difference in hydraulic characteristics between the reach to be improved and the reach upstream that contains the bed material. The relationship between bed material availability and channel stability is discussed in NEH, Section 3, Chapter 4, Transportation of Sediment by Water, pages 4-36 and 4-37.

The channel investigation should include an estimate of the volume of bed material available in the bed of the stream and where the deposit is located with reference to the proposed improvement. If the bottom of the deposit cannot be reached, this should be indicated in the notes. Representative samples of the bed material are to be obtained. The amount required for various gradations of material is given in Table 3-2, NEH, Section 8, Chapter 3. A sample of the surface armor should be obtained, if present, as well as a sample of the underlying finer accumulation.

For analysis of stability, the geologist is expected to define whether discharges critical for stability should be defined as "clear water" or sediment laden flows. In most instances, the wash load will not be of sufficient concentration to classify the flows as other than "clear water". An examination of suspended load records on a regional basis will provide good clues as to the likelihood of significant concentrations. A knowledge of the watershed will also serve to indicate the extent of critical erosion. Reservoirs will reduce concentrations to relatively low levels.

The influence of bed material on the stability of channels is discussed in NEH, Section 3, Chapter 4, pages 4-25 to 4-28.

Where bedrock is to be removed from the channel, the potential effects on stream banks and bed both upstream and downstream must be considered. Where rock is to be removed from a channel section, its disposal must be planned. Alternative disposal methods may include wasting in a preselected area, use in channel construction (riprap, filters), salvage, etc. Local highway agencies are often valuable sources of information concerning rock quality and ease of removal.

Site Investigation Reports

Completion of the Stratigraphic-Unit Profiles

The first step in preparation of site investigation reports is to complete the longitudinal profile and description of stratigraphic units in the channel reach to be improved. As stated on page 3-11, this is

Sheet ____ of ____
Date ____________

WORK SHEET
FOR
LOGGING AND SAMPLING DATA-GEOLOGIC INVESTIGATIONS FOR CHANNEL IMPROVEMENT

Watershed *Cottonwood* Stream *Cottonwood Creek* County *Jones* State *Any*

Site location by survey stationing *65+60* Investigation site number *8* Method of investigation *backhoe*

Log of Unit

Depth (ft) From	To	
0	*3*	STRATIGRAPHIC UNIT NO. *1*
		Description: *A recent deposit of dry, firm, homogeneous, light brown silt (ML), low in organic*
		matter. The deposit breaks down easily into discrete particles and is identified in the field
		as noncoherent. This is the same stratigraphic unit as at investigation site 7, from 0-3.5 ft.
3.0	*6.5*	STRATIGRAPHIC UNIT NO. *3*
		Description: *A dark grey, moist, firm, homogeneous silty clay (CL) relatively free of jointing,*
		roots or other factors that may affect erosion. The unit is identified in the field as coherent.
6.5	*8.0*	*Contains yellow mottling indicative of poor drainage conditions. This stratigraphic unit*
		(3.0 to 8.0 ft.) was identified between 3.5 and 7.5 ft. at site 7. Water table at 7.0 ft.
		STRATIGRAPHIC UNIT NO. ____
		Description:

Figure 3-3

to be developed in tentative form during the preliminary investigation of the site. With the benefit of the additional information obtained during logging and sampling, the original profile is to be refined to include more precise information on the extent, thickness and position in the profile of stratigraphic units. The survey stationing, bank line, and proposed invert level are to be plotted, as are the locations and logs of pits or drill holes. The upper and lower elevations of stratigraphic units will be shown at the appropriate station site. When the correlation of units between pit or drill hole sites has been determined, correlation lines are drawn as shown on Figure 3-4. In this example, it is implied that correlation between test hole sites has been facilitated by exposures in stream banks. Additional test holes would be required where such exposures are not available.

The "U" on the right of the profile test hole location indicates that an undisturbed sample was obtained. A disturbed sample may or may not be obtained at the same site depending upon whether enough material is available in the undisturbed sample for classification. The investigators have identified this stratigraphic unit as consisting of soil which would resist erosion as a mass rather than as discrete particles. The "D" indicates that a disturbed sample has been obtained. The investigators have made a field determination that the unit is non-coherent and that erosion characteristics are dependent on the individual particle characteristics. The stratigraphic unit delineations on Figure 3-4 are not dependent on the indicated investigation sites alone but include determinations from observations along stream banks and from hand borings. When the test data have been obtained, results pertinent to the erosion or stability characteristics may be shown or listed on attached report sheets with the data clearly identified with the appropriate units. Ground water, if encountered, should be shown at each of the investigation test hole locations.

The longitudinal profile with stratigraphic unit interpretations is primarily for purposes of stability evaluation, and without it the designer does not know that interpretations of soil characteristics have been made between investigation sites.

The logs of test pits or drill holes and the plan map showing the location of these investigation sites should accompany the profiles as supporting and supplemental information and a copy will be placed in the design folio.

The geologic profile and plan map should show the locations and the extent of outcropping bedrock. They should also show its verified or estimated depth below the present streambed and its relation to the proposed channel invert. Existing erosion or deposition features associated with the outcrop (either up or down stream) should also be shown.

Analysis of Channel Stability

With the longitudinal profiles, logs and laboratory test data available, the geologist and engineer are in a position to make an analysis of the stability of the proposed channel improvement. In addition, information is needed on the design discharge if bed-material load is not involved as a problem, and the hydrographs for design as well as more frequent discharges if bed-material load has been determined as a problem. Chapter 6 of this Technical Release or designated supplements are to be used for determining the stability of channels that are not affected by bed material transport. NEH, Section 3, Chapter 4, is to be used for the analysis of bed-material transport that may affect channel stability.

Report Outline and Documentation

The longitudinal profile, the plan view, logs, field and laboratory test results, and stability analysis are to form the documentation for a brief report based upon the following outline. Geologists and engineers are both involved in an analysis of these problems. They should work closely together in the determinations and report preparation.

A. Conclusions and Recommendations

B. Description of the geomorphology of the channel improvement area

   Basis for stratigraphic unit delineation with reference to longitudinal profiles or reasons for lack of stratigraphic unit delineation.

C. Copies of logs and test data

D. Criteria used to establish stability

   1. Condition of flow to be used in stability analysis: clear or sediment-laden water. Basis for criteria used.

   2. Bed material evaluation procedure used among those recommended in NEH, Section 3, Chapter 4.

E. Stability analysis

   1. Stability under proposed as-built conditions

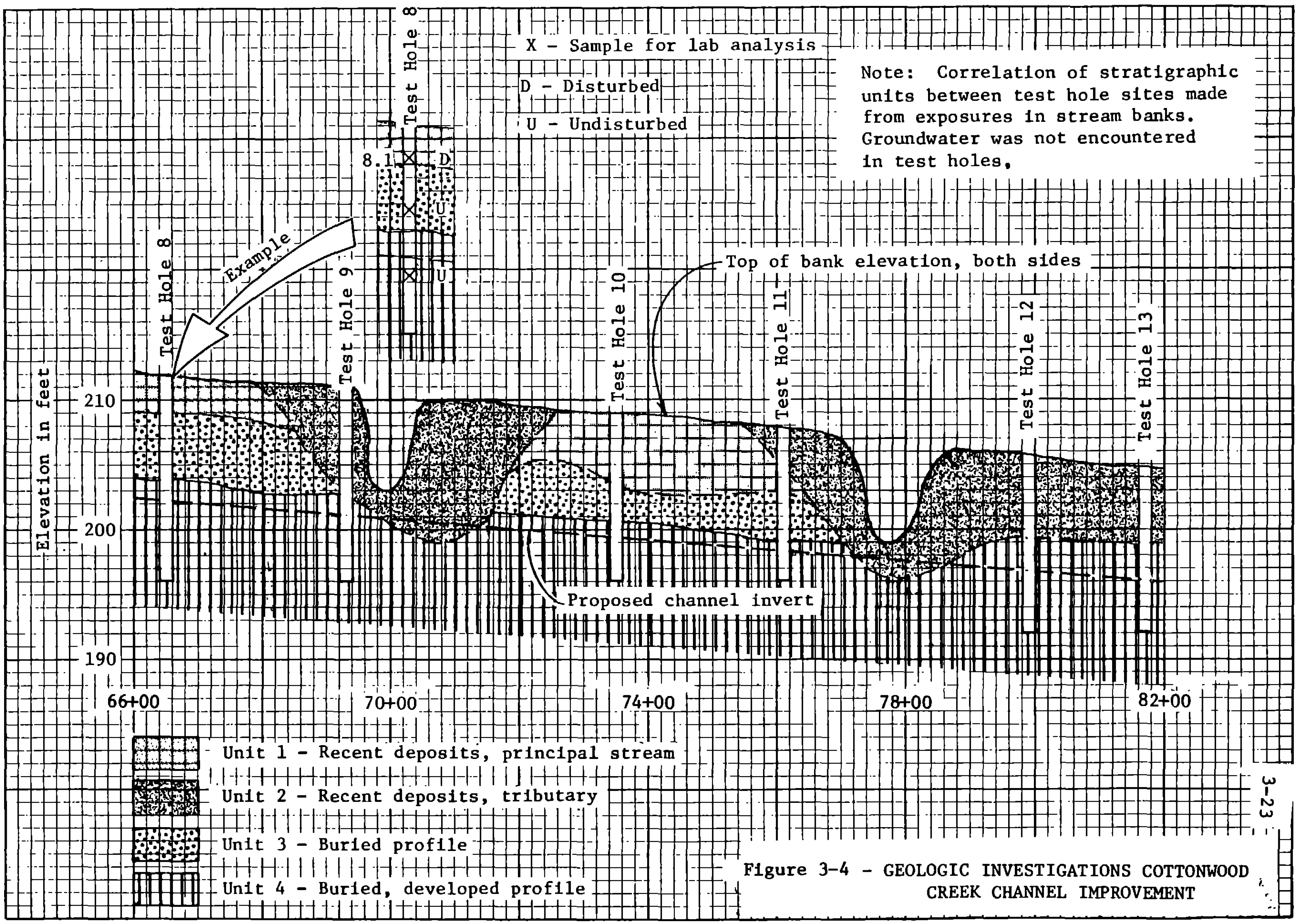

Figure 3-4 - GEOLOGIC INVESTIGATIONS COTTONWOOD CREEK CHANNEL IMPROVEMENT

a. Location of projected unstable reaches, if unprotected; bed and/or banks by stationing.

(1) Elevation of unit or units with reference to invert level.

(2) Cause of predicted conditions, such as tractive stress or velocity in excess of allowable or internal seepage forces.

b. Discuss unique conditions not fitting into the assumptions made in Technical Release 25.

2. Bed material sediment problems if these influence stability of proposed channel improvements.

3. Effect of program installations, such as floodwater, floodwater detention or multipurpose structures, debris basins and grade control structures on downstream reaches.

Evaluation in the Construction Stage

Unexpected discontinuities or unaccounted variations in soil characteristics affecting stability may be revealed during excavations for improvements. It should be standard procedure for the geologist, design engineer and soils engineer to examine the channel during or immediately after excavation. With the profiles at hand, the geologist should make any pertinent adjustments in unit delineation and interpretations over those prepared during the design investigations. Any significant alterations pertaining to stability should be immediately called to the attention of the project and construction engineers.

Testing

Purposes of Testing

The purposes of testing soils from channel projects are fourfold:

1. To develop design values and correlation between properties of fine-grained soils and field evaluation and classification of erosional activity in constructed or natural channels now in use.

2. To provide values for use in the design of stable channels in gravels and coarse sands from the standpoint of erosional activity.

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3. To provide values for the design of channel banks from the standpoint of shear strength and sliding.

4. To evaluate earth loads on structural members.

Test Procedures

Tests may be conducted in the field and/or in the laboratory. Testing procedures should conform to standards of the American Society for Testing Materials (ASTM) or American Association of State Highway Officials (AASHO).

Types of Tests

Two categories of tests are presented---primary tests and supplementary tests. Primary tests are those that provide values that may be used directly (1) to evaluate erosion by either the tractive force, limiting velocity, or tractive power procedures discussed in Chapter 6, or (2) to analyze bank slope stability problems discussed in Chapter 6. Supplementary tests are those that will provide supporting or indicator information that may be of assistance in evaluating erosional resistance and bank stability. These latter tests may be made at local option; results from these tests are not needed to use the design criteria set forth in this Guide, but they may provide valuable clues to the designer.

Index and engineering properties tests are listed in Table 3-1 for erosional analyses and in Table 3-2 for bank stability analyses. Field tests that may be used where conditions warrant are discussed in NEH, Section 8, Chapter 1.

Classification. - - Soils should be fully described and classified according to the Unified Soil Classification System (ASTM D-2488).

Particle size distribution. - - A mechanical analysis should be made on selected samples representing soils that are or may be exposed to erosional forces in channel bottoms and banks to determine the distribution of particle sizes. The coarse fraction should be analyzed using the following U. S. Standard Sieves as the minimum number of size separations: No. 200, 140, 40, 20, 10, 4 and 3/8", 3/4", 1 1/2", and 3". Other size separations may be introduced if considered necessary. The fine fraction should be analyzed using the standard hydrometer tests. Results should be reported on the basis of total material.

Soil consistency tests. - - The liquid limit, plastic limit, and plasticity index should be determined for all soils, except clean sands and gravels, in order to differentiate between materials of appreciable plasticity and slightly plastic or non-plastic materials.

Table 3-1

Erosional Analyses

A. Index Tests

1. Classification 1/

   a. Mechanical analysis

   b. Liquid limit

   c. Plastic limit

   Plasticity index

2. Dispersion

   a. Field tests to identify possible problem areas

   b. Lab tests for soils with more than 15% >.005 mm and for a P.I. >8

3. Soluble salts

4. Natural dry unit weight

B. Engineering Properties Tests 2/

1. Permeability

2. Shear strength

   a. Unconfined compression 3/

   b. Vane shear

1/ For tractive force, limiting velocity or tractive power procedures.

2/ These tests may be made at local option. Results will provide supporting information which may assist in evaluating erosional stability.

3/ For tractive power procedure only.

Table 3-2

Bank Slope Stability Analyses

A. Index Tests

   1. Classification

      a. Mechanical analysis

      b. Liquid limit
                                    Plasticity index
      c. Plastic limit

   2. Specific gravity

      a. Particles smaller than No. 4 sieve

      b. Particles larger than No. 4 sieve

   3. Natural dry unit weight and natural moisture content

   4. Compaction

B. Engineering Properties Tests

   1. Shear

      a. Unconfined Compression

      b. Direct

      c. Triaxial

      d. Vane

   2. Permeability

C. Other Tests 1/

   1. Dispersion

   2. Soluble salts

   3. Linear shrinkage

1/ These tests may be made at local option. Results will provide supporting information which may assist in evaluating bank stability.

Specific gravity. - - When soils are composed of particles having 95 percent or more (by dry weight) smaller than the No. 4 sieve (4.76 mm.) and when compaction or shear tests are required, specific gravity tests should be performed on the minus No. 4 fraction.

When soils contain five percent or more (by dry weight) of hard particles larger than the No. 4 sieve and when compaction or shear tests are required, absorption and bulk specific gravity (saturated, surface dry) tests should be performed on the coarse materials as well as on the fraction smaller than the No. 4 sieve.

Results of specific gravity tests are needed for computation of porosity, void ratio, and associated air-water-solid relationships; also results are needed for adjustment of moisture-density relationships for compacted materials having five percent or more larger than the No. 4 sieve (by dry weight).

Strength considerations. - - Some measure of inter-particle attraction may prove helpful in evaluating erosional stability of channels on the basis of the tractive force theory. Although shear strength obtained by various tests has been used by some investigators, it may not be a true indicator of inter-particle attraction.

Strength values for use in slope stability analyses of banks should be governed by site conditions and the method of analysis to be used. Therefore, some latitude is needed by the engineer in the selection of appropriate tests to be performed at a given site for determination of shear strength parameters.

1. Unconfined compression test. This test is commonly called a q_u test and may be performed on saturated, undisturbed specimens. The test may be made for two purposes: (1) to obtain unconfined compressive strength values for use in estimating erosional stability (by the tractive power method) of those soils in which individual particles cohere under natural field conditions, and (2) to obtain values of cohesion (no load strength) for use in analyzing the stability of banks from a strength standpoint. In the latter case, the test is generally applicable to highly plastic soils, but it may be used for other soils where the in-place strength will simulate ultimate field conditions.

   The q_u test should not be performed on those soils in which natural planes of weakness are an inherent characteristic of the soil, as exemplified by stiff, fissured clay or soils having blocky or columnar structural development.

The effects of weathering should be considered in using q_u test results from samples taken at a depth which will later be unloaded and exposed to the atmosphere -- both in an erosional evaluation and in a bank stability analysis. This consideration will be a matter of judgment and local experience.

2. Direct shear tests. Drained, direct shear tests are recommended for determining the strength parameters for use in the analysis of banks against sliding when the effect of pore water pressure is included as a separate item. Cohesive and non-cohesive soils may be tested in this manner.

3. Triaxial shear tests. Consolidated, undrained, triaxial shear tests may be used, but will generally not be required, for strength parameters unless fractured or structurally cleaved soils are involved.

   Consolidated, undrained, triaxial tests may be made without pore pressure measurements when pore pressures which are anticipated in the field can be simulated during testing.

   When triaxial shear tests are used instead of direct shear tests and when pore pressures are considered separately in the stability analysis, it will be necessary to perform consolidated, drained tests or to perform consolidated, undrained tests with pore pressure measurements.

4. Vane shear tests. The vane shear test is a field test which is well adapted to obtaining in-place shear strength of saturated, plastic soils that do not contain gravel. Shear strength values obtained by vane shear tests may be used for bank stability analyses. This type of test is especially applicable when soils are sensitive and when it is difficult to secure and transport undisturbed samples. Vane tests should include results on remolded strength as well as peak strength.

Natural dry unit weight and moisture content. - - The natural dry unit weight and natural moisture content should be recorded for all undisturbed samples. These determinations can be made in conjunction with shear test operations. Proper interpretations of shear test results are dependent upon data on variations in natural dry unit weight and moisture content.

The erosional resistance of many soils, particularly those which are weakly coherent, increases as dry unit weight (density) increases. The natural (in-place) dry unit weight of soils exposed in the channel will provide valuable guidance in the review and evaluation of the performance of existing channels functioning under similar conditions to a planned channel.

Compaction (moisture-density relationship). - - The moisture-density relationship of soils (for a given compactive effort) should be known before shear testing of disturbed (borrow) materials can be instigated or before field control of moisture and density can be maintained. Compaction and field control may be necessary either (1) to assure adequate density and shear strength in levee or dike fills or (2) to increase the erosional resistance of low density, weakly coherent soils which form the bottoms and sides of channels.

Compaction tests should be performed on disturbed samples of selected representative soils on which shear tests are required for design and/or field control of moisture and density are required during construction operations. Compaction tests are also used to evaluate collapse potential of soils. Most highly collapsible soils are also generally highly erosive.

Permeability. - - In those cases where seepage from channel banks may be a problem, the permeability of natural materials may be required to construct flow nets and to analyze seepage forces. Permeability tests should be performed on undisturbed materials which are representative of soils in the problem area.

Field permeability tests will usually be the most effective means of determining permeability rates (a) in non-coherent soils which are difficult to sample representatively, handle and transport to the laboratory and (b) in lenticular and highly stratified soils.

Methods of making field permeability tests are described in NEH, Section 8[5/], Chapter 2; Section 16[2/], Chapter 2; Special Publication SP-SW-0262 of the American Society of Agricultural Engineers[7/]; and the Earth Manual, First Edition, U. S. Bureau of Reclamation[8/].

A permeability test may also be performed as a secondary and companion test to the unconfined compression test when the purpose of the latter is to provide values for use in an erosional analyses of coherent or cemented soils[9/] (see Chapter 6). The permeability test described hereafter is not necessary when the unconfined compression test is made for the purpose of providing values for a bank stability analysis.

A permeability test, as a companion test to the unconfined compression test, should be performed in a falling head permeameter on undisturbed specimens having a ratio of height to diameter equal to approximately two. As soon as consistent head readings are obtained, the test should be discontinued; the specimen should be removed from the permeameter immediately and subjected to an unconfined compression test to determine the coherence of the saturated specimen.

Dispersion. - - The terms dispersive clay or dispersion refer to the stability of soil aggregates when exposed to water. Dispersive soils are highly erosive. When dispersed soils are exposed to weathering, they are easily distinguished by field observation. They tend to melt down like sugar, develop land forms that resemble typical "badland" relief (macro or micro) and form "slick spots" that are generally bare of vegetation.

There are several tests that identify dispersive soils.

1. The field "crumb" test identifies dispersive soils by the formation of a colloidal cloud around a small clod of moist soil dropped into distilled water or very dilute sodium hydroxide.

2. The laboratory dispersion test identifies most dispersive soils by comparing the amount of 0.005 mm. material in a suspension without deflocculating chemicals to the total amount of 0.005 mm. material determined by mechanical analysis.

3. The field dispersion test is an abbreviated version of the laboratory dispersion test.

4. Chemical tests identify dispersive soils by comparing the amount of soluble sodium and the total salt content in the saturation extract or pore fluid of the soil.

None of these tests always correlates with the others or with field performance. It is sometimes necessary to run all tests in order to properly evaluate the adverse effects of dispersive soils.

Whenever dispersion is suspected in an area, laboratory tests should be run to verify and evaluate the problem.

Soluble salts. - - Soils that contain considerable soluble salts, particularly gypsum, are highly erodible and may slough badly. It appears that the recrystallization of gypsum and other salts during sedimentation results in an open, honeycombed soil structure that may be compared to loessial materials. These soils have sufficient stability to stand on vertical banks as long as they are dry, but they slough and erode rapidly when exposed to moisture. Sodium salts have a dispersive effect on soils and generally increase the erodibility of soils. Although high soluble salt content in soils tend to keep the soil in a flocculated or aggregated (erosion resistant) state, these salts would generally be removed or leached back into channel banks during high flows, possibly leaving a deflocculated, dispersed, erodible surface layer. In arid and semi-arid regions, the soluble salt content may be determined for representative samples, even though results of this test cannot be used quantitatively at present.

Linear shrinkage. - - Coherent soils which have linear shrinkage factors in the range of ten percent or more are generally subject to considerable density and strength changes with variations in moisture content. Results from linear shrinkage tests on coherent soils serve as indicators of potential problems related to bank stability.

Field Sample and Test Specimen Requirements

The quantity of materials needed for testing depends primarily on such factors as (1) number, type and purpose of tests to be performed, (2) particle size characteristics of the material, and (3) homogeneity of the natural materials, particularly when undisturbed specimens are to be tested. The minimum number of test specimens for one test is given below:

Unconfined compression	1
Direct shear	3
Triaxial shear	3
Permeability	1

It is suggested that undisturbed samples be large enough so that an additional test specimen will be available as a spare. In the case of small diameter tube samples, duplicate sampling should be obtained.

Sample sizes and specimen requirements for testing of soils from channel projects are given in Table 3-3 (Disturbed Samples) and 3-4 (Undisturbed Samples). The specimen sizes given in Table 3-4 are those which are actually required for testing; these sizes do not include allowances for trimming.

Table 3-3

Minimum Size of Disturbed Samples

Required for Testing Purposes

Test	Maximum Size to be Included in Samples	Gradation Characteristics of Natural Material		
		90-100% Passing #4 Sieve	50-90% Passing #4 Sieve	0-50% Passing #4 Sieve
Mechanical analysis, liquid limit, plastic limit, specific gravity, dispersion, soluble salts, linear shrinkage	3 in.	4 lb.	10 lb. 1/	40 lb. 1/
Compaction and shear	3 in.	25 lb.	40 lb. natural material	2/

1/ Provide estimates of the percent of oversized material (plus 3 inch size) excluded from samples shipped to laboratory.

2/ Forty pounds of these materials will be adequate for all applicable tests.

Table 3-4

Minimum Specimen Requirements (After Trimming) for Testing Undisturbed Samples

Maximum Particle Size	Test 1/	Size of Specimen
No. 4 sieve (4.76 mm.) 2/	Unconfined compression	1.4" dia. x 3" long
	Direct shear	2.0" x 2.0" x 1" thick
	Triaxial shear	1.4" dia. x 3" long
	Permeability	2" dia.
1/2 inch 2/	Unconfined compression	2.8" dia. x 6.0" long
	Triaxial shear	2.8" dia. x 6.0" long
	Permeability	4.0" dia.
1 inch 2/	Triaxial shear	4.0" dia. x 8.0" long
	Permeability	6.0" dia.
1 1/2 inches	Triaxial shear	6.0" dia. x 12.0" long
	Permeability	6.0" dia.

1/ Table 3-3 gives the sizes of materials which are required for mechanical analysis, liquid limit, plastic limit, specific gravity, dispersion, soluble salts and linear shrinkage. If undisturbed samples are not large enough to meet these requirements, disturbed samples of the sizes indicated should be taken at the same elevation from an adjacent hole to represent the undisturbed materials.

2/ If materials have scattered particles larger than the sizes shown, it may still be possible to trim these test specimens without going to a larger specimen size.

Appendix to Chapter 3

OUTLINE TO PLAN SITE INVESTIGATIONS AND PREPARE REPORTS FOR CHANNEL IMPROVEMENTS

I. Introduction

A. Investigational considerations

1. Channel improvements may be many miles in length.
2. Depths to channel bottom are shallow, usually less than 20 feet.
3. Correlation is necessary to predict presence of boundary materials between holes to avoid inadequate design.
4. An understanding of the geomorphology of alluvial deposits and the identification of stratigraphic units at each site are necessary for correlation of deposits with similar erodibility.

B. Definition of a stratigraphic unit

A stratigraphic unit is defined in this instance as an identifiable stratum of alluvium or other soil material whose susceptibility to erosion is a reflection of the original materials, mode of accumulation and the changes that have occurred since deposition or soil formation.

II. Geomorphology of deposits

A. Significance of the stratigraphic unit and factors that affect the characteristics and erodibility or resistance of stratigraphic units.

1. The characteristics of the original deposits.
2. Age of the deposit.
3. The environment since deposition.

B. Identification of stratigraphic units

1. Primary identifying characteristics are color, particle size, particle size distribution, plasticity, cementation, density, dispersion, structure, and thickness.

2. A stratigraphic unit occupies the same position relative to other beds in a sequence.

3. A stratigraphic unit usually occupies the same position across the valley as it does near the present channel.

C. Geomorphic history determines the erosion resistance of stratigraphic units

1. The particle size distribution and texture of the deposit are the primary determinants of erosion resistance initially.

2. Changes in the character of the material occur with time such as:

    a. The movement downward of fine particles.

    b. Weathering.

    c. Consolidation by overburden, alternate wetting and drying, desiccation, etc.

    d. Cementation.

    e. The accumulation of humus.

3. The geomorphic history results in a characteristic erosion resistance for each stratigraphic unit.

4. The unit may consist of individual particles without connecting bonds.

a. A grain size (as D_{50}, D_{65}, D_{75}) is the quantitative identifier of erosion resistance.

b. Disturbance does not affect the erosion resistance.

5. The unit may consist of particles that cohere or that do have a connecting bond.

   a. The degree of coherence is strongly affected by the geomorphic history.

   b. Disturbance destroys some of the affects of geomorphic history on the unit's erosion resistance.

D. Discontinuities in stratigraphic units

1. The stratigraphic unit may completely disappear from one test hole site to another. This is identified as a discontinuity.

2. The stratigraphic unit may be interrupted by erosion and replaced by different materials. This is also identified as a discontinuity.

3. The stratigraphic unit may locally thin or thicken. This is a variation within a continuous unit, <u>not</u> a discontinuity.

E. Stratigraphic units without internal continuity

1. Some stratigraphic units vary widely in grain size and other characteristics within a short distance.

2. Such variations cannot be correlated between one test hole and another.

3. Such units are identified as lacking internal continuity.

III. Data requirements and observations preliminary to site investigation

A. Data requirements prior to investigations

1. Longitudinal profile of channel reaches to be improved and includes:

   Survey stationing and profiles of existing and proposed channel invert (bottom) and top of bank

2. Flood plain map to include:

   a. Existing channel.

   b. Proposed channel.

   c. Survey base line.

3. When to evaluate stability of existing channels prior to field investigations.

   a. Capacity, slope, frequency at bank full, and other hydraulic measurements are similar to proposed improvement.

   b. Channel boundary soils are exposed to hydraulic forces.

   c. Stability of channel under specified flood discharges and frequencies can be identified.

   d. Water table fluctuations affect bank stability.

B. Reconnaissance of site for channel modifications

1. Inspect soils to and below proposed channel invert level. Study exposures along stream banks and in stream bed. Supplement by test pits or borings where necessary.

2. Interpret geomorphology, tentatively identify stratigraphic units. Geologic, soil survey and topographic maps are very useful for study preliminary to the investigation.

3. Identify places of observation on longitudinal profile.

4. Prepare tentative stratigraphic unit delineation on profile from observations made above.

IV. Site investigations

A. Determine the nature and intensity of site investigations. Factors that affect the intensity of site investigations:

1. Site limitations may require channel lining independent of soil characteristics. Investigations limited to determination of site preparation and construction problems.

2. A potential high hazard to life and property can require a closer spacing of test holes, more intensive soil sampling and testing than a low hazard site.

3. A complex geomorphology will require a closer spacing of test holes, more sampling and testing than a site with few stratigraphic units that are continuous.

4. Environmental considerations.

B. Location of sites for logging and sampling

1. Evaluate information obtained under Item A above.

2. Determine the sample types to be obtained

a. For any specific reach, consider whether coherent soils will be exposed at the toe and/or lower bank.

b. If the answer under 2a is affirmative, plan undisturbed soil sampling of this and overlying coherent stratigraphic units.

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c. If the answer under 2a is negative, plan disturbed soil sampling of this and overlying stratigraphic units, whatever the characteristics of the latter.

C. Select equipment for logging and sampling

1. Choose backhoe or dragline for site investigations where possible and if water table is below invert.

2. Choose drilling equipment if depth is greater than practical for excavation equipment, because of high water table, or if in-place observations are unnecessary or impossible.

3. Select appropriate equipment and containers for undisturbed samples.

D. Data requirements at test holes

1. Log test holes (A suggested worksheet is included as Figure 3-3 of Chapter 3 (revised) TR-25) to depths of at least three feet below invert.

   a. Log test holes by stratigraphic units.

   b. Log appearance and condition in accord with ASTM D-2488 (Unified Soil Classification System).

2. Identify elevation of water table if encountered.

3. Record results of field tests.

4. Obtain representative samples of stratigraphic units. Sampling in undisturbed or disturbed state depends on determinations under IV B 2 above.

5. Supplement to site investigations plan.

   A break in continuity of stratigraphic units, changes in dispersion test results or other significant changes in soil characteristics between test holes requires an additional hole or holes to delineate changes.

F. Determine availability of bed material

1. Sand or gravel can aggrade an improved channel.
2. The hazard to the improved channel depends on whether expected flows will:
   a. Exceed the allowable tractive force for the characteristic grain sizes.
   b. The supply of bed material is sufficient to cause aggradation problems.
3. Measure or estimate depth of bed material in reach to be improved and in channel upstream. Note if bottom cannot be reached.
4. Obtain representative bed-material samples for determination of size distribution.

G. Define "clear water" or "sediment load" flow condition where required.

1. Evaluation of channel stability (allowable velocity or critical tractive force) may depend on how flow is classified.

2. For appropriate classification consider:

   a. Suspended sediment load records in the area.

   b. Severity of erosion in the watershed.

   c. Reservoirs or other structures affecting sediment load.

V. Site investigation report

A. Organization of field data

1. Adjust longitudinal profile of stratigraphic units prepared in tentative form as indicated in Item III B 4 to reflect more detailed observations under IV, Site Investigations.

2. Attach logs of test holes and results of field tests.

B. Report on findings

1. Prepare brief statement on site geomorphology to support delineations on longitudinal profile.

2. Prepare brief statement concerning findings under IV F. - availability of bed material.

3. Prepare brief statement on findings under IV.G. - "clear water" or "sediment load" flow conditions.

VI. Field and laboratory soil testing

A. Field testing

1. In-place density

2. In-place moisture content

3. Vane shear

4. Permeability

5. Dispersion of clay soils - Crumb test

B. Laboratory testing

1. Index tests

a. Classification

(1) Mechanical analysis

(2) Liquid limit

(3) Plastic limit

b. Specific gravity

c. Compaction

2. Engineering properties tests

a. Strength

(1) Unconfined compression

(2) Direct shear

(3) Triaxial shear

b. Permeability

3. Other tests

a. Shrink-swell

(1) Shrinkage limit

(2) COLE

(3) Linear shrinkage

(4) Free swell

b. Soluble salts

c. Base exchange

d. Dry unit weight - natural moisture

e. Dispersion

(1) Laboratory dispersion test

(2) Crumb test

(3) Chemical tests

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CHAPTER 4. DETERMINING DESIGN DISCHARGES

General

Procedures for determining peak rates and volumes of runoff are available in the NEH, Section 4. [17] Procedures for determining channel capacity for irrigation canals and laterals may be found in NEH, Section 15,[18] and for drainage channels in NEH, Section 16[2].

The adaptation of hydrology will be related to two principles in channel design. One principle is the design of a channel for a peak rate of discharge for a selected frequency of occurrence. The second principle is to determine the rate of discharge required to remove a volume of runoff within a specified time limit.

Flood routing requirements will be different for the two principles. The requirements will differ, too, for other needs as illustrated in Table 4-1.

Procedure for Determining Required Channel Capacity

The usual practice in determining the capacity requirement for channels is to make estimates in the planning stage that are sufficiently detailed for final design. A reconsideration of capacity requirement in the design stage will ordinarily be necessary only if there is a departure from the original plan.

Channels Designed for a Peak Rate and Selected Frequency

These channels are planned and constructed to contain the peak rate of runoff for a design flood of a given frequency. The hydrology for this type of channel involves estimates of local runoff for the design frequency and flood routing through the channel system.

The method of determining runoff and flood hydrographs from subwatersheds for the desired frequency can be selected from those described in NEH, Section 4,[17] Chapters 5, 10, 16 and 18. Chapter 5 describes the use of stream flow data for estimating the rate of discharge. Chapter 10 covers "Estimation of Direct Runoff from Rainfall," for cases where stream flow data are not available or adequate. Rainfall amounts for selected frequencies and durations, as found in the Weather Bureau's Technical Paper No. 40, Rainfall Frequency Atlas of the United States,[19] are generally adequate to provide a basis for an estimate of direct runoff from a specified area. Chapter 16 of NEH, Section 4,[17] covers development of hydrographs, and Chapter 18 describes statistical methods for determining the frequency of flood events.

TABLE 4-1

Investigational Procedure	Flood Routing Required by Project Purpose: Containment of a Peak Rate of Runoff of Specified Freq.	Removal of a Volume of Runoff Within a Time Limit	Open Ditch1/ Drainage
1. Determination of design discharge	Channel 2/	None	None
2. Economic evaluation of flood damage reduction			
a. Based on percent chance of occurrence	Channel and overbank 3/	Channel and overbank	None
b. Based on drainage criteria	None	None	None
3. Determination of effects of program on downstream flood peaks	Channel and overbank	Channel and overbank	Channel and overbank 4/

1/ Open ditch for either surface or subsurface drainage.

2/ Channel routing indicates that only storage characteristics of channel are considered in routing computations.

3/ Channel and overbank routing indicates that storage characteristics of both channel and overbank areas are considered in routing computations.

4/ Open ditch channels are frequently designed from drainage criteria which only deals with peak rates of runoff. If a routing is required, design hydrographs must be developed.

The "Convex" routing method described in NEH, Section 4, Chapter 17,17/ is appropriate for establishing the design flood discharge capacity. The permissible mean velocity, channel reach length and the hydrograph of inflow to the reach, are required for channel routing.

A check should be made in every case to determine if the channel will be carrying ground-water flow. If so, an estimate of the discharge rate of ground-water flow should be added to the design capacities, computed above. NEH, Section 4,[17] Chapter 6, describes the separation of ground-water flow from surface flow of recorded flow data.

Channels Designed to Remove a Volume of Runoff Within a Specified Time Limit

The function of these channels is to remove the volume of runoff causing overbank flooding within a period of time that will prevent damage. The required discharge capacity may be calculated by the empirical procedure described in NEH, Section 16,[2] Chapter 6. These procedures are adapted primarily to locations where comparatively level land area is substantial with respect to the watershed area, where watershed boundaries are difficult to delineate, where it is not necessary to flood route and where channels designed on similar sites have provided adequate protection. It may also be based on a design flood of a given frequency.

Drainage Capacity Related to Frequency

General

It is sometimes necessary to estimate the frequency of protection provided by a channel designed by drainage criteria. The frequency may be related to either a peak discharge or a design flood volume for a specified time interval.

Drainage Capacity Related to Annual Flood Peak Frequency

The frequency, equivalent to drainage criteria, may vary between individual reaches and should be estimated for each reach. The flood frequency equivalent may be obtained by the following procedure from NEH, Section 4.

1. If stream flow data are available, the annual flood peaks, minus ground-water flow, may be related to frequency by the procedure described in Chapter 18. Since the bankfull capacity of the channel can be computed, this discharge can be related to the frequency curve developed from annual flood peaks. Average ground-water discharge that the channel will carry may be estimated by procedure described in NEH, Section 4[17], Chapter 6.

2. If stream flow data are not available, the annual flood may be estimated from rainfall. Obtain rainfall amounts for selected storm durations from Weather Bureau Technical Paper No. 40.[19] Select two or more storm frequencies that will encompass the design channel capacity in all reaches. For each selected

rainfall frequency, compute the volume of direct runoff, develop subwatershed hydrographs, and flood route through channel reaches. Plot the routed peak discharge versus volume of runoff in equivalent inches of depth over the watershed area for the selected frequencies, for each reach. See Figure 4-1. The plotted points are described by a straight line originating at zero. The selected frequencies are shown for their respective volume of runoff. The heavy dash lines with arrows indicate entering the graph with the given design discharge for each reach and reading the volume of runoff. The volume of runoff will in turn relate to a specific storm frequency. This is the storm frequency at which the peak discharge can be contained below a damaging stage.

Drainage Capacity Related to Flood Volume Frequencies

A procedure for determining the frequency of a flood volume that can be removed from the overbank areas by channels with a capacity based on drainage criteria, involves the following assumptions:

1. On-farm drainage and tributary laterals have been constructed with outlets into the designed channel.

2. The volume of storage in the channel network is negligible.

3. The overbank areas are broad and almost flat wherein the variation in the overbank stage with respect to time is relatively small.

Analysis with stream flow data. - - When stream flow data are available, a volume-duration-frequency analysis should be made. Volume-duration-frequency analyses have been provided to many states for streams with drainage areas under 1,000 square miles, by the Central Technical Unit, Hydrology Branch, Washington office, and should be used when available. This analysis can be made according to procedure contained in NEH, Section 4[17/], Chapter 18. The analysis can be made for any duration; however, a duration of 24 hours will be used for comparison. Volume-duration-frequency curves are shown for 24, 48, and 72-hour durations in Figure 4-2.

Using the dimensions of the proposed channel, compute the discharge rate for the average overbank stage of flooding for each reach. This computation can best be made at road crossings or other control points where the outflow is confined to the new channel. The average overbank flood stage can usually be estimated within 10 percent at properly selected points such as this. Estimate the average rate of discharge out of the reach by the slope-area

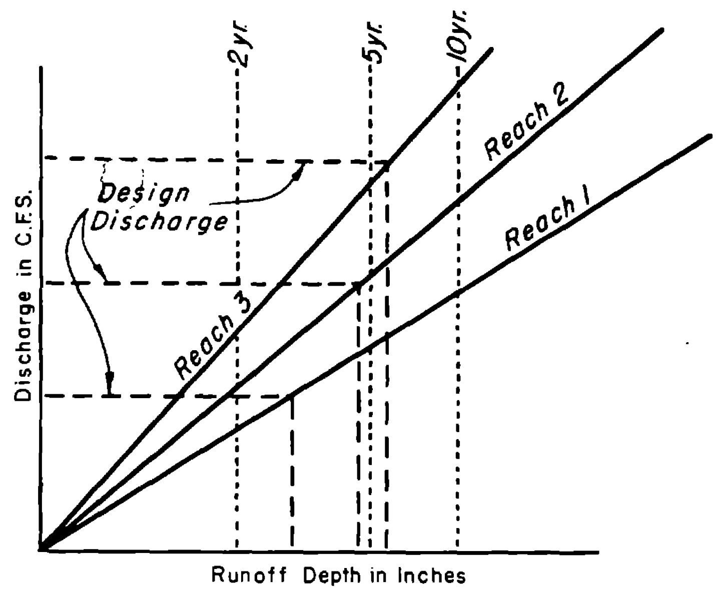

FIGURE 4-1
ROUTED PEAK DISCHARGE VERSUS
AVERAGE DEPTH OF RUNOFF, BY REACH

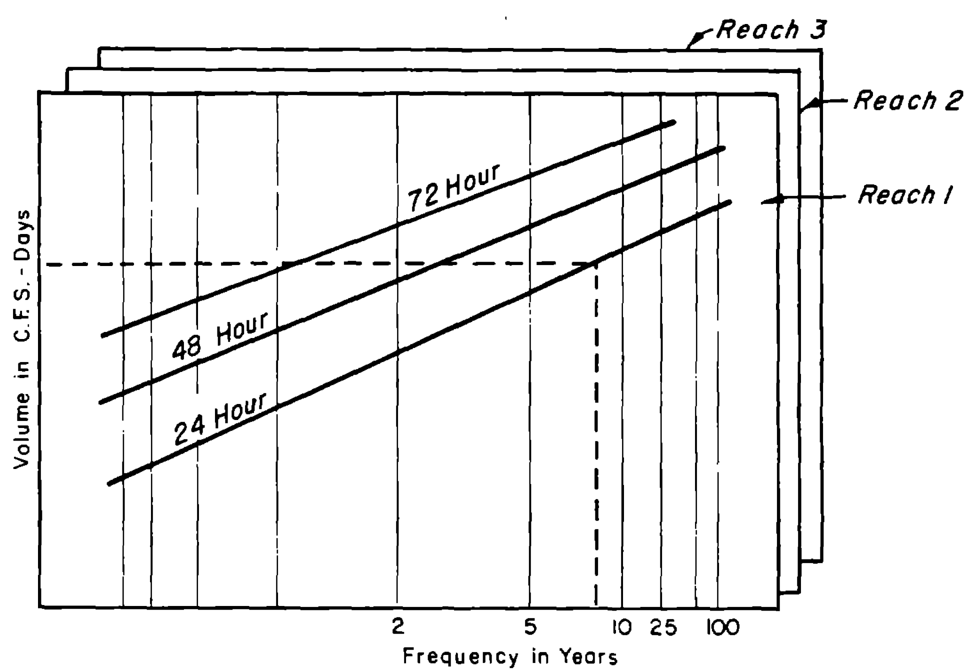

FIGURE 4-2
VOLUME DURATION FREQUENCY ANALYSES

method. This rate of discharge multiplied by 24 hours equals the flood volume removed in cfs-day units. This flood volume is compared with the 24-hour volume-duration-frequency curve from stream flow data to determine the frequency equivalent of the design drainage capacity. (See Figure 4-2.)

The frequency equivalent may be expressed for either of two situations. In many northern states it may refer to the winter and spring snow-melt season or to the summer rain-storm season.

Analysis without stream flow data. - - Where stream flow data are not available, the volume-duration-frequency curves for a 24-hour period may be derived from Weather Bureau rainfall data. The Weather Bureau's Technical Paper No. 40[19] contains maps showing the amount of rainfall expected to occur during a 24-hour period for various frequencies. This may be converted to runoff depth in inches by the procedures in NEH, Section 4[17], Chapter 10. The volume of runoff may be converted to cfs-days by the following conversion factor:

$$\text{Inches depth} \div 0.03719 = \text{cfs-days per square mile}$$

Volume in cfs-days plotted versus frequency provides a graph similar to Figure 4-2. Precipitation amounts for durations longer than 24 hours may be obtained from an analysis of individual precipitation records or by an extrapolation on log paper if this has proven to be valid in the area under consideration. The required frequency equivalent for a channel designed by drainage criteria can be obtained in a manner similar to that for gaged data. See U. S. Weather Bureau TP-49, Two-to-Ten-Day Precipitation for Return Periods of 2 to 100 years in the United States.

This method is limited to channels with drainage areas less than 400 square miles.

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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[20], 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[2]. 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 5[20], 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 16[2])

Step 8. - - Compute the water surface profile for the best apparent cross section. Hydraulic design methods outlined in NEH, Section 16[2], 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 reproportioning 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 ($s_o < 0.001$ ft./ft.) Table 6.1, NEH Section 16[2], 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[4]. 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[22]. 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).

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

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, 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.

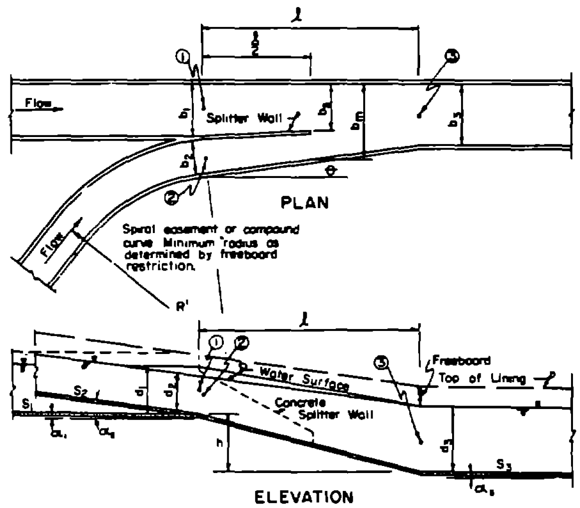
Channel outlet. - - The SAF Basin is the most satisfactory outlet structure where rectangular R/C channels at supercritical flow discharge into an earth section. It is also permissible to use a R/C transition to a trapezoidal rock-lined section. The rock lining should extend a sufficient distance downstream 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.^{2}
- 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.^{2}
- 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_a = average wetted perimeter between two sections — ft.
- P_m = wetted perimeter at middle section — ft.
- Q = rate of discharge — cfs.
- R_a = average hydraulic radius between two sections — ft.
- R_m = hydraulic radius at middle section — ft.
- R' = radius of curve to centerline — ft.
- S_c = critical slope of channel invert — --
- S = channel invert slope — --
- V_a = average velocity between two sections — fps.
- V_m = velocity at middle section — fps.
- B = 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

$$\frac{Q_3^2}{gA_3}\cos\alpha_3+\left(\frac{b_3}{2}+\frac{Bd_3}{3}\right)\left(d_3^2\right)\cos^2\alpha_3=\frac{Q_1^2}{gA_1}\cos\alpha_1+\frac{Q_2^2}{gA_2}\cos\alpha_2\cos\theta+\left(\frac{b_1}{2}+\frac{Bd_1}{3}\right)\left(d_1^2\right)\cos^2\alpha_1+A_a h-\frac{P_a \ell n^2 V_a^2}{2.21\,R_a^{1/3}}$$

Unequal Width Main Channel

$$\frac{Q_3^2}{gA_3}\cos\alpha_3+\left(\frac{b_3}{2}+\frac{Bd_3}{3}\right)\left(d_3^2\right)\cos^2\alpha_3=\frac{Q_1^2}{gA_1}\cos\alpha_1+\frac{Q_2^2}{gA_2}\cos\alpha_2\cos\theta+\left(\frac{b_1}{2}+\frac{Bd_1}{3}\right)\left(d_1^2\right)\cos^2\alpha_1+(b_m+Bd_m)\,d_m h-\frac{P_m \ell n^2 V_m^2}{2.21\,R_m^{1/3}}$$

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

$$\frac{Q_3^2}{gA_3}\cos\alpha_3+\frac{b_3 d_3^2}{2}\cos^2\alpha_3-\frac{b_3 d_3 h}{2}=\frac{Q_1^2}{gA}\cos\alpha_1+\frac{Q_2^2}{gA_2}\cos\alpha_2\cos\theta+\frac{b_1 d_1 h}{2}+\frac{b_1 d_1^2}{2}\cos^2\alpha_1-\frac{P_a \ell n^2 V_a^2}{2.21\,R_a^{1/3}}$$

Unequal Width Main Channel

$$\frac{Q_3^2}{gA_3}\cos\alpha_3+\frac{b_3 d_3^2}{2}\cos^2\alpha_3-\frac{b_3 d_3 h}{2}=\frac{Q_1^2}{gA_1}\cos\alpha_1+\frac{Q_2^2}{gA_2}\cos\alpha_2\cos\theta+\frac{b_3 d_1 h}{2}+\frac{b_1 d_1^2}{2}\cos^2\alpha_1+\frac{(b_3-b_1)}{2}\left(d_2^2\right)\cos^2\alpha_2-\frac{P_m \ell n^2 V_m^2}{2.21\,R_m^{1/3}}$$

SPECIAL CASE

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

Constant Width Main Channel

$$\frac{Q_3^2}{gA_3}+\frac{b_3 d_3^2}{2}-\frac{b_3 d_3 h}{2}=\frac{Q_1^2}{gA_1}+\frac{Q_2^2}{gA_2}+\frac{b_1 d_1 h}{2}+\frac{b_1 d_1^2}{2}-\frac{P_a \ell n^2 V_a^2}{2.21\,R_a^{1/3}}$$

Unequal Width Main Channel

$$\frac{Q_3^2}{gA_3}+\frac{b_3 d_3^2}{2}-\frac{b_3 d_3 h}{2}=\frac{Q_1^2}{gA_1}+\frac{Q_2^2}{gA_2}+\frac{b_3 d_1 h}{2}+\frac{b_1 d_1^2}{2}+\frac{(b_3-b_1)}{2}\left(d_2^2\right)-\frac{P_m \ell n^2 V_m^2}{2.21\,R_m^{1/3}}$$

Criteria

$\tan\alpha < 0.1$

Convergence angle θ less than 10°

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

$\theta \leq 10^\circ$

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

Height of sidewall at ③ $= 1.2\,d_3 + 0.25\,d_{c_3}\left[1-11.1\left(\dfrac{S_3}{S_{c_3}}-1\right)^2\right]+1.2\,\dfrac{V_2^2\,b_2}{gR_2}$

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

$\ell = cV_2\left[b_1+b_2-b_3\right]^{1/3}$

$b_3 \geq 0.8\,(b_1+b_2)$

$b_s=\dfrac{b_1\,(b_1+b_2+b_3)}{2(b_1+b_2)}$ when $b_1 > b_2$

$b_s=\dfrac{b_2\,(b_1+b_2+b_3)}{2(b_1+b_2)}$ when $b_1 < b_2$

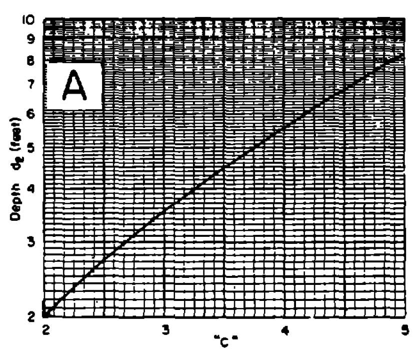


FIGURE 5-1
HYDRAULICS JUNCTION STRUCTURES

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° with the center line, nor diverge at an angle greater than 25°.

3. In well designed transitions, losses in addition to friction should not exceed .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

$$\frac{V_1^2}{2g} + \frac{P_1}{W} + z_1 = \frac{V_2^2}{2g} + \frac{P_2}{W} + z_2 + h_f \qquad \text{(Eq. 5.1-1)}$$

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:

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

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 Δ W.S. equals $1.10\Delta h_v$ for inlets and ΔW.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

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

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

   ΔW.S. equals $1.10 \Delta h_v$ (inlets)

   ΔW.S. equals $.80 \Delta h_v$ (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 Δ 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 reaches.

5. Compute Δh_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

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

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

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.

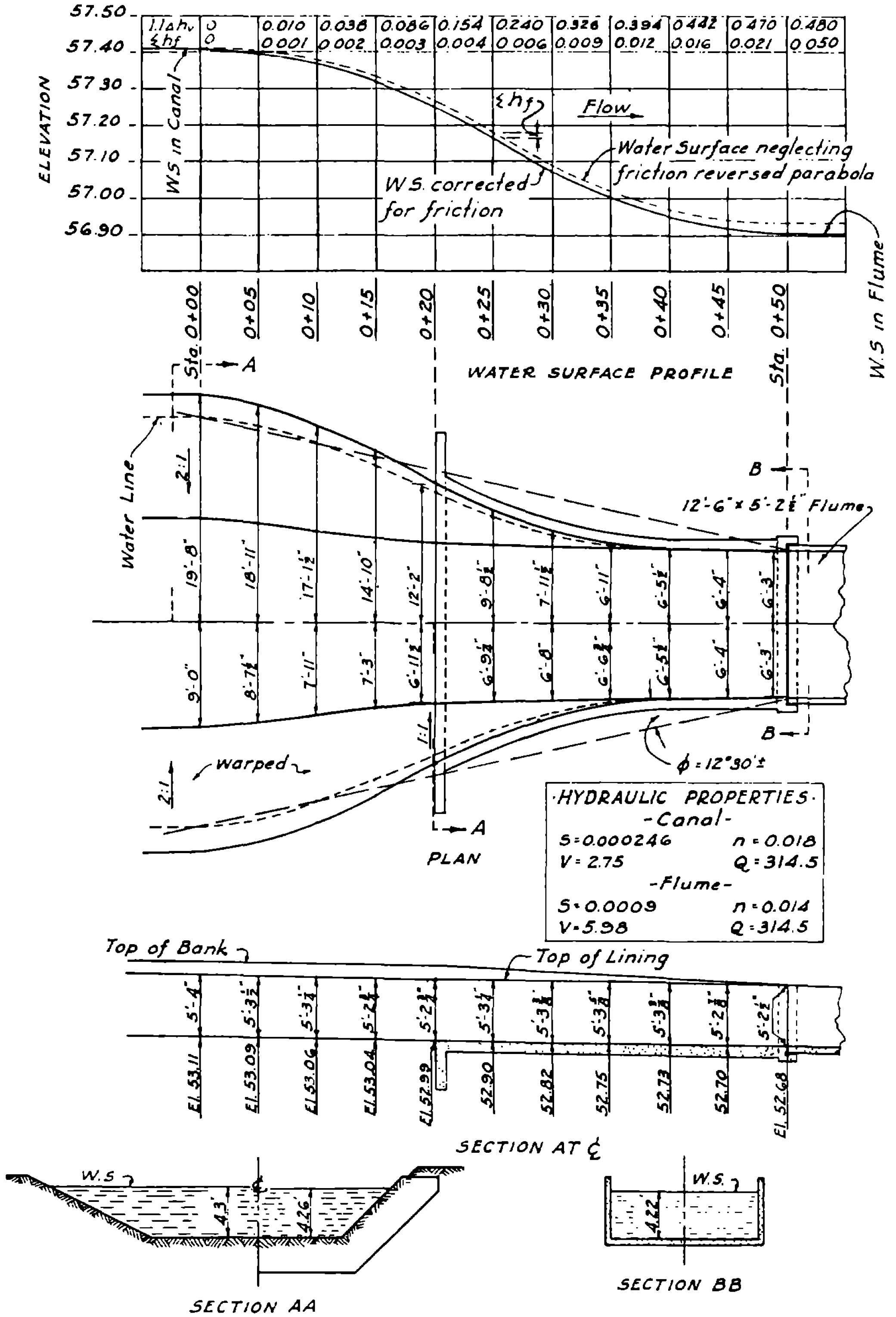
Line	Item	**0 + 00	0 + 05	0 + 10	0 + 15	0 + 20	0 + 25	0 + 30	0 + 35	0 + 40	0 + 45	0 + 50
1	Δ WS = Drop in WS		0.010	0.038	0.086	0.154	0.240	0.326	0.394	0.442	0.470	0.480
2	Δh_v = WS ÷ 1.1		0.009	0.035	0.079	0.140	0.218	0.296	0.357	0.401	0.427	0.436
3	$h_v = 0.117 + \Delta h_v$		0.126	0.152	0.196	0.257	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 ÷ 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 ÷ 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-Δ 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 x H	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"	17'1 1/2"	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



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[20/].

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[2/], 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.)

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	Δ 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	$V = \sqrt{2 g h_v}$	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 $= n^2 v^2 \div 2208 R^{4/3}$	.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	Σh_f		.00124	.00278	.00498	.00870	.01606	.03186	.06238	.10622	.15494
13	W.S. Elev.=103.68-Δ W.S.-Σh_f	103.68	103.649	103.597	103.465	103.221	102.704	101.668	100.418	99.994	99.885
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	3.08
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

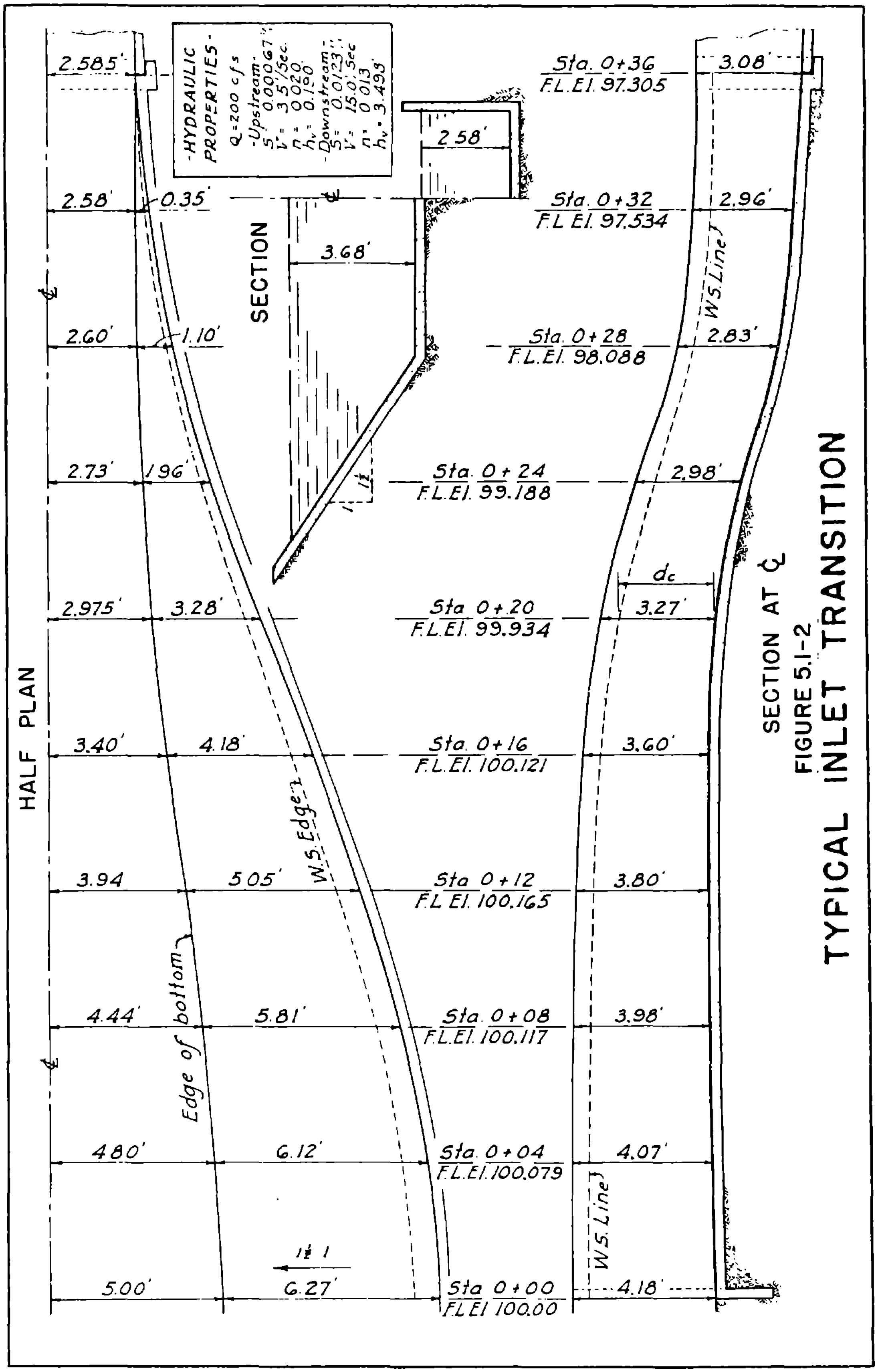


FIGURE 5.1-2
TYPICAL INLET TRANSITION

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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.

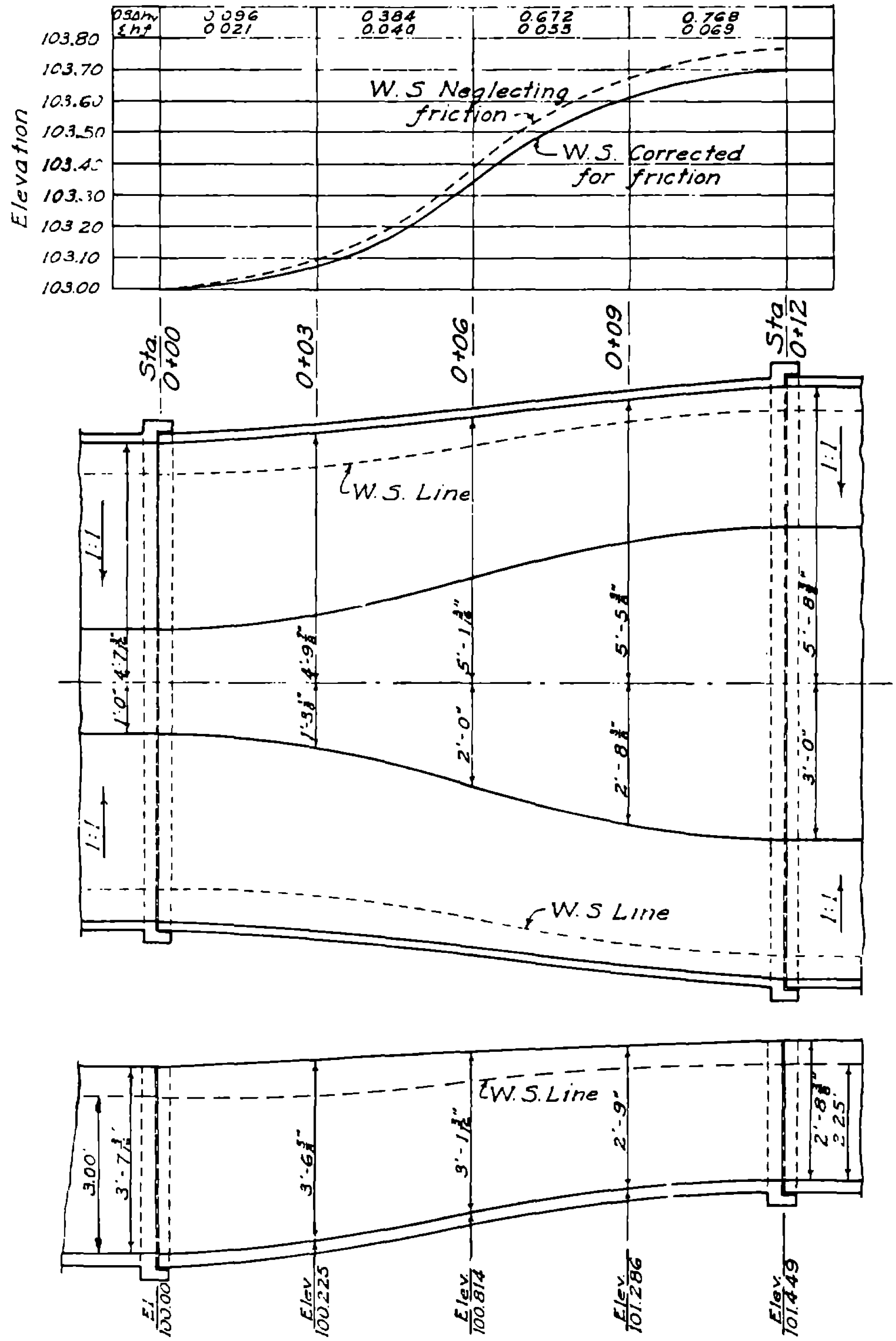
Line	Item	Station				
		*1	2	3	4	5
1	Δ W.S. = Rise in W.S.		0.096	0.384	0.672	0.768
2	$\Delta h_v = \Delta \text{W.S.} \div 0.80$		0.120	0.480	0.840	0.960
3	$h_v = 2.76 - \Delta h_v$	2.760	2.640	2.280	1.920	1.800
4	$V = \sqrt{2gh}$	13.33	13.09	12.11	11.11	10.77
5	Area = $Q \div V$	15.00	15.34	16.52	18.00	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	Σh_f =		.021	.040	.055	.069
13	W.S. El. = $103 + \Delta \text{W.S.} - \Sigma h_f$	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 %	n = 0.012
V = 13.33'/sec	hv = 2.760'	V = 10.77'/sec	hv = 1.800'

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 maps, etc. Suggested minimum radii of curvature are cited in NEH, Section 16[2/], 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[20].

Hydraulic Design

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

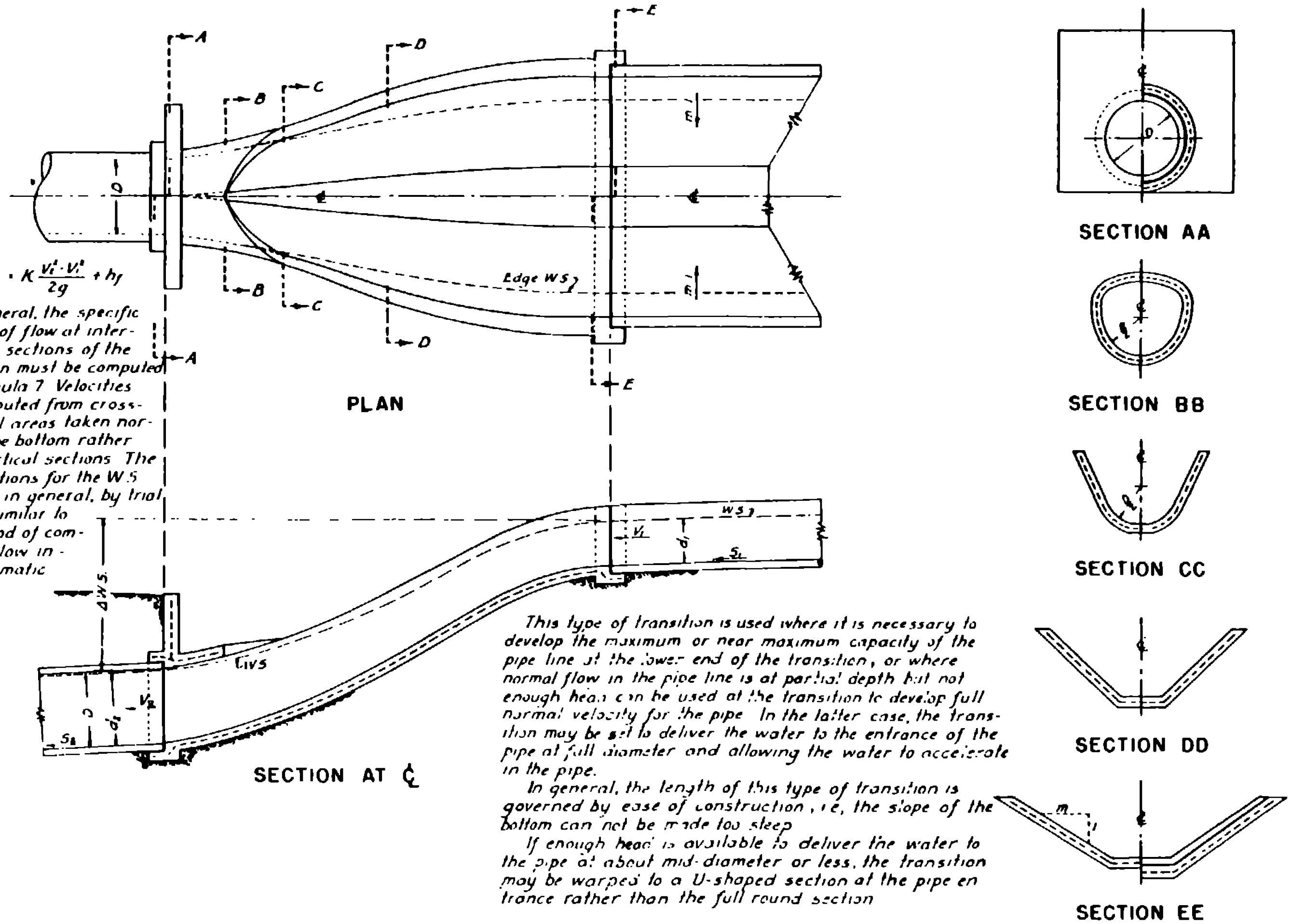
Subcritical Flow

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[2], 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.)

FIGURE 5.1-4

TYPICAL INLET TRANSITION TO PIPE LINE



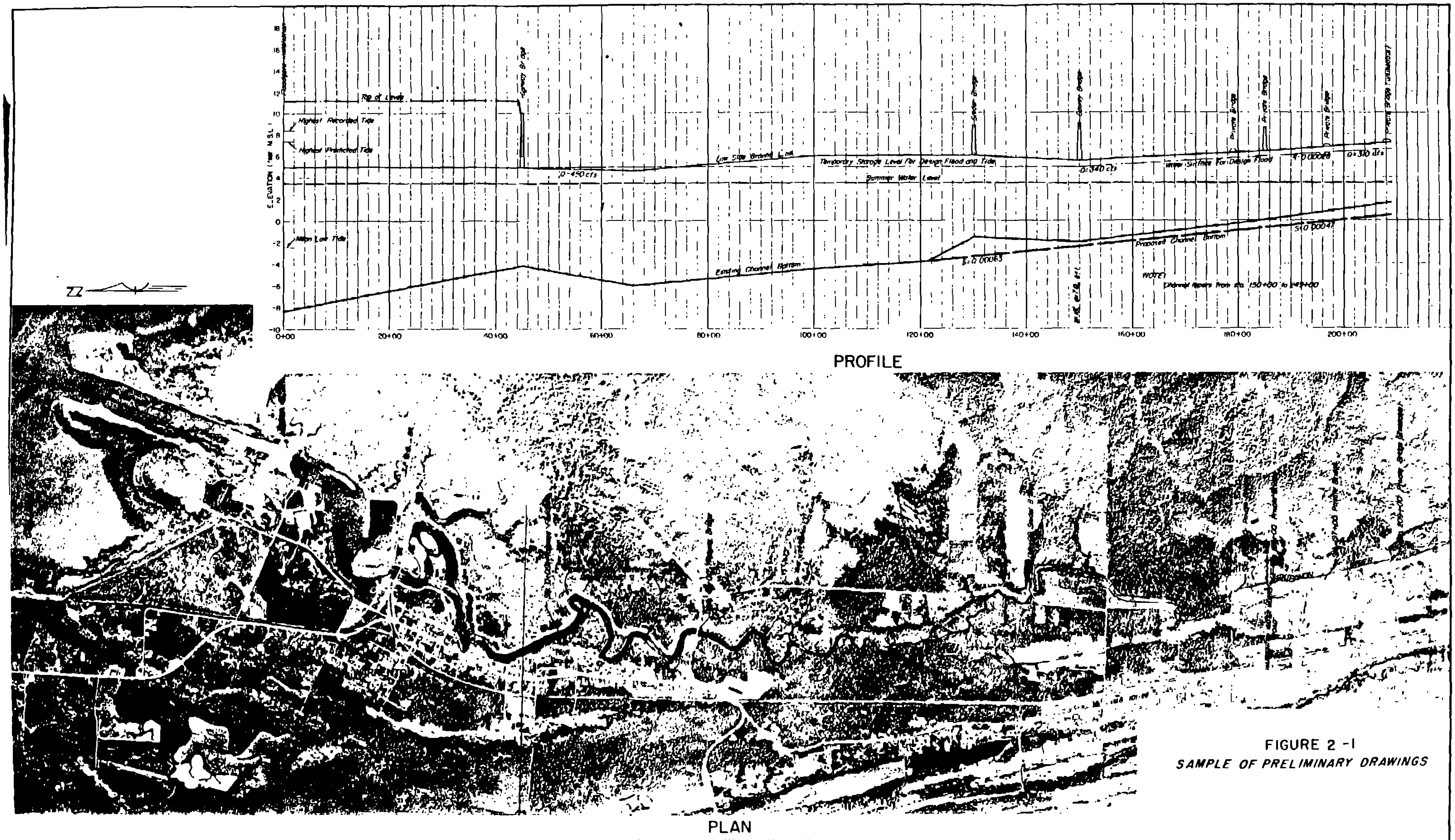
$$\Delta WS = K\frac{V_1^2 - V_2^2}{2g} + h_f$$

In general, the specific energy of flow at intermediate sections of the transition must be computed by Formula 7. Velocities are computed from cross-sectional areas taken normal to the bottom rather than vertical sections. The computations for the W.S. line are, in general, by trial & error similar to the method of computing flow in non-prismatic chutes

This type of transition is used where it is necessary to develop the maximum or near maximum capacity of the pipe line at the lower end of the transition, or where normal flow in the pipe line is at partial depth but not enough head can be used at the transition to develop full normal velocity for the pipe. In the latter case, the transition may be set to deliver the water to the entrance of the pipe at full diameter and allowing the water to accelerate in the pipe.

In general, the length of this type of transition is governed by ease of construction, i.e., the slope of the bottom can not be made too steep.

If enough head is available to deliver the water to the pipe at about mid-diameter or less, the transition may be warped to a U-shaped section at the pipe entrance rather than the full round section.



PROFILE

PLAN

FIGURE 2 -1
SAMPLE OF PRELIMINARY DRAWINGS

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° 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:

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

   W.S. equals $0.75 \Delta h_v$ (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.

247-927 O - 78 - 9

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 equals 1.44 feet

V_c equals 6.5 feet per second

1. Length of transition

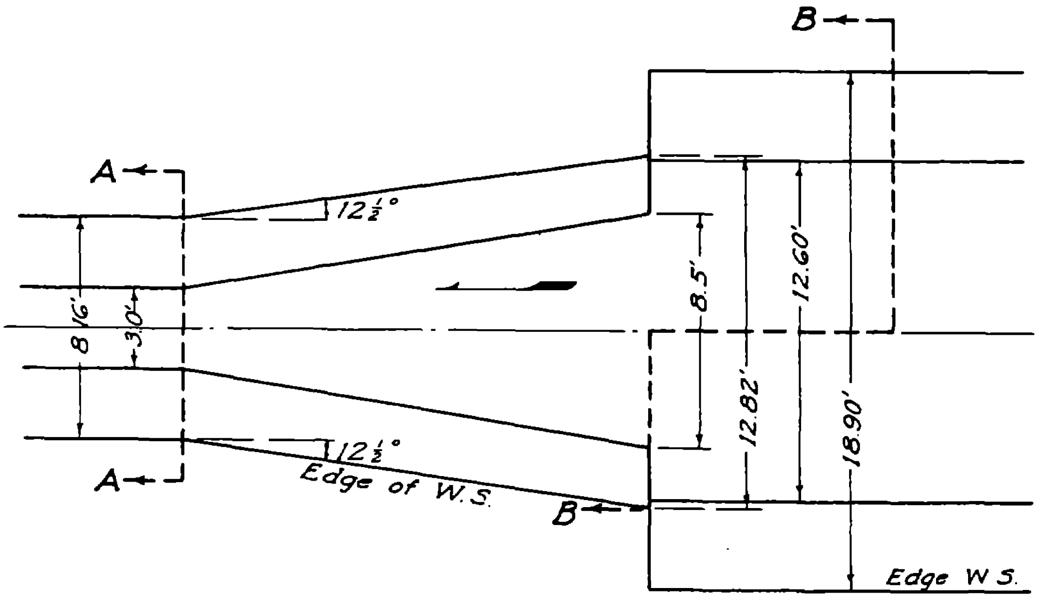
   2.33 x cot 12-1/2° equals 2.33 x 4.51 equals 10.5 feet say 10.0 ft.

2. W.S. equals 1.15 (Δh_v)

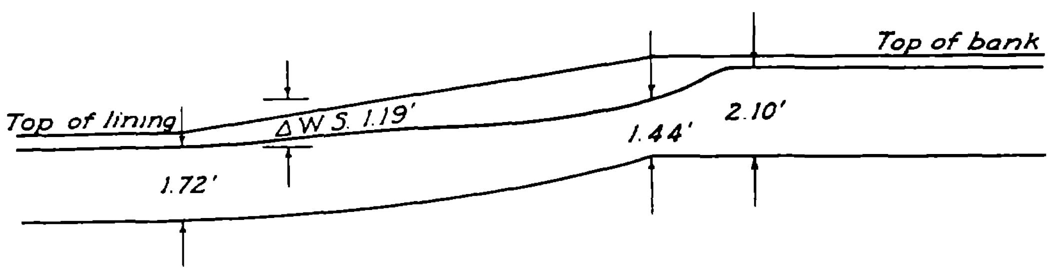
   equals 1.15 (1.035) equals 1.19 feet (See Fig. 5.1-5)

FIGURE 5.1-5

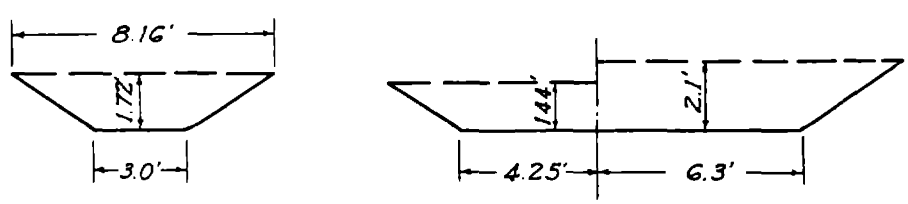
TYPICAL TRANSITION WITH STRAIGHT SIDES



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 supercritical 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.

* 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

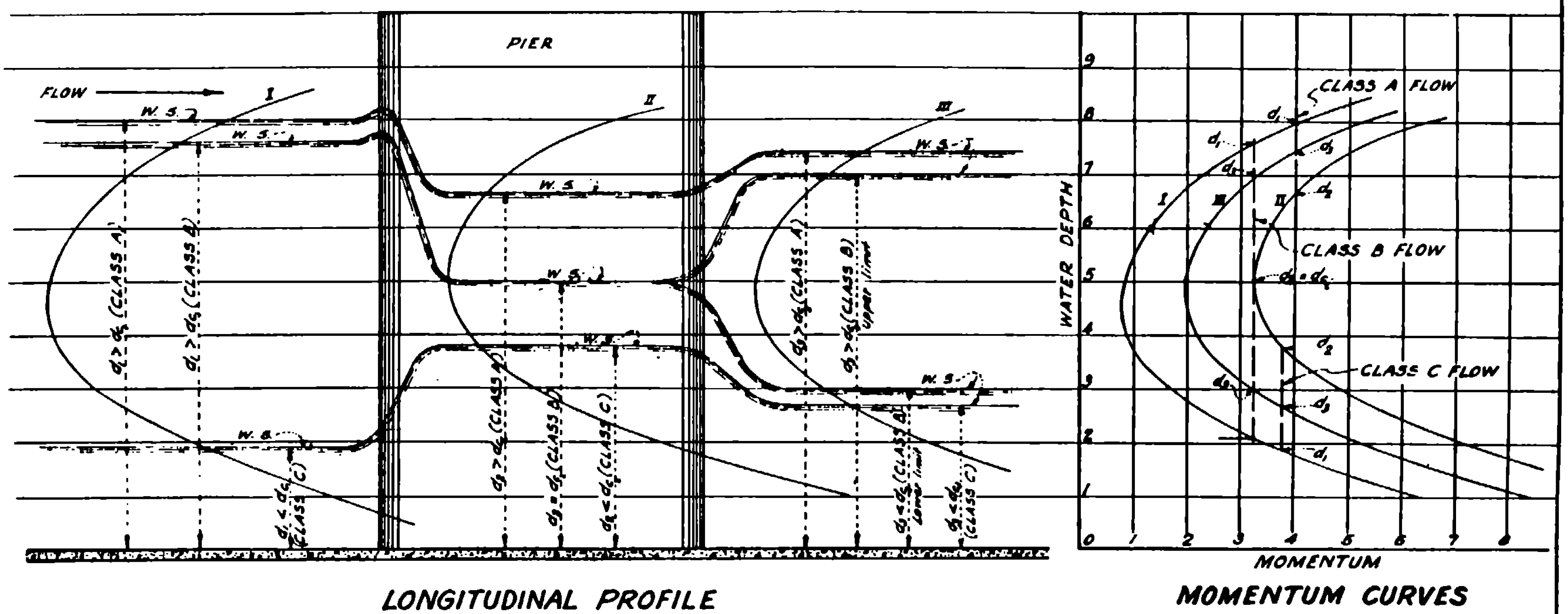
$$m_1 - m_0 + \frac{Q^2}{gA_1} - \frac{A_0}{A_1}\frac{Q^2}{gA_1} = m_2 + \frac{Q^2}{gA_2}, \text{ or}$$

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

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

$$m_2 + \frac{Q^2}{gA_2} + m_0 = m_3 + \frac{Q^2}{gA_3}, \text{ or}$$

$$m_2 + \frac{Q^2}{gA_2} = m_3 - m_0 + \frac{Q^2}{gA_3}$$



GENERAL MOMENTUM EQUATION:

$$\underbrace{m_1 - m_p + \frac{Q^2}{gA_1}(A_1 - A_p)}_{I} = \underbrace{m_2 - m_p + \frac{Q^2}{g(A_1 - A_p)}}_{II} = \underbrace{m_3 - m_p + \frac{Q^2}{gA_3}}_{III}$$

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.
- $d_{c_1} = d_{c_3}$ = Critical in unobstructed channel.

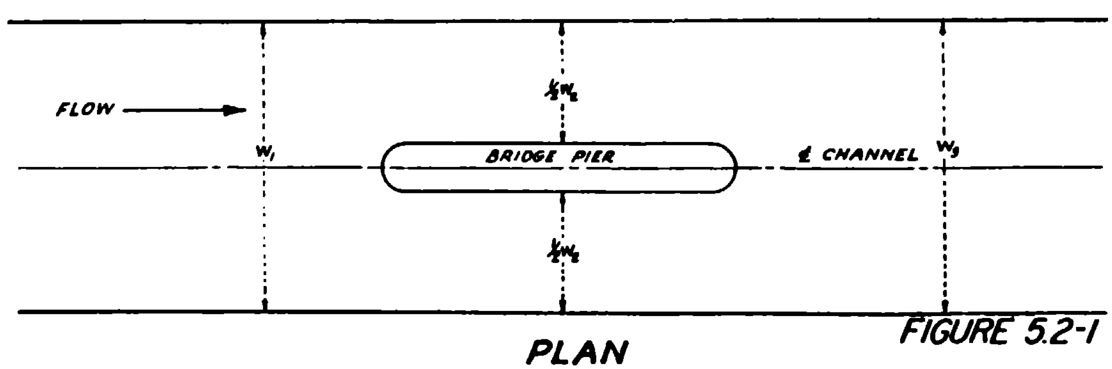


FIGURE 5.2-1 BRIDGE PIER LOSSES BY THE MOMENTUM METHOD

A

B

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:

$$I = M_1 - M_0 + \frac{Q^2\,(A_1 - A_0)}{gA^2{}_1}$$

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

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

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

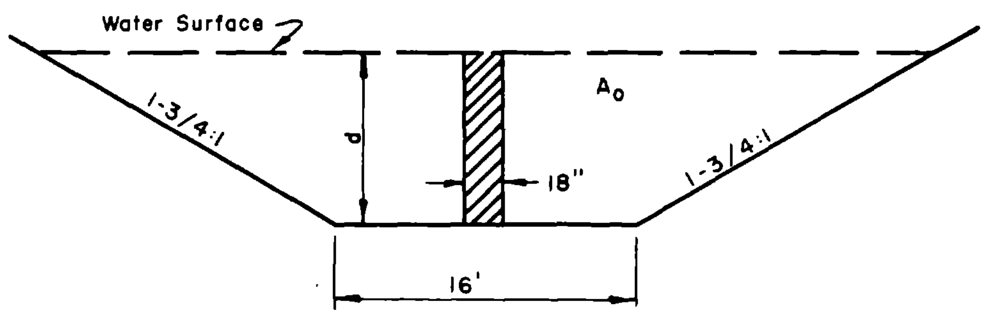


Fig. 5.2-2. Cross-section of channel showing center bridge pier.

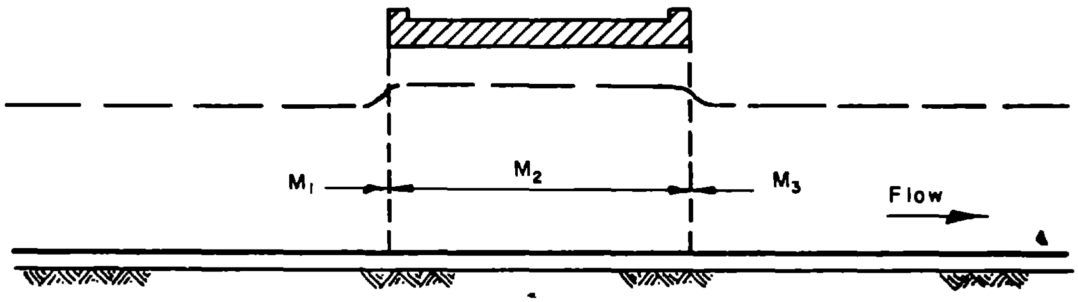


Fig. 5.2-3. Longitudinal section along centerline of channel indicates three locations: I - immediately upstream, II - Under bridge, III - Immediately downstream.

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.

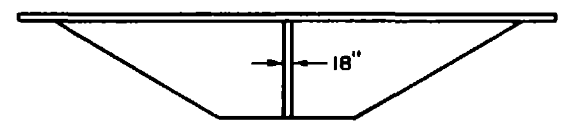
The momentum method of computing the approximate change of water surface is not dependent upon coefficients "K" as are necessary in the formulas derived by Nagler, Weisbach, Rehbock and others (see USDA Technical Bulletin No. 429, "Pile Trestles as Channel Obstructions" and USDA Technical Bulletin No. 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	$A_1 = A_3$	$M_1 = M_3$	A_0	M_0	$A_1 - A_0$	$M_1 - M_0$ (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_1 = Static moment in unobstructed channel

M_0 = Static moment of obstruction

$M_1 - M_0$ = Static moment in obstructed channel

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

$\frac{Q^2}{g\ (A_1\ \ A_2}$ = Kinetic momentum in obstructed channel

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

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

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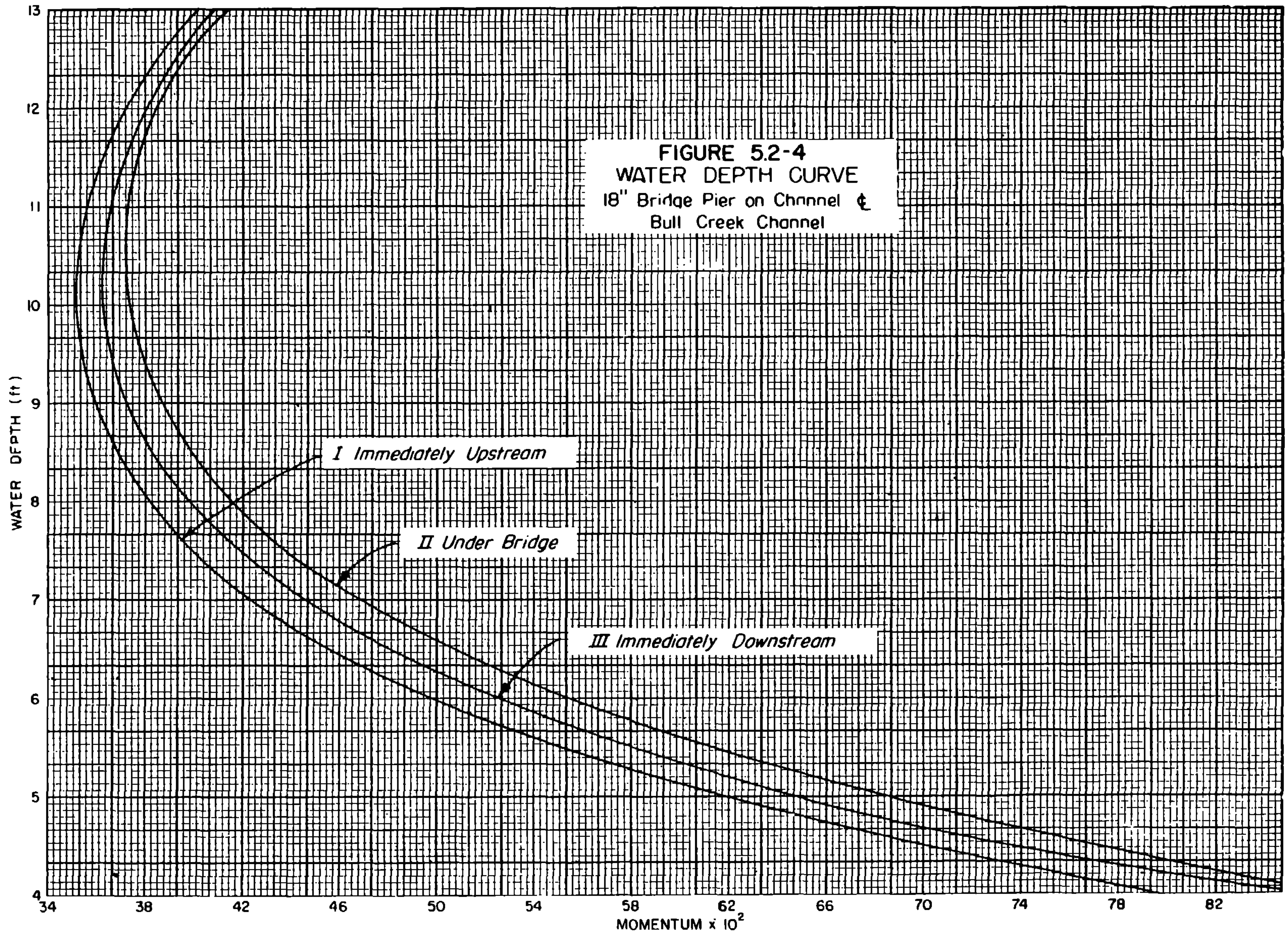

FIGURE 5.2-4
WATER DEPTH CURVE
18" Bridge Pier on Channel ℄
Bull Creek Channel

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CHAPTER 6 - STABILITY DESIGN

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CHAPTER 6. STABILITY EVALUATION AND DESIGN

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

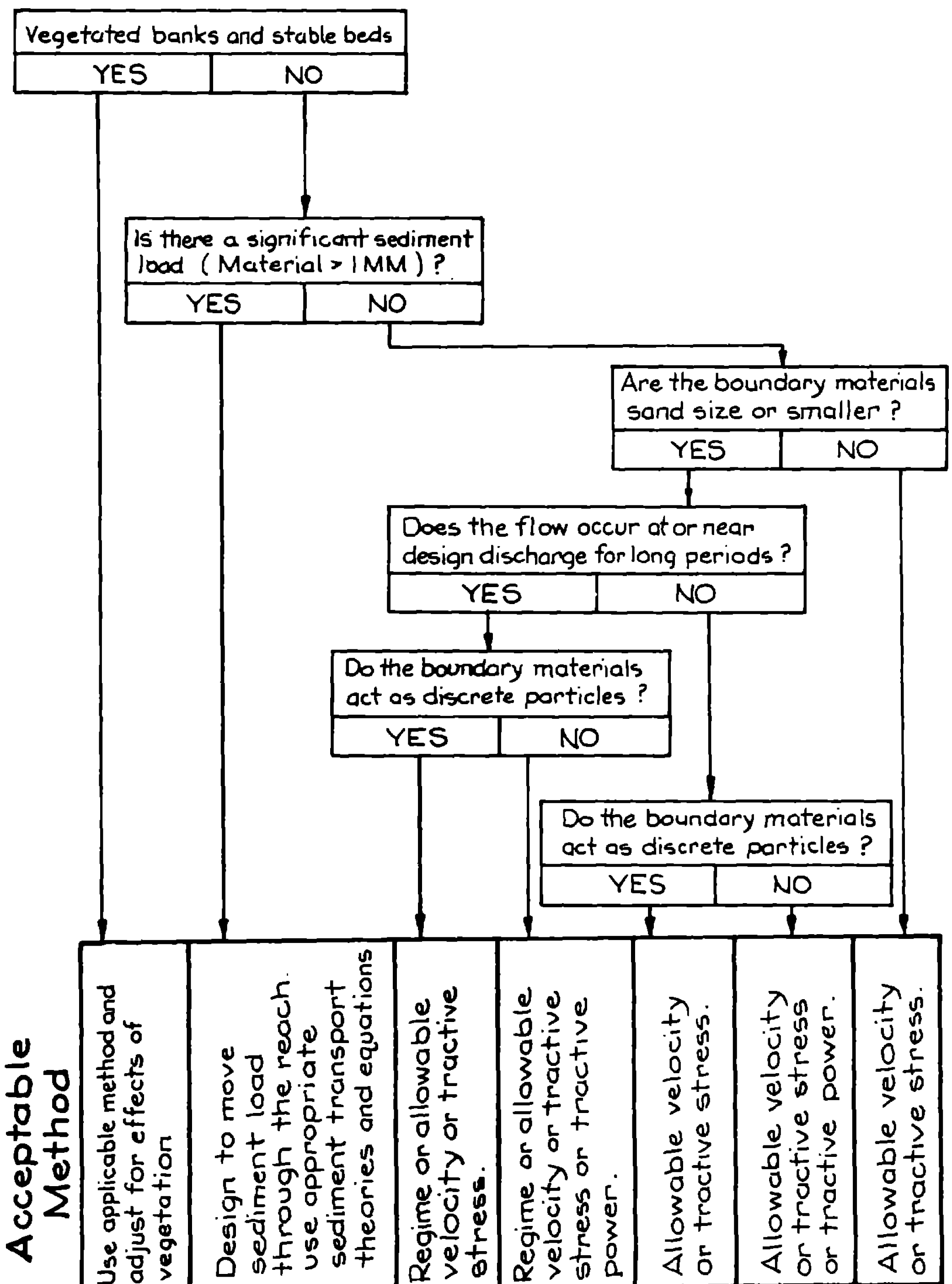


Figure 6-1

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.[52] 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

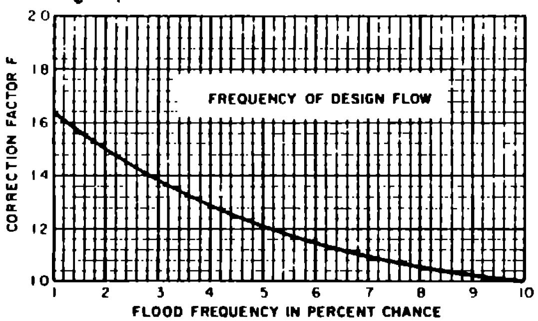
This method of testing the erosion resistance of earth channels is based on data collected by several investigators.

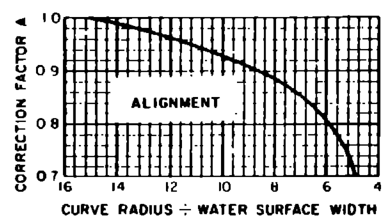
Figure 6-2 shows "Allowable Velocities for Unprotected Earth Channels" developed chiefly from data by Fortier and Scobey[24], Lane[25], by investigators in the U.S.S.R.[26] and others. The allowable velocities determined from Figure 6-2 refer to channels formed in earth with no vegetative or structural 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.

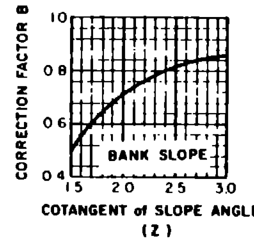
Stability is influenced by the concentration of fine material carried by the flow in suspension. There are two distinct types of flow depending on concentration of material in suspension.

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

6-4







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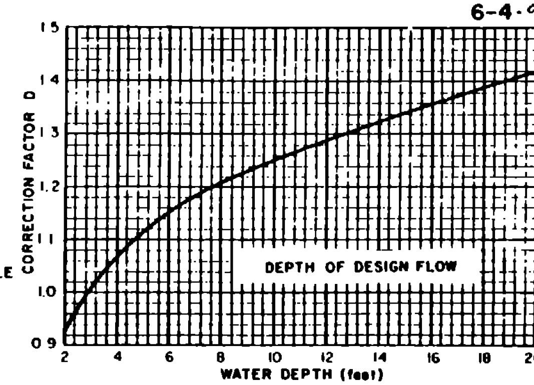
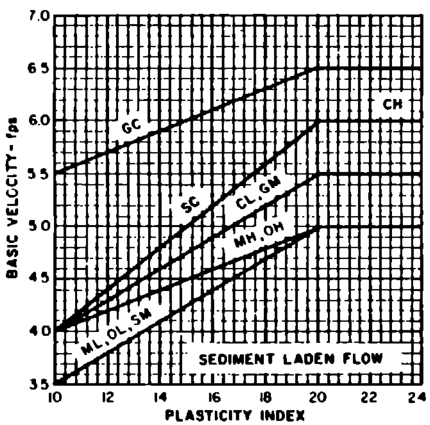
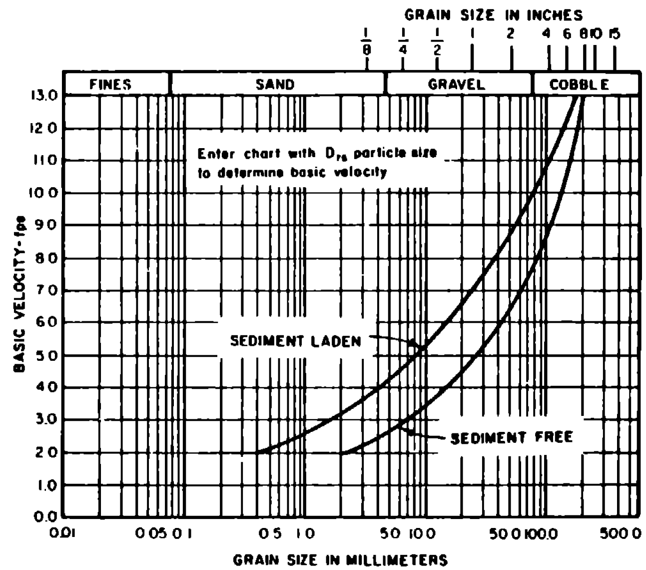
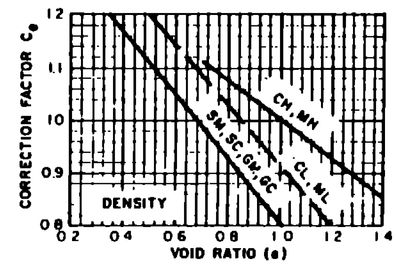




BASIC VELOCITIES FOR COHERENT EARTH MATERIALS (v_b)

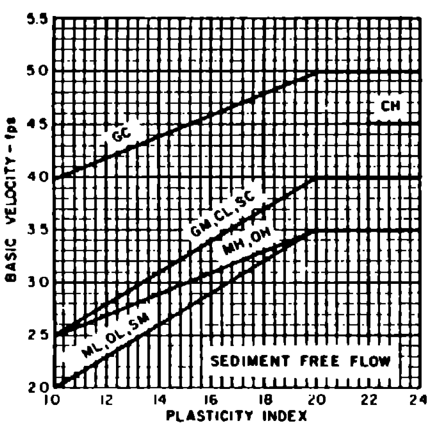


BASIC VELOCITY FOR DISCRETE PARTICLES OF EARTH MATERIALS (v_b)



NOTES:

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



BASIC VELOCITIES FOR COHERENT EARTH MATERIALS (v_b)

ALLOWABLE VELOCITIES FOR UNPROTECTED EARTH CHANNELS

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

Revised -

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:

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

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.

$V_b = 6.7$ 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

$V_a = V_bDB = (6.7)(1.22)(0.72)$ straight reaches

$= 5.88$ fps

$V_a = V_bDBA = (6.7)(1.22)(0.72)(0.89)$ curved reach

$= 5.24$ fps

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 fps (from Figure 6-2)

for the 2 percent chance flood

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

Density correction, compute e.

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

C_e = 1.0 (from Figure 6-2)

Frequency correction, F = 1.5 (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)

$V_a = V_b D C_e F$ Straight reach

$V_a = (3.7)(1.22)(1.0)(1.5) = 6.77$ fps

$V_a = V_b D C_e F A$ Curved reach

$V_a = (3.7)(1.22)(1.0)(1.5)(0.89) = 6.03$ 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.

$V_a = V_b D C_e$ Straight reaches

$V_a = (3.7)(1.19)(1.0) = 4.40$ fps

$V_a = V_b D C_e A$ Curved reaches

$V_a = (3.7)(1.19)(1.0)(0.90) = 3.96$ 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.

$V_b = 9.0$ fps (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-build 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_e = 0.97

Allowable velocity for straight channel = $V_a = V_b DAFC_e$
$V_a = (3.33)(1.23)(1.0)(1.0)(0.97) = 3.97$ fps

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

Allowable velocity for sharpest curve = $V_a = V_b DAFC_e$
$V_a = (3.33)(1.23)(0.75)(1.0)(0.97) = 2.98$ 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 $V_a = (V_b)$ DAB

V_b for sediment laden flow (20,000 ppm) = 5.3 ft/sec

V_b for sediment free flow (1,000 ppm) = 3.4 ft/sec

V_b 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
$V_a = V_b DAB = (3.5)(1.23)(1.0)(0.71) = 3.06$ fps

Allowable velocity for sharpest curve
$V_a = V_b$ DAB = (3.5)(1.25)(0.75)(0.71) = 2.29 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[25/] method is used. When fine grained soils are involved, a method derived from the work of Keulegan and modified by Einstein[27/], and Vanoni and Brooks[48/] 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 impedence to flow including grain roughness, form roughness, vegetation, curved alignment, etc. Lane's[25/] 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.

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

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

$$\tau_\infty = \gamma_w \, d s_t \qquad \text{(Eq. 6-4)}$$

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

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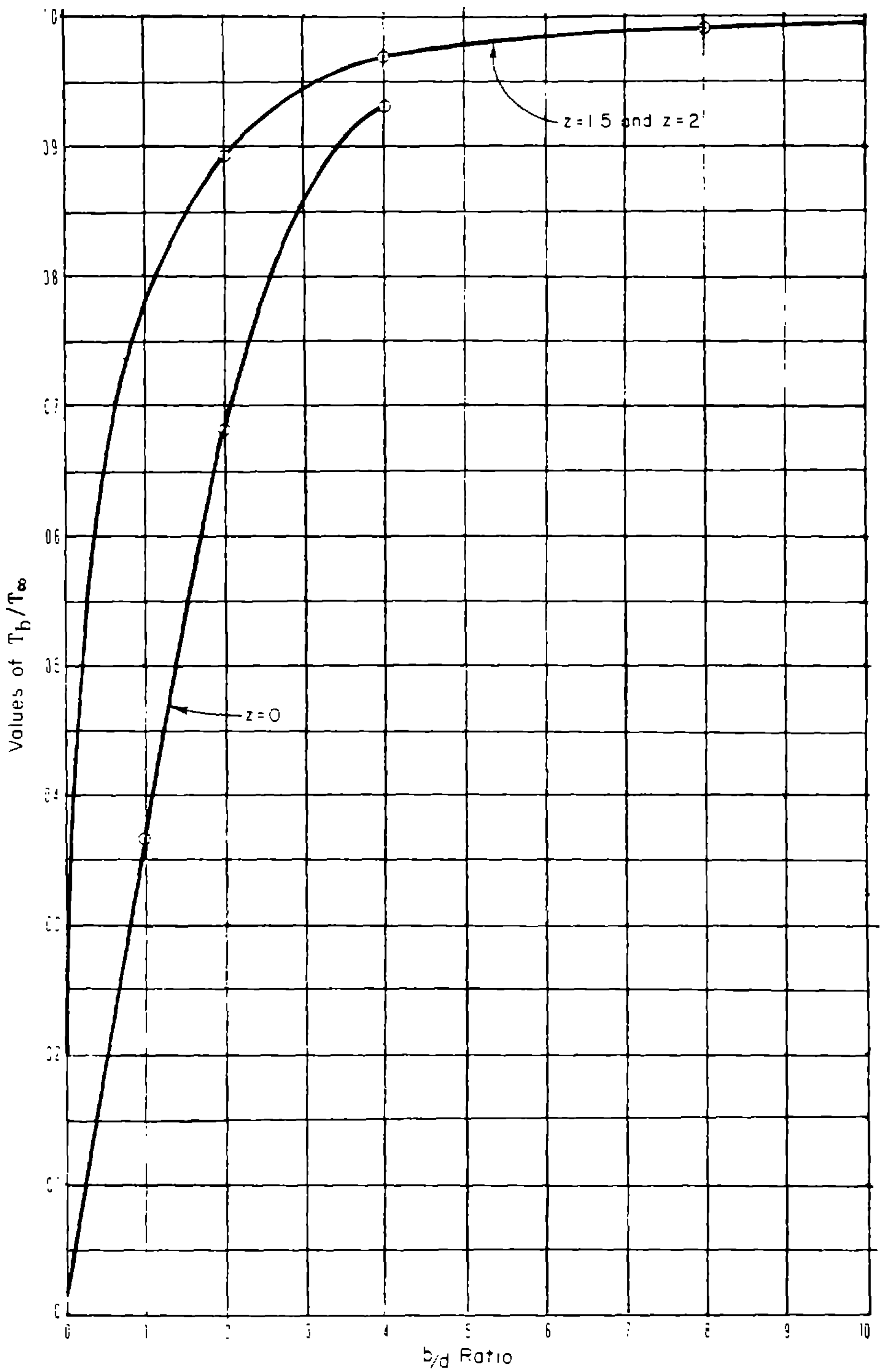
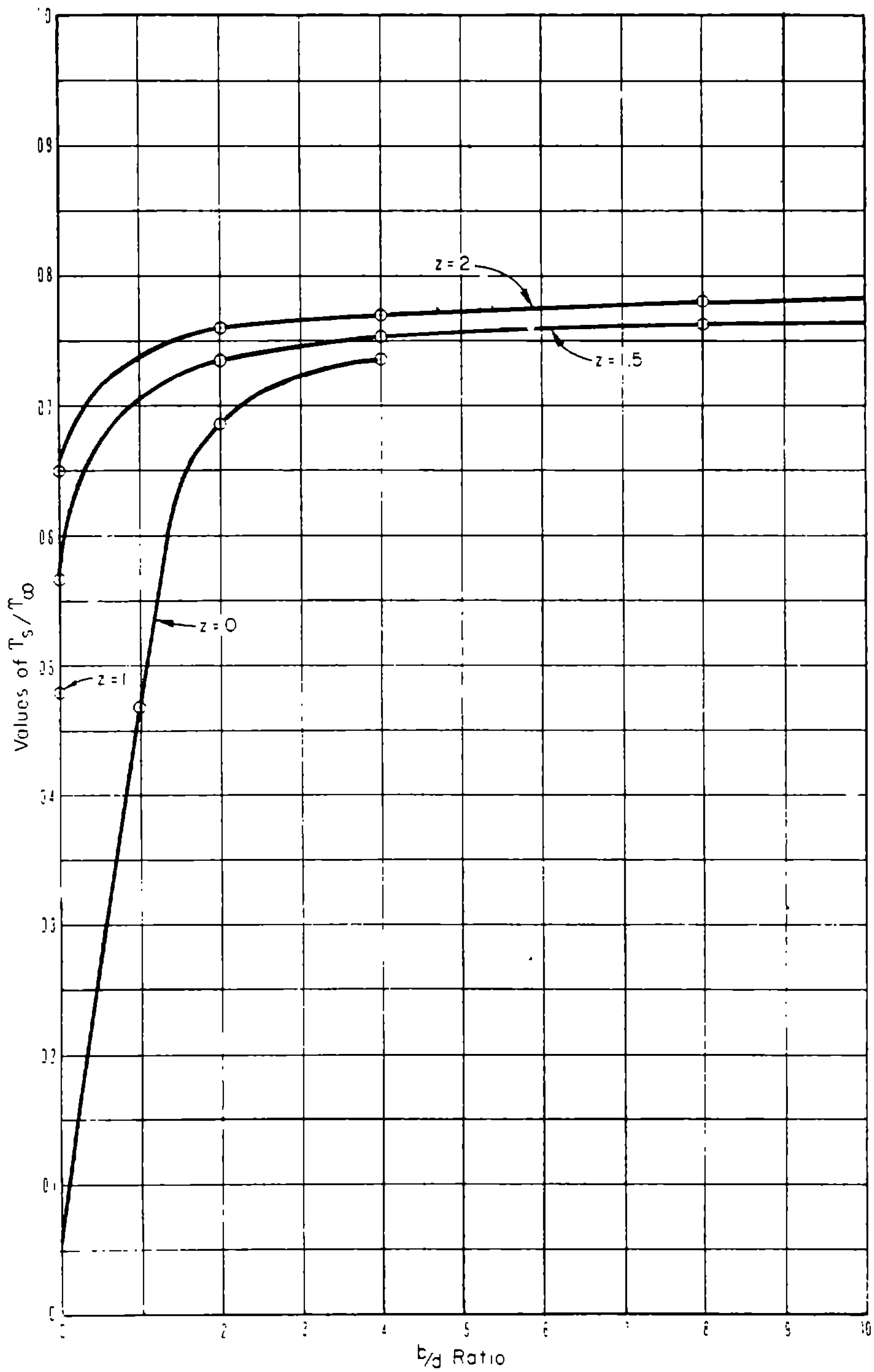
HYDRAULICS: Channel stability; actual maximum tractive stress, T_b, on bed of straight trapezoidal channels



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

FIGURE 6-3

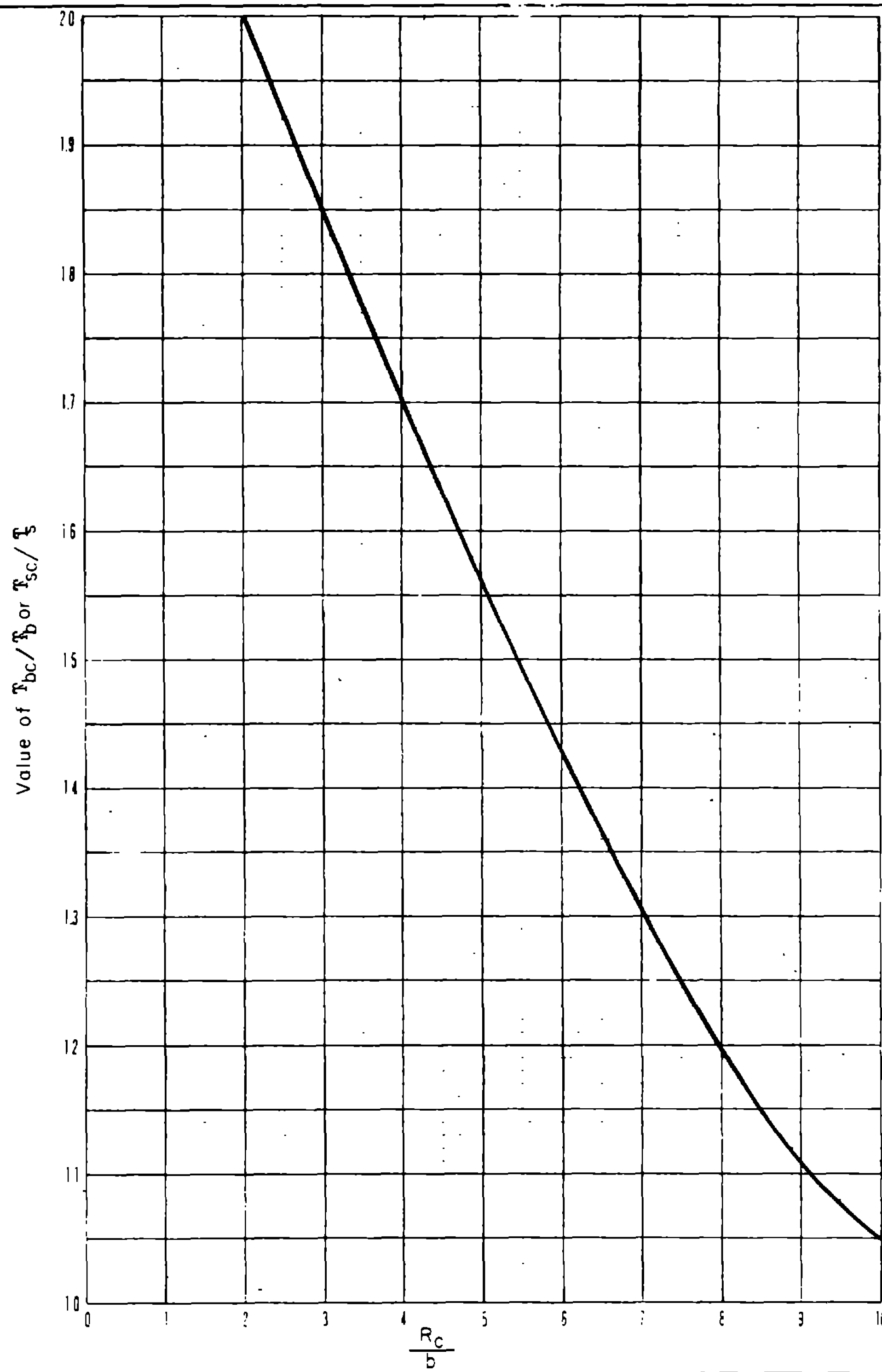
HYDRAULICS: Channel stability; actual maximum tractive stress, τ_s, on sides of straight trapezoidal channels



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

FIGURE 6-4

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



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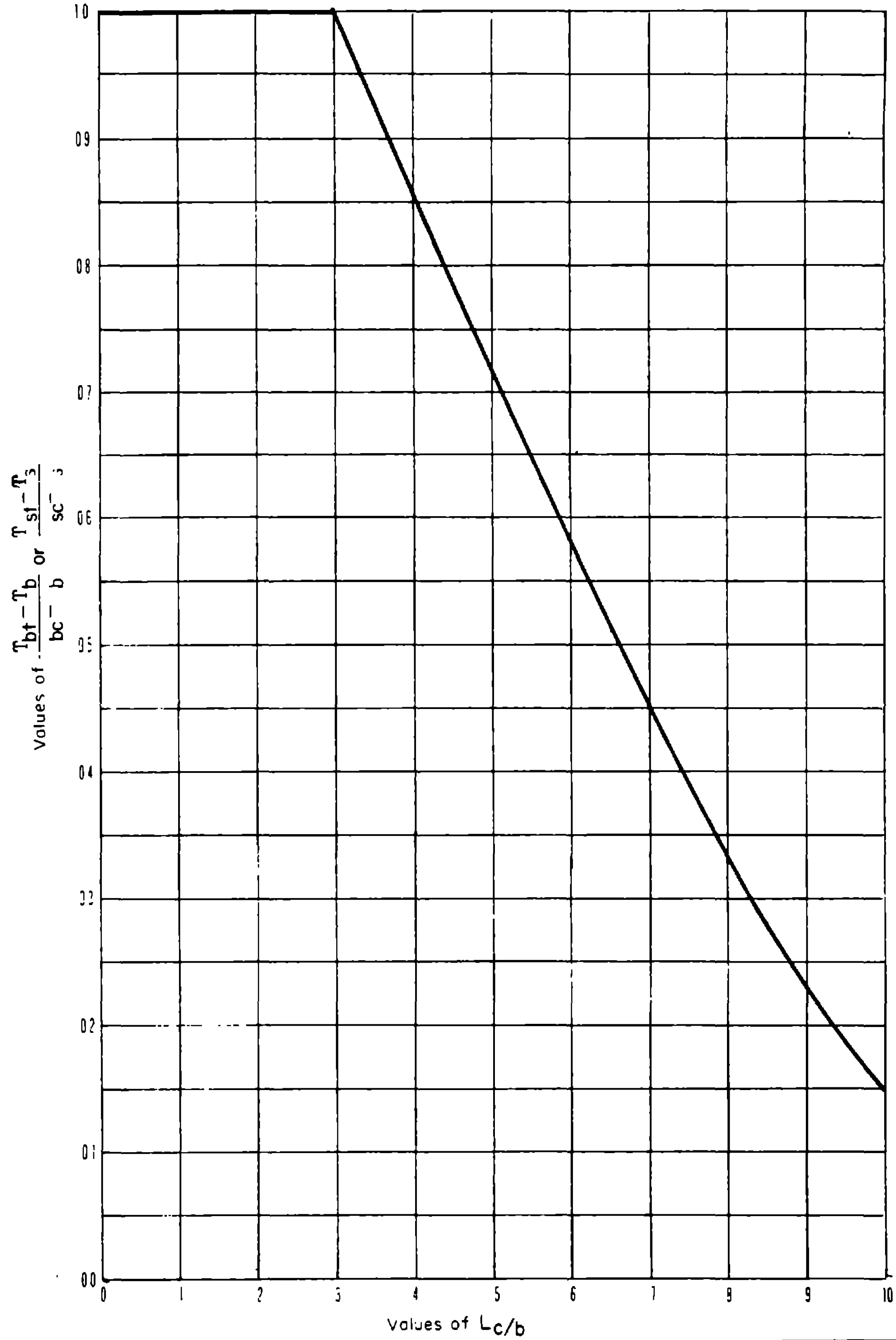
Nece, R.E., Givler, C.A., and Drinker, P.A., Measurement of Boundary Shear Stress in an Open Channel Curve with a Suface Pitot Tube. M.I.T. Tech note No. 6, August 1959

FIGURE 6-5

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



REFERENCE
Nece, R.E., Givler, 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, τ_{Lb}, composed of soil particles with discrete, single grain behavior with a given D_{75} is:

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

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

The allowable tractive stress for channel sides τ_{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 ϕ_R with the horizontal is

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

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

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 γ_s of the constituents of the material having a grain size larger than the D_{75} size is significantly different than 160 lb/ft^3, the limiting tractive stress τ_{Lb} and τ_{Ls} as given by equations (6-5) and (6-6) should be multiplied by the factor.

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

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

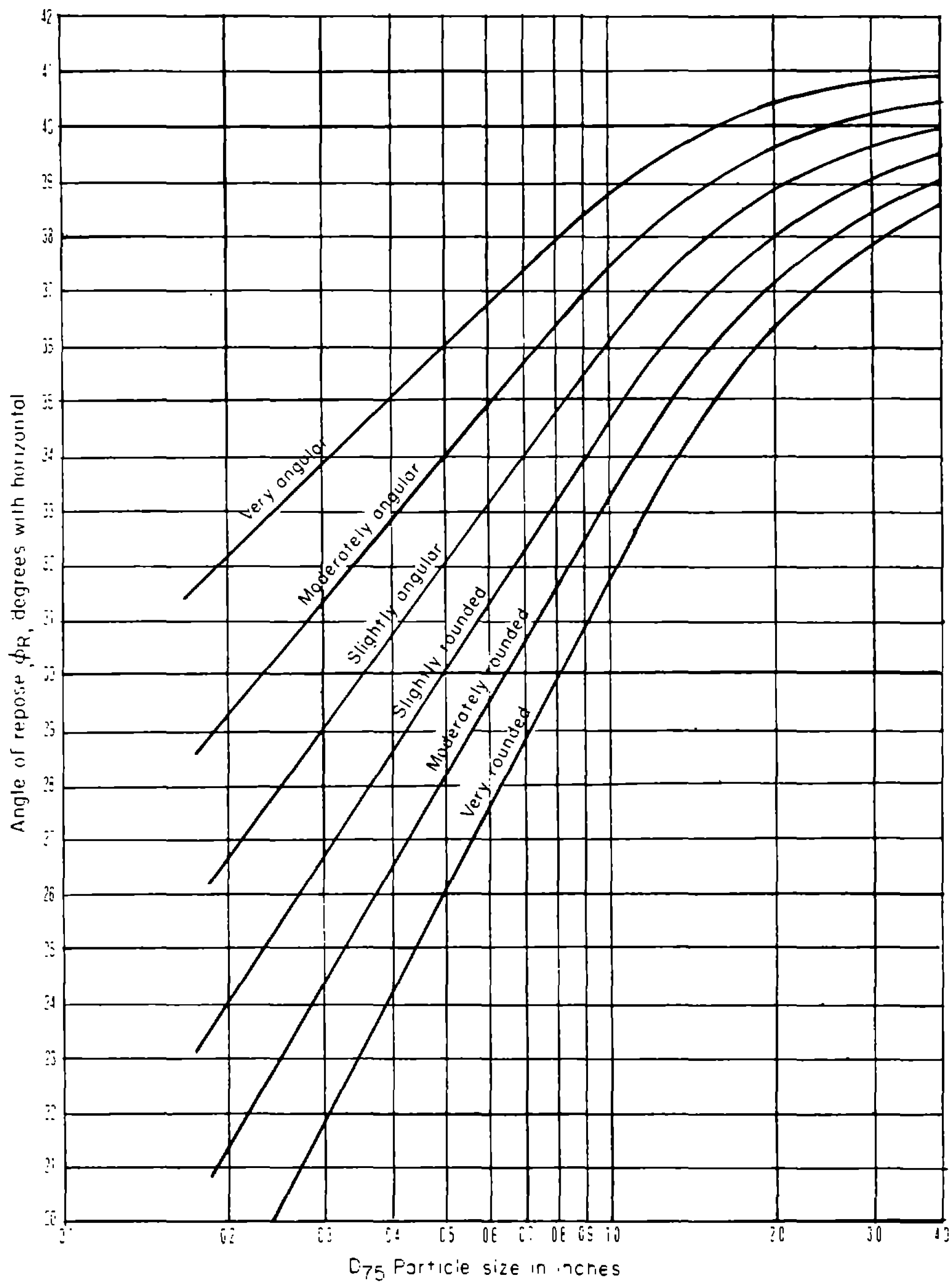
A. Determination of Actual Tractive Stress

1. Reference tractive stress

The expression for the reference tractive stress is:

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

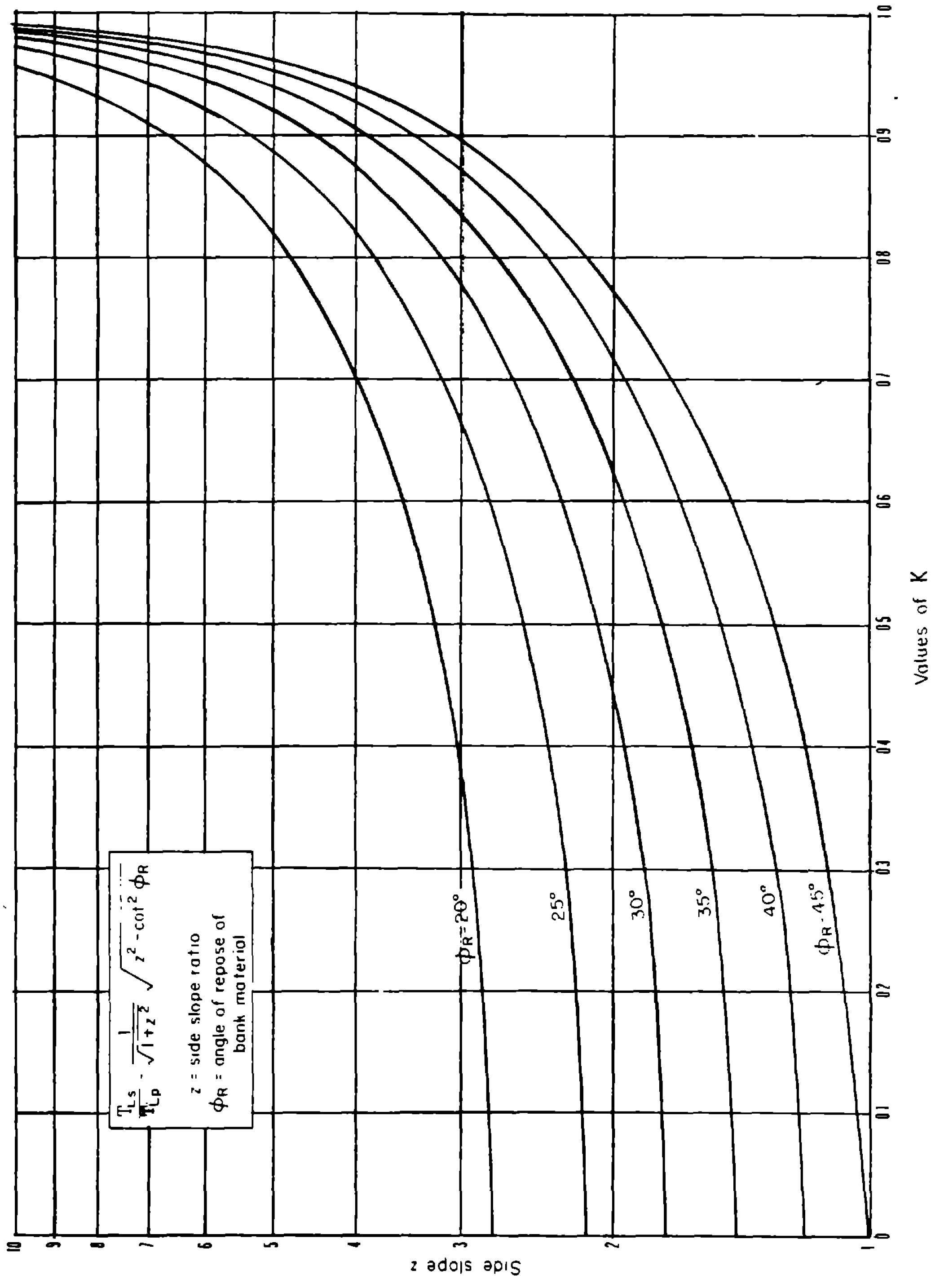
HYDRAULICS: Channel stability; angle of respose, ϕ_R, for non-cohesive materials



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

FIGURE 6-7

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



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

FIGURE 6-8

In a given situation γ 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.[27]

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

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 δ. This thickness is determined from the equation

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

A relationship between x and k_s/δ has been developed empirically by Einstein[27] 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[48] 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 ρ and kinematic viscosity of the water ν can be obtained.

The computation of reference tractive stress (τ) 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 τ, the reference tractive stress, to obtain values for the maximum tractive stress on the sides and bed of trapezoidal channels in fine grained soils.

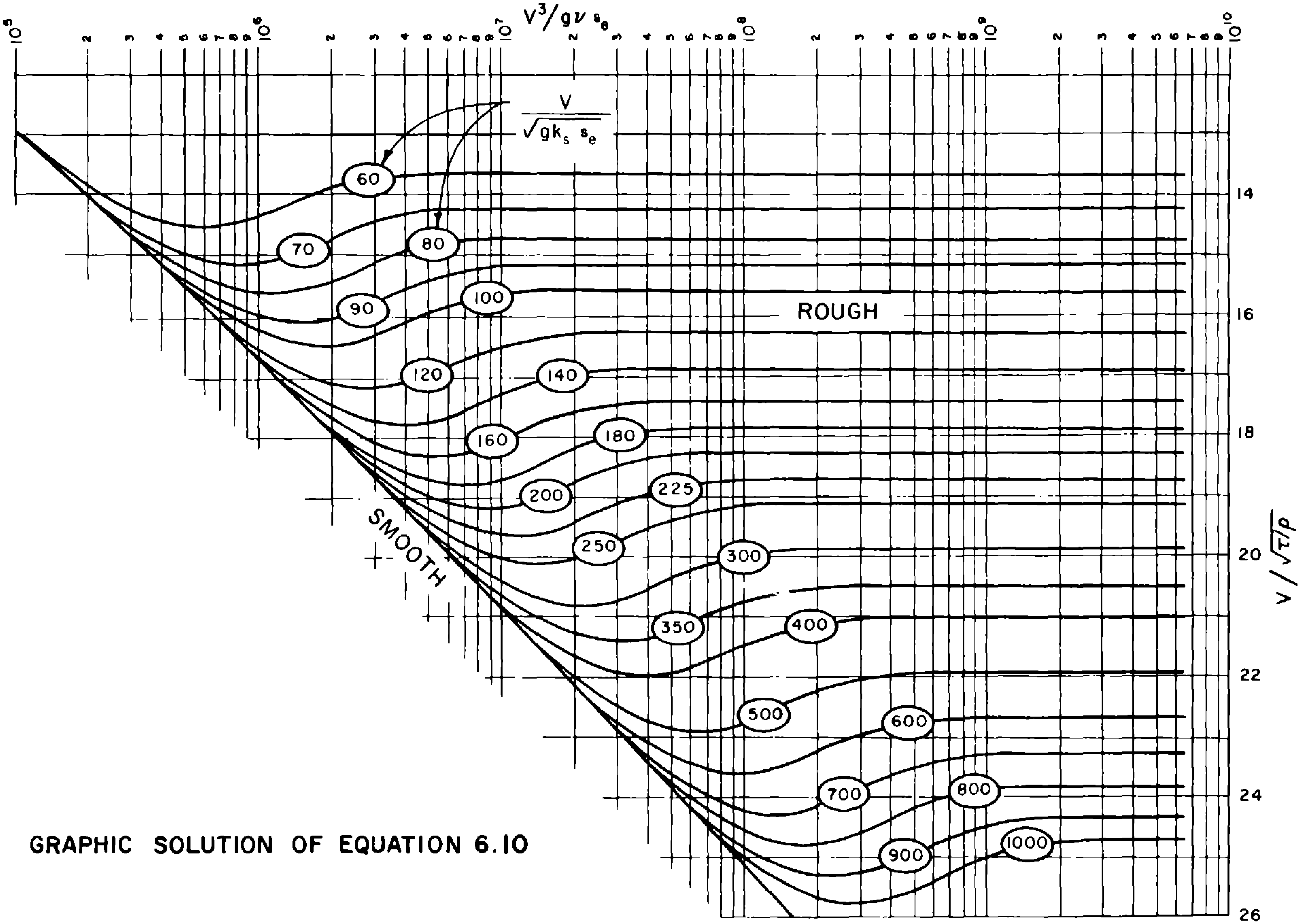
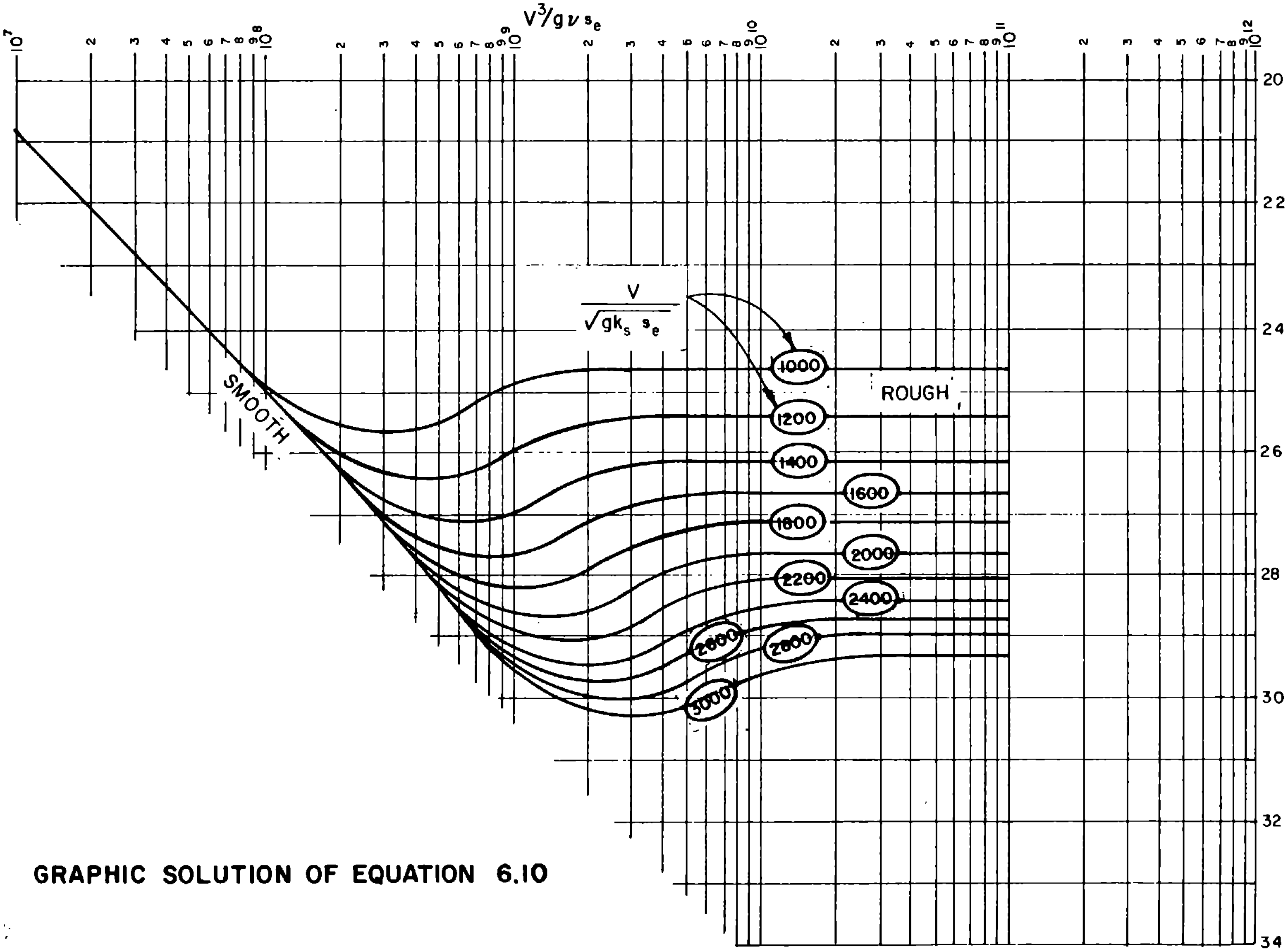


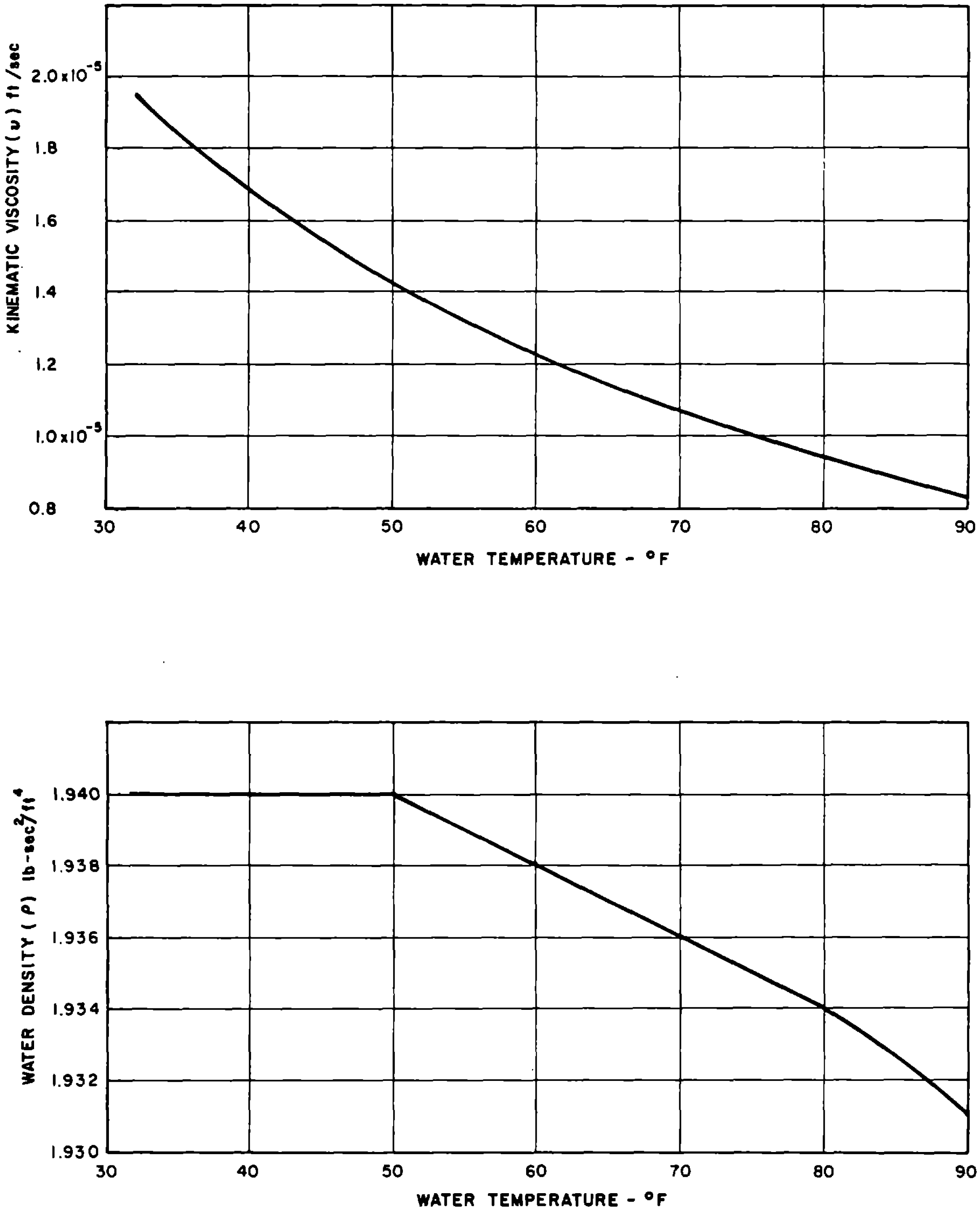
FIGURE 6-9

GRAPHIC SOLUTION OF EQUATION 6.10



GRAPHIC SOLUTION OF EQUATION 6.10

FIGURE 6-10



Values of ρ and ν for various water temperatures

FIGURE 6-11

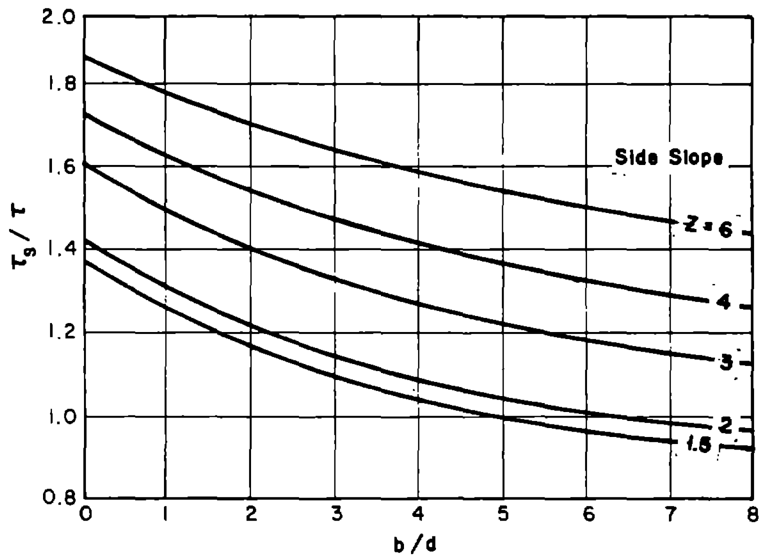


FIGURE 6-12 : Applied Maximum Tractive Stresses, τ_s, On Sides Of Straight Trapezoidal Channels.

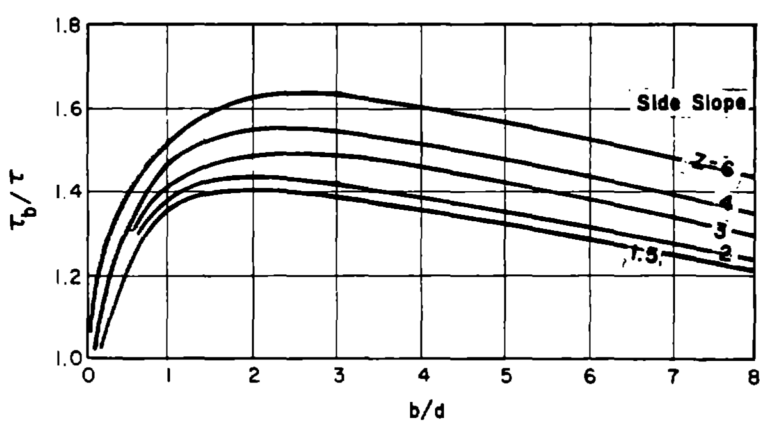


FIGURE 6-13 : Applied Maximum Tractive Stresses, τ_b, On Bed Of Straight Trapezoidal Channels.

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

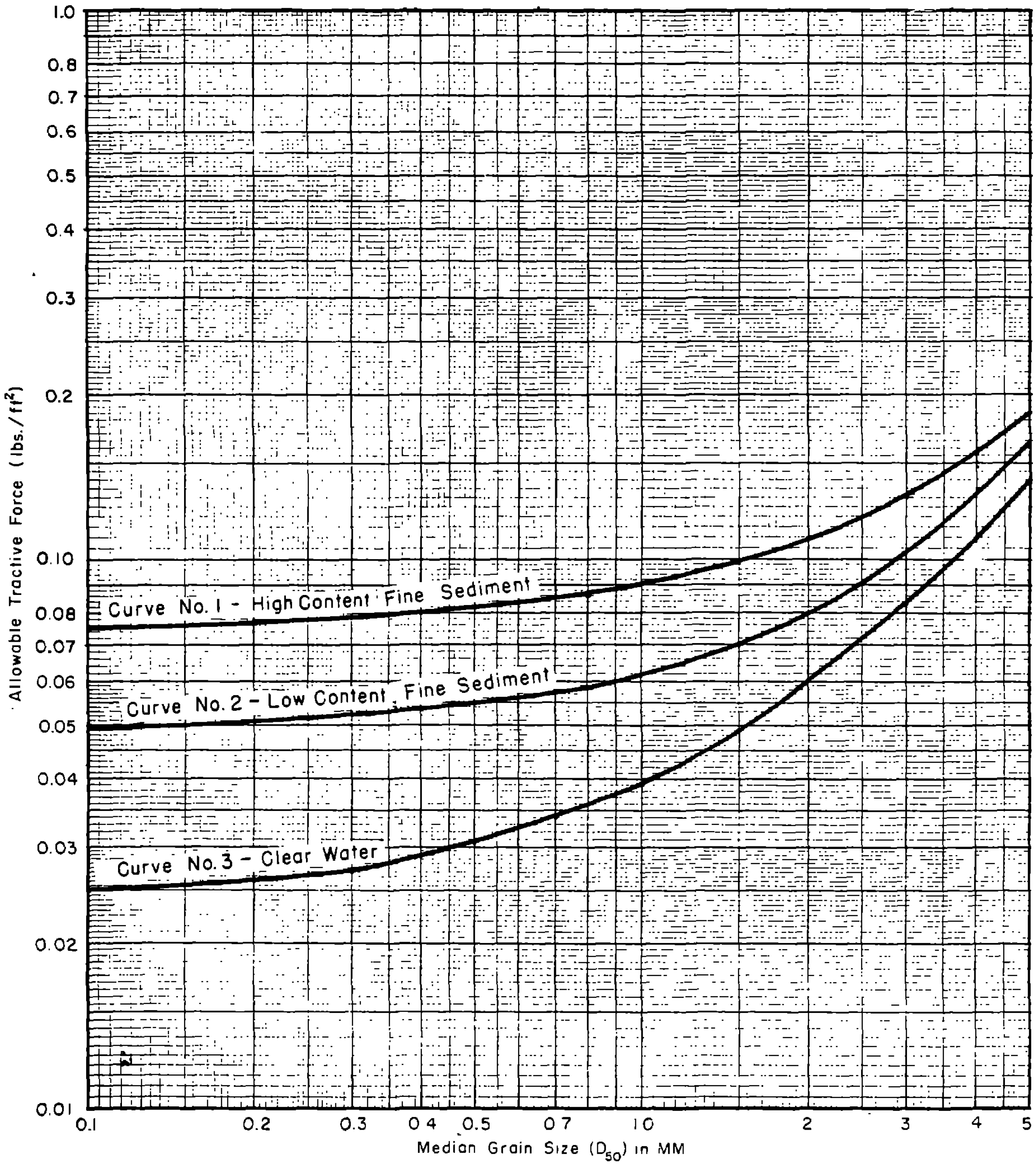


FIGURE 6-14

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[25/]. 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 °F and read the density ρ and the kinematic viscosity of the water ν.

3. Compute $\dfrac{V^3}{g\nu s_e}$.

4. Compute $\dfrac{V}{\sqrt{g k_s s_e}}$.

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 $\dfrac{V}{\sqrt{\tau/\rho}}$.

6. Compute τ from $\dfrac{V}{\sqrt{\tau/\rho}}$, V and ρ

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

where the terms are defined in the glossary.

<u>Examples - Tractive Stress Approach</u>

<u>Example 6-5</u>

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.

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

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

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

b/d (ratio of bottom width to depth) = 18/3.5 = 5.14

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

R_c/b (radius of curve/bottom width) = 150/18 = 8.33

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

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

actual $\tau_s = (0.179)(0.76) = 0.136$ psf

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

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

Solving for allowable tractive stress -

$\phi_R = 38.4°$ (From Figure 6-7) K = 0.45 (From Figure 6-8)

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

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.

247-927 O - 78 - 2

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

$V^3/g\nu s_e = 3.23^3/((32.2)(1.42 \times 10^{-5})(0.0026)) = 2.83 \times 10^7$

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

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

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

b/d (ratio of bottom width to depth) = 18/3.5 = 5.14

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

Actual Tractive Stresses:

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

Allowable Tractive Stresses:

$D_{50} = 0.127$ 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

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

For example

$\tau_b = 0.6$ psf

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

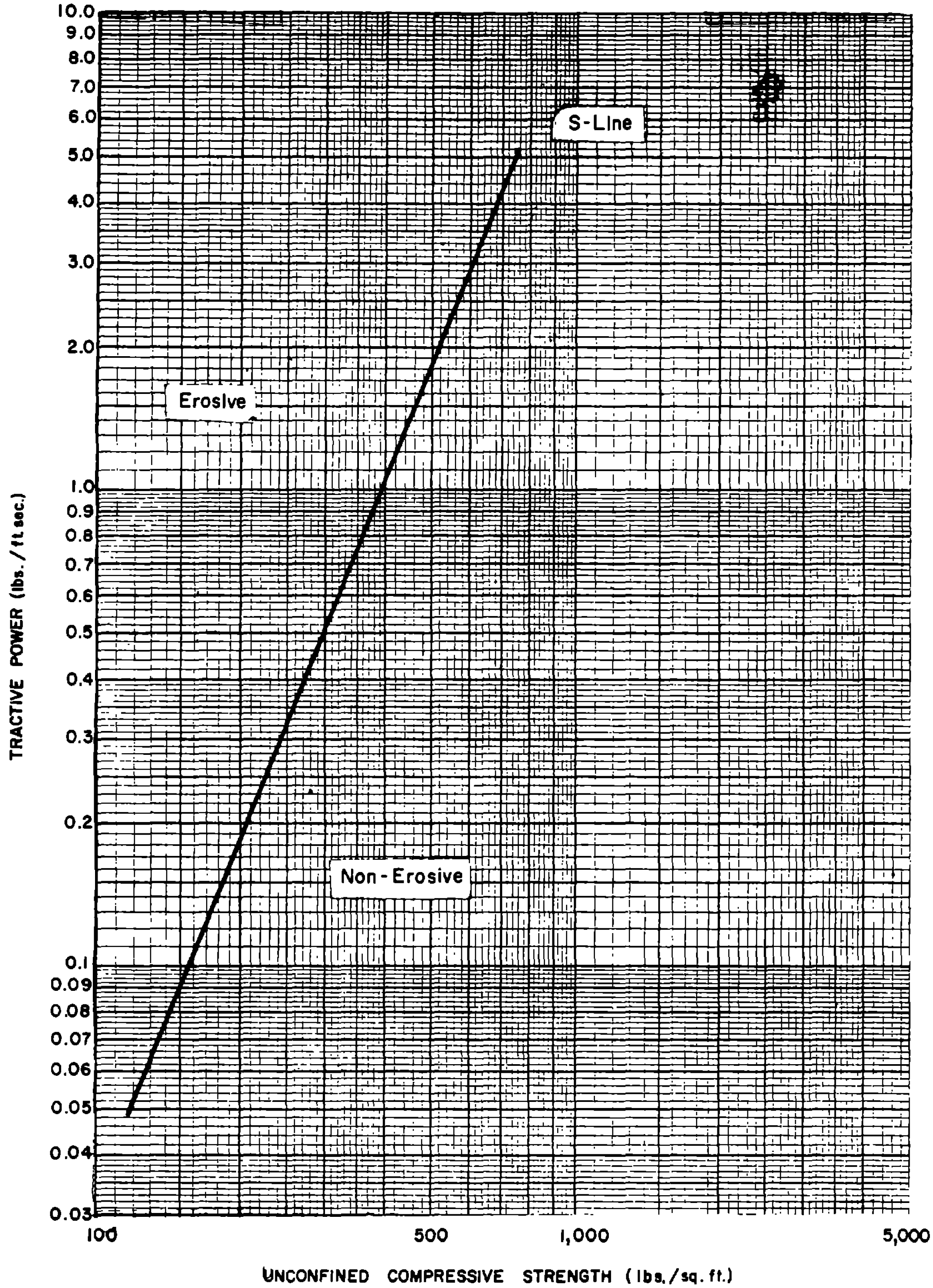


Figure 6-15
Unconfined Compressive Strength And Tractive Power As Related To Channel Stability

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.

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

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

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

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

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

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

$\tau_b/\tau = 1.45$ Figure 6-13

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

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

Use the larger of τ_s or τ_b to compute tractive power.

$\tau_b V = (0.0548)(3.77) = 0.207$ - 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.

The Modified Regime Approach

General

The regime theory approach to evaluating channel stability is based on observations of the results in various parts of the world of natural processes causing continuous adjustments of channels. The predictive equations are largely empirical. This method of analysis is limited to flow in alluvial channels.

Chien[49] defines an alluvial channel as one that contains a bed of loose sediment of the same type that is moved along the bed. Such a channel bed seldom remains flat and even. Bars and ripples are developed at the bed surfaces at low stages. They become longer when the discharge increases and eventually may disappear at high flows. At unusually high flows, large, nearly symmetrical, sand bars may appear again, accompanied by surface waves in phase with the bottom undulations. The sand bars and ripples represent another type of roughness, in addition to the roughness of the grains which compose the channel bed. The problem of determining the relationship between slope, depth, velocity, and boundary roughness is complicated by this phenomenon because the roughness not only defines the flow, but the flow itself also molds the roughness.

Blench[50] refers to alluvial channels as those with mobile boundaries. They are the channels that are capable of self-adjustment and have formed their geometric shape by moving boundary material. Materials of at least part of the boundary are moved at some stage of flow. They make at least part of their boundaries from their transported load, and part of their transported load comes from their boundaries.

The equations used in the regime approach have been developed by studying statistics obtained by physical observations of canal systems. Those observations included channel dimension and geometry, and the discharges of streams that were "silt stable," that is, canals that through a succession of years remained free of excessive sediment deposits and did not scour excessively. These equations empirically correlate the capacity of the stream to transport sediment with its main hydraulic characteristics. A "silt stable" or regime stream is known to deposit sediment throughout some stage of flow and to scour during other stages. The proponents of the regime theory approach are satisfied if the net result of deposit and scour is zero at the end of every flow cycle.

When nature or man imposes rigid boundaries in a channel system, the natural laws of alluvial flow are partially or totally negated. Consequently the regime approach cannot be used to analyze rigid boundary channel systems. However, they can be used to determine channel proportions such that the channel can be expected to remain relatively stable.

The procedure and equations presented here are to a large extent from Simons and Albertson.[51]

Three types of mobile boundary materials encompass most, alluvial channels encountered in SCS work. These are: (Note- for this approach a soil is classed as cohesive if the PI is greater than 7.)

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

A relationship exists between sediment load, Froude number, and channel stability. The Froude number is determined by:

$$F = \frac{V}{\sqrt{g\ d}} \qquad \text{(Eq. 6-12)}$$

According to Simons and Albertson[51/] the Froude number has to be less than 0.3 in type A, B, or C channels to avoid excessive scour.

The relationships between channel geometry and slope are determined by the following regime equations as modified by Simons and Albertson:

$$d = 1.23\ R \quad \text{(For R from 1 to 7)} \qquad \text{(Eq. 6-13)}$$

$$d = 2.11 + 0.934\ R \quad \text{(For R from 7 to 12)} \qquad \text{(Eq. 6-14)}$$

$$W = 0.9\ P \qquad \text{(Eq. 6-15)}$$

$$W = 0.92\ W_T - 2.0 \qquad \text{(Eq. 6-16)}$$

Equations 6-13, 6-14, 6-15, and 6-16 apply to all Type A, B, and C alluvial channels.

Equations 6-17, 6-18, 6-19, 6-20, and 6-21 were derived from data in the Simons and Albertson paper.

Equation	Coefficients by Channel Type			
	A	B	C	
$P = C_1 Q^{0.512}$	3.30	2.51	2.12	(Eq. 6-17)
$R = C_2 Q^{0.361}$	0.37	0.43	0.51	(Eq. 6-18)
$A = C_3 Q^{0.873}$	1.22	1.08	1.08	(Eq. 6-19)
$V = C_4 (R^2S)^{1/3}$	13.9	16.1	16.0	(Eq. 6-20)
$\frac{W}{d} = C_5 Q^{0.151}$	6.5	4.3	3.0	(Eq. 6-21)

where all symbols are as defined in the glossary.

These equations result in some general relationships, approximate width-depth ratios and velocity limitations that if followed will result in "silt stable" channels under the conditions described.

Determination of an acceptable safe slope for a channel is about the most difficult decision in channel design. Values of the slope determined from Manning's equation with a reasonable value of n, cross section geometry consistent with the modified regime equations, and a velocity resulting in a Froude number of less than 0.3 should be compared with the slope determined from equation 6-20.

Procedure - Modified Regime Approach

The use of the modified regime theory to evaluate 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 character of the earth materials forming the banks and bed of the design reach and the reach upstream.

3. Evaluate the sediment transport carrying capacity in the design reach to (a) determine the sediment concentration and (b) test the possibility for aggradation.

4. Check to see if the modified regime approach is applicable. (Use Figure 6-1)

5. Determine the channel geometry and acceptable safe slope using equations 6-12 through 6-20 with the appropriate constants.

6. Check the slope determined using Manning's equation with a realistic value of n, cross section geometry consistent with that determined in step 5 and a velocity resulting in a Froude number of less than 0.3.

Example 6-8 - Modified Regime Approach

Given: A type B channel to convey 600 cfs at bank full stage. Use 2:1 side slopes and assume that n = 0.022.

Determine: Design the channel.

Solution: Step 1 - Compute $P = 2.51Q^{0.512}$ . . . . Eq. 6-17

$P = (2.51)(600)^{0.512} = 66.4$ ft.

Step 2 - Compute $R = 0.43Q^{0.361}$ . . . . Eq. 6-18

$R = (0.43)(600)^{0.361} = 4.33$ ft.

Step 3 - Compute A = PR or use Eq. 6-19

$A = (66.4)(4.33) = 288$ sq. ft.

Step 4 - Compute $V = Q \div A = 600 \div 288 = 2.08$ fps

Step 5 - Compute d - Since R is less than 7

Use $d = 1.23R$ . . . . Eq. 6-13

$d = 1.23\ (4.33) = 5.3$ ft.

Step 6 - Compute the Froude Number.

$$F = \frac{V}{\sqrt{gd}}$$ . . . . Eq. 6-12

$$= \frac{2.08}{\sqrt{32.2(5.3)}} = 0.159$$

F < 0.3 - Design meets this requirement for stability

Step 7 - Compute bottom width

$W = 0.9\ P$ . . . . Eq. 6-15

$W = 0.9(66.4) = 59.76$ ft.

$W = 0.92\ W_T - 2.0$ . . . . Eq. 6-16

$59.76 = 0.92\ W_T - 2.0$

$W_T = 67.1$ ft.

For 2:1 side slopes-

$b = 67.1 - (4)(5.3) = 45.9$ ft.

Use $b = 45$ ft.

Step 8 - Find the slope of the channel bottom which is needed to cause the channel to be in regime.

$V = C_4\ (R^2 s_o)^{1/3}$ . . . . Eq. 6-20

$2.08 = 16.1\ ((4.33)^2 s_o)^{1/3}$

$s_o = 0.000114$

Step 9 - Find the slope of the channel which is needed to provide capacity assuming uniform flow and Manning's equation. Compute $AR^{2/3}$ using the above values for depth, side slope, and bottom width.

$P = b + 2d\sqrt{z^2 + 1} = 45 + 2(5.3)\sqrt{5} = 68.7$ ft.

$A = bd + zd^2 = 45\ (5.3) + 2(5.3)^2 = 294.7$ ft.^{2}

$$R = \frac{A}{P} = \frac{294.7}{68.7} = 4.29 \text{ ft.}$$

$AR^{2/3} = (294.7)(4.29)^{2/3} = 778$

Compute s_o

$$Q = \frac{1.486}{n}\ AR^{2/3}\ s_o^{1/2}$$

$$600 = \frac{1.486}{0.022}\ (778) s_o^{1/2}$$

$s_o = 0.00013$

Step 10 - Select slope to be used.

Since two values for the slope have been determined it is necessary to choose a slope that falls between the two values. If a slope flatter than 0.00013 is selected either the width or depth must be increased to provide the needed capacity. In any case the channel will not match the regime relationships exactly but the two slope values are sufficiently close so that the design should be satisfactory. Use the following parameters:

$s_o = 0.00013$

$d = 5.3$ ft.

$b = 45.0$ ft.

Equation 6-21 could have been used to get an idea of what a reasonable width to depth relationship would be from regime methods.

$$W/d = 4.3Q^{0.151}$$

$$W/d = (4.3)(600)^{0.151} = 11.3$$

The same ratio would be obtained by dividing the value for W obtained in Step 7 by the depth obtained in Step 5.

$$\frac{W}{d} = \frac{59.76}{5.3} = 11.3$$

Channel Stability With Respect to Sediment Transport

A channel transporting sediment during flow is considered to be stable if the rate of sediment transport is such that the overall equilibrium of the channel is maintained. This requires that scour and aggradation are maintained between prescribed limits. Bedload transport equations have been developed for predicting the rate of transport under equilibrium conditions. In these equations transport is related to stream discharge per foot of channel width. A procedure is presented in this section for determiningn relative rates of scour or deposition using variations in mean velocity.

Application of Bedload Transport Equations

A number of equations have been developed to compute rates of bedload sediment transport. The more widely used include the Einstein Bedload Function[27/], the Meyer-Peter and Muller formula[28/], and the Schoklitsch[29/] equations. A comparison of the measured and computed sediment loads indicates that the most reliable involves depth-integrated samples of suspended load and computations employing the Einstein bedload function. This is known as the

Modified Einstein Procedure[30]. However, the field data required in use of this procedure are not ordinarily available.

The Einstein bedload function and the Meyer-Peter and Muller formula for computing bedload transport have been determined to be about equally adapted for this purpose in the range from medium-size sand to gravel. The equations for computation of equilibrium bedload transport are given in the references cited.

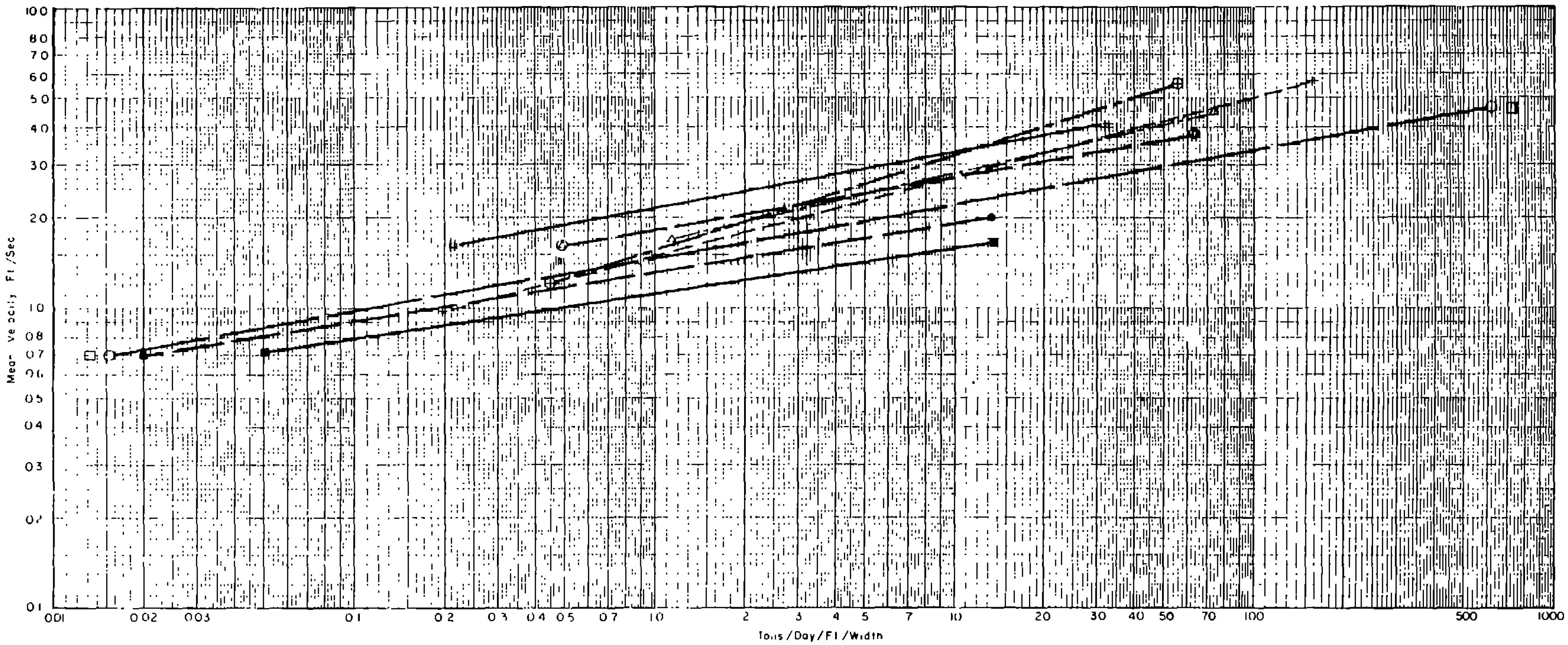
Sediment Transport in Sand Bed Streams Not in Equilibrium

The procedures described in this section are recommended for determining the effect of channel changes on stability. They are based on research which shows that the rate of bedload sediment transport is strongly related to mean velocity. Figure 6-16 shows this relationship for fine and medium sand sizes. Factors which create differences in mean velocity from one reach to another cause differences in rates of bedload transport. If the changes in rates are substantial in amount and in duration, an unstable condition is established.

Bedload sediment transport in sand bed streams with variable roughness. Numerous studies have indicated that the roughness coefficient n varies in a sandbed stream as the bed form changes in response to the formation of ripples, dunes and anti-dunes. No generally accepted method has been developed for predicting what the n value will be at any given discharge or velocity. In the approach used here, mean velocity is related to an approximation of tractive force for broad, shallow flow - - the product of depth, slope and unit weight of water. The relationship is established by adaptation of data presented by Dawdy[31]. In his paper, the hydraulic radii as related to mean velocity are shown for a number of sand bed streams. Figure 6-17 shows a plotting of mean velocity related to the product of slope, hydraulic radius and unit weight of water for five of the streams. It is assumed that hydraulic radii in these relatively broad, flat-bedded streams are equivalent to depth for purposes of computing tractive force in this procedure. More data are needed to define the curve of Figure 6-17 for sediment with the median size coarser than 0.5 mm. Data from one stream with a median sand size of 0.8 mm. indicates a deviation from this curve.

Procedure- Channel Stability with respect to Sediment Transport

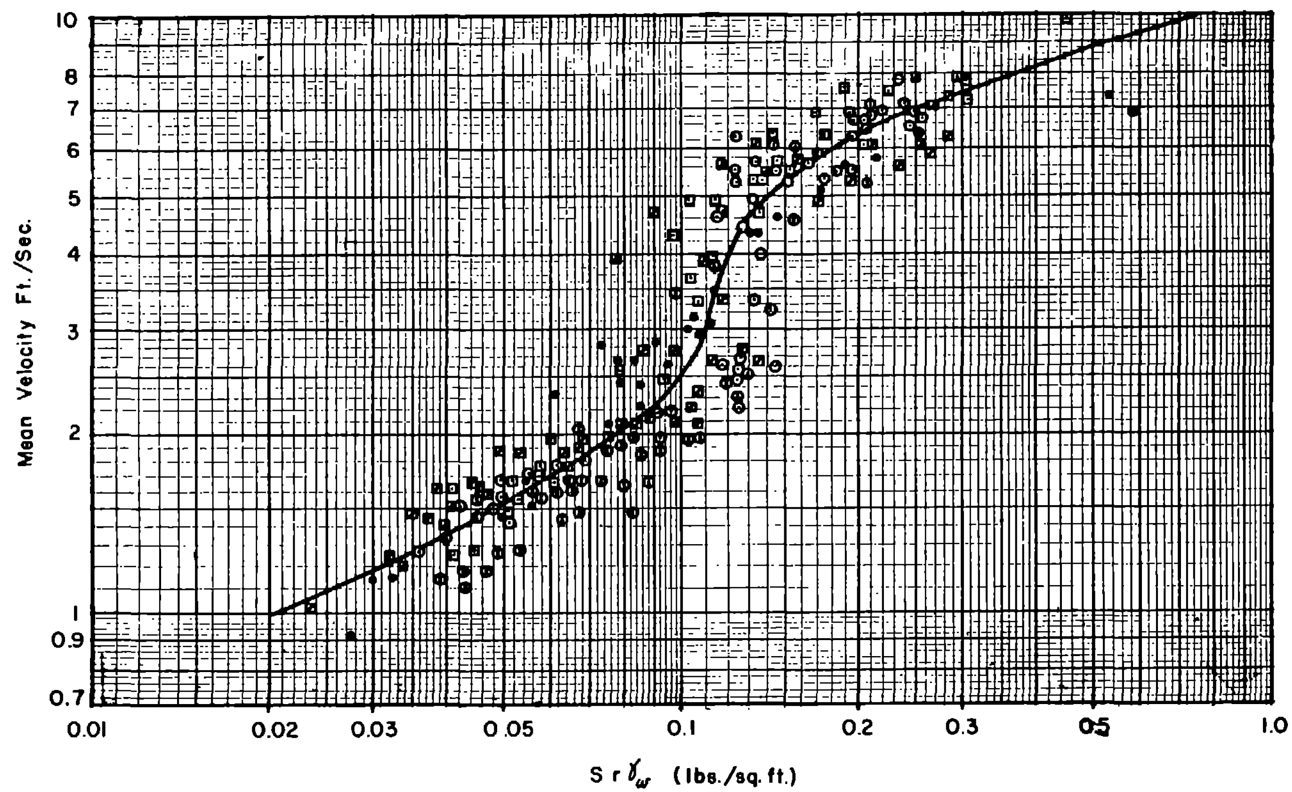
The following procedure may be used to determine whether unstable conditions will occur under projected channel conditions where variable bed roughness occurs:



NO.	D_{50} SIZE	MEASURING POINT	WATER DEPTH	REFERENCE
1	○ 0 45 mm	SAND FLUME	0 27'- 1 0'	USGS WS PAPER 1488-A 61
2	△ 0 594 mm	SAND-FLUME	0 074' 0 346'	CAL TECH REPORT NO KHR 2 JAN 61
3	⊕ 0 233 mm	SAND-FLUME	0 145'-0 356'	CAL TECH REPORT NO KHR 2 JAN 61
4	□ 0 145 mm	SAND-FLUME	0 240'-0 242'	CAL TECH REPORT NO E-68 DEC 57
5	■ 0 137 mm	SAND-FLUME	0 203'-0 302'	CAL TECH REPORT NO E-68 DEC 57
6	● 0 137 mm	SAND FLUME	0 524'-0 553'	CAL TECH REPORT NO E-68 DEC 57
7	ǂ 0 130 mm	RIO GRANDE AT BERNALILLO		USBR SED CONF PAPER NO 32 DEC 63
8	# 0 27 mm	COLORADO R AT ADOBE		USBR-SED CONF PAPER NO 32 DEC 63
9	⊕	AVE CURVE - FIVE MILE CR WY NIOBRARA R MIDDLE LOUP, NEB		TRANS AGU VOL 38 NO 5 OCT 57

Figure 6-16

RELATIONSHIP BETWEEN MEAN VELOCITY AND SEDIMENT TRANSPORT ON AND NEAR STREAM BED



⊠	0.40 mm	SAND	PIGEON ROOST CR., MISS.
⦶	0.32 mm	SAND	REPUBLICAN R., NEB.
⊡	0.26 mm	SAND	MIDDLE LOUP R., NEB.
⊙	0.30 mm	SAND	RIO GRANDE R., N.M.
•	0.50 mm	SAND	SOUTH FK. POWDER R., WYO.

FIGURE 6-17

RELATION OF MEAN VELOCITY TO PRODUCT OF SLOPE HYRAULIC RADIUS AND UNIT WEIGHT OF WATER

FINE AND MEDIUM SIZE SAND BED STREAMS DATA ADOPTED FROM U.S. GEOL. SURVEY WATER SUPPLY PAPER 1498 C, FIG. 5, 6, 8, 9, 10

1. Determine whether a full supply of bedload will be introduced into the reach by methods described in Chapter 3, NEH Section 3 - Chapter 4 and Geologic Note 2.

2. Compute mean velocities for various stages of flow for a hydrograph or series of hydrographs at cross sections typical of stream reaches to be compared. The recommended method of determining the influence of variable bed roughness and bank roughness on mean velocity is explained in the example of the procedure given below.

3. Select one characteristic velocity-bedload transport curve from Figure 6-16 or construct a new one from available data.

4. Compute rates of bedload transport for each reach.

5. Where scour or aggradation may occur, revise design, such as changing projected channel slope, width and depth. The design may have to provide sufficient channel freeboard for low flow aggradation to insure capacity during large flows. Reliance on the removal of the low flow deposits prior to peaking of higher flows should be approached with caution since the sequence of flows or the condition of the channel at the time of any flood occurrence cannot be predicted.

Example 6-9

Assume that a flood detention reservoir will be built on a sand bed stream with a median size of bed material of 0.15 mm. With the reservoir installed, improvement of the channel two miles downstream will be required to allow controlled runoff without erosion of the banks. The distance from the dam to the beginning of the reach to be improved is great enough to enable the flow to become fully loaded with bed material. The energy gradient in the unimproved reach is 0.003 feet per foot. The stream banks are nearly vertical and free of vegetation.

Rights-of-way limitations show that a 60-foot bottom width improved channel meets requirements. It is proposed to protect erodible banks with riprap on a slope of 2-1/2:1. The energy gradient in this reach is computed to be 0.0025.

The riprapped slopes and the narrower and lower gradient section result in a change in velocity over that of the upstream section. Table 6-1 and its supplement, Table 6-2, show the procedure used in computing these velocities. The formula in Table 6-2 was originally presented by Horton[32]. The computation of the values in each of the columns is as follows.

Column 1 - Depths are chosen to provide a range of flows within the two channel sections up to the maximum proposed reservoir release rate.

Columns 2 and 3 (80-foot section), and Columns 6 and 7 (60-foot section) give the cross-sectional area at the specified depths and side slopes. These data were obtained from hydraulic tables such as those prepared by the Corps of Engineers.

Columns 4 and 8 are approximations of tractive force using the energy gradients equal to 0.003 or 0.0025, respectively; depth (column 1), and the unit weight of water, 62.4 pounds per cubic foot.

Columns 5 and 9 are obtained from Figure 6-17, the mean velocities being read from the intersection of the product values in columns 4 and 8 with the curve.

The remaining calculations determine the correction of velocity of the 60-foot section due to riprap. Velocities that are attributable to the depth, energy gradient and bed roughness only are reflected in the velocities in Columns 5 and 9. Column 10 shows the n values related to the velocities in Column 9. In this example the n for the velocities in Column 9 were obtained from the Table of Values of nv Corresponding to Different Values of R (radius) and s (slope) in Manning's Formula in Kings "Handbook of Hydraulics."[33] These n values are used in the formula given at the head of Table 6-2 for computing roughness of the improved channel, accounting for both riprap and bottom roughness.

The method of obtaining the n in Column 11, corrected for roughness due to the riprapped side slopes, is given in Table 6-2.

The corrected mean velocity of Column 12 is determined from Manning's formula, using corrected n of Column 11, an energy gradient of 0.0025 and the appropriate R in Column 6.

The available reservoir storage capacity and the hydrology of the site indicates that 500 cfs is the maximum desired capacity of the principal spillway. Figure 6-18 gives the design release rate for the proposed reservoir. It is assumed that there are no significant uncontrolled flows entering the stream between the reservoir and improved channel.

In design of the improved channel and in programming reservoir releases, it is necessary to determine (1) if the proposed reservoir releases will provide equilibrium bedload transport through the improved channel; and (2) if scour or aggradation will occur and the relative rate of its occurence.

The foregoing calculations have provided data on velocities for a range of depths through the reaches represented by the two channel sections. The following steps are necessary to determine how the changes in velocity for the same discharge passing through the two reaches affect their capacity to transport bedload sediment.

Velocity-area curves for the two stream reaches were prepared from the data in Columns 3 and 5 (for the 80-foot wide channel), and Columns 7 and 12 (60-foot wide channel). The curves in Figure 6-19 provide information that enables calculations and plotting of discharges as related to velocities. In the velocity-discharge curves, Figure 6-20 discharges

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Table 6-1 - Mean Velocity Computations - Two Channel Sections

Channel - 80' bottom width; Energy gradient - 0.003 ft./foot; 1/4:1 Side Slopes					Channel - 60' bottom width; Energy gradient - 0.0025 ft./foot; 2 1/2:1 Side Slopes						
1	2	3	4	5	6	7	8	9	10	11	12
									"n" Related to Velocity	Corrected for Riprap	
Depth Ft.	Hyd. Radius Ft.	Area Sq. Ft.	Tractive Force Lbs/sq.ft.	Mean Veloc. fps	Hyd. Radius Ft.	Area Sq. Ft.	Tractive Force Lbs/sq.ft.	Mean Veloc. fps		"n"	Mean Velocity fps
0.2	0.2	16	0.037	1.3	0.2	12.1	0.031	1.2	0.0212	0.0220	1.15
0.4	0.4	32	0.075	1.8	0.39	24.4	0.062	1.7	0.0233	0.0240	1.65
0.6	0.59	48	0.112	3.4	0.58	36.9	0.094	2.3	0.0225	0.0231	2.24
0.8	0.79	64.2	0.150	5.2	0.77	49.6	0.125	4.4	0.0142	0.0170	3.68
1.0	0.98	80.25	0.187	6.1	0.96	62.5	0.156	5.4	0.0134	0.0160	4.45
1.4	1.36	112.25	0.262	7.1	1.32	88.9	0.218	6.6	0.0135	0.0165	5.42
1.6	1.54	128.6	0.300	7.4	1.49	102.4	0.250	7.0	0.0137	0.0172	5.64
2.0	1.91	161.0	0.374	8.0	1.84	130.0	0.312	7.4	0.0151	0.0185	6.03
2.6	2.46	209.7	0.487	8.8	2.34	172.9	0.406	8.2	0.0160	0.0202	6.47
3.0	2.81	242.2	0.562	9.2	2.66	202.5	0.468	8.5	0.0168	0.0213	6.69
3.6	3.33	291.2	0.674	9.7	3.13	248.4	0.562	9.2	0.0173	0.0221	7.18
4.0	3.67	324.0	0.749	10.0	3.43	280.0	0.624	9.5	0.0178	0.0230	7.34

Table 6-2 - Calculation of n adjusted for 2 1/2 to 1 side slopes (riprapped)

The formula used for obtaining the corrected n in Column 11 is:

$$n = \left(\frac{P_1\, n_1^{3/2} + P_2\, n_2^{3/2}}{\underline{P}} \right)^{2/3}$$

where n_1 = roughness coefficient of the individual lining material

n_2 = roughness coefficient of riprapped banks

P_1 = wetted perimeter associated with roughness coefficient n (bottom width)

d = depth

z = slope of banks

$P_2 = (1 + z^2)^{1/2}$ (2d) (wetted perimeter of the banks)

$\underline{P}$ = total wetted perimeter ($P_1 + P_2$ etc.)

1	2	3	4	5	6	7	8	9	10	11
P_1	n_1	$n_1^{3/2}$	d	z	$(1+z^2)^{1/2}$	P_2	n_2	$n_2^{3/2}$	$\underline{P}$	Corrected n
60	0.0212	0.0030	0.2	2.5	2.692	1.077	0.035	0.0065	61.077	0.0220
"	0.0233	0.0036	0.4	"	"	2.154	"	"	62.154	0.0240
"	0.0225	0.0034	0.6	"	"	3.230	"	"	63.230	0.0231
"	0.0142	0.0017	0.8	"	"	4.307	"	"	64.307	0.0170
"	0.0134	0.0016	1.0	"	"	5.384	"	"	65.384	0.0160
"	0.0135	0.0016	1.4	"	"	7.540	"	"	67.540	0.0165
"	0.0137	0.0016	1.6	"	"	8.614	"	"	68.614	0.0172
"	0.0151	0.0018	2.0	"	"	10.770	"	"	70.770	0.0185
"	0.0160	0.0020	2.6	"	"	14.00	"	"	74.00	0.0202
"	0.0168	0.0022	3.0	"	"	16.15	"	"	76.15	0.0213
"	0.0173	0.0023	3.6	"	"	19.38	"	"	79.38	0.0221
"	0.0178	0.0024	4.0	"	"	21.54	"	"	81.54	0.0230

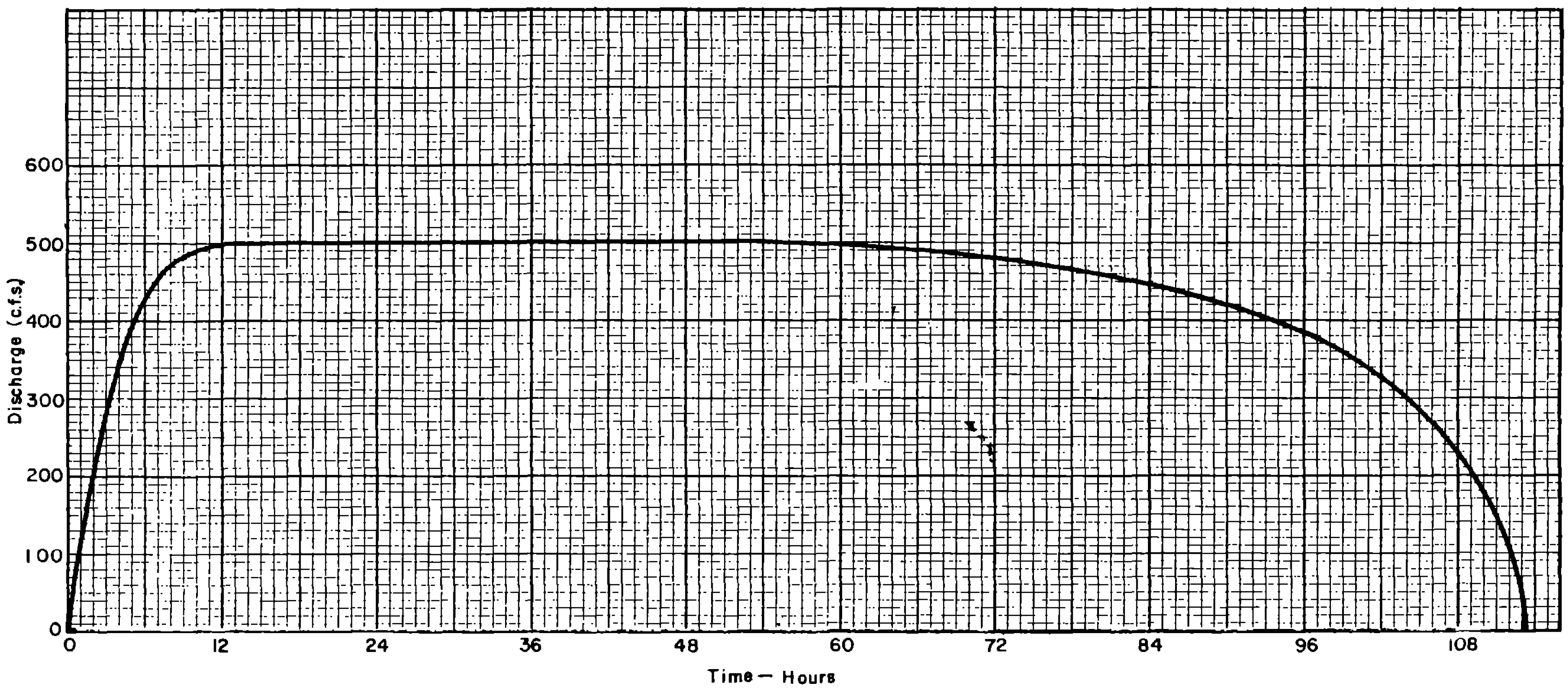


FIGURE 6-18
RESERVOIR RELEASE HYDROGRAPH
PRINCIPAL SPILLWAY

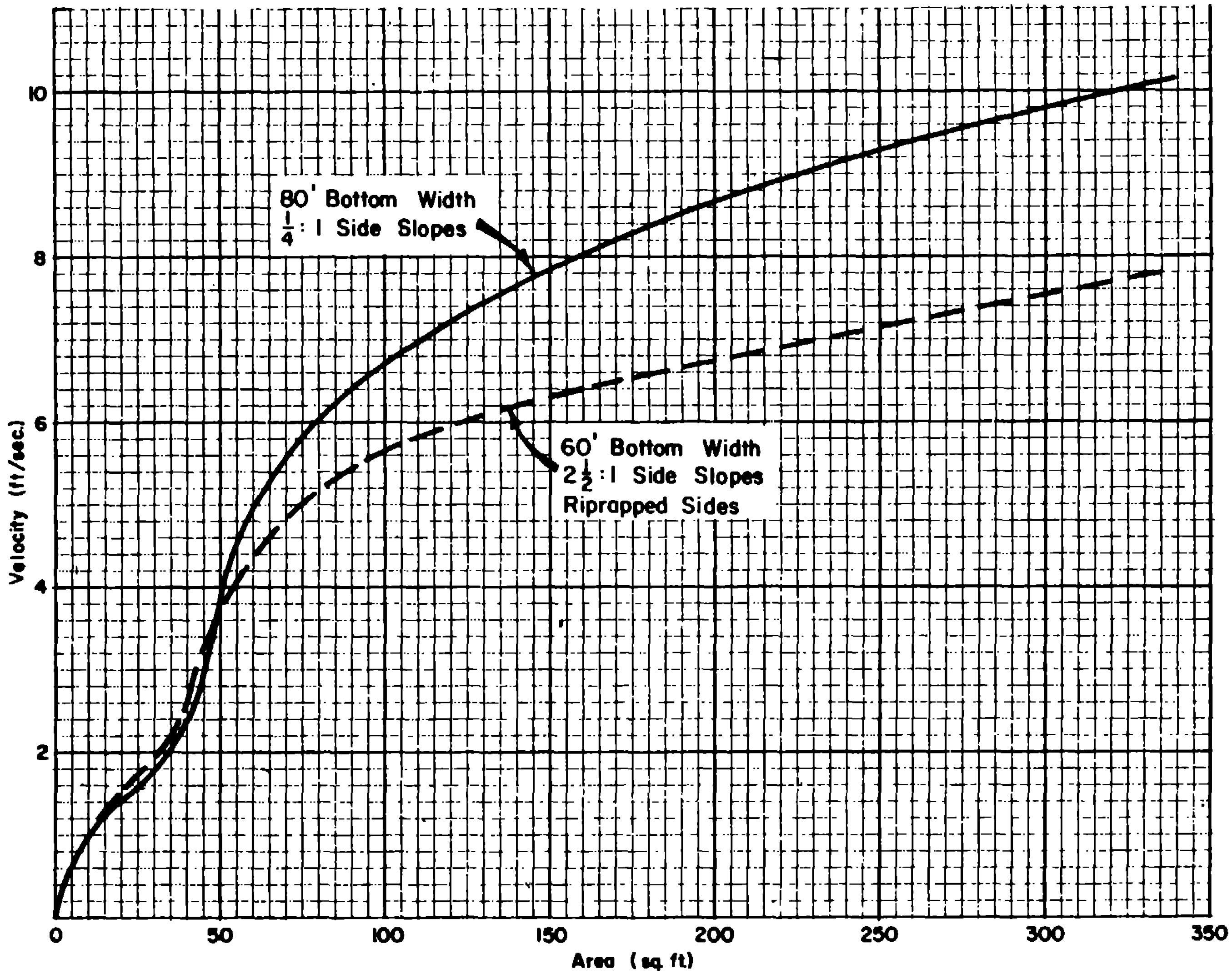


FIGURE 6-19
VELOCITY-AREA CURVE

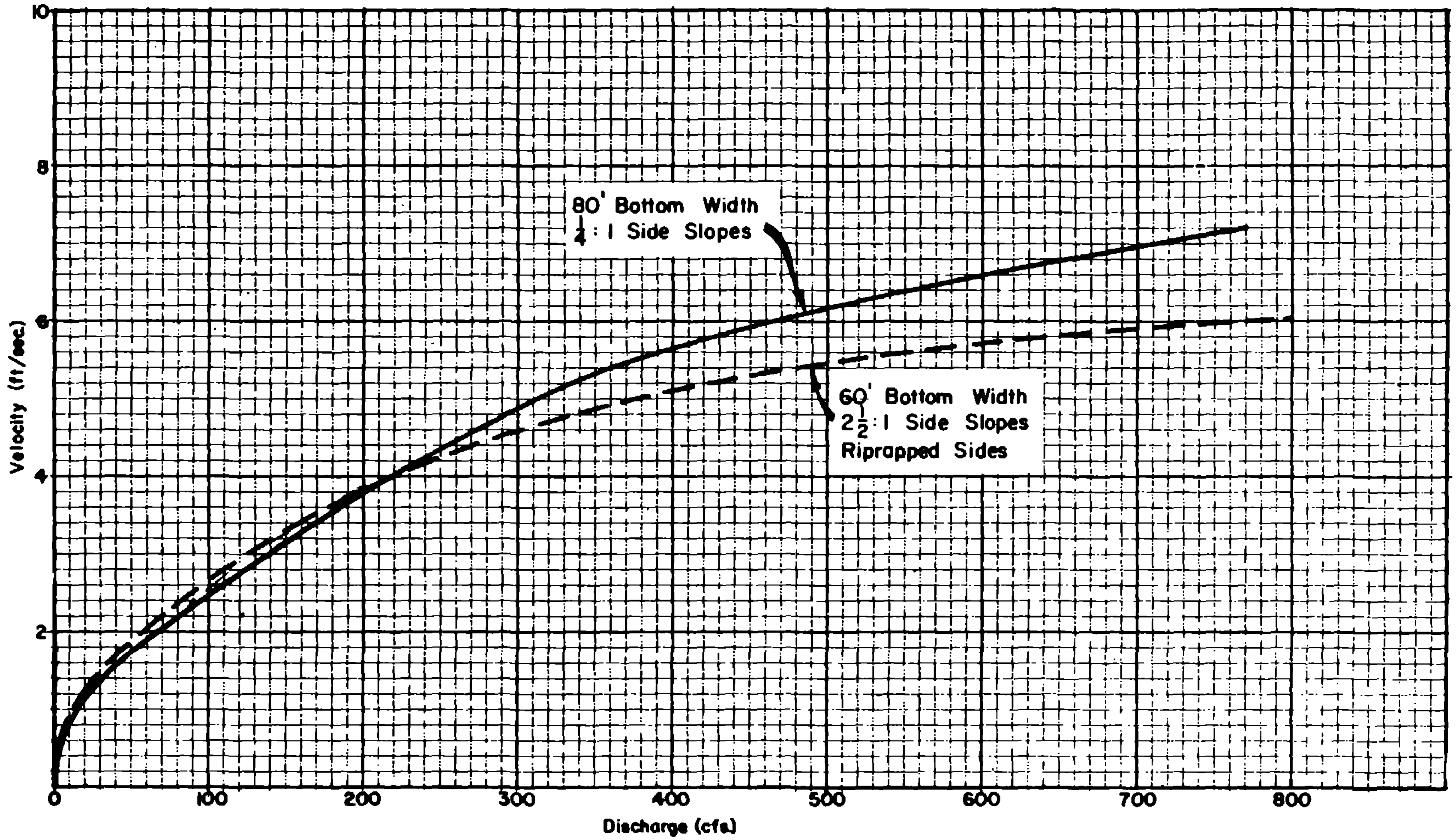


FIGURE 6-20
VELOCITY - DISCHARGE CURVE

were obtained for each 0.5 fps increase in velocity from the product of the cross-sectional area and equivalent velocity for the two stream sections shown on Figure 6-20.

The next step required selection of a mean velocity-bedload sediment transport rating curve from Figure 6-16 because of similarity in median grain size. Curve number 7 was chosen as the more applicable to this problem. Selection of another curve would result in relatively comparable qualitative results but they would differ quantitatively. However, qualitative results provide useful information for solution of this problem.

The discharge-bedload sediment transport curves of Figure 6-21 were derived from Figure 6-16 and Figure 6-20, in the following manner: Velocities for the same discharges in both stream reaches were read from the curve of Figure 6-20. In the example, the discharges selected were spaced sufficiently close (every 50-75 cfs. change) to provide adequate plotting points for drawing a curve. The sediment transport for the velocities relating to the discharges were read from Curve 7, Figure 6-16. The resulting data enabled plotting of the discharge-bedload sediment transport curves of Figure 6-21.

The remainder of the steps in this problem are indicated in Table 6-3, which shows bedload sediment transport determinations for the 80-foot bottom width and 60-foot bottom width stream reaches. Column 1 gives the range in discharge for a number of segments of the reservoir release hydrograph of Figure 6-18. The segments are so selected as to facilitate location of a point (Column 2) reflecting a mean value for the range. The elapsed time covered on the hydrograph by the discharge range in Column 1 is given in Column 3. Since sediment transport is in tons per day per foot of width on Figure 6-16 and Figure 6-21 the elapsed time is converted to percent of 24 hours in Column 4. Bedload sediment transport in tons per day per foot of width in Column 5 is the intersection of the mid-point value in Column 2 with the curve on Figure 6-21 for the appropriate channel reach. Bedload sediment transport is the product of the data in Columns 4 and 5 and the bottom width of the respective reach.

The results of the procedure applied to the example show that equilibrium transport could be maintained if the maximum reservoir release were about 150 cfs. Beyond that discharge, the improved section would aggrade with about 57 percent of the incoming bedload sediment moving through. At a maximum release rate of 500 cfs. aggradation could soon fill the channel, depending on the frequency of reservoir release and length of reach over which the deposit would accumulate. Presuming the release rate could not be reduced to 150 cfs., a great reduction in aggradation could be achieved by a change to about 300 cfs. maximum reservoir release. This is evident from the data on Table 6-3 and the increased difference in transport between the two reaches at the higher release rates.

Table 6-3
Bedload Sediment Transport
80-foot channel, energy gradient 0.003 ft./ft., 1/4:1 side slopes

1	2	3	4	5	6
Range In Discharge cfs	Mid-Point of Range cfs	Elapsed Time Hrs.	Time % of 24 Hrs.	Sediment Transport per ft.width tons/day	Bedload Sediment Transport Col. 4 x 5 x Bottom Width Tons
0 - 200	100	1	4.2	7	24
200 - 400	285	4	9.6	78	599
400 - 500	475	7	29.1	200	4,656
500 - 500	500	30	124.7	213	21,248
500 - 485	492	24	100	210	16,800
485 - 460	474	12	50	200	8,000
460 - 418	441	12	50	180	7,200
418 - 327	380	12	50	142	6,480
327 - 240	290	6	25	82	1,640
240 - 0	155	6	25	18	720
		Total bedload sediment transport			67,367

60-foot channel, energy gradient 0.0025 ft./ft., 2-1/2:1 side slopes

1	2	3	4	5	6
0 - 200	100	1	4.2	9	43
200 - 400	285	4	9.6	67	386
400 - 500	475	7	29.1	140	2,450
500 - 500	500	30	124.7	145	14,460
500 - 485	492	24	100	142	8,510
485 - 460	474	12	50	139	4,170
460 - 418	441	12	50	130	3,900
418 - 327	380	12	50	106	3,180
327 - 240	290	6	25	69	1,035
240 - 0	155	6	25	21	315
		Total bedload sediment transport			38,449

The effect of widening the improved reach to that of the upstream reach would need to be re-evaluated because the n attributable to the channel would be changed. Determination of the most efficient channel dimension and reservoir release commensurate with the site limitation may require several trial and error computations.

Bedload sediment transport in sand bed streams with constant roughness. - - The following procedure may be used in sand bed streams with a median size larger than 0.5 mm. and with constant bed roughness. The steps to be taken are the same as those given in the example 6-9 except for the variable roughness computations.

Example 6-10

Assume that stabilization requires construction of a concrete-lined channel from the edge of the foothills of a tributary across the floodplain to its junction with the main stream. The natural channel within the foothills contains a full supply of coarse sand with a median size of 1.0 mm. The width of the stream averages 28 feet and has a gradient of 0.0195 feet per foot. The slope of an alluvial fan just downstream from the foothill zone is 0.014 feet per foot. The tributary joins another tributary which has a grade of 0.006 feet per foot at the junction.

The hydrology and hydraulics of the proposed improvements show that an 8 foot wide rectangular section will be required to handle the tributary flow on a grade of 0.014 feet per foot, whereas a rectangular section 14 feet wide will be required below the junction of the two streams. Determination of relative rates of bedload transport in the natural channel, tributary section and at the junction of the tributaries is necessary to predict whether the lined sections will carry the introduced bedload or whether a debris basin may be required.

The estimated constant n for the 3 stream sections is 0.027 for the 28' bottom-width sand bed stream above the concrete lining and 0.014 for the lined sections.

Figure 6-22 shows a synthetic hydrograph for a relatively frequent event and Table 6-4 presents the mean velocity and discharge for stages of flow that would be experienced during the runoff. These correspond to Figure 6-18 and Table 6-1 in the Procedural Guide. The derivation of Figure 6-23 (Velocity-Area Curve) and Figure 6-24 (Velocity-Discharge Curve) are described for their counterparts, Figures 6-19 and 6-20 in the Guide. Curve 9, Figure 16 of the latter was extended beyond the data range in order to estimate sediment transport at higher velocities. Rates of sediment transport per foot of width on Figure 6-18 for the 3 described channel sections is equivalent to Figure 6-21 of Guide.

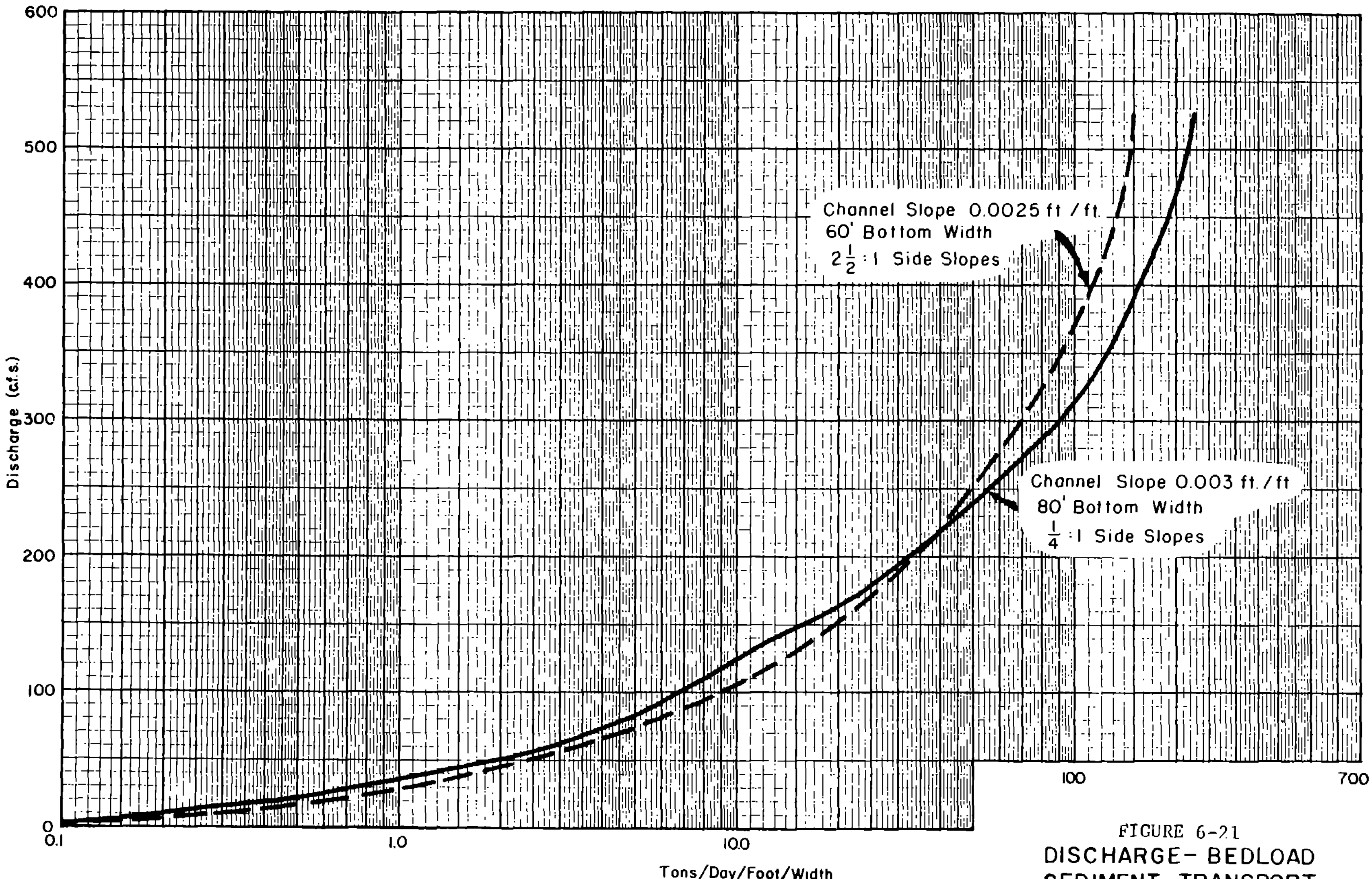


FIGURE 6-21
DISCHARGE- BEDLOAD SEDIMENT TRANSPORT

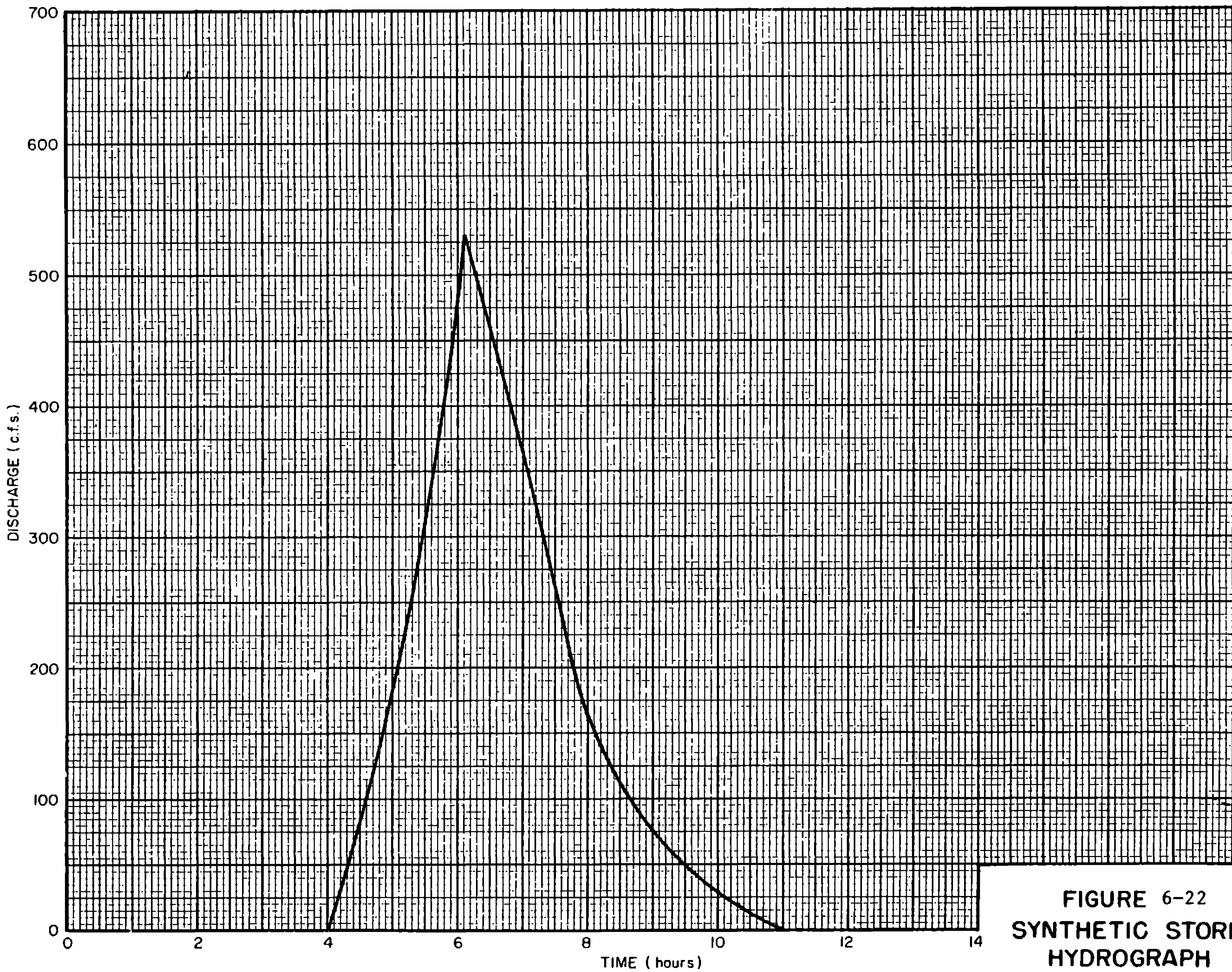


FIGURE 6-22
SYNTHETIC STORM HYDROGRAPH

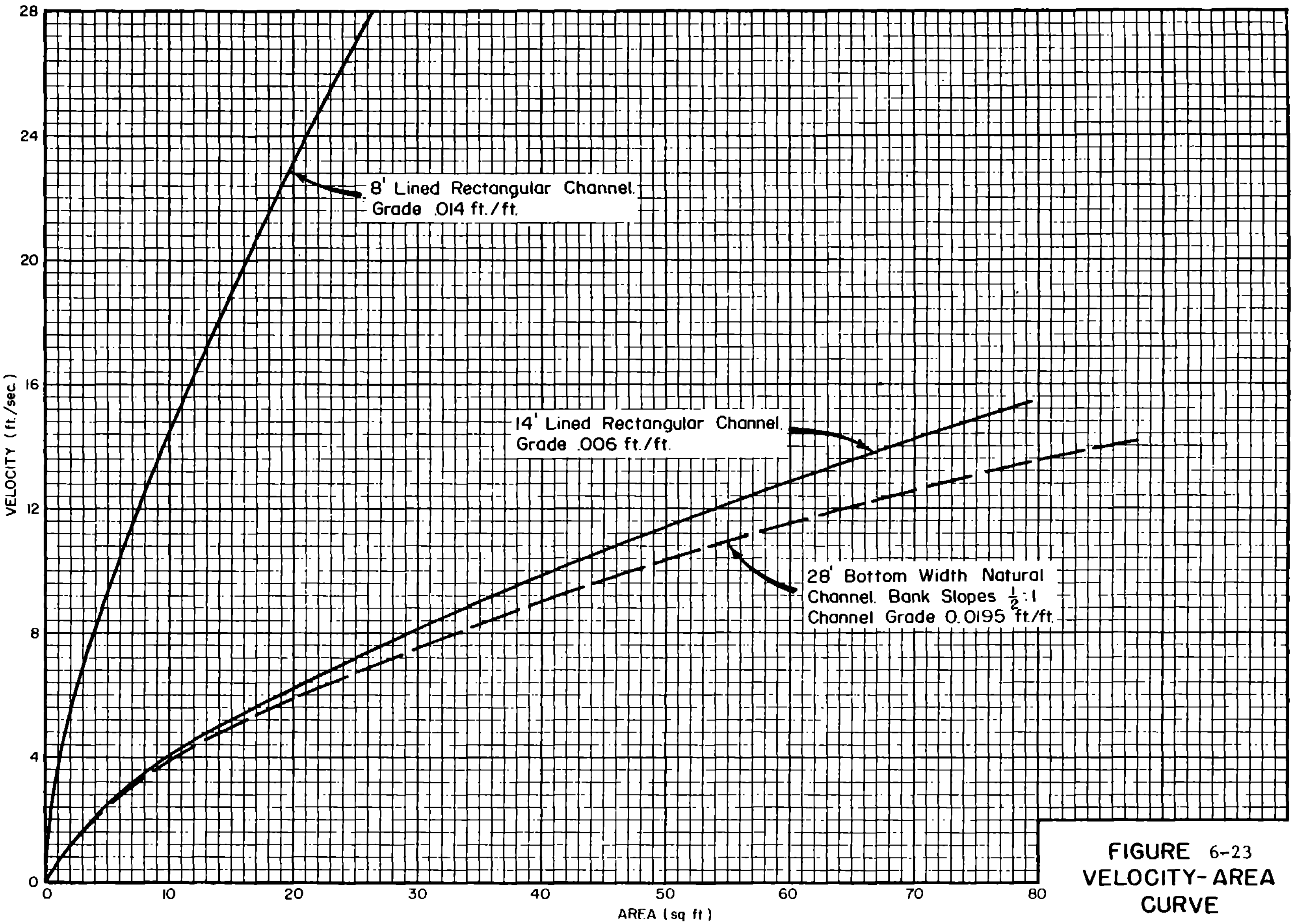


FIGURE 6-23
VELOCITY-AREA
CURVE

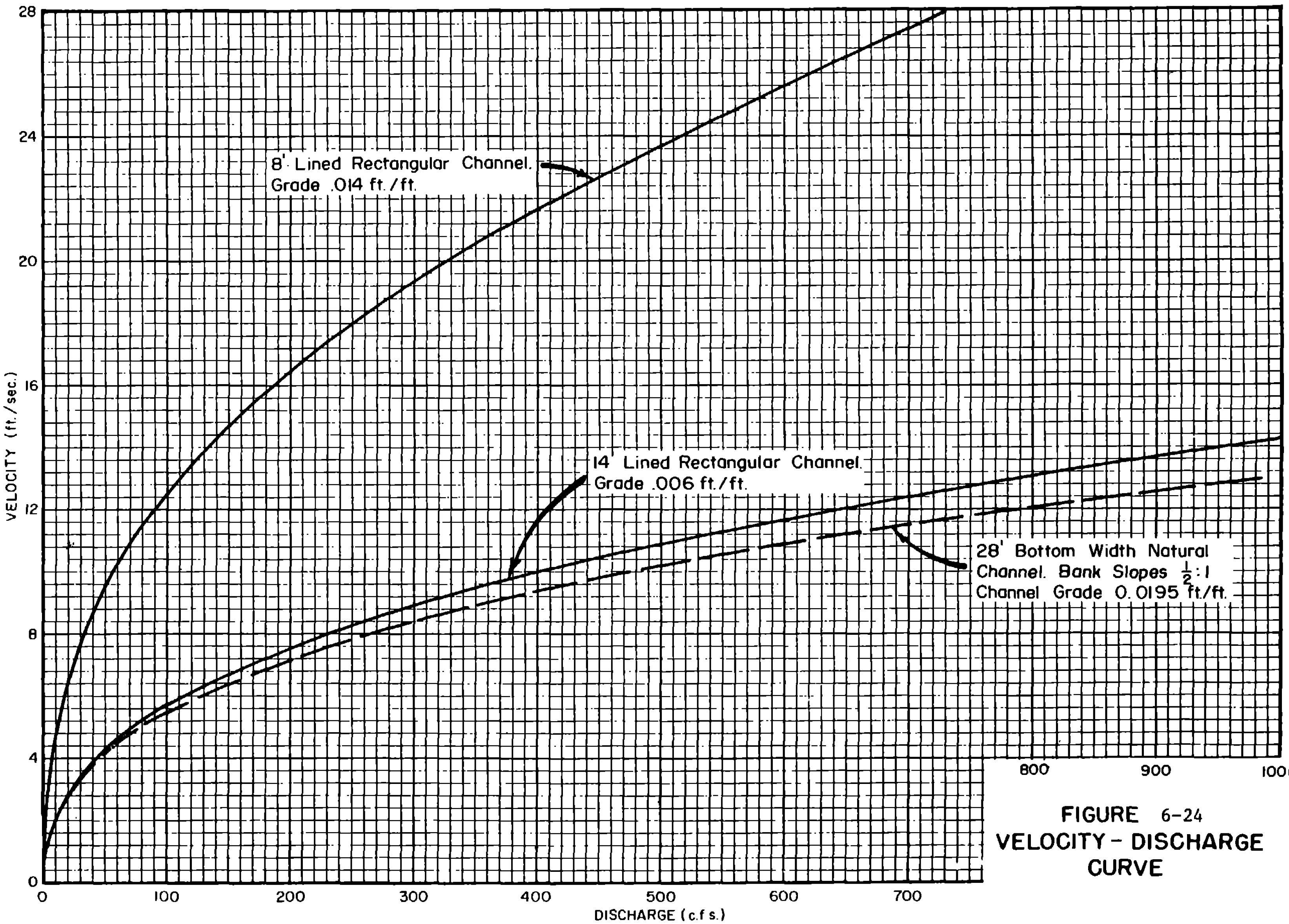


FIGURE 6-24
VELOCITY - DISCHARGE CURVE

Table 6-4 - Mean Velocity Computations - Three Channel Sections

Natural Channel - 28' bottom width Slope - 0.0195 ft./ft. 1/2:1 Side Slopes n -0.027					Channel 8' Rectangular Slope -0.014 ft./ft. n -0.014			Lined Rectangular Channel 14' width Slope -0.006 ft./ft. n -0.014		
1	2	3	4	5	6	7	8	9	10	11
Depth	Hyd. Radius	Area	Mean Velocity	Discharge	Area	Mean Velocity	Discharge	Area	Velocity	Discharge
ft.	ft.	sq.ft.	fps	cfs	sq.ft.	fps	cfs	sq.ft.	fps	cfs
0.2	0.20	5.62	2.61	15	1.6	4.30	7	2.8	1.67	5
0.4	0.39	11.28	4.09	46	3.2	6.80	22	5.6	2.65	15
0.6	0.58	16.98	5.29	90	4.8	8.92	43	8.4	3.48	35
0.8	0.76	22.72	6.38	145	6.4	10.84	69	10.2	4.21	43
1.0	0.94	28.50	7.37	210	8.0	12.57	101	14.0	4.89	68
1.2	1.12	34.32	8.23	282	9.6	14.39	138	16.8	5.53	93
1.4	1.29	40.18	9.10	366	11.2	15.70	175	19.6	6.13	120
1.6	1.46	46.08	9.79	451	12.8	17.15	220	22.4	6.67	149
1.8	1.62	52.02	10.55	550	14.4	18.50	266	25.2	7.22	182
2.0					16.0	19.90	318	28.0	7.76	218
2.2					17.6	21.2	373	30.8	8.28	255
2.4					19.2	22.4	430			
2.6								36.6	9.25	338
2.8					22.4	24.8	555			
3.0								42.0	10.2	429
3.4								47.6	11.1	529

Table 6-5 gives the results of Bedload Sediment Transport calculations in the 3 sections. The results show that the steeper concrete lined section between the tributary junction and the natural channel can carry more than 3 times the bedload sediment introduced by the storm. However, the channel below the junction can carry little more than half the amount introduced from the natural channel. The result would be a plugging at the junction and backfilling of sediment into the contributing channel. About 50 percent additional inflow of relatively sediment free water from the other tributary would be necessary for prevention of a plug forming at the junction. A basic reason for this problem developing is indicated by inspection of Figure 6-25. The sediment transport curves for the 14' lined channel at grade of 0.006 feet per foot shows rates of transport slightly in excess of that for the 28' incoming channel section per foot width. Since the latter is twice as wide, transport over the whole width substantially exceeds that of the lined section. The need for construction of a debris basin to trap the bedload sediment is indicated in this example.

Slope (Bank) Stability Analysis

General

A slope stability analysis from the standpoint of strength will not be required for all channel banks - - in fact, it may not be required for the majority of channels. However, soil conditions and all of the forces that may act on a slope should be carefully considered in deciding whether or not an analysis is needed. In some cases, it may not be economically feasible to initially construct channel banks in accordance with design requirements imposed by high water table, quicksand or "soupy" conditions, or other adverse soil and seepage pressure conditions.

It should also be recognized that it may be impractical to provide absolutely safe channel slope designs for every foot of many channel sections. Surface and subsurface investigations, sampling and testing of soils at channel sites may not be as intensive as at dam sites. Stability analyses and slope design will generally have to be based on dominant conditions with adequate provision for maintenance of trouble spots that may show up during or after construction or that may not be large enough to warrant variation in the design of the overall project. However, in some situations where seepage conditions may be limited in extent and in critical areas, the design based on dominant conditions may be modified by drainage appurtenances or by a change in slope inclination.

The banks of existing channels in similar soils and under similar conditions should be studied. Past experience with channel banks under similar conditions should be reviewed.

Table 6-5 - Bedload Sediment Transport - 3 Stream Sections

Hydrograph Data (From Fig. 1)				28' Bottom Width- Natural Channel Slope-0.0195 ft/ft 1/2:1 Side Slopes		8' Rectangular Lined Channel Slope-0.014 ft/ft		14' Rectangular Lined Channel Slope-0.006 ft/ft	
Range In Discharge cfs	Mid-Point of Range cfs	Elapsed Time Hrs.	Time % of 24 Hrs.	Sediment Transport Per ft. width tons/day	Bedload Sediment Transport Col. 4 x Col. 5 x width tons	Sediment Transport Per ft. width tons/day	Bedload Sediment Transport Col. 4 x Col. 7 x width tons	Sediment Transport Per ft. width tons/day	Bedload Sediment Transport Col. 4 x Col. 9 x width tons
1	2	3	4	5	6	7	8	9	10
0-200	100	1	4.2	54	64	650	218	61	56
200-400	300	0.7	2.9	200	218	2,550	591	238	97
400-555	460	0.4	1.67	332	155	4,200	560	410	96
555-450	480	0.4	1.67	360	168	4,600	614	432	101
450-250	360	1.0	4.2	250	295	3,100	1,040	303	178
250-160	205	0.5	2.08	120	70	1,550	258	145	42
160- 25	72	1	4.2	36	42	460	154	40	24
25- 0	12	0.5	2.08	6	--	60	10	---	--
					1,012		3,435		574

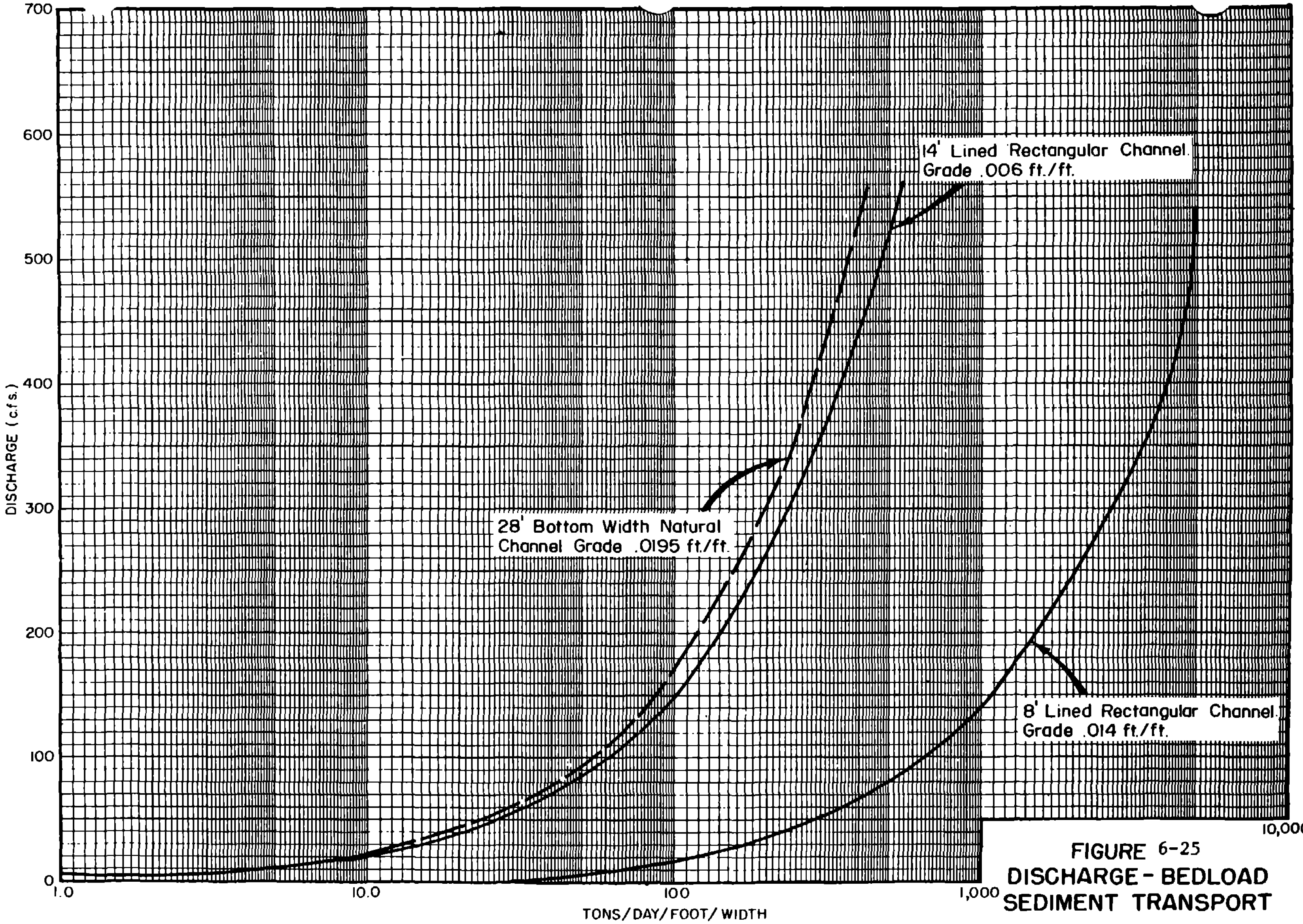


FIGURE 6-25
DISCHARGE - BEDLOAD
SEDIMENT TRANSPORT

The design of most channel banks from the standpoint of strength will probably depend largely upon local experience and past performance; detailed analyses generally will be limited:

1. to those sections where critical soil or stress conditions are anticipated,

2. to new areas in which experience is lacking,

3. to high hazard areas where failure would cause severe damage.

Too often instability of channels is blamed on erosional activity or on bank sloughing. Actually, many channel bank failures involve a combination of erosion and shear failure, such as:

1. degradation of the channel bottom,

2. undercutting of a bank because of channel obstructions, improper curvature, or other factors that direct channel currents toward the bank,

3. loss of toe support for a slope from internal erosion (piping).

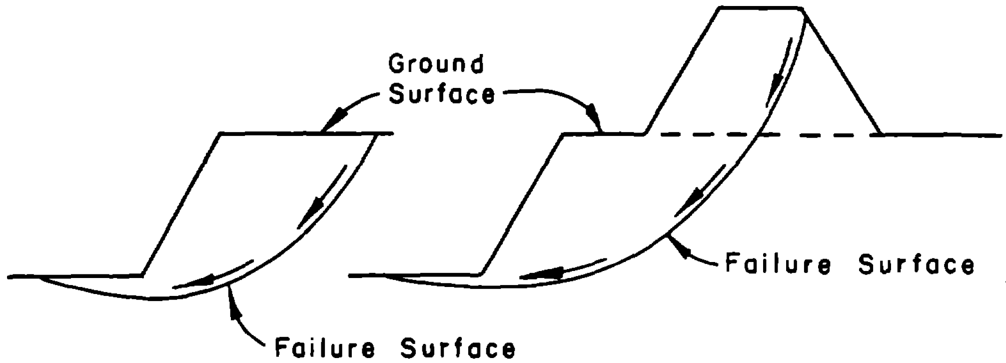
In these cases, the resistance to sliding is reduced, and the possibility of slope failure is increased, even though the original slope was stable before erosion occurred. These factors should be carefully considered in evaluating past performance of channels.

The object of a stability analysis is to determine the factor of safety for the most critical combination of stresses and boundary conditions anticipated. A good estimate of the location of the critical surface can usually be made by considering that the failure surface will tend to follow the path of least resistance, e.g., through or along material with the lowest shear strength.

Types of Slides and Methods of Analysis

No one method of slope stability analysis is applicable to all conditions. The type of potential slope failure and the location of the critical zone or plane of weakness generally dictate the method of analysis to be used.

Rotational slides. - - Rotational slides are those in which the sliding soil mass moves on a circular arc failure surface through any section of the slope or the channel bottom.



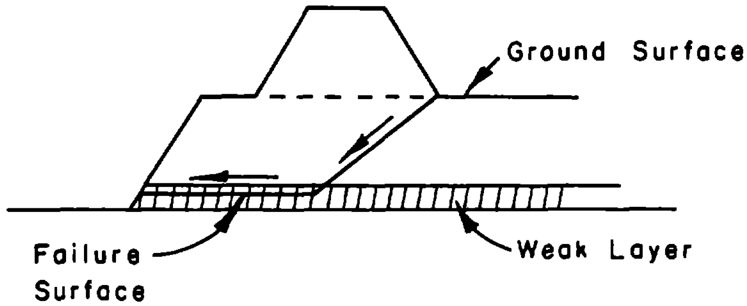
This type of failure generally occurs in plastic soils.

A Swedish slice method[34] is used to analyze rotational slides on circular failure arcs through levees, natural banks, or combinations thereof. This method is most applicable to soils that exhibit cohesion; it may be used in the no seepage, steady seepage and rapid drawdown conditions.

Although Taylor's[34] charts of stability numbers do not consider seepage, these charts may be used for rough determinations and preliminary solutions in homogeneous cohesive soils, provided a conservative factor of safety is used. If a channel is stable from an erosional standpoint and deep drying cracks are not likely to occur, bank stability in homogeneous cohesive soils is generally not a problem for channel depths less than about eight feet.

Janbu[35] has developed an analytical method for cohesive materials for cases of $\phi = 0$ and $\phi \neq 0$. Charts and graphs are presented for wet slopes in which submergence and drawdown are considered and for dry slopes. Effects of surcharge and tension cracks are also included. This method is limited to homogeneous levees, with single layer foundations or base materials, and no seepage.

Translatory slides. - - Translatory slides are those in which the soil mass moves on a zone of weakness that can be identified as the base for sliding.



This type generally occurs as a slide in a weak clay seam (stratum) or where uplift pressure is excessive.

1. The sliding wedge method[36] is applicable to translatory slides along weak planes at or above the bottom of the channel and roughly parallel to the ground surface. This method is used in connection with either the no seepage or the steady seepage conditions; it is difficult to evaluate the pore water effects in this method.

2. The infinite slope method of analysis is applicable to slopes of non-cohesive materials that are subject to either steady seepage or rapid drawdown conditions. This method of analysis assumes the soil mass slides parallel to the slope.

Boundary Conditions and Parameters Affecting Slope Stability

Effects of Water

The stability of channel banks is affected by the amount of water in the soil mass, the pressure head on the water, and the directional movement of water in the pores of the soil. The weight of the soil varies with moisture content which in turn affects the forces acting on the soil mass. The effect of moisture in terms of pore pressure alters the resistance of soil to sliding failure, i.e., seepage pressure will lower strength whereas surface tension in moist soil will increase the strength in relation to the saturated strength.

Steady seepage. - - When gravitational water moves through saturated soils, seepage forces are set up by the frictional drag exerted on the soil particles. These forces are functions of the head losses or hydraulic gradient through the soil mass. When flow moves from the bank into the channel, the resultant seepage forces decrease the stability of the bank.

The maximum anticipated elevation to which ground water may develop or the maximum pressure that may develop in shallow aquifers should be considered. The water level and pressure conditions at the time of investigation may not be the most critical condition. For example, downstream from a storage dam the ground water level in the valley may rise considerably after the dam is constructed. When water table conditions are involved in channel bank stability studies, water movement should generally be considered in a horizontal direction. Banks consisting of fine sands and non-cohesive silts are especially prone to slough under high water table conditions; stability of banks in these materials may not be achieved until the ground water has been lowered.

Under conditions of permanent low ground water where no seepage flow is assumed out of the bank, seepage forces can be neglected in the slope stability analysis. However, prolonged heavy rains can saturate a portion of the soil profile, especially if the profile is stratified, and bring water forces into consideration.

Drawdown. - - Drawdown is the lowering of the water level against a channel bank. When the water stands for some time against an earth slope, such as an irrigation canal, the soil becomes saturated. Rapid drawdown presupposes a sufficiently quick withdrawal of the water in the channel so that the soil in the banks remains saturated. Outflow from the banks is considered to move horizontally.

Shear Strength

The results of drained shear tests produce the best strength parameters for stability analysis of channel banks when all seepage forces are considered. Shear strengths from saturated unconfined compression tests or vane shear tests may be used for highly plastic soils. Results from consolidated, undrained shear tests may be used in lieu of results from drained shear tests when the former are considered adequate and representative or when pore water pressures are measured.

Unloading by excavation and the subsequent weathering of some bank materials may lead to swelling, cracking, decrease in density, and loss of shear strength. Under these conditions, the shear strength obtained from tests on unweathered samples must be adjusted downward on the basis of knowledge of the material, past experience, and judgement.[37]

Unless shear tests have been made on materials in spoil banks or levees, the shear strength of these materials should be ignored in the resisting forces. The weight of such materials must be considered in the driving forces, however.

Seismic Forces

The effect of earthquake shocks can be ignored in the stability analyses of channel banks in a large portion of the United States. However, in some areas (the Western States in particular) seismic effects should be evaluated as a design factor, when a slide would result in costly property damage or loss of life.

The designer should review the following references for information on earthquake history and seismic effects on dams:

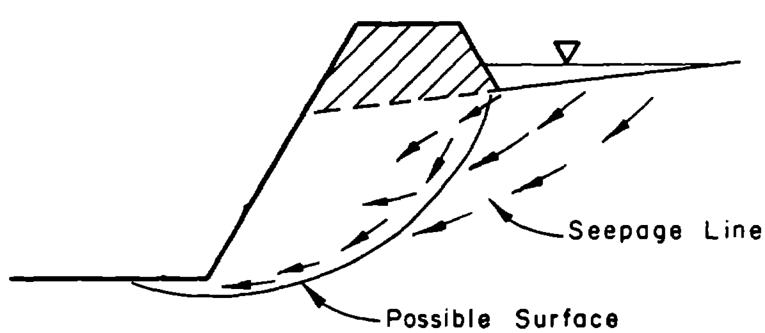
> "Earthquake History of the United States," U. S. Department of Commerce, Coast and Geodetic Survey Bulletin No. 41-1.
>
> "Seismic Stability of Earth Dams" by E. E. Esmiol, U.S.B.R. Technical Memorandum No. 641.

It is suggested that seismic loadings be obtained from Figure 17 of Technical Memorandum No. 641. These loads are assumed to act horizontally in the direction of instability and should be applied to the worst condition other than rapid drawdown.

In lieu of values from Figure 17, earthquake effects may be included by the addition of a horizontally directed inertial force of 0.1 g, i.e., the stress increase is 0.1 of the weight of material above the slip surface.

Surcharge

Surcharge loads, such as levees, spoil banks and roadways near the top of channel banks should be avoided or minimized when possible, especially those conditions shown in the figure below. In this situation, the driving forces are increased by the weight of the excavated material placed at the top of the bank. In addition, free runoff of surface water is prevented. The seeping water from the land side of the spoil bank weakens the soil in the zone of possible failure and increases its unit weight. The resisting forces are decreased, and the driving forces are increased.



When levees or spoil banks are located away from the edge of the channel bank so as to leave a berm at the ground surface, the forces tending to cause shear failure or sloughing of the channel bank are considerably reduced. NEH, Section 16[2/], pages 6-18 and 6-19, contains a discussion on natural ground berms and spoil banks.

If surcharge loads will exist, they should be considered in the stability analysis of channel banks. The largest anticipated value of the unit weight of soil in levees and spoil banks should be used in the analyses. Unit weights will vary with soil types, moisture contents, and methods of placement. For example, the unit weight of materials placed by dragline may vary considerably from the unit weight of material placed by hauling equipment.

The line load, plus the appropriate roadbed surcharge load, should be included in the stability analysis when roads will be located adjacent to banks, on berms or on levees of channel projects.

Tension Cracks

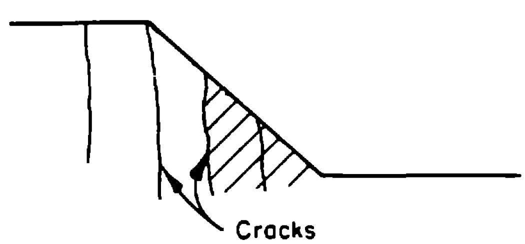
Experience has shown that the upper portion of most cohesive slopes is initially in a state of tension. The depth to which tension extends can be roughly estimated by the following equation:

$$H_c = \frac{2\,c_m}{\gamma_m} \tan(45^\circ + \frac{\phi m}{2})$$

where terms are as defined in glossary.

In a stability analysis, the depth of tension cracks should not be extended below the water table or over one-half the height of the slope. For a vertical bank, the location of greatest tensile stress is back from the edge of the cut a distance equal to approximately one-half of the height. The cohesion portion of the shear strength should not be used in the zone of cracking. The hydrostatic pressure created by water in the cracks should be added to the driving forces.

Cracks caused by excessive shrinkage may exist in some soils to greater depths than the tension zone previously discussed. The soils generally most susceptible to shrinkage cracking are clays having liquid limits greater than 40 and plasticity indexes greater than 20.



In clay soils where the water table is low, shrinkage cracks may develop in channel banks as shown in the above sketch; regardless of the slope of the bank. Blocks of soil, as indicated by the cross-hatched lines, are further weakened by water in the channel and eventually are moved from the bank. Under these conditions, channel banks tend to become vertical. A similar situation occurs in dry soils having a columnar structure, such as loess.

None of the methods of stability analysis presented in this section are directly applicable to a solution involving the shrinkage condition.

Factors of Safety Against Sliding

The end result of all methods of slope stability analyses is a comparison of the forces that cause sliding with those that resist sliding. The ratio of the resisting forces to the driving forces is the factor of safety against sliding.

The minimum acceptable factor of safety is dependent:

1. on the method of analysis used,

2. on whether all loads and forces on the banks have been considered and included in the analysis,

3. on strength parameters that may have to be correlated or estimated to a considerable extent because of limited intensity of investigation and testing.

Piping

Piping is the movement of soil particles by percolating water and the subsequent development of internal channels or pipes. The formation of pipes in the periphery of a channel reduces support for the toe of the slope; this loss of support may eventually create an unstable bank.

When hydrostatic pressure exists in a sub-stratum at a planned project site, this pressure may become excessive when overburden materials are removed, with the result that heaving and piping may occur in the bottom and/or banks of channel. After excavation of the channel, the effective weight of the soil overlying the stratum under hydrostatic pressure must be greater than the uplift pressure if the channel is to be stable. In order to make an analysis, the hydrostatic pressure must be determined by piezometers or other means.

Water impounded in a reservoir may increase the uplift pressure on a channel downstream from a dam. In those cases where a less pervious blanket overlies a more pervious stratum, the uplift analysis may be made by blanket equations.[38]

In the case where an aquifer lies above the bottom of a channel, it may be necessary to construct a flow net to determine the exit gradient for use in a piping analysis.

The minimum acceptable factor of safety against heaving and piping is generally 1.5.

Stabilizing Measures

General

When the preliminary design for an earth channel indicates that the allowable tractive force and velocity will be exceeded, consideration should be given to vegetation or structural stabilization.

Stabilizing measures can be classified broadly into three groups - - bank protection, channel linings, and grade control structures.

Bank protection and channel linings protect the channel surfaces from erosion caused by movement of water and transported materials and from shallow surface sliding.

Grade control structures may be used to reduce the channel bottom grade with a resulting reduction in velocity and scour, to control overfalls at the head end of channels, and to control the discharge from tributary channels.

The selection of a particular measure or combination of measures should be based on sound engineering and agronomic principles for each particular situation since channel stabilizing problems can vary considerably from one location to another.

Except in narrowed channels, protective elements should approximate natural roughness. Revetments should be as coarse in texture as natural banks. Retards, baffles and jetties should simulate the effect of trees and boulders along natural banks and in overflow channels.

Bank Protection

Under certain conditions channel stability may be obtained by providing protection to the banks only. Examples are at sharp changes in alignment and at bridges, culverts, or grade control structures where the bottom is stable.

Vegetation[21/ 39/ 40/]

Vegetation may be considered for sites suitable to good vegetative growth. It can be used alone or in conjunction with structural measures to provide a more effective and permanent type of protection.

It may be necessary to use temporary materials to protect the seedlings or plants against erosion from wind and runoff during the period of establishment.

Permissible velocities for vegetative cover are given in Table 3 of SCS-TP-61[21]. This table indicates a range of velocities from 2.5 f.p.s. for easily erodible soil, to 8.0 f.p.s. for erosion-resistant soil. Velocities exceeding 5.0 f.p.s. should not be used except where good cover and proper maintenance are assured.

Conditioned Earth

Conditioned earth may be used to increase the stability of channels with stable bottoms by providing denser, more erosion-resistant soil in the channel banks. Earth banks may be "conditioned" by the following methods: (See page 29 of [39])

1. Compacting the existing soil in the shaped channel to a greater density.

2. Over-excavating to a larger cross section than necessary, and placing a compacted less permeable soil as a lining. Both methods 1 and 2 are not easily adaptable to slopes steeper than 3 to 1.

3. By adding chemicals to the soil.

Revetments[41] [42] [43]

Revetments of various types may be used to stabilize channel banks.

Retards and permeable jetties. - Retards and permeable jetties are extensive or multiple-unit structures composed of open forms like piling, fencing, and unit frames. However, their function and alignment are different.

Retards are placed parallel to erodible banks of channels on stable gradients where the prime purpose is to lessen the tangental or impinging stream velocities sufficiently to prevent erosion of the bank and to induce deposition. As a remedial measure, the prime purpose may be deposition near the bank in deep channels or restoration of an eroded bank by accretion.

Retards may be used alone (see Fig. 189 of [43]) if the bank will be protected by deposition behind the retard, or by establishment of vegetation, otherwise they should be used in combination with an armor protection. (See Fig. 190 of [43]) Retards may permit use of a lighter type of armor or they may be used as toe protection of armor revetments when a good foundation for the revetment is impractical because of high water or extreme depths of poor soil materials.

On tangent reaches where the channel is narrow, retards may, by slowing the velocity on one side, affect an increase in velocity on the other. In wide reaches of a meandering stream retards may reduce an impinging attack as well as have beneficial effect on the opposite bank by slowing a rebounding high velocity wave.

Permeable jetties are placed at an angle with the channel bank and are generally used in meandering streams to direct the current away from the bank. (See Fig. 191 of [43/]) They encourage deposition of bed material and growth of vegetation, but where retards build a narrow strip in front of the bank, permeable jetties cover a wider area roughly limited by the envelope of the outer ends.

Timber piling. - - Timber piling retards and jetties may be of single, double or triple rows of piles with the outside or upstream row faced with wire mesh or woven wire fencing material which adds to the retarding effect by trapping light brush or debris. This type of retard is particularly adapted to larger channels where the piles will remain in the water, removed from the fire hazard of brushy banks. The number of pile rows and amount of wire may be varied to control the deposition of material. In leveed channels, it is often desirable to discourage accretion in order not to constrict the channel but still provide sufficient retarding effect to prevent loss of bank protection such as vegetation or small rock riprap. When used as jetties, the purpose is to encourage deposition of material and protect vegetation. Assuming negligible fire hazard, the wood may be treated with preservative to provide a long life (Fig. 195 of [43/]).

Fence types. - - For smaller channels or areas of less frequent flood flow attack, such as overflow areas, single and double rows of various types of fencing may be used. (Figs. 202-205 of [43/]) All metal types, such as pipe-and-wire or rail-and-wire, are more suitable when conditions are conducive to the growth of brush that presents a serious fire hazard to wooden posts. Details of typical designs of pipe-and-wire retards are found in Figs. 206 and 207 of [43/].

The principal difference between fence retards and ordinary woven wire fences is the posts of retards must be driven sufficiently deep to avoid loss by scour.

When it is necessary to reduce the permeability as an aid in directing the stream, as is frequently required at earth fills behind bridge abutments, self-adjusting wire baskets[43/] may be used and filled with alternate layers of rock and brush.

Permeability can be varied to meet the requirements of the location. For single fences, the factor most readily varied is the pattern of the wire mesh. For multiple fences, the mesh pattern can be varied or the space between fences can be filled to any desired height.

Making optimum use of local materials, this fill may be brush ballasted by rock, or rock alone.

Jacks and tetrahedrons. - These devices are skeletal frames adaptable to permeable retards and jetties by tying a number of similar units together in the desired alignment.

They serve best in meandering channels that carry considerable bedload during flood stages. Impedance of the stream along the string of units causes deposition of bed material, especially at the crest of flow and during falling stages. Beds of such streams often scour on the rising stage, undercutting the units and causing their subsidence, and rotation when one leg or side is undercut more than the other. Deposition on the falling stage usually restores the former bed and partially or completely buries the units. However, in the lowered and rotated position, they may be completely effective during future flood flows.

Selection of jacks and tetrahedrons may be influenced by location in or near urban or recreational areas. Unless the units will be screened by natural vegetation, attention should be given to their appearance. Where units may become "attractive nuisances," details should avoid sharp points and edges or other features dangerous to children.

Rock riprap (not grouted). - This kind of protection consists of rock courses placed either directly upon the bank slopes or on gravel filters on bank slopes. (See Fig. 152 of 43/)

Where stones of sufficient size and quality are available, it may be the most economical type of revetment and has the following advantages:

a. It is flexible.

b. Local damage or loss is easily repaired by the addition of rock.

c. Appearance is natural, hence acceptable in recreational areas.

d. Vegetation may grow through the rocks adding structural value to the bank material and restoring natural roughness.

e. Additional thickness can be provided at the toe to offset possible scour when it is not feasible to found it upon a solid foundation.

f. Wave runup is less (as much as 70%) than with smooth linings.

g. It is salvable. The rock may be stockpiled and re-used if necessary.

h. Rock slope protection, more than any other type, adopts a non-uniform widely varying material to a structural purpose, with gravity alone holding the stone together.

Rock riprap should:

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

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

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

Rock and wire mattress (gabion revetment). - This type of bank protection consists of connected flat mats fabricated of wire mesh or woven wire fencing filled with rock and adequately anchored to the bank. (See Fig. 363 of [43/])

As a revetment, its application has been limited to locations where the rock economically available is too small for ordinary rock riprap, or where grouted protection is unsuited because of fineness of stone or insecurity of bedding or foundation. Alternatives of wire strength and mat sizes make rock and wire mattresses adaptable to a wide range of exposure to hydraulic forces, but the lighter exposures are served more economically by reticulated revetment.

The most common use of rock and wire mattresses has been to provide flexible toe protection for other types of bank protection as shown in Figs. 179-181 of [43/]. The mat will adjust itself by flexure and subsidence, and block the progress of erosion and scour that might threaten the toe of the bank. This type has not performed well on curves (Figs. 182, 325 of [43/]), where settlement requires extending or shortening of the length of the mat. It is more adapted to tangent reaches when the mat has sufficient strength to hang suspended when deep or uneven scour occurs. Its life and that of the bank protection above depend on the durability and strength of the wire. Therefore, rock and wire mats should have a longer service life in drier climates and mature channels carrying mud and silt (but not gravel and stones that would abraid and shorten the life of the wire mesh).

Considering the high cost of the labor involved, the questionable service life of the wire, and the efficiency of modern methods of excavating for toe protection, use of this type of bank protection has declined.

Reticulated revetment. - Wire-mesh netting is useful in revetment work to confine rock that by itself would be too light to resist the erosive forces of the stream flow. It may be used as a cover for banks over which a layer of rocky material has been placed. The size of the mesh must be small enough to confine the majority of the stones. Although some small stones may wash through the netting, there will remain a top layer of larger stones which, in turn, will confine the small stones underneath.

The netting is placed over the rocky slope and pinned by means of short lengths of reinforcing bar hooked at the top. Brush may grow through the wire and provide additional anchorage. An application of this type is shown in Fig. 187 and typical design details are shown in Fig. 188 of 43/. If the channel bed material is gravel, the wire may serve as a flexible toe protection by extending it into the channel bed and weighting the toe end.

Sacked concrete riprap. - This method of protection consists of facing the banks with sacks filled with dry concrete mix. Much hand labor is required but it is simple to construct and adaptable to almost any contour. A photograph of this type of installation is shown in Fig. 169 and typical plans adapted to several slopes are reproduced in Figs. 170 and 171 of 43/.

Sacked concrete is an expensive but commonly used type of revetment. Where both ledge rock and gravel are readily available, sacked concrete may cost four to five times as much as an equal quantity of rock. It is almost never used unless suitable stream gravel is available at the location and satisfactory rock is not.

Dry pack may be an excellent device for subaqueous placement, for initial foundation, or repair of undercuts. It is also adaptable for protection or repair of small areas.

In many locations, the smoothness of sacked concrete is very undesirable and its use may require surface roughening. Projecting dowel bars and honeycombed surface concrete have been used for this purpose.

Portland cement concrete articulated block. - This type of revetment consists of small precast concrete blocks held together to form a flexible mat. A typical installation is shown in Fig. 115 of 43/.

In this type of installation, the blocks contain wire-mesh reinforcement with rebars extending out from each edge and bent into an eye at one edge and a hook at the opposite edge. As the block is placed, the open hooks are put through the eyes of the adjacent blocks and closed. It is easily placed and is desirable from an appearance standpoint. This type of fabrication becomes complicated for curved contours as the blocks must be cast in different sizes for each row. Use of this type has been most successful for toe protection on tangent sections.

Grouted rock riprap. - This type of revetment consists of rock riprap having voids filled with portland cement concrete grout to form a monolithic armor. A photograph of this type of installation is shown in Figs. 112, 159 and a typical plan in Fig. 160 of [43]. It has application in areas where rock of sufficient size for ordinary rock riprap is not economically available. It also will generally reduce the quantity of rock needed for a given job. Grouting not only protects the stones from the full force of high velocity water but integrates a greater mass to resist its pressure.

Grouting will usually more than double the cost per unit volume of stone, but the use of smaller stones in grouted rock slope protection than in an equivalent protection using ungrouted stones permits a lesser thickness of protection which may offset to some extent the cost of the grout.

As this type of protection is rigid without high strength, support by the banks must be maintained. Slopes steeper than the angle of repose of the bank material are risky.

Asphalt concrete paving. - This type of revetment consists of a facing of asphalt concrete usually reinforced by wire mesh. (See Fig. 175 of [43]) Such revetment is very susceptible to damage from hydrostatic pressure behind the pavement and should not be used unless relief from this condition can be provided at reasonable cost.

It has found most use in bank linings where drawdown is not rapid and water pressure acts to maintain close contact between the paving and the bank. It has been used without reinforcement as a lining for small drainage ditches where it is placed and compacted by hand. (See Fig. 176 of [43].)

Concrete paving. - This method of protection consists of paving the bank slopes with reinforced portland cement concrete. A photograph of this type of installation is shown in Fig. 162 and typical plans are shown in Figs. 163 and 164 of [43].

It is particularly adaptable to locations where the hydraulic efficiency of smooth surfaces is important. On a cubic yard basis, the cost is high but as the thickness is generally only 3 to 6 inches, the cost on a basis of area covered will usually be less than for sacked concrete slope protection. This is especially so when sufficiently large quantities are involved and alignment will permit the use of mass production equipment such as slip-form pavers.

Because of the rigidty of portland cement concrete slope paving, its foundation must be good and the bank slopes stable.

Bulkheads. - In bank protection, a bulkhead is constructed along a steep slope to retain the bank from sliding as well as to protect it against erosion. (Fig. 226 of 43/.)

Walls. - - The commonest bulkhead in bridge practice is the wingwall (or endwall) serving as a transition from a rectangular constriction to a trapezoidal channel. The commonest forms (Fig. 227 of 43/) are:

1. Straight Endwall

   This type has no transitional value but protects approach against eddy erosion; it is suitable only for low velocity in poorly defined channel.

2. Straight Wingwall

   This type also has no transitional value but protects steep banks which support the approach embankment.

3. Oblique Wingwall

   This is a conventional transition; it is efficient and economical for well-defined channels and moderate velocity. Flare angle in degrees should be limited to $\frac{300}{V}$ for converging and $\frac{150}{V}$ for diverging flow, where V is the velocity in f.p.s. through the constricted section.

4. Tapered Wall

   Tapering the grade of the parapet of either the straight or oblique wingwall is common practice for streams of moderately low velocities. By matching the surcharge slope to the natural bank, the transition progressively exposes this slope to the low velocity boundary of the varied flow.

5. Warped Wall

   Tapering the slope of the wall from vertical at the abutment to a stable-bank slope at the end of the wall makes an excellent transition for moderate to high velocity.

6. Returned Wall

Building the standard cantilever wall on a curved alignment returned from the abutment is an economical solution for a combination of a vulnerable approach embankment projecting into a channel with durable banks.

Cribs. - - Timber and concrete cribs are used for bulkheads in locations where some flexibility is desirable or permissible (Figs. 229-233 of 43/). Using backfill for stability, cribs are economical in the use of structural materials. Their rough surfaces are advantageous in all natural locations where banks are exposed to high velocities.

Piling. - - Timber, concrete and steel piling are used for bulkheads depending on deep penetration of foundation materials for all or parts of their stability. Any of the three materials is adaptable to sheet piling or a sheathed system of post or column piles. (Fig. 234 of 43/.)

Channel Linings

Channel linings are used to protect the entire channel surface.

Vegetation is the most commonly used protection for channels with infrequent flow, relatively low velocities, and where a good stand can be established and maintained.

Ungrouted rock riprap may be used for channel lining where soils are not suitable to vegetative growth. Such lining is applicable to the inlet and outlet of channel structures for stabilization of bottom and banks.

Where it is necessary to conserve water by limiting or eliminating seepage, where high velocity flow occurs, or where channel operation at high hydraulic efficiency is required, durable, relatively impervious linings such as concrete or asphaltic concrete may be used. For relatively short reaches grouted rock riprap may be used. Such linings may be required where channel right-of-way is limited.

The usual shape of cross sections for vegetated cover, ungrouted rock, grouted rock, asphaltic concrete, or other non-structural sections is trapezoidal. For reinforced concrete, the cross section may be rectangular or trapezoidal. The type of protection selected will depend almost entirely on economics. The only sure way to select the most economical material is to prepare a preliminary design for each and compare annual costs.

As a guide to trial selection of the type of lining, the following approximate criteria are presented:

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1. Rectangular reinforced concrete channels will show the least annual cost when velocities are high, rights-of-way are expensive, and wall heights are 15 feet or less.

2. Trapezoidal R/C channels are most economical for the above conditions when right-of-way costs are more moderate and channel wall heights are quite great (usually over 15 feet).

3. Loose rock lining is efficient when velocities are not so great as to require extremely large rock and thick sections, and where rock and filter material are available from nearby sources.

4. Grouted rock lining is generally economical only for short reached of high velocity flow where extremely large rock would be required for loose rock lining.

5. Channel linings constructed of asphaltic concrete, pneumatically applied mortar, pre-cast R/C slabs are usually economical on an annual cost basis only in special situations of availability, short project life requirements, etc.

Grade Control Structures[43/] [44/] [45/] [46/] [47/]

Various types of structures may be used to reduce the gradient in channel reaches where the channel materials will not resist the erosive forces. They can be divided into two classes - - open top structures and closed conduit structures.

Open top structures, such as drop spillways and chutes, may be considered for use with any size channels.

Closed conduit structures, such as culverts, hooded inlets and drop inlets are generally used in relatively small channels.

In the design of grade control structures the site configuration, foundation, conditions, availability of construction materials, hydraulic and structural adequacy, and economic factors should be considered.

Generally, the design procedure for grade control requirements should include the following:

1. For the channel reach selected, determine the total fall between upstream and downstream limits.

2. For the design discharge, the selected channel dimensions and type of channel protection generally determine the maximum stable channel gradient.[20/] [21/]

3. Using the total fall, length of channel reach, and stable grade, determine the amount of fall to be controlled by structure(s).

4. Select the type and size of grade control structures needed, based on site configuration, foundation conditions, availability of construction materials, hydraulic and structural adequacy and economic factors.

Since channel dimensions and type of protection directly affect the stable grade and the amount of fall to be controlled by structures, alternate designs should be made to select the most practical and economical overall plan.

Open Top Structures

Straight drop spillway[4/] [46/]. - - This type of structure is efficient for the control of relatively low heads normally up to 10 feet. It is very stable for heads less than 10 feet and the likelihood of serious structural damage is more remote than for other types of structures. However, a stable grade below the structure is essential to stability.

A rectangular weir is less susceptible to clogging by debris than the openings of other structures of comparative discharge capacities. When properly constructed, maintenance costs are lower for straight drops than for other types of grade control structures for most embankment and foundation soil conditions. It is relatively easy to construct.

Limitations to the use of the drop spillway are:

a. It is more costly than some other types of structures where the required discharge capacity is less than 100 c.f.s.

b. When the total head or drop is greater than 10 feet, it becomes costly to stabilize this type of drop structure against sliding.

c. It is not a favorable structure where it is desired to use temporary spillway storage to obtain a large reduction in discharge.

Box inlet drop spillway[46/]. - - The box inlet drop spillway can be used for the same purposes as a straight drop spillway. One of its greatest uses is for grade and erosion control in open channels where the width of outlet is limited. It can also serve as a tile outlet at the head end of the channel.

It is particularly adapted to narrow channels where it is necessary to pass large flows of water. The long crest of the box inlet permits large flows to pass over it with relatively low heads, and the width of the spillway need be little, if any, greater than that of the exit channel. The box inlet drop spillway can be easily combined with a bridge to provide a road crossing. The high portion of the sidewalls can be used as abutments for the bridge.

The structural design of the box inlet drop spillway is more complex than for straight drop spillways.

Island-type spillway[46/]. - - The island-type spillway consists of a drop structure in the channel with earth emergency spillways for carrying storm flow around the structure. Either the straight drop spillway or the box inlet drop spillway can be used. When the weir length of the structure is greater than the bottom width of the channel, the box inlet drop spillway should be considered. This type of spillway is adaptable for use at the head end of channels to control the overfall. It is particularly adapted to site conditions where the design runoff volume is greater than the capacity of the outlet channel into which the structure empties. The use of this type of grade control structure is limited to areas where there is sufficient nearly level land on either side of the channel for use as earth spillways. Topography of the ground must be such that the path of overflow around the structure will return to the channel locations a short distance below the structure without causing damage to the land or channel banks.

The island-type spillway is proportioned so that the channel will be full before the overflow around the dam enters the channel, thereby eliminating the possibilities of bank erosion from flow over the bank. To accomplish this, the crest of the weir must be set below the bottom elevation of the earth spillway, a distance sufficient to provide a weir notch capacity between these two points equal to the bank full capacity of the channel at the place where the flow from the auxiliary spillway will enter the channel. Larger flows will then pass around the earth embankment of the drop spillway forming an island composed of the drop spillway and the headwall extension levees. The waterway above the structure must have the same capacity as the channel below the dam at the point of overflow. The island spillway should be so proportioned that earth spillways will begin to flow as soon as the channel capacity flow has been reached. In order to force overflow water away from the dam and protect the fill from washing out around the dam, levees extending each way from the dam must be provided.

The island-type structure permits the use of a spillway having a capacity less than would be required to handle the total runoff peak discharge. It requires the construction of auxiliary spillways in areas that may be cropland where maintenance of the correct grade and elevation is difficult.

Concrete chute spillway[44/] [46/]. - - The concrete chute is particularly adapted to high overfalls where a full flow structure is required and where site conditions do not permit the use of a detention-type structure.

Chutes may be more economical than drop inlet structures of the same capacity and drop when larger capacities are required.

Closed Conduit Structures

Hooded inlet spillway[46/]. - - The hooded inlet spillway is best adapted for use at sites where the pipe can be installed in the original ground. Construction is complicated when the pipe is placed in the embankment.

The hooded inlet spillway will flow completely full for conduit slopes up to 36 percent (the limit of present tests) if the length of the hood is properly selected and the head on the inlet is sufficient. As compared with the drop inlet, it has the advantage that no riser is required and there is less fill over the pipe. It is simple to fabricate and install and is comparatively low in cost.

Drop inlet spillway[46/]. - - The drop inlet is an efficient structure in the control of relatively high heads. It is well adapted to sites providing an appreciable amount of temporary storage above the inlet. It may also be used in connection with relatively low heads, as in the case of a drop inlet on a road culvert.

For high heads, drop inlets require less material than a drop spillway under similar circumstances. Where an appreciable amount of temporary storage is available, the capacity of the structure can be materially reduced. Besides affecting a reduction in cost, this reduction of discharge results in a lower peak channel flow below, and can be a favorable factor in channel grade stabilization and flood control.

Drop inlets are subject to plugging by debris. They are limited to locations where satisfactory earth embankments and emergency spillways can be constructed.

Culvert drop box[46/]. - - Drop boxes are used to control gradients above culverts in either natural or constructed channels and, in addition, they may serve as an outlet structure for tile lines in drainage systems. Cattle ramps can be incorporated into the design of the box when the culvert is used as a cattle pass. The drop box is very effective for roadway erosion control.

The drop box is one of the most economical structures for controlling overfalls because the existing culvert and roadway embankment replaces the outlet portion of the typical drop spillway. It has the advantage of the box inlet drop spillway in that weir length can be fitted to a narrow waterway.

Other Structures

General

A comprehensive channel design frequently requires the incorporation of one or more of the following structures and/or practices:

1. channel crossings;

2. channel junction structures;

3. side inlet structures;

4. water level control structures.

Channel Crossings

Channel crossings are required where private or public roadways pass over the channel. Structures used for this purpose are stream fords, culverts, and bridges.

Stream fords[46]. - - Stream fords are installed on the channel surface. They provide the most economical type of crossing. They can be constructed of reinforced concrete, compacted rock, or broken concrete.

Stream fords are best suited for use in the upper ends of channels. They should not be installed where deep flows of long duration will prevent normal use.

Culverts. - - Culverts of concrete or metal pipes also provide an economical crossing when used at locations where the flow is relatively small, and where serious resistance to the flow of water is not a limiting factor in overall channel design. For hydraulic design, see page 6-29 of [2].

Bridges. - - Bridges of concrete or timber should be used when necessary on most open channels that are designed to capacity on low gradients. Since they do not offer serious resistance to the flow of water, they are preferred over culverts, especially for high flows. For hydraulic design, see page 6-32 of [2].

Channel Junction Structures

Where two main channels join, wave formation can be minimized if the two flows at the junction are as nearly parallel as possible. The design criteria for structures at a junction of 2 trapezoidal or 2 rectangular channels is shown on Fig. 5-1.

Side Inlet Structures

Provisions should always be made for lowering surface water from adjoining fields to the main channel without serious erosion. Pipe spillways, drop spillways, and chutes are the more common types of structures used for side inlets.

Side inlet structures should empty into areas recessed in the banks of the main channel. Construction in this manner will minimize damage by the movement of floodwater, debris, or ice, and also will cause less retardance of flow in the main channel.

Pipe spillways[46/]. - - Pipe spillways can be used advantageously to convey water from bank of levees and continuous spoil banks into a channel. The hooded inlet is most efficient where discharge capacity is a problem. The flared inlet is less efficient but facilitates the passing of debris such as corn stalks and grasses. The pipe drop inlet is efficient and can be used as a tile outlet. When the required pipe size exceeds 48-inches in diameter, an open top structure should be considered for economy.

Drop spillways[46/]. - - Drop spillways are generally used where the volume of water to be handled is large. They can be used as a tile outlet structure. The drop spillway fits conditions where there is no spoil bank and functions well at the head end of a channel.

Reinforced concrete chutes[46/]. - - Concrete chutes function well where the volume of water to be handled is large and the overfall is such that a drop spillway will not be economical.

Vegetated chute[46/]. - - This type of chute should be limited to small watersheds and sites where good, dense sod can be developed and maintained. The water course below the chute must be stable. When the channel below the chute is narrow or conditions at the lower end of the chute may not be favorable to establish and maintain vegetation because of poor soil or rocky or wet conditions or siltation from adjacent channels or streams, a toewall should be used. The toewall will raise the end of the sod chute above these unfavorable conditions and permit the maintenance of a good vegetation. The toewall is a small drop spillway with a headwall generally 1 to 2 feet in height.

A vegetated chute is economical since material and construction costs are generally low. Use is limited to sites where the velocity of flow in the chute is low enough to maintain the vegetative cover. This generally limits the use of vegetated chutes to small water courses with low overfalls where there is no long, sustained flow.

Riprap chute[46]. - - A rock riprap chute provides a more stable outlet than a vegetated chute. The use of native rock may make it less expensive than a pipe or concrete structure of comparable size. It is a permanent type facility requiring less maintenance than a vegetated chute.

Rock riprap lined chutes are limited to areas where suitable durable cobbles or rock are available for construction. It requires careful adherence to the basic details of design in their construction to obtain satisfactory performance and stability.

Gabion chute[46]. - - The gabion chute is similar to the riprap chute except that the rock is placed in wire baskets. It is particularly adaptable to unstable foundation conditions because of its ability to adjust and retain its general section with displacement or compression of the foundation. The opportunity to fill it with native rock and cobbles makes its cost favorable in comparison with reinforced concrete. Generally, by the time the long-lasting wire baskets deteriorate, the structure will be so well established and bound together that it will remain indefinitely without the need for added protection.

Water Level Control Structures[46]

Water level control structures are used to regulate and maintain water in channels for water table control or for flooding land surfaces. The control is accomplished by use of gates or stop logs that can be fitted into several types of structures. The most common types used are drop spillways, box inlets or culverts, and open flumes.

Design Features Related to Maintenance

Channels must be properly maintained to function as designed. Maintenance can be made easier and more effective if certain features are incorporated in the design.[2]

Added Depth or Capacity for Deposition

Allowance should be made in the design for initial sloughing and sedimentation. Quite often during the first year after construction, the channel bottom will be raised from sloughings left by construction equipment. Soil and seepage conditions affect bank sloughing and silting. The sedimentation problem must be considered in the design so that depth and capacity will be provided over a period of years in line with economy.

Relationship of Side Slopes to Maintenance Methods

The slope of channel banks may be dependent on the type of maintenance as well as on soil conditions; for example, 3:1 slopes or flatter are usually needed for banks to be mowed.[2]

Berms

Berms may be used to facilitate maintenance by:

1. Preventing material from washing or rolling into the channel.

2. Providing work areas and facilitating spreading of spoil banks.

3. Providing access roadways.

Berm design may follow the general practice of the locality where the channel is to be constructed, provided proper loading and soil conditions are used. Guidance to minimum berm widths is given on NEH, Section 16[2], page 6-19, and National Standard and Specification Guide for Dikes and Levees.

Maintenance Roadways

Roadways should be provided for access to the channel with maintenance equipment and for inspection. They can be located on berms, spoil banks, or levees. On channels in excess of 20' top width, roadways may be required on both sides of the channel. The roadway should be wide enough to handle all maintenance equipment and should slope away from the channel.

Spoil

It is good practice to spread spoil banks to the extent that they can be maintained properly and can be used in the same manner as the adjoining area. The degree to which the spoil is spread depends upon the local conditions.[29]

Entrance of Side Surface Water to Channel

Side surface water should not be allowed to spill over the channel bank without protection. Interception ditches should be provided to control local drainage on the land side of the berms or spoil banks throughout the length of the project. These ditches should be graded toward collection points to drain into the channel through lined chutes, pipe drops and culverts, or over drop spillways.

Seeding

The berms and spoil should be seeded. Quite often the channel side slopes are also seeded. The extent to which seeding is done depends upon the location of the channel and local desires. Side slope seeding is accepted as good practice, particularly when flat side slopes are used so that both seeding and maintenance can be done economically.

Pilot Channels

Occasionally pilot channels are used to facilitate construction of a channel system as designed. The principal function of a pilot channel is to lower the water table sufficiently to permit deeper excavations to be made with greater safety and economy. This is accomplished by excavating the pilot channel as deep below the water table as practical without causing excessive sloughing of the banks. Construction is then deferred until the water table is lowered and the banks become more stable.

GLOSSARY OF SYMBOLS

A - alignment factor to adjust the basic velocity because of the effects of curvature of the channel.

A - area of flow. (ft^2)

b - bottom width of a channel (feet).

b_T - water surface width (feet).

B - bank slope factor to adjust the basic velocity because of the effects of different bank slopes.

C - sediment concentration in parts per million by weight.

C_1, C_2, C_3, C_4, C_5 - coefficients used to determine channel proportions and slope when using the modified regime equations.

C_e - Density factor to adjust the basic velocity because of variations in the density of soil materials in the channel boundary.

c_m - cohesion intercept at natural moisture (psf).

d - depth of flow (feet).

d_c - critical depth of flow (feet).

d_m - mean depth of flow (feet).

D - depth factor to adjust basic velocity because of the effects of the depth of flow.

D_s - the particle diameter of which s% of the sample is smaller.

F - frequency factor to adjust the basic velocity because of the effect of infrequent flood flows.

F - Froude number = $\dfrac{V}{\sqrt{gd_m}}$

g - acceleration due to gravity (fps^2).

G - specific gravity.

H_c - depth of tension crack (feet).

k_s - characteristic length of roughness element, for granular material.

$k_s = D_{65}$ size in feet.

K - coefficient modifying tractive force for gravitational forces on coarse, noncohesive materials on channel sides.

n - Manning's coefficient.

n_t - Manning's coefficient for roughness of soil grains.

P - wetted perimeter.

PI - Plasticity index.

q_u - unconfined compressive strength.

Q - discharge (cfs).

Q_s - sediment transport rate (tons/day).

R - hydraulic radius - feet

R_c - radius of curvature of central section of compound curve.

R_t - hydraulic radius associated with grain roughness of the soil.

s_o - slope of channel bottom.

s_c - critical slope.

s_e - energy gradient

s_t - rate of friction head loss because of tractive stress acting on bed and side materials.

V - average velocity (fps).

V_a - allowable velocity (fps).

V_b - basic velocity (fps).

V_c - critical velocity (fps).

W - average width of flow - ft.

W_T - top width of flow - ft.

x - factor describing effect of ratio $\frac{k_s}{\delta}$ on flow resistance.

z - cotangent of side slope angle.

T - factor to correct allowable tractive force for materials with D_{75} > 0.25" for unit weights different than 160 pcf.

γ - unit weight of water (pcf).

γ_d - dry unit weight (pcf).

γ_m - moist unit weight (pcf).

γ_s - unit weight of particles larger than 0.25" (pcf).

γ_w - unit weight of water (62.4 pcf).

δ - thickness of laminar sublayer $= \frac{11.6\nu}{\sqrt{gR_t s_e}}$

ϕ - angle of shearing resistance.

ϕ_m - angle of shearing resistance at natural moisture content.

ϕ_r - angle of repose of coarse noncohesive materials.

ν - kinematic viscosity of water (ft^2/sec).

ρ - water density ($lb\text{-}sec^2/ft^4$).

τ - reference tractive stress (psf).

τ_∞ - tractive stress in an infinitely wide channel (psf).

τ_b - maximum tractive stress on the channel bed (psf).

τ_s - maximum tractive stress on the channel sides (psf).

τ_{bc} - maximum tractive stress on the bed in a curved reach (psf).

τ_{sc} - maximum tractive stress on the sides in a curved reach (psf).

τ_{Lb} - allowable tractive stress along the bed. (psf)

τ_{Ls} - allowable tractive stress along the sides (psf).

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TECHNICAL RELEASE No. 25, CHAPTER 6. APPENDIX A.

Stream Armor Design Concepts

Purpose

This appendix (1) explains the underlying physical processes affecting armoring, (2) describes different SCS-approved math models available, and (3) presents an example illustrating one way to estimate armoring.

The various math models for critical and recommended allowable tractive stress discussed in this appendix are accepted in the engineering profession; they differ mainly in choice of a safety factor, scope of application, or both. Two different math models for recommended allowable tractive stress are used in SCS. They differ solely in their safety factors. The armor designer is free to select the most applicable model.

Actual transverse tractive stress of each situation must be determined through a hydraulic analysis. The example in this appendix uses a simplistic model to determine the hydraulic radius. In real situations, actual cross-sectional geometry and, possibly, precise water surface profile calculations are required. However, this requirement does not invalidate the concepts illustrated by the example.

Physical Processes

Armoring is a well-known natural phenomenon. Furthermore, its important features already are used in some engineering structures, for example, riprap. Armoring is sometimes called hydraulic sorting. It is a limiting or special case of sediment transport. It has been studied by various scientists over the years, (e.g., A. Shields, A. Strickler, E. Lane, H. Einstein, and others). Understanding the primary principles of armoring is still developing and is leading to various math models and procedures for field application.

Armoring is the result of the dynamic interaction of unsteady fluid flow and a mobile bed composed generally of a broad range of discrete particles.

At low flows, the boundary is stationary; as the flow increases, however, the smallest particles begin to move. As the flow increases further, larger particles also begin to move but at a lower velocity. Finally, the discharge can increase to a point where the entire boundary is moving, although the larger particles move more slowly than the smaller. As the flow decreases, the process reverses itself; but if the smaller particles are not replaced, the bed is left degraded and coarser.

Armoring occurs when smaller particles are transported from the boundary but not replaced and coarser particles are exposed but not transported. Whether true armoring occurs depends on whether the exposed coarser particles originated at their present position or upstream. If they originated upstream, what has occurred is not armoring but sediment transport by unsteady flow.

A design of a stable channel that depends upon armoring for stability can be a contradiction unless the armor surface has already been established and will not be disturbed during construction. Otherwise, degradation must occur before a complete armor surface can exist, and resulting eroding bed material contributes a downstream sediment load to the system. Furthermore, this degradation causes undercutting of the toes of the bank, which can lead to bank sloughing. Ultimate design value of armoring may be that it is the last line of defense against the more extreme events that otherwise may completely unravel a channel and possibly lead to ecological disaster or catastrophic failure of important cultural features.

Math Models

The math models developed by Shields and Strickler provide the basis for the armoring design procedure. The procedure was verified by Lane's field work. The designer must analyze (1) the active or driving forces and (2) the passive or resisting forces. The analysis of active forces consists of determining the hydraulics or depth of flow and determining the boundary roughness shear or tractive stress. The latter determination is necessary because not all energy loss is due to boundary roughness. Bends or changes in cross-sectional area cause energy loss through internal fluid shear.

SCS has adopted Manning's equation to estimate the rate of total energy loss (S_e); i.e.,

$$S_e = [(Q\ n_e)/(1.486\ AR^{2/3})]^2$$

where

$n_e \equiv$ retardance coefficient for total energy loss.

Furthermore, SCS has adopted the Manning-Strickler equation to estimate the energy loss due to boundary roughness, (S_t); i.e.,

$$S_t = [(Q\ n_t)/(1.486\ AR^{2/3})]^2$$

where

$n_t = K_m d_m^{1/6}$ the Strickler equation -- retardance coefficient due to boundary roughness only.

$d_m \equiv$ a characteristic boundary particle size.

$K_m \equiv$ empirical coefficient relating d_m to n_t.

Units for K_m must be consistent with units chosen for d_m.

Report 108 of the National Cooperative Highway Research Program recommends using $K_m = 0.0395$, with $d_m = d_{50}$ expressed in feet. The K_m value is the same as the default value for C_n in Eq. 2 of TR-59, "Hydraulic Design of Riprap Gradient Control Structures."

This leads to the following formula for actual average transverse stress ($\overline{\tau}_{act.}$):

$$\overline{\tau}_{act.} = \gamma R\ S_t$$

where

$\gamma = 62.4\ \#/ft^3$

$R \equiv$ hydraulic radius, ft.

$S_t = (n_t/n_e)^2\ S_e$; Eq. 6-3, TR-25.

Shield's work establishes the critical relationship between the active and passive forces; i.e., it relates the critical fluid tractive stress (τ_c) for incipient motion to the gravitational resisting force. It was verified for coarse grained materials (d_m > 6 mm) by Lane's study of prototype field canals and for discrete particle material (d_m > .1 mm) by Report 108.

Lane reports:

Critical tractive stress, $\tau_c = 6\ d_m$; where d_m is in feet and τ_c is in psf.

This critical tractive stress is nearly identical with Shield's work for $d_m > \frac{1}{4}$ inch.

Lane recommends:

Allowable tractive stress, $\tau_{all.} = 4.8\ d_m$; d_m, τ_c same units as above.

This allowable tractive stress is conservative with respect to Shield's work for $d_m > 4$ mm and gives results identical to those from Eq. 6-5 of TR-25.

Report 108 reports:

Critical tractive stress, $\tau_c = 5\ d_m$; d_m, τ_c same units as above.

This critical tractive stress is conservative with respect to Shield's work for $d_m > 4$ mm.

Report 108 recommends:

Allowable tractive stress, $\tau_{all.} = 4\ d_m$; d_m, τ_c same units as above.

This allowable tractive stress is conservative with respect to Shield's work for $d_m > 2$ mm and gives results identical to those from Eq. 24 of TR-59, setting the FS value equal to 1 and using the default value for C_{50}.

For armoring design analysis, the characteristic armor particle size (d_m) is chosen from the coarser portion of the original material since most of the fine material will be hydraulically removed. Usually $d_m = d_{90}$; therefore, m = 90. Furthermore, for design purposes, all material smaller than the d_m is assumed to be sorted out. Therefore, the depth of degradation (D_d) is

$D_d = d_m/[(100 - m)/100] = 10\ d_{90}$ (see page 6-31).

This assumption has a physical interpretation. The d_{90} size of the original bed material (before armoring) will become the d_{50} of the final exposed surface bed material (after armoring).

Example

This example illustrates the armoring design concept. The uniform flow-unit slice assumption was made for convenience in computing the depth of flow; it may not be valid for most field applications. Furthermore, it is not a conservative assumption, from a stability viewpoint, for subcritical but supernormal flows. Also, the choice of the numeric value for the modifying value (n_o), which accounts for energy loss due to factors other than boundary roughness, should be determined reach-by-reach for each application. (See NEH-5, Supplement B, for guidance). The smaller the n_o value, the more conservative the design from a stability viewpoint.

Problem: A concrete emergency spillway is planned to discharge onto an alluvial valley floor of at least 6 feet of homogeneous material. What maximum steady-state unit discharge would limit scour by permitting armoring to the d_{90} size material? What would be the expected depth of scour? The valley slope (S_o) is 0.00520 ft/ft, the d_{90} is 110 millimeters, and the modifying value (n_o) is assumed to be 0.005. Assume uniform flow-unit slice principles are applicable; therefore, the hydraulic radius is equal to the depth of flow ($y = R$), the rate of total energy loss is equal to the valley slope ($S_e = S_o$), and the actual transverse tractive stress is uniformly distributed ($\overline{\tau}_{act.} \leq \tau_{all.}$). Use the recommended allowable tractive stress formula from Report 108 that is compatible with TR-59. Use $K_m = 0.0395$.

Given:

$S_o = 0.00520$

$m = 90$

$d_m = d_{90} = 110 \text{ mm} = 0.3609 \text{ ft.}$

$n_o = 0.005$

Required:

(a) q_{max} for $\overline{\tau}_{act.} = \tau_{all.}$

(b) D_d for $m = 90$

Solution:

(a) $n_t = K_m d_m^{1/6}$

$= 0.0395(0.3609)^{1/6}$

$= 0.0333$

$n_e = n_t + n_o$ (see 7th step, page B.6, Supplement B, NEH-5)

$= 0.0333 + 0.005$

$= 0.0383$

$S_e = S_o = 0.00520$

$S_t = (n_t/n_e)^2 \; S_e$

$= (0.0333/0.0383)^2 \cdot 0.00520$

$= 0.00393$

$\tau_{all.} = 4 \; d_m$

$= 4 \cdot 0.03609$

$= 1.444$ pounds per sq. ft.

$\tau_{act.} = \tau_{all.}$

$\gamma R \; S_t = 4 \; d_m$

$R = (4 \; d_m)/(\gamma \; S_t)$

$= 1.444/(62.4 \cdot 0.00393)$

$= 5.884$ ft

$y = R = 5.884$ ft

$q_{max} = (1.486/n_e) y^{5/3} \sqrt{S_o}$

$= (1.486/0.0383)(5.884)^{5/3} \sqrt{0.00520}$

$= 53.6$ cfs/ft

(b) $D_d = d_m/[(100 - m)/100]$

$= 10 \cdot 0.3609$

$= 3.61$ ft

Therefore, the maximum steady-state unit discharge that would limit scour by permitting armoring is approximately 54 cfs/ft. The expected depth of degradation before complete armoring (one layer) is almost 4 feet.

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Chapter 7

ENVIRONMENTAL CONSIDERATIONS IN CHANNEL DESIGN, INSTALLATION, AND MAINTENANCE

Introduction

A channel improvement project causes a change in the surrounding environment. The change may be abrupt or gradual; localized or broader in impact; biological, social, or cultural in nature. Whatever the changes they must be recognized and considered in arriving at the decision to modify a channel. The optimum channel improvement project is one that is based upon a careful assessment of the functional demands of the project, the needs for protection and enhancement of affected resources, and a justification that considers environmental and economic values.

The information presented here provides guidance for the proper recognition, protection, and enhancement of the affected fish, wildlife and recreation factors, during the planning, design, installation, and maintenance of channel projects.

Scope

The decision to develop a channel project will be made only after careful consideration of the impact of the project upon the environment. Once the decision has been made that a channel project is necessary, it is then equally important that the channel designer, builder, and those responsible for maintenance recognize the environmental factors that may be affected by the project. All those involved must know the techniques and measures available to protect and enhance these environmental values. This is the primary purpose for the material that follows.

Fish, wildlife and recreation factors are listed and described. Additional factors will need to be identified in local areas. The factors have not been rated or priorities assigned. This is the job of the appropriate specialist; i.e., engineer, biologist, forester, agronomist, range conservationist, geologist, recreation specialist, landscape architect, naturalist, etc. The development of the design requires a joint effort by all the concerned disciplines and the sponsors of the project. The final product then will result in minimized detrimental effects and, when possible, in the enhancement of some of the factors. There may be occasions when the decision to install a channel project may have to be reconsidered after all the factors are carefully evaluated.

The techniques and measures are presented to provide guidance in the development of the project in a manner that will protect or enhance the environmental factors identified as being affected by the project. These techniques and measures are not criteria. The features or factors under consideration in a given area generally will dictate the criteria for the project. The designer, builder, and those responsible for maintenance in their use of the various techniques and measures must constantly be aware of the functional, economical and environmental aspects of the project.

The functional and economic aspects of a project cannot be separated from the environmental aspects. Drainage, flooding, vector and phreatophyte problems, and the economical aspects related to these problems, also are critical "environmental factors." They must be given consideration at the time the decision is made on the need for a channel.

A. Wildlife Resources

The fundamental needs of wildlife are food, cover, and water. Food is needed at all seasons of the year, but the types of food vary between seasons. Spring nesting and winter escape cover are critical items for most species. Seldom can a wildlife species find its needs in a single vegetative type. Thus, a mixing or interspersion of vegetative types is important within the daily cruising range of a particular species. One or more important elements of wildlife habitat always are found along or adjacent to streams. Frequently, critical woody cover is present next to the stream.

The needs for food, cover, and water vary depending upon the species. Guidelines for meeting these needs should be established by States or areas. (See Appendix D. for sample.)

B. Fish Resources*

1. Water quality is a limiting factor for fish production. The individual States have established water quality standards in line with national guidelines. The following items affect fish species suitability, production, and survival.

    a. Temperature is an important physical factor. Summer water temperatures commonly vary as much as 10° in a 24-hour period. In general, summer temperatures should be between 50° to 70° for cold-water species. Egg hatching success is best for trout between 45° to 55°. Warm-water species need summer temperatures between 70° to 90°.

    Removal of shade tends to raise water temperatures while the maintenance of vegetation for shade may keep the water cooler. Water temperature is raised when velocity and depth are reduced. Water temperature may be affected by release of water from upstream impoundments.

* See Appendices B and C for Fish Stream Investigation Guides.

b. Turbidity caused by inorganic material, such as clay, is detrimental to fish production. Such material destroys spawning areas by sedimentation and reducing growth of bottom organisms. Adult fish generally can withstand high levels of turbidity for short periods of time, but prolonged exposure may cause mortality.

   It is reported that turbidity as high as 245 mg/L is not harmful to fish. In fact, fish thrive in water with turbidities that range over 400 mg/L and average 200 mg/L. Turbidities of 3,000 mg/L are considered dangerous to fish when maintained over a 10-day period. Trout eggs were destroyed with 2,000 mg/L turbidity for six days. Symptoms of fish stress appear as turbidity approaches 20,000 mg/L; death between 50,000 and 200,000 mg/L. At turbidities causing death, the opercular cavities were found to be matted with soil and the gills had a layer of soil in them.

c. Oxygen requirements for subsidence of fingerling and adult salmon and trout are about 6 p.p.m. dissolved oxygen. Incubating eggs require a minimum of 8 p.p.m. Warm-water species require about 3 p.p.m. Water at or near oxygen saturation, for its temperature and elevation, is always satisfactory. Oxygen is put into water by direct absorption from the atmosphere, photosynthesis of growing plants, and by tumbling action of stream or waterfalls and turbulence generated at drop inlets or drop spillways. Turbidity, reduced flow, and non-tumbling action reduce oxygen.

d. Carbon dioxide is another of the basic factors determining productivity of waters. It is necessary in photosynthesis and for keeping minerals, such as calcium, in solution. High carbon dioxide levels reduce the ability of fish to take up oxygen and to dispose of carbon dioxide from the body. Concentrations of carbon dioxide should be kept below 25 p.p.m.

   Carbon dioxide is put in water by direct absorption from the atmosphere, decomposing organic matter, and respiration of plants and animals. It is removed by photosynthesis, agitation of water, evaporation, and rise of bubbles from depths.

e. pH is a measure of the acid intensity in water. The scale of reading is from 0 to 14. Optimum fish production lies between 6.5 and 8.5. Values below 5 and above 9 affect the ability of fish to take oxygen from the water source. Water pH is changed if an acid layer of soil is exposed in stream bottom or sides.

2. These stream channel features affect fish production, species suitability, and survival.

   a. Bottom Material - The bottom material of a stream is important from the standpoint of food production and natural spawning. The following yield in grams of food per square foot in terms of different stream bottom m materials has been recorded: silt - 3.07; cobble - 2.47; coarse gravel - 1.51; fine gravel - 0.93; and sand - 0.1.

      Coarse and fine gravel beds in riffles are best for trout to deposit their spawn successfully. Most warm-water fish spawn in sand or silt beds in water less than 3 feet deep and with little or no current.

   b. Water Types

      (1) Riffle - Section of stream containing gravel and/or rubble, in which surface water is at least slightly turbulent and current is swift enough that the surface of the gravel and cobble is kept fairly free from sand and silt.

      Riffles are essential for trout spawning and food production. Riffles should occur at intervals equal to every 5 to 7 channel widths. The current in the riffle should be swift enough to carry away sediment. The bed material in riffles should be larger than in pools so as to provide for aeration of the water. A water depth of 6 inches is desirable.

      (2) Pool[1/] - Section of stream deeper and usually wider than normal with appreciably slower current than immediate upstream or downstream areas and possessing adequate cover (sheer depth or physical condition) for protection of fish. Stream bottom usually is a mixture of silt and coarse sand.

      Pools are valuable as resting and refuge areas. Some surface feeding also is done.

      (3) Flat[1/] - Section of stream with current too slow to be classed as riffle and too shallow to be classed as a pool. Stream bottom usually composed of sand and finer materials with coarse cobbles, boulders, or bedrock occasionally evident.

      (4) Cascades or Bedrock - Section of stream without pools, the bottom consisting primarily of bedrock with little cobble, gravel, or other such material present. Current usually faster than in riffles.

[1/] Warm-water streams normally contain only these water types.

c. Stream Side Vegetation - This item pertains to the relation of vegetation to stream shade and fish shelter. Low shrubs and grasses provide shade for small streams, but do not over-shade them. Such vegetation does not clog streams by falling in the water, and it provides hiding cover for fish if allowed to hang over the bank into the water.

Trees are necessary for shade along streams over 30 feet wide since low shrubs and grasses shade only a small portion of this width.

An ideal situation, along small streams, is enough trees for aesthetic purposes and low shrubs and grasses providing shade and cover. Along large streams, trees for about 40 percent of the stream length, on both sides should be present. There probably are situations where the presence of trees well back from the water's edge furnishes shade almost as good as comparable ones closer to the stream. This would be true especially on the east side of north-south flowing stream and the south side of east-west streams.

d. Velocities - Tolerable water velocity for fish is governed by several factors, chiefly, by the species of fish, size of fish, and the distance and frequency of resting areas. Boulders, pools, deflectors, etc. provide resting areas.

C. Recreation Resources

There are numerous opportunities for recreation along and in channels. Many of these opportunities have been discussed under the heading of wildlife, fish, or aesthetic resources.

For optimum use a recreation resource must satisfy several key factors:

1. Proximity - All recreation activities are distance-related with respect to the user's home. Generally the greater the population within a 50-mile radius (or an hour's drive), the more the area may be used.

2. Access - The public road system needs to provide access to the potential recreation resource. The degree of access of the area can be gauged by determining the portion that is within one mile of an all weather road. Again, the greater the degree of access the more likely the recreation area will be used.

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3. Ownership - The ownership and land-use pattern of the area have a bearing on the potentials for developing recreation opportunities. The area must be of sufficient size to support a public recreation activity. The landowner must be interested. The area will have to be accessible to the public for heavy use. Sometimes the resource will be used by the landowner and his family.

Specific factors for each activity are:

1. Fishing - Stream fishing has a special quality for some fisherman. All of the environmental factors listed under "Fish Resources" are essential if fishing opportunities are to be provided.

   Many factors besides the abundance of fish and accessibility enter into stream fishing quality. The unique scenic setting, sounds of nature, the sight of a riffle or pool, the wooded reach, the open meadow, and overall diversity of a landscape are qualities appreciated by the stream fisherman and enjoyed by many others. Streams that are partially brush and tree lined offer the fisherman the opportunity to exercise his skill.

2. Hunting - Hunting opportunities often are enhanced by the presence of streams or channels. In some localities the vegetative cover along channels (brush and trees) provides the only cover available for hunting. The brush and trees provide, for some species, the only avenues for hunting, escape cover from predators, or the opportunity for protected movement. The grasses and legumes provide the essential nesting cover required for ample populations. All of the environmental factors already described under "Wildlife Resources" are essential if the hunting opportunity is to be established.

   The combination of food, cover, and water enhances the opportunities for hunting success.

   Channels and the accompaning marshes offer opportunities to hunt many species of waterfowl, rail, and woodcock. In some areas they provide the only habitat for these species.

3. Swimming - Swimming in streams and channels is an activity practiced in many parts of the country, particularly if ponds and lakes are relatively scarce. Desirable environmental factors are good water quality, pH between 6.5 and 8.3, coliform count below 800, clear water with minimum in flow of 650 gallons of water per bather per day (number of bathers $\div$ 1,000 = inflow cfs). State regulations may require higher standards. Shade and desirable soils (sands) will enhance the desirability of the area.

   Stream "pools" may be developed fully with all facilities (bathhouse, beach, etc.) present or they may be simply the local swimming "hole."

Depth should be at least 5 feet, greater if diving is permitted. The shore line should have a slope of less than 10 percent, 2-4 percent is best.

4. Boating and Canoeing - Boating streams should be at least 2 feet deep for rowboats or 3 feet for boats with outboard motors. A good width is at least 2-1/2 times the length of boat allowed. Narrower streams, however, can be utilized. Stream channels with minimum depths must be free of obstructions.

   Canoeing streams may have depths as shallow as 6 inches for short stretches or 18 inches for a major portion. Canoeists are not adverse to portaging (carrying) for short stretches where water is too shallow. Good widths are 17 feet, but widths of 6 feet are acceptable. Some authorities recommend an average flow of 100 cfs in order to be suitable but this is dependent upon depth, width, and gradient. While many canoeing streams have white water and pools in combination, flat water streams attract thousands of users for canoeing and boating alike.

5. Hiking and Walking - Many of the same characteristics that enhance a canoeing or boating stream are desirable for hiking along a stream. Cascades, riffles, white water and pools, shrubs and trees with a variety of color and shape add to the aesthetic value.

   Over-water walkways and bridges which permit extensive observation are interesting features. A trail should be approximately 4 feet in width, sufficient to allow two people to walk side by side. A grade of 10 percent or less is recommended.

6. Painting and Photography - The thousands of photographs and paintings depicting landscape scenes with streams attest to man's interest in viewing his environment. Natural curves, a variety of landscapes, water courses meandering through a variety of vegetative types and vistas all provide professional and amateur artists with ample subject matter.

7. Camping - The environmental factors previously discussed may be used as guides. Vacation camping may be a profitable recreation activity if other recreation opportunities (fishing, swimming, boating, etc.) also are available. Transient campground may be feasible without these activities if the size is within 3 miles of a major highway. In either case the necessary land area is 10-15 acres. Soils in the area should be suitable for septic tanks and roads. An adequate potable water supply should be available. Characteristics that limit an area's usefulness for camp sites are susceptibility to flooding, impermeable hardpan layers, shallow soil over bedrock, restrictions to natural drainage, erosion hazard, and inability to support and sustain vegetative cover. Slopes should be less than 15 percent, preferably less than 8 percent.

8. Botanizing - On occasion, an area adjacent to a water course may be of particular interest to botanists and other nature lovers. The species of plants may, but need not be, of rare or exotic nature.

9. Bird Watching - The environmental factors discussed under "Wildlife Resources" should be used in determining whether the opportunities for bird-watching exists. The thousands of birdwatchers in the country often visit water courses during their bird counts. In some areas the plant species may compose a specialized habitat for a particular species of bird. Fields of one crop reduce the number and species using the area. The shrubs and trees bordering a channel have added value in crop areas. Landscape variations are much more attractive to songbirds than areas of a single crop.

10. Specimen Collecting - Artifacts from cache pits or Indian mounds, fossils, decorative rocks, or desirable mineral specimens are of interest to a number of people, institutions, and agencies. Collectors also may find driftwood, burntwood, and tree roots of value or interest.

A guide for evaluating channels for general recreation development is included in Appendix A. See Fig. 1

Protection and Enhancement Techniques and Measures

Design

1. Alignment, Capacity, and Grade - Channels generally will follow existing alignment except where stability, environmental, or cost factors clearly dictate an alternative course. For instance, a section of the channel or floodway may be relocated in order to bypass important fish or wildlife habitat.

   Natural streams and constructed channels need to convey water discharges of all magnitudes from base flow through floodflow without significant damage to the channel or to fish habitat. In order to protect a desirable existing stream channel, higher frequency floodflows could be carried out of banks or on a separate alignment. As the floodflow channel would be dry most of the time, it could be designed to include farming or reforestation within the right of way.

   Water often is used as a receiving medium for various waste discharges and yet its quality must be maintained so that it is suitable for instream recreational uses as well as out of stream needs for municipal water supply, irrigation, cooling, washing, and dilution. Reservoir releases of stored water can provide low flow augmentation to prevent waste discharges from exceeding acceptable concentrations and to provide recreation and fish habitat water requirements. Oxbow and wetland flood storage, with controlled releases, also can be used for this purpose.

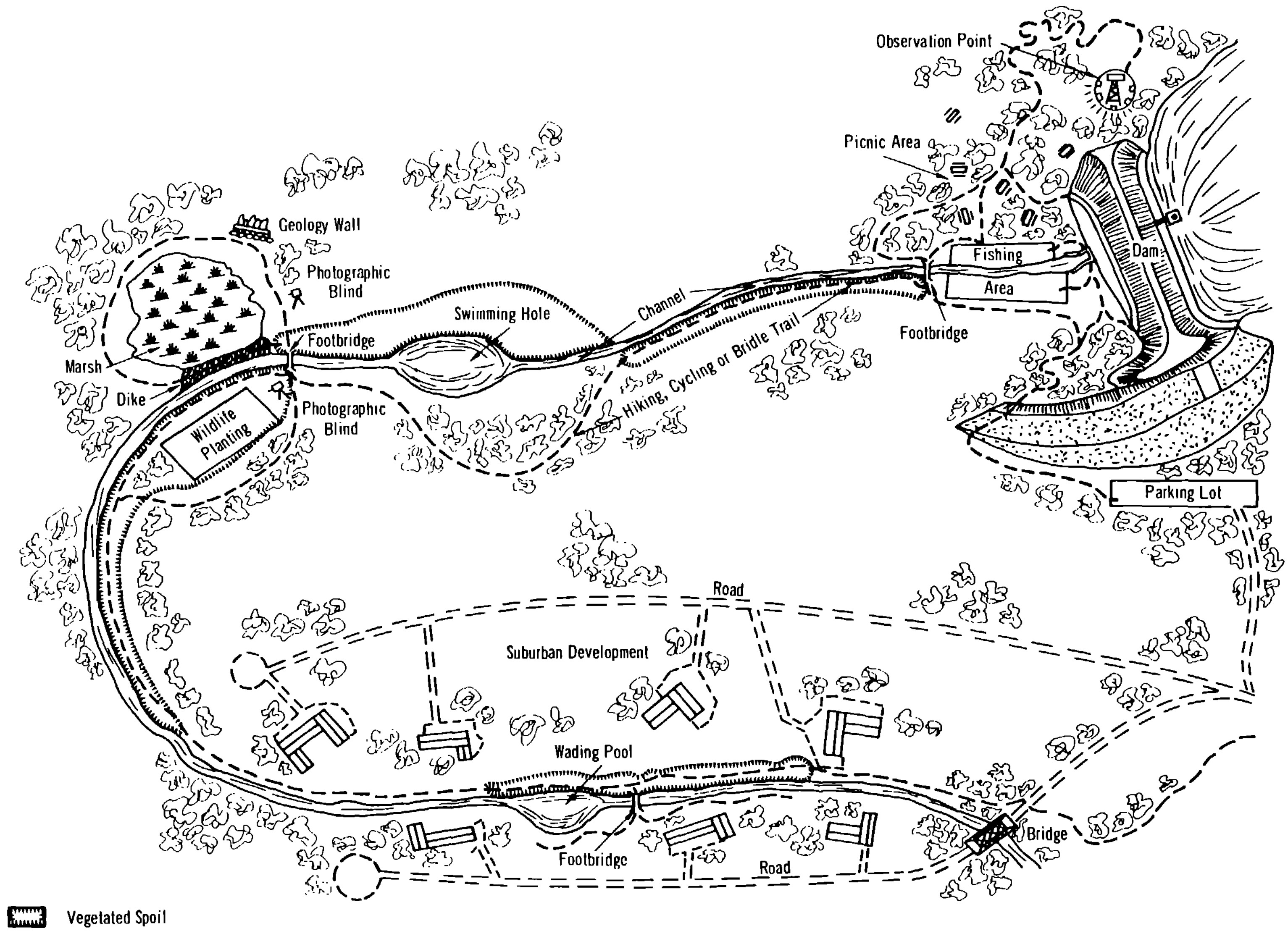


Figure 1 CHANNEL RECREATION POSSIBILITIES

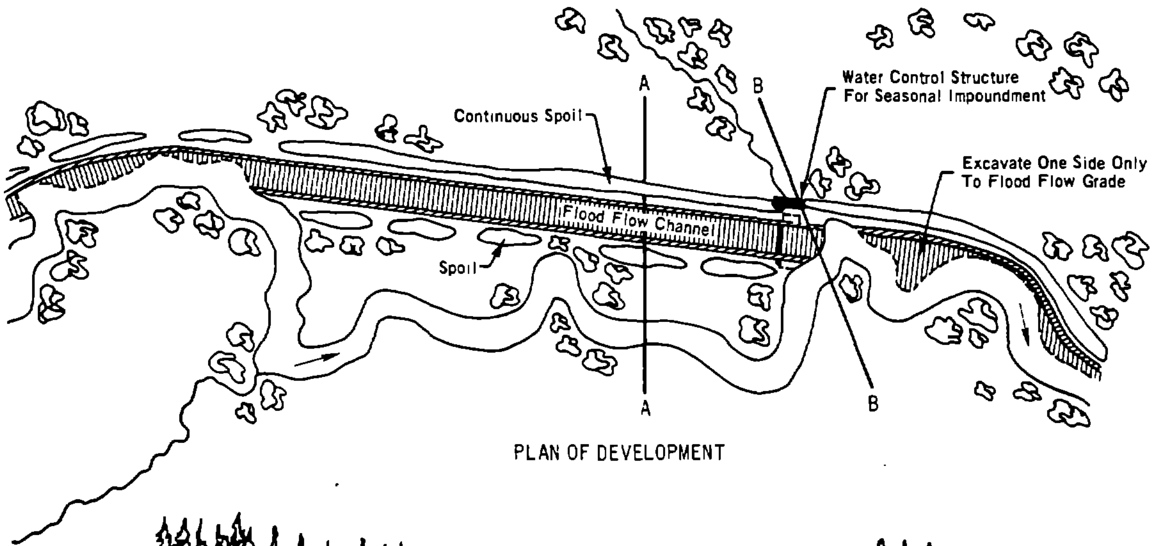
Sediment loads delivered to the channel by tributary streams or other sources need to be controlled as part of the overall design. A stable channel has a limited sediment carrying capacity and downstream uses of the water may necessitate further limitation of sediment load. Sediment traps may be needed at delivery points or at intermediate points along the channel to provide the required water quality and stability.

In certain reaches, the channel slope may need to be flattened to obtain stability in highly erosive soils or can be steepened to make maximum use of erosion resistant soils. Where gravel armoring or riprap is needed, or just available, full use should be made of its ability to withstand higher velocities. The channel slope variations and rock protection will allow for the inclusion of pools and riffles and also provide control of meander development.

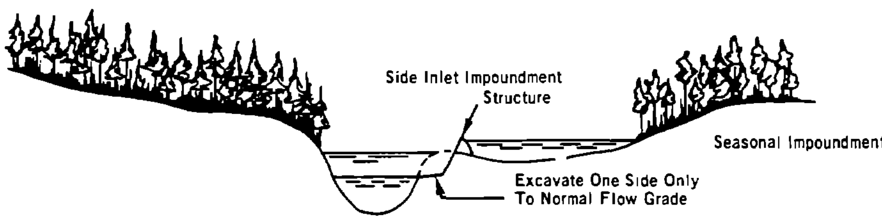
2. Channel Section - The low flow channel cross section should approach a natural stream condition. (Figure 2) The side slopes and bottom width can be allowed to vary to take advantage of existing conditions. Where possible, the side slope on the outside of the channel curve could be steepened and the side slope on the inside of the channel curve flattened to duplicate a naturally developed sinuous waterway. Use the onsite large boulders in riprap sections or at selected points for fish habitat development. Large slab rocks or boulders can be used to create near vertical banks and on trout streams for wing deflectors and bank cover devices to improve the fishing potential. (Figure 3) The channel bottom width can be varied in conjunction with the bed slope to aid in the development of deep pools, cascades, low velocity sections, and sections of high velocity rips that would simulate natural conditions and also take advantage of localized variations of in-bank capacity and stability. Width restrictions also could be satisfied in this manner.

   Figure 4 shows the cross sections, meander pattern, bed contours, and bottom profile that can develop in a natural stream. The meander parameters shown are average values but could be used for preliminary proportioning and alignment of a constructed channel. Other factors, such as discharge, character and amount of bed load, general valley slope and the resistance of bed and banks to erosion, also need to be considered to develop the final layout.

3. Spoil Placement - Channel excavation spoil should be utilized in a manner most appropriate for the controlling reach conditions. In general, excavated materials should be placed so as to reduce to a minimum the required clearing and disturbed areas and to provide wildlife habitat. (Figure 5) The template sections shown on the drawings and the specifications should provide guidance as to typical sections, approved spoil disposal methods, maintenance limits and construction limits.

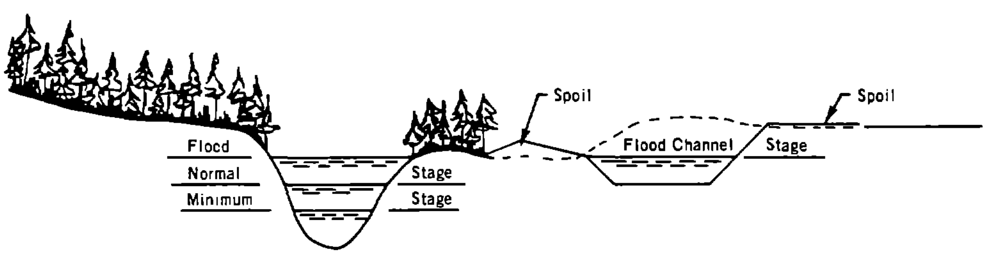


PLAN OF DEVELOPMENT



Cross Section B–B

ONE-SIDED CONSTRUCTION FOR ADJACENT STREAM CHANNEL & FLOOD FLOW CHANNEL



Cross Section A–A

UNDISTURBED STREAM CHANNEL & SEPARATE FLOOD FLOW CHANNEL

Figure 2 SEPARATE LOW FLOW AND FLOOD FLOW CHANNELS

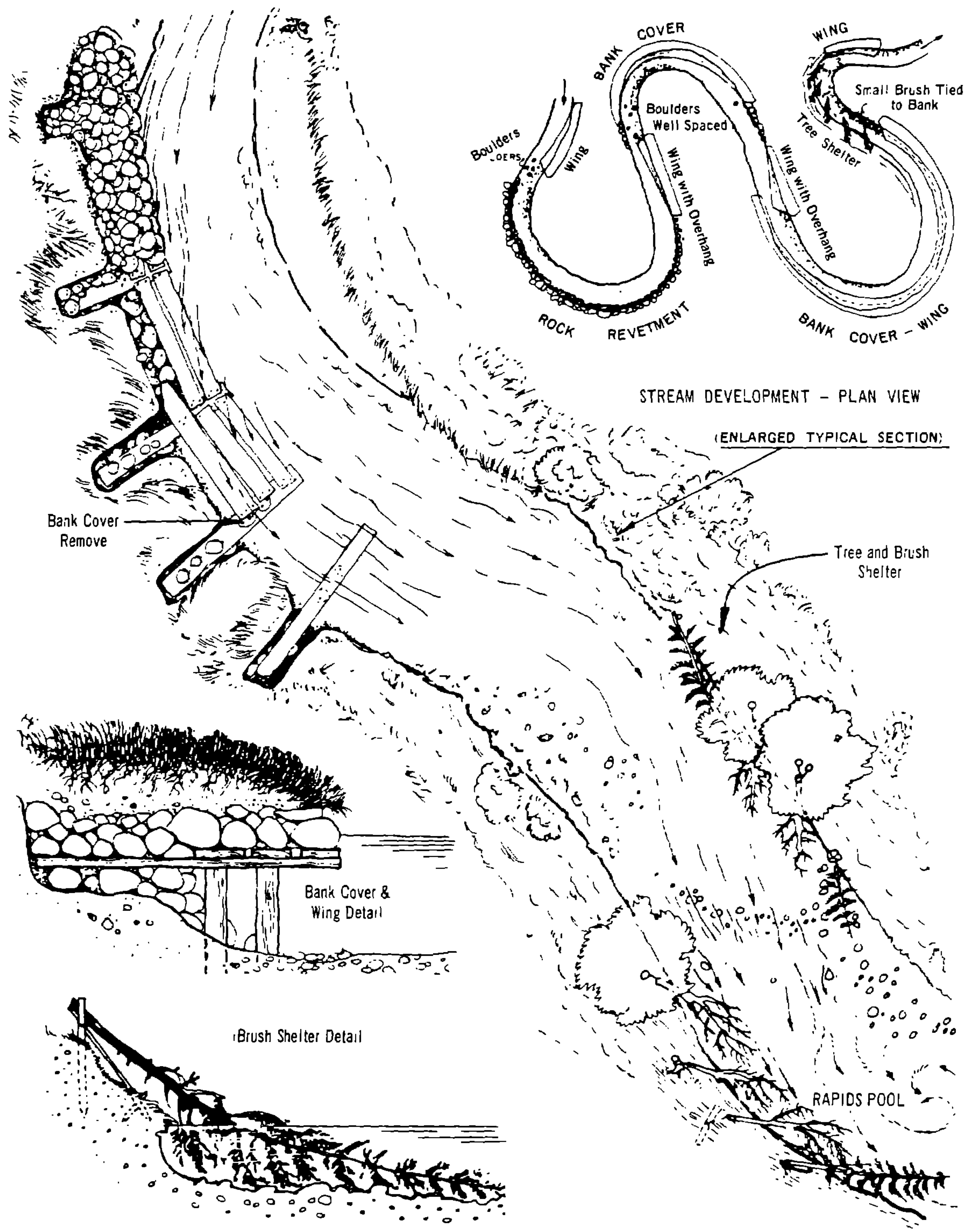
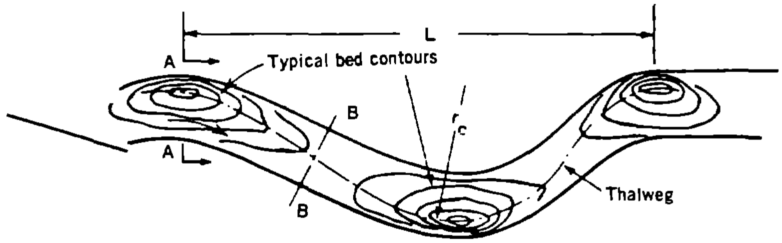
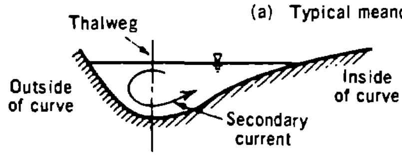


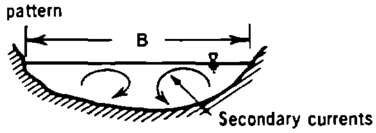
Figure 3 FISH HABITAT DEVELOPMENT



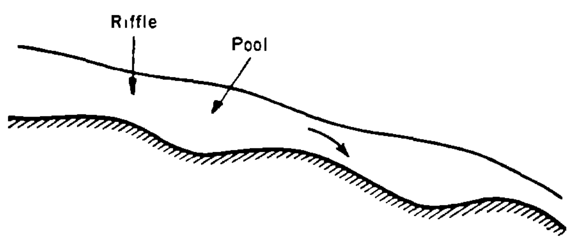
(a) Typical meander pattern



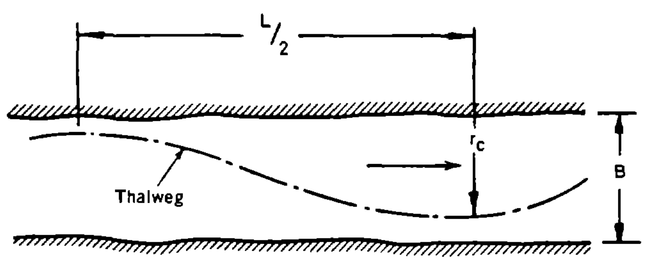
(b) Section A–A



(c) Typical section of straight channel, or crossover B–B



(d) Pool–and–riffle formation (VERTICAL MEANDER PATTERN)



(e) Concealed meander pattern in a straight channel

r_c = radius of curvature in Thalweg

APPROXIMATE MEANDER PARAMETERS

$\frac{L}{B} = 7$ TO 11

$\frac{r_c}{B} = 2$ TO 3

Figure 4 NATURAL CHANNEL - MEANDER PATTERNS WITH POOL AND RIFFLE DEVELOPMENT

Figure 5 SPOIL BANK DEVELOPMENT FOR WILDLIFE HABITAT

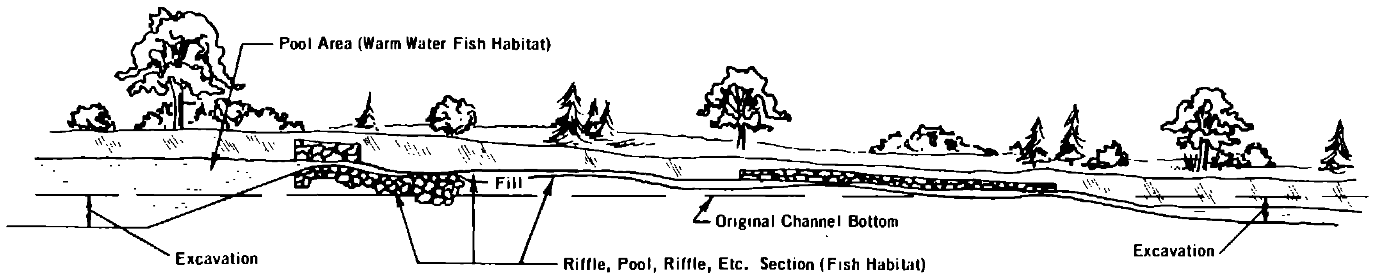
4. Structural Measures - Channels with flat enough gradients so that stability is not a design problem could, if desirable, be laid out in pond-riffle-pool manner. The additional grade provided by ponding would allow the inclusion of a series of riffles and pools that could add to the fish habitat. See Figure 6.

5. Vegetation

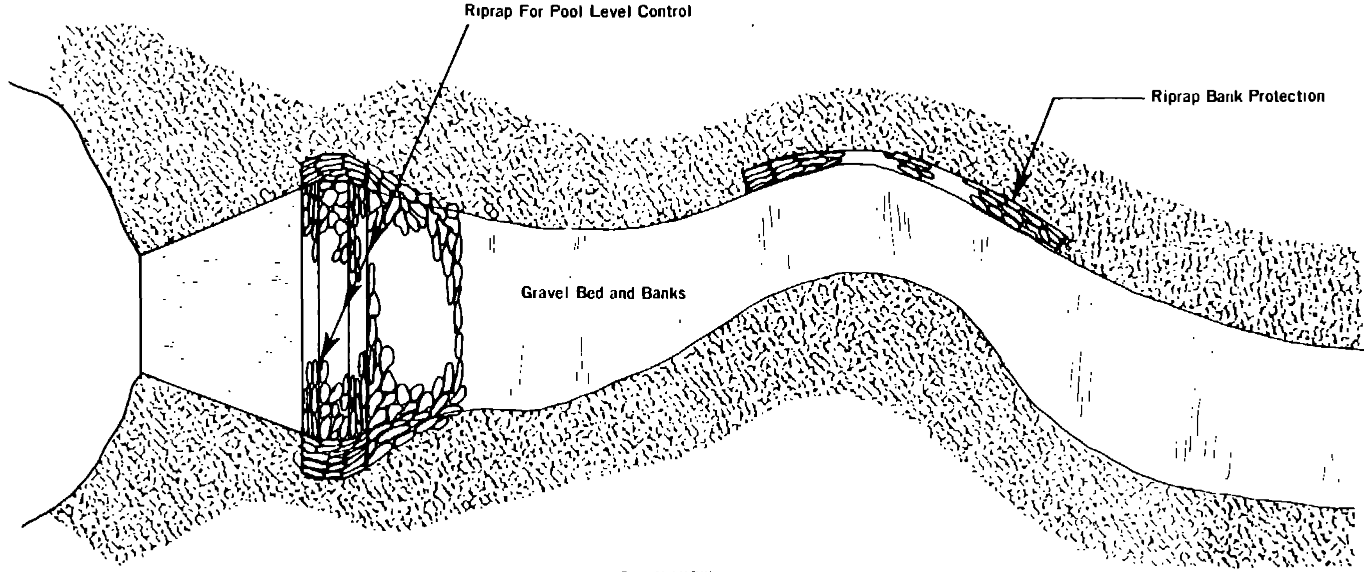
   a. Wildlife habitat seedings and plantings can be used outside the channel section, on the spoil sections or in odd corners of suitable size where the wildlife potential can be enhanced and the disturbed areas stabilized. Wildlife habitat mitigation areas, when included, and the normal habitat seedings can be laid out in discontinuous blocks, irregularly sized to provide a maximum of edges.

   b. Recreation areas could be selected for special treatment that would greatly add to the utility of the project. Functional planting can be used to screen noise and direct pedestrian traffic.

      Figure 1 shows some examples of activity areas that might be included along a channel. A hiking, cycling, or bridle trail along the berm, or stream side fishing supplemented with a few picnic tables for the rest of the family. A dike to preserve a marsh enhanced by a wildlife planting. A swimming "hole" for light use or a wading pool in a suburban development. "Green" areas through suburban areas can be developed into playgrounds or neighborhood parks. Long channels with sufficient depth may be used for canoeing.



CHANNEL PROFILE – BUILT IN GRADE AND BOTTOM CONFIGURATION



PLAN VIEW

Figure 6 CHANNEL SEGMENT WITH POND, RIFFLE, POOL LAYOUT

APPENDIX A

Evaluating Channels for Recreation Developments

A copy of the chart and work sheet used in this approach follow. The use of this approach requires a cursory on-site investigation. Ratings are determined on the basis of judgment and estimations.

It should be noted that this evaluation is for a "general" recreation development. Evaluation of a site for specific recreation activities entails consideration of many additional key elements or a change in the criteria already outlined. Water flow criteria, for example, would be different if swimming, canoeing, or hiking were the specific activities under consideration. Additional key elements, such as length of channel, size of adjacent land area, width of adjacent land area, soils, depth of water are some additional key elements required for specific activities. Recreation developments should be planned to preserve and intensify the landscape character. Planning should not introduce a disturbing, inharmonious item into the landscape, whether it is a facility or an activity. The planner should ensure that the planned project comprises a complex of functions related to the best features of the site. For this reason, evaluation and planning for specific recreation activities should be undertaken by the appropriate specialist.

EVALUATING CHANNELS AND ADJACENT AREAS FOR RECREATION DEVELOPMENTS

Key Elements	Multiplier M	Rating (R)			
		High 4	Moderate 3	Fair 2	Poor 1
Water Quality	4*	No pollution coli count below 200	ph 6.5-8.3 coli count 200=600/100 ml	ph 8.3-9 or 5-6.5 coli count 600=800/100 ml	ph 9 + or 5 - coli count 800 +/100 ml
Aesthetics (vista, natural attractions nearby, visual appearance of channel)	4*	Excellent	Good	Fair	Poor
Water Flow	4*	Little variation with riffles and pools	Little variation	Intermittent	Lacking sufficient flow during primary use period
Adjacent Land Flora	3	Wooded or open with 12" trees forming a 10-20% canopy	Open with grass and small trees forming a 10-20% canopy	Open with grass and brush	Open - grass only
Distance From Users	3	Under 1/2 Hr.	1/2-3/4 Hr.	3/4-1 Hr.	Over 1 Hr.
Distance to Surfaced Road	2	Under 3 miles	3-5 miles	5-10 miles	Over 10 miles
Width of Channel at Normal Water Surface	2	20' +	11-19'	6-10'	6' -

Formula $\Sigma(M \times R)$ = Score

Maximum Possible Score 88
High Potential 67 - 88
Medium Potential 45 - 66
Low Potential 22 - 44

* If any key element with a multiplier of 4 is rated as "Poor" (1) that element must be considered limiting. Further consideration of other key elements is unnecessary. The area is generally considered unsuitable for recreation development.

SAMPLE WORKSHEET

EVALUATING CHANNELS AND ADJACENT AREAS FOR RECREATION DEVELOPMENTS

State ________________ County ______________ Township _________

River Basin _________________________ Watershed _________________

Location or Job No. ______________________________

Key Elements	Multiplier (M)	x	Rating (R)	
Water Quality *	4	x		=
Aesthetics *	4	x		=
Water Flow *	4	x		=
Adjacent Land Flora	3	x		=
Distance From Users	3	x		=
Distance to Surfaced Road	2	x		=
Width of Channel at Normal Water Surface	2	x		=

Total Score ________

Maximum Possible Score	88
High Potential	67 - 88
Medium Potential	45 - 66
Low Potential	22 - 44

* If any key element with a multiplier of 4 is rated as "Poor" (1) that element must be considered limiting. Further consideration of other key elements is unnecessary. The area is generally considered unsuitable for recreation development.

APPENDIX B

Sample
Fish Stream Investigation Guide

Introduction

This guide provides a systematic approach to fish stream investigation.

Ten stream features are used to obtain a biological rating and six use factors are used to obtain a use rating.

Each stream feature is recorded and rated on a field work sheet, using predetermined criteria. The ten features have been assigned importance factors based on their individual importance to the total stream character. The final rating is weighted average of the individual stream features.

The use rating is a judgment based on six items related to fishing.

Instructions

Heading - Show watershed, major water courses, and important tributaries.

Biological Investigation

1. Designated Reaches - The appraiser should divide the stream into reaches from mouth to upper limit. The two designated points setting the limits of each reach should be easily identifiable on the ground by designated roads, natural markers, or points selected due to some physical land or stream characteristic. (No rating)

   Length - The length must be measured in feet. (No rating)

   a. Average width (ft.) is measured at normal flow or depth.
      Importance factor of 2.
      Rating - 1 point for each foot of average width. (Maximum 10)

   b. The acreage of the stream is calculated by multiplying the length (ft.) by average width (ft.) and dividing by 43560.
      Importance factor of 3.
      Rating - 3 points for 1/2 acre or less (warm water - 1 acre or less)
      3-5 points for 1/2 acre to 1 acre (warm water - 1-2 acres)
      6-10 points for 1 acre plus (warm water - 2 acres plus)

(Increase or decrease one point for each 1/4 acre) (warm water - 1/2 acre)

c. Flow - Constant flow is year-long and intermittent only a portion of a year.
Importance factor of 10.
Rating - Intermittent flow
2 points base score. Subtract 1 point for each 15-day period (or fraction thereof) without flow.

Constant flow
2 points base score. (Maximum 10)

Cold - Add 1 point for each 1 inch of average flow depth in riffle.

Warm - Add 2 points for each 5 inches average flow depth in riffle.

2. Water chemistry is measured in parts per million. Temperature is recorded in degrees F.

Water chemistry is a limiting factor and overrides all other factors if any quality condition falls outside fish requirements.

3. The pool riffle ratio is calculated by determining the feet of each reach in riffles, pools, flats, and cascades or bedrock and calculating the percent of total length. These are defined as follows:

Riffle - Section of stream containing gravel and/or rubble, in which surface water is at least slightly turbulent and current is swift enough that the surface of the gravel and rubble is kept fairly free from sand and silt. (Disregard bottom material for warm water stream.)

Pool - Section of stream deeper and usually wider than normal with appreciably slower current than immediate upstream or downstream areas and possessing adequate cover (sheer depth or physical condition) for protection of fish. Stream bottom usually a mixture of silt and coarse sand.

Flat - Section of stream with current too slow to be classed as riffle and too shallow to be classed as a pool. Stream bottom usually composed of sand or finer materials, with coarse rubble, boulders, or bedrock occasionally evident.

Cascades or Bedrock - Section of stream without pools, consisting primarily of bedrock with little rubble, gravel, or other such material present. Current usually more swift than in riffles.

The sums of a, b, c, and d should equal 100 percent.

Importance factor of 10.

247-527 O - 78 - 19

Rating - (Consider quality of pools and riffles for in-group rating.)

8-10 if pool-riffle ratio is at least 35 percent pools and 35 percent riffles.

4-8 if less than 35 percent of stream is in pools and 35 percent or more is riffles <u>or</u> if more than 35 percent is in pools and 35 percent or less is in riffles.

1-3 if less than 35 percent in pools and less than 35 percent in riffles (Must be in this range if intermittent flow.)

4. <u>The water source</u> - Place a check mark to indicate water sources for each reach. Number of sources probably will decrease as one progresses upstream.

   Importance factor of 5.
   If source is springs and seeps rate 10.
   If source is runoff drainage rate 3.
   If source is lakes or ponds rate 2.
   Any combination - 5.

5. <u>Dominant vegetation</u> - This item pertains to the relation of vegetation to stream shade and fish shelter, not wildlife habitat. Record in feet for right and left bank.

   Importance factor of 5.
   Rate each side independently and average rating.
   For each reach and total stream length award 2 points for each 10 percent of tree and shrub type. (Maximum 10)

6. <u>Turbidity</u> - Express as <u>clear</u> if bottom is distinctly seen through 4 or more feet of water, <u>slightly turbid</u> if bottom can be seen at from 1- to 4-foot depth, and <u>turbid</u> if bottom is only visible at less than 1 foot. (If organic stain of fertility prevents appraisal of this item, so note and disregard this item.)

   Importance factor of 5.

   Rating - Clear 10.
   Slightly Turbid 5.
   Turbid 1.

7. <u>Sediment</u> - This item reflects the amount of sediment deposits visible in the stream bottom. Sediment influences pool-riffle ratio, width, acreage, turbidity and possibly other factors. Therefore, it is felt the rating of these items will reflect the effect of sediment. (No rating)

8. Check all visible or detectable sources of pollution, including siltation, altering water quality.

Water pollution (as water chemistry) is a limiting factor if severe enough to affect fish life or cause undesirable aesthetic quality.

Final

Multiply rating assigned to each element by the importance factor, add totals, and divide by total of importance factors.

BIOLOGICAL SCORESHEET

Stream Feature	Importance Factor	x Rating	=	Score
1 a.	2	x ______	=	______
1 b.	3	x ______	=	______
1 c.	10	x ______	=	______
2 (Limiting factor)				
3	10	x ______	=	______
4	5	x ______	=	______
5	5	x ______	=	______
6	5	x ______	=	______
7 (No rating)				
8 (Limiting factor)				
	Total 40			______ Total

= Final Grade

$40 \overline{)\text{Total Score}}$

The final stream grade is based on a scale of 1 - 10 (10 being the highest possible grade). A grade of less than 5 indicates a stream with a low biological value. The process identifies characteristics by reach and by evaluating them, management needs and potential can be identified.

Use Investigation

1. Fish Resources - Name species and specify category.

2. Access - This is physical access due to terrain, streambank vegetation, aquatic vegetation, debris, etc.

3. Public Access - Indicate 0, 1, 2, or 3 in blank.

4. Ownership - Check if public ownership, etc.

5. and 6. Fishing Pressure and Success - Record based on local knowledge, special studies, use evidence, and information in 1 through 4.

The final rating indicates value of stream fishing area and is one of judgment made by the investigator based on the information in items 1 through 6. The present use rating may be limited by access factors and a potential use rating substituted for it if so desired.

Source References

1. Lagler, Karl F. (1952) Freshwater Fishery Biology, William C. Brown Company, Dubuque, Iowa.

2. Seehorn, Monte E. (1970) A Survey Procedure for Evaluating Stream Fisheries, 24th Annual Convention, Southern Division, American Fisheries Society, Atlanta, Georgia.

3. Unpublished - A Guide to Stream Appraisal, UD - RTSC, TSC Advisory BIOL - UD-11, 1967.

FISH-STREAM INVESTIGATION GUIDE
(Worksheet for In-Service Use Only)

Watershed:____________________ Stream:________________

Tributary to:__________________ Important branches or tribu-

taries:________ ________ ________ ________ __________

Biological Investigation

1. Designated Reaches:

			Length (Ft.)	(a) Av.Width (Ft.)	(b) Ac. - Area	(c) 1/ Flow
(1)	From__________	To__________				
(2)	From__________	To__________				
(3)	From__________	To__________				
(4)	From__________	To__________				
(5)	From__________	To__________				
(6)	From__________	To__________				
Etc.		Total				

2. Water Chemistry and Temperature

Reach	Pheno ALK	MO ALK	T. Hardness	T D S	pH	O_2	Temp - H_2O
1.							
2.							
3.							
4.							
5.							
6.							

Date______________________ Time ______________ Air Temperature__________ Weather __________

1/ Constant Intermittent
(with average depth)

3. Pool-Riffle Ratio

	Reach 1	2	3	4	5	6	Total	Percent
a. Ft. in riffles								
b. Ft. in pools								
c. Ft. in flats								
d. Ft. in cascades or bedrock								

4. Water Source(s)

	Reach 1	2	3	4	5	6
Runoff/drainage						
Lake/pond						
Springs/seeps						
Marshes						
Tide						

5. Dominant vegetation within 25 feet of normal water level on each side (Note: Identify all important types)

Dominant Vegetation Types	Reach 1 R	1 L	2 R	2 L	3 R	3 L	4 R	4 L	5 R	5 L	6 R	6 L
a. Marsh/bog plants												
b. Wild grass/weeds												
c. Shrubs(understory)												
d. Trees (overstory)												
e. Pasture or hay												
f. Crop field												
g. ____________												
Total												

6. Water clear, slightly turbid, or turbid:

1 ________ 2 ________ 3 ________ 4 ________ 5 ________ 6 ________ 7 ________
(Record for other than flood flow)

7. Degree of siltation: (Slight or none, moderate, or extremely heavy)
1 ________ 2 ________ 3 ________ 4 ________ 5 ________ 6 ________ 7 ________

8. Evidence of Pollution: (Check)

	1	2	3	4	5	6
a. Sewage						
b. Animal effluents						
c. Industrial wastes						
d. Eroding areas						

USE INVESTIGATION

1. Important fish resources
(Name: STK. = stocked annually; mig. = migrant (spawner); res. = permanent resident in reach)

1 ________( ) ________( ) ________( ) ________( ) ________( ) ________( ) ________( )
2 ________( ) ________( ) ________( ) ________( ) ________( ) ________( ) ________( )
3 ________( ) ________( ) ________( ) ________( ) ________( ) ________( ) ________( )
4 ________( ) ________( ) ________( ) ________( ) ________( ) ________( ) ________( )
5 ________( ) ________( ) ________( ) ________( ) ________( ) ________( ) ________( )
6 ________( ) ________( ) ________( ) ________( ) ________( ) ________( ) ________( )

2. Access for fishing us U = unlimited, R = restricted, O = none in reach:

1________ 2 ________ 3 ________ 4 ________ 5 ________ 6 ________ 7 ________

3. Public fishing is allowed to the approximate extent of O = none, 1 = less than 1/2, 2 = more than 1/2, 3 = all of rest

1________ 2 ________ 3 ________ 4 ________ 5 ________ 6 ________ 7 ________

4. Public agency has fishing ownership, easement, right-of-way on reach:

1 ________ 22________ 3________ 4 ________ 5 ________ 6 ________ 7 ________

5. Fishing pressure (Check)

	1	2	3	4	5	6
High	________	________	________	________	________	________
Moderate	________	________	________	________	________	________
Low	________	________	________	________	________	________

6. Fishing success (Check)

Good	________	________	________	________	________	________
Fair	________	________	________	________	________	________
Poor	________	________	________	________	________	________

APPENDIX C

POOR QUALITY RECOGNITION GUIDE

Quality Class No.	Pool		
	Length	Depth	Shelter 1/
1	Greater than a.c.w. 2/	2' or deeper	Abundant 3/
	Greater than a.c.w.	3' or deeper	Exposed 4/
2	Greater than a.c.w.	2' or deeper	Exposed
	Greater than a.c.w.	$<2'$	Intermediate 5/
	Greater than a.c.w.	$<2'$	Abundant
3	Equal to a.c.w.	$<2'$	Intermediate
	Equal to a.c.w.	$<2'$	Abundant
4	Equal to a.c.w.	Shallow 6/	Exposed
	Less than a.c.w.	Shallow	Abundant
	Less than a.c.w.	Shallow	Intermediate
	Less than a.c.w.	$<2'$	Intermediate
	Less than a.c.w.	2' or deeper	Abundant
5	Less than a.c.w.	Shallow	Exposed

1/ Logs, stumps, boulders, and vegetation in or overhanging pool or overhanging banks.

2/ Average channel width.

3/ More than one-half perimeter of pool has cover.

4/ Less than one-quarter of pool perimeter has cover.

5/ One-quarter to one-half perimeter of pool has cover.

6/ Approximately equal to average stream depth.

APPENDIX D

Habitat Requirements

(The material below was developed to illustrate the type of information that the designer needs to have available. These data can be prepared for States or Regions and then can cover the appropriate species.)

Listed below are some specific habitat requirements for the more common wildlife species. These descriptions can be used to identify existing habitat, judge quality, and determine effects of modification.

1. Pheasants - A favorable land use pattern for pheasants consists of: (a) 60-80 percent in grain and seed crops, (b) 10-30 percent in grasses and legumes, (c) 5-10 percent in brush and woods, and (d) 3-5 percent in permanently protected herbaceous weedy cover.

   Crop fields ten acres or more in size, growing barley, buckwheat, corn, grain sorghum, oats, proso millet, soybeans, or wheat, should dominate the area. At least two acres of grasses and legumes (alfalfa, crownvetch, orchardgrass, reed canarygrass, sericea lespedeza, smooth brome, switchgrass, or timothy) per 100 acres should be present. Extra growth should be present on at least two 1/8 acre plots within 100 feet of a vegetative change. It is desirable to have at least one 1/2 acre strip of annual weeds per 100 acres and at least 1 acre of brushy thickets or woods.

2. Bobwhite Quail - Bobwhite usually thrive best where there are numerous small fields of grain and seed crops, interspersed with grassland, weedy patches, and brush or woodland areas. They seldom are abundant in extensive and continuous areas of cropland, grassland, or dense woodland.

   At least one-half acre of grain and seed crops (corn, cowpeas, lespedeza, millet) should be present within 100 feet of woody cover per each 100 acres of habitat or a one-half acre patch of wild herbaceous plants (panic grass, ragweed, croton, partridge pea). A combination of the two is best. Unmowed grass areas, 1/10 to 1/4 acres near food and cover, provide necessary nesting sites for a covey range. Woody cover totaling about 1 acre per 100 is necessary.

3. Ruffed Grouse - This grouse is a bird primarily of woodland edges and openings rather than dense woods. Hardwood trees should predominate. Hardwood trees, such as alder, apple, beech, birch,

APPENDIX C

POOR QUALITY RECOGNITION GUIDE

Quality Class No.	Pool		
	Length	Depth	Shelter 1/
1	Greater than a.c.w. 2/	2' or deeper	Abundant 3/
	Greater than a.c.w.	3' or deeper	Exposed 4/
2	Greater than a.c.w.	2' or deeper	Exposed
	Greater than a.c.w.	<2'	Intermediate 5/
	Greater than a.c.w.	<2'	Abundant
3	Equal to a.c.w.	<2'	Intermediate
	Equal to a.c.w.	<2'	Abundant
4	Equal to a.c.w.	Shallow 6/	Exposed
	Less than a.c.w.	Shallow	Abundant
	Less than a.c.w.	Shallow	Intermediate
	Less than a.c.w.	<2'	Intermediate
	Less than a.c.w.	2' or deeper	Abundant
5	Less than a.c.w.	Shallow	Exposed

1/ Logs, stumps, boulders, and vegetation in or overhanging pool or overhanging banks.

2/ Average channel width.

3/ More than one-half perimeter of pool has cover.

4/ Less than one-quarter of pool perimeter has cover.

5/ One-quarter to one-half perimeter of pool has cover.

6/ Approximately equal to average stream depth.

APPENDIX D

Habitat Requirements

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Listed below are some specific habitat requirements for the more common wildlife species. These descriptions can be used to identify existing habitat, judge quality, and determine effects of modification.

1. Pheasants - A favorable land use pattern for pheasants consists of: (a) 60-80 percent in grain and seed crops, (b) 10-30 percent in grasses and legumes, (c) 5-10 percent in brush and woods, and (d) 3-5 percent in permanently protected herbaceous weedy cover.

   Crop fields ten acres or more in size, growing barley, buckwheat, corn, grain sorghum, oats, proso millet, soybeans, or wheat, should dominate the area. At least two acres of grasses and legumes (alfalfa, crownvetch, orchardgrass, reed canarygrass, sericea lespedeza, smooth brome, switchgrass, or timothy) per 100 acres should be present. Extra growth should be present on at least two 1/8 acre plots within 100 feet of a vegetative change. It is desirable to have at least one 1/2 acre strip of annual weeds per 100 acres and at least 1 acre of brushy thickets or woods.

2. Bobwhite Quail - Bobwhite usually thrive best where there are numerous small fields of grain and seed crops, interspersed with grassland, weedy patches, and brush or woodland areas. They seldom are abundant in extensive and continuous areas of cropland, grassland, or dense woodland.

   At least one-half acre of grain and seed crops (corn, cowpeas, lespedeza, millet) should be present within 100 feet of woody cover per each 100 acres of habitat or a one-half acre patch of wild herbaceous plants (panic grass, ragweed, croton, partridge pea). A combination of the two is best. Unmowed grass areas, 1/10 to 1/4 acres near food and cover, provide necessary nesting sites for a covey range. Woody cover totaling about 1 acre per 100 is necessary.

3. Ruffed Grouse - This grouse is a bird primarily of woodland edges and openings rather than dense woods. Hardwood trees should predominate. Hardwood trees, such as alder, apple, beech, birch,

cherry, mountain-ash, oaks, and poplars, should be well distributed throughout the woodland. Openings within each 200 acres of woodlands should occupy at least 1 percent of the area and not exceed 15 percent. Such openings are more valuable if a variety of vegetation, such as perennial weeds, wild grass, shrubby evergreens, and hardwood brush, grow in them.

4. Cottontails - Individual cottontail rabbits usually spend their entire lifetime in less than 10 acres of habitat. Therefore, a wide variety of vegetation on small-sized areas is desirable for good habitat. Four or five small patches (one-tenth to one-quarter acres) of grasses and legumes, wild or domesticated, in and around woody and brushy or cultivated fields, provide good food and cover. At least 50 percent of any area of rabbit habitat should be in hardwood trees and shrubs.

5. Woodcock - Habitat for woodcock may be described as a mixture of grass, perennial weeds, shrubby evergreens, and brushy thickets, generally occurring on moist soils having fairly high fertility and considerable organic matter (condition favorable to earthworm abundance).

   Brushy thickets should occupy 40 - 50 percent of the area, and should not exceed 15 feet in height. At least 500 square feet per acre of brush should be in grass and perennial weeds and brush should not exceed 2 feet in height for 50 feet around openings. Several small clumps of evergreen, shrubs near grassy areas provide nesting and brood rearing sites.

6. White-tailed Deer - This deer is an animal of forest lands broken by small clearings, lakes, swamps, crop fields, cut-over areas, pastures, hay meadows, etc., which create edge or allow sunlight to reach the ground so that shrubs and bushes will grow. Deer prefer open forest which provides plentiful understory vegetation.

   One 5-acre opening per each 200 acres of woodland provides grasses and legumes for summer and spring food. In addition, one 5- to 20-acre opening per each 200 acres of growing woodlands, shrubs, and vines increases the food supply.

7. Tree Squirrels - The fox squirrel is largely an inhabitant of mature, somewhat open, hardwood forests and woodlots. The gray squirrel lives primarily in large unbroken bottomland hardwood forests. The ranges overlap considerably, but normally one species predominates. Both species occupy two types of nests - den and leaf. Den trees provide the best protection. Two or three good den trees per acre are desirable. Nuts and acorns are staple foods; seed, buds, and fruits also are eaten. Each animal requires about a pound and a half of food per week. Nut-bearing trees should be well distributed throughout woodland, at least two per acre.

8. Non-Game Land Birds - Non-game birds include a great variety of species which are found in nearly every kind of vegetative community. These

species eat all kinds of foods, have many adaptations for nesting, feeding, escape, migration, etc. Therefore, a variety of habitats will support a variety of species. As a rule, the greater the variety of plant forms on a given tract of land, the larger the number of bird species and the more the individuals. Such areas may be grassy areas, grain and seed cropfields, weedy spots, brush areas, and woodlands. They also may be lawns, pastures, meadows, fencerows, small woodlots, barnyards, pond edges, etc.

9. Waterfowl and Other Wetland Birds and Muskrats

   a. Ducks (mallard, pintail, black duck, teal, wood duck): Ducks require several different vegetative types and water conditions for nesting, rearing broods, adult moult, and feeding:

      (1) Courting, pairing, and mating (mid-winter and early spring, before nesting activities) require little or no vegetative cover, as these activities generally are performed on small open-water areas with bare shorelines. Mating habitat usually is one or several small, shallow, open-water ponds.

      (2) Nesting (March to May). Mallards, teal, and black ducks nest on the ground, usually within 150 yards of water, but sometimes farther away in medium-height vegetation, such as alfalfa, redtop, and other grasses as well as emergent type wetland grasses, sedges, and rushes. Wood ducks nest in trees along water or waterways where they prefer a hollow or natural cavity in the trunk.

      (3) Rearing the broods (May to early September). Immediately after the young are hatched, the hen leads them from the nest to a water area several feet deep, surrounded by or interspersed with marsh plants, such as bulrushes, sedges, cattails, and other aquatic plants. Wood ducks desire woody cover along streams or ponds.

      (4) Loafing. Ducks spend a great deal of their time loafing, sunning, or preening themselves on mud flats, knolls, or small islands. Wood ducks commonly perch in trees.

      (5) Feeding. Black ducks, mallards, teal, and wood ducks feed primarily on plant seeds. They occasionally take snails, insects, and herbaceous vegetation. Important plants are corn, buckwheat, sorghum, barley, pondweeds, wildrice, millets, bulrushes, smartweeds, naiad, and white and pin oak acorns. Water, of course, is an essential element of the habitat.

      Swamp or marsh areas, 1 acre and larger, are more valuable.

The following water conditions are desirable on feeding a and brood rearing areas: At least 50 percent of the area with water less than 3 feet deep; small (500-2,000 square feet) open-water areas scattered throughout, about 5 per acre.

b. Muskrats

Muskrats are semi-aquatic and need water to live. The entrance to muskrat houses (or dens) is normally 4 to 16 inches beneath the water level. Muskrats use open water for travel, and their diet is composed chiefly of leaves and roots of emergent and aquatic vegetation.

c. Wading Birds - Consideration is given here to four families of birds:
Ardeidae (Herons and Bitterns); Ciconiidae (Storks and Wood Ibises); Threskiornithidae (Ibises and Spoonbills); and Gruidae (Cranes).

Obviously, many species of wildlife, such as other water birds -- amphibians, reptiles, fishes, and some mammals, particularly the marsh and swamp dwellers -- benefit from management of wading bird habitat.

The wading bird group feeds largely on small aquatic life found along edges and in extensive shallows of lakes and swampy areas. All kinds of small fish, both game and rough fish, have been found in food habitat studies. Salamanders, frogs, and aquatic insects make up a part of the diet. Sluggish swimmers and prolific producers, such as gambusia (top minnows), make good food sources.

Herons and the like usually nest in groups of a few to several hundred or more. These rookeries may have several species nesting in one tree. The nesting birds prefer to build over water several feet deep. Cypress swamps with enough underbrush, such as buttonbush, make desired nesting sites.

It is generally believed that water fluctuation plays an important role in nesting behavior. Plentiful water supply in early spring apparently stimulates breeding. Low water levels concentrating the food source 4 to 6 weeks later provide good feeding conditions when the young are in the nest.

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