David G. Phillips

Design of Open Channels

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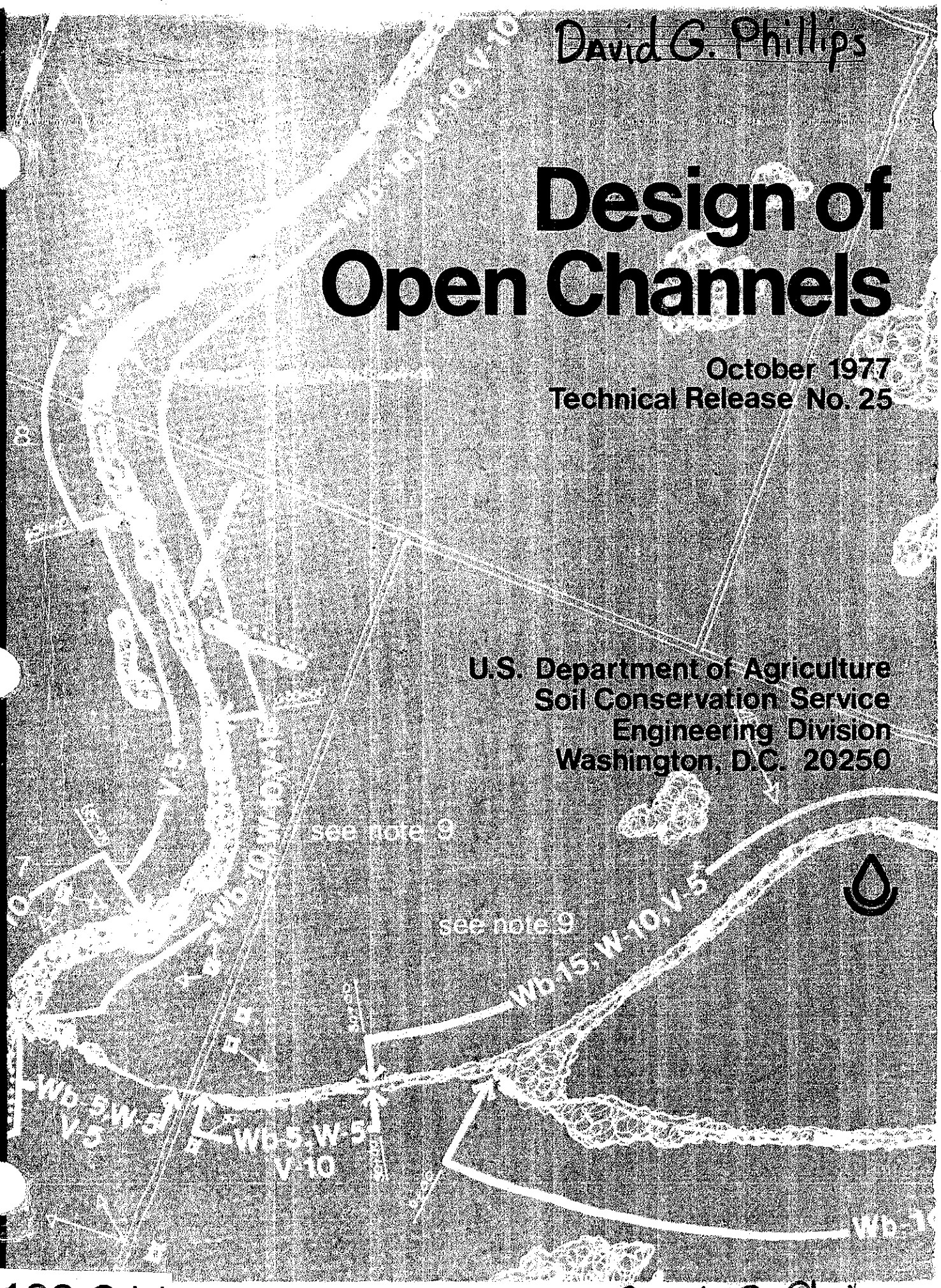

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TECHNICAL RELEASE NOTICE 25-5

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.

DESIGN OF OPEN CHANNELS

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TECHNICAL RELEASE

NUMBER 25

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.

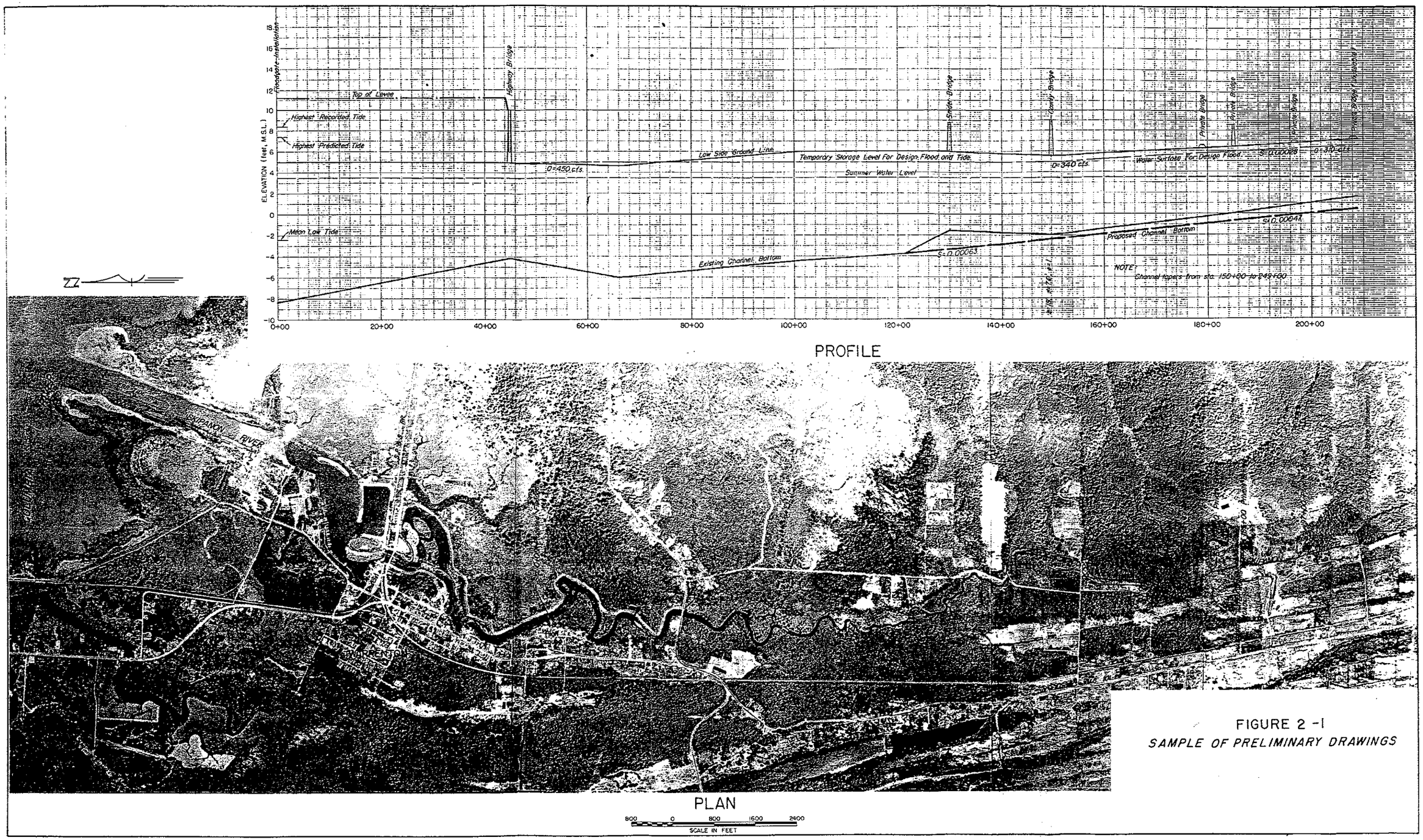

FIGURE 2 -1
SAMPLE OF PRELIMINARY DRAWINGS

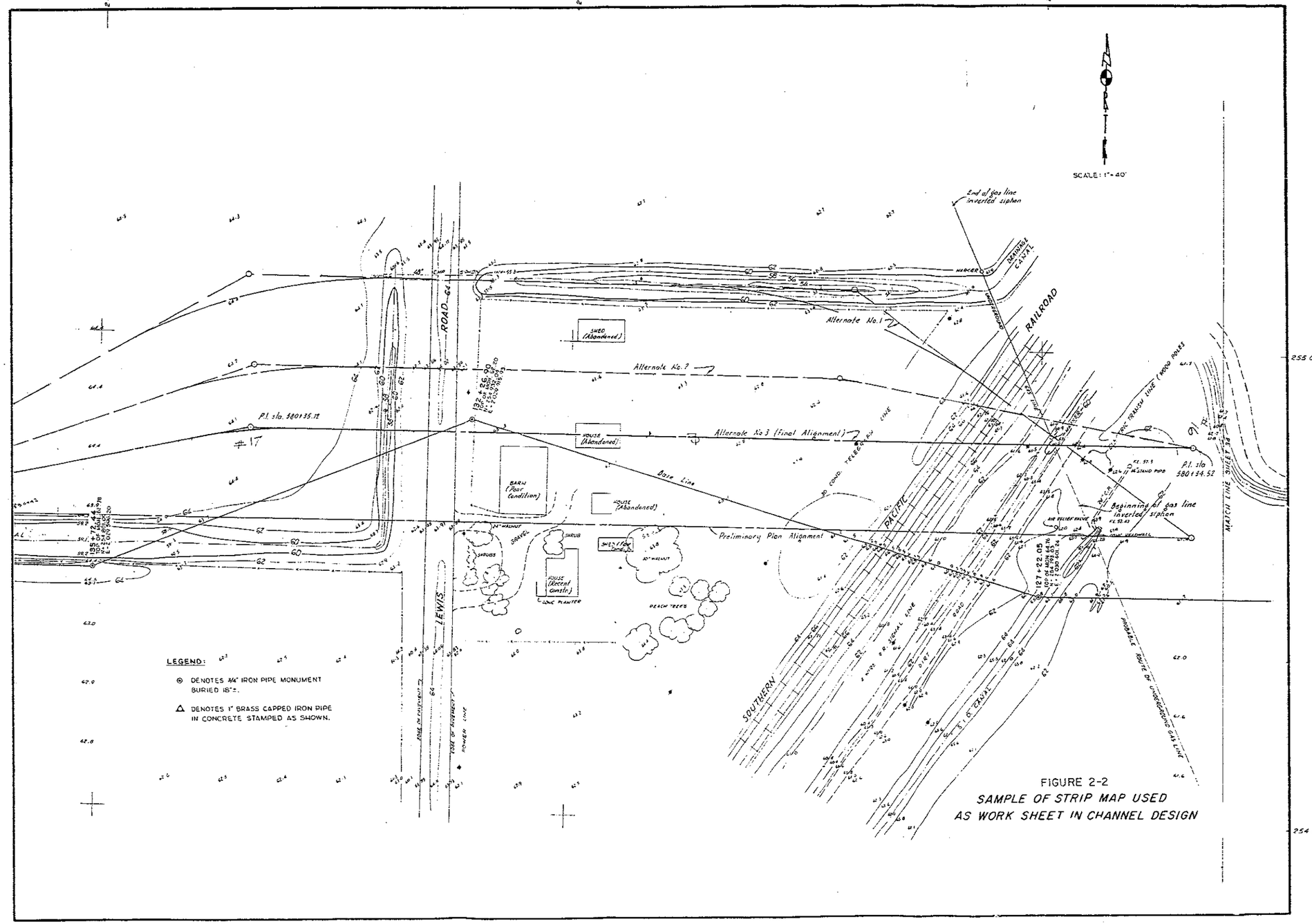

FIGURE 2-2
SAMPLE OF STRIP MAP USED AS WORK SHEET IN CHANNEL DESIGN

247-327 O - 78 (Face blank p. 30) No. 2

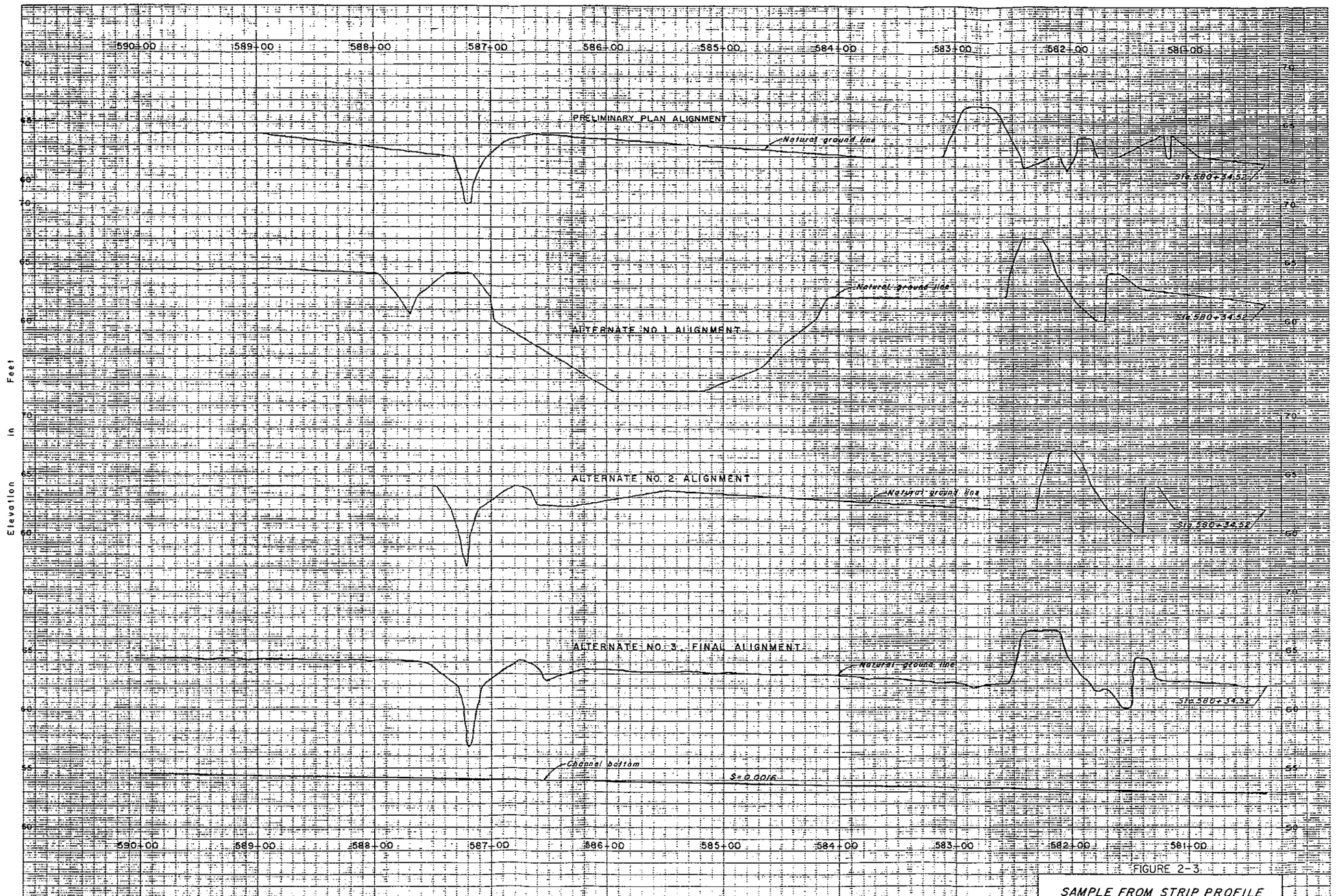

FIGURE 2-3

SAMPLE FROM STRIP PROFILE
(DESIGN WORK SHEET)

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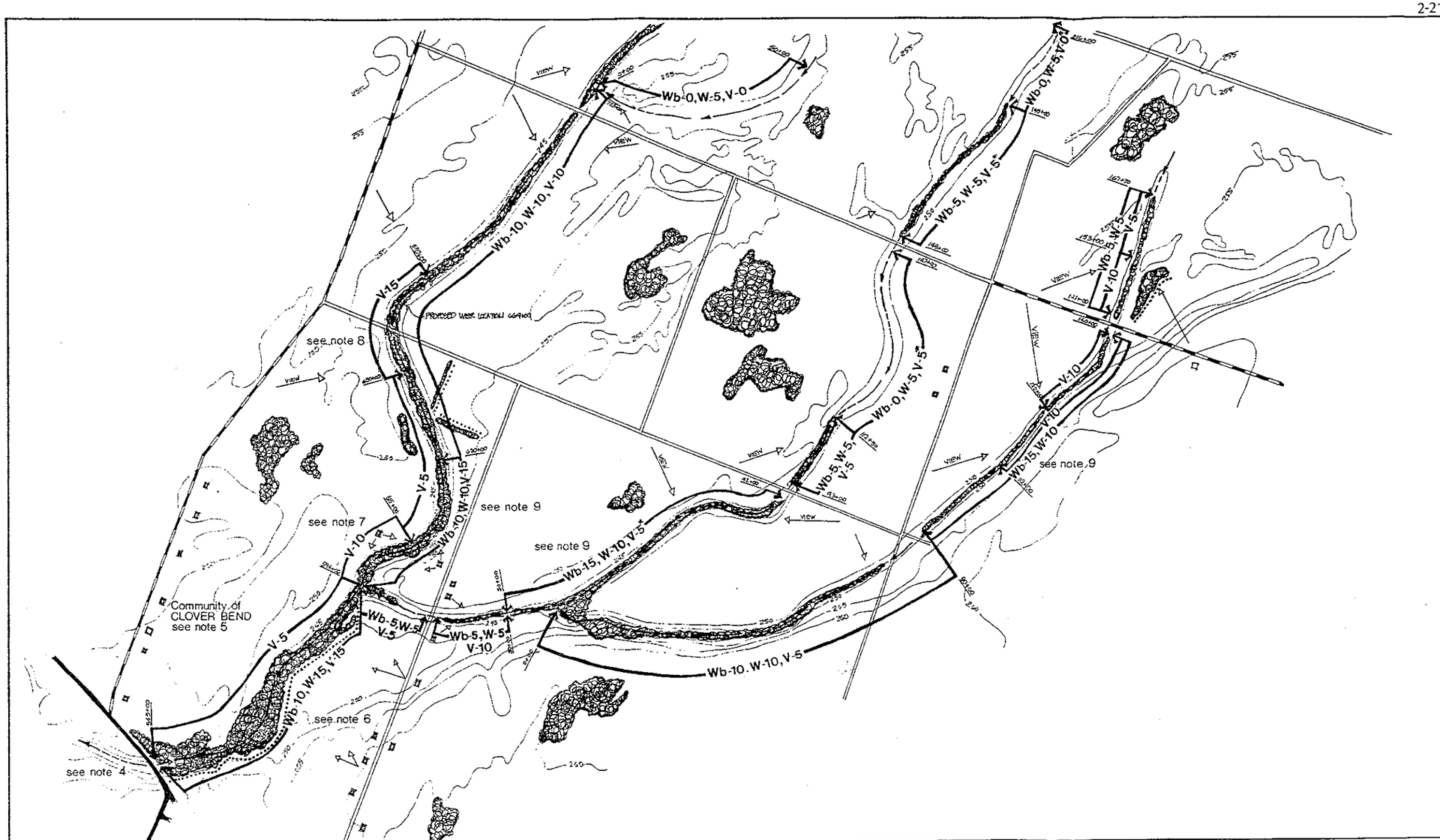


NOTES

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 @ 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
- VIEW — SIGHT LINES OF SIGNIFICANCE
- 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

Designed	Date	Approved by	
Drawn		Title	
Traced		Title	
Checked		Sheet No. of	Drawing No.

Form SCS-314 (November 1955)

247-927 O - 78 (Face blank p. 300) No. 4

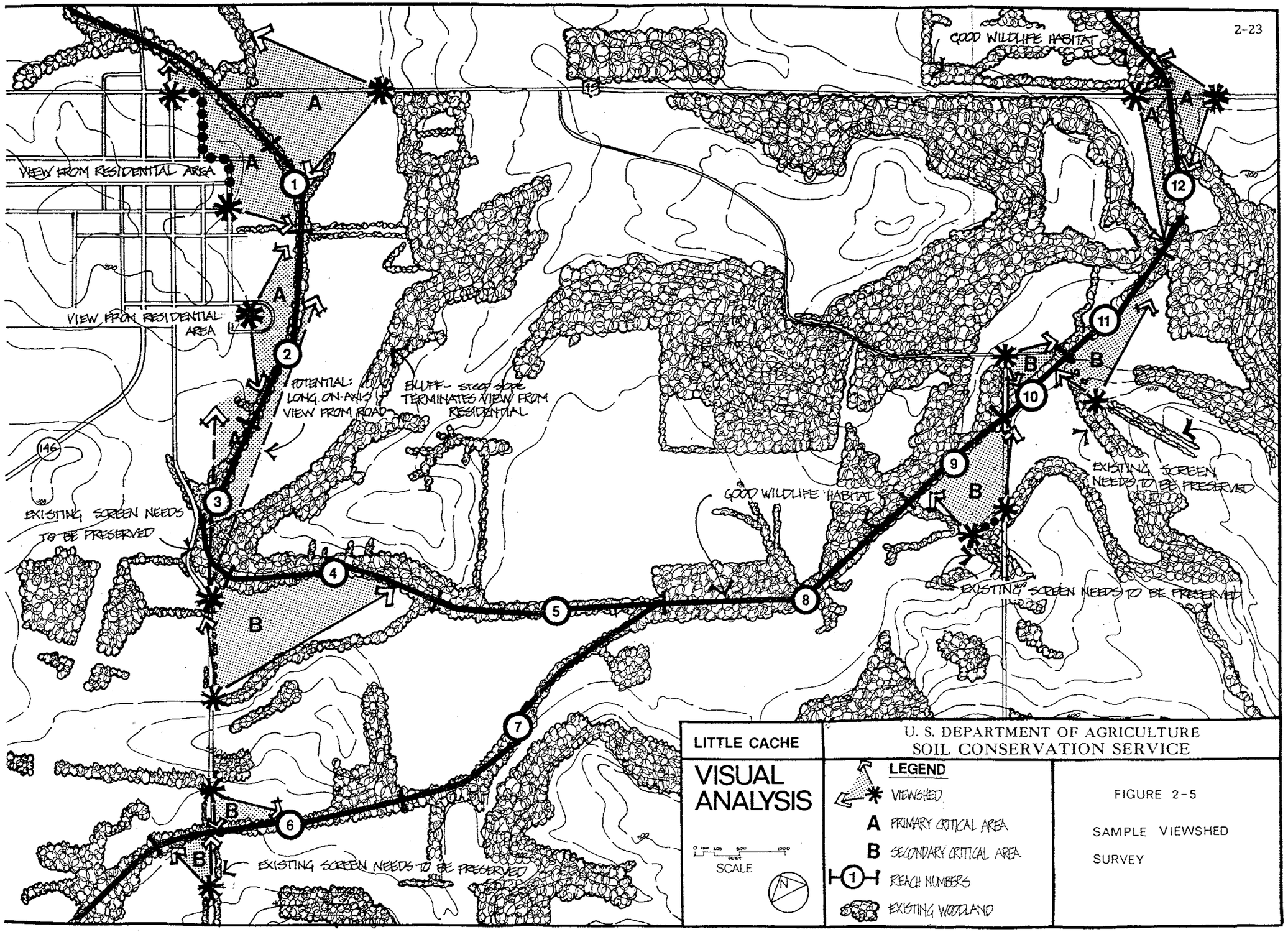
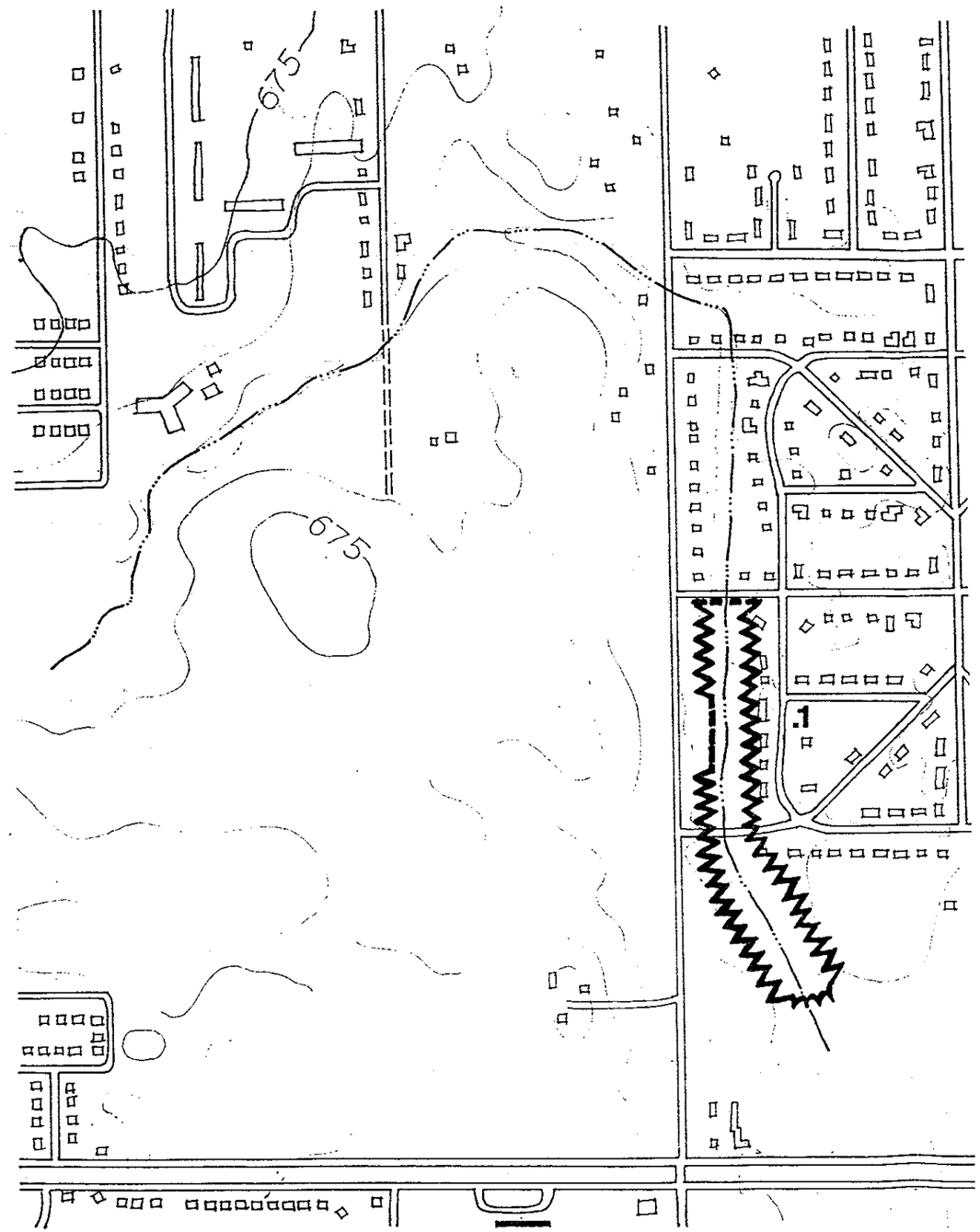


FIGURE 2-5

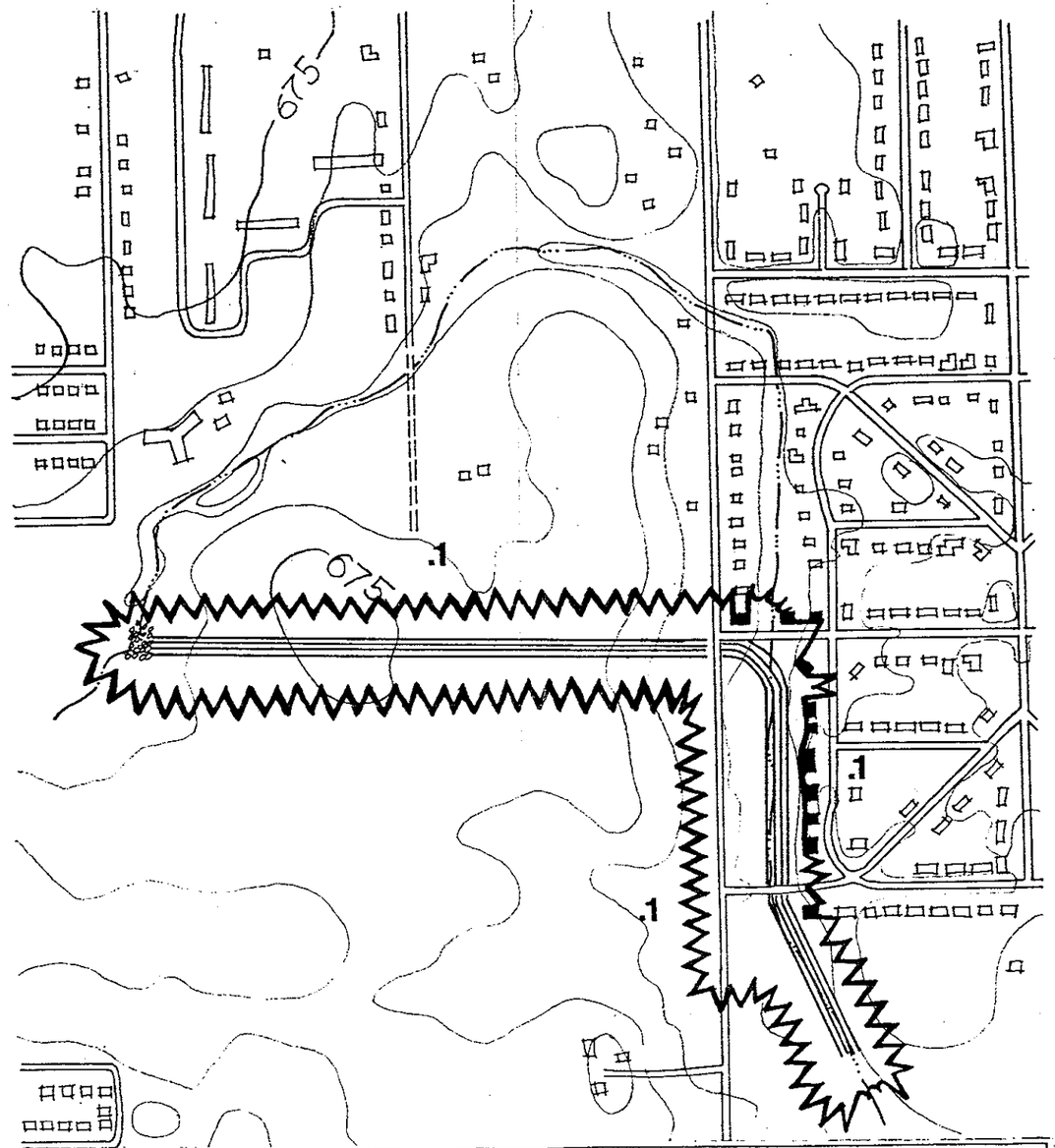
SAMPLE VIEWSHED SURVEY

O - 78 (Face blank p. 30) No. 5

Visual Conditions Before Construction: Winter



Visual Conditions After Construction: Winter



DIVERSION CHANNEL
VISUAL ANALYSIS

SCALE IN FEET
600 0 300 600 900

20 ACRES

FIGURE 2-6

KEY

- VISUAL EDGE
- PARTIALLY OBSTRUCTED VIEW
- VISUAL EDGE AT MAX. OF X MILES
- VEGETATION VISUAL EDGE
- BUILDING VISUAL EDGE

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.

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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 are 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 - APPENDIX

OUTLINE TO PLAN SITE INVESTIGATIONS AND PREPARE REPORTS FOR CHANNEL IMPROVEMENT

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 semi-arid 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

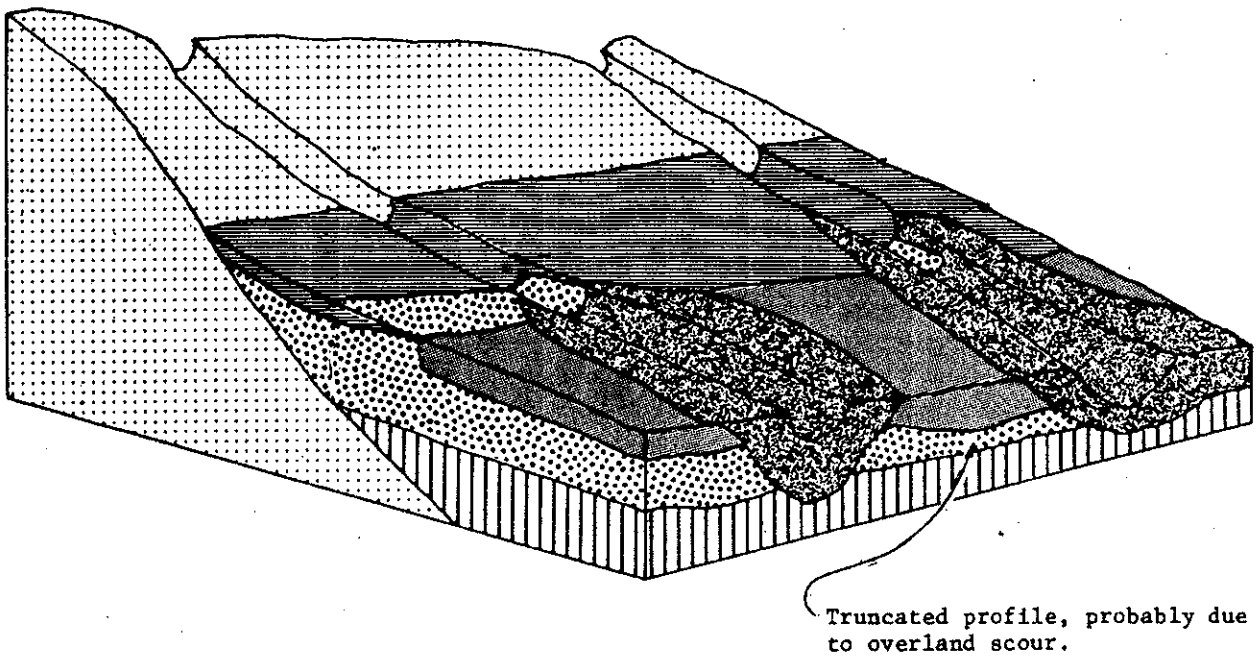
In certain geomorphic environments, the sediment deposited varies widely in texture within a short distance. If there is any unit identification, it is to the effect that there is a range in particle sizes from silt to gravel, all noncoherent, except for possibly some lens cementation. Such deposits 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.



Unit 1 - Recent deposits, principal stream

Unit 2 - Recent deposits, tributary

Unit 3 - Buried soil profile

Unit 4 - Buried, developed soil profile

Unit 5 - Terrace deposits

Unit 6 - Bedrock or residual soil

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.

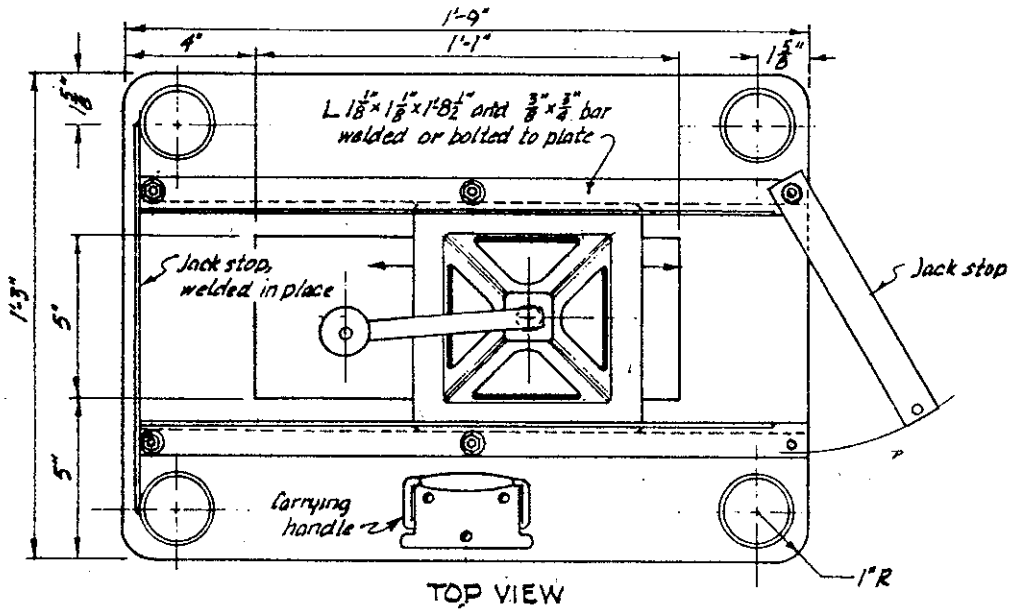
Careful selection of investigation tools and procedures for investigations are necessary to obtain samples and logs of the natural, undisturbed in-place materials. Augers, excavation equipment and many laboratory procedures disturb the soils and provide data which result in erroneous interpretations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

Selection of Equipment for Logging and Sampling

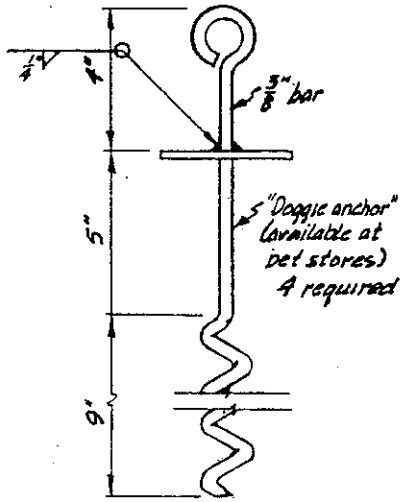
The frequently limited depth required in field investigations for channel improvements and the advantages afforded by in-place examination of the alluvial profile indicates that at least a framework of pit excavations is desirable. Safety precautions must be rigidly maintained. The backhoe is the preferred 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.

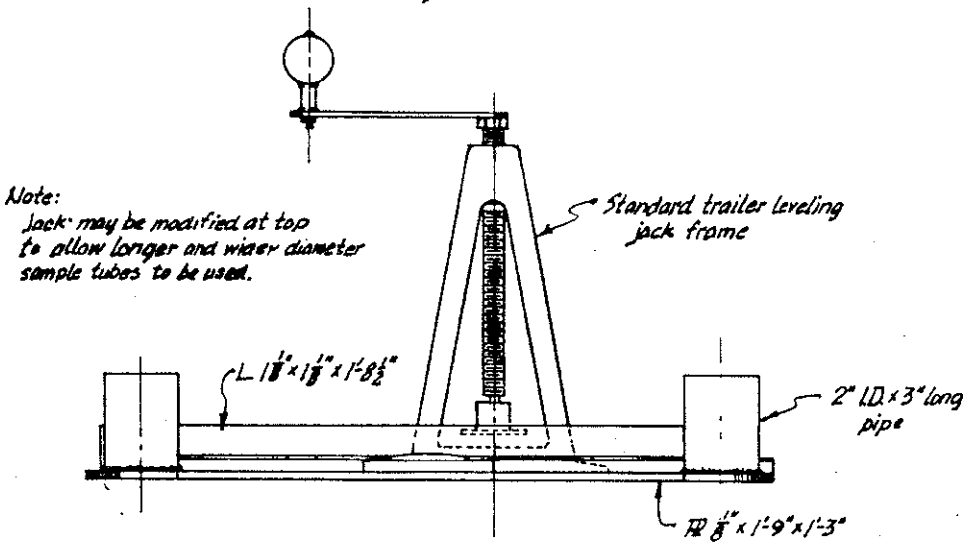
Sampling equipment is described in NEH, Section 8, Chapter 2. In instances where drilling equipment is used, push tube sampling of relatively soft fine-grained soils or Denison core barrel sampling of hard fine-grained soils are recommended. Special equipment has been devised for undisturbed sampling 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

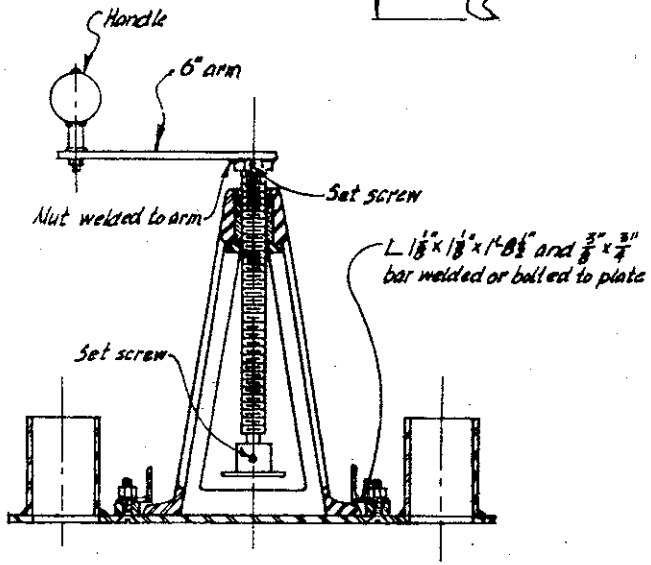


TOP VIEW





SIDE VIEW



CROSS SECTION

SCALE: 3" = 1'-0"

Fig 3-2 – UNDISTURBED-SAMPLE CYLINDER PRESS

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

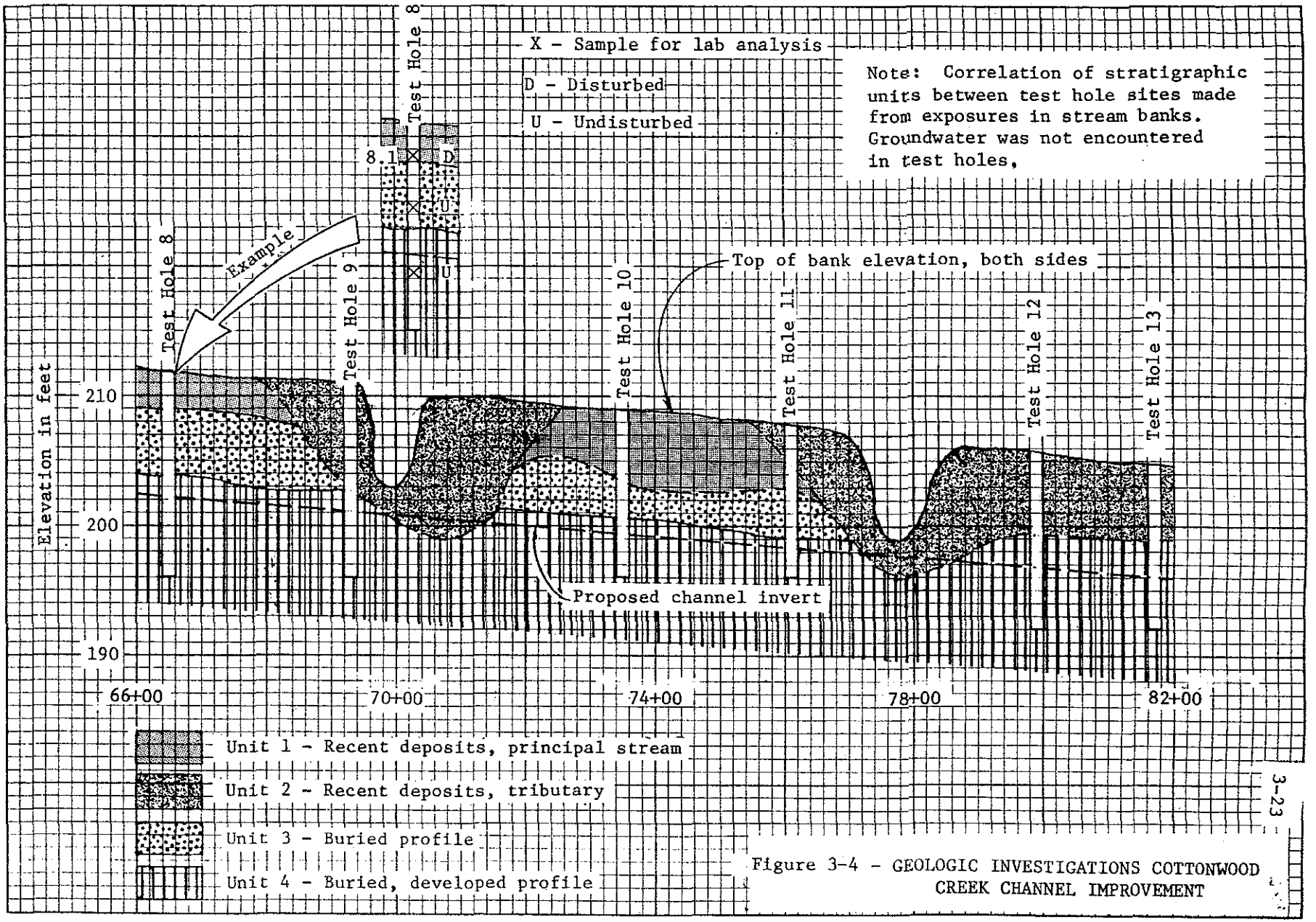


Figure 3-4 - GEOLOGIC INVESTIGATIONS COTTONWOOD CREEK CHANNEL IMPROVEMENT

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

   c. Plastic limit

   Plasticity index

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 <u>always</u> 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

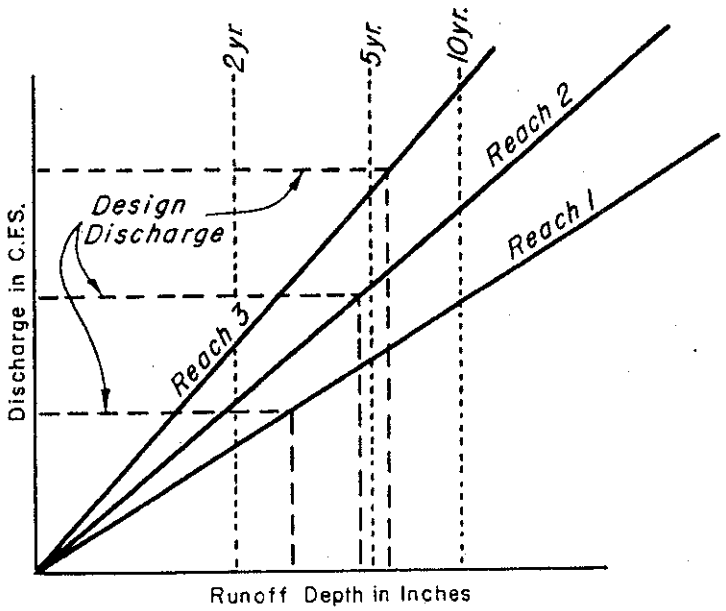

FIGURE 4-1
ROUTED PEAK DISCHARGE VERSUS
AVERAGE DEPTH OF RUNOFF, BY REACH

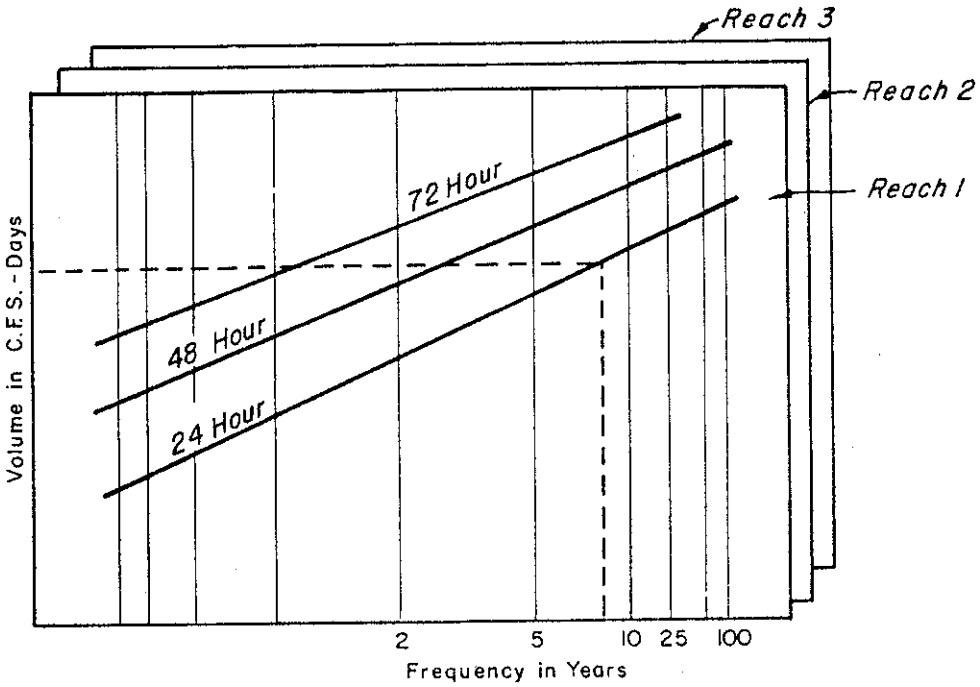

FIGURE 4-2
VOLUME DURATION FREQUENCY ANALYSES

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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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.)

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.)

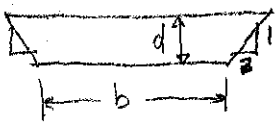
In closely controlled irrigation canals or channels where overbank flooding is permissible at frequent intervals, the above freeboard criteria may be disregarded.

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.

does not take radius of curve into account

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:



$$s = \frac{V^2 \overbrace{(b + 2\,zd)}^{\text{total width}}}{2\,(gR - 2zV^2)}$$

z = cot so angle

Ex 1/3 cot θ = 3/1 3=z

where the terms are as defined in the glossary.

A =

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:

channel actual slope

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

critical slope

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.

subcritical

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).

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

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.

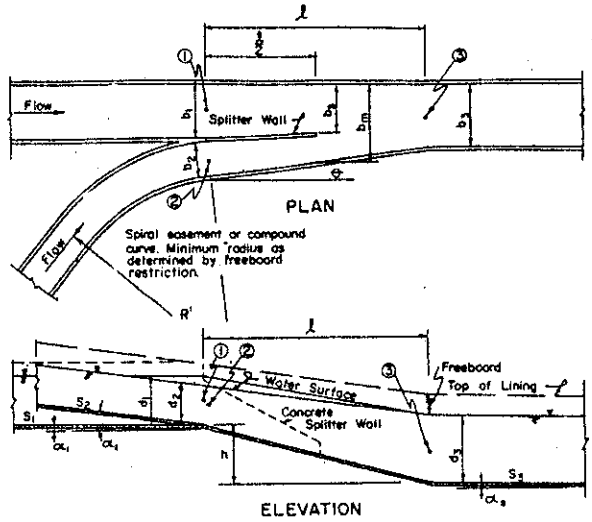
Channel outlet. - - The SAF Basin is the most satisfactory outlet structure where rectangular R/C channels at supercritical flow discharge into an earth section. It is also permissible to use a R/C transition to a trapezoidal rock-lined section. The rock lining should extend a sufficient distance downstream 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.
Z	= 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

$$\frac{Q_3^2}{gA_3}\cos\alpha_3 + \left(\frac{b_3}{2} + \frac{Zd_3}{3}\right)\left(d_3^2\right)\cos^2\alpha_3 = \frac{Q_1^2}{gA_1}\cos\alpha_1 + \frac{Q_2^2}{gA_2}\cos\alpha_2\cos\theta + \left(\frac{b_1}{2} + \frac{Zd_1}{3}\right)\left(d_1^2\right)\cos^2\alpha_1 + A_a h - \frac{P_a \ell n^2 V_a^2}{2.21\, R_a^{1/3}}$$

Unequal Width Main Channel

$$\frac{Q_3^2}{gA_3}\cos\alpha_3 + \left(\frac{b_3}{2} + \frac{Zd_3}{3}\right)\left(d_3^2\right)\cos^2\alpha_3 = \frac{Q_1^2}{gA_1}\cos\alpha_1 + \frac{Q_2^2}{gA_2}\cos\alpha_2\cos\theta + \left(\frac{b_2}{2} + \frac{Zd_1}{3}\right)\left(d_1^2\right)\cos^2\alpha_1 + (b_3 + Zd_m)\, d_m h - \frac{P_m \ell n^2 V_m^2}{2.21\, R_m^{1/3}}$$

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

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

Unequal Width Main Channel

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

SPECIAL CASE

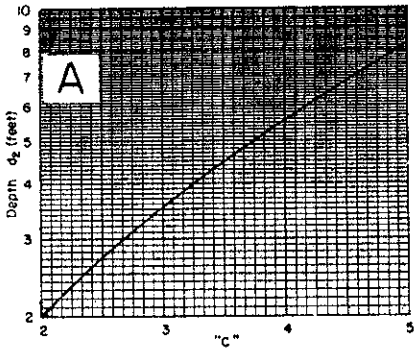
Limiting Criteria - Rectangular Channels
Supercritical Velocity Flow in All Branches.

Constant Width Main Channel

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

Unequal Width Main Channel

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



Criteria

$\tan\alpha < 0.1$

Convergence angle θ less than 10°

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

$\theta \lesssim 10^\circ$

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

Height of sidewall at ③ $= 1.2\, d_3 + 0.25\, d_{c_3}\left[1 - 11.1\left(\frac{S_3}{S_{c_3}} - 1\right)^2\right] + 1.2\,\frac{V_2^2 b_2}{gX_2}$

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

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

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

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

$b_e = \frac{b_2\,(b_1 + b_2 + b_3)}{2(b_1 + b_2)}$ 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.

   $V^2/2g$

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

$$\frac{V_1^2}{2g} + \frac{P_1}{W} + Z_1 = \frac{V_2^2}{2g} + \frac{P_2}{W} + Z_2 + h_f \qquad \text{(Eq. 5.1-1)}$$

units of length

$\frac{P_2}{W} =$

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

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

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 ΔW.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. See fig 5.1-1

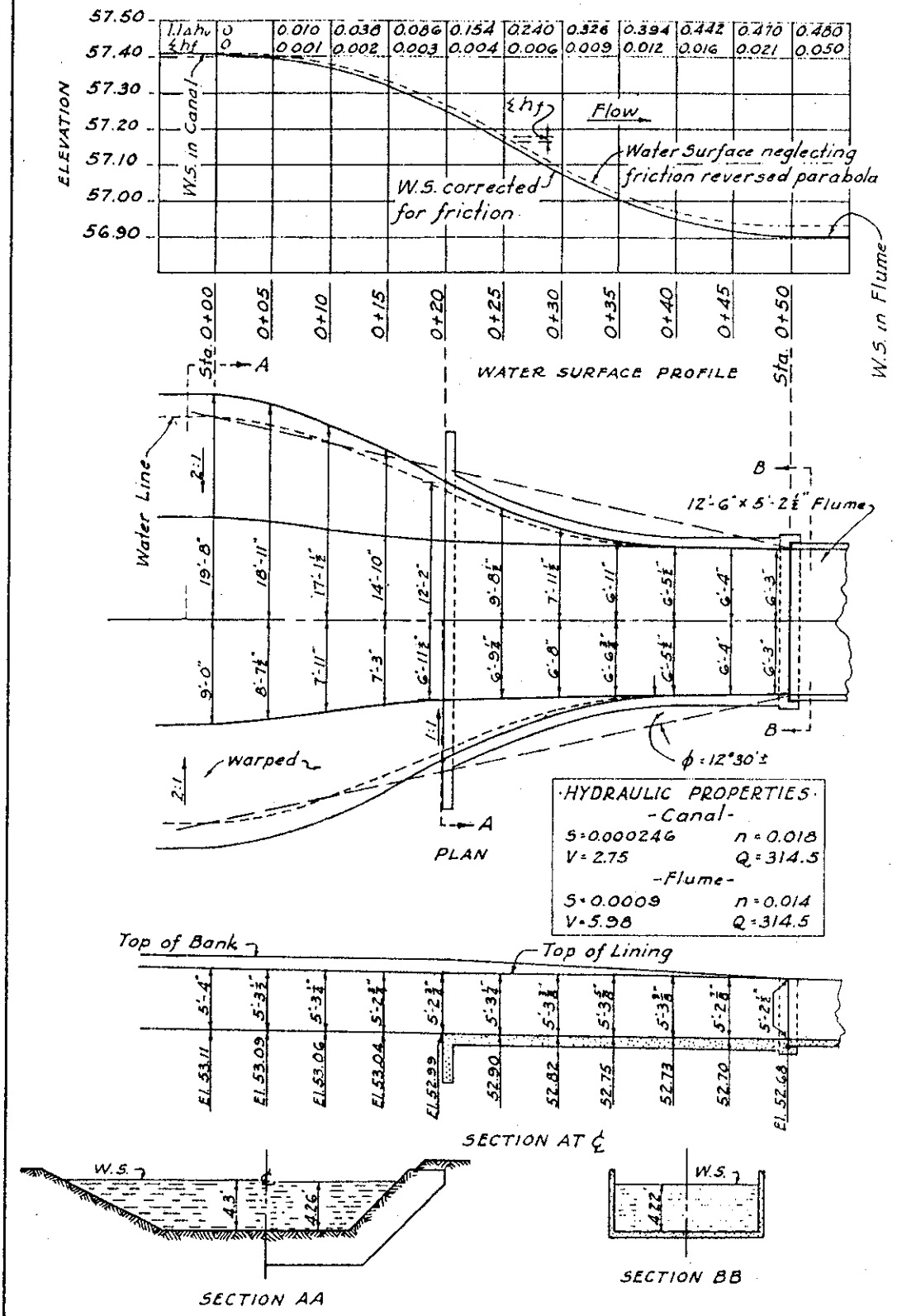
Line	Item											
		**0 + 00	0 + 05	0 + 10	0 + 15	0 + 20	0 + 25	0 + 30	0 + 35	0 + 40	0 + 45	0 + 50
1	Δ WS = Drop in WS		0.010	0.038	0.086	0.154	0.240	0.326	0.394	0.442	0.470	0.480
2	Δh_v = WS ÷ 1.1		0.009	0.035	0.079	0.140	0.218	0.296	0.357	0.401	0.427	0.436
3	h_v = 0.117 + Δh_v		0.126	0.152	0.196	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	Σh_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-Σh_f	57.410	57.399	57.370	57.321	57.252	57.164	57.075	57.003	56.951	56.919	56.905
14	Grade = WS Elev - d	53.110	53.090	53.060	53.041	52.992	52.900	52.823	52.750	52.726	52.699	52.685
15	0.5 T - 0.5 B	8.600	8.375	7.510	6.210	4.270	2.368	1.050	0.284			
16	Side slopes	2.000	1.945	1.744	1.447	1.000	0.554	0.247	0.067			
17	Height of lining = H	5.330	5.295	5.270	5.234	5.228	5.265	5.287	5.305	5.274	5.236	5.205
18	0.5W-0.5B=Side slope xH	10.660	10.310	9.210	7.575	5.228	2.920	1.305	0.354			
19	0.5 W = 0.5 top width	19.660	18.935	17.127	14.825	12.186	9.691	7.972	6.917	6.458	6.315	6.250
20	0.5 W to nearest 1/2 in.	19'8"	18'11"	17'11 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



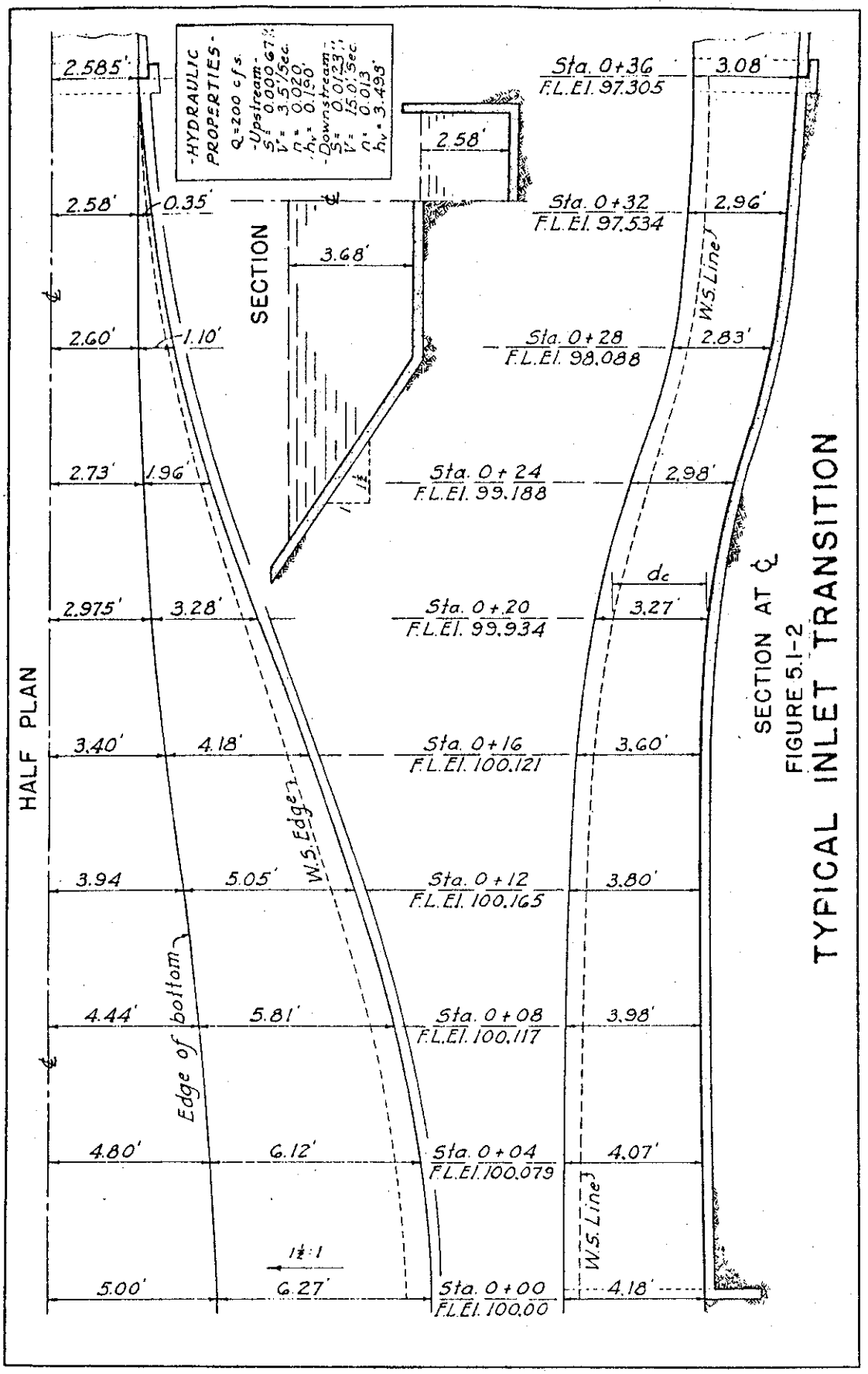
Design an inlet transition from an earth channel with a discharge of 200 C.F.S., bottom width 10.0 feet, depth 3.68 feet, side slopes 1-1/2:1, area 57.11 square feet and a velocity of 3.5 feet per second to a rectangular concrete lined channel with a width of 5.17 feet, depth 2.58 feet, area 13.34 square 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	$V = \sqrt{2 \quad gh_v}$	3.50	3.74	4.11	4.95	6.21	8.27	11.31	14.12	14.89	15.00
5	A = Q/V	57.14	53.48	48.66	40.40	32.21	24.18	17.68	14.16	13.43	13.33
6	0.5 T (Measured)	10.52	10.18	9.52	8.32	7.00	5.755	4.40	3.48	2.87	2.585
7	0.5 b (Measured)	5.00	4.80	4.44	3.94	3.40	2.975	2.73	2.60	2.58	2.585
8	Average Width = (.5 T + .5b)	15.52	14.98	13.96	12.26	10.40	8.73	7.13	6.08	5.45	5.17
9	d = A/Av. Width	3.68	3.57	3.48	3.30	3.10	2.77	2.48	2.33	2.46	2.58
10	**f=Friction Slope=$n^2v^2 \div 2208R^{4/3}$	.00028	.00034	.00043	.00068	.00119	.00249	.00541	.00985	.01206	.0123
11	h_f = 4 (Average f)		.00124	.00154	.00220	.00372	.00736	.01580	.03052	.04384	.04872
12	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


-HYDRAULIC PROPERTIES-
Q=200 c.f.s.
-Upstream-
S= 0.000677
V= 3.5'/Sec.
n= 0.020
hv= 0.190
-Downstream-
S= 0.0123
V= 15.0'/Sec.
n= 0.013
hv= 3.495'
SECTION
HALF PLAN
Sta. 0+36
F.L.El. 97.305
Sta. 0+32
F.L.El. 97.534
Sta. 0+28
F.L.El. 98.088
Sta. 0+24
F.L.El. 99.188
Sta. 0+20
F.L.El. 99.934
Sta. 0+16
F.L.El. 100.121
Sta. 0+12
F.L.El. 100.165
Sta. 0+08
F.L.El. 100.117
Sta. 0+04
F.L.El. 100.079
Sta. 0+00
F.L.El. 100.00
W.S. Edge
Edge of bottom
W.S. Line
SECTION AT ℄
FIGURE 5.1-2
TYPICAL INLET TRANSITION

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.

$V^2/2g$

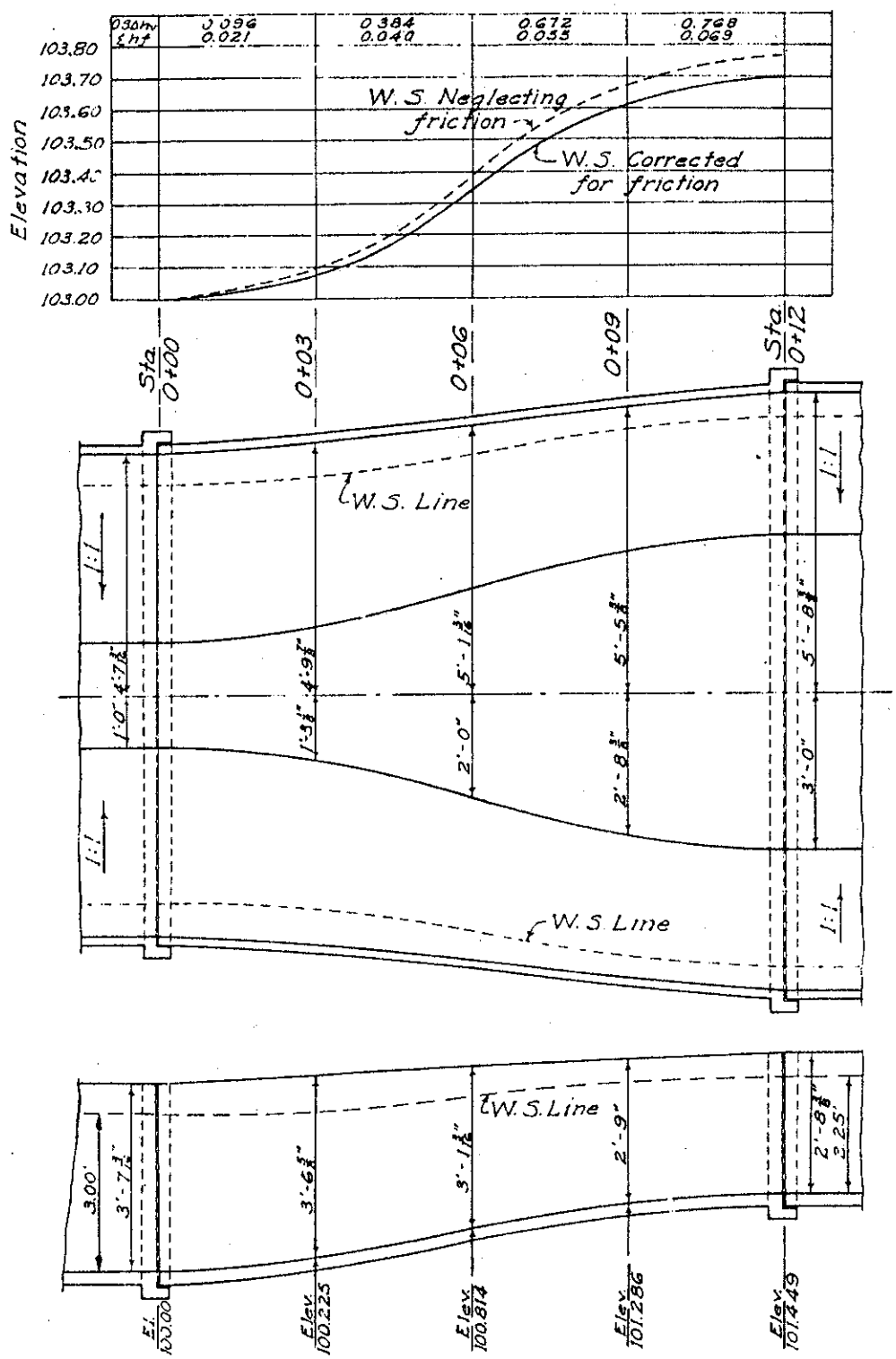
Line	Item	Station				
		*1	2	3	4	5
1	$\triangle$ W.S. = Rise in W.S.		0.096	0.384	0.672	0.768
2	$\triangle h_v = \triangle$ W.S. $\div$ 0.80		0.120	0.480	0.840	0.960
3	$h_v = 2.76 - \triangle h_v$	2.760	2.640	2.280	1.920	1.800
4	$V = \sqrt{2gh}$	13.33	13.09	12.11	11.11	10.77
5	Area = Q $\div$ V	15.00	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 + $\triangle$ W.S. - Σh_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



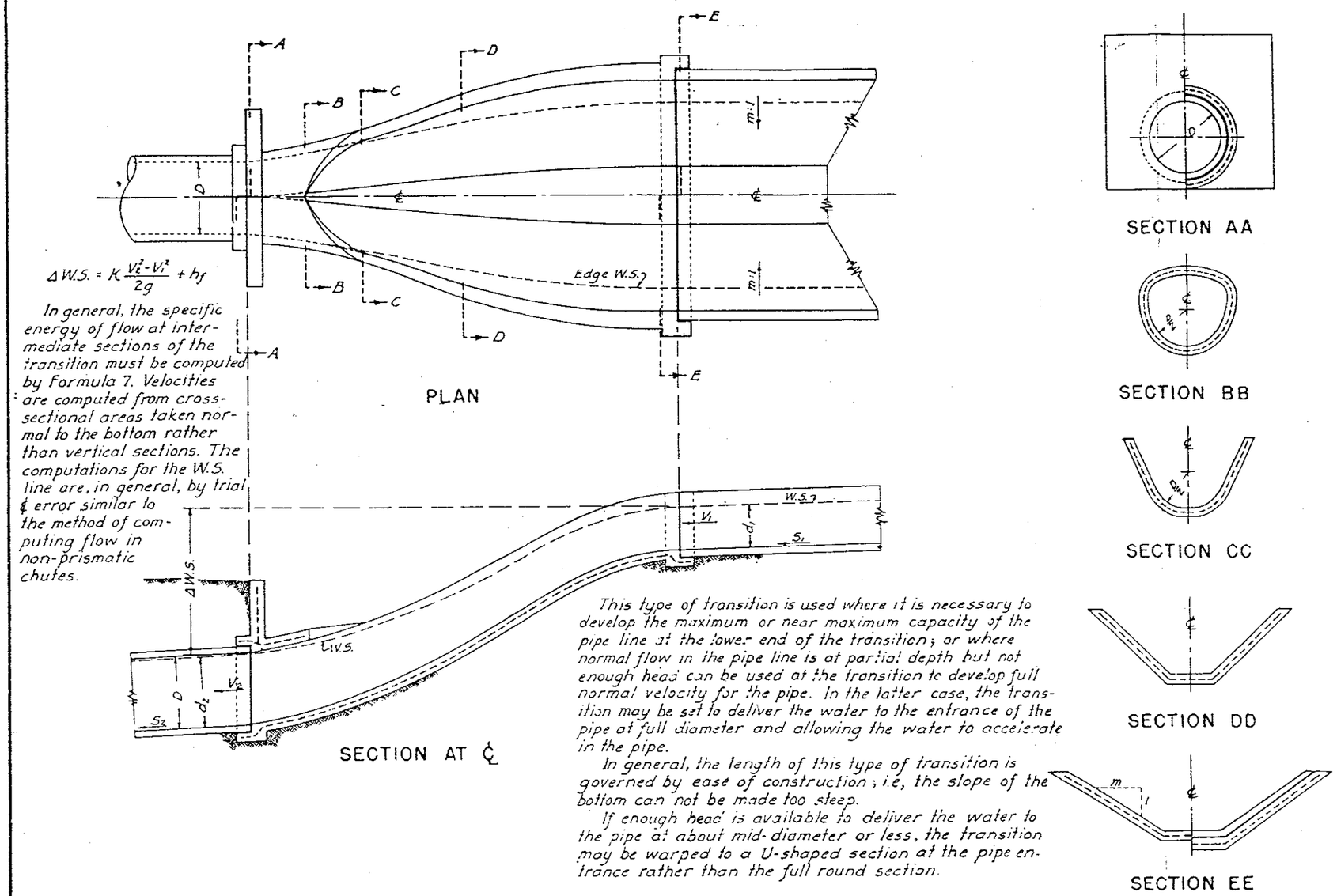
HYDRAULIC PROPERTIES

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'

FIGURE 5.1-4

TYPICAL INLET TRANSITION TO PIPE LINE



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

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.

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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.

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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

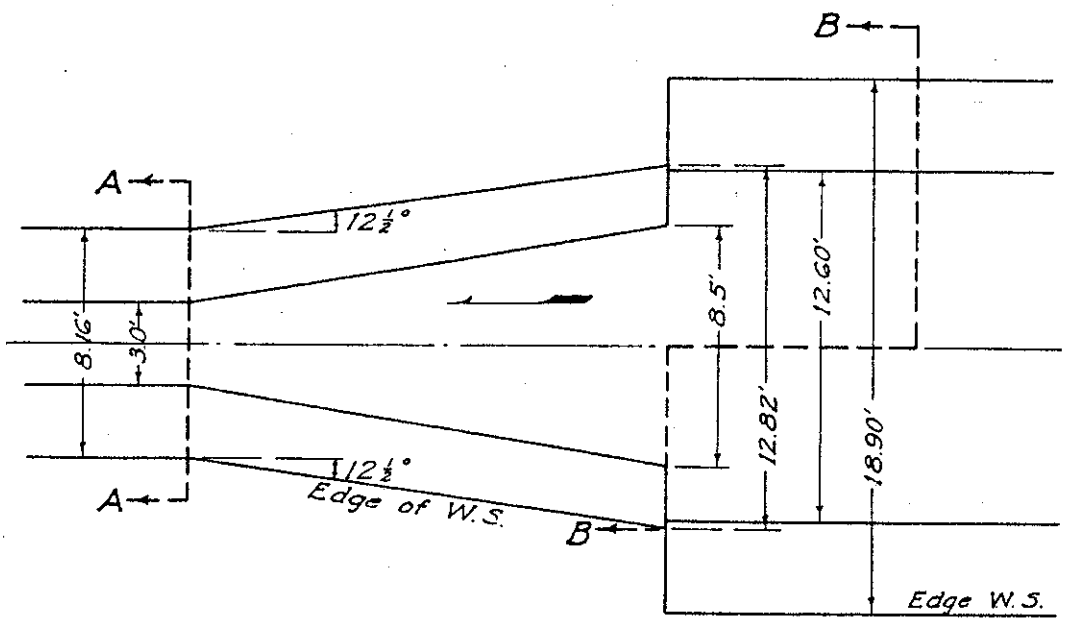
   2.33 x cot 12-1/2° equals 2.33 x 4.51 equals 10.5 feet say 10.0 ft.

2. W.S. equals 1.15 (Δh_v)

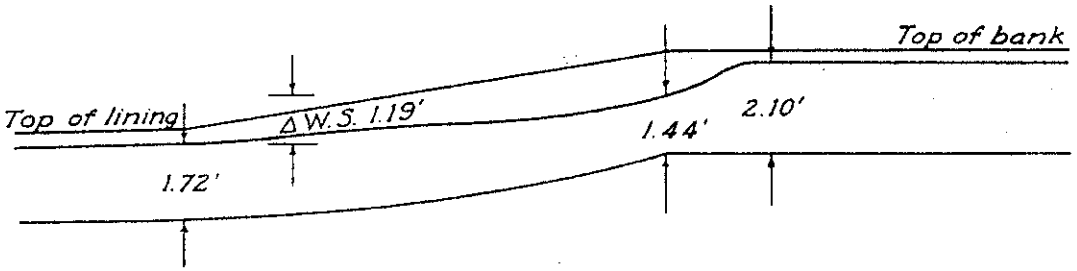
   equals 1.15 (1.035) equals 1.19 feet (See Fig. 5.1-5)

FIGURE 5.1-5

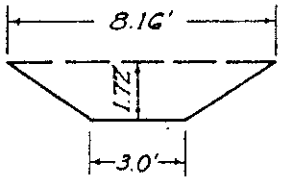
TYPICAL TRANSITION WITH STRAIGHT SIDES



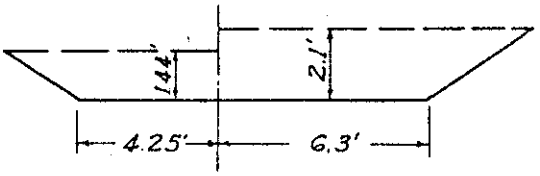
Plan Showing Bottom & Lines at W.S.



Side Elevation Showing Bottom & Approx. W.S.



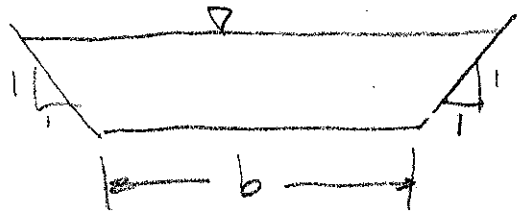
Section A-A



Section B-B

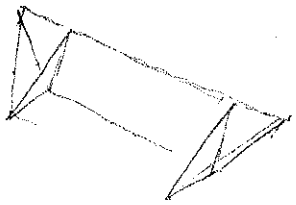
Let $b = 8$ ft

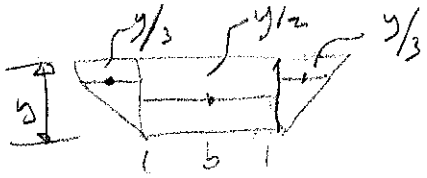
$Q = 160$ cfs



$Q = AV$, $V = Q/A$ $\quad A = by + y^2$

$$E = y + V^2/2g = y + \frac{Q^2/A^2}{2g} = y + \frac{Q^2}{2(by+y^2)g}$$





State pressure force

Ave pressure = $P_{ave} \times A_{rea}$ = $\int p\,dA = \int \gamma h\,dA = \gamma \bar{h} A$

= centroidal Axis depth × Area

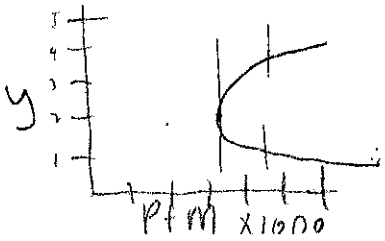
$$P = \left[2\left[\tfrac{y}{3} \cdot \tfrac{1}{2}y^2\right] + \tfrac{y}{2}\left[by\right]\right]\gamma = \gamma\left(\tfrac{1}{3}y^3 + \tfrac{b}{2}y^2\right)$$

$$m = \rho Q V = \rho Q^2/A$$

$$P + m = \gamma\left(y^3/3 + by^2/2\right) + \rho Q^2/A$$

y vs P+m

y	$\gamma y^3/3$	$\gamma b y^2/2$	y^2	A	$m = \rho Q^2/A$	$P+m$
1	20.8	299.6	1	9	5518.22	5788.62



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

$$\frac{Q^2}{gA} = \frac{A^6/g^2}{A/g^2 \times A^2} = A^3$$

Q = discharge in cubic feet per second

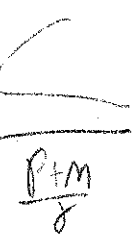
g = acceleration of gravity in feet per second per second

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.

$$\underbrace{P+M}_{\text{Pressure force + momentum flux}} = \frac{\gamma}{2}y^2 + \rho q V = \frac{1}{2}\gamma y^2 + \frac{\gamma}{g}\frac{q^2}{y} \quad (V = q/y)$$

$$\frac{P+M}{\gamma} = \frac{y^2}{2} + \frac{q^2}{gy}$$



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

upstream loss momentum at section

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

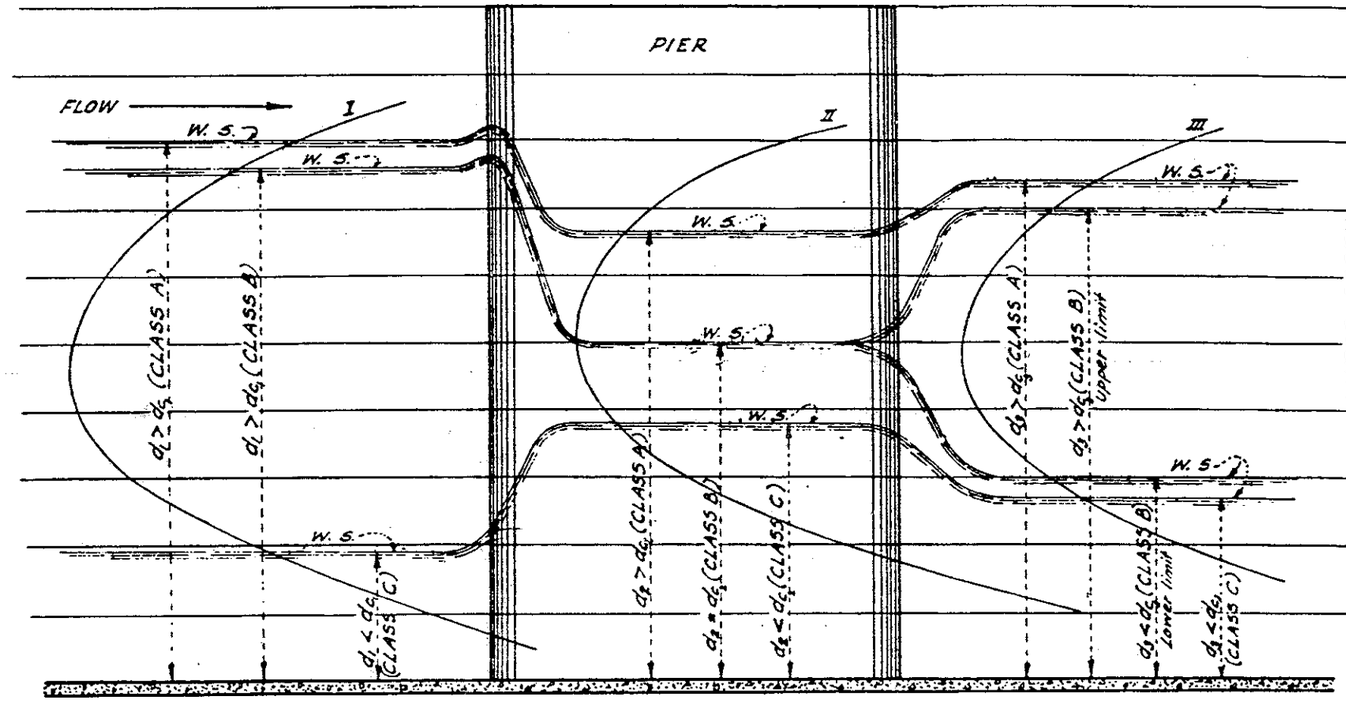
$m_1 - m_2 = m_2$

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

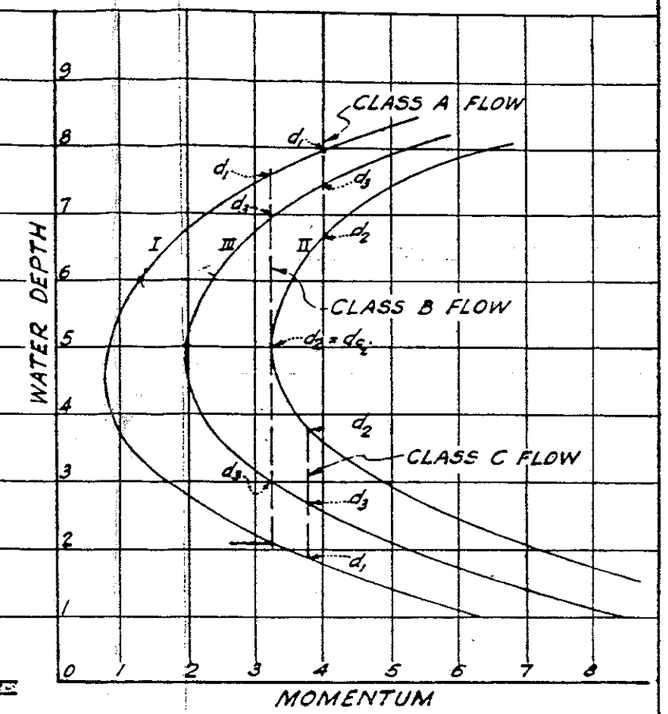
Total momentum within the constricted section plus static pressure on the downstream obstructed area must equal the total momentum in the downstream channel, or

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

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



LONGITUDINAL PROFILE



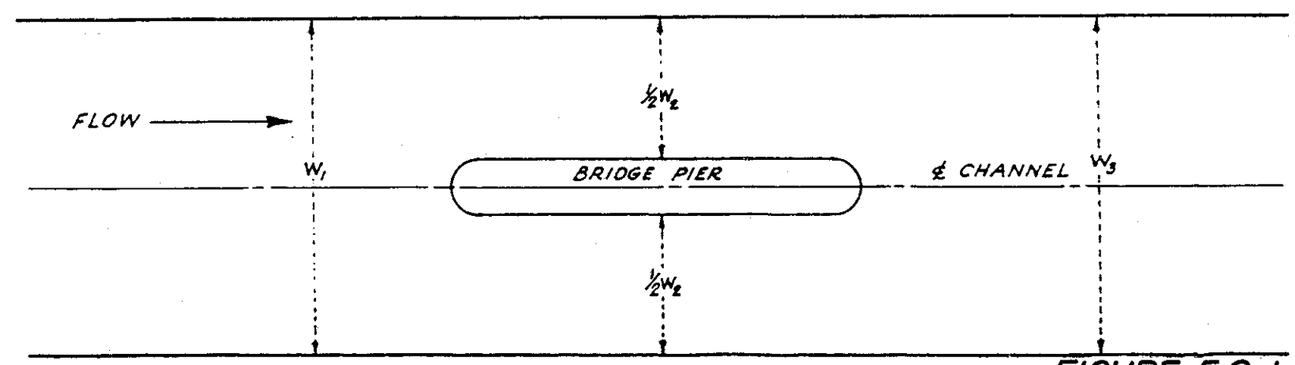
MOMENTUM CURVES

GENERAL MOMENTUM EQUATION:

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

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_2} = 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.



PLAN

FIGURE 5.2-1 BRIDGE PIER LOSSES BY THE MOMENTUM METHOD

247-927 O - 78 (Face p. 134)

The general momentum equation follows:

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

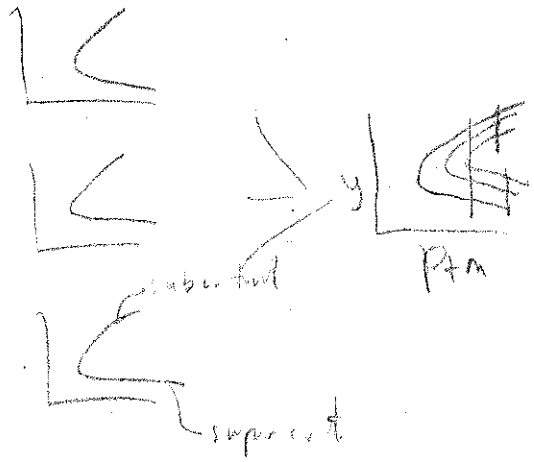
The above equations cannot be solved as presented, and it is necessary that a simpler method be used. The total sum of momentum and hydrostatic pressure for each section (Fig. 5.2-3) for equal depths of flow past each section should first be determined. Using equal depths, $A_1 - A_0 = A_2 = A_3 - A_0$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

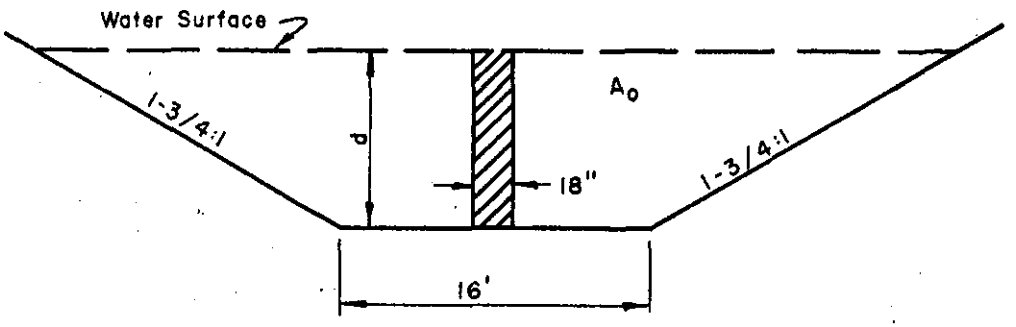


Fig. 5.2-2. Cross-section of channel showing center bridge pier.

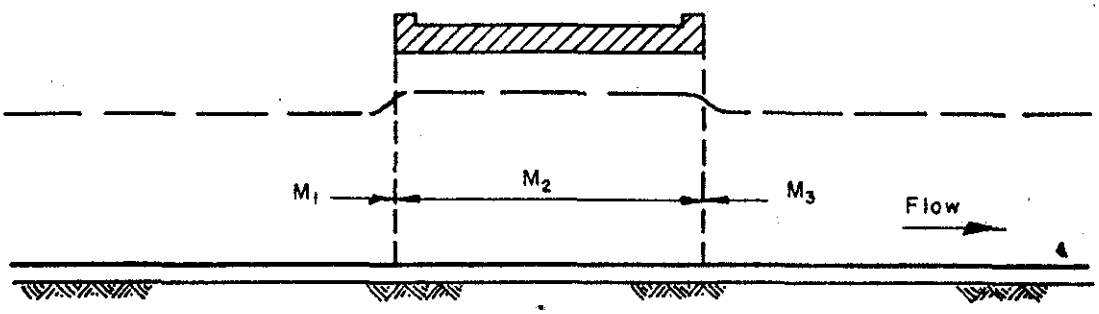


Fig. 5.2-3. Longitudinal section along centerline of channel indicates three locations: I - immediately upstream, II - Under bridge, III - Immediately downstream.

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.

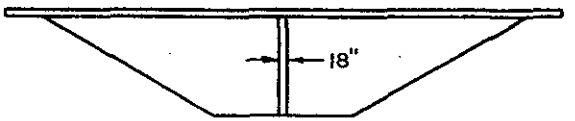
The water surface profile, above and below bridges, can be computed by the standard step method, as described in a report "Technical Memorandum - Water Surface Computation in Open Channels" by R. F. Wong, Los Angeles District Corps of Engineers, and also given in King's Handbook of Hydraulics.

Table 5.2-1 MOMENTUM LOSSES AT BRIDGES

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

Section



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

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

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

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

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

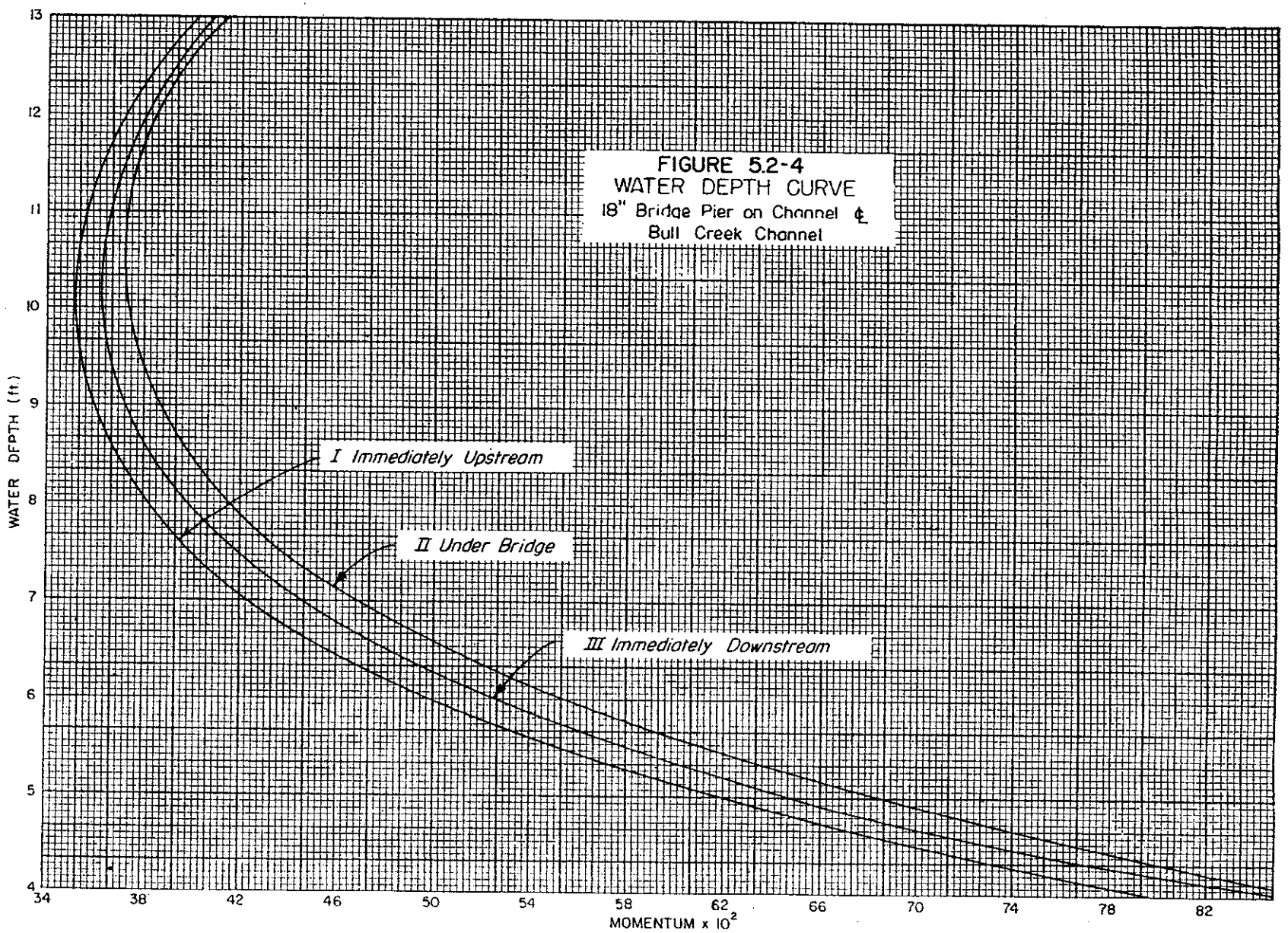


FIGURE 5.2-4
WATER DEPTH CURVE
18" Bridge Pier on Channel ℄
Bull Creek Channel
I Immediately Upstream
II Under Bridge
III Immediately Downstream
WATER DEPTH (ft.)
13
12
11
10
9
8
7
6
5
4
34
38
42
46
50
54
58
62
66
70
74
78
82
MOMENTUM x 10²

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

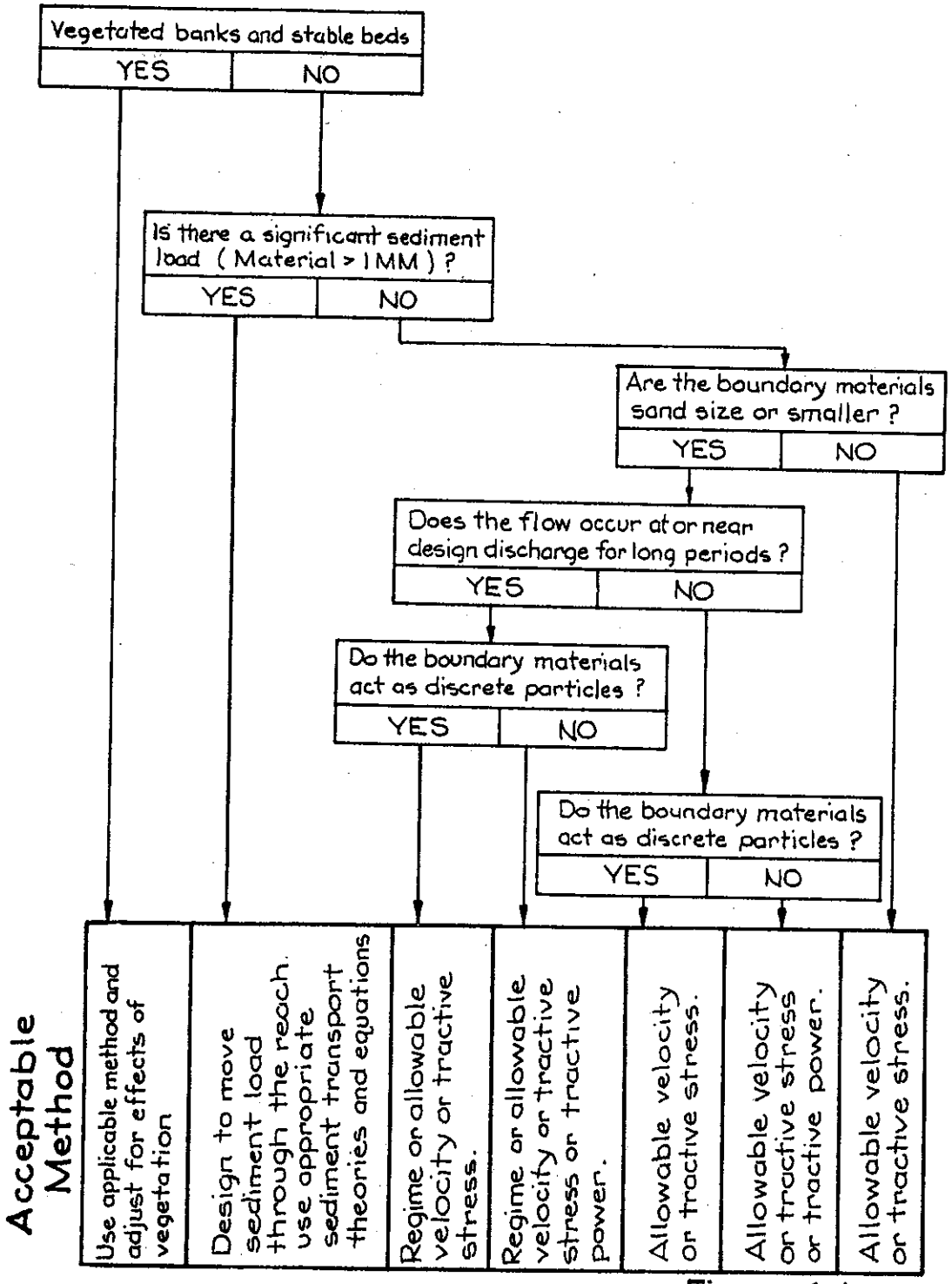


Figure 6-1

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

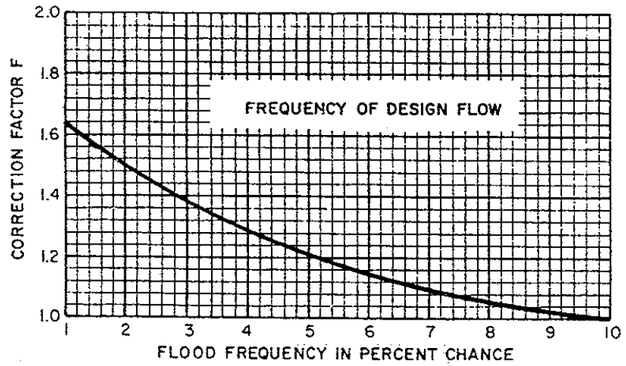
General

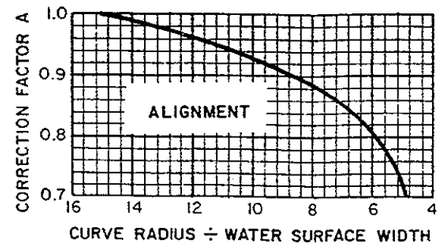
This method of testing the erosion resistance of earth channels is based on data collected by several investigators.

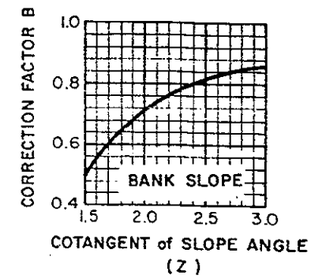
Figure 6-2 shows "Allowable Velocities for Unprotected Earth Channels" developed chiefly from data by Fortier and Scobey[24/], Lane[25/], by investigators in the U.S.S.R.[26/] and others. The allowable velocities determined from Figure 6-2 refer to channels formed in earth with no vegetative or structural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

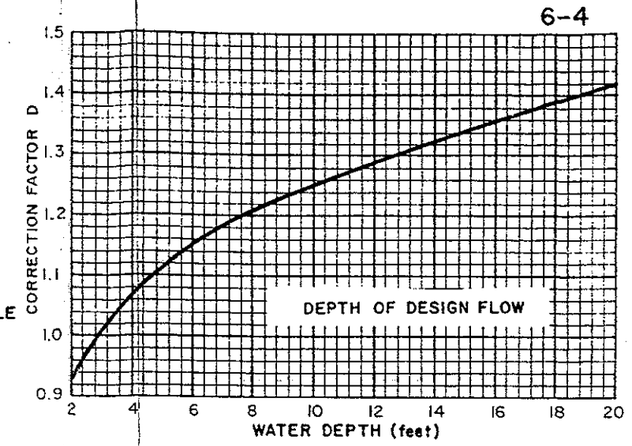
Stability is influenced by the concentration of fine material carried by the flow in suspension. There are two distinct types of flow depending on concentration of material in suspension.

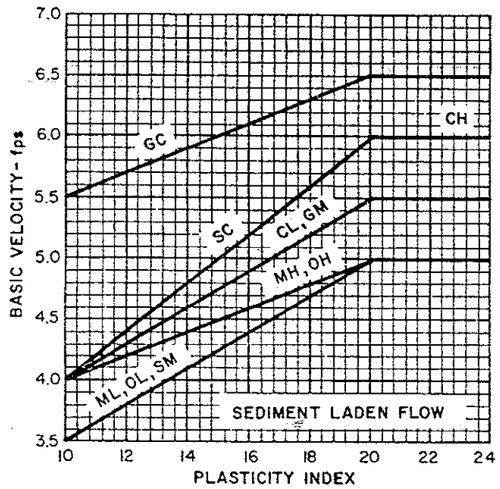
1. Sediment free flow is defined as the condition in which fine material is carried in suspension by the flow at concentrations so low that it



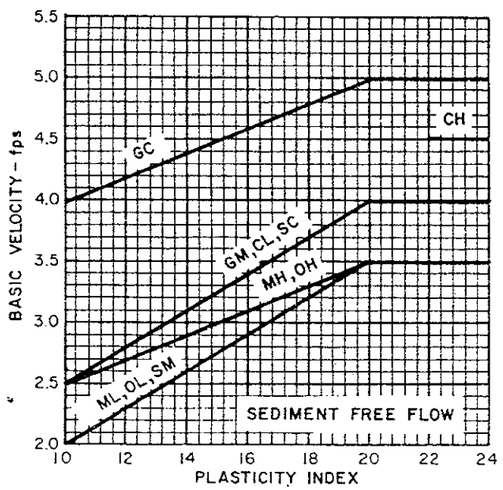








BASIC VELOCITIES FOR COHERENT EARTH MATERIALS (v_b)



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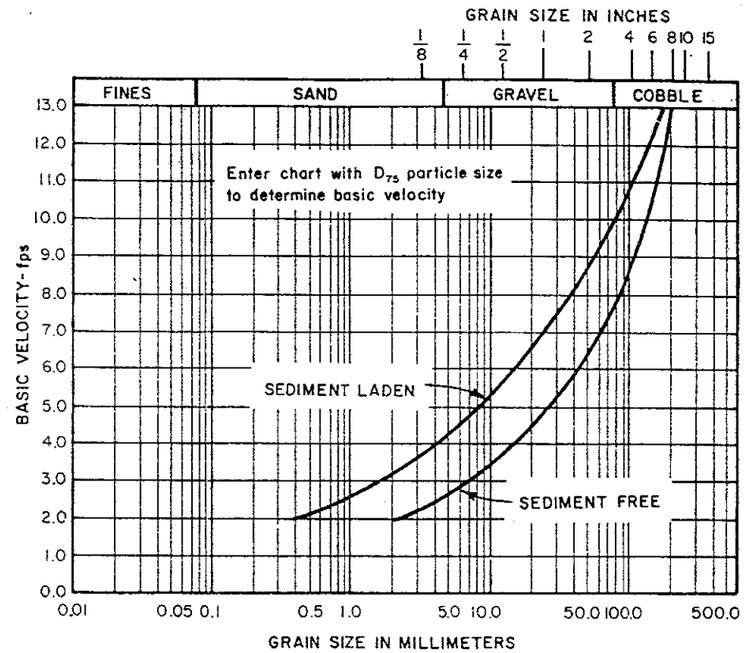
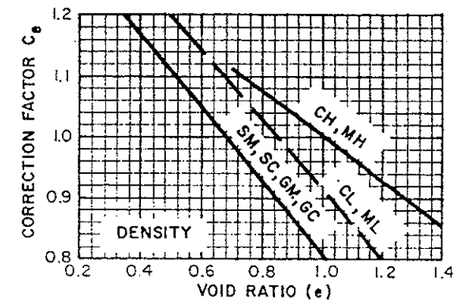


BASIC VELOCITY FOR DISCRETE PARTICLES OF EARTH MATERIALS (v_b)



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

NOTES:

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

→ conservative approach. For $d_{75} < 0.4$ mm depth of flow is not considered, however, Tractive Stress or Power will allow a higher velocity, flow depth rechecked.

FIGURE 6-2

ALLOWABLE VELOCITIES FOR UNPROTECTED EARTH CHANNELS

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

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

   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

$V_a = V_b DB = (6.7)(1.22)(0.72)$ straight reaches

$= 5.88$ fps

$V_a = V_b DBA = (6.7)(1.22)(0.72)(0.89)$ 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

$V_a = V_b DB = (9.0)(1.22)(0.72) = 7.91$ fps for straight reaches

$V_a = V_b DBA = (9.0)(1.22)(0.72)(0.89) = 7.04$ 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. <u>GM Layer</u>

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.

Restrictions:
1. channel does not aggrade
2. no bed load and no deposition on banks
3. Non-cohesive, discrete particle soils only

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. γS

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.

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

Appropriate for particle roughness

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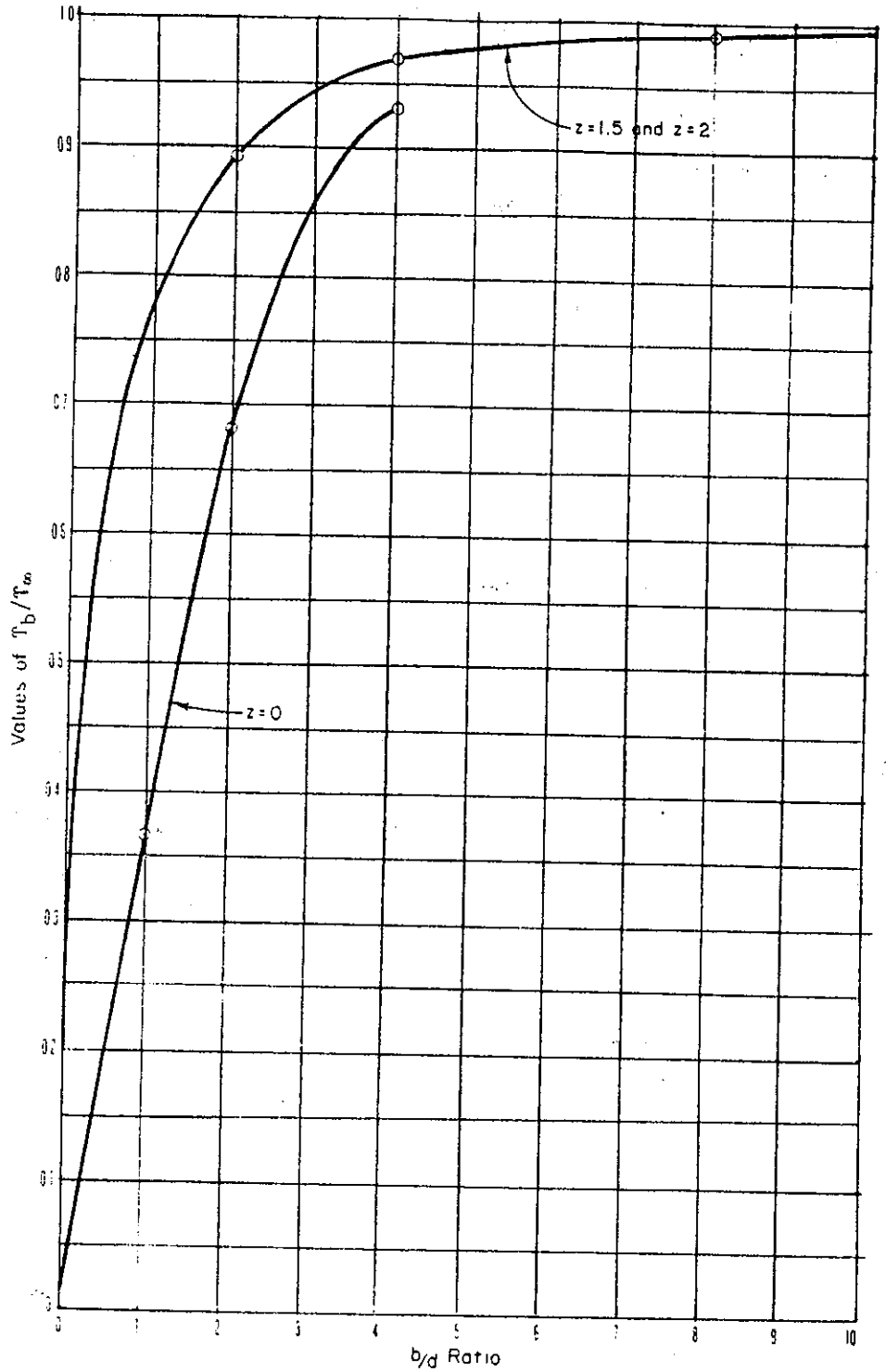
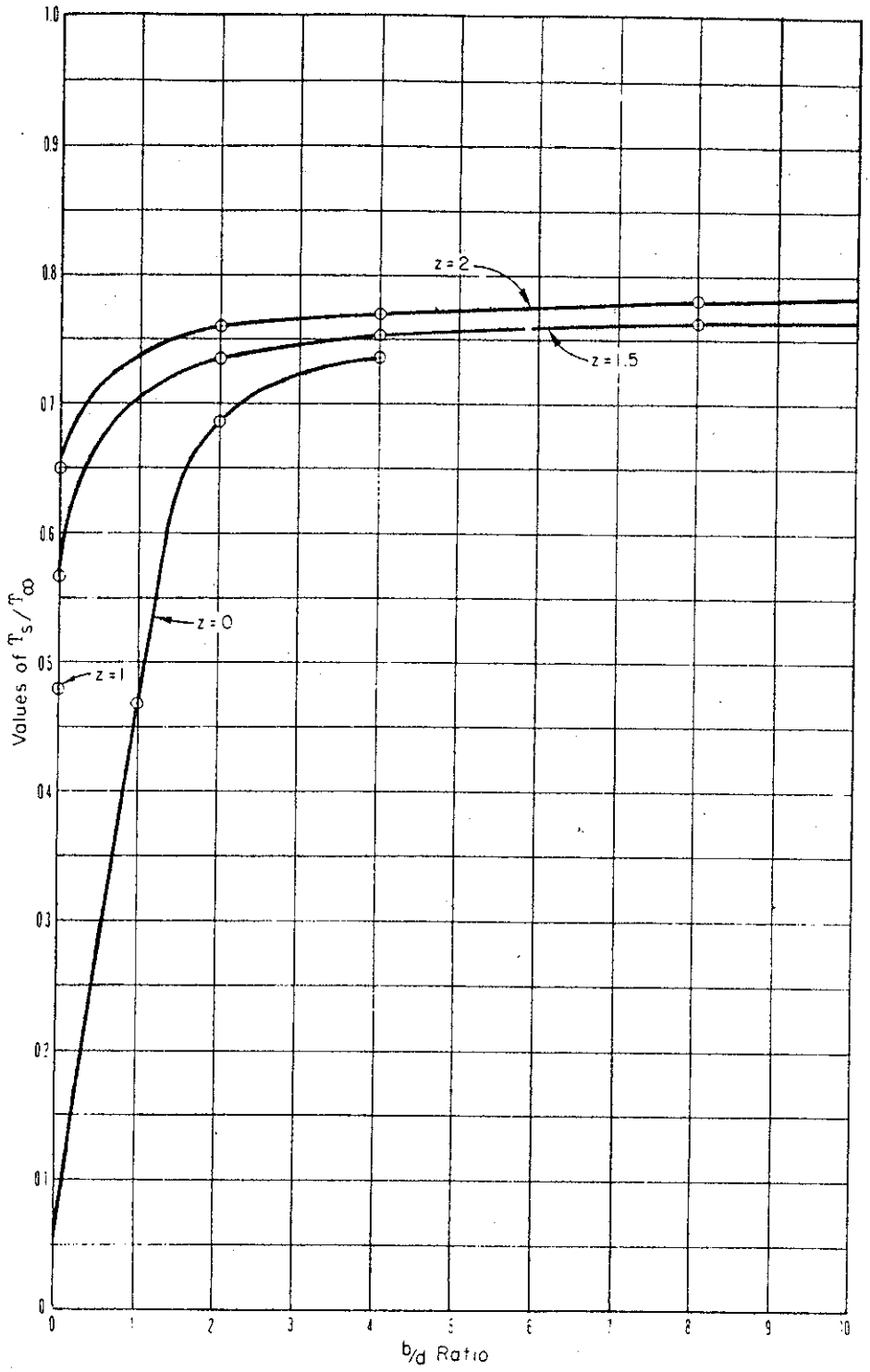
HYDRAULICS: Channel stability; actual maximum tractive stress, τ_b, on bed of straight trapezoidal channels



REFERENCE
Bureau of Reclamation Progress Report on Results of Studies for Design of Stable Channels, HYD-348

FIGURE 6-3

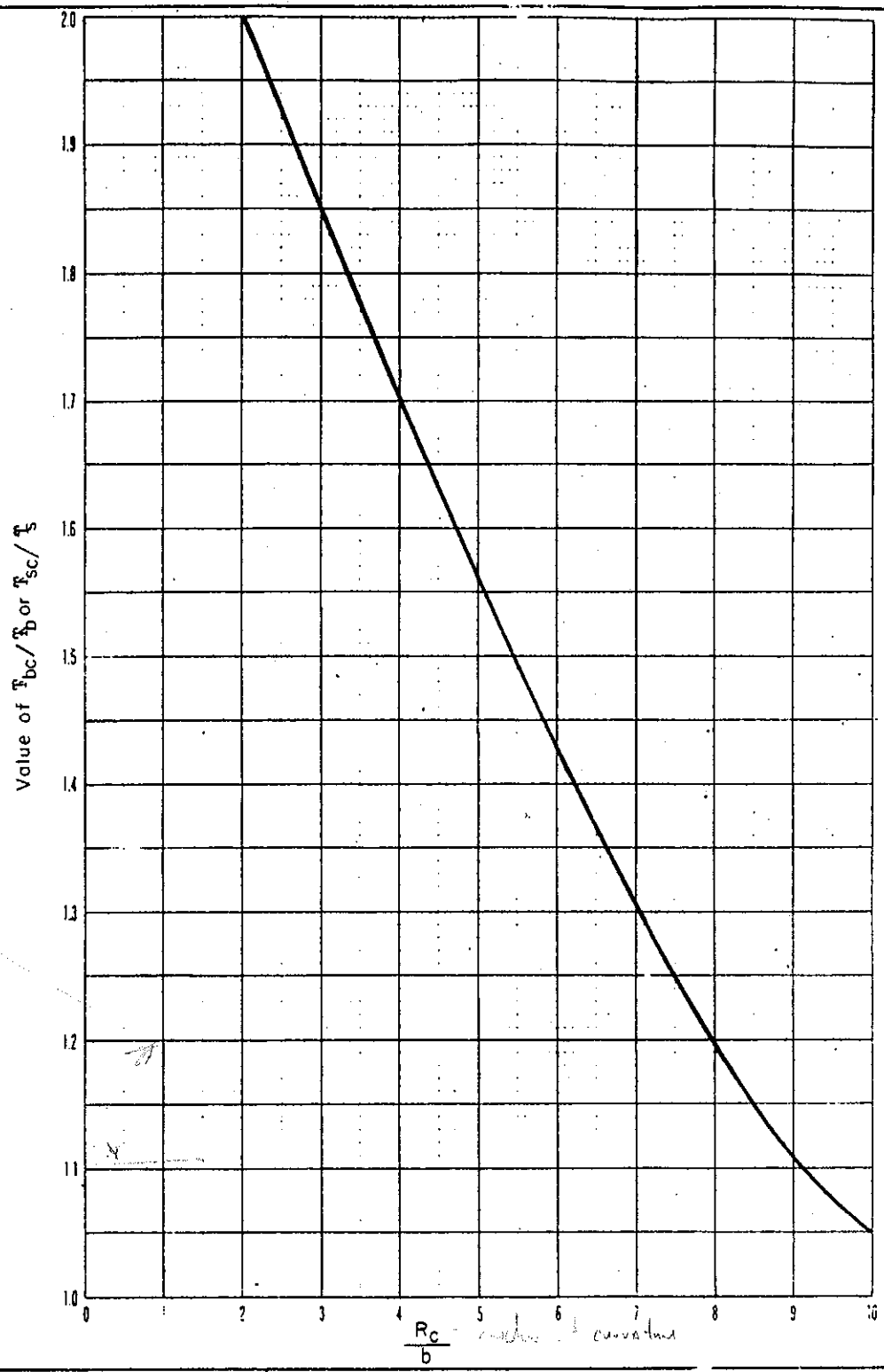
HYDRAULICS: Channel stability; actual maximum tractive stress, T_s, on sides of straight trapezoidal channels



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

FIGURE 6-4

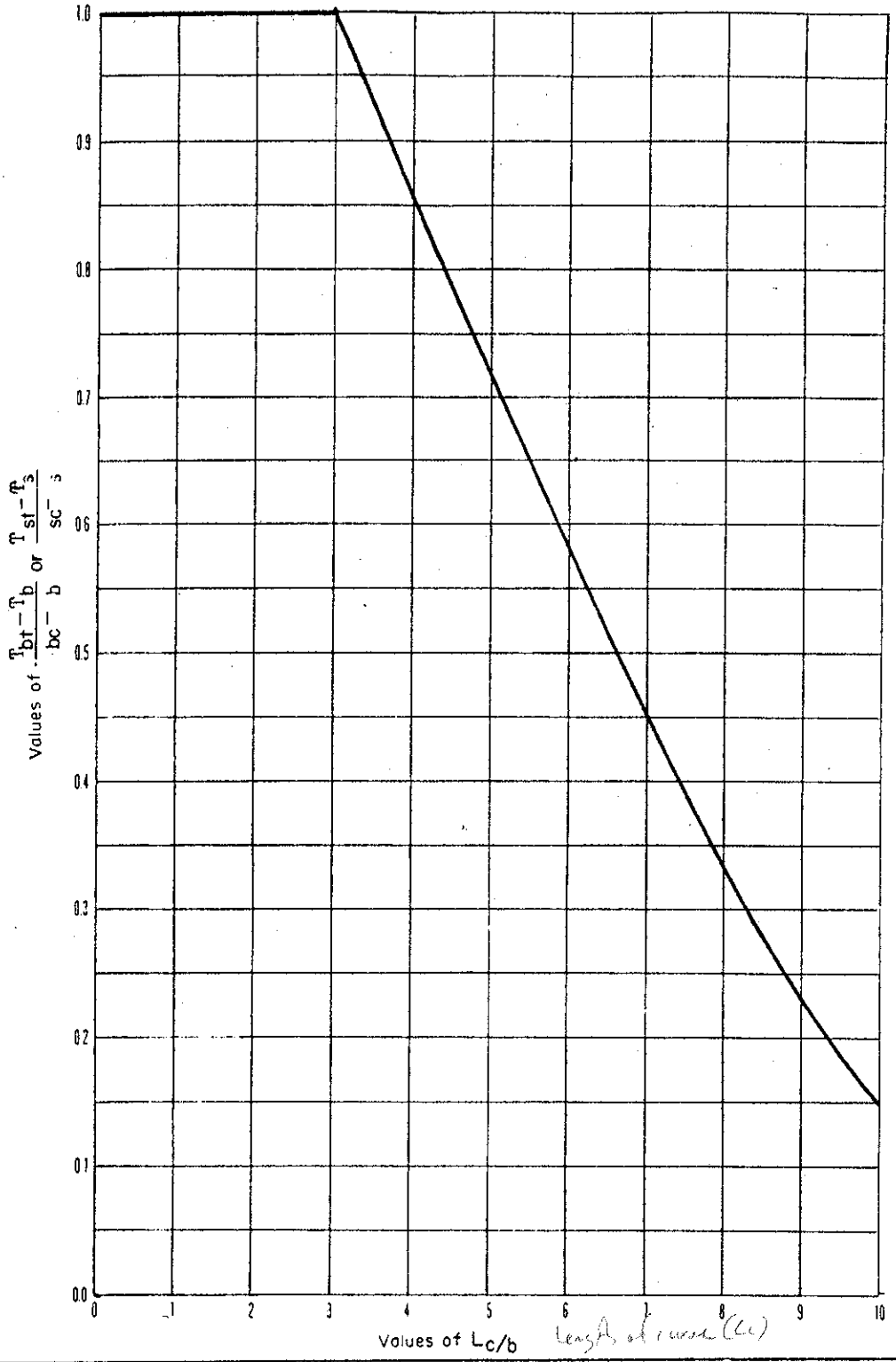
HYDRAULICS: Channel stability; actual maximum tractive stress, τ_{bc} and τ_{sc}, on bed and sides of trapezoidal channels within a curved reach



REFERENCE
Lane, Emory W., Design of Stable Channels Transaction, ASCE, vol 120, 1955
Nece, R E, Gyler, C A, and Drinker, P A, Measurement of Boundary Shear Stress in an Open Channel Curve with a Surface Pitot Tube M. I. T. Technical Note (no 6), August 1959

FIGURE 6-5

HYDRAULICS: Channel stability; actual maximum tractive stresses Y_{bt} and Y_{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 in. < D_{75} < 5.0 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

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

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

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.

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

<u>Fine Grained Soils - D_{75} < 1/4 inch</u> 6.35 mm > #4 sieve

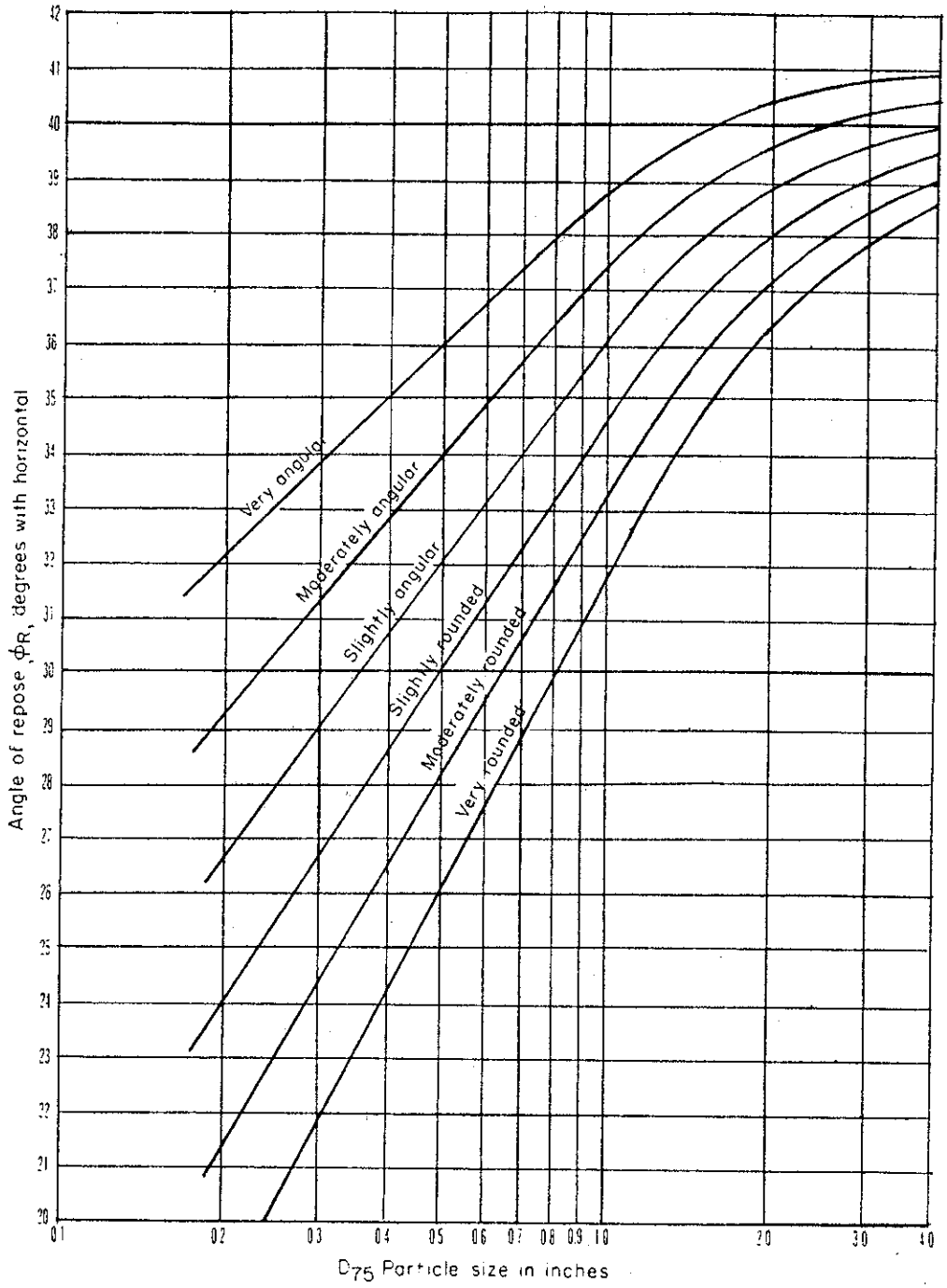
A. Determination of Actual Tractive Stress

1. Reference tractive stress

The expression for the reference tractive stress is:

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

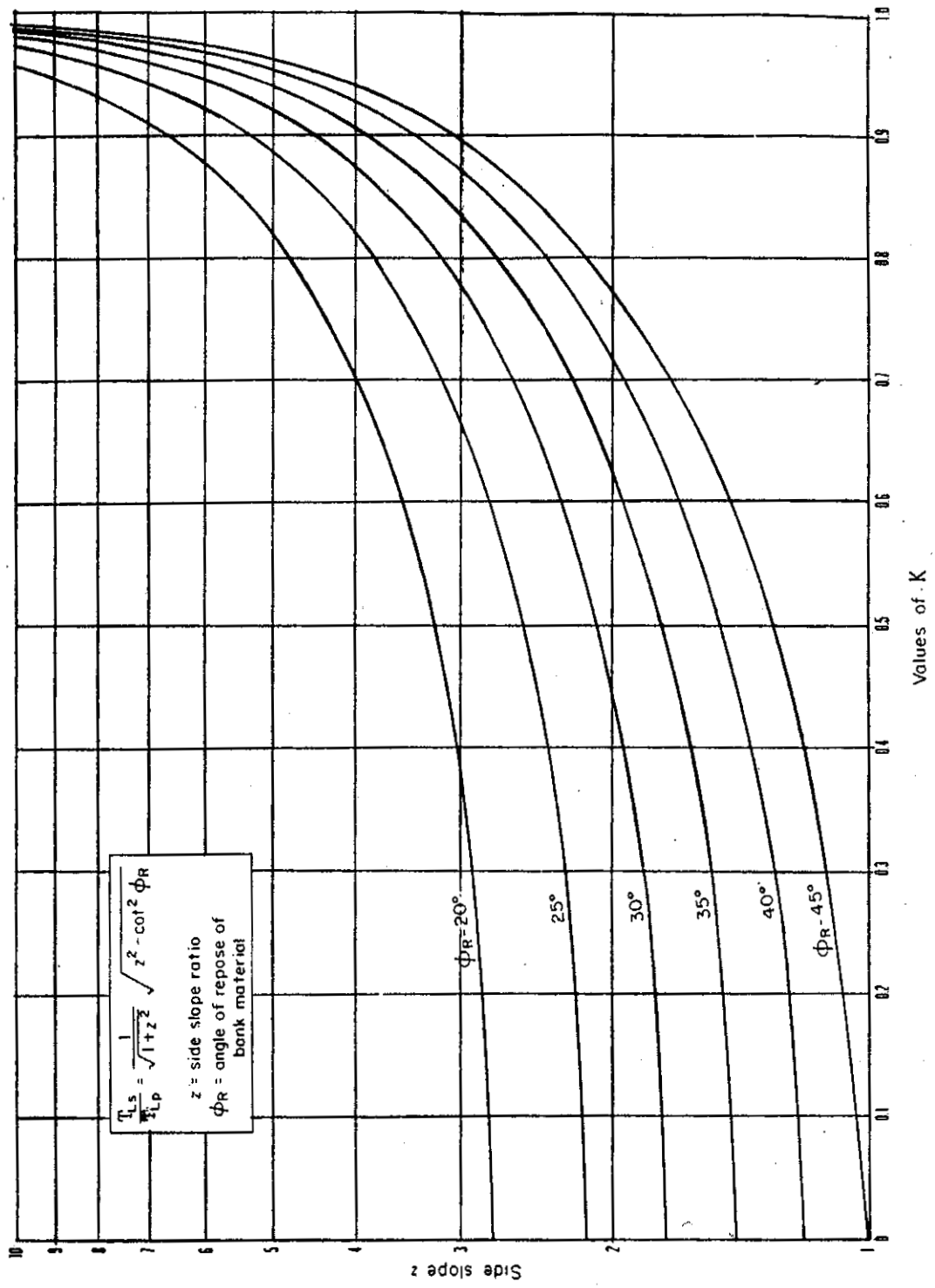
HYDRAULICS: Channel stability; angle of respose, ϕ_R, for non-cohesive materials



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

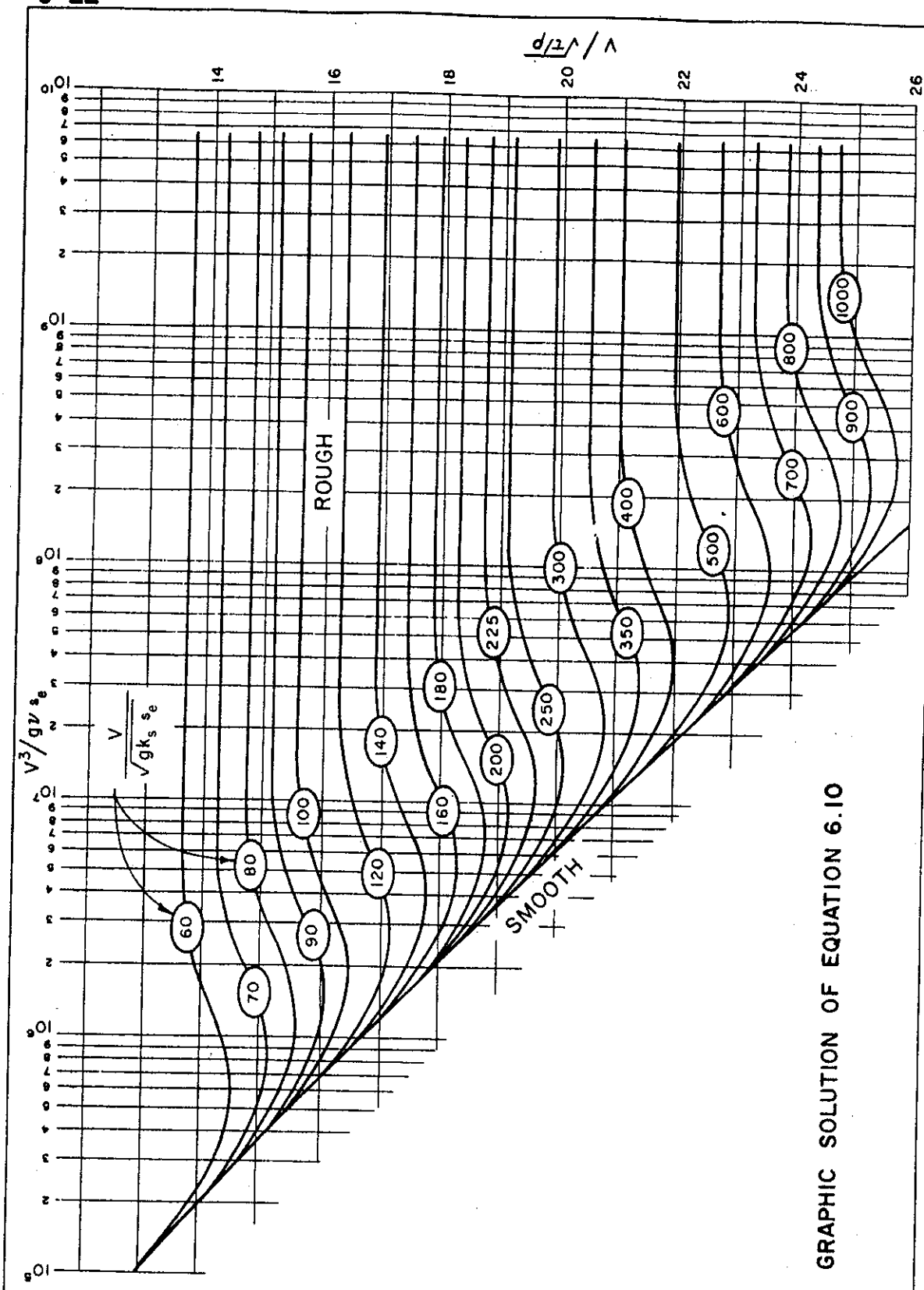
Figure 6-11 gives curves from which values of density ρ and kinematic viscosity of the water ν can be obtained.

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.



GRAPHIC SOLUTION OF EQUATION 6.10

FIGURE 6-9

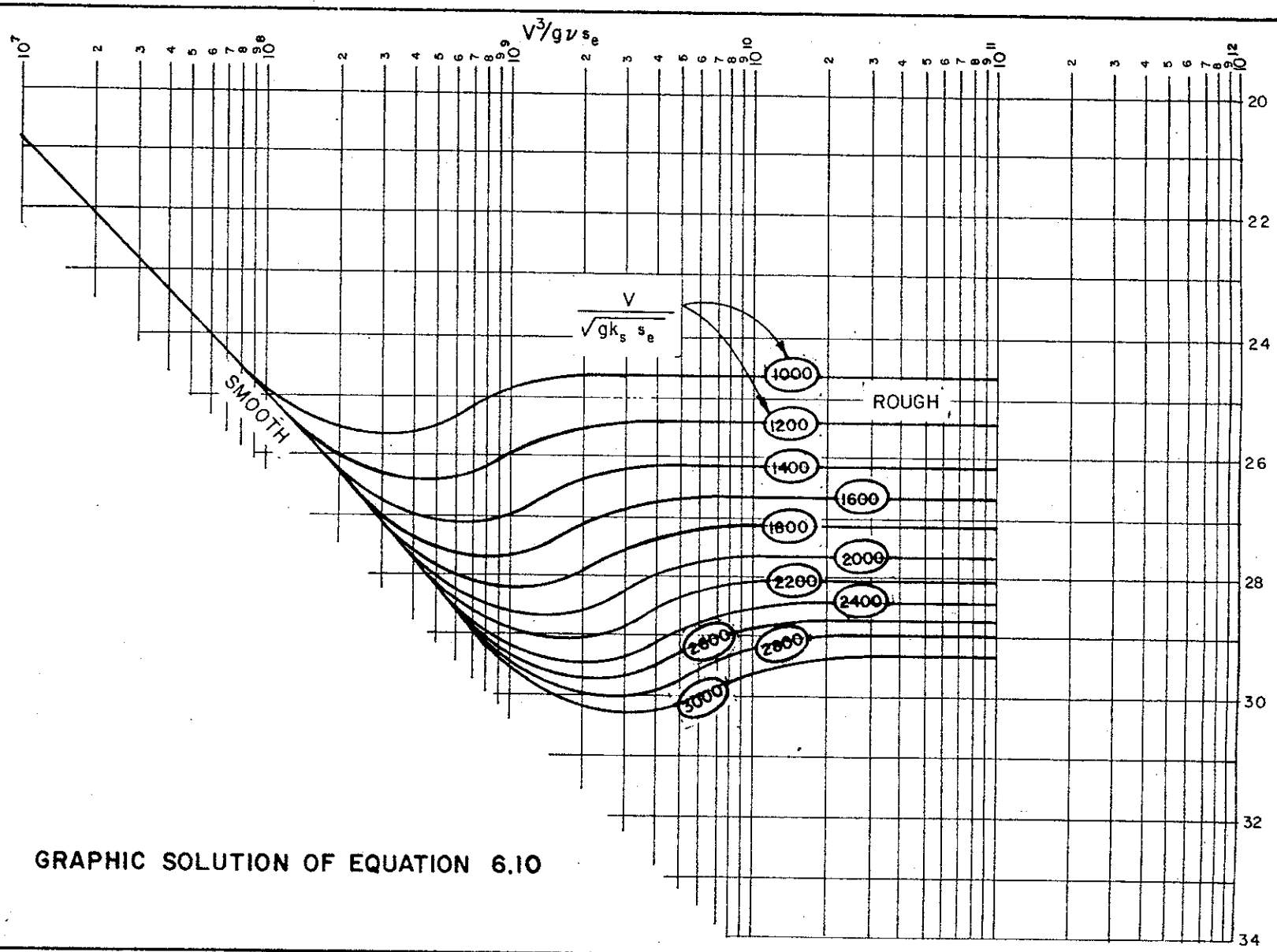
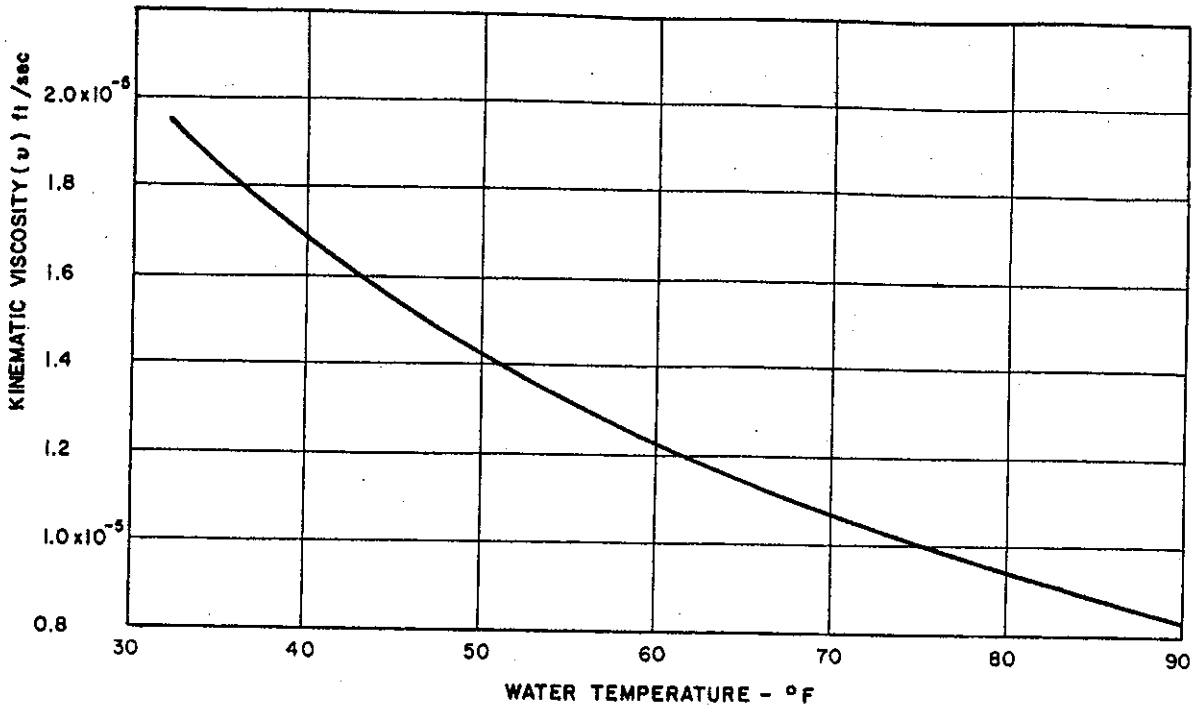
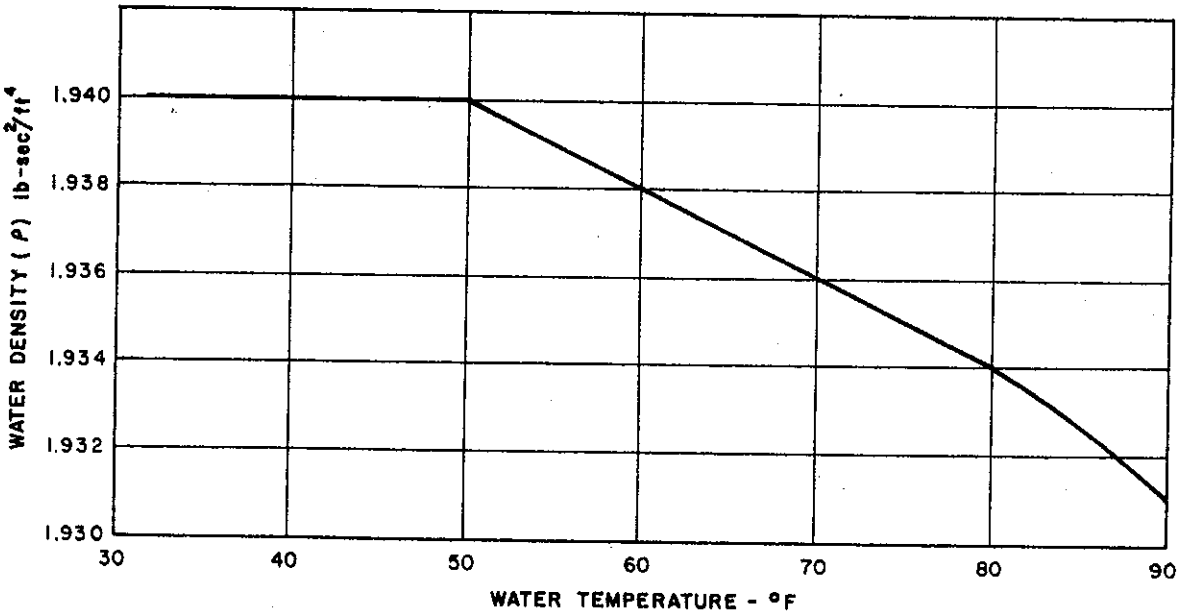


FIGURE 6-10

GRAPHIC SOLUTION OF EQUATION 6.10





Values of ρ and ν for various water temperatures

FIGURE 6-11

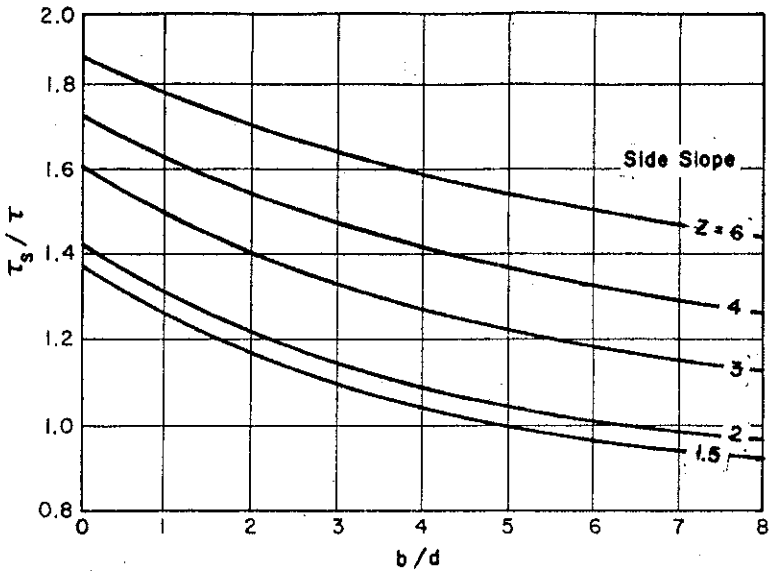


FIGURE 6-12 : Applied Maximum Tractive Stresses, τ_s, On Sides Of Straight Trapezoidal Channels.

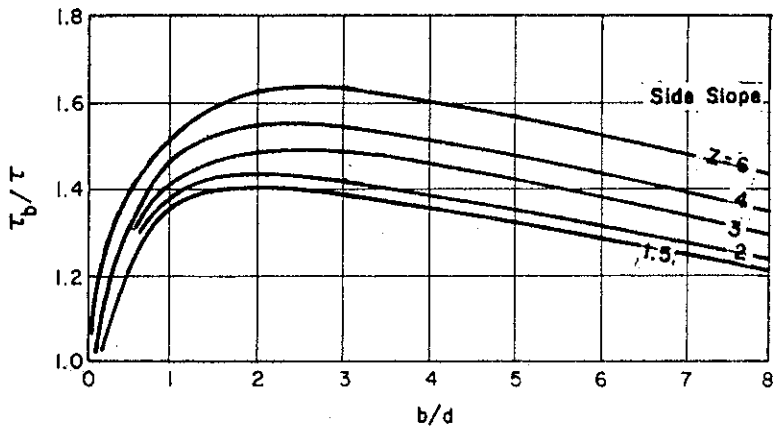


FIGURE 6-13 : Applied Maximum Tractive Stresses, τ_b, On Bed Of Straight Trapezoidal Channels.

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

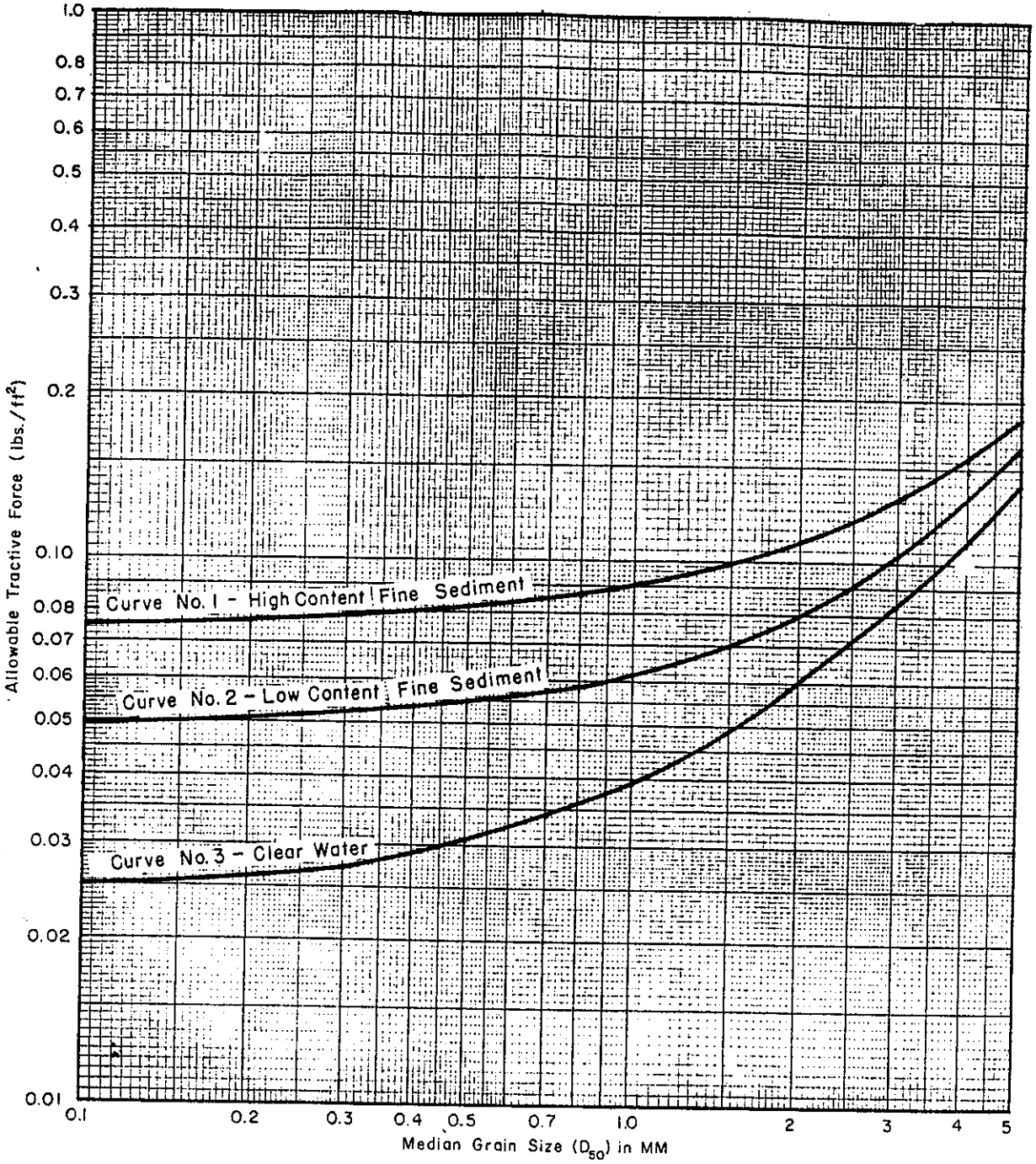


FIGURE 6-14

ALLOWABLE TRACTIVE STRESS

NON-COHESIVE SOILS, $D_{75} < 0.25''$

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 ~#4 sieve = 4.75 mm = 0.19 in

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 $\dfrac{V}{\sqrt{gk_s s_e}}$.

5. Enter the graph in Figure 6-9 (or Figure 6-10) with the computed values in steps 2 and 3 above and read the value of $\dfrac{V}{\sqrt{\tau/\rho}}$.

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.

Examples - Tractive Stress Approach

Example 6-5

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

Determine: The actual and allowable tractive stress.

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

$n_t = (0.90)^{1/6}/39 = 0.0252$ (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 - 12

$\nu = 1.42 \times 10^{-5}$ 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

$\tau_s/\tau \simeq 1.0$; $\tau_b/\tau = 1.31$ (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

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

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

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

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

Tractive Power Approach

(E. Manasota floodway)

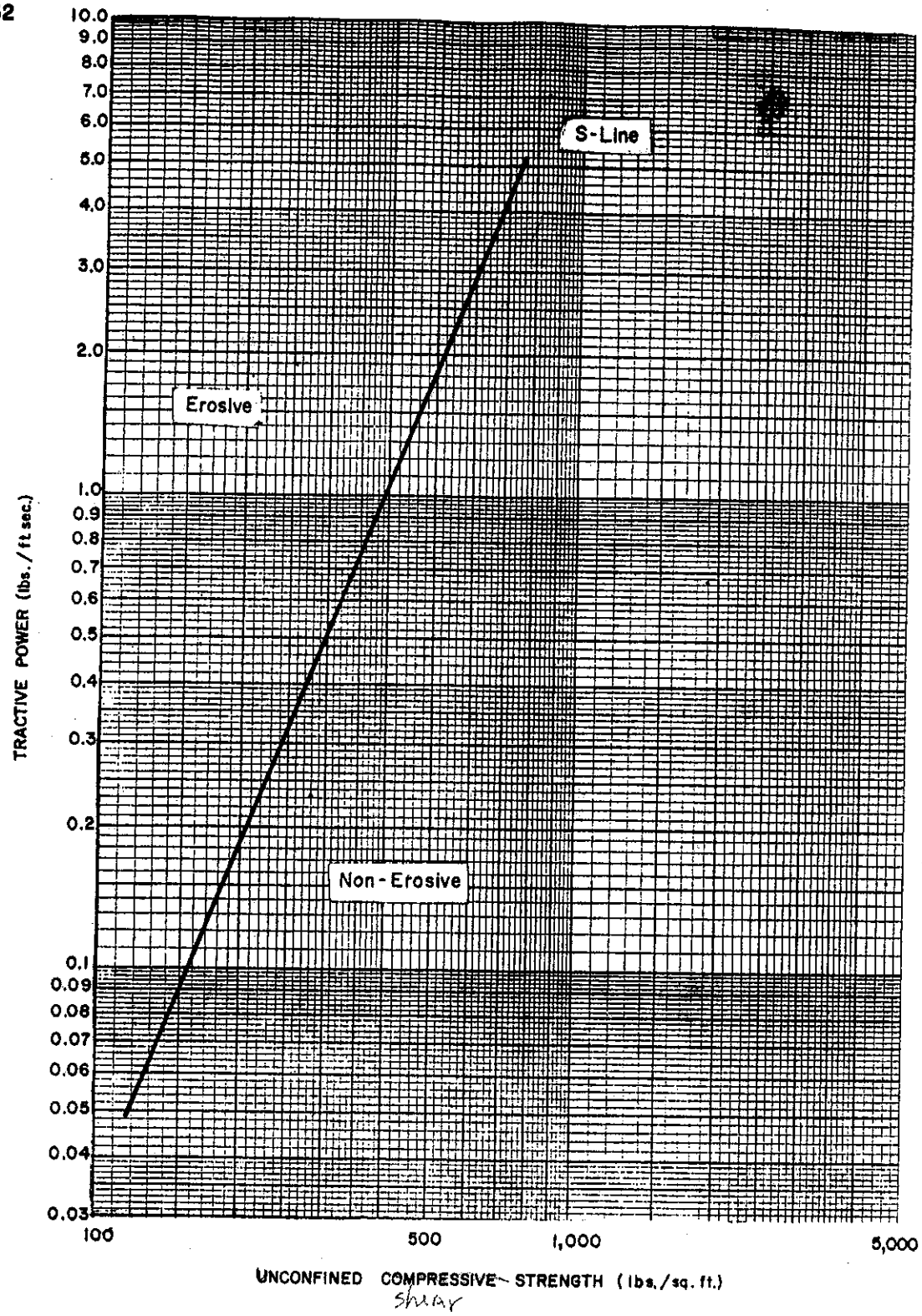
General

Many channels have failed using this approach

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.



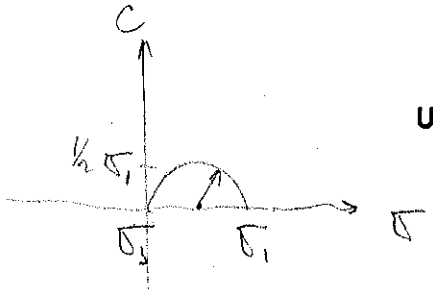


Figure 6-15
Unconfined Compressive Strength And Tractive Power As Related To Channel Stability

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

good for irrigation ditches

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.

Limitations

1. No significant sediment load > 1mm
2. boundary material ≤ sand
3. long duration flow
4. not supplied water by designers
5. no vegetation not 1/4 on banks
6. Valid for some Q
7. Transport flow $F_r < 0.3$

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.

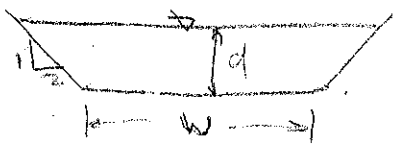
Determine: Design the channel.

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 ÷ A = 600 ÷ 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.

$R = \frac{A}{P} = \frac{(d \times w) + 2[\frac{1}{2} d 2d]}{w + \sqrt{5}d}$

$= \frac{dw + 2d^2}{w + \sqrt{5}d}$

if $F = \frac{V}{\sqrt{gd}} = 0.3$

$V = 0.3\sqrt{gd} = 1.7\sqrt{d}$

if $d = 7$, $V = 4.5 \Rightarrow A = Q/V = 600/4.5 = 133.33$

$A = dw + 2d^2 =$

$133.33 = 7w + 2(7)^2$

$w = 5.05$ ft

$R = 6.99$

$2 = 0.43(600)^{0.361} + (4.33)$

Step 6 - Compute the Froude Number.

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

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

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

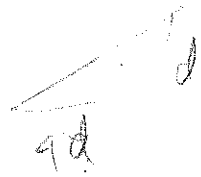
$59.76 = 0.92\ W_T - 2.0$

$W_T = 67.1$ 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}

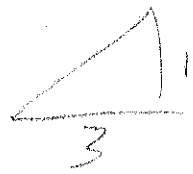
$R = \frac{A}{P} = \frac{294.7}{68.7} = 4.29$ ft.

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

Compute s_o

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

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



$s_o = 0.00013$

$A = \frac{1}{2} d (3d)$

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.

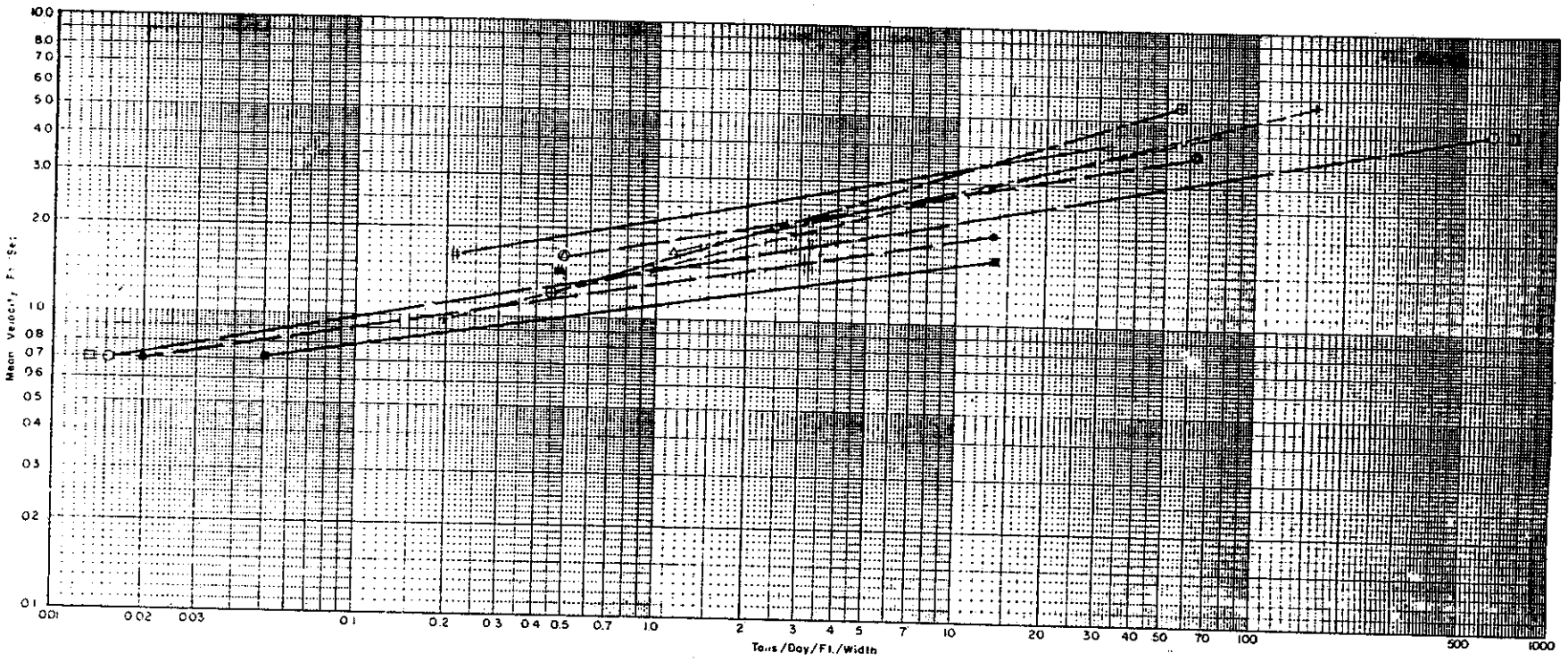
Sediment Transport in Sand Bed Streams Not in Equilibrium

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

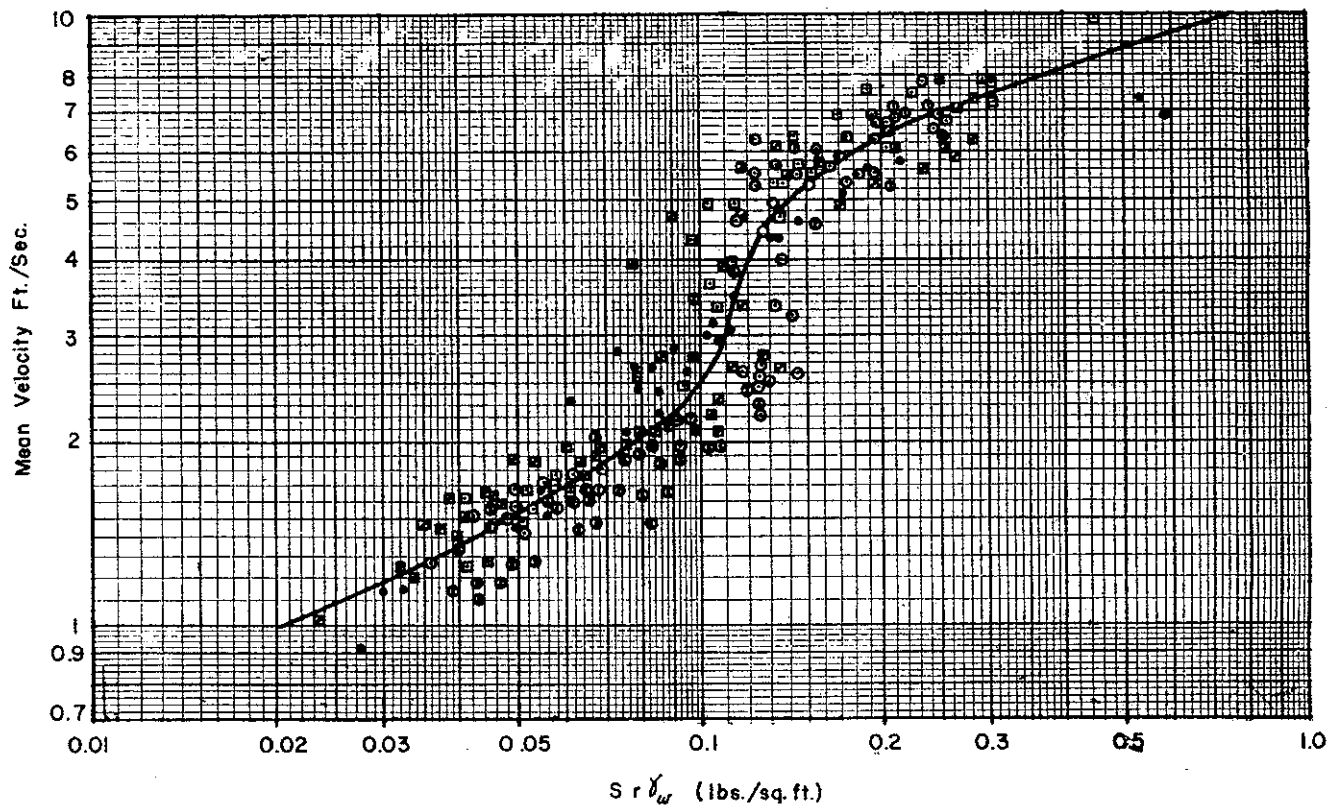
The following procedure may be used to determine whether unstable conditions will occur under projected channel conditions where variable bed roughness occurs:



NO	D_{35} SIZE	MEASURING POINT	WATER DEPTH	REFERENCE
1	○ 0.45 mm	SAND-FLUME	0.27'- 1.0'	U.S.G.S W.S. PAPER 1468-A 61
2	△ 0.594 mm	SAND-FLUME	0.074'-0.346'	CAL TECH. REPORT NO. K.H.R.2 JAN.61
3	◊ 0.233 mm	SAND-FLUME	0.145'-0.356'	CAL TECH REPORT NO K.H.R 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	+ 0.130 mm	RIO GRANDE AT BERNALILLO		U.S.B.R.-SED. CONF. PAPER NO. 32 DEC 63
8	# 0.27 mm	COLORADO R. AT ADOBE		U.S.B.R.-SED CONF PAPER NO. 32 DEC 63
9	⊕	AVE CURVE - FIVE MILE CR WY		TRANS AGU. VOL 38 NO 5 OCT 57
		NIOBRARA R MIDDLE LOUP, NEB		

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

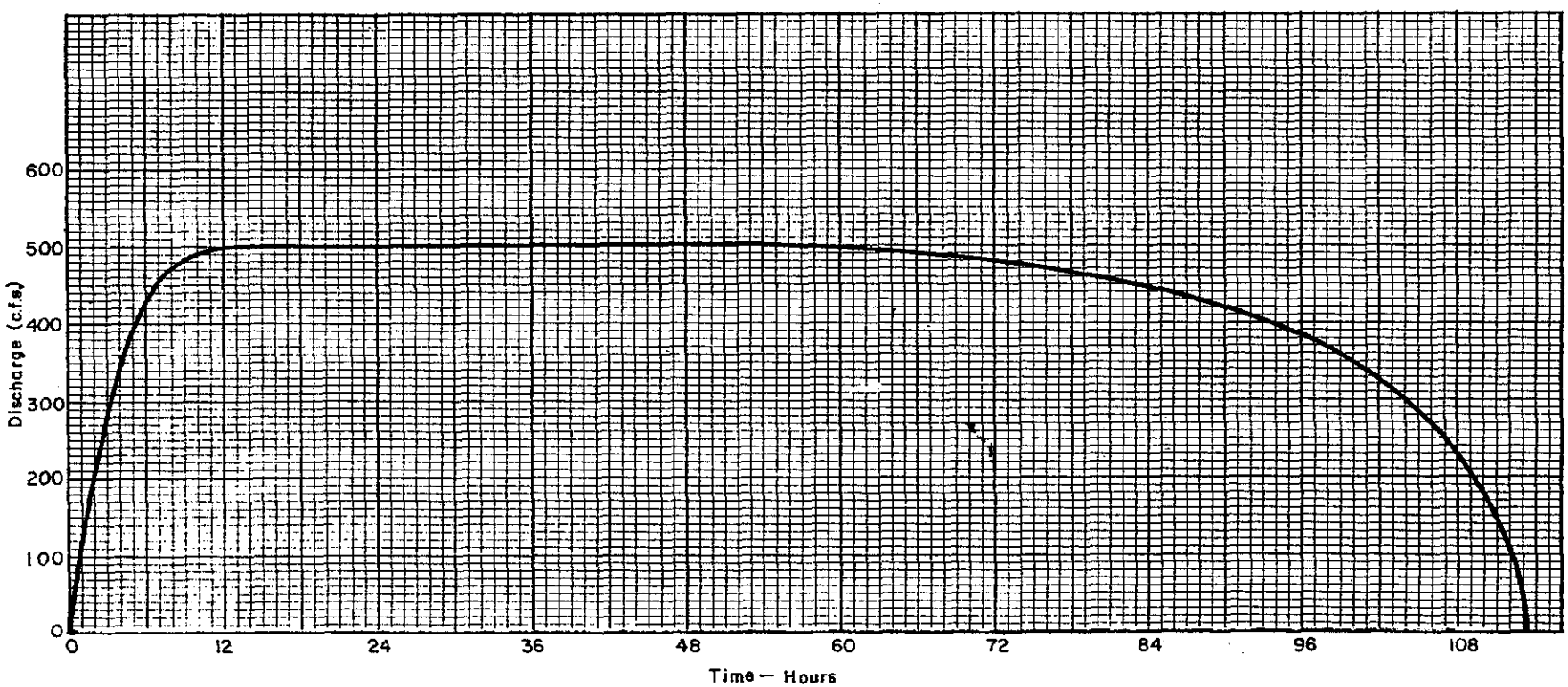

FIGURE 6-18
RESERVOIR RELEASE HYDROGRAPH
PRINCIPAL SPILLWAY

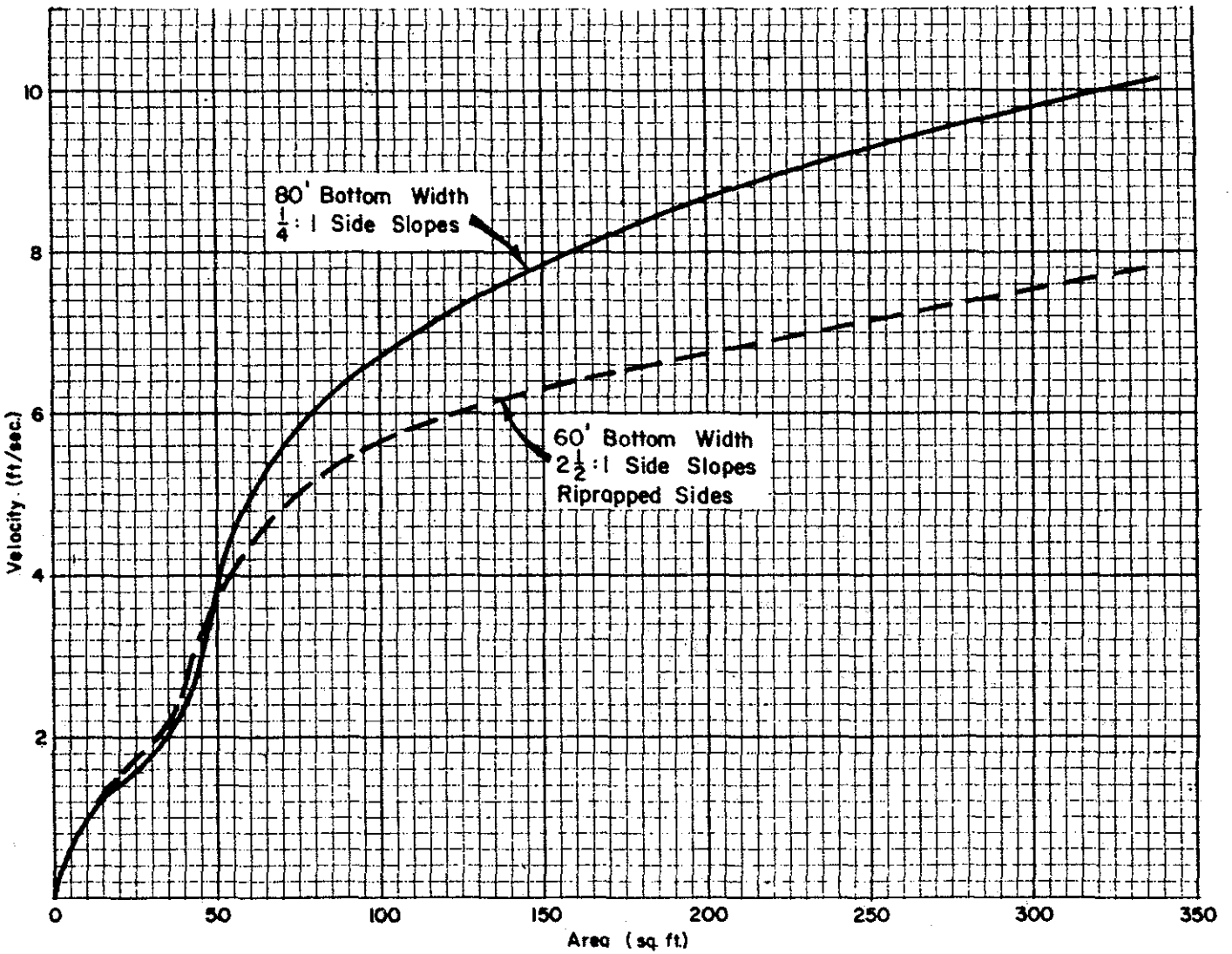

FIGURE 6-19
VELOCITY - AREA CURVE

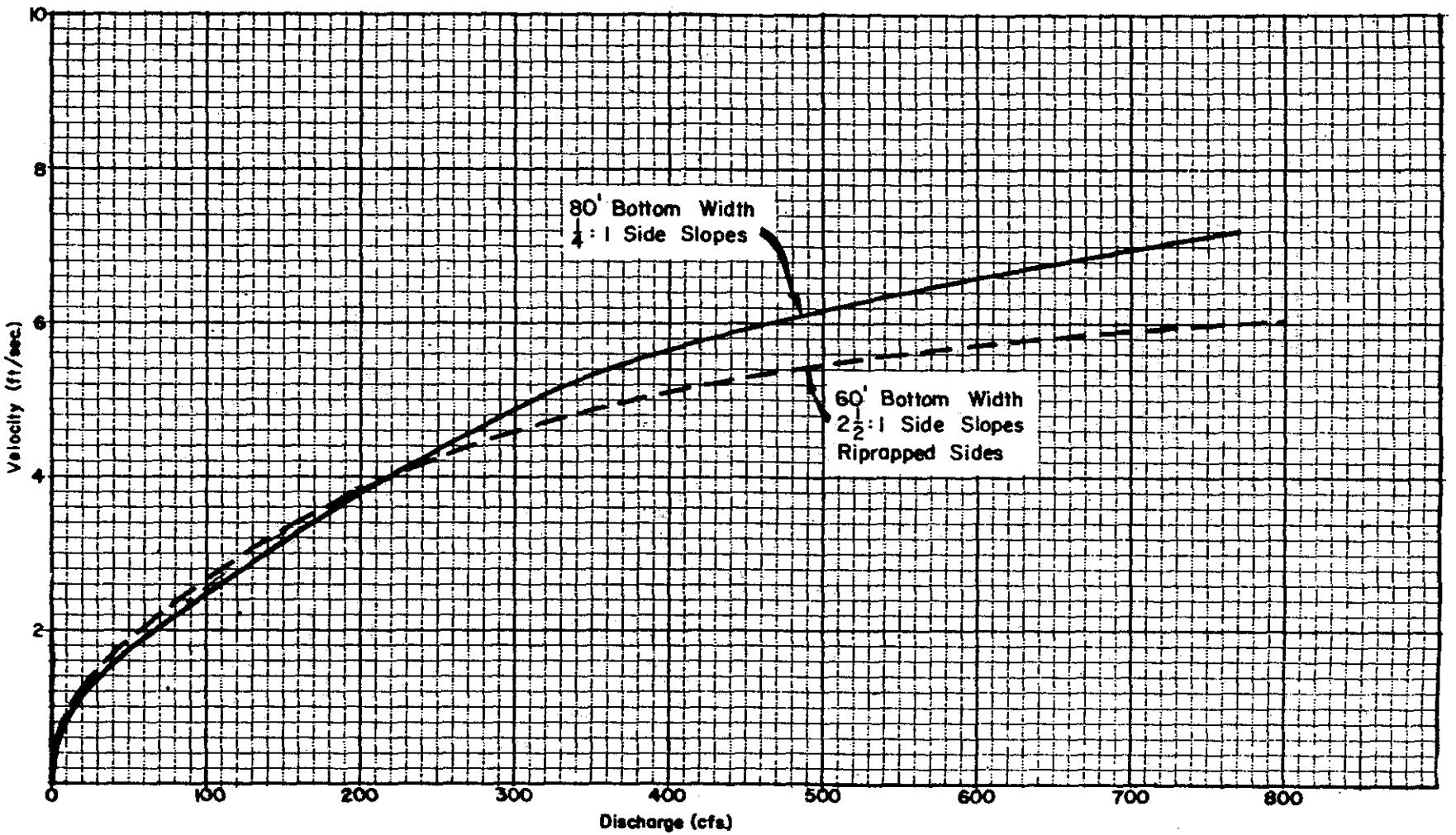

FIGURE 6-20
VELOCITY - DISCHARGE CURVE

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

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.

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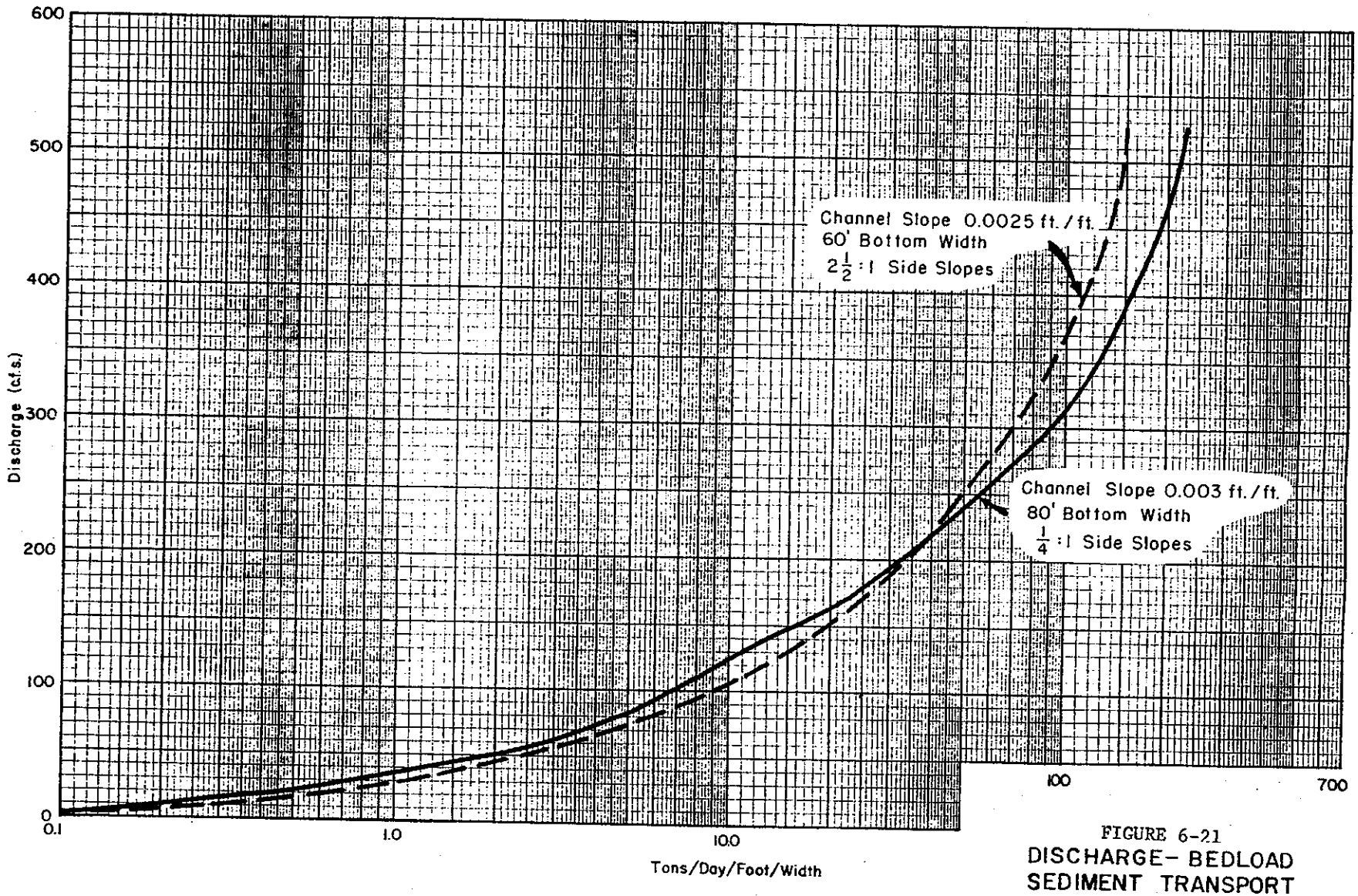


FIGURE 6-21
DISCHARGE- BEDLOAD
SEDIMENT TRANSPORT

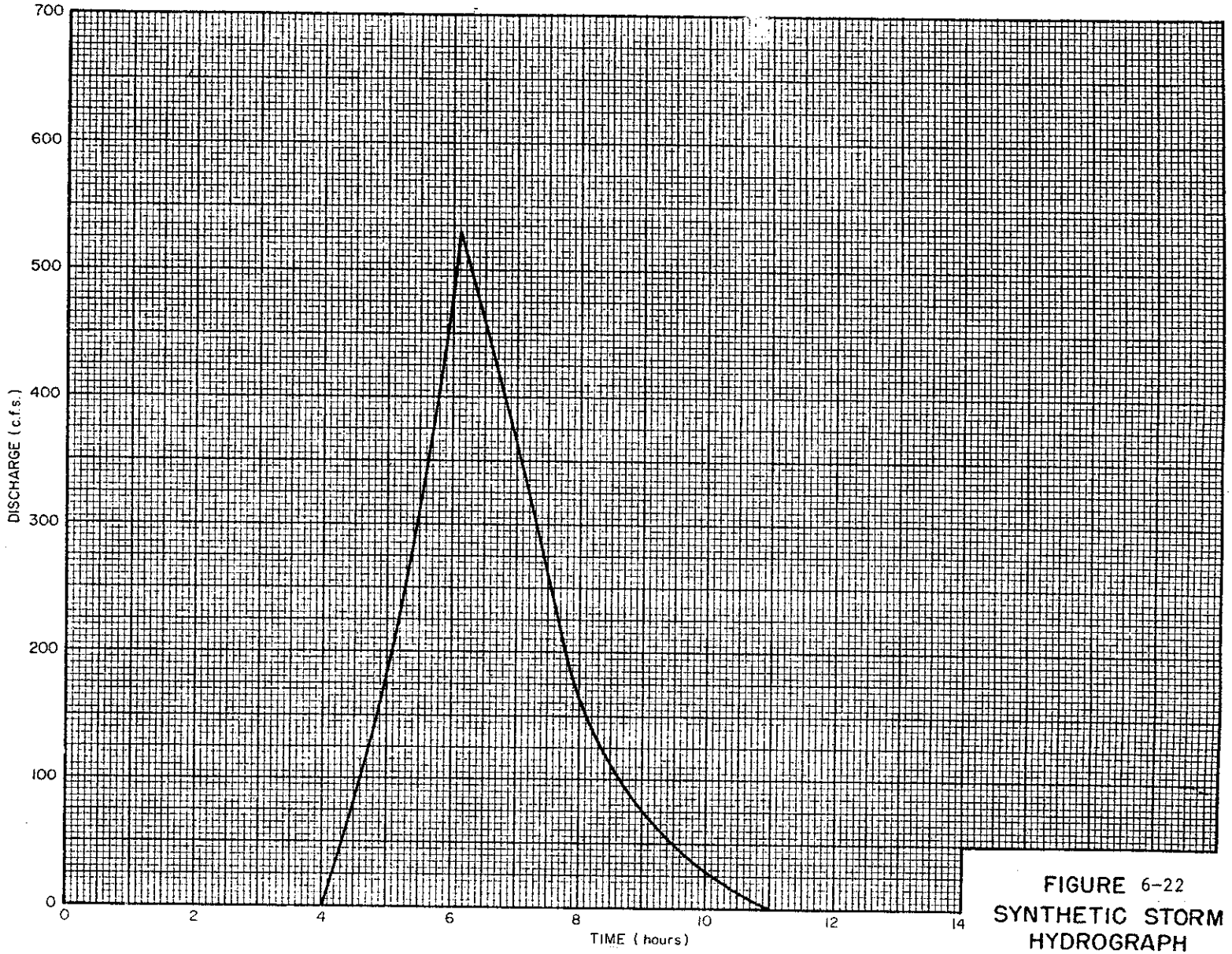


FIGURE 6-22
SYNTHETIC STORM
HYDROGRAPH

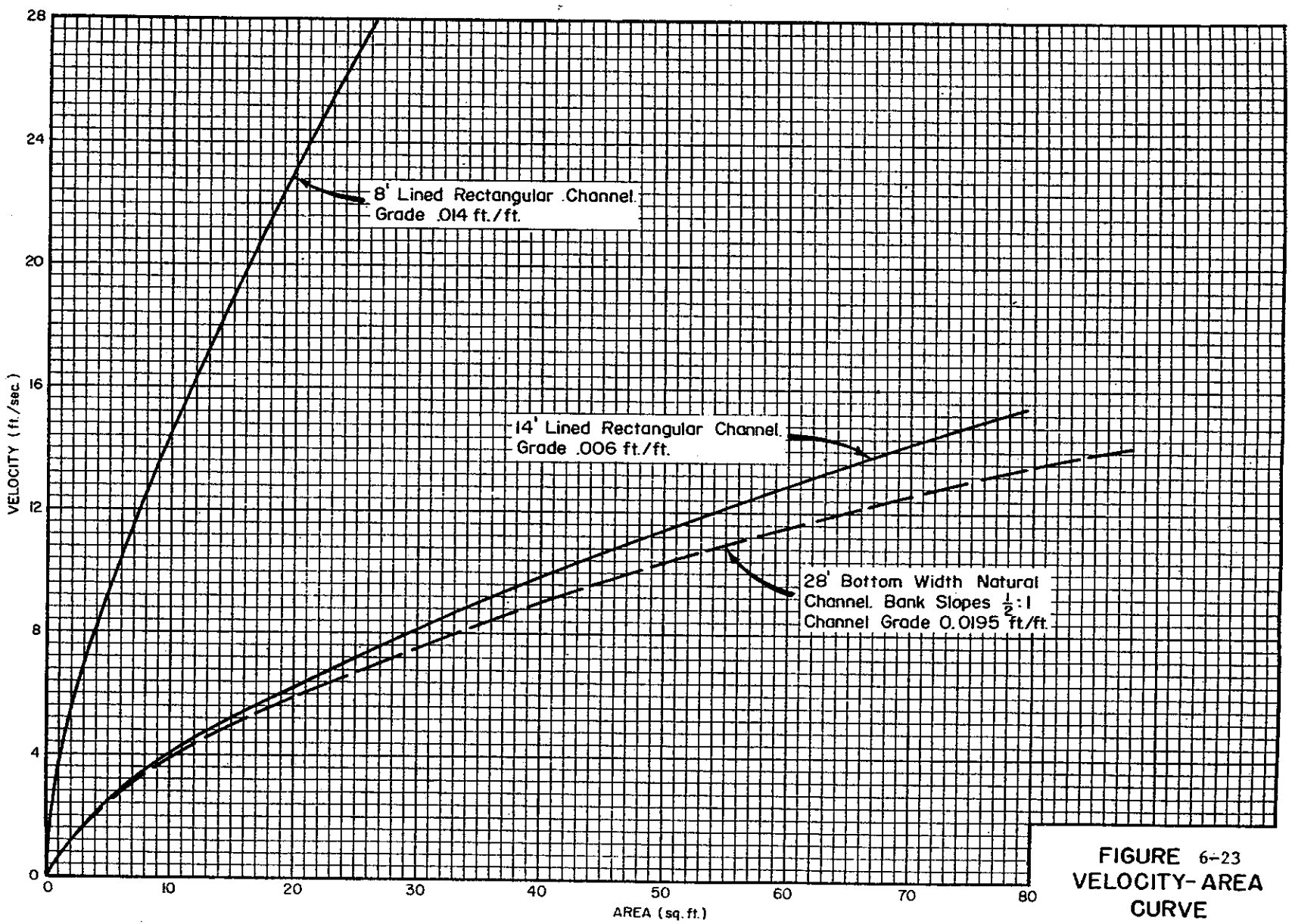


FIGURE 6-23
VELOCITY-AREA CURVE

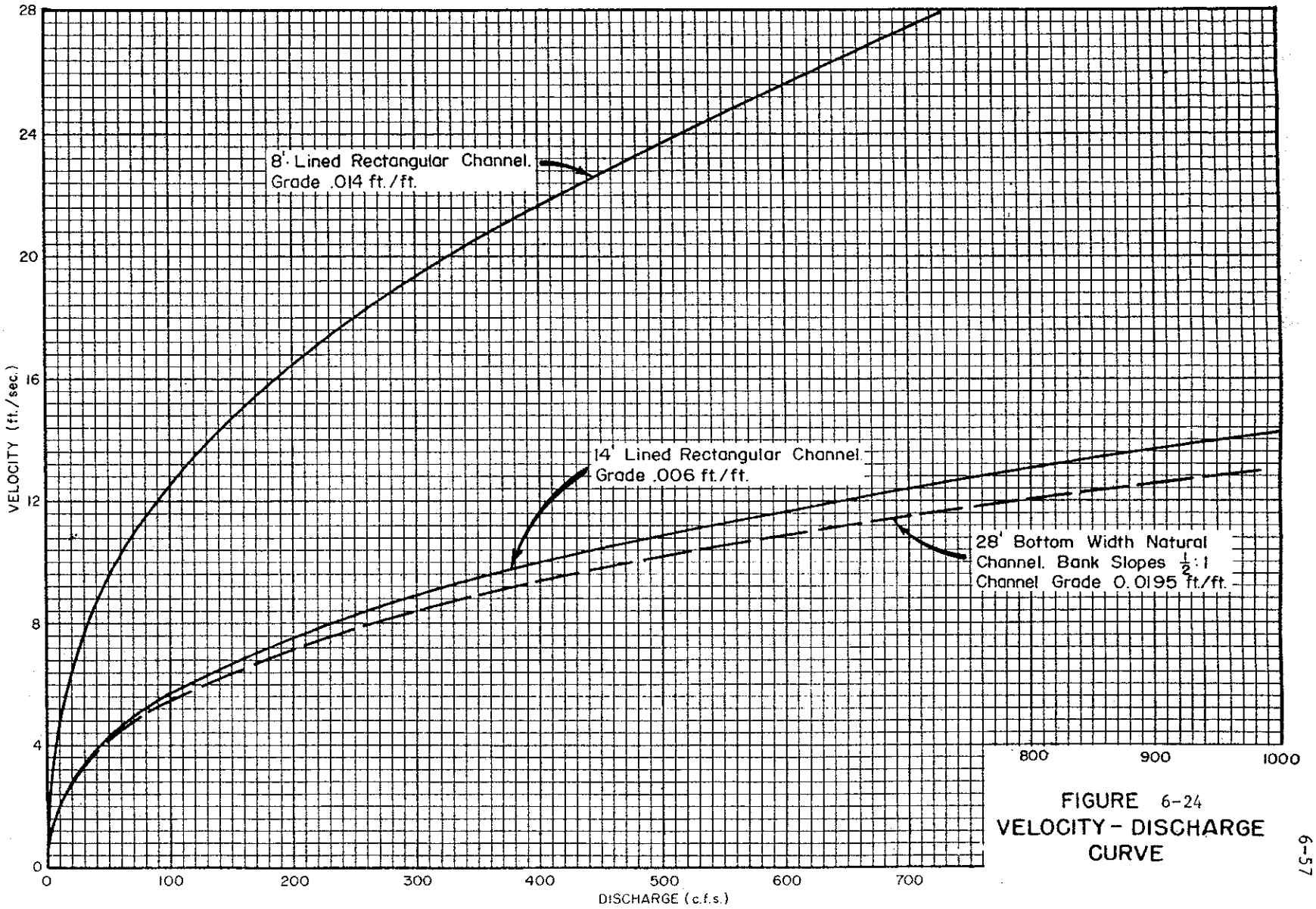


FIGURE 6-24
VELOCITY - DISCHARGE CURVE

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

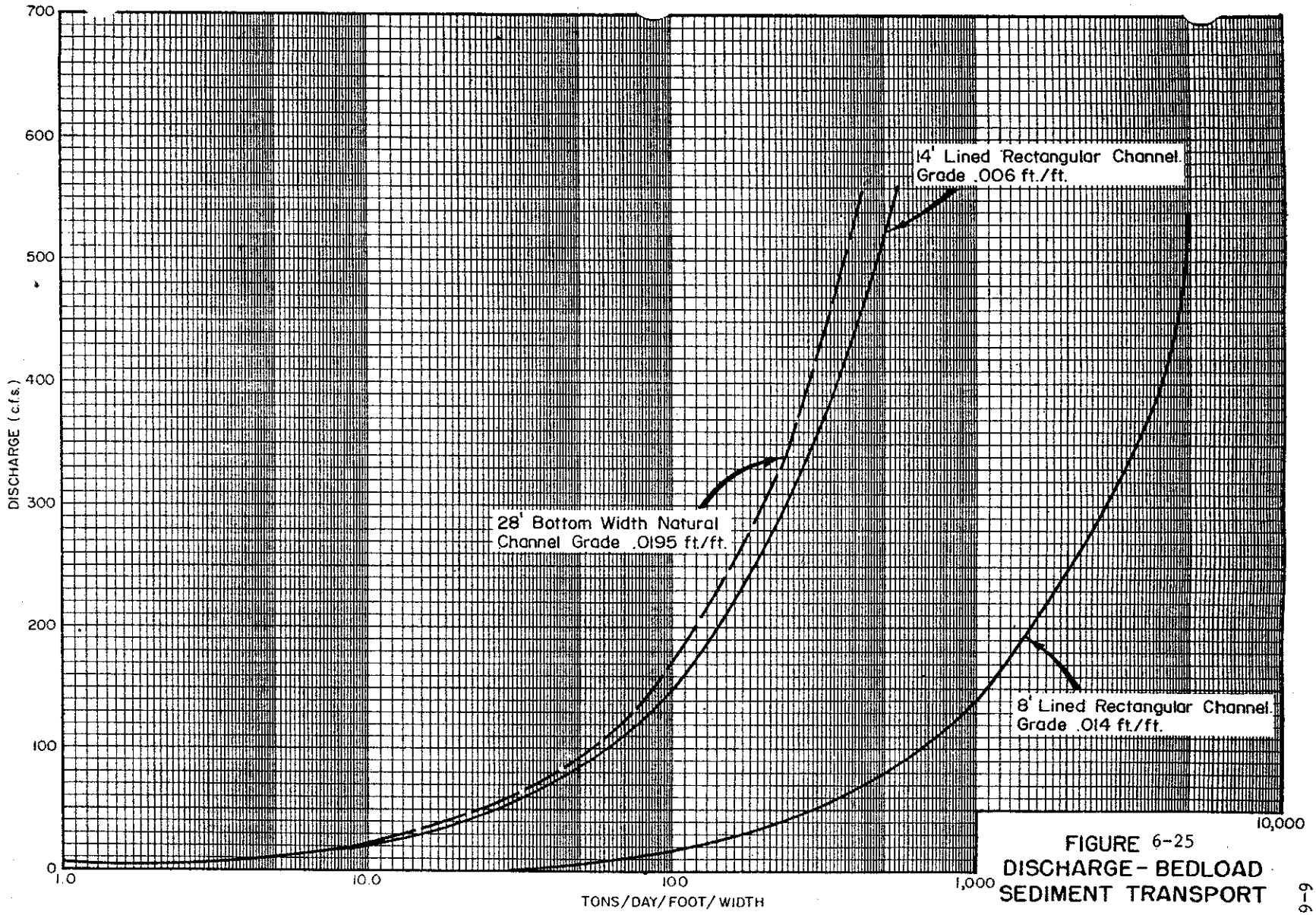


FIGURE 6-25
DISCHARGE - BEDLOAD
SEDIMENT TRANSPORT

247-927 O - 78 - 14

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).

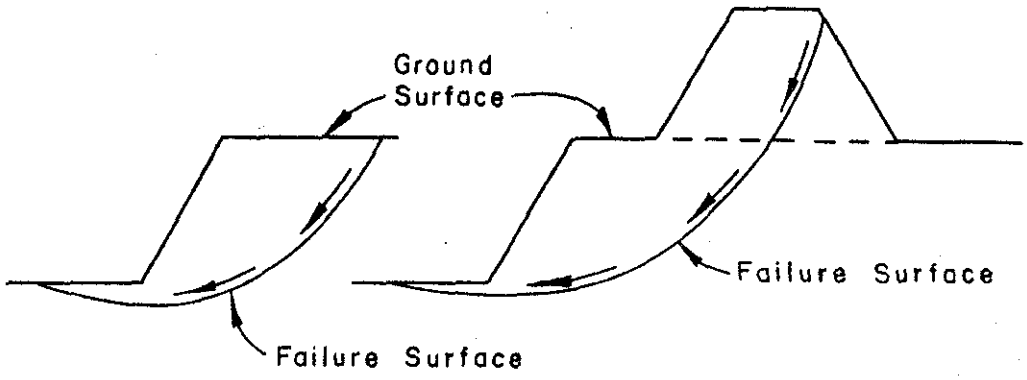
In these cases, the resistance to sliding is reduced, and the possibility of slope failure is increased, even though the original slope was stable before erosion occurred. These factors should be carefully considered in evaluating past performance of channels.

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.



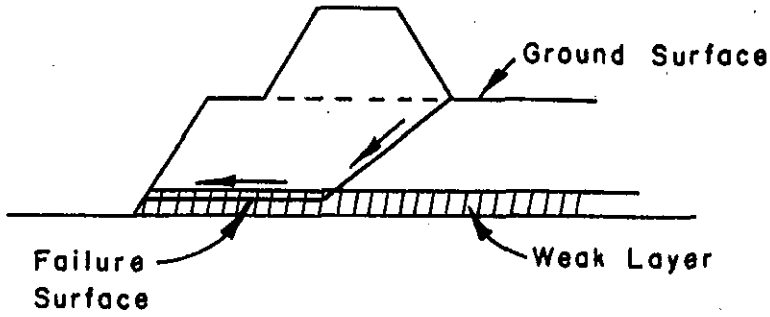
This type of failure generally occurs in plastic soils.

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 $\emptyset = 0$ and $\emptyset \neq 0$. 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:

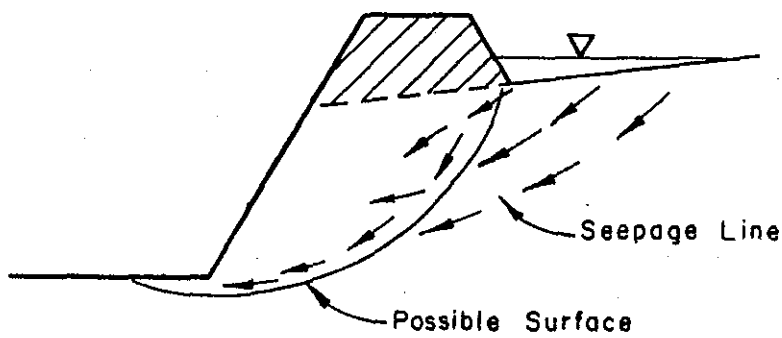
> "Earthquake History of the United States," U. S. Department of Commerce, Coast and Geodetic Survey Bulletin No. 41-1.
>
> "Seismic Stability of Earth Dams" by E. E. Esmiol, U.S.B.R. Technical Memorandum No. 641.

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

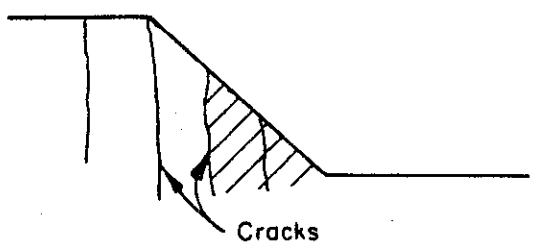
Experience has shown that the upper portion of most cohesive slopes is initially in a state of tension. The depth to which tension extends can be roughly estimated by the following equation:

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

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 = $\frac{V}{\sqrt{gd_m}}$

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 $= \dfrac{11.6\nu}{\sqrt{gR_t s_e}}$

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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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 ÷ 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.

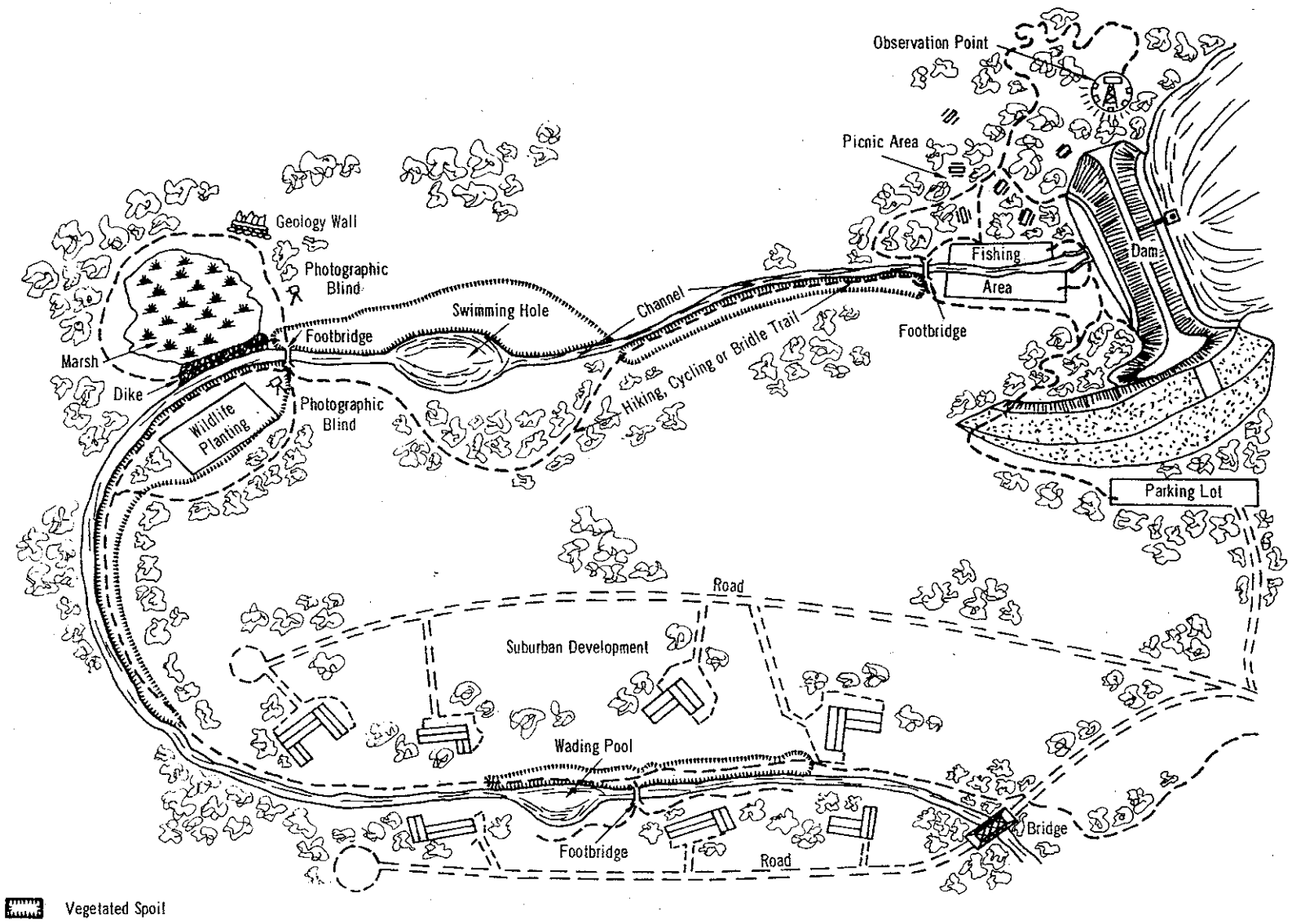


Figure 1 CHANNEL RECREATION POSSIBILITIES

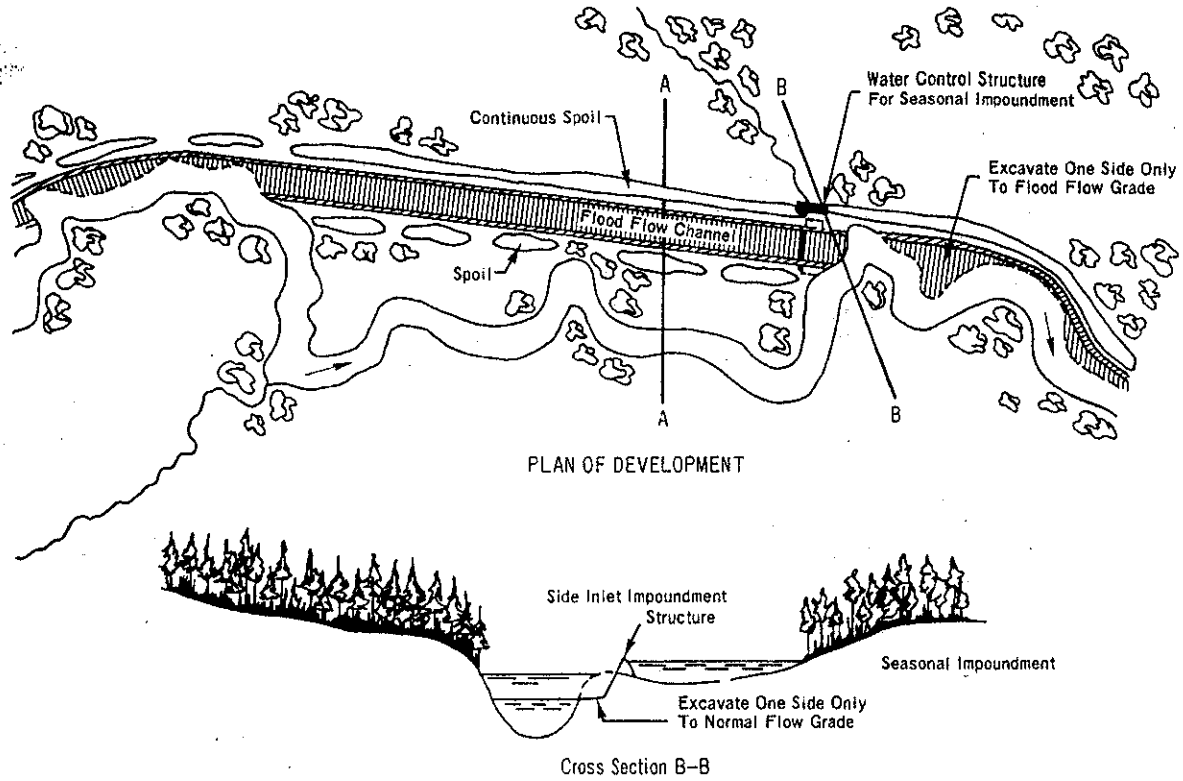
Sediment loads delivered to the channel by tributary streams or other sources need to be controlled as part of the overall design. A stable channel has a limited sediment carrying capacity and downstream uses of the water may necessitate further limitation of sediment load. Sediment traps may be needed 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.

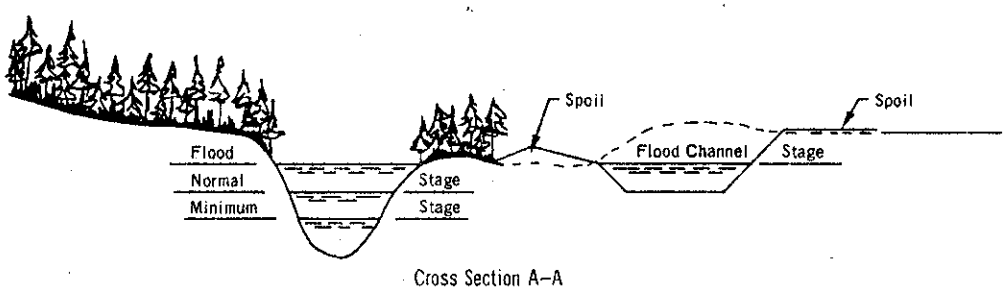
2. Channel Section - The low flow channel cross section should approach a natural stream condition. (Figure 2) The side slopes and bottom width can be allowed to vary to take advantage of existing conditions. Where possible, the side slope on the outside of the channel curve could be steepened and the 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.



ONE-SIDED CONSTRUCTION FOR ADJACENT STREAM CHANNEL & FLOOD FLOW CHANNEL



UNDISTURBED STREAM CHANNEL & SEPARATE FLOOD FLOW CHANNEL

Figure 2 SEPARATE LOW FLOW AND FLOOD FLOW CHANNELS

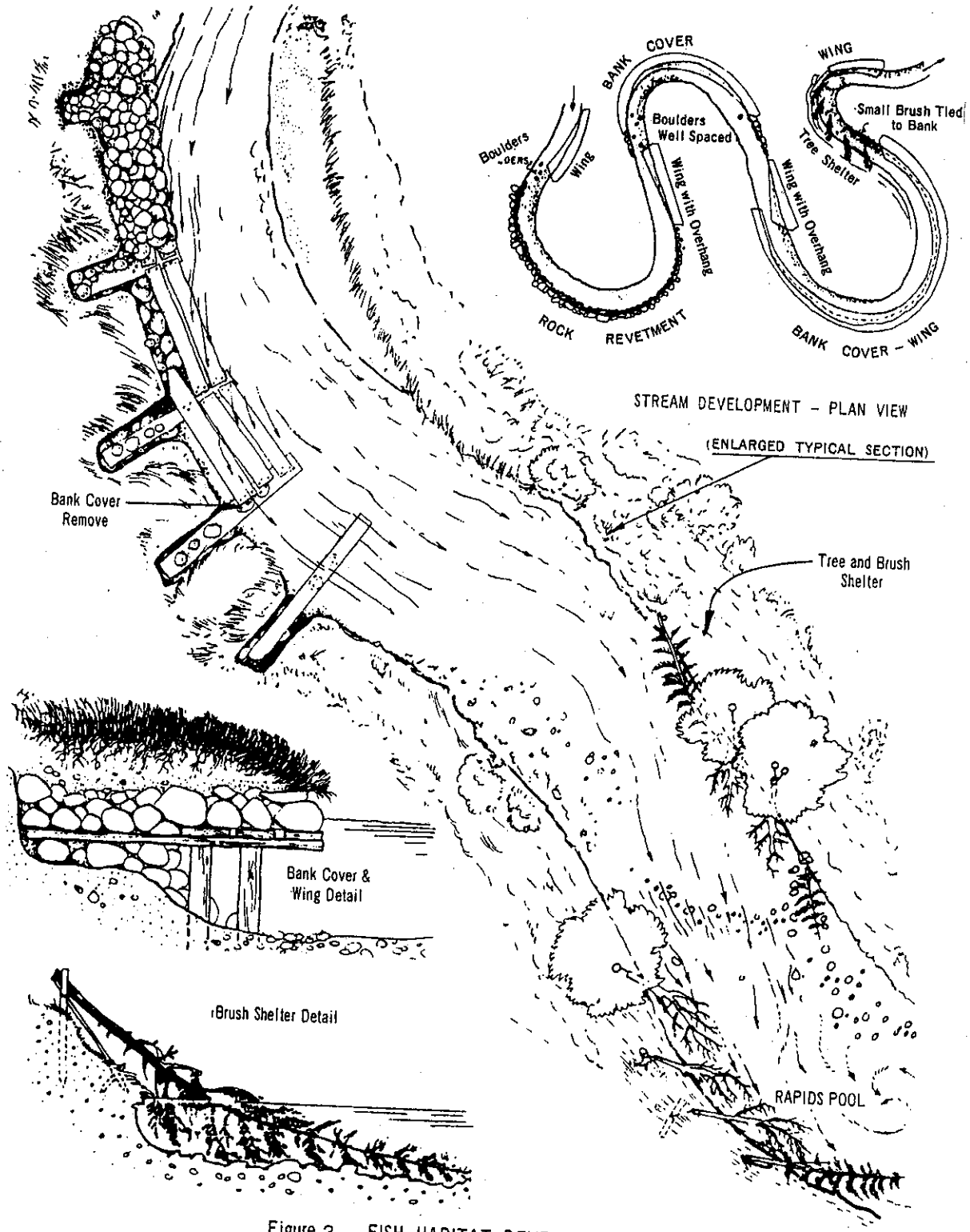
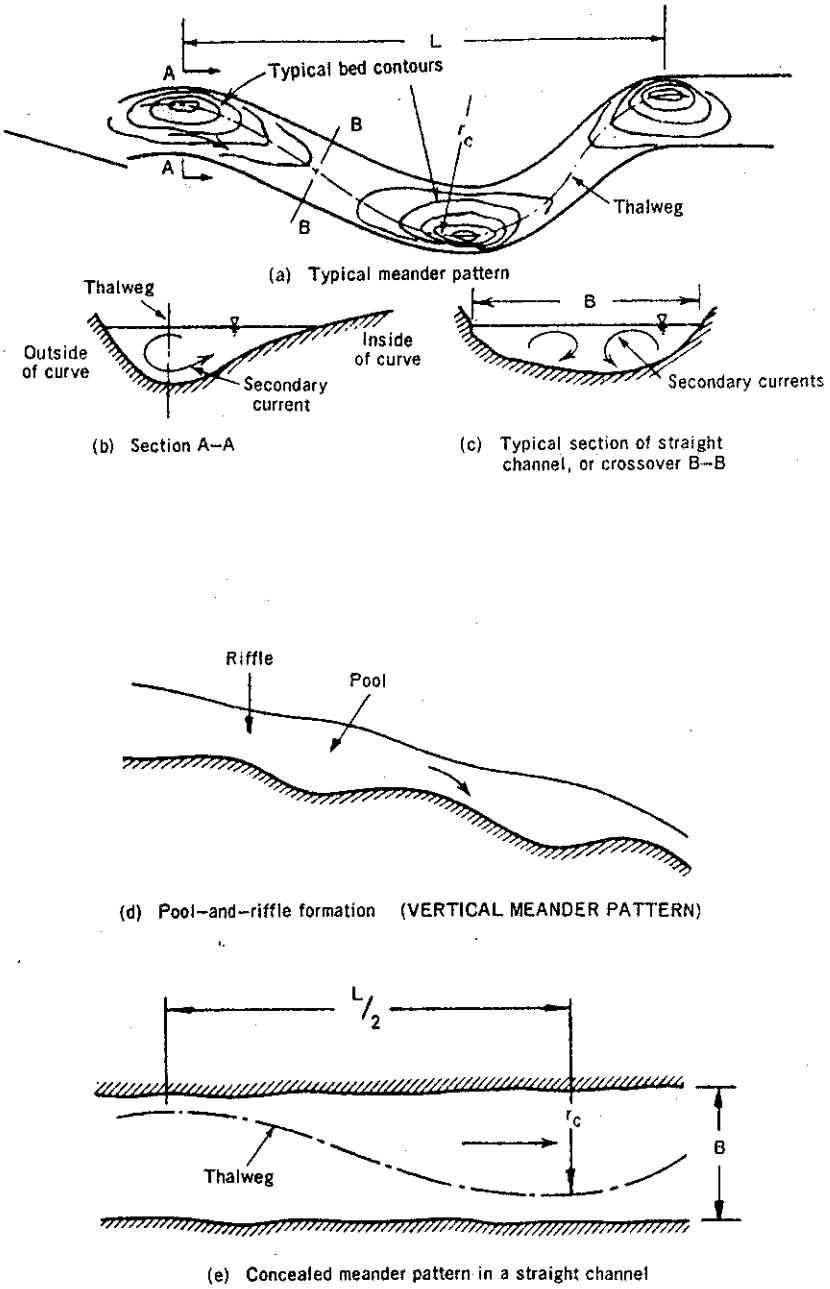


Figure 3 FISH HABITAT DEVELOPMENT



APPROXIMATE MEANDER PARAMETERS

r_c = radius of curvature in Thalweg

$\frac{L}{B}$ = 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

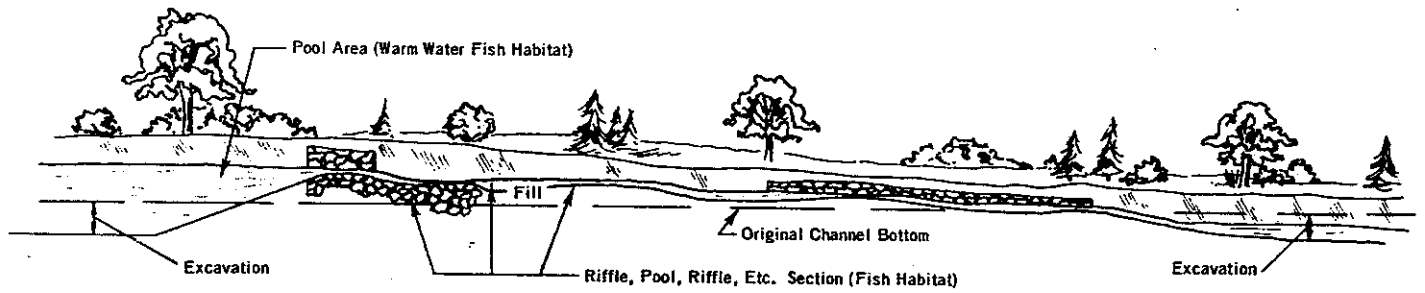
4. Structural Measures - Channels with flat enough gradients so that stability is not a design problem could, if desirable, be laid out in pond-riffle-pool manner. The additional grade provided by ponding would allow the inclusion of a series of riffles and pools that could add to the fish habitat. See Figure 6.

5. Vegetation

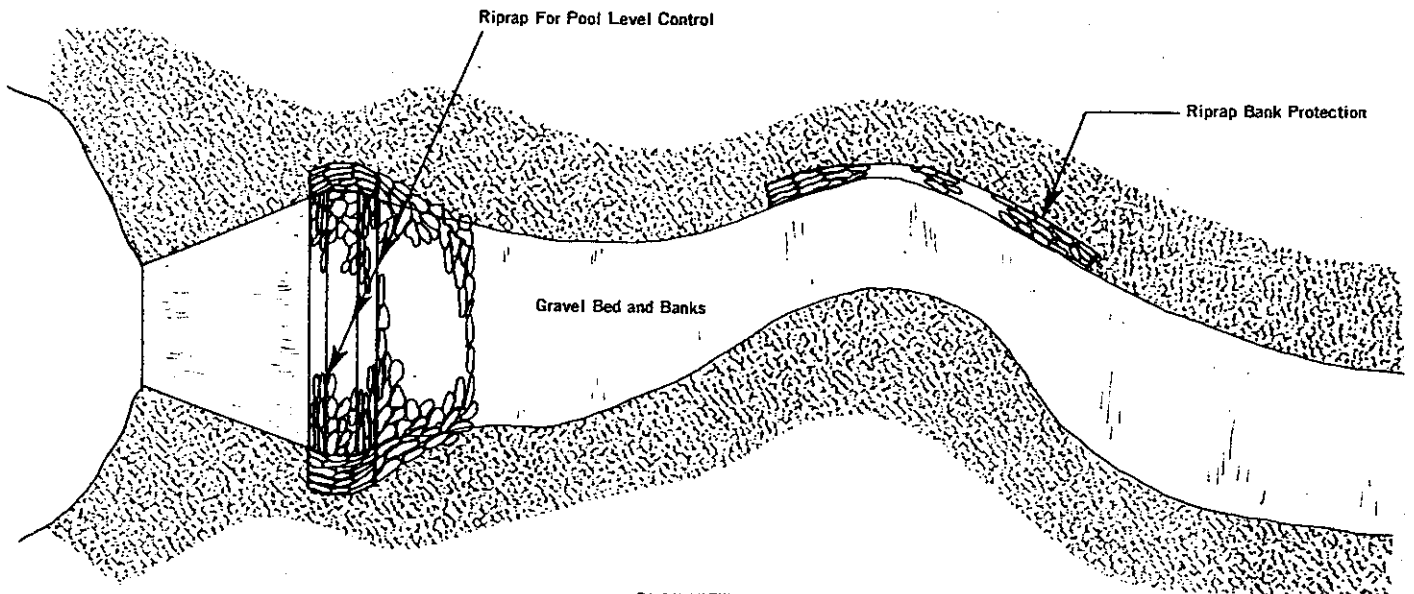
   a. Wildlife habitat seedings and plantings can be used outside the channel section, on the spoil sections or in odd corners of suitable size where the wildlife potential can be enhanced and the disturbed areas stabilized. Wildlife habitat mitigation areas, when included, and the normal habitat seedings 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	Multi-plier 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 near-by, visual appearance of channel)	4*	Excellent	Good	Fair	Poor
Water Flow	4*	Little variation with riffles and pools	Little variation	Inter-mittent	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 form-ing 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.

247-927 O - 78 - 18

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 or 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. The water source - 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. Dominant vegetation - 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. Turbidity - Express as clear if bottom is distinctly seen through 4 or more feet of water, slightly turbid if bottom can be seen at from 1- to 4-foot depth, and turbid 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. Sediment - 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

$$\frac{\text{Total Score}}{40}$$

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 tributaries:_____ _____ _____ _____ _____

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 0 = 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,

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.

USDA-SCS-HYATTSVILLE, MD. 1971

U. S. GOVERNMENT PRINTING OFFICE : 1977 O - 247-927

AGENDA
ENGINEERING WORKSHOP
CHANNEL STABILIZATION AND BANK PROTECTION
February 5-7, 1990 - SCS Conference Room
Phoenix, Arizona

Monday, February 5 - Chairperson, Stephanie Yard

CHANNEL STABILIZATION

9:00 - 9:10	Welcome and Introductions	Ralph Arrington
9:10 - 9:15	Remarks	Don Phillips
9:15 - 10:00	Watershed Geomorphology	Aubrey Sanders
10:00 - 11:30	Tractive Stress Method	Lee Saele
11:30 - 12:30	Lunch	
12:30 - 2:00	Tractive Stress Method (Cont'd)	Lee Saele
2:00 - 2:15	Break	
2:15 - 4:00	Tractive Power Method	John Harrington

Tuesday, February 6 - Chairperson, Carmella Apodaca

CHANNEL STABILIZATION - CONT'D

7:00 - 8:00	Allowable Velocity Method	Tom Jayo
8:00 - 9:00	Modified Regime Method	Aubrey Sanders
9:00 - 9:15	Break	
9:15 - 11:30	Modified Regime Method (Cont'd)	Aubrey Sanders
11:30 - 12:30	Lunch	
12:30 - 1:30	Environmental Considerations Fish & Wildlife	Dave Seery
1:30 - 2:00	Vegetative Channels (Film)	Steve Jones
2:00 - 2:15	Break	
2:15 - 2:45	Discussion of vegetative Channels	Lee Saele & Steve Jones
2:45 - 4:00	Grade Control Structures	Lee Saele

Wednesday, February 7 - Chairperson, Jack Burgin

BANK PROTECTION

7:00 - 7:15	Policy & Job Approval	Ralph Arrington
7:15 - 7:30	Summary of 1979 & 1983 EWP Work	John Weaver
7:30 - 8:00	California Video	John Weaver
8:00 - 8:15	Arizona Video	John Weaver
8:15 - 8:45	Vegetative Bank Protection	Dave Seery
8:45 - 9:15	Rock Riprap	Stephanie Yard
9:15 - 9:30	Break	
9:30 - 10:00	Pipe & Fence	Jack Elder
10:00 - 10:30	Kellner Jacks	John Weaver
10:30 - 11:00	Spider Jacks	Steve Jones
11:00 - 11:30	Deflecting Jetties	John Weaver
11:30 - 12:30	Lunch	
12:30 - 1:00	Soil Cement	John Harrington
1:00 - 1:30	Gabions	Lee Hardy
1:30 - 2:00	CE Streambank Protection	John Weaver
2:00 - 2:15	Break	
2:15 - 2:45	Streambank Protection in WNTC	Lee Saele
2:45 - 3:00	Workshop Evaluation	Ralph Arrington
3:00	Adjourn	

ENGINEERING WORKSHOP
February 5-7, 1990

Participants

Carmella Apodaca, Civil Engineer, Tucson, Arizona

Ralph Arrington, State Conservation Engineer, Phoenix, Arizona

Jack Burgin, Civil Engineering Technician, Phoenix, Arizona

Blake Covey, Civil Engineering Technician, Mesa, Arizona

Jack Elder, Area Engineer, Tucson, Arizona

Leland Hardy, Water Management Engineer, Phoenix, Arizona

John Harrington, State Design Engineer, Phoenix, Arizona

Noller Herbert, Project Engineer, Mesa, Arizona

Tom Jayo, Civil Engineer, Phoenix, Arizona

Steve Jones, Area Engineer, Flagstaff, Arizona

Harry Millsaps, Hydraulic Engineer, Phoenix, Arizona

Don Paulus, Planning Engineer, Phoenix, Arizona

Albert Rutledge, Construction Inspector, Mesa, Arizona

Aubrey Sanders, Geologist, Phoenix, Arizona

Lee Saele, Design Engineer, WNTC, Portland, Oregon

John Sullivan, Construction Engineer, Phoenix, Arizona

John Weaver, Asst. State Conservation Engineer, Phoenix, Arizona

Stephanie Yard, Civil Engineer, Flagstaff, Arizona

Dave Seery, Biologist, Flagstaff, Arizona

Flood Control District (Nine Persons), Phoenix, Arizona

TRACTIVE FORCE

THE TRACTIVE FORCE METHOD OF STABILITY ANALYSIS WAS DEVELOPED FOR THE THRESHOLD CONDITIONS OF SEDIMENT TRANSPORT AND IS BASED ON THE CONCEPT THAT MOVEMENT OF THE MATERIALS IN THE BED AND BANKS OF A CHANNEL WILL NOT OCCUR IF CERTAIN CONDITIONS OF FLOW WITH RESPECT TO THE SOIL MATERIALS ARE MET.

I. RESTRICTIONS

1. The tractive force method is restricted to channels which do not aggrade. (do not carry heavy sediment.)

   Channels designed by the tractive force theory carry water which is free of bed load and carry suspended sediment that does not deposit as a coating on the banks or bed of the channel.

2. This method is also restricted to channels which have bed and banks made up of noncohesive, discrete soil particles.

II. DEFINITION OF TRACTIVE FORCE

When water flows in a channel, a drag force is developed that acts on the channel boundary in the direction of flow. This force, which is simply the tangential pull of water on the wetted boundary area, is known as the tractive force. The tractive force per unit boundary area is the tractive stress.

III. DERIVATION OF THE BASIC TRACTIVE STRESS FORMULA

straight, smooth channel, relatively wide (no side corners)
small slope (α), normal flow depth bed slope = WS slope

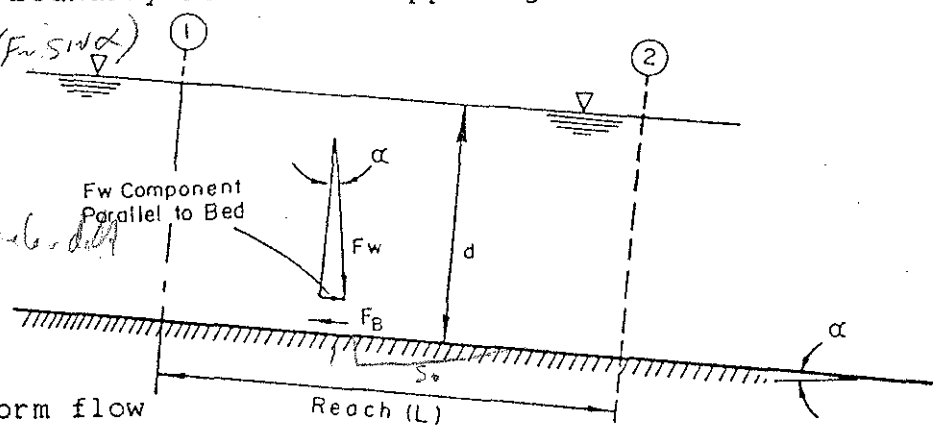
The idea is to relate the tractive stress excerted on the boundary to the boundary resistance opposing motion.

weight of water moving downstream ($F_w \sin\alpha$)
= frictional component

small α, $\sin = \tan$

$F_w \sin\alpha = (\text{Vol}, \rho_{water}) S_o$ S_o = bed slope

$= (A L)(\text{weight}) S_o$



Assume: a) uniform flow

analogy to wood block moving down plane - solve for friction friction tractive force

b) slope is small

Weight of water acting downslope

$$F_w \sin\alpha = (\text{vol.})(\delta_w)\, S_o$$

$$= A\, L\, \delta_w\, S_o \qquad (1)$$

where A = cross sectional area

Frictional resistance of bed (F_B) = τ_o × (boundary area)

$$\tau_o \times (\text{boundary area}) = \tau_o\, P\, L \qquad (2)$$

P = wetted perimeter

τ_o = tractive shear stress = force of water on bed

Equating the weight of water acting downslope to the frictional resistance of the bed, equation 1, to equation 2, results;

$$\tau_o\, P\, L = A\, L\, \delta_w\, S_o$$

$$\tau_o = \frac{A}{P}\, \delta_w\, S_o$$

$$= \delta_w\, R\, S_o \qquad (3)$$

where R equals the hydraulic radius

<u>τ_o is referred to as the average tractive stress</u>

Stable Channels Workshop, Az., Feb. 1990

EVALUATION OF N

A. Single Grain Roughness

1. Cowan's Empirical Method (average "n")

NEH-5, Hydraulics - Supplement B Nat'l Engineering Handbook

EDS-FWS pp 6-2, 6-3

Guide for Selecting Roughness "n", good, pictures
Guy Fasken, SCS, Lincoln, Neb. 196X

Roughness Characteristics of Natural Channels,
USGS W-SP 1849 Available

Open Channel Hydraulics - Chow pp 110-125 best for grain

2. Stickler's Formula - basic "n" value as a function of boundary particle size; No adjustment for hydraulic radius.

$$n = \frac{D_{50}^{1/6}}{29.3}$$ D_{50} in feet - Chow pp 205,206

$$n_t = \frac{D_{75}^{1/6}}{39}$$ $D_{\cancel{50}\,75}$ in inches - TR-25, p 6-12

B. Composite Roughness for varying n values (x-section)

$$n = \left(\frac{\sum_{i=1}^{m} P_i n_i^{3/2}}{P} \right)^{2/3}$$

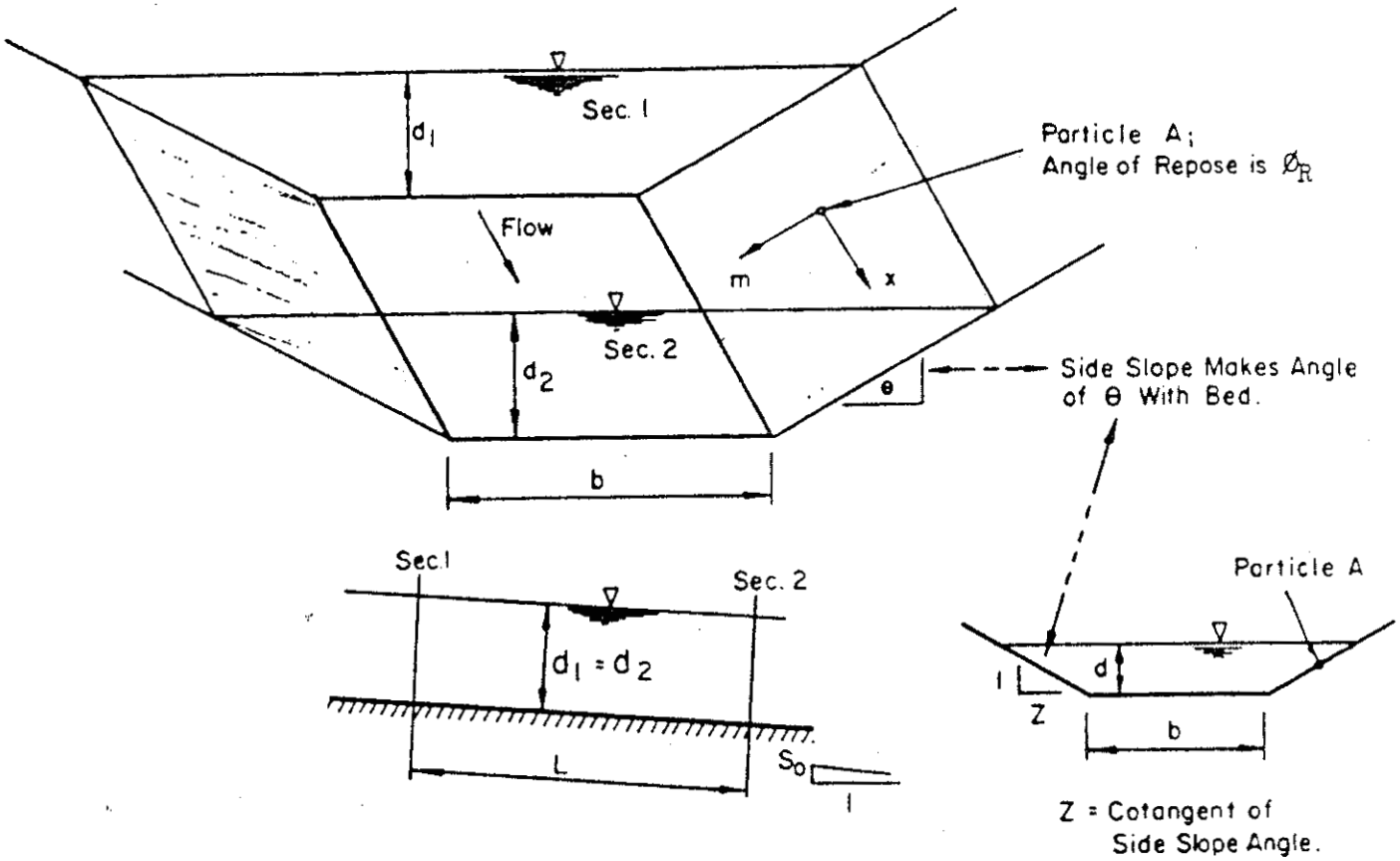
EDS -FWS p 6-3
TR-25 p 6-47
Chow p 136

$$n = \left(\frac{\sum_{i=1}^{m} P_i n_i^{2}}{P} \right)^{1/2}$$

Chow p 136

NOTE: The use of composite roughness is implicit in the derivation of equation 6-3, TR-25.

TRACTIVE FORCE RATIO FOR COARSE, NON-COHESIVE SOIL MATERIAL



Forces on particle A

1. Force of water tending to move particle downstream in the direction of flow (X direction).

2. Force of gravity tending to move particle down the sloping side of the bank (m direction).

$$K = \frac{\tau_{L_s}}{\tau_{L_b}} = \cos\theta \sqrt{1 - \frac{\tan^2\theta}{\tan^2\phi_R}}$$

Formula originally given by USBR. Refer to references 12 and 4.

where τ_{L_s} = critical (allowable) τ on the bank

and τ_{L_b} = critical (allowable) τ on a level bed.

$$\kappa = \sqrt{\cos^2\theta - \frac{\sin^2\theta}{\tan \phi_R}} = \sqrt{1 - \sin\theta \left(1 + \frac{1}{\tan^2\phi_R}\right)}$$

$$\kappa = \sqrt{1 - \sin^2\theta \left(\frac{\sin^2\phi_R + \cos^2\phi_R}{\sin^2\phi_R}\right)} = \sqrt{1 - \frac{\sin^2\theta}{\sin^2\phi_R}}$$

$$\kappa = \sqrt{\frac{z^2 - \cot^2\phi_R}{1 + z^2}}$$

Formula of SCS on page 6-~~15~~18 of TR-25[8]

TR-25

Coarse-Grained Discrete $(0.25'' \text{ mm} \leq D_{75} \leq 5.0'' \text{ mm})$

Maximum Tractive Stress

w = weight of water, d = depth of flow, S_e = energy slope

1. $\tau_\infty = \omega d S_t$, Reference Tractive Stress where

hydraulically wide channel

$$S_t = \left(\frac{n_t}{n}\right)^2 S_e \text{ and } n_t = \frac{(D_{75})^{1/6}}{39} \text{ with } D_{75} \text{ expressed in inches}$$

2. $\tau_{bed} = \frac{\tau_b}{\tau_\infty} \times \tau_\infty$, Fig. 6-3, TR-25

   $\tau_s = \frac{\tau_s}{\tau_\infty} \times \tau_\infty$, Fig. 6-4, TR-25

   Accounts for various b/d and z values

3. Correction Factors for Curvature

   Within curved reach,

   $\tau_{bc} = \frac{\tau_{bc}}{\tau_b} \times \tau_b$, Fig. 6-5, TR-25 (bed curve)

   $\tau_{sc} = \frac{\tau_{sc}}{\tau_s} \times \tau_s$, Fig. 6-5, TR-25 (side slope curve)

   Downstream of Curved Reach (Fig. 6-6, TR-25)

   $\tau_{bt} = \frac{\tau_{bt} - \tau_b}{\tau_{bc} - \tau_b} \times (\tau_{bc} - \tau_b) + \tau_b$ (bed transition)

   $\tau_{st} = \frac{\tau_{st} - \tau_s}{\tau_{sc} - \tau_s} \times (\tau_{sc} - \tau_s) + \tau_s$ (slope transition)

Allowable Tractive Stress

always larger than actual for stability

1. $\tau_{Lb} = 0.4\, D_{75}$, with D_{75} expressed in inches

   slope of τ_c vs diameter particle

2. $\tau_{Ls} = K\ \tau_{Lb} = K(0.4\, D_{75})$

   K = f (side slope)

   Where $K = f(\phi_R, z)$, Fig. 6-8, TR-25

   and $\phi_R = f(D_{75}, \text{angularity})$, Fig. 6-7, TR-25

   compare to shear ($\tau_b, \tau_s, \tau_{bc}, \tau_{sc}, \tau_{bt}, \tau_{st}$)

Check Stability

$$\tau_b, \tau_s, \tau_{bc}, \tau_{sc}, \tau_{bt}, \tau_{st} < \tau_{Allowable}$$

CHANNEL DESIGN WORKSHOP
AZ. 2/90 L.M. SAELE

Tractive Force (Analysis Problem) 1 8

Stable Earth Channels
Problem 1

Given: A channel is to be constructed through an area of intense cultivation.

Q_{design} = 262 cfs (bankfull)

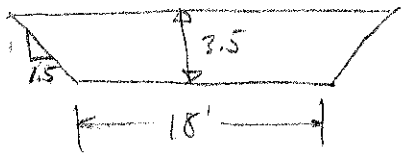
$p = 262/3.23 =$

Q_{10} = 100 cfs or 50% of design Q (SDS)

$5n$

$3.5 \overline{)18}$
1.5

b = 18 ft.



d = 3.5 ft.

z = 1½ : 1

V = 3.23 fps

S = 0.0026

n (aged) = 0.045

The bed and bank material is GM that is nonplastic, with $\boxed{D_{75} = 0.90 \text{ inches}}$. The gravel is very angular

$s = r\theta$
$\theta = s/r \times 180/\pi$
$= \frac{72}{150} \times \frac{180}{\pi} \approx 30°$

There is one curve in the reach with a radius of 150 ft and a length of 72 ft.

Required: Check the stability of the reach for the given design discharge using tractive stress procedures.

$30/18 = \approx 1.8$

$\frac{t_b =}{\tau_s}$ = 0.975 = t_{bed}
0.76 = τ_{side}

$\tau_{bc}/\tau_b = 2.0 = \tau_{sc}/\tau_s$

$L_c/b = 72/18 \approx 4 \Rightarrow \frac{\tau_{bt} - \tau_s}{bc - b} = 0.85$; $\frac{\tau_{st} - \tau_s}{sc - s} = 0.85$

$\tau_o = \gamma_w d S_f = 62.4 \times 3.5 \times 0.0026 = 0.75$?

$\tau_b = 0.74$; $\tau_{bc} = 1.5$;
$\tau_s = 0.6$; $\tau_{sc} = 1.2$;

Stable Earth Channels
Problem 1
Solution (Aged)

Since $D_{75} > 1/4$ inch, use the Lane Method

1. Find the Actual tractive stress in straight reach

$$n_t = \frac{(D_{75})^{1/6}}{39} = \frac{(0.90)^{1/6}}{39} = 0.0252 \quad \text{(from Eq. 6-2, TR-25)}$$

$$S_t = \left(\frac{n_t}{n}\right)^2 S_e \quad \text{(from Eq. 6-3, TR-25)}$$

$$= (0.0252/.045)^2 \times .0026 = 0.00082$$

$$\tau_\infty = \gamma_w d S_t \quad \text{(from Eq. 6-4, TR-25)}$$

$$= (62.4)(3.5)(0.00082) = 0.179 \text{ psf}$$

$$b/d = 18/3.5 = 5.14$$

$$\tau_b/\tau_\infty = 0.98 \quad \text{(from Fig. 6-3, TR-25)}$$

$$\tau_s/\tau_\infty = 0.76 \quad \text{(from Fig 6-4, TR-25)}$$

$$\text{Actual } \tau_b = \tau_\infty (\tau_b/\tau_\infty)$$

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

$$\text{Actual } \tau_s = \tau_\infty (\tau_s/\tau_\infty)$$

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

2. Find the Actual tractive stress in the curve

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

$$\tau_{bc}/\tau_b = \tau_{sc}/\tau_s = 1.17 \quad \text{(from Fig. 6-5, TR-25)}$$

Stable Earth Channels
Problem 1
Solution (Aged)

Actual $\tau_{bc} = \tau_b(\tau_{bc}/\tau_b)$

$= (0.175)(1.17) = 0.208$ psf

Actual $\tau_{sc} = \tau_s(\tau_{sc}/\tau_s)$

$= (\quad)(\quad) = 0.16$ psf

3. Find the Actual tractive stress in straight reach immediately downstream from curve

L_c/b (length of curve / bottom width) = 72/18 = 4.0

$$\frac{\tau_{bt} - \tau_b}{\tau_{bc} - \tau_b} = \frac{\tau_{st} - \tau_s}{\tau_{sc} - \tau_s} = 0.86$$ (from Fig 6-6, TR-25)

$$\text{Actual } \tau_{bt} = \left(\frac{\tau_{bt} - \tau_b}{\tau_{bc} - \tau_b}\right)(\tau_{bc} - \tau_b) + \tau_b$$

$$= (0.81)(0.205 - 0.175) + 0.175 = 0.201 \text{ psf}$$

$$\text{Actual } \tau_{st} = \left(\frac{\tau_{st} - \tau_s}{\tau_{sc} - \tau_s}\right)(\tau_{sc} - \tau_s) + \tau_s$$

$$= (0.86)(0.159 - 0.136) + 0.136 = 0.156 \text{ psf}$$

4. Find the Allowable tractive stress

$\phi_R = 38.4°$ (from Fig. 6-7, TR-25)

$K = 0.45$ (from Fig 6-8 or Eq. 6-7 TR-25)

Tractive Force (Analysis Problem) 4 8

$\tau_{Lb} = 0.4\, D_{75}\, T$ $T = 1.0$ * Stable Earth Channels Problem 1 Solution (Aged)

$\tau_{Ls} = 0.4\, D_{75}\, K\, T$

(critical) Allowable $\tau_{Lb} = (0.4)(D_{75})$ (from Eq. 6-5, TR-25)

$= (0.4)(0.90) = 0.360$ psf

Allowable $\tau_{Ls} = (0.4)(K)(D_{75})$ (from Eq. 6-6, TR-25)

$= (0.4)(0.45)(0.90) = 0.162$ psf

5. Check stability

$\tau_b, \tau_{bc}, \tau_{bt} < \tau_{Lb}$ ∴ bed is stable

$\tau_s, \tau_{sc}, \tau_{st} < \tau_{Ls}$ ∴ sides are stable

* Equat. 6-8, TR-25
Assume unit weight of material > D_{75} = 160 #/ft³
∴ T = 1.0

(2.65 − 1.00 / 1.65)
($\gamma_s - \gamma_w$)

Tractive Force (Analysis Problem) 5 8

Not fully aged channel (As Built)

Stable Earth Channels
Problem 1
Solution (As-built)

1. Find the Actual tractive stress in straight reach

use $Q = Q_{design}/2 = 131$ cfs (Ref. NEH 2 EPS 5E2)

$n_t = \frac{D_{75}^{1/6}}{39} =$ (from Eq. 6-2; TR-25)

only difference in this problem

basic n = n_t = (Ref. pg 6-13, TR-25)
irreg .005 (from NEH 5, Appendix B)

$n = 0.030$

determine new flow depth

$Q = 131$ cfs
$n =$
$s = .0026$
$b = 18$ ft
$z = 1\ 1/2$

$Q = 1.49\, AR^{2/3} s^{1/2}/n$

required $AR^{2/3} = Qn/1.49 s^{1/2}$

$= 131(\quad)/1.49\,(.0026)^{1/2} = 51.73$

$AR^{2/3} = (bd + d^2 z)\left[\frac{bd + d^2 z}{b + 2d\sqrt{z^2+1}}\right]^{2/3}$

$51.73 = (18d + 1.5d^2)\left[\frac{18d + 1.5d^2}{18 + 3.61d}\right]^{2/3}$

$d = 1.827 \Rightarrow AR^{2/3} = 51.71$ OK

$s_t = \left(\frac{n_t}{n}\right)^2 s_e = (\quad/\quad)^2(.0026) =$

Tractive Force (Analysis Problem) 6 8

Stable Earth Channels
Problem 1
Solution (As-built)

$\tau_\infty = \gamma_w d S_t$ (from Eq. 6-4, TR-25)

$= (62.4)(1.83)(\quad) =$ psf

$b/d = 18/1.83 = 9.84$

$\tau_b/\tau_\infty =$ (from Fig. 6-3, TR-25)

$\tau_s/\tau_\infty =$ (from Fig. 6-4, TR-25)

Actual $\tau_b = \tau_\infty(\tau_b/\tau_\infty)$

$= \quad (.99) =$ _____ psf

Actual $\tau_s = \tau_\infty(\tau_s/\tau_\infty)$

$= \quad (.76) =$ _____ psf

2. Find the Actual tractive stress in the curve

$R_c/b = 150/18 = 8.33$

$\tau_{bc}/\tau_b = \tau_{sc}/\tau_s =$ (from Fig. 6-5, TR-25)

Actual $\tau_{bc} = \tau_b(\tau_{bc}/\tau_b)$

$= 0.207(\quad) =$ _____ psf

Actual $\tau_{sc} = \tau_s(\tau_{sc}/\tau_s)$

$= .159(\quad) =$ _____ psf

Stable Earth Channels
Problem 1
Solution (As-built)

3. Find the Actual tractive stress in straight reach immediately downstream from curve.

$L_c/b = 72/18 = 4.0$

$$\frac{\tau_{bt} - \tau_b}{\tau_{bc} - \tau_b} = \frac{\tau_{st} - \tau_s}{\tau_{sc} - \tau_s} =$$ (from Fig 6-6, TR-25)

$$\text{Actual } \tau_{bt} = \left(\frac{\tau_{bt} - \tau_b}{\tau_{bc} - \tau_b}\right)(\tau_{bc} - \tau_b) + \tau_b$$

$$= (\quad)(0.242 - 0.207) + 0.207 = \underline{\qquad\qquad} \text{ psf}$$

$$\text{Actual } \tau_{st} = \left(\frac{\tau_{st} - \tau_s}{\tau_{sc} - \tau_s}\right)(\tau_{sc} - \tau_s) + \tau_s$$

$$= (\quad)(0.186 - 0.159) + 0.159 = \underline{\qquad\qquad} \text{ psf}$$

4. Find the Allowable tractive stress

$\phi_R = 38.4°$

$K = 0.45$

$$\text{Allowable } \tau_{Lb} = 0.4\, D_{75}$$

$$= 0.4(0.90) = 0.360 \text{ psf}$$

$$\text{Allowable } \tau_{Ls} = 0.4\, K D_{75}$$

$$= 0.4(0.45)(0.90) = 0.162 \text{ psf}$$

5. Check stability

τ_b, τ_{bc}, τ_{bt} $< \tau_{Lb}$ $\therefore$ bed is stable

$\tau_s < \tau_{Ls}$ but τ_{sc}, $\tau_{st} > \tau_{Ls}$ $\therefore$ sides in curve and just downstream of curve are not stable.

worked out problem

Tractive Force (Analysis Problem) 1 8

Stable Earth Channels
Problem 1

Given: A channel is to be constructed through an area of intense cultivation.

Q_{design} = 242 cfs (bankfull)

Q_{10} = 100 cfs

b = 18 ft.

d = 3.5 ft.

z = 1 1/2 : 1

V = 3.23 fps

S = 0.0026

n (aged) = 0.045

The bed and bank material is GM that is nonplastic, with D_{75} = 0.90 inches. The gravel is very angular

There is one curve in the reach with a radius of 150 ft and a length of 72 ft.

Required: Check the stability of the reach for the given design discharge using tractive stress procedures.

Stable Earth Channels
Problem 1
Solution (Aged)

Since $D_{75} > 1/4$ inch, use the Lane Method

1. Find the Actual tractive stress in straight reach

$n_t = \frac{(D_{75})^{1/6}}{39} = \frac{(0.90)^{1/6}}{39} = .0252$ (from Eq. 6-2, TR-25)

$S_t = \left(\frac{n_t}{n}\right)^2 S_e$ (from Eq. 6-3, TR-25)

$= (.0252/.045)^2 \times .0026 = .00082$

$\tau_\infty = \gamma_w d S_t$ (from Eq. 6-4, TR-25)

$= (62.4)(3.5)(.00082) = 0.179$ psf

$b/d = 18/3.5 = 5.14$

$\tau_b/\tau_\infty = 0.98$ (from Fig. 6-3, TR-25)

$\tau_s/\tau_\infty = 0.76$ (from Fig 6-4, TR-25)

Actual $\tau_b = \tau_\infty (\tau_b/\tau_\infty)$

$= (0.179)(0.98) = 0.175$ psf

Actual $\tau_s = \tau_\infty (\tau_s/\tau_\infty)$

$= (0.179)(0.76) = 0.136$ psf

2. Find the Actual tractive stress in the curve

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

$\tau_{bc}/\tau_b = \tau_{sc}/\tau_s = 1.17$ (from Fig. 6-5, TR-25)

$$\text{Actual } \tau_{bc} = \tau_b(\tau_{bc}/\tau_b)$$

$$= (0.175)(1.17) = 0.205 \text{ psf}$$

$$\text{Actual } \tau_{sc} = \tau_s(\tau_{sc}/\tau_s)$$

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

3. Find the Actual tractive stress in straight reach immediately downstream from curve

L_c/b (length of curve / bottom width) = 72/18 = 4.0

$$\frac{\tau_{bt} - \tau_b}{\tau_{bc} - \tau_b} = \frac{\tau_{st} - \tau_s}{\tau_{sc} - \tau_s} = 0.86$$ (from Fig 6-6, TR-25)

$$\text{Actual } \tau_{bt} = \left(\frac{\tau_{bt} - \tau_b}{\tau_{bc} - \tau_b}\right)(\tau_{bc} - \tau_b) + \tau_b$$

$$= (0.86)(0.205 - 0.175) + 0.175 = 0.201 \text{ psf}$$

$$\text{Actual } \tau_{st} = \left(\frac{\tau_{st} - \tau_s}{\tau_{sc} - \tau_s}\right)(\tau_{sc} - \tau_s) + \tau_s$$

$$= (0.86)(0.159 - 0.136) + 0.136 = 0.156 \text{ psf}$$

4. Find the Allowable tractive stress

$\phi_R = 38.4°$ (from Fig. 6-7, TR-25)

$K = 0.45$ (from Fig 6-8 or Eq. 6-7 TR-25)

Tractive Force (Analysis Problem) 4 8

$\tau_{Lb} = 0.4\ D_{75}\ T$ $T = 1.0$ Stable Earth Channels Problem 1 Solution (Aged)

$\tau_{Ls} = 0.4\ D_{75}\ K\ T$

Allowable $\tau_{Lb} = (0.4)(D_{75})$ (from Eq. 6-5, TR-25)

$= (0.4)(0.90) = 0.360$ psf

Allowable $\tau_{Ls} = (0.4)(K)(D_{75})$ (from Eq. 6-6, TR-25)

$= (0.4)(0.45)(0.90) = 0.162$ psf

5. Check stability

$\tau_b, \tau_{bc}, \tau_{bt} < \tau_{Lb}$ ∴ bed is stable

$\tau_s, \tau_{sc}, \tau_{st} < \tau_{Ls}$ ∴ sides are stable

Stable Earth Channels
Problem 1
Solution (As-built)

1. Find the Actual tractive stress in straight reach

use $Q = Q_{design}/2 = 131$ cfs (Ref. NEH 2 EPS 552)

$n_t = \frac{D_{75}^{1/6}}{39} = .0252$ (from Eq. 6-2, TR-25)

basic $n = n_t = 0.025$ (Ref. pg 6-13, TR-25)
irreg $\underline{.005}$ (from NEH 5, Appendix B)

$n = 0.030$

determine new flow depth

$Q = 131$ cfs
$n = .03$
$s = .0026$
$b = 18$ ft
$z = 1\ 1/2$

$Q = 1.49\, AR^{2/3} s^{1/2}/n$

required $AR^{2/3} = Qn/1.49 s^{1/2}$

$= 131(.03)/1.49(.0026)^{1/2} = 51.73$

$AR^{2/3} = (bd + d^2 z)\left[\frac{bd + d^2 z}{b + 2d\sqrt{z^2+1}}\right]^{2/3}$

$51.73 = (18d + 1.5d^2)\left[\frac{18d + 1.5d^2}{18 + 3.61d}\right]^{2/3}$

$d = 1.827 \Rightarrow AR^{2/3} = 51.71$ OK

$s_t = \left(\frac{n_t}{n}\right)^2 s_e = (.0252/.03)^2(.0026) = .00185$

Stable Earth Channels
Problem 1
Solution (As-built)

$\tau_\infty = \gamma_w d S_t$ (from Eq. 6-4, TR-25)

$= (62.4)(1.83)(.00185) = 0.209$ psf

$b/d = 18/1.83 = 9.84$

$\tau_b/\tau_\infty = 0.99$ (from Fig. 6-3, TR-25)

$\tau_s/\tau_\infty = 0.76$ (from Fig. 6-4, TR-25)

Actual $\tau_b = \tau_\infty(\tau_b/\tau_\infty)$

$= 0.209(.99) = \underline{\underline{0.207 \text{ psf}}}$

Actual $\tau_s = \tau_\infty(\tau_s/\tau_\infty)$

$= 0.209(.76) = \underline{\underline{0.159 \text{ psf}}}$

2. Find the Actual tractive stress in the curve

$R_c/b = 150/18 = 8.33$

$\tau_{bc}/\tau_b = \tau_{sc}/\tau_s = 1.17$ (from Fig. 6-5, TR-25)

Actual $\tau_{bc} = \tau_b(\tau_{bc}/\tau_b)$

$= 0.207(1.17) = \underline{\underline{0.242 \text{ psf}}}$

Actual $\tau_{sc} = \tau_s(\tau_{sc}/\tau_s)$

$= .159(1.17) = \underline{\underline{0.186 \text{ psf}}}$

Stable Earth Channels
Problem 1
Solution (As-built)

3. Find the Actual tractive stress in straight reach immediately downstream from curve.

$L_c/b = 72/18 = 4.0$

$$\frac{\tau_{bt} - \tau_b}{\tau_{bc} - \tau_b} = \frac{\tau_{st} - \tau_s}{\tau_{sc} - \tau_s} = 0.86$$ (from Fig 6-6, TR-25)

$$\text{Actual } \tau_{bt} = \left(\frac{\tau_{bt} - \tau_b}{\tau_{bc} - \tau_b}\right)(\tau_{bc} - \tau_b) + \tau_b$$

$$= (0.86)(0.242 - 0.207) + 0.207 = \underline{0.237 \text{ psf}}$$

$$\text{Actual } \tau_{st} = \left(\frac{\tau_{st} - \tau_s}{\tau_{sc} - \tau_s}\right)(\tau_{sc} - \tau_s) + \tau_s$$

$$= (0.86)(0.186 - 0.159) + 0.159 = \underline{0.182 \text{ psf}}$$

4. Find the Allowable tractive stress

$\phi_R = 38.4°$

$K = 0.45$

$$\text{Allowable } \tau_{Lb} = 0.4\, D_{75}$$

$$= 0.4(0.90) = 0.360 \text{ psf}$$

$$\text{Allowable } \tau_{Ls} = 0.4\, K D_{75}$$

$$= 0.4(0.45)(0.90) = 0.162 \text{ psf}$$

5. Check stability

τ_b, τ_{bc}, τ_{bt} $< \tau_{Lb}$ $\therefore$ bed is stable

$\tau_s < \tau_{Ls}$ but τ_{sc}, $\tau_{st} > \tau_{Ls}$ $\therefore$ sides in curve and just downstream of curve are not stable.

TR-25
Fine-Grained Discrete $\left(D_{75} < \begin{matrix} 0.25" \\ 6.35mm \end{matrix}\right)$

Maximum Tractive Stress

1. $\tau = \omega R_t S_e$ Reference Tractive Stress R_t = wetted perimeter (different, accounts for bed forms etc.)

$\tau = \dfrac{V^2 \rho}{\left(\dfrac{V}{\sqrt{\tau/\rho}}\right)^2}$ Reference Tractive Stress using Figs. 6-9 and 6-10, TR-25 Assume temp of water use 6-11.

Know V, g, ρ, S_e, $[K_s = D_{65}(ft)]$

a.) ρ, Fig. 6-11, TR-25

b.) ν, Fig. 6-11, TR-25 get $V/\sqrt{\tau/\rho} \Rightarrow \tau = ?$

c.) Calc. $\dfrac{V^3}{g \nu S_e}$ and $\dfrac{V}{\sqrt{g\ K_s\ S_e}}$

$K_s = D_{65}$ in ft.; if $D_{50} < 0.10mm$

use $D_{65} = 0.10mm = .000328$ ft.

d.) $\dfrac{V}{\sqrt{\tau/\rho}}$, Fig. 6-9 or 6-10

2. $\tau_b = \dfrac{\tau_b}{\tau} \times \tau$, Fig. 6-13, TR-25

$\tau_s = \dfrac{\tau_s}{\tau} \times \tau$, Fig. 6-12, TR-25

Accounts for Various b/d and z values

3. Correction Factors for Curvature

Within Curved Reach,
(Fig. 6-5, TR-25)
Same as for straight

$$\tau_{bc} = \frac{\tau_{bc}}{\tau_b} \times \tau_b$$

$$\tau_{sc} = \frac{\tau_{sc}}{\tau_s} \times \tau_s$$

Downstream of Curved Reach
(Fig. 6-6, TR-25)
Same

$$\tau_{bt} = \frac{\tau_{bt}-\tau_b}{\tau_{bc}-\tau_b} \times (\tau_{bc}-\tau_b) + \tau_b$$

$$\tau_{st} = \frac{\tau_{st}-\tau_s}{\tau_{sc}-\tau_s} \times (\tau_{st}-\tau_s) + \tau_s$$

Allowable Tractive Stress

1. $\tau_{Lb} = \tau_{Ls}$, Fig. 6-14, TR-25

For $D_{50} > 5$mm, use $\tau_{Lb} = \tau_{Ls}$ @ $D_{50} = 5$mm.

For $D_{50} < 0.1$mm, use $\tau_{Lb} = \tau_{Ls}$ @ $D_{50} = 0.1$mm

Check Stability

$\tau_b, \tau_s, \tau_{bc}, \tau_{sc}, \tau_{bt}, \tau_{st} < \tau_{Allowable}$

Tractive Force (Analysis Problem) 1 3

Stable Earth Channels
Problem 2

Given: A channel is to be constructed through an area of intense cultivation.

Q_{design} = 250 cfs (bankfull)

b = 15 ft

d = 3.7 ft

z = 1½ : 1

V = 3.26 fps

S = 0.0026

n (aged) = 0.045

Design Temp. = 50°F

The channel will be cut in nonplastic SM soil with D_{75} = 0.035 inches, D_{65} = 0.01075 inches (0.273 mm), and D_{50} = 0.127 mm.

There are no curves in the reach. Sediment load is quite light in this locality, in the range of clear water criteria.

Required: Determine the actual tractive stress and the allowable tractive stress for the given conditions.

Tractive Force (Analysis Problem) 2 3

Stable Earth Channels
Problem
Solution (Aged)

Since D_{75} = .035 in < 1/4 inch use the reference tractive stress method.

1. Find the actual tractive stress.

$\nu = 1.92 \times 10^{-5}$ ft²/sec (from Fig 6-11, TR-25)

$\rho = 1.940$ lb.sec²/ft⁴ (from Fig 6-11, TR-25)

$$\frac{V^3}{g\nu S_e} = 3.26^3 / ((32.2)(1.92\times10^{-5})(0.0026))$$

$$= 2.91 \times 10^7$$

$$\frac{V}{\sqrt{g K_s S_e}} = 3.26 / \sqrt{(32.2)(0.01075 \times 12)(.0026)}$$

d_{65} in mm = 376 = 376

$$\frac{V}{\sqrt{\tau/\rho}} = 21.6$$ (from Fig 6-9, TR-25)

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

$$= (3.26^2)(1.94)/(21.6)^2 = 0.0442 \text{ psf}$$

$b/d = 15/3.7 = 4.05$

$\tau_s/\tau = 1.04$ (from Fig 6-12, TR-25)

$\tau_b/\tau = 1.31$

Actual $\tau_s = \frac{\tau_s}{\tau} \times \tau = (\quad)(\quad) =$ _____ psf

Actual $\tau_b = \frac{\tau_b}{\tau} \times \tau = (\quad)(\quad) =$ _____ psf

2. Find the allowable tractive stress

For D_{50} = 0.127 mm and assuming clear water flow (curve No.)

Allowable τ = 0.025 psf

$\tau_s > \tau_{Allow}$ and $\tau_b > \tau_{Allow}$

$\therefore$ both the bed and banks of the channel are unstable.

Tractive Force (Analysis Problem) 1 3

Stable Earth Channels
Problem 2

Given: A channel is to be constructed through an area of intense cultivation.

Q_{design} = 250 cfs (bankfull)

b = 15 ft

d = 3.7 ft

z = 1½ : 1

V = 3.26 fps

S = 0.0026

n (aged) = 0.045

Design Temp. = 50°F

The channel will be cut in nonplastic SM soil with D_{75} = 0.035 inches, D_{65} = 0.01075 inches (0.273 mm), and D_{50} = 0.127 mm.

There are no curves in the reach. Sediment load is quite light in this locality, in the range of clear water criteria.

Required: Determine the actual tractive stress and the allowable tractive stress for the given conditions.

Tractive Force (Analysis Problem) 2 3

Stable Earth Channels
Problem
Solution (Aged)

Since D_{75} = .035 in < 1/4 inch use the reference tractive stress method.

1. Find the actual tractive stress.

$\nu = 1.42 \times 10^{-5}\ ft^2/sec$ (from Fig 6-11, TR-25)

$\rho = 1.940\ lb.sec^2/ft^4$ (from Fig 6-11, TR-25)

$$\frac{V^3}{g\nu S_e} = 3.26^3/((32.2)(1.42\times10^{-5})(.0026))$$

$$= 2.91 \times 10^7$$

$$\frac{V}{\sqrt{g K_s S_e}} = 3.26/\sqrt{(32.2)(.01075/12)(.0026)}$$

$$= 376$$

$$\frac{V}{\sqrt{\tau/\rho}} = 21.6$$ (from Fig 6-9, TR-25)

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

$$= (3.26^2)(1.94)/(21.6)^2 = 0.0442\ psf$$

$b/d = 15/3.7 = 4.05$

$\tau_s/\tau = 1.04$ (from Fig 6-12, TR-25)

$\tau_b/\tau = 1.36$

Actual $\tau_s = \frac{\tau_s}{\tau} \times \tau = (1.04)(.0442) = \underline{\underline{.0460\ psf}}$

Actual $\tau_b = \frac{\tau_b}{\tau} \times \tau = (1.36)(.0442) = \underline{\underline{.0601\ psf}}$

STATE AZ		PROJECT
BY	DATE	
SUBJECT TRACTIVE STRESS TR-25		SHEET 1 of ___

$d_{75} > 1/4''$ Use $\tau_\infty = \gamma_w \, d \, s_t$

Q = _____ cfs Se = _____ f/f

b = _____ ft Sample No. = _____

z = _____ :1 Station = _____

n = _____ d_{75} = _____ MM

d = _____ ft d_{65} = _____ MM

V = _____ f/s d_{50} = _____ MM

1. $S_t = \left(\frac{n_t}{n}\right)^2 S_e$

$n_t = \frac{(d_{n5})^{1/6}}{39} = \frac{(\quad)}{39} =$ _____

$S_t = (\quad)(\quad) =$ _____

$\tau_\infty = (62.4)(\quad)(\quad) =$ _____ psf

2. $\tau_b = \tau_\infty \left(\tau_b/\tau_\infty\right)$

b/d = _____ = _____.

$\tau_b/\tau_\infty =$ _____ (Fig. 6-3)

$\tau_b = (\quad)(\quad) =$ _____ psf

STATE AZ		PROJECT
BY	DATE	
SUBJECT TRACTIVE STRESS TR-25		SHEET 2 of ____

3. $\tau_{bc} = \tau_b \left(\tau_{bc}/\tau_b\right)$

$R_{c/b}$ = _____ = _____

τ_{bc}/τ_b = _____ (Fig. 6-5)

τ_{bc} = ( )( ) = _____ psf

4. $\tau_{bt} = \left(\dfrac{\tau_{bt} - \tau_b}{\tau_{bc} - \tau_b}\right)(\tau_{bc} - \tau_b) + \tau_b$

$L_{c/b}$ = _____ = _____

$\left(\dfrac{\tau_{bt} - \tau_b}{\tau_{bc} - \tau_b}\right)$ = _____ (Fig. 6-6)

τ_{bt} = ( )( )+( ) = _____ psf

5. $\tau_s = \tau_\infty \left(\tau_s/\tau_\infty\right)$

b/d = _____ = _____

τ_s/τ_∞ = _____ (Fig. 6-4)

τ_s = ( )( ) = _____ psf

STATE AZ		PROJECT
BY	DATE	
SUBJECT TRACTIVE STRESS TR-25		SHEET 3 of ___

6. $\tau_{sc} = \tau_s \left(\tau_{sc}/\tau_s\right)$

R_c/b = _____ = _____.

τ_{sc}/τ_s = _____ (Fig. 6-5)

τ_{sc} = ( )( ) = _____ psf

7. $\tau_{st} = \left(\frac{\tau_{st} - \tau_s}{\tau_{sc} - \tau_s}\right)(\tau_{sc} - \tau_s) + \tau_s$

L_c/b = _____ = _____.

$\left(\frac{\tau_{st} - \tau_s}{\tau_{sc} - \tau_s}\right)$ = _____ (Fig. 6-6

τ_{st} = ( )( )+( ) = _____ psf

8. τ allow

$\tau_{Lb} = 0.4\ d_{75}$ = (0.4)( ) = _____ psf

$\tau_{Ls} = 0.4\ d_{75}\ K$

d_{75} = _____ ins.

ϕ_R = _____ Fig. 6-7

K = _____ Fig. 6-8

τ_{Ls} = (0.4)( )( ) = _____ psf

STATE AZ		PROJECT	
BY	DATE		
SUBJECT TRACTIVE STRESS TR-25			SHEET 4 of ____

9. Check Stability

τ_b = _____

$\tau_{b\,c}$ = _____

τ_{bt} = _____

τ_{Lb} = _____

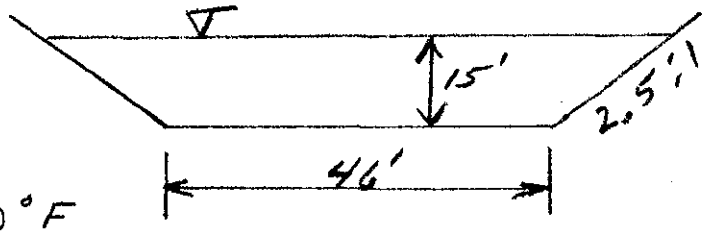
τ_s = _____

τ_{sc} = _____

τ_{st} = _____

τ_{Ls} = _____

STATE		PROJECT		
BY	DATE	CHECKED BY	DATE	JOB NO.
SUBJECT				SHEET ____ OF ____



$T = 50°F$

$Q = 4750$ cfs

$V = 3.77$ fps

$n = 0.03$

$D_{75} = 0.15$ mm

$D_{65} = 0.00256$ inch (0.065 mm)

$\frac{q_u}{2} = 400$ psf = shear strength

$S_e = 0.00042$

Determine: Actual tractive power and stability of channel.

STATE AZ		PROJECT
BY	DATE	
SUBJECT TRACTIVE POWER TR-25		SHEET 1 of

$d_{75} < 1/4''$ Use τ as REF. STRESS

$Q =$ 4750 cfs
$b =$ 46 ft
$z =$ 2.5 :1
$n =$ 0.03
$d =$ 15 ft
$V =$ 3.77 f/s

$S_e =$ 0.00042 f/f
τ water = 50 °F.
shear UC Strength = 400 lbs/ft²
Sample No. =
Station =

$d_{75} =$ 0.15 MM
$d_{65} =$ MM , 00256 IN
$d_{50} =$ MM
$K_s = d_{65}$ in ft

1. $\tau = \left(\dfrac{\frac{V^2 \rho}{V}}{\sqrt{\tau/\rho}}\right)^2$

$\rho =$ 1.940 $\dfrac{lb\text{-}Sec^2}{ft^4}$ (Fig. 6-11)

$\nu =$ 1.42×10^{-5} ft/Sec (Fig. 6-11)

$\dfrac{V^3}{g \nu S_e} =$ _____ = 2.79×10^8

$\dfrac{V}{\sqrt{g K_s S_e}} =$ $\sqrt{}$ = 2,220

$\dfrac{V}{\sqrt{\tau/\rho}} =$ 27 (Fig. ~~6-9~~ 6-10)

$\tau = \left(\dfrac{3.77}{27}\right)^2 1.94 =$ 0.0378 psf

STATE		PROJECT
BY	DATE	
SUBJECT	TRACTIVE POWER TR-25	SHEET 2 of ___

2. $\tau_b = \tau\,(\tau_b/\tau)$ Tractive stress on bottom

$b/d = \frac{46}{15} = 3.07$

$\tau_b/\tau = 1.45$ (Fig. 6-13)

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

3. $\tau_{bc} = \tau_b\,(\tau_{bc}/\tau_b)$ No curves on this problem

$R_c/b = ____ = ____$

$\tau_{bc}/\tau_b = ____$ (Fig. 6-5)

$\tau_{bc} = (\quad)(\quad) = ____$ psf

4. $\tau_{bt} = \left(\frac{\tau_{bt} - \tau_b}{\tau_{bc} - \tau_b}\right)(\tau_{bc} - \tau_b) + \tau_b$

$L_c/b = ____ = ____$

$\left(\frac{\tau_{bt} - \tau_b}{\tau_{bc} - \tau_b}\right) = ____$ (Fig. 6-6)

$\tau_{bt} = (\quad)(\quad) + (\quad) = ____$ psf

STATE AZ	PROJECT	
BY	DATE	
SUBJECT TRACTIVE POWER TR-25		SHEET 3 of ___

5. $\tau_s = \tau\,(\tau_s/\tau)$. Tractive stress on sideslope

$b/d =$ _____ $=$ 3.07 .

$\tau_s/\tau =$ 1.21 (Fig. 6-12)

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

6. $\tau_{sc} = \tau_s \left(\frac{\tau_{sc}}{\tau_s}\right)$ NO CURVE

$R_c/b =$ _____ $=$ _____ .

$\tau_{sc}/\tau_s =$ _____ (Fig. 6-5)

$\tau_{sc} = (\quad)(\quad) =$ _____ psf

7. $\tau_{st} = \left(\frac{\tau_{st}-\tau_s}{\tau_{sc}-\tau_s}\right)(\tau_{sc}-\tau_s)+\tau_s$

$L_c/b =$ _____ $=$ _____ .

$\left(\frac{\tau_{st}-\tau_s}{\tau_{sc}-\tau_s}\right) =$ _____ (Fig. 6-6)

$\tau_{st} = (\quad)(\quad)+(\quad) =$ _____ psf

STATE AZ		PROJECT
BY	DATE	
SUBJECT TRACTIVE POWER TR-25		SHEET 4 of ___

8. Calculate Tractive Power

T.P. = τ_b, bc, ctc x V

τ_b = (0.0548) (3.77) = 0.207

τ_{bc} = (_____) (_____) = _____

τ_{bt} = (_____) (_____) = _____

τ_s = (0.0458) (3.77) = 0.173

τ_{sc} = (_____) (_____) = _____

τ_{st} = (_____) (_____) = _____

9. Check each against Fig. 6-15

Shear ~~UCS~~ = 400 lb/ft

τ all. = 1.0 lbs/ft. sec.

→ Watershed Geomorphology - Origin of landscapes, processes by which they have been formed

Article Author CE Magazine JAN 1990

Overlooked flow parameters for flood cell design:

Sediment Transfer, critical flow? Manning's "n"?

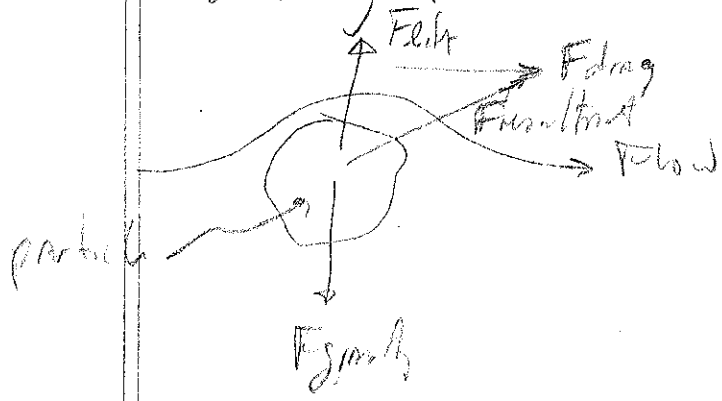
do banks actually erode during floods?, natural vegetation

★ Design with natural processes in mind!

→ Tractive Stress Method

H/O 1,2,3 Sed movement = f(velocity)^n can be visually observed

Threshold of



Forces: 1. resisting - (weight) resists movement

2. fluid - tends to move particle

★ Three approaches for channel design

1. GA power law $V_c = K(F_w)^{1/6}$

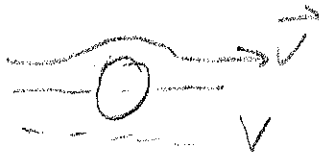
~ function of weight

Volume = f(diam)^3

then $V_c = K(D)^{1/2}$

~ particle diameter

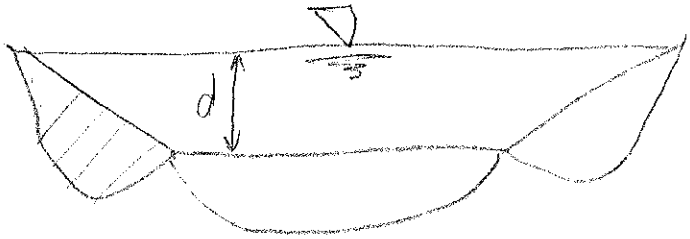
2. Hydraulic lift
Vel over particle is >> than upper side particle



causes lift. not developed

3. Drag-Force Concept (Tractive Force) see 4/0 (1)
most common see 4/0 (2)

Actual "n" value for channel is greater than the n value for particles under consideration



$\tau_{ave} = \gamma R S$
$\tau = k \gamma d S$

and τ_c is part of diameter, called a $d = 5$ mm (4")

→ $\tau_c = 0.9 d_{75}$ where $d_{75} > 5$ mm no FS.

τ_c selected < τ_c bed

Open channel turbulent → trans. Manning
laminar → trans. Darcy

☆ Tractive Power = f(velocity, d, S)
remember $\tau_o = \gamma_w d S_f$

Tractive Power = τ_o Vel = $\frac{lb}{ft^2} \times \frac{ft}{sec} = \frac{lb\ ft}{sec} \cdot \frac{1}{ft^2}$ = Power/Area
(1 hp = 550 $\frac{ft\text{-}lb}{sec}$ = power)
= f (compressive strength)

remember shear strength · f'c conc. 3000-4000 psi
soil ≈ 6 psi

soil must be 100% saturated

Tractive Power - based on observations from several stable channels around the world. [illegible]

DESIGN PROCEDURE

Tractive Stress

When the same material exists in the channel bottom and sides, stability limitations can be determined in terms of maximum depth.

One procedure is as follows:

1. Assume uniform flow. $-D_{75} > 1/4$ inches.

2. Find maximum depth for stability.

   a. Compute $n_t = \dfrac{D_{75}^{1/6}}{39}$

   b. Compute $s_t = \left(\dfrac{n_t}{n}\right)^2 s_e$

      where s_e is the slope of the hydraulic control line.

   c. Compute K.

   d. Assume that b/d ratio is greater than 4. From Figs. 6-3 & 6-4, a conservative value of $\dfrac{T_s}{T_\infty} = 0.78$ and $\dfrac{T_b}{T_\infty} = 0.99$.

      Then $\dfrac{T_s}{T_b} = 0.79$.

   e. Compare K with 0.79.

      (1) If $K > 0.79$, stability of the bed will control.

      (2) If $K < 0.79$, stability of the sides will control.

   f. Solve for maximum d from the relations

      (1) bed - $d = \dfrac{T_{Lb}}{(0.99)\gamma s_t}$

      (2) sides - $d = \dfrac{T_{Ls}}{0.78\ \gamma s_t}$

3. Compute maximum depth for stability - aged condition.

   a. Compute n_t and s_t for the aged condition.

   b. Follow steps (c - f) of step 2 using s_t associated with the aged condition and determine maximum depth for stability for the aged condition.

4. Using Manning chart or trial solution solve for the required bottom width to provide capacity for the aged condition.

5. Check b/d ratios to insure that the assumption in step 2 is valid.

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 $\dfrac{V}{\sqrt{gk_s s_e}}$.

5. Enter the graph in Figure 6-9 (or Figure 6-10) with the computed values in steps 2 and 3 above and read the value of $\dfrac{V}{\sqrt{\tau/\rho}}$.

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.

Given: A straight reach of channel is to be designed. The material in the bed and banks is non-cohesive and has a D_{75} size of 0.27 inches. The particles are moderately angular.

$Z = 2$

Slope of the hydraulic control line = 0.0008

$Q_{(design)} = 500$ c.f.s.

$Q_{10} = 100$ c.f.s.

$n(\text{as built}) = 0.026$ $\qquad n_t + .005 = .021 + .005 = .026$

$n(\text{aged}) = 0.035$

Find: Select depth and bottom width of a channel to meet capacity and stability requirements.

Solution

Given: A straight reach of channel is to be designed. The material in the bed and banks is non-cohesive and has a D_{75} size of 0.27 inches. The particles are moderately angular.

$Z = 2$

slope of the hydraulic control line = 0.0008

$Q_{(design)} = 500$ c.f.s.

$Q_{10} = 100$ c.f.s.

$n(\text{as built}) = 0.026$

$n(\text{aged}) = 0.035$

Find: Select depth and bottom width of a channel to meet capacity and stability requirements.

Solution:

1. Assume uniform flow.

2. As built condition. Find max. depth for stability.

$$n_t = \frac{D_{75}^{1/6}}{39} \quad \frac{(0.27)^{1/6}}{39} = 0.021$$

$$S_t = \left(\frac{n_t}{n}\right)^2 S_e = \left(\frac{0.021}{0.026}\right)^2 0.0008 = 0.000522$$

Allowable tractive stress.

$\emptyset = 30.8^{o}$, $K = 0.49$ (Sheets 3 and 4 of ES-140) (Figs. 6-7, 6-8)

$T_{Ls} = (0.49)(0.4)(0.27) = 0.053$ #/ft^2

Assume that $T_s = 0.8T_\infty$ - This is conservative and reasonable for $\frac{b}{d}$ ratios greater than 2.

Let $T_{Ls} = T_s = 0.8T_\infty = 0.053 = 0.8\gamma d S_t = (0.8)(62.4)(0.000522)d$

$$d = \frac{0.053}{(0.000522)(0.8)(62.4)} = 2.03 \text{ ft.}$$

Q for checking as-built stability = 250 cfs

Determine b for Q = 250, d = 2.03', S_0 = 0.0008, n = 0.026, Z = 2

$$AR^{2/3} = \frac{Qn}{1.486S^{1/2}} = \frac{250(.026)}{1.486(.0283)} = 155$$

try b = 30'	A = 69.1	WP = 39.1	R = 1.77	$R^{2/3}$ = 1.46	$AR^{2/3}$ = 101	NG
50	109.7	59.1	1.86	1.51	165	NG
47	103.7	56.1	1.85	1.51	156	<u>OK</u>

check - $T_\infty = \frac{.053}{0.8} = 0.066$

$\frac{b}{d} = \frac{47}{2.03} = 23$ $\frac{T_s}{T_\infty} = 0.78$ $\frac{T_b}{T_\infty} = 1$

$T_s = 0.78(0.066) = 0.051$ #/ft^2 $T_{Ls} = 0.053$ #/ft^2 <u>OK</u>

$T_b = 0.066$ #/ft^2 $T_{Lb} = 0.4(0.27) = 0.108$#/ft^2 <u>OK</u>

3. Aged condition - since depth of flow is to be determined, both aged conditions are met. Find max. depth for stability.

$$S_t = \left(\frac{n_t}{n}\right)^2 S_e = \left(\frac{0.021}{0.035}\right)^2 0.0008 = 0.000288$$

$T_{Ls} = 0.053 = T_s = 0.8T_\infty = (0.8)\ \gamma\ ds_t = (0.8)(62.4)(0.000288)d$

$$d_{(max)} = \frac{0.053}{(0.8)\ (62.4)\ (0.000288)} = 3.69 \text{ ft.}$$

Find bottom width for capacity.

From Drawing No. 4-R-22873-3 for Q = 500, n = 0.035, d = 3.69 ft.

S = 0.0008 read b = 45 ft., but min. b = 47' from above.

This will not reduce the depth enough to require recomputation

$$\frac{b}{d} = \frac{47}{3.69} = 13$$

therefore, as before $T_s = \cancel{0.51}\ 0.051\ \#/ft^2$ $T_{Ls} = 0.053\ \#/ft^2$

$T_b = 0.066$ $T_{Lb} = 0.108$

4. Ans. Use a 47 ft. bottom width - depth of 3.69 ft.

STABILITY DESIGN OF GRASS LINED CHANNELS

```
A:\>DIR

 Volume in drive A is RSFILES
 Directory of  A:\

SUPLST            3328  12-28-84   9:26p
SALDATDY          1280   3-29-85  12:23a
SALDATMO          1280   3-29-85  12:23a
SALDATYR          1280   3-29-85  12:23a
SALDDISC          1280  12-28-84  10:12p
R1               97536  12-28-84   9:32p
R2               74368  12-28-84   9:47p
R3               86720  12-28-84   9:49p
ENGLISH  BAS     24064   1-01-80  12:06a
METRIC   BAS     24192   1-01-80   4:18a
       10 File(s)     176128 bytes free

A:\>
```

Computer program developed by ARS

1. LOAD "BASICA" INTO RAM
2. LOAD > "ENGLISH" or "METRIC"
3. Enter data as prompted on screen

EFFECTIVE STRESS DESIGN OF GRASS-LINED CHANNELS
(DIRECT DESIGN ROUTINE)
ENGLISH UNIT VERSION

ENTER CROSS-SECTIONAL SHAPE
TRAPEZOIDAL ---------- 'T'
TRIANGULAR ---------- 'TR'
PARABOLIC ---------- 'P'
?T

INPUT CHANNEL BED SLOPE? .003
ENTER COTANGENT OF BANK SIDE SLOPE ?3
ENTER LIMITING MINIMUM BED WIDTH (DEFAULT APPROXIMATES MIN. FLOW AREA)?1

12ft

SCREEN 1

1

SOIL SURFACE LAYER INFORMATION
OPTIONS AVAILABLE FOR ENTRY OF SOIL ERODIBILITY

OPTION	CODE
Basic soil PROPERTIES (plasticity, density, gradation)	P
Allowable effective STRESS plus soil grain roughness	S
Reference bare soil permissible VELOCITY	V

ENTER INPUT OPTION (Default = 'P')? P
Soil types from unified soil classification system
Do not use double classifications such as CL-ML
SURFACE SOIL CLASSIFICATION ? CL
PLASTICITY INDEX? 13
ENTER VOID RATIO (default is to a void ratio correction of 1.0)? 1

more [illegible] → e ?

SCREEN 2

```
                    BOUNDARY COVER INFORMATION
TYPE OF COVER (descriptive name)? GRASS
THE FLOW RETARDANCE POTENTIAL OF THE VEGETATION MAY BE ENTERED AS;
            Stem length and density data                    LD
            SCS retardance class                           SCS
            Direct input of the retardance Curve Index      DI
MEANS OF ESTIMATING VEGETAL FLOW RETARDANCE (LD, SCS, or DI)? SCS
            ENTER COVER CONDITIONS FOR STABILITY CALCULATIONS
RETARDANCE CLASS ? C
VEGETAL COVER FACTOR ? .5      50 % cover

CHANNEL DISCHARGE  (cfs) ? 130
```

SCREEN 3

```
INPUT DATA FOR TRAPEZOIDAL CHANNEL
   CHANNEL BED SLOPE = .003  ft/ft
   DESIGN DISCHARGE = 130  cfs
   MINIMUM BED WIDTH OF 12  ft
   BANK SIDE SLOPES OF  3 :1
SOIL BOUNDARY IS CLASSIFIED AS CL BY UNIFIED SOIL CLASSIFICATION
   PLASTICITY INDEX =  13
   VOID RATIO =  1
GRASS VEGETAL COVER
   SCS RETARDANCE  CLASS   C
   VEGETAL COVER FACTOR =  .5


IF DATA IS CORRECT HIT ANY KEY TO CONTINUE, IF NOT HIT 'C' TO CORRECT
```

SCREEN 4

OUTPUT

GRASS-LINED TRAPEZOIDAL CHANNEL

SOLUTION FOR STABILITY
(establishes geometric control)

CL SOIL BOUNDARY
PLASTICITY INDEX = 13
VOID RATIO = 1
ALLOWABLE EFFECTIVE STRESS = .0343 lb/sq.ft
SOIL GRAIN ROUGHNESS COEFFICIENT = .0156
GRASS COVER FALLS IN SCS RETARDANCE CLASS 'C'

CURVE INDEX	MANNING COEFFIC.	AVERAGE VELOCITY ft/s	VOLUMETRIC DISCHARGE cu.ft/s	COVER FACTOR	EFFECTIVE SOIL STRESS lb/sq.ft
5.60	0.043	2.73	130.00	0.50	0.0303

BOTTOM WIDTH ft	TOP WIDTH ft	FLOW DEPTH ft	SIDE SLOPE z:1	X-SECT. AREA sq.ft	HYDRAULIC RADIUS ft	BED SLOPE ft/ft
12.0	26.7	2.46	3.00	47.6	1.73	0.003

* The limiting value of BED WIDTH was found to control channel dimensions
Stability is probably NOT the limiting factor in this case

SCREEN 5

ENTER COVER CONDITIONS FOR CAPACITY CALCULATIONS
RETARDANCE CLASS ? ~~D~~ B or higher

SCREEN 6

SOLUTION FOR CAPACITY
(establishes required flow depth)
GRASS COVER FALLS IN SCS RETARDANCE CLASS 'D'

CURVE INDEX	MANNING COEFFIC.	AVERAGE VELOCITY ft/s	VOLUMETRIC DISCHARGE cu.ft/s
4.44	0.034	3.21	130.00

BOTTOM WIDTH ft	TOP WIDTH ft	FLOW DEPTH ft	SIDE SLOPE z:1	X-SECT. AREA sq.ft	HYDRAULIC RADIUS ft	BED SLOPE ft/ft
12.0	25.1	2.18	3.00	40.5	1.57	0.003

SCREEN 7 Program ends

When to Apply:

1. Tractive stress —
   Non-cohesive soils (PI < 10
   velocity range 1.5 fps — 10 fps
   remember 2 different grain sizes
   d_{75} < ¼" → 1.5 < vel < 3.5
   d_{75} > ¼" – 5" 1.5 < vel < 10

2. Allowable Velocity Approach
   plastic soils PI ≥ 10
   Velocity range 2 fps – 6 fps

3. Modified Regime
   Velocity: 1.5 — 2.5 fps
   long reaches (irrigation channels)
   low plastic soils
   try 1 or 2 first.

4. Tractive Power (must have close correlation to other methods
   Soils
   be very careful at v > 9 fps

Allowable Velocity Method

design = f (V_{allow})

find V_{allow} that does not erode, may carry silt

Modified Regime Approach

Regime – stream in equilibrium, Δ sediment remains the same. Channels that are "in regime" may change but the net change over time is zero.

1. Calculate V or S from empirical equations regime stable
2. Compare to V or S required for hydraulic performance
3. When these V's or S's are ≈ equal, the channel is in "regime" or equilibrium

Permissible Velocity Method

1

TABLE 7-3. MAXIMUM PERMISSIBLE VELOCITIES RECOMMENDED BY FORTIER AND SCOBEY AND THE CORRESPONDING UNIT-TRACTIVE-FORCE VALUES CONVERTED BY THE U.S. BUREAU OF RECLAMATION*

(For straight channels of small slope, after aging) & depths < 3 ft.

Material	n	Clear water		Water transporting colloidal silts	
		V, fps	τ_0, lb/ft²	V, fps	τ_0, lb/ft²
Fine sand, colloidal	0.020	1.50	0.027	2.50	0.075
Sandy loam, noncolloidal	0.020	1.75	0.037	2.50	0.075
Silt loam, noncolloidal	0.020	2.00	0.048	3.00	0.11
Alluvial silts, noncolloidal	0.020	2.00	0.048	3.50	0.15
Ordinary firm loam	0.020	2.50	0.075	3.50	0.15
Volcanic ash	0.020	2.50	0.075	3.50	0.15
Stiff clay, very colloidal	0.025	3.75	0.26	5.00	0.46
Alluvial silts, colloidal	0.025	3.75	0.26	5.00	0.46
Shales and hardpans	0.025	6.00	0.67	6.00	0.67
Fine gravel	0.020	2.50	0.075	5.00	0.32
Graded loam to cobbles when noncolloidal	0.030	3.75	0.38	5.00	0.66
Graded silts to cobbles when colloidal	0.030	4.00	0.43	5.50	0.80
Coarse gravel, noncolloidal	0.025	4.00	0.30	6.00	0.67
Cobbles and shingles	0.035	5.00	0.91	5.50	1.10

* The Fortier and Scobey values were recommended for use in 1926 by the Special Committee on Irrigation Research of the American Society of Civil Engineers.

STATE AZ		PROJECT ENGINEERING WORKSHOP		
BY Tom JAYO	DATE 1/90	CHECKED BY	DATE	JOB NO.
SUBJECT ALLOWABLE VELOCITY METHOD				SHEET 3 OF

CHARACTERISTICS
OF A
STABLE CHANNEL

1. Neither aggrades or degrades beyond tolerable limits.

2. Banks do not erode such that cross section changes appreciably.

3. Excessive sediment bars do not develop.

4. Gullies do not form or enlarge because of entry of uncontrolled surface flow to the channel.

FROM
EPS 582

$$\begin{pmatrix}\text{Plan}\\ \text{Agree.}\end{pmatrix} + \begin{pmatrix}\text{Des.}\\ \text{Docum}\end{pmatrix} + \begin{pmatrix}\text{Proj}\\ \text{Agree}\end{pmatrix} + \begin{pmatrix}\text{O\&M}\\ \text{Agree}\end{pmatrix} \neq \begin{matrix}\text{Des.}\\ \text{Def.}\end{matrix}$$

FORMULA TO
MINIMIZE DESIGN DEFICIENCIES
AND
MAXIMIZE UNDERSTANDING OF O&M.

STATE AZ		PROJECT ENGINEERING WORKSHOP		
BY Tom Jayo	DATE 11/90	CHECKED BY	DATE	JOB NO.
SUBJECT ALLOWABLE VELOCITY METHOD				SHEET 4 OF

DESIGN OBJECTIVES

1. Limits of aggradation & degradation quantity vs event
2. Bank stability – Must maintain stability
3. Sediment bars – keep to a minimum
4. Uncontrolled entry – any? stablize

SUMMARY

Clear, concise – reach by reach basis
Design reports are important.

ANTICIPATED PERFORMANCE & O&M

1. Sed. removal or repairs or armoring frequency ~ amounts
2. Problem bank areas
3. Sed bar removal – where, when
4. Entry problems – risk, repairs

STATE AZ		PROJECT ENGINEERING WORKSHOP		
BY TOM DAYO	DATE 2/90	CHECKED BY	DATE	JOB NO.
SUBJECT ALLOWABLE VELOCITY METHOD				SHEET 5 OF

SOURCES OF CRITERIA

1. CPM - 11

A. SETS UP NHCP
B. MINIMUM ENGINEERING
C. REQUIRES ANY DEVIATIONS VIA NEM 501.30

2. NEM 531.09 - REQUIRES GEOLOGIC INVESTIGATIONS TO BE IN ACCORDANCE WITH:

A. TR-25
B. NEH - SEC 8 - GEOLOGY
C. NEH - SEC 16 - DRAINAGE

3. NEM 501.30 SETS UP PROCEDURE FOR DEVIATION FROM TR 25 AND PS 582 - NEED IS DETERMINED BY S.C.E. & APPROVED BY THE HEAD, WNBC ENGINEERING STAFF.

PRACTICE STD 582 - OPEN CHANNEL USING TR-25 AS REF CRITERIA

DEFINES STABLE CHANNEL -

DESIGN FOR:

1. AS BUILT: BANKFULL, Q DES OR 10YR - LEAST BUT NOT < 50% Q_{DESIGN} - MAY BE INCREASED FOR RAPID VEGETATION.

2. AGED: BANKFULL, Q_{DESIGN} - GREATEST BUT NOT > 100 YR.

STABILITY ANALYSIS THAT ARE FLOW RELATED ARE NOT REQUIRED IF V < 2 FPS.

DISCRETE
VS
NON-DISCRETE

Difference is mainly in the degree of bond or cohesion between individual particles

DISCRETE or non cohesive

1. Consists mainly of larger, uncemented particles -

2. Generally cobbles, gravels, sands, and coarser non plastic silts.

3. Resistance to erosion depends on:

   a. size
   b. shape
   c. fall velocity
   d. mass density
   e. specific weight
   f. specific gravity

NON - DISCRETE

or coherent or cohesive

1. Consists mainly of materials that have a clay or organic binder - or some type of cementation

2. Generally clays, clayey sands, clayey silts, mucks, peats, boulder clays

3 Resistance to erosion depends on:

a. Particle properties

b. strength of bond where bond effects are much greater than physical characteristics of individual particles

CHANNEL EVALUATION PROCEDURAL GUIDE - TR-25

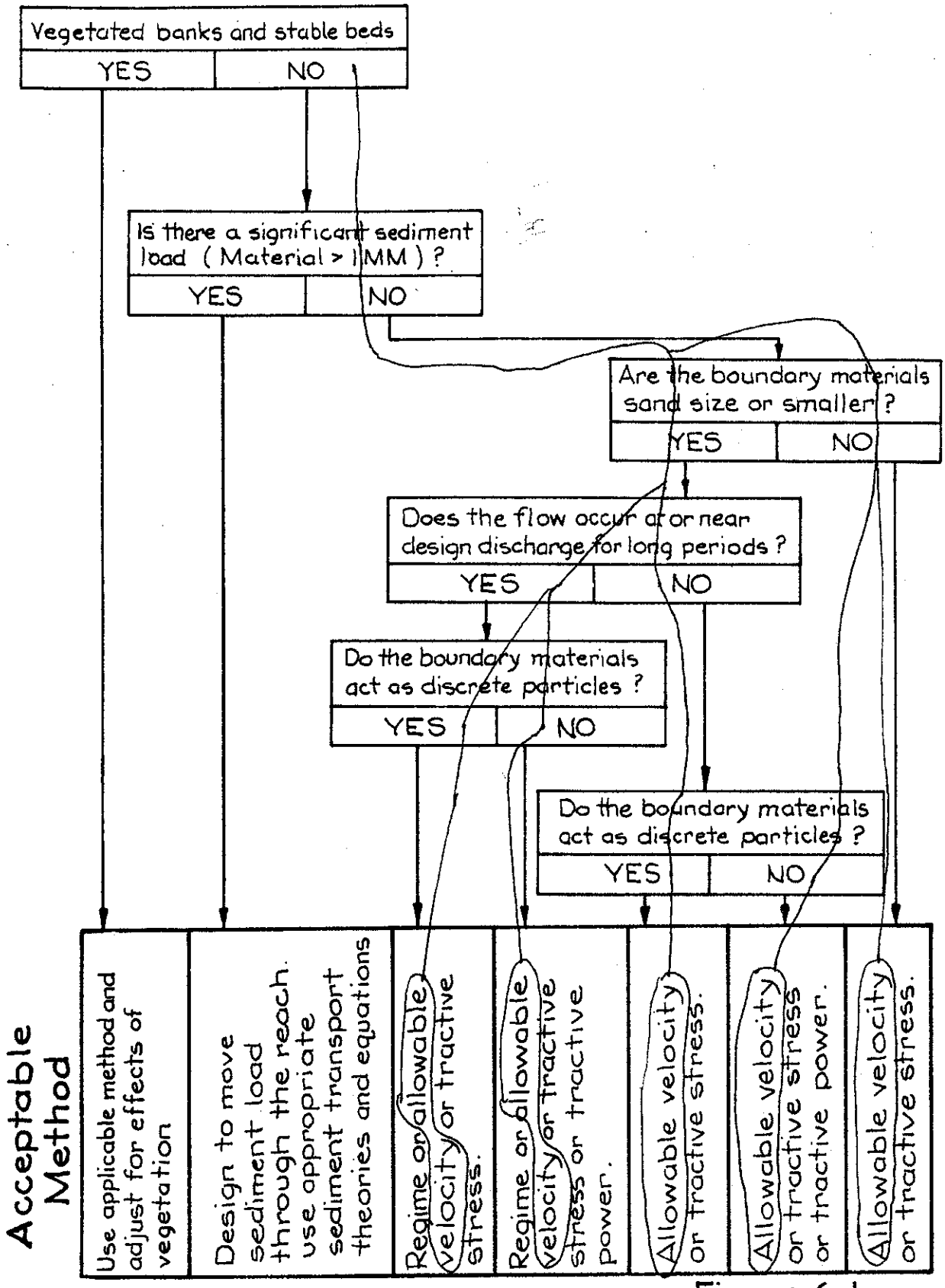


Figure 6-1

10

ALLOWABLE VELOCITIES FOR UNPROTECTED EARTH CHANNELS	
CHANNEL BOUNDARY MATERIALS	ALLOWABLE VELOCITY
DISCRETE PARTICLES Sediment Laden Flow ≥ 20,000 PPM Medium Sands $D_{75} > 0.4$ mm Fine Sands $D_{75} < 0.4$ mm	Basic velocity chart value x D x A x B 2.0 fps - Fine Sands with max Sediment
Sediment Free Flow ≥ 1,000 PPM Coarse Sand $D_{75} > 2.0$ mm Medium Sand $D_{75} < 2.0$ mm	Basic velocity chart value x D x A x B 2.0 fps - Medium Sand max w/o Sediment
COHERENT EARTH MATERIALS PI > 10 PI < 10	Basic velocity chart value x D x A x F x C_e 2.0 fps - Low PI w or w/o Sediment max

1

FROM FIG. 6-2
TR-25

ALLOWABLE VELOCITIES
FOR UNPROTECTED EARTH CHANNELS

PROCEDURE

1. ALLOWABLE VELOCITY

FOR STABILITY
- No sign. sed load (> 1.0 mm)
- Rigid Boundary
- Clear H_2O ($\leq$ 1,000 ppm fines)
- Sed laden ($\geq$ 20,000 ppm fines)
- TR-25

$V_{allowable\ for\ channel\ material} > V_{actual\ due\ to\ water\ flow}$

TERMINOLOGY

1) Synonomous Terms - Channel Boundary Material

1. Non Cohesive = Discrete particles
2. Cohesive = coherent mass = non discrete

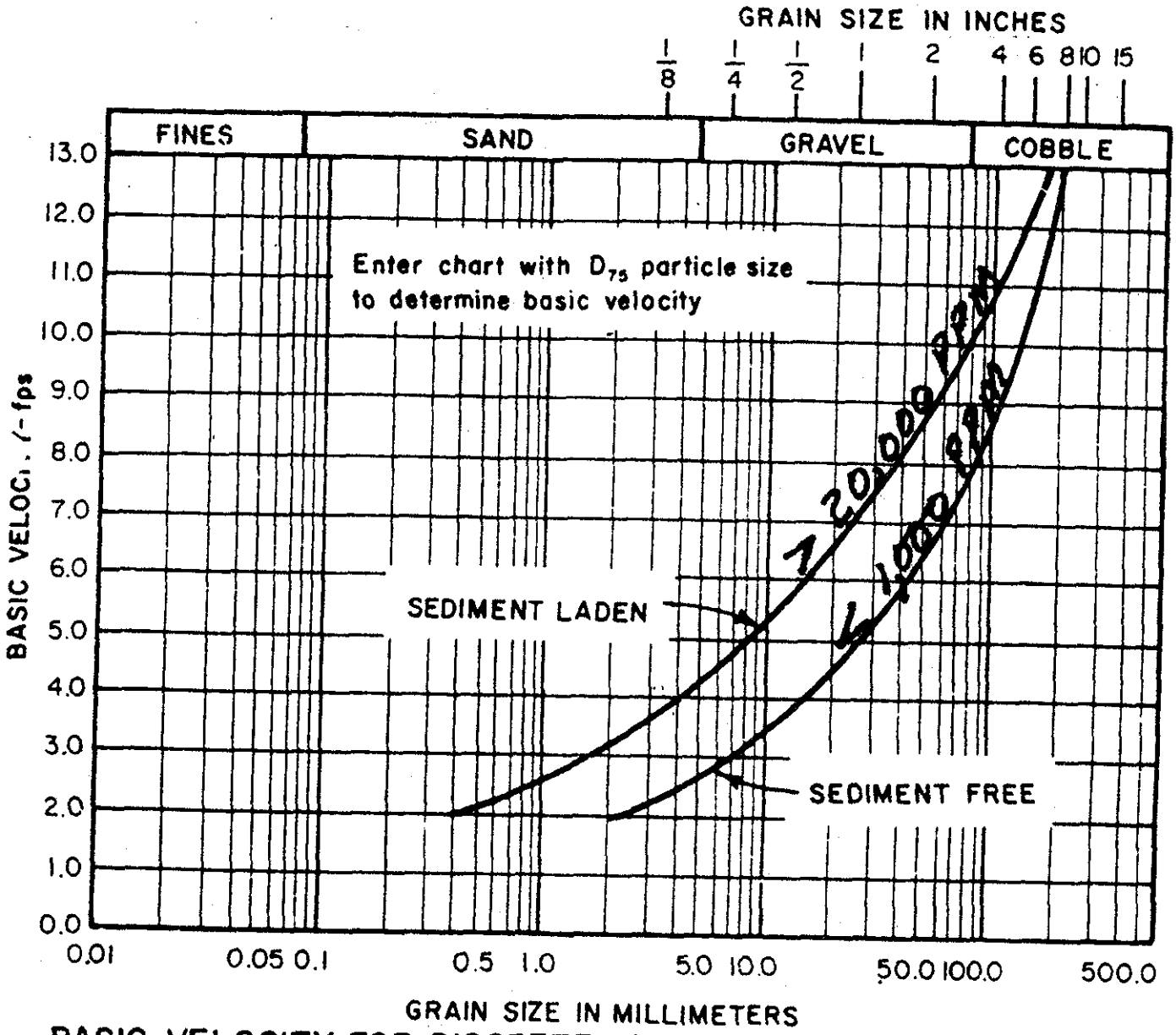
1. Sediment free flow = condition in which fine material is carried in suspension by the flow at concentrations so low that it has no effect on channel stability. $C < 1000$ ppm are sediment free flows.

2. Sediment laden flow = 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 $C \geq 20{,}000$ ppm by weight

$$C_{ppm} = 370 \frac{Q_s\ (\text{tons/day})}{Q\ \text{cfs}} \qquad D_s < 0.074\ \text{mm}\ (-200)$$

ALLOWABLE VELOCITY PROCEDURE

FIG. 6.2 - METHOD TO OBTAIN ALLOWABLE VELOCITY FOR UNPROTECTED EARTH MATERIALS FOR DISCRETE PARTICLES WITH NO BED LOAD TRANSPORT AND ONLY FINE SUSPENDED SEDIMENT CARRIED BY THE FLOW.



BASIC VELOCITY FOR DISCRETE PARTICLES OF EARTH MATERIALS (v_b)

D_{75}

Need Geologist input

Linearly interpolate for suspended sediment concentrations between 1000 PPM & 20,000 PPM

ALLOWABLE VELOCITY PROCEDURE

FOR DISCRETE PARTICLES

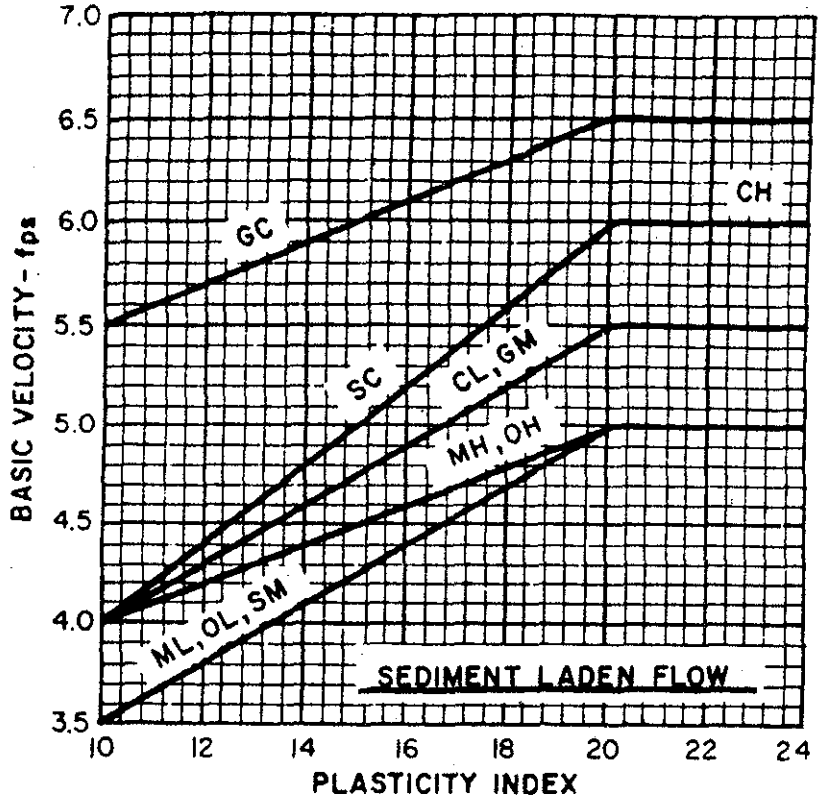
SEDIMENT LADEN FLOW ($C \geq 20{,}000$ PPM)	• $V_{ALLOWABLE} = V_b \times A \times D \times B$ D_{75} OF CHANNEL BOUNDARY MATERIAL > 0.40 mm .016" • $V_{ALLOWABLE} = V_b = 2$ fps D_{75} OF CHANNEL BOUNDARY MATERIAL ≤ 0.40 mm
SEDIMENT FREE FLOW ($C \leq 1{,}000$ PPM)	• $V_{ALLOWABLE} = V_b \times A \times D \times B$ D_{75} OF CHANNEL BOUNDARY MATERIAL > 2.0 mm .079" • $V_{ALLOWABLE} = V_b = 2$ fps D_{75} OF CHANNEL BOUNDARY MATERIAL ≤ 2.0 mm

ALLOWABLE VELOCITY METHOD

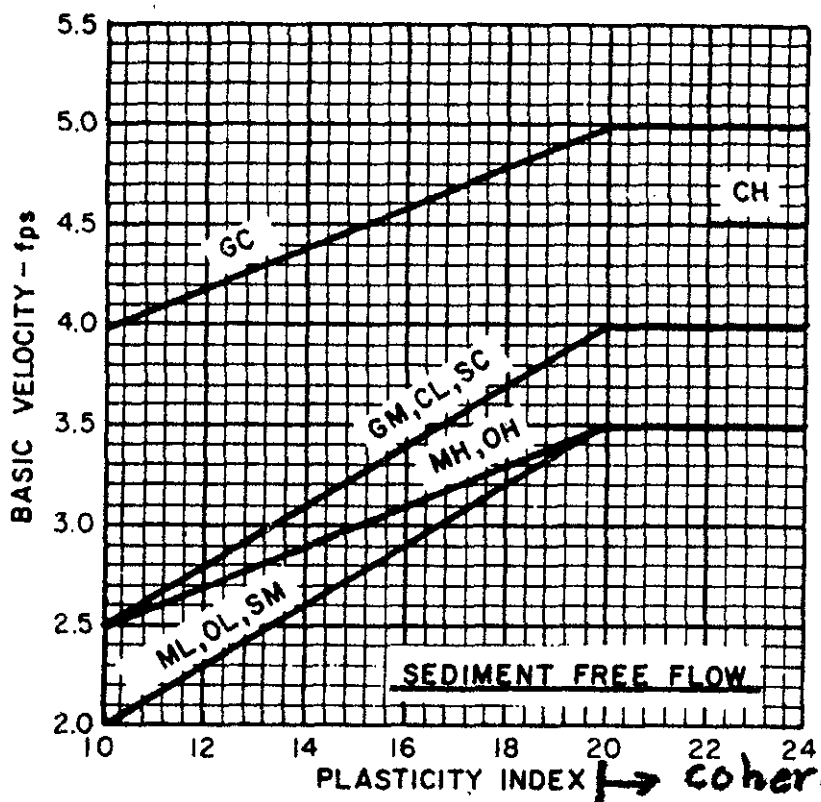
FOR COHERENT SOILS

SEDIMENT LADEN FLOW ($C \geq 20{,}000$ PPM)	• $V_{ALLOWABLE} = V_b \times A \times D \times C_e \times F$ $PI > 10$ • $V_{ALLOWABLE} = V_b = 2$ fps $PI < 10$
SEDIMENT FREE FLOW ($C \leq 1000$ PPM)	• $V_{ALLOWABLE} = V_b \times A \times D \times C_e \times F$ $PI > 10$ • $V_{ALLOWABLE} = V_b = 2$ fps $PI < 10$

FIG. 6-2 - METHOD TO OBTAIN ALLOWABLE VELOCITY FOR UNPROTECTED EARTH MATERIALS FOR COHERENT MASS WITH NO BED LOAD TRANSPORT AND ONLY FINE SUSPENDED SEDIMENT CARRIED BY THE FLOW.



BASIC VELOCITIES FOR COHERENT EARTH MATERIALS (v_b)



Need Geologist input

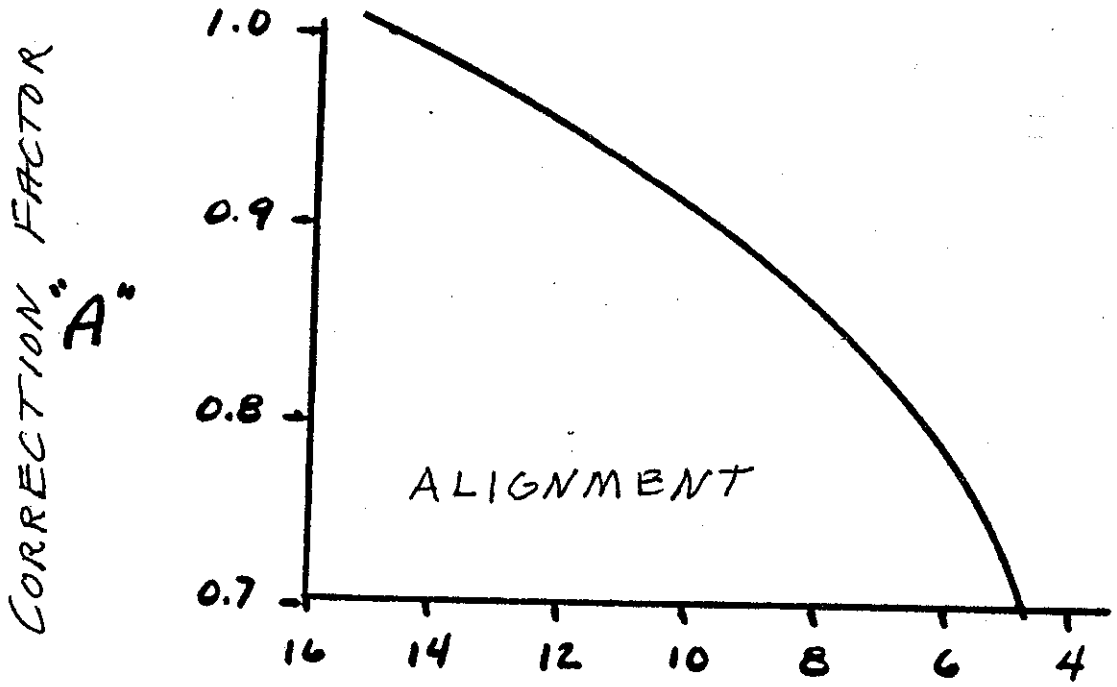
→ Coherent Material - use tractive power

BASIC VELOCITIES FOR COHERENT EARTH MATERIALS (v_b)

"V_b" for coherent Soils

look carefully CPI
use more than 1 sample

ALIGNMENT, CURVATURE



$$R/T = \frac{\text{Radius (curve) } ℄}{\text{Water Surface Width}}$$

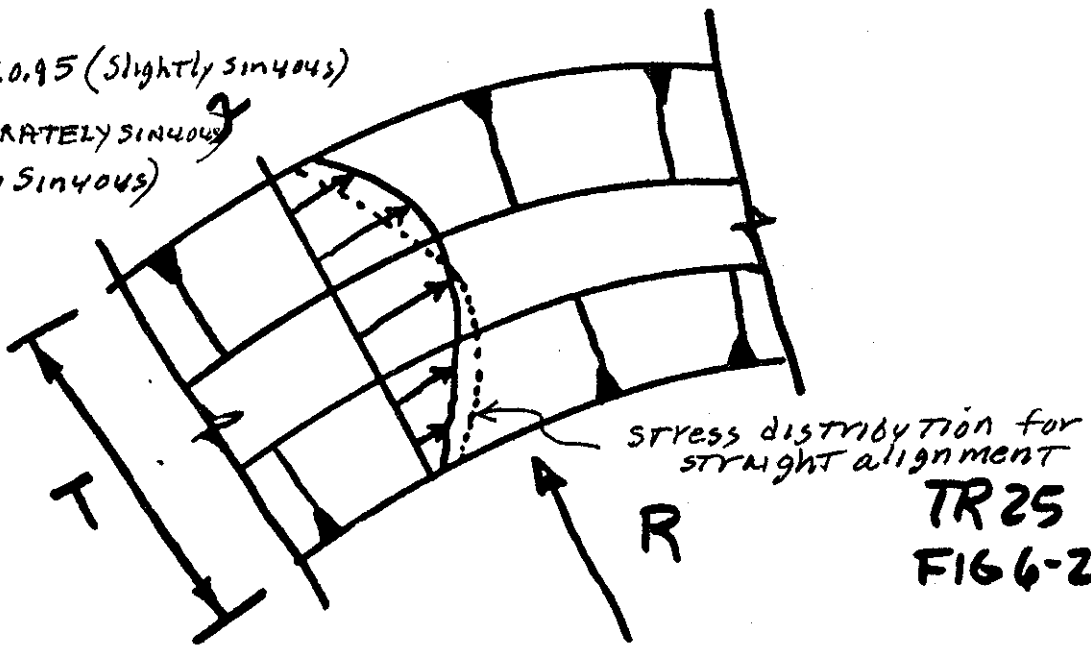
"A" for all conditions

- $A \leq 1.00$
- LAWE:

$V_{ALLOWABLE} = V_b \times 0.95$ (Slightly sinuous)

$= V_b \times 0.87$ (MODERATELY SINUOUS)

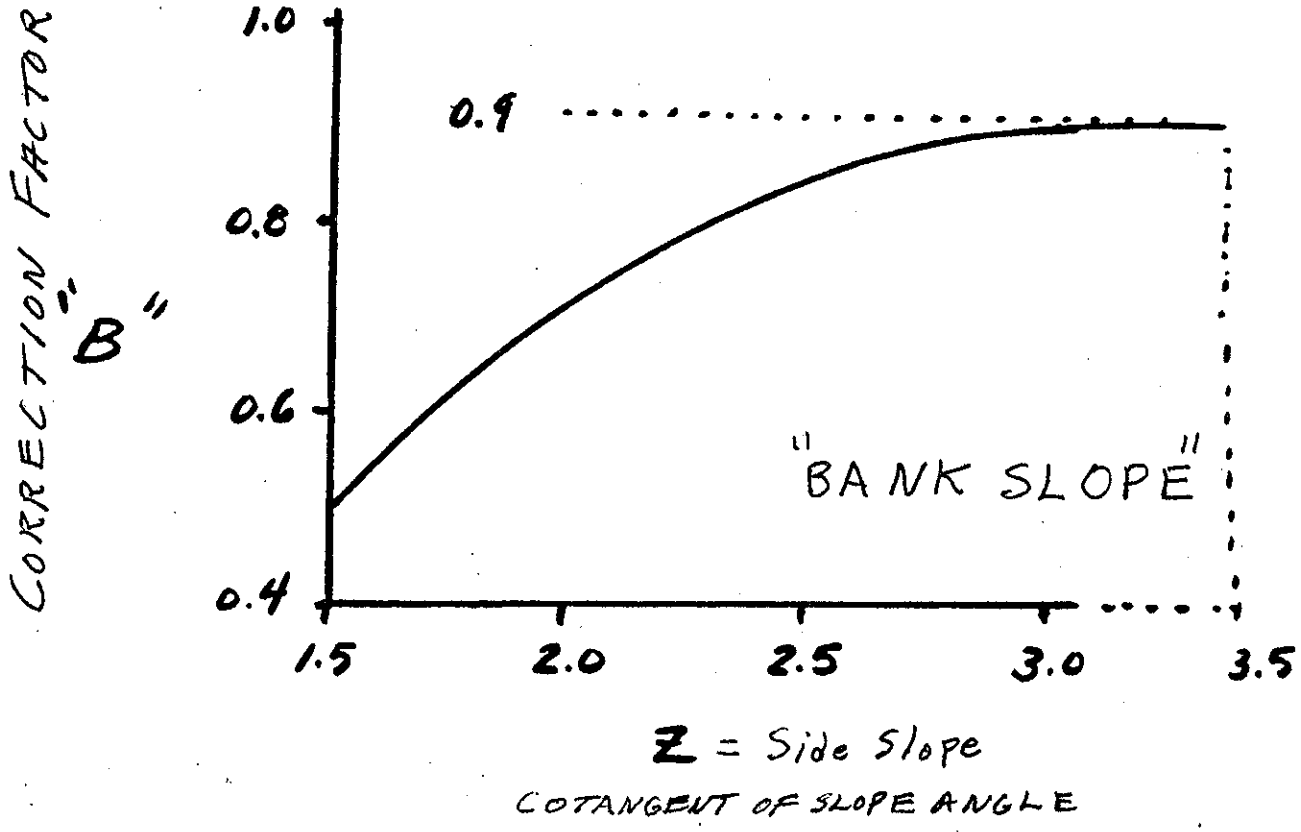
$= V_b \times 0.78$ (VERY SINUOUS)



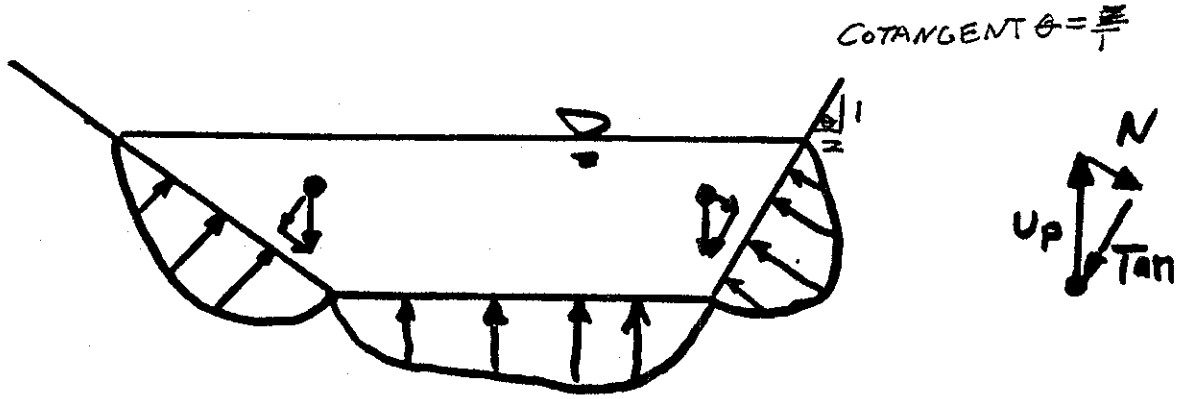
TR25
FIG 6-2

CORRECTION FACTOR B

BANK SLOPE, GRAVITY FORCE, SLIDE DOWN EFFECT



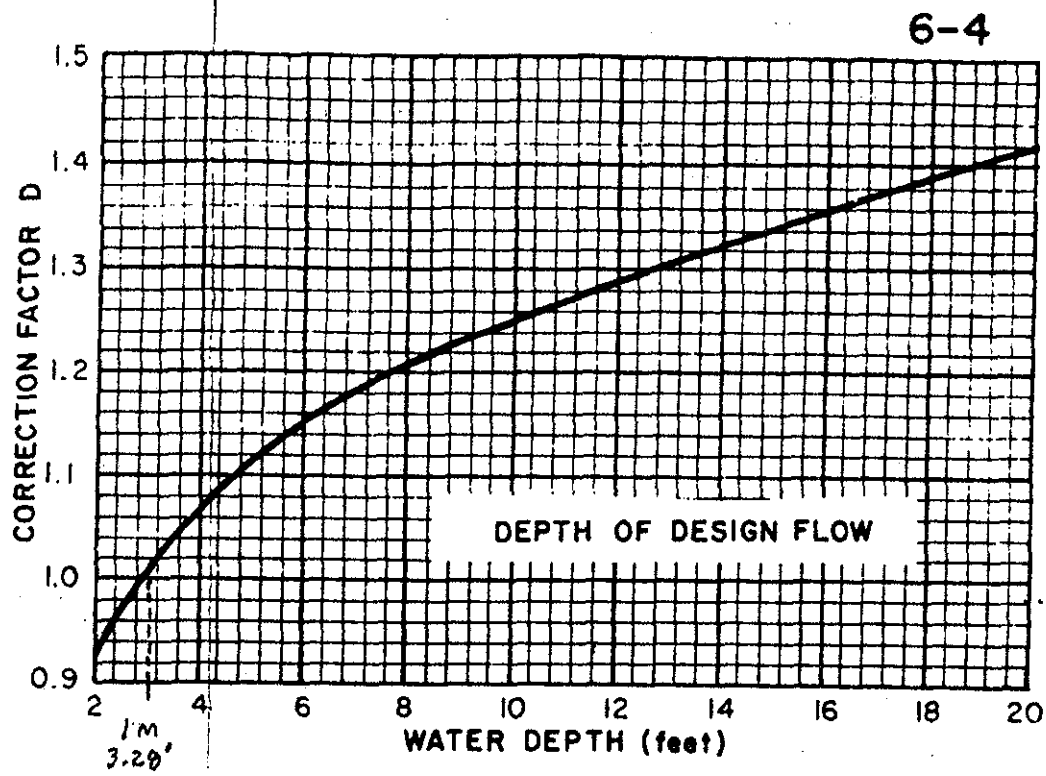
"B" for <u>Discrete particles</u> (only)



TR 25
FIG 6-2

CORRECTION FACTOR D

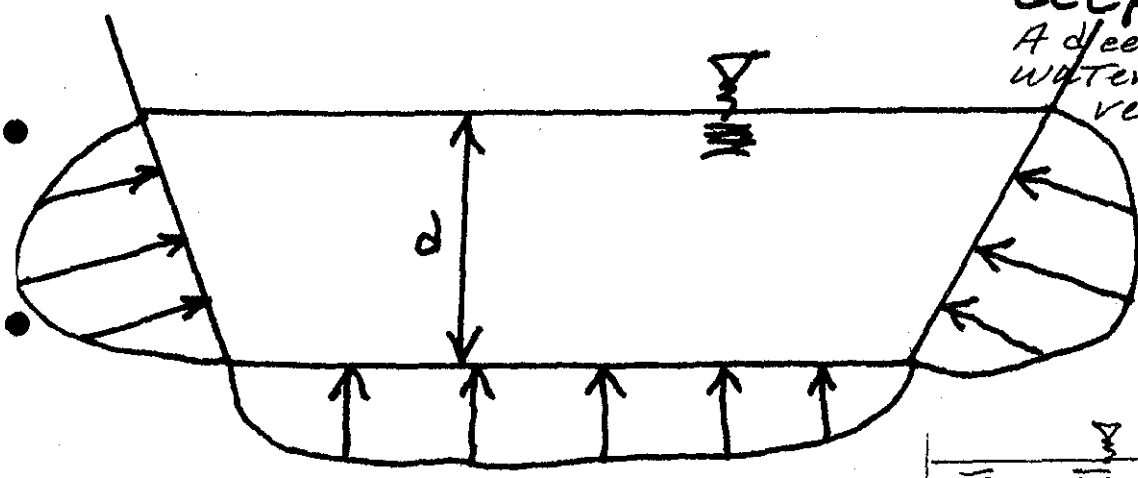
DEPTH OF FLOW

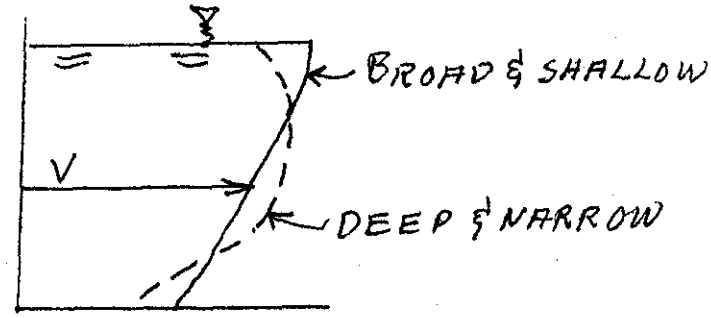


"D" for all conditions

Stress distribution more favorable with deeper water

A deep channel can convey water at a greater area velocity than a shallow channel



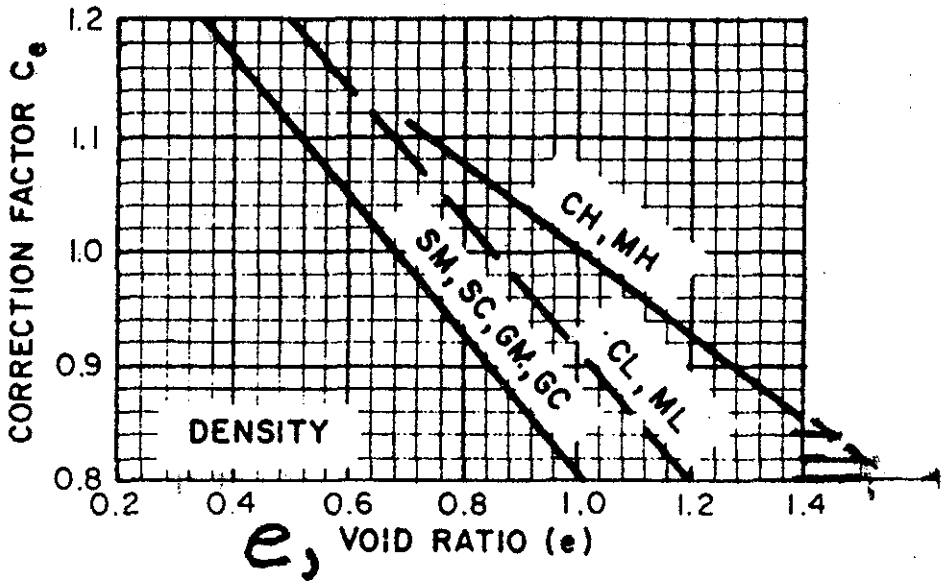


TR 25 - FIG 6-2

CORRECTION FACTOR Ce

VOID RATIO, DENSITY CORRECTION FACTOR

Ce



FOR COHERENT MATERIALS (ONLY)

$$e = \frac{G_s \gamma_w}{\gamma_d} - 1$$

γ_d = dry density = $\frac{\gamma_{wet}}{1+w}$

G_s = Specific gravity of solids?

γ_w = 62.4 (pcf)

$se = wG_s$

- DEALS WITH CLOSENESS OR PACKING RATHER THAN THE DENSITY
- DATA FROM AN UNKNOWN RUSSIAN AUTHOR.
- Ce APPLIES TO ALL SOIL MATERIALS EXCEPT CLEAN SANDS AND GRAVELS (CONTAINING < 5% PASSING #200)

CORRECTION FACTOR F

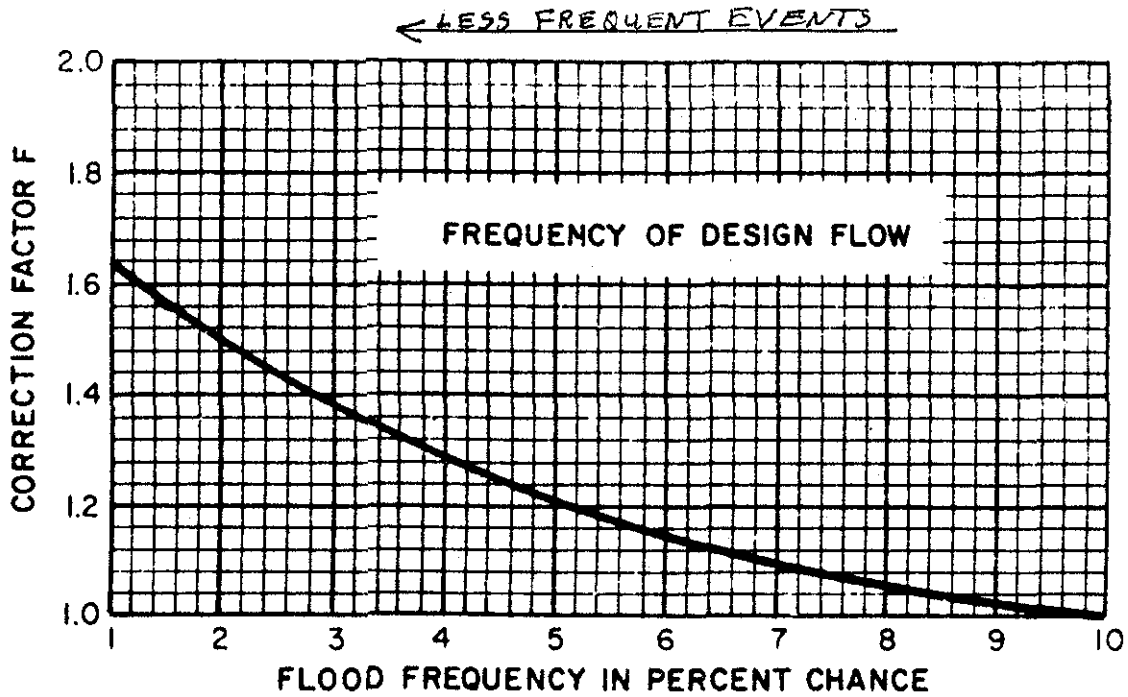
FREQUENCY OF DESIGN FLOW

(FOR COHERENT MASS PARTICLES ONLY)

NOTES -

1.0 "IN NO CASE SHOULD THE ALLOWABLE VELOCITY BE EXCEEDED WHEN THE 10% CHANCE DISCHARGE OCCURS, REGARDLESS OF THE DESIGN FLOW FREQUENCY.

THUS - A CHANNEL COULD BE STABLE AT THE DESIGN FLOW AND NOT STABLE AT A FREQUENCY = 10%



T_R = 100 yr T_R = 20 yr T_R = 10 yr

T_R = RETURN PERIOD IN YEARS

"F" for cohesive soils

Flood frequency risk factor

FOR (FREQ > 10%), F = 1.0.

TR 25 FIG 6-2

ALLOWABLE VELOCITY

Example Prob 1

Given: Channel $b = 15'$, $z = 2$, $d = 4.5'$
coarse sandy gravel, $D_{75} = 7mm = 1/4''\pm$
fine sediment conc. = 25,000 PPM, 10% Q.
incoming sediment > 1mm is very small.

Determine: Allowable Velocity

1. sandy gravel will perform as discrete particles therefore use V_{basic} for Discrete particles.

2. Sed. conc. > 20,000 PPM and $D_{75} > 0.4$ mm
therefore use:

$$V_{all} = V_{basic} \times D \times A \times B$$

$V_{basic} = 4.8$ fps, $D = 1.09$, $A = 1$, $B = 0.7$
(F not needed)

$$V_{all} = 4.8(1.09)(1)(0.7) = \underline{\underline{3.66 \text{ fps}}}$$

3. What is V_{all} if a 200' radius curve is installed?

Water surface Top $= T = b + 2zd = 15 + 2(2)(4.5) = 33'$

$$\frac{R}{T} = \frac{200}{33} = 6.06$$

therefore from Alignment Curve $A = 0.78$

$$V_{all} = 4.8(1.09)(1)(0.7)(0.78) = \underline{\underline{2.85 \text{ fps}}}$$

ALLOWABLE VELOCITY

Example Prob. 2

Given: Channel $b = 15'$, $Z = 2$, $d = 4.5'$
sandy clayey gravel, PI ≥ 15
$\gamma_d = 122$ pcf., fine sediment concentration very low (< 1000 PPM)
10% Q, incoming sediment > 1mm is very low.
G_s = Specific gravity = 2.7

Determine: Allowable velocity

1. sandy clayey gravel will perform as a coherent material (GC)

2. PI > 10 Therefore V_{basic} for coherent soils

$$V_{allow} = V_{basic} \times D \times A \times F \times C_e$$

$V_{basic} = 4.49$ fps, $D = 1.09$, $A = 1$, $F = 1$, $C_e = 1.18^{(-)}$

$$e = \frac{G_s \gamma_w}{\gamma_d} - 1 = \frac{2.7(62.4)}{122} - 1 = 0.38$$

$$V_{all} = 4.49(1.09)(1)(1.18) = \underline{\underline{5.78 \text{ fps}}}$$

Example Prob 3

Given: An open channel through a reach of GW soils has a $D_{75} = 18mm$.

The hydraulic design requires a depth of 3', $z = 3$ and a bottom width of $b = 20'$ for the 10 year design discharge of $Q = 325$ CFS.

The channel has no curves

Find: According to allowable velocity criteria is the reach stable?

a) For the present sediment laden flow with Conc. = 20,000 PPM.

b) For sediment free water when a structure, urban development, and erosion control measures reduce fine sediments to < 1000 PPM.

Solution:

a) 1. GW soils will act like discrete particles.

For sediment laden flow and $D_{75} > 0.4mm$

Use $V_{all} = V_{basic} \times D \times A \times B$

$V_B = 6.3$ fps, $D = 1.01$, $A = 1.0$, $B = 0.86$

$V_{all} = (6.3)(1.01)(1.0)(0.86) = 5.47$ fps.

$$V_{actual} = \frac{Q}{A} = \frac{325}{20(3)+3(3)^2} = \frac{325}{87} = 3.74 \text{ fps} < 5.47 \text{ fps}$$ OK

b) 1. For sediment free flow ($D_{75} > 2mm$)

Use $V_{all} = V_{basic} \times D \times A \times B$

$V_b = 4.2$ fps, $D = 1.01$, $A = 1.0$, $B = 0.86$

$V_{all} = 4.2(1.01)(1.0)(0.86) = 3.65$ fps

$V_{act} = 3.74 > 3.65$ fps $\therefore$ No Good

ALLOWABLE VELOCITY

Example Prob 4

Given: channel with depth = 3', z = 3, bottom width b = 20'
10 year design discharge = 325 cfs
channel in CL soils having a PI = 18

Find: Is the reach stable for
a) Sediment laden flow with Conc = 20,000 ppm
b) Sediment free flow after developments are in.

Assume $\gamma_d = 105$ pcf and $G_s = 2.7$

Solution - Channel material is a coherent material and PI > 10 therefore use

$$V_{all} = V_{basic} \times D \times A \times F \times C_e$$

a) Sediment laden

$V_b = 5.2$, $D = 1.09$, $A = 1$, $F = 1.0$, $C_e = 1.14$

$$e = \frac{G_s \gamma_w}{\gamma_d} - 1 = \frac{2.7(62.4)}{105} - 1 = 0.6$$

$$V_{all} = (5.2)(1.01)(1)(1)(1.14) = 5.99 \text{ fps}$$

$$V_{actual} = \frac{Q}{A} = \frac{325}{20(3)+3(3)^2} = \frac{325}{87} = \underline{\underline{3.74 \text{ fps} < 5.99 \text{ fps}}} \quad \underline{\underline{ok}}$$

b) Sed free

$V_b = 3.7$ fps

$$V_{all} = 3.7(1.01)(1)(1)(1.14) = 4.26 \text{ fps}$$

$$\underline{\underline{3.74_{act} < 4.26_{all}}} \quad \underline{\underline{ok}}$$

ALLOWABLE VELOCITY

Example Prob 5

Given: channel downstream of principal spillway of a Dam. Channel materials are SP's and SM's. D_{75} varied from 2.5mm to 4mm, $Z=2$, trap efficiency of the dam is high, there are few fines in any of the sediment yielding areas in the drainage area. At the 1% design flow, $Q = 140$ CFS, $d = 3.5'$ and $V = 2.5$ fps; at the 10% flow $Q = 30$ cfs, $d = 1.0'$ and $V = 2$ fps.

Find: (a) check the reach for stability for each flow.
(b) An alternate location for the channel would place it in SC materials, $PI = 20$, $\gamma_d = 110$ pcf and $G_s = 2.7$. Is this location preferable?

Solution

(a) 1. High trap efficiency of dam and low fines in sediment producing areas suggests very little 1mm sediment load and < 1000 ppm fines.

2. Described channel soils will act as discrete particles. D_{75} range indicates the use of

$$V_{all} = V_{basic} \times D \times A \times B$$

1% $V_b = 2.0$ fps (least D_{75}), $D = 1.04$, $A = 1.0$, $B = 0.7$

$V_{all} = (2)(1.04)(1.0)(0.7) = 1.46$ fps < 2.5 fps$_{act}$ - NO Good

10% No difference - 1.46 fps < 2.0 fps$_{act}$ NO Good
(No correction is made for F)

b) $V_{all} = V_b \times D \times A \times F \times C_e$

$$e = \frac{G_s \gamma_w}{\gamma_d} - 1 = \frac{2.7(62.4)}{110} - 1 = 0.53$$

1% $V_{all} = (4.0)(1.04)(1)(1.64)(1.09) = 7.44$ fps

7.44 fps > 2.5 fps OK

10% $V_{all} = 4.0(0.93)(1)(1.00)(1.09) = 4.05$ fps

4.05 fps > 2 fps OK

U.S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
333 WALLER AVENUE
LEXINGTON, KENTUCKY 40504

Low Cost Structures for Limited Resource Producers

Practice: Grade Stabilization Structure 410

Wire Panel Grade Control Structure

Utilization: Used to lower surface water at side inlet locations and prevent erosion problems commonly referred to as "head cuts".

Advantages:

1. Relatively inexpensive.
2. Availability of materials.
3. Installation can be achieved without heavy equipment or experienced personnel.

Limitations:

1. Net drop of 4 feet or less.
2. 60 cfs capacity or less.

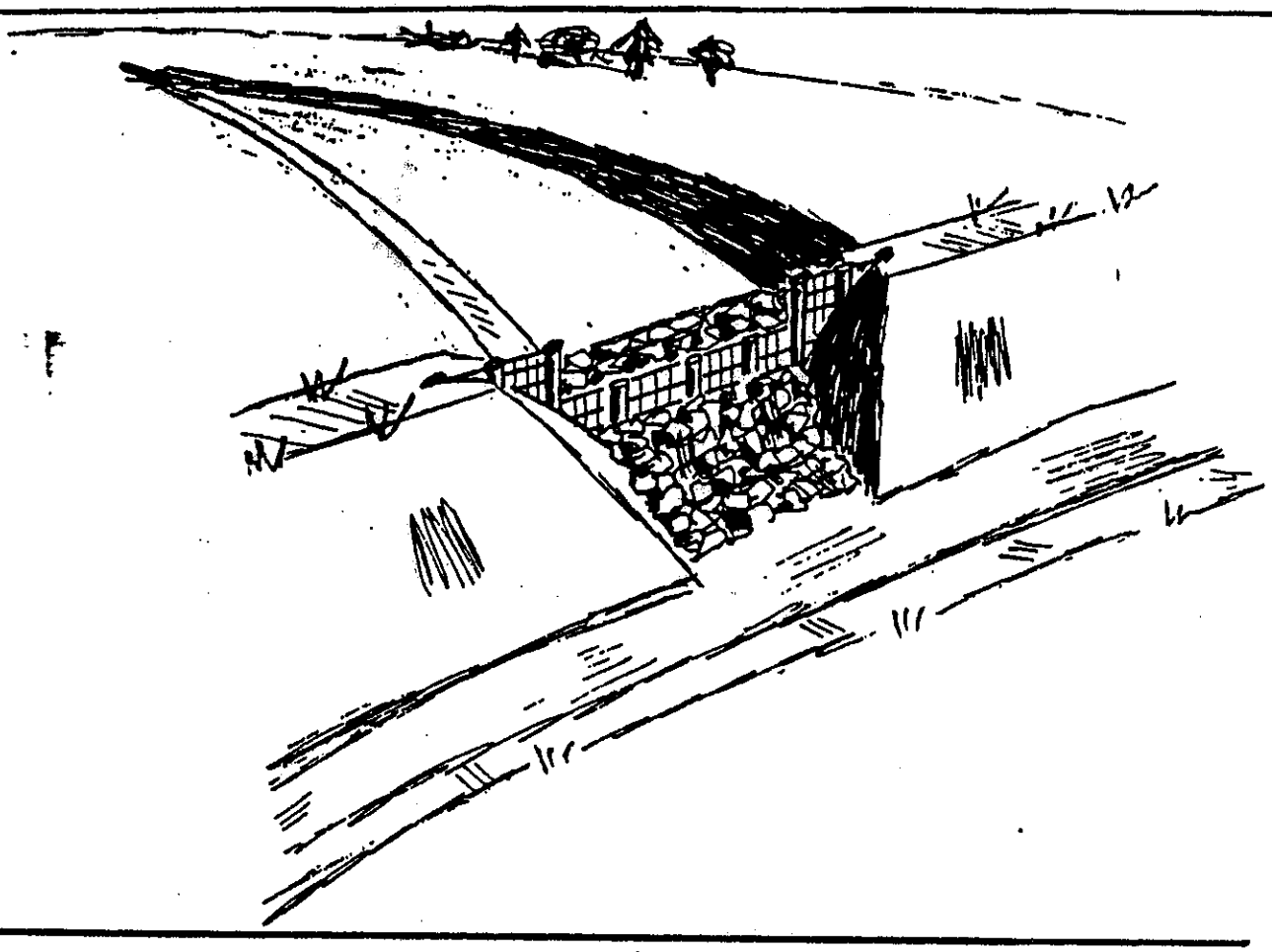
Construction Materials:

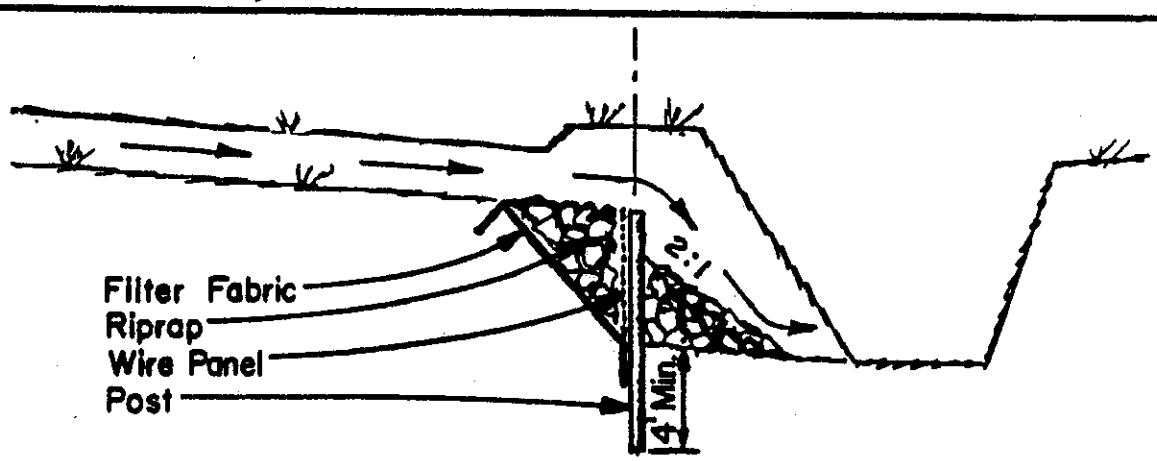
1. Post, cut pipe or other material locally available.
2. Wire panels commonly referred to as hog or cattle panels. These are readily available at most farm supply outlets.
3. Filter fabric - non-woven needle punched available at most construction suppliers.
4. Riprap

Research: No research has been conducted on this type of structure other than simple stability analysis. Approximately 100 of these have been successfully installed during the past 2-3 years.

For Further Information, Contact:

Joe Davis, SCE
Soil Conservation Service
333 Waller Avenue
Lexington, KY 40504





Location

Field no., description or sketch

WIRE PANEL GRADE CONTROL STRUCTURE

Cooperator _____

Approved _____ Date _____

Workshop Prob

Given:

A channel is proposed in an area of medium sands to carry a bank full discharge of 200 cfs. n from NEH 5, Supp B is 0.024; side slopes will be 3:1

Determine:

1. Channel geometry that will satisfy regime and hydraulics according to SCS modified regime. Where will you set So and why?

2. Recheck with a higher flow of 1500 cfs. For this discharge n increases to 0.028.

   Where now will we set the slope and why?

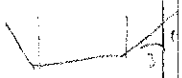
Workshop Problem Modified Regime

1. $P = 3.30(200)^{0.512} = 49.73$ ft.
2. $R = 0.37(200)^{0.361} = 2.51$ ft
3. $A = 1.22(200)^{0.873} = 129.50$ ft^2
4. $V = 200/129.50 = 1.61$ fps
5. $d = 1.23R = 1.23(2.51) = 3.09$ ft
6. $F = V/\sqrt{gd} = 1.61/\sqrt{32.2 \times 3.09} = 0.161$ ok

7. $W = 0.9P = 0.9(49.73) = 44.76$ ft

$W = 0.92 W_t - 2.0 \Rightarrow W_t = (W + 2.0)/0.92$

total width $= (44.76 + 2)/0.92 = 50.82$



for 3:1 side slopes:

$b = 50.82 - 6(3.09) = 32.28$ ft $\Rightarrow 32.3$ ft.

(d)

8. $V = C_q (R^2 S_0)^{1/3}$ $\quad S_0 = (V/C_q)^3/R^2$

$= (1.61/13.9)^3/2.51^2 = 2.47 \times 10^{-9}$

9. $P = b + 2d\sqrt{z^2+1} = 32.3 + 2(3.09)\sqrt{(3)^2+1} = 51.82$ ft

$A = bd + zd^2 = 32.3(3.09) + 3(3.09)^2 = 128.4$ ft^2

$R = A/P = 128.4/51.82 = 2.48$ ft.

$AR^{2/3} = (128.4)(2.48)^{2/3} = 235$

$Q = \frac{1.49}{n} AR^{2/3} S_0^{1/2} \qquad S_0 = \left[\frac{Qn}{1.49} \frac{1}{AR^{2/3}}\right]^2$

$S_0 = \left[\frac{200(0.024)}{1.49} \frac{1}{235}\right]^2 = 1.88 \times 10^{-4}$ ft/ft

Prob. 1. Solution

$Q = 200$ CFS $\quad n = 0.024 \quad z = 3$

1. $P = C_1 Q^{0.512} = (3.3)(200)^{0.512} = 49.73$

2. $R = C_2 Q^{0.361} = (0.37)(200)^{0.361} = 2.51$

3. $A = C_3 Q^{0.873} = (1.22)(200)^{.873} = 124.5$

4. $V = Q/A = 200/124.5 = 1.61$

5. $S_o = \frac{V^3}{C_4^3 R^2} = (1.61)^3/(13.9)^3(2.51)^2 = 0.000245$

6. $d = 1.23 R = 1.23 \times 2.51 = 3.09$

7. $F = V/\sqrt{gd} = 0.16 < .3$ O.K.

8. $W = 0.9 P = 0.9 \times 49.73 = 44.76$

9. $W_T = W + 2.0/0.92 = 44.76 + 2.0/0.92 = 50.8$

10. $b = W_T - 2zd = 50.8 - (2)(3)(3.09) = 32.26$ use 32

11. $P = b + 2d\sqrt{z^2+1} = 32 + (2)(3.09)[(3)^2+1]^{1/2} = 51.54$

12. $A = bd + zd^2 = (32)(3.09) + (3)(3.09)^2 = 127.5$

13. $S = Q^2 n^2 / 2.2 A^2 R^{4/3} = (200)^2(.024)^2/(2.2)(127.5)^2(2.51)^{4/3} =$

$.000193$

$$R = \frac{A}{P} = \frac{127.5}{51.54} = 2.47$$

PROBLEM 2 SOLUTION

1/ $P = (3.3)(1500)^{.512} = 139.53$

2/ $R = (0.37)(1500)^{.361} = 5.18$

3/ $A = (1.22)(1500)^{.873}$ 722.92

4 $V = Q/A = 1500/$ $= 2.07$

5. $S_o = (2.07)^3/(13.9)^3(5.18)^2 = 0.00012$

6. $d = 1.23 \times 5.18 = 6.37$

7. $F = 2.07/(32.2 \times 6.37)^{1/2} = 0.14.6 < 0.3$ ok

8. $W = 0.9 \times 139.53 = 125.58$

9. $W_r = 125.58 + 2.0/0.92 = 138.67$

10. $b = 138.67 - (2)(3)(6.37) = 100.45$ use 100

11. $P = 100 + (2)(6.37)[9+1]^{1/2} = 140.29$

12 $A = (100)(6.37) + (3)(6.37)^2 = 758.73$

13. $R = A/P = 758.73/140.29 = 5.41$

14. $S = (1500)^2(.028)^2/2.2 \times 758.73^2 \times 5.41^{4/3} = 0.00015$

But using 0.00015 & figure you would have capacity & some aggragation

TRAPEZOIDAL CHANNEL

Given:	Runoff	Q =	55 cfs
	Grade	=	3.0 percent
	Side slope	=	3:1
	Vegetative cover	=	Kentucky bluegrass
	Condition of vegetation		
	Good stand-mowed (3″ — 4″)	=	D curve retardance } TABLE #4
	Good stand-headed (6″ — 12″)	=	C curve retardance } TABLE #4
	Permissible velocity	=	5 ft./sec.

Step 1 - Stability - D curve retardance

Q = 55 cfs.

V_{max} = 5 ft./sec.

$$A = \frac{Q}{V_{max}} = \frac{55}{5} = 11 \text{ sq. ft.}$$

Try Bottom Width (b) = 12 feet

$A = bD + zD^2$ AREA OF A TRAPEZOID

$11 = 12D + 3D^2$

Solve for D by use of the quadratic equation.

$$ax^2 + bx + c = 0$$

$$x = \frac{-b \pm \sqrt{b^2 - 4ac}}{2a}$$

$$D' = \frac{-12 \pm \sqrt{12^2 - 4(3)(-11)}}{2(3)}$$

$$= \frac{-12 + 16.61}{6} = \frac{4.61}{6}$$

$$= 0.77 \text{ feet}$$

Hydraulic Radius

$$r = \frac{\text{area } (A)}{\text{wetted perimeter } (P)} = \frac{bd + zD^2}{b + 2D\sqrt{z^2 + 1}}$$

$$r = \frac{12(0.77) + 3\,(0.77^2)}{12 + 2(0.77)\sqrt{3^2 + 1}}$$

$$r = \frac{9.24 + 1.78}{12 + 4.87}$$

$$r = \frac{11.02}{16.87} = 0.65$$

$$Vr = 5(0.65) = 3.25$$

FIG 13

for $Vr = 3.25$ and D retardance, $n = 0.04$

$$V = \frac{1.486}{n}\, r^{2/3}\, s^{1/2} \text{ (Manning's equation)}$$

where n is the roughness coefficient, r is the hydraulic radius, and s is the hydraulic gradient.

$$V = \frac{1.486}{0.04}\,(0.65^{2/3})\,(.03^{1/2}) = 4.83 \text{ ft./sec.}$$

— Bottom Width = 10 feet

$11 = 10D + 3D^2$

$D = 0.87$ feet

$r = 0.71$

$Vr = 3.55$

$n = 0.04$ (see figure I-14)

$V = \frac{1.486}{n} r^{2/3} S^{1/2} = 5.15$ which is greater than V_{max}

Therefore, select design bottom width = 12 feet

Velocity = 4.83 ft./sec. for D retardance

Depth = 0.8 feet

Step 2 - Capacity (C retardance).

Try $D = 0.9$ feet

$$A = (12)(0.9) + 3(.9^2) = 13.23$$

$$r = \frac{A}{P} = \frac{13.23}{12+2(.9)\sqrt{3^2+1}} = 0.75$$

Assume $V = 4.4$ ft./sec.

$$Vr = (4.4)(0.75) = 3.30$$

For $Vr = 3.30$ and C retardance, $n = 0.046$

$$V = \frac{1.486}{0.046}(0.75^{2/3})(.03^{1/2}) = 4.62 \text{ ft./sec.}$$

Since 4.62 ft./sec. is greater than the assumed V,

assume $V = 4.6$ ft./sec.

$$Vr = (4.6)(0.75) = 3.45$$

$$n = 0.046$$

$$V = \frac{1.486}{.046}(0.75^{2/3})(.03^{1/2}) = 4.62 \text{ ft./sec.}$$

This is close enough.

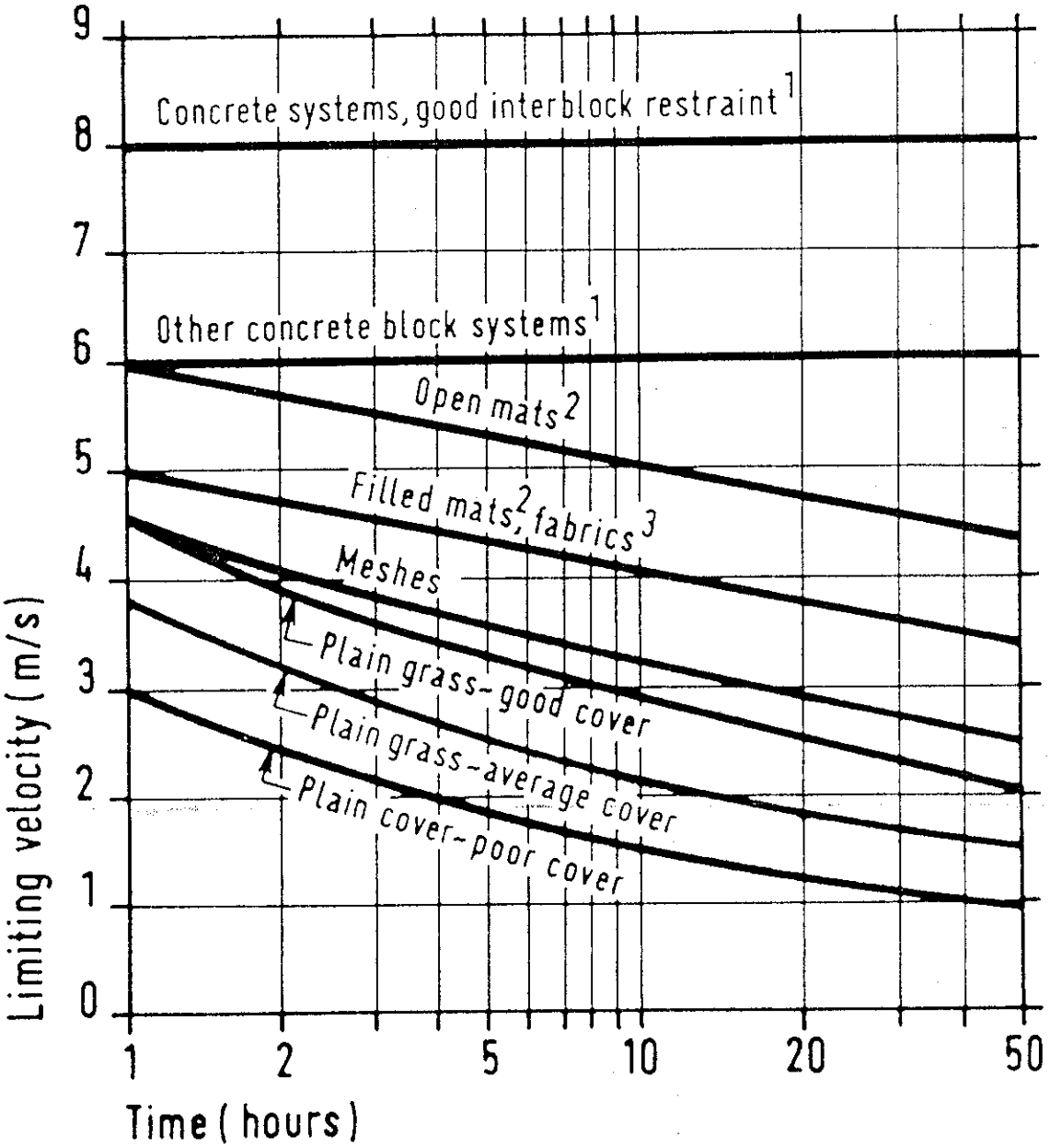
Therefore, dimensions and velocities are as follows:

Bottom width = 12 feet

Side slopes = 3:1

For D retardance - V = 4.83 ft./sec.
D = 0.8 feet

For C retardance - V = 4.62 ft./sec.
D = 0.9 feet



Recommended limiting values for erosion resistance of plain and reinforced grass

Notes:
1. Minimum superficial mass 135 kg/m², see Section 4.3.3 for other criteria.
2. Minimum nominal thickness 20 mm.
3. Installed within 20 mm of soil surface, or in conjunction with a surface mesh.
4. See Section 4.3.2 for other criteria for geotextile reinforcement.
5. These graphs should only be used for erosion resistance to unidirectional flow. Values are based on available experience and information at date of this report.
6. All reinforced grass values assume well-established, good grass cover.
7. Other criteria (such as short-term protection, ease of installation and management, susceptibility to vandalism, etc) must be considered in choice of reinforcement.

Karl Larson

UNITED STATES DEPARTMENT OF AGRICULTURE

SOIL CONSERVATION SERVICE

HANDBOOK OF CHANNEL DESIGN FOR SOIL AND WATER CONSERVATION

Prepared By

Stillwater Outdoor Hydraulic Laboratory

Stillwater, Oklahoma

In cooperation with the
Oklahoma Agricultural Experiment Station

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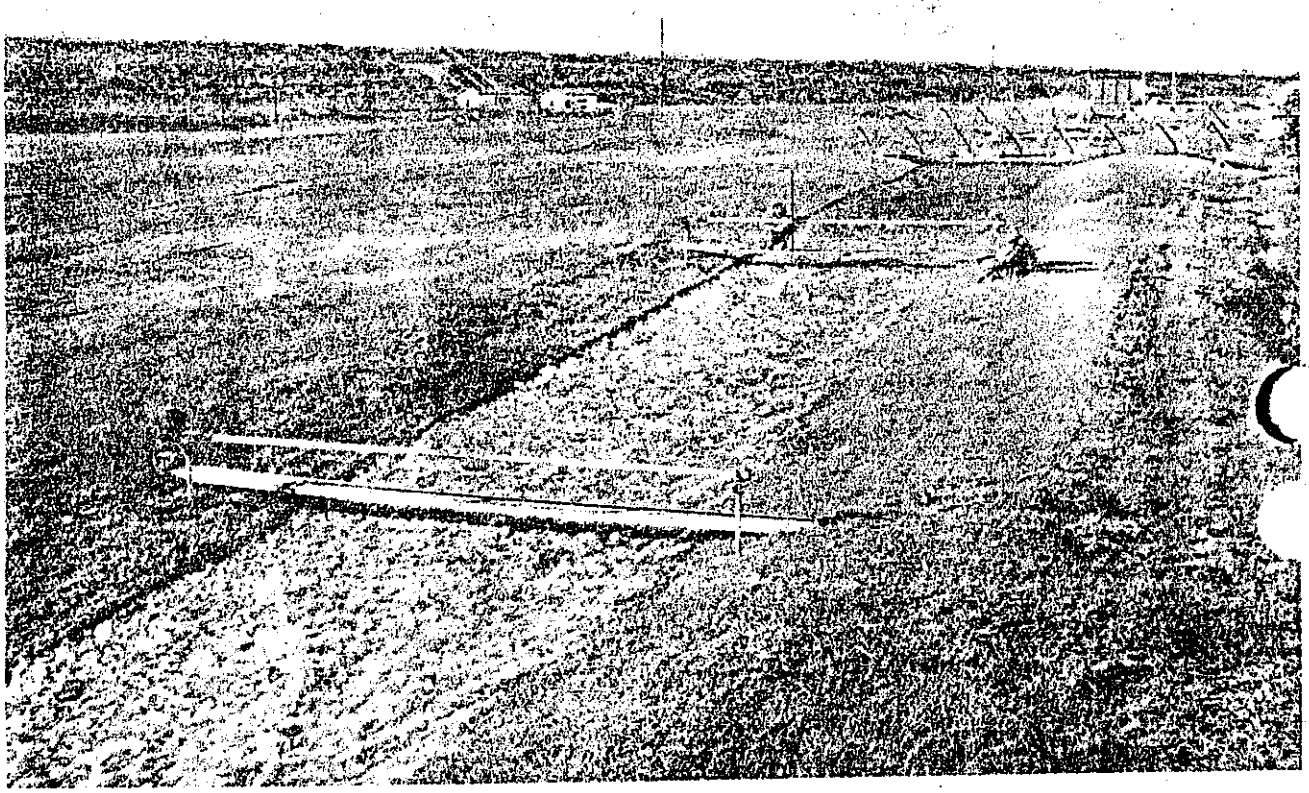

Large trapezoidal channel lined with bermudagrass being tested at the Stillwater Outdoor Hydraulic Laboratory

HANDBOOK OF CHANNEL DESIGN FOR SOIL AND WATER CONSERVATION[1]

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Introduction

The factors to consider in open-channel flow and stability, together with graphical methods for designing conservation channels, are presented in this handbook. The purpose is to furnish technicians of the Soil Conservation Service with the most recent and complete information to aid them in the design of channels to be lined with vegetation.

Complete graphical methods are presented that deal with retardance to flow offered by vegetation as a function of the depth and velocity of flow. In addition, as an aid in designing channels where the retardance may be considered constant, e.g., one with a concrete lining, a simple nomographic solution of the Manning formula has been developed. The dimensions of trapezoidal (side slopes 1:1, 1.5:1, 2:1, 2.5:1, 3:1, 4:1, 5:1, and 6:1), triangular, and parabolic channels are determined graphically. The solution for parabolic channels permits the determination of the width for any depth and the approximate side slope at each depth.

The appendix contains pertinent experimental results from which the degrees of vegetal retardance were determined and recommendations based. A list of references is included.

The method of vegetal-channel design was developed by the Stillwater Outdoor Hydraulic Laboratory of the Soil Conservation Service. The stability recommendations and capacity-design charts for completely vegetated channels are based on experimental results from this laboratory, from its former location near Spartanburg, S. C., and from the Oklahoma Agricultural Experiment Station. Information concerning Kentucky bluegrass was obtained from the Soil Conservation Experiment Station at Columbia, Mo.

Although developed primarily for use in the Western Gulf Region, this handbook contains values of Manning's n, general design charts, and graphical solutions that are applicable in all regions.

Open-Channel Flow

General

An open channel is defined as any conduit in which water flows with a free-water surface. Rivers, canals, and uncovered flumes are open channels. Pipes, drains, sewers, etc., act as open channels when flowing partially full. Following are definitions of a number of common terms describing flow in open channels:

Steady flow--When the discharge or rate of flow remains constant at a cross section, the flow is described as steady. Flows in drainage channels and rivers tend to be steady except during surface-runoff periods.

Unsteady flow--When the discharge is changing at a cross section, the flow is unsteady. Intermittent runoff into terrace outlets, meadow strips, diversion channels, and pond spillways is generally unsteady.

Uniform flow--When the flow is steady and the mean velocity is the same at each succeeding cross section, the flow is uniform. The channel must have a constant cross section and slope. In designing ordinary vegetation-lined channels the flow is considered to be uniform at the maximum discharge.

Nonuniform flow--When the mean velocity changes from cross section to cross section, the flow is nonuniform. Flow in the upper portions of steep channels near the entrance is nonuniform. In a channel of changing cross section the flow is nonuniform.

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[1] Early in 1946 it was determined that sufficient information was available from experiments on vegetation-lined channels (made by the Soil Conservation Service at Spartanburg, S. C., from 1937 to 1941 and at Stillwater, Okla., since 1941) to permit the development of a handbook pertaining especially to the design of channels lined with vegetation. This handbook was prepared by Vernon J. Palmer, Agricultural Engineer, Research Project OK-R-3, assisted by William P. Law, Jr., formerly of the Soil Conservation Service.

In this revised edition, the information contained in the earlier publication has been supplemented by data, graphical methods, and design charts useful in the design of vegetated channels having extremely low retardance values. This material was prepared by W. O. Ree, Hydraulic Engineer, Division of Drainage and Water Control, Soil Conservation Service--Research.

The Stillwater Outdoor Hydraulic Laboratory, Stillwater, Okla., was transferred to Agricultural Research Service of the U. S. Department of Agriculture Dec. 1, 1953.

Notation

Q - Rate of discharge or flow in cubic feet per second.
A - Cross-sectional area of the flow in square feet.
V - Velocity of flow (mean) in feet per second.
n - Coefficient of retardance used in flow formulas. In vegetation-lined channels this coefficient contains not only the effect of lining roughness but also the effect of irregularities in bed surface, channel shape, slope, and alinement on the flow. The effect of blocking out a portion of the cross section by vegetation is also included.
R - Hydraulic radius in feet. It is the cross-sectional area A divided by the wetted perimeter.
S - Slope of energy gradient in feet per foot. In vegetation-lined channels where the flow is considered to be uniform, the bed slope in feet per foot of length may be used. The use of vertical drop and slope length in computing S is satisfactory in most vegetated channels.
d - Depth of flow in feet.
D - Depth of channel in feet after freeboard is added.
t - Width of water surface when the water is at depth d.
z - Side slope. Ratio of horizontal to vertical.
b - Bottom width of a trapezoidal channel in feet.

Character of Flow

Flow in conservation channels is turbulent. Turbulence exists when the direction and magnitude of the velocity at any point within a fluid varies irregularly with time. Considerable energy may be expended in this action. Eddying and "boiling" are visible forms of energy loss. These disturbances in the fluid are produced and maintained largely by roughness and irregularities of the bed and oscillation and sometimes severe whipping of stems and leaves of vegetation by the flow. The more violent the disturbances, the greater is the retardance to the flow and the greater are the forces acting to scour the bed.

The velocity V, as computed from Q/A or estimated from flow formulas, is the mean velocity of all components parallel to the axis of the channel. These velocity components in any vertical range from a maximum near the surface to zero at the bed. With vegetation, the velocity distribution from surface to bed may be very nonuniform, increasingly so as the cover becomes taller, stiffer, and "bunchier."

In vegetation-lined channels, there are wide differences in velocity throughout a cross section. The water at the edges of the channel will be flowing through the vegetation at low velocities. In the deeper portions of the channel, where the vegetation has been bent over and submerged, resistance to flow will be less and velocities will be higher. The least resistance will be encountered where the flow is deepest. The velocity as ordinarily determined, is the mean for the entire cross section. It is apparent that channel shape affects velocity distribution. For the san mean velocity, the maximum velocity in the center of a triangular channel could be much higher than in a trapezoidal or parabolic channel. For this reason, a reduction in permissible velocities with triangular channels is desirable.

Flow in field channels lined with vegetation generally has a rough water surface. The more irregular the bed and vegetal growth and the higher the velocity, the greater is the surface roughness. The amount of aeration is also dependent upon these factors. A freeboard (vertical distance from the maximum water surface to the channel berm) allows for these conditions as well as for differences between estimated and actual discharge and capacity. Freeboards of 0.5 foot are commonly used for vegetation-lined channels.

Estimating Velocity

The design of conservation channels will be based on a mean velocity determined from the Manning formula: $V = \frac{1.486}{n} R^{2/3} S^{1/2}$. Refer to the previous section, on Notation for an explanation of the terms in this expression.

The coefficient known as Manning's n contai the effect of all the retarding influences on th flow. Kutter's n, which is used in an older, mo complicated expression known as the Kutter formula and is no more accurate than Manning's for estimating the velocity, is equal to Manning n only for a hydraulic radius of 3.28 feet. For other values of hydraulic radii, they are not identical and should not be used interchangeably. Values of Manning's n for various kinds of open channels are contained in table 1 in the Appendix.

When the channel is to be lined with vegetation, the design problem is complicated by a roughness or retardance element that changes its general position and oscillates continuously in the flow to an extent that is dependent upon the depth and velocity. To aid in designing channels of this type, a criterion has been developed for estimating n that considers the depth and velocity of the flow. A subsequent section presents this criterion together with a simple, complete, and graphical method of application.

The Manning formula will apply to uniform flow in channels. In designing ordinary vegetation-lined channels, this condition of uniform flow can be assumed to exist. For some steep channels and all channels with artificial linings (example: concrete, sheet metal, wood), careful consideration must be given to (1) inlet and outlet design and their effect on the flow in the channel, (2) changes in grade and alinement, and (3) changes in cross-sectional shape. Recourse to King's "Handbook of Hydraulics" or other qualified texts on hydraulics is recommended.

Stability of Channels

Nonvegetal Channels

With channel linings of concrete, metal, wood, etc., stability against erosion is generally not a consideration unless abrasive material is being carried in high-velocity flow. In earth channels, bare and vegetation-lined, resistance against erosion by flowing water is of principal concern. The difficulty of providing adequate protection increases with increases in both velocity and bed slope.

To aid in designing canals and irrigation and drainage channels, the Special Committee on Irrigation Hydraulics of the American Society of Civil Engineers authorized Samuel Fortier and F. C. Scobey in 1926 to prepare a paper presenting what was known about permissible velocities in bare channels of mild slope. King's "Handbook of Hydraulics" contains their conclusions. The problem is complex, depending not only upon the bed material and velocity of flow but also upon the gradation of bed material, the presence or absence of colloidal material (which acts as a cementing agent) on the bed, the age of the canal (well-seasoned canals will withstand much higher velocities than new ones), the depth of flow, and whether or not the water is clear or is transporting colloidal silts or non-colloidal material. Permissible canal velocities after aging, as recommended by Fortier and Scobey, are presented in table 2. They have been specified to apply, in a general sense, to canals with long tangents (for sinuous channels, reduce values by 25 percent) and to depths 3 feet or less (for greater depths an increase of velocity of 0.5 foot per second is allowable). An additional limitation not stated is that they apply only to channels of mild slope. A slope of 0.2 percent is suggested as a reasonable upper limit.

Stability of Vegetation-Lined Channels

A common misconception has been that vegetation, bent over and completely submerged by flowing water, shingles the bed and forms a protective shield. Observations through vertical glass walls in experimental channels lined with such physically different vegetations as bermudagrass (Cynodon dactylon), weeping lovegrass (Eragrostis curvula) and yellow bluestem (Andropogon ischaemum), have revealed that vegetation remains up in the flow, waving and whipping back and forth. The severity of the action is a function of the velocity, vertical distribution (as affected by slope), depth of flow, and roughness of the bed.

Vegetation protects the channel by reducing the velocity near the bed. Dense stands of long-stemmed vegetations will produce deep mats, and velocities will be low in these vegetal zones. It follows that a uniform, sod-forming vegetation having a dense, relatively deep root system will offer the greatest protection against scour. However, consideration must be given to the ability of a vegetation to reassume its normal growing position and withstand and recover from excessive deposition. Bermudagrass possesses these properties and, as known by experience, is one of the best covers to use.

With vegetal linings there are factors such as adequate stand establishment, lack of uniformity of cover, uncertainty of density and climate, rodent control, maintenance of stand, and control of deposition, the evaluation of which requires experience and good judgment on the part of the technician. Since these factors are so many and varied, it is possible to present only a few general principles of vegetated-channel stability:

(a) Uniformity of cover is extremely important. The stability of a sparsely covered area is the stability of the entire channel. Weak areas should be fertilized, sodded, and treated as necessary to produce adequate cover.

(b) In construction, if practical, the topsoil should be preserved and replaced, particularly over the center portion of the channel where the flow will be deepest. When the subsoil is particularly unfavorable to vegetal growth, either a layer of good topsoil or manuring and development of a favorable soil condition is necessary.

(c) Fertilization and proper soil preparation should be done on newly constructed channels to insure rapid growth.

(d) Maintenance by mowing or controlled grazing will help to assure continuance of the desired density and uniformity of cover.

(e) Timely repair of eroded areas and protection against damage by rodents cannot be stressed too highly.

(f) Control of deposition is the first consideration of the technician when designing the channel and selecting the cover. Bunch-grasses and open covers like alfalfa, kudzu, etc., offer less retardance to low-silting flows than sod covers. Keeping covers short will reduce deposition. Wide, flat trapezoidal channels will be the most difficult to maintain from the deposition standpoint. Parabolic and triangular sections are less subject to deposition. (Unless side slopes are 6:1 or flatter, triangular sections are generally not recommended because of the undesirable velocity distribution.)

(g) If possible, keep water diverted from the channel until vegetation is well established.

To serve in designing earth channels to be lined with vegetation, permissible velocities have been determined experimentally for many different covers. Prime factors determining permissible velocities are: (a) Physical nature of vegetation (type and distribution of root growth and top growth and physical condition), (b) erodibility of soil, (c) uniformity of cover, and (d) bed slope.

Permissible velocities for bunchgrasses and other nonuniform covers are lower than where sod-forming covers are used, because (a) areas bare of vegetation exist, (b) bunchgrasses generally produce the effect of very rough beds that seriously disturb the smoothness of flow, and (c) bunchgrasses lack a dense, uniform root system.

Flow disturbances grow in severity with increase in bed slope. The ability of vegetation to provide adequate protection decreases with increase in slope.

Permissible velocities to use in design are contained in table 3. Many have been determined by experiment, others by reasonable extension and interpolation.

Design of Bare and other Nonvegetal Channels

The application of the Manning formula to hard-surfaced channels of concrete, sheet metal, wood, etc., and to earth channels free of bottom vegetation is straightforward. For the former, n can usually be assumed as constant for a given type of lining irrespective of the slope and shape of channel and depth of flow. For earth channels with bank vegetation, the selection of n for the smaller channels must be based on the width of the water surface, since some bank vegetation partially submerged has a relatively greater retarding effect in narrow channels. Table 1 contains values of Manning's n for common hard-surfaced linings and various conditions of earth channels.

A nomographic solution of the Manning formula, figure 1, and the channel-dimension diagrams for trapezoidal, triangular, and parabolic-shaped channels, figures 2 to 12 in the Appendix, permit rapid solution of all ordinary channel problems where uniform flow is involved. The following examples illustrate the solution of several types of problems:

Example 1

Given: Q = 50 cubic feet per second
S = 0.0002 foot per foot

Find: The bottom width, depth of flow, and mean velocity of a drainage channel with 1.5:1 side slopes. Channel to be maintained regularly and kept free of excessive vegetal growth. Estimate n to be 0.04.

Solution: Requires one or more trial solutions. The steps are: (1) Select a velocity, (2) compute the area required from Q/V, (3) determine the required R from the nomograph (fig. 1), and (4) determine the bottom width and depth from the 1:5:1 dimension diagram (fig. 3).

Item	Trial Solutions		
	1	2	3
V - feet per second (selected)	1.0	0.9	0.95
A - square feet (from Q/V)	50.0	55.6	52.7
R - feet (from *fig. 1*)	2.60	2.20	2.40
b - feet (from *fig. 3*)	*	15	7.5
D - feet (from *fig. 3*)		2.85	4.1

*Bottom width less than 2 feet. Section not practical.

Dimensions as determined by solutions 2 and 3 are practical. Choice will depend upon depth and bottom width of channel desired. If still not satisfactory, change V slightly and redetermine dimensions.

Example 2

Given: Q = 50 cubic feet per second

Find: The bed slope required to maintain a velocity of 2.5 feet per second in a trapezoidal channel with side slopes 3:1 and bottom width of 10 feet. Determine the depth of flow. Use $n = 0.04$.

Solution: The area required is Q/V or 20 square feet. Enter figure 6 (3:1 dimension diagram) with A = 20 and b = 10, and find R = 1.08 feet. In the nomograph (fig. 1) extend a line from R = 1.08 through n = 0.04 to the pivot line. Then from this point on the pivot line, extend a line through the velocity scale at 2.5 to the slope line. The slope required is 0.004 foot per foot. Depth of flow (from fig. 6) will be 1.[illegible] feet.

Example 3

Given: An existing drainage channel of these dimensions: Side slopes 3:1, bottom width 12 feet, depth 3.5 feet, bed slope 0.0008 foot per foot. Channel has not been well maintained and some willows and other vegetal growth exist. Estimate n to be 0.06.

Find: Present capacity allowing a 0.5 foot freeboard.

Solution: Depth of flow will be 3.0 feet. From the 3:1 dimension chart (fig. 6) R = 2.05 feet and A = 63 square feet. From nomograph (fig. 1), V = 1.15 feet per second. Then channel capacity = 63 x 1.15 = 72.4 cubic feet per second.

Design of Vegetation-Lined Channels

General

The material in this section applies to the design of terrace outlets, meadow strips,

diversion channels, pond-spillway channels, and other completely vegetated channels subject to intermittent flow. The design problem is more complicated than for nonvegetal channels, since the use of a constant value for Manning's n is not usually correct where vegetal linings are used. Under the influence of velocity and depth of flow, vegetation tends to bend and oscillate continuously. The retardance to flow varies as these factors change.

It has been determined experimentally in both large and small channels, in channels of different cross-sectional shape and bed slope, and with different vegetations, that the Manning retardance coefficient n varies with VR, the product of velocity and hydraulic radius. This relationship will be referred to as the n-VR relationship. It is the basis of the vegetal-channel design method presented in this handbook.

Five general n-VR curves have been selected to apply to groups of vegetations. These curves, figure 13, were selected from a study of the experimental results presented graphically in the Appendix.

The various vegetations for which experimental n-VR relationships are available are classified in table 4 according to their degree of retardance. When a cover can be expected to be significantly shorter or much less dense than that described in the table, use the next lower degree of retardance in estimating capacity.

Above some minimum level of cover density, the tallness of the vegetation overshadows the effect of differences in density on retardance. For these conditions, only limitations as to tallness and to density conditions need be considered. Except for very sparse coverage, table 5 may be used to judge the degree of retardance.

The design of vegetation-lined channels requires that n be compatible with the value of VR. To accomplish this easily, graphical solutions of the Manning formula that apply to the five degrees of retardance, A, B, C, D, and E are presented in figures 14 to 18. These apply to flow with the vegetation completely submerged or nearly so. For shallow flow through upright vegetation with no submergence, Manning's n ceases to be related to VR. Some results of low-flow studies have been reported in the Transactions of the American Geophysical Union (April 1946).

To aid in channel design, graphical solutions of dimensions of trapezoidal (side slopes 1:1, 1.5:1, 2:1, 2.5:1, 3:1, 4:1, 5:1, and 6:1), triangular, and parabolic channels are included in figures 2 to 12. Complete examples for using the graphical solutions are presented in subsequent sections.

The parabolic channel, offering distinct advantages for control of scour and deposition, deserves explanation. Most natural channels tend to assume this shape. The parabolic channel is defined by $d = k(t/2)^2$, where d is the depth, t the width at depth d, and k is a shape constant.

As soon as d and t have been determined for maximum flow (from fig. 11), k is automatically evaluated (a fixed point on the pivot line in fig. 12). It remains constant for the parabolic channel in question. Other combinations of depth and width for that channel must give the same value of k.

To determine other values of t and d use the nomograph in figure 12. A specific parabolic channel will have only one intersection point on the pivot line. When this is located by a straight line through the d and t for maximum flow, it serves as a pivot point that allows width to be readily determined for any other depth. In the example illustrated on the nomograph, the maximum flow was at a depth of 1.95 feet and top width of 32 feet. A 0.5-foot freeboard required a total depth of 2.45 feet. The top width at this depth is graphically determined to be 35 feet.

As an aid in judging the shape of the channel, a side-slope line is incorporated in the nomograph in figure 12. Extension of a line through a specific pivot point to this slope scale gives the approximate side slope at the depth being considered. In an initial design, a check of the side slope at maximum depth is recommended so as to assure that a steep, impractical shape is not being selected.

Selection of Channel Shape

The technician generally has some degree of freedom in the selection of channel shape. Following are some considerations that affect his decision:

(a) Equipment available for construction
If the channel is to be built with a grader, a trapezoidal (if the bottom width is greater than the minimum cut the blade will make) or triangular shape will generally be easiest to construct. A plow, bulldozer, or fresno are more suitable for construction of parabolic shapes. If a plow or ordinary grader is to be used, side slopes steeper than 4:1 cannot be constructed readily.

(b) Maintenance (by mowing)
If the vegetation in a channel is to be maintained by mowing, side slopes 3:1, 4:1, or flatter will be found desirable. A triangular channel, or relatively flat parabolic channel, will be easier to mow than a small trapezoidal channel with a bottom narrower than the length of a sickle bar.

(c) Crossing channel with equipment
Where a channel is to be crossed occasionally, with farm machinery, its side slopes should be 4:1 or flatter.

(d) Depth of excavation
Broad-bottomed trapezoidal channels require the shallowest excavation. To carry the same quantity of water, parabolic channels require a slightly deeper center excavation and triangular channels the deepest. Where depth of excavation is a limiting factor, these inherent differences between shapes should be considered.

(e) Stability
Trapezoidal channels designed and constructed with wide flat bottoms tend to silt during periods between peak discharges, thereby reducing their capacity. This silting, which occurs unevenly, results in channeling of intermediate flows and a consequent entrenching action. On the other hand, triangular channels decrease the opportunity for silting by concentrating the low flows. Since the high flows are also concentrated in the "V" of the channel, the higher velocities are more likely to produce damaging scour. The "V" of the triangular channel is the critical area. A common objection is that trickle flows of long duration seriously weaken the vegetation in the "V."

The triangular and, particularly, the trapezoidal cross sections will not maintain their shape as well as the parabolic under ordinary conditions of intermittent runoff. In fact, the trapezoidal and triangular sections tend to become parabolic in shape, due to the normal actions of channel flow, deposition, and bank erosion.

The parabolic cross section approximates the cross-sectional shape naturally assumed by many old channels. For conservation channels, this appears to be a very desirable compromise between the other two shapes, combining most of the strong points of each, with none of the undesirable characteristics of either.

Selection of Vegetation

In addition to climate and soil, which determine the type of vegetation that will grow and survive in any particular region, there are local factors that influence the selection of a vegetal lining for a channel. From a stability standpoint these, in approximate order of importance, are:

(a) Discharge to be handled
In general, the greater the discharge the better is the vegetal lining required.

(b) Bed slope
Permissible velocities decrease with increase in slope. As bed slopes increase channeling of the flow is more likely to occur. For this reason, bunchgrasses should not be used alone on slopes steeper than 5 percent. For slopes above 5 percent, only sod-forming covers should be used on the channel bed where the main flow occurs.

(c) Establishment
Ease of establishment and time required to develop a protective cover are extremely important considerations in selecting a vegetation. Bermudagrass and weeping lovegrass establish covers rapidly. Sometimes annuals are desirable for more immediate or earlier protection, with native grasses seeded as soon as possible. Generally, any type of temporary cover during establishment of permanent cover would be better than none at all.

(d) Suitability to farmer
If a sod cover is required, as determined by discharge and slope, but is objectionable to the farmer because of likely spreading, a combination channel might be selected. This type of channel will have, for example, bermudagrass on the bed and partially up on the sides and weeping lovegrass on the upper reaches of the side and on the berm.

(e) Deposition
Deposition may be controlled to some ex by the selection of vegetation. Low, shal. flows encounter very high retardance when flowing through sod covers like bermudagrass. A dense sod cover keeps the flow from channeling. This results in low velocities which are conducive to excessive deposition. Only when the vegetation bends and submerges will high, nonsilting velocities develop.
Bunchgrasses and "open" covers like alfalfa, lespedeza, and kudzu offer less resistance to shallow flows than sod-forming covers. Velocities are higher, owing primarily to development of channeled flow. Less opportunity for deposition exists. These covers, however, offer less protection than sod covers and are limited to lesser slopes.

Illustration of Graphical Method of Design

Graphical solutions are provided in this handbook to permit rapid design of trapezoidal, triangular, and parabolic channels with five different degrees of vegetal retardance. Care has been taken to make the parabolic solution complete and as easy to use as the other two types of sections.

The degree of vegetal retardance depends largely on the tallness and density of cover, particularly the tallness. The selection of the degree of retardance for a given channel will depend mostly upon the tallness of the cover chosen. Generally, after the cover is selected, the retardance with a good uncut condition will be the one to use for capacity determination. Since a condition offering less protection exists at least during the establishment period, it is advisable to use the next lower degree of retardance when designing for stability. The procedure is to design first for stability, then, with the next higher retardance, determine the increase in depth necessary to contain the maximum discharge.

In the following example, dimensions of a trapezoidal channel with 4:1 side slopes, a triangular channel, and a parabolic channel are determined graphically for a given discharge and bed slope. Weeping lovegrass is selected as the channel lining.

Given: Q = 100 cubic feet per second
S = 3 percent
Cover - weeping lovegrass

Find: Dimensions of a trapezoidal channel with 4:1 side slopes, a triangular channel, and a parabolic channel that will carry this flow on a 3-percent bed slope. Uniform flow is considered.

Solution: Refer to table 3 and find that the permissible velocity for an average stand of weeping lovegrass on a silt-loam soil is 3.5 feet per second. Table 4 classifies weeping lovegrass under retardance A at the end of the second season. (The channels should be designed for stability, using retardance B and then provide sufficient additional depth to allow for retardance A.)

The first step is to design for stability, using retardance B. Enter the design graph for retardance B (fig. 15) with V = 3.5 and determine R = 0.99 foot. The area required is Q/V = 28.6 square feet. From dimension charts (fig. 7, 10, and 11) the following dimensions are determined:

Shape	Side slope	Bottom width b	Depth d	Width t
		feet	*feet*	*feet*
Trapezoidal.....	4:1	18	1.25	--
Triangular.....	7:1	0	2.0	--
Parabolic.....	--	--	1.5	28

The second step is to determine the depth of flow in the above channels under retardance A. (A cut-and-try procedure is necessary since all dimensions except depth and top width are determined.)

(a) For trapezoidal channel with 4:1 side slope and b = 18 feet: Try d = 1.6 feet. Enter figure 7 with d = 1.6 and b = 18 and find R = 1.25 and A = 39. Enter figure 14 (design chart for retardance A) with R = 1.25 and S = 3 percent and determine V = 2.25. Then capacity with d = 1.6 is AV = 39 x 2.25 = 87.6 cubic feet per second. Since this is significantly less than the 100 required, a new, slightly higher d should be investigated. Try d = 1.65. Proceeding again through figures 7 and 14 find R = 1.28, A = 41, and V = 2.5. Capacity with d = 1.65 is 103 which is adequate. Adding freeboard to this depth will give the total depth of channel.

(b) For triangular channel with 7:1 side slopes: Try d = 2.5 feet. Enter figure 10 with z = 7 and d = 2.5 and find R = 1.25 and A = 44. Enter figure 14 with R = 1.25 and S = 3 percent and find V = 2.25. Then capacity with d = 2.5 feet is 99 cubic feet per second which is adequate. Total depth of channel will be 2.5 plus freeboard.

(c) For parabolic channel with the channel shape determined by d = 1.5 feet and t = 28 feet: In the nomograph (fig. 12) project a line from d = 1.5 through t = 28 to the pivot line. The point established on this line remains fixed for this particular channel. Assuming d = 1.9 for retardance A, project (fig. 12) a straight line between this depth and the fixed point on the pivot line and determine t = 31 from the width scale. Then from figure 11 with d = 1.9 and t = 31 determine R = 1.26 and A = 40. Enter figure 14 (retardance A) with R = 1.26 and find V = 2.3. Then capacity at d = 1.9 equals 40 x 2.3 = 92 cubic feet per second which is too low. Try d = 1.95. From figure 12, t = 32. From figure 11, R = 1.28 and A = 42. From figure 14, V = 2.55. Then capacity = 107 which is satisfactory.

With parabolic channels, additional information such as the top width after a freeboard is added and widths for lesser depths is helpful. Continuing with the example: Allowing a freeboard of 0.5 foot, the following data were obtained for this parabolic channel from the nomograph, figure 12. A straight line is extended to the slope scale (through the fixed pivot point) for various combinations of d and t to arrive at the approximate side slope.

Depth d	Width t	Approximate side slope
feet	*feet*	
0.5.....	16	8:1
1.0.....	23	5.6:1
1.5.....	27	4.6:1
1.95 (Maximum w. s.).....	32	4.1:1
2.4 (with freeboard).....	35	3.7:1

The side slope at maximum depth (after freeboard is added) is important. If too steep, a reduction in velocity would be necessary to obtain a flatter, workable section. The above data are also useful in laying out and checking the channel in the field.

When a trial velocity does not yield a satisfactory cross section (bottom width too narrow with trapezoidal channels, or side slopes too steep with triangular and parabolic channels), a change in v of only 0.5 foot per second is recommended where the trial velocity is greater than 2. Dimensions will be affected greatly by this small change. Use still smaller increments of change for velocities lower than 2 feet per second.

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(2) ———— and Palmer, V. J.
1948. Results of Tests on Vegetated Waterways. Okla. Agr. Expt. Sta. Misc. Pub. 12, 43 pp., illus.

(3) King, H. W.
1939. Handbook of Hydraulics. 3d edition, 617 pp., illus. New York and London.

(4) Palmer, V. J.
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(5)
1946. Retardance Coefficients for Low Flow in Channels Lined with Vegetation. Amer. Geophy. Union Trans. 27: 187-197, illus.

(6) Ramser, C. E.
1929. Flow of Water in Drainage Channels. U. S. Dept. Agr. Tech. Bul. 129, 101 pp., illus.

(7) Ree, W. O., and Palmer, V. J.
1949. Flow of Water in Channels Protected by Vegetative Linings. U. S. Dept. Agr. Tech. Bul. 967, 115 pp., illus.

(8) Scobey, F. C.
1933. The Flow of Water in Flumes. U. S. Dept. Agr. Tech. Bul. 393, 98 pp., illus.

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1939. Flow of Water in Irrigation and Similar Canals. U. S. Dept. Agr. Tech. Bul. 652, 78 pp., illus.

(10) Smith, D. D.
1946. Bluegrass Terrace Outlet Channel Design. Agr. Engin. 27: 125-130, illus.

Appendix

TABLE 1.--Values of Manning's *n*

Item	Manning's *n*
Nonvegetal channels[1]:	
Wooden flumes--planed (no battens)	0.010 - 0.014
Wooden flumes--unplaned (no battens)	.011 - .015
Metal flumes--smooth	.011 - .015
Metal flumes--corrugated	.022 - .030
Vitrified clay tile	.012 - .017
Concrete--finished	.011 - .014
Concrete--unfinished	.015 - .020
Masonry--rubble	.017 - .030
Masonry--dry-rubble	.025 - .035
Earth channels subject to intermittent flow:	
Bare--unable to support vegetal growth or vegetal growth is not permitted by farm operations	
Straight and uniform	.017 - .025
Dredged (rough bed)	.025 - .035
Vegetal lining--use a retardance coefficient *n* related to the product of velocity and hydraulic radius. Use figures 14 to 18 to solve graphically the Manning formula for five degrees of vegetal retardance so that *n* will be compatible with VR. Use tables 4 and 5 as a guide in selecting the degree of retardance.	
Earth channels subject to continuous flow[2]:	
Canals and ditches with continuous flow or sufficiently frequent flow to keep vegetation from growing on the bed.	
A. Channels with unusually good alinement, relatively smooth beds and side slopes. Banks fairly well vegetated but no vegetation or large obstructions on the bed.	
Product of top width and depth	
Up to 10	.030 - .040
10 to 20	.020 - .030
Greater than 20	.017 - .025
B. Channels with average bed and side-slope roughness and alinement. Banks fairly well vegetated. Average maintenance	
Product of top width and depth	
Up to 10	.045 - .050
10 to 20	.040 - .045
Greater than 20	.030 - .040
C. Channels choked with weeds and willows. Values of *n* as high as 0.15 may develop.	

[1] Selected from table of Horton's values of *n* in King's "Handbook of Hydraulics."
[2] Reference to U. S. Dept. Agr. Technical Bulletins by C. E. Ramser and F. C. Scobey (see references) is recommended.

TABLE 2.--Permissible canal velocities after aging; for channels with linings other than vegetation

Original material excavated	Clear water, no detritus	Water transporting colloidal silts	Water transporting noncolloidal silts, sands, gravels, or rock fragments
	Ft. per sec.	*Ft. per sec.*	*Ft. per sec.*
Fine sand, noncolloidal	1.50	2.50	1.50
Sandy loam, noncolloidal	1.75	2.50	2.00
Silt loam, noncolloidal	2.00	3.00	2.00
Alluvial silts, noncolloidal	2.00	3.50	2.00
Ordinary firm loam	2.50	3.50	2.25
Volcanic ash	2.50	3.50	2.00
Fine gravel	2.50	5.00	3.75
Stiff clay, very colloidal	3.75	5.00	3.00
Graded, loam to cobbles, noncolloidal	3.75	5.00	5.00
Alluvial silts, colloidal	3.75	5.00	3.00
Graded, silt to cobbles, colloidal	4.00	5.50	5.00
Coarse gravel, noncolloidal	4.00	6.00	6.50
Cobbles and shingles	5.00	5.50	6.50
Shales and hardpans	6.00	6.00	5.00

[1] Recommended in 1926 by Special Committee on Irrigation Research, American Society of Civil Engineers.

Although not specifically stated in the original recommendations, these values apply only to channels with mild bed slopes.

TABLE 3.--Permissible velocities for channels lined with vegetation[1]

The values apply to average, uniform stands of each type of cover.

Cover	Slope range[2]	Permissible velocity: Erosion resistant soils	Permissible velocity: Easily eroded soils
	Percent	*Ft. per sec.*	*Ft. per sec.*
Bermudagrass	0-5	8	6
	5-10	7	5
	over 10	6	4
Buffalograss, Kentucky bluegrass, Smooth brome, Blue grama	0-5	7	5
	5-10	6	4
	over 10	5	3
Grass mixture	[2] 0-5	5	4
	5-10	4	3
Lespedeza sericea, Weeping lovegrass, Yellow bluestem, Kudzu, Alfalfa, Crabgrass	[3] 0-5	3.5	2.5
Common lespedeza[4], Sudangrass[4]	[5] 0-5	3.5	2.5

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

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

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

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

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

TABLE 4.--Classification of vegetal covers as to degree of retardance

Note: Covers classified have been tested in experimental channels. Covers were green and generally uniform.

Retardance	Cover	Condition
A	Weeping lovegrass.....	Excellent stand, tall, (average 30 inches)
	Yellow bluestem Ischaemum...	Do tall, (average 36 inches)
B	Kudzu.....	Very dense growth, uncut
	Bermudagrass.....	Good stand, tall, (average 12 inches)
	Native grass mixture (little bluestem, blue grama, and other long and short midwest grasses).....	Good stand, unmowed
	Weeping lovegrass.....	Good stand, tall, (average 24 inches)
	Lespedeza sericea.....	Good stand, not woody, tall (average 19 inches)
	Alfalfa.....	Good stand, uncut, (average 11 inches)
	Weeping lovegrass.....	Good stand, mowed, (average 13 inches)
	Kudzu.....	Dense growth, uncut
	Blue grama.....	Good stand, uncut, (average 13 inches)
C	Crabgrass.....	Fair stand, uncut (10 to 48 inches)
	Bermudagrass.....	Good stand, mowed (average 6 inches)
	Common lespedeza.....	Good stand, uncut (average 11 inches)
	Grass-legume mixture--summer (orchard grass, redtop, Italian ryegrass, and common lespedeza).....	Good stand, uncut (6 to 8 inches)
	Centipedegrass.....	Very dense cover (average 6 inches)
	Kentucky bluegrass.....	Good stand, headed (6 to 12 inches)
D	Bermudagrass.....	Good stand, cut to 2.5-inch height
	Common lespedeza.....	Excellent stand, uncut (average 4.5 inches)
	Buffalograss.....	Good stand, uncut (3 to 6 inches)
	Grass-legume mixture--fall, spring (Orchardgrass, redtop, Italian ryegrass, and common lespedeza).....	Good stand, uncut (4 to 5 inches)
	Lespedeza sericea.....	After cutting to 2-inch height. Very good stand before cutting.
E	Bermudagrass.....	Good stand, cut to 1.5 inches height
	Bermudagrass.....	Burned stubble.

TABLE 5.--Guide to selection of vegetal retardance

Stand	Average length of vegetation	Degree of retardance	Stand	Average length of vegetation	Degree of retardance
Good.....	Longer than 30"	A	Fair.....	Longer than 30"	B
	11 to 24"	B		11 to 24"	C
	6 to 10"	C		6 to 10"	D
	2 to 6"	D		2 to 6"	D
	Less than 2"	E		Less than 2"	E

TABLE 6.--Source of experimental results presented in figures 19 to 22

Identification number	Source[1]	Channel	Expt.	Date
1.....	a	U6	3	Fall, 1945
2.....	a	U4	1	Fall, 1945
3.....	a	U2	2	Fall, 1945
4.....	b	B2-9	2	Fall, 1939
5.....	b	B2-14B	1	Summer, 1940
		B2-10B	1	Summer, 1941
6.....	c	1D, 4D, 7B	1	Fall, 1943
7.....	a	U6	1	Fall, 1944
8.....	c	2C, 4B	1	Fall, 1943
9.....	c	1D, 2D, 4B, 7F, 10D	1	Fall, 1940
10.....	b	B2-9	1	Fall, 1938
11.....	a	U5	3	Fall, 1945
12.....	a	L1A	2	Fall, 1945
		FC3B	1	Fall, 1945
13.....	b	B2-6	1	Fall, 1938
14.....	b	B2-5	1	Fall, 1938
15.....	b	B2-16B	1	Summer, 1940
		B2-12B	1	Summer, 1941
16.....	b	B1-4	1	Fall, 1939
17.....	d	-	-	Summers, 1943-44
18.....	a	U3	3	Fall, 1945
19.....	c	Results from a group of channels varying in slope from 1 to 10 percent		Fall, 1942
20.....	b	B2-15B	1	Summer, 1940
21.....	b	B2-12C	1	Spring, 1941
		B2-16C	1	Spring, 1940
		B2-16A	1	Fall, 1940
22.....	b	B2-14A	1	Fall, 1940

[1] Source — Investigator

a Stillwater Outdoor Hydraulic Laboratory, Soil Conservation Service, Research Project OK-R-3, Stillwater, Okla.

b Spartanburg Outdoor Hydraulic Laboratory, Soil Conservation Service, Research Project SC-R-2, Spartanburg, S.C.

c Oklahoma A & M Agricultural Experiment Station, Stillwater, Okla.

d Soil Conservation Experiment Station, Soil Conservation Service, Research Project, Columbia, Mo.

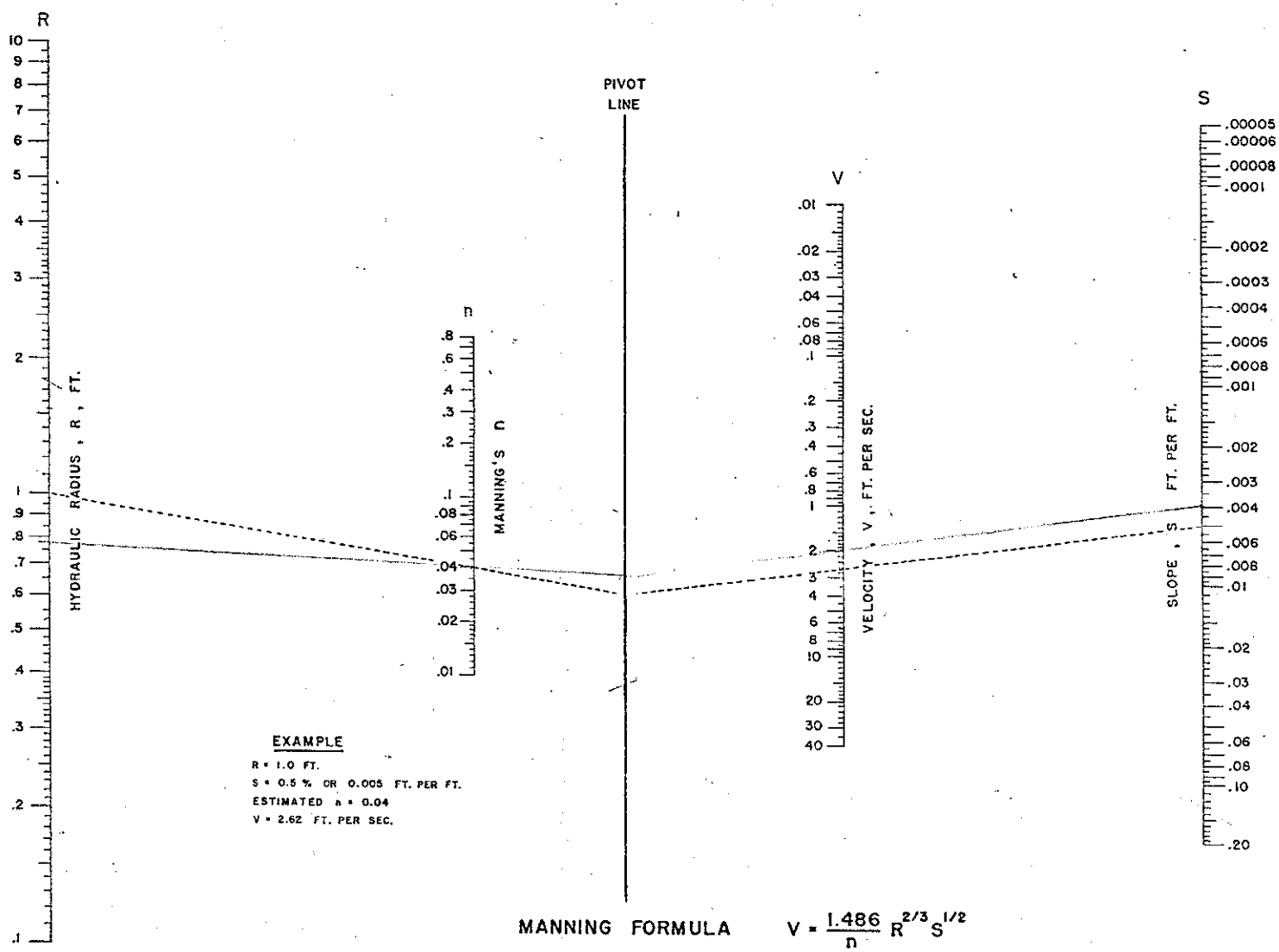


Figure 1.--Solution of the Manning formula.

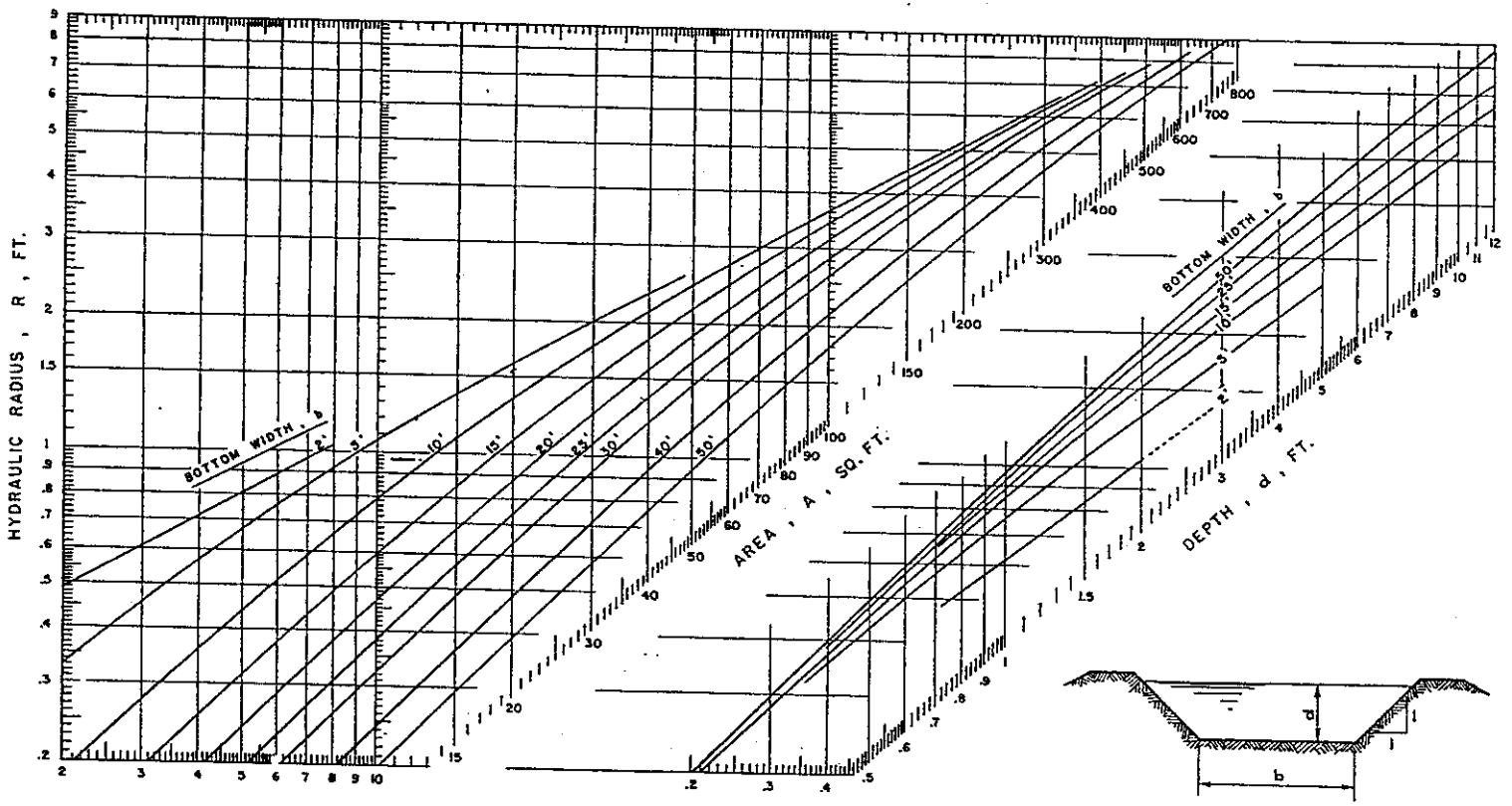


Figure 2.--Dimensions of trapezoidal channels with 1 to 1 side slopes.

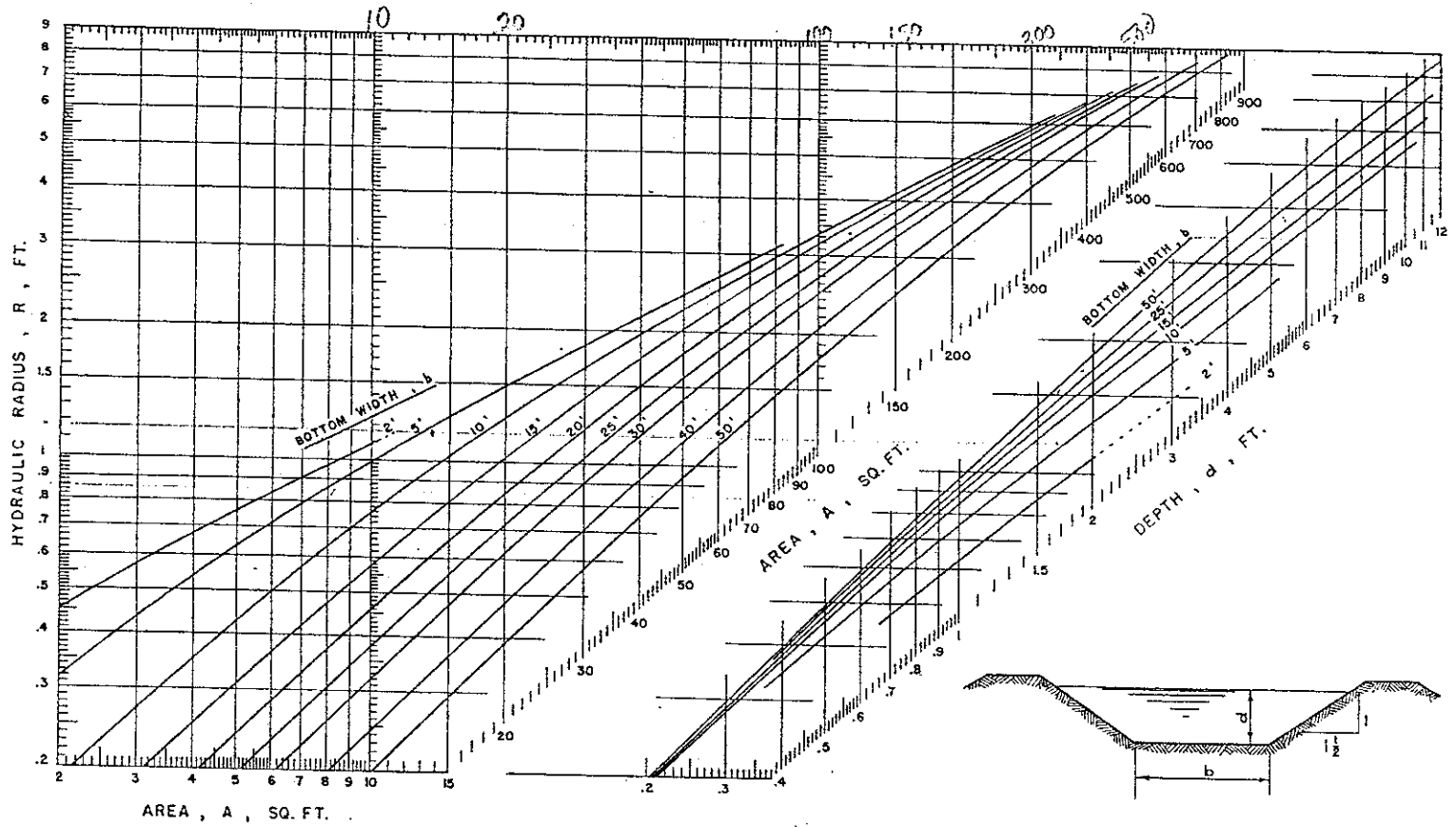


Figure 3.--Dimensions of trapezoidal channels with 1-1/2 to 1 side slopes.

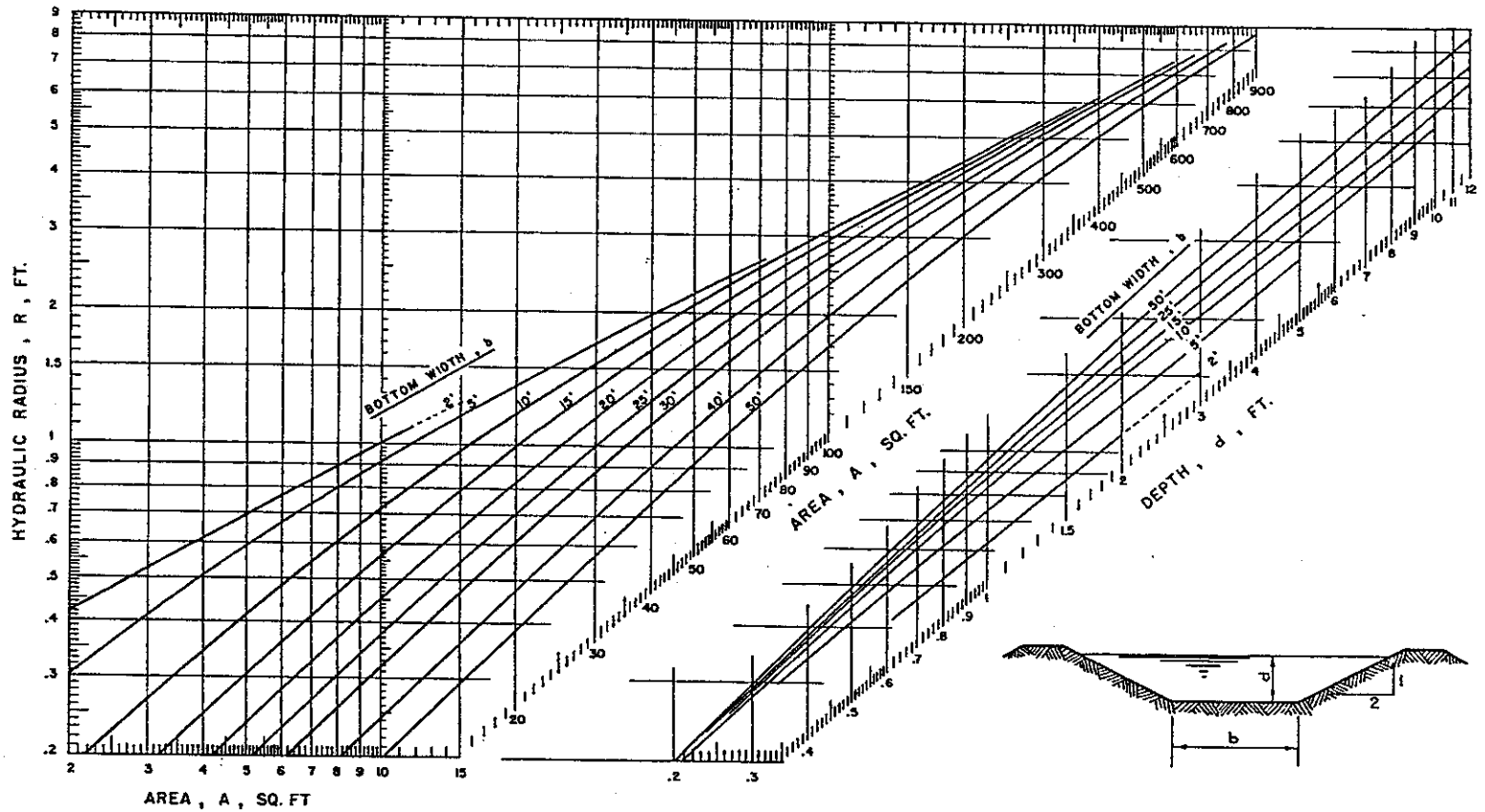


Figure 4.--Dimensions of trapezoidal channels with 2 to 1 side slopes.

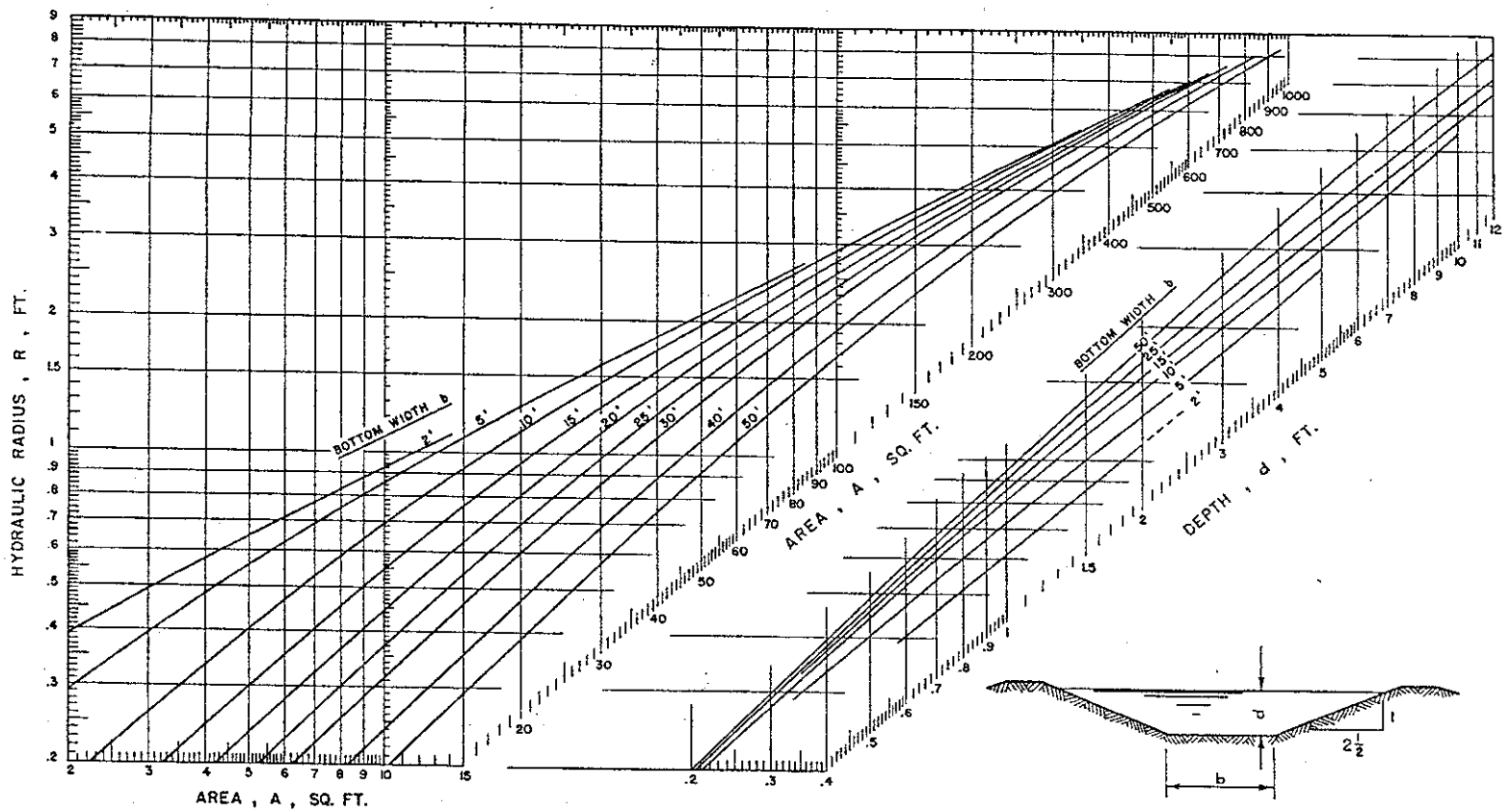


Figure 5.--Dimensions of trapezoidal channels with 2-1/2 to 1 side slopes.

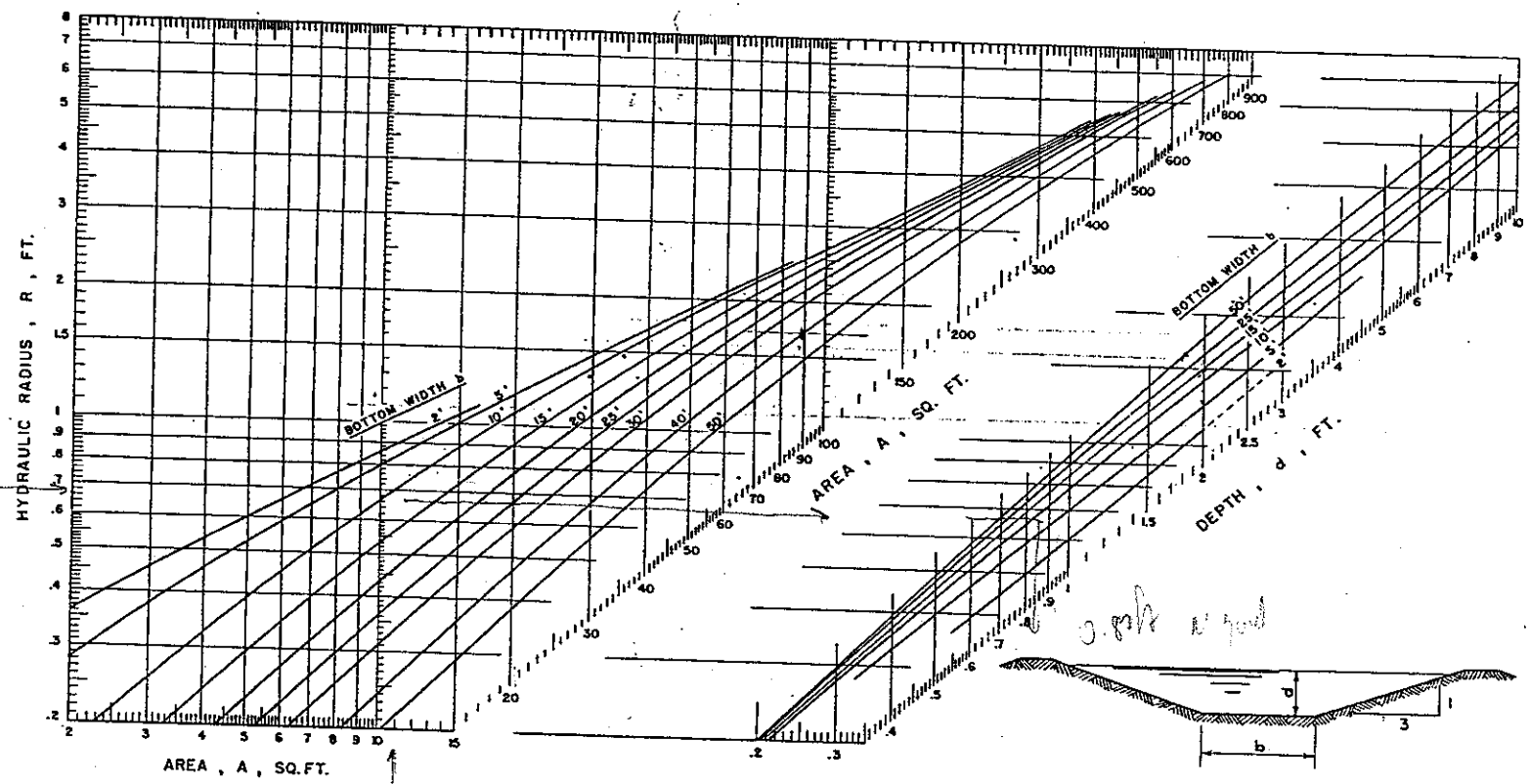


Figure 6.--Dimensions of trapezoidal channels with 3 to 1 side slopes.

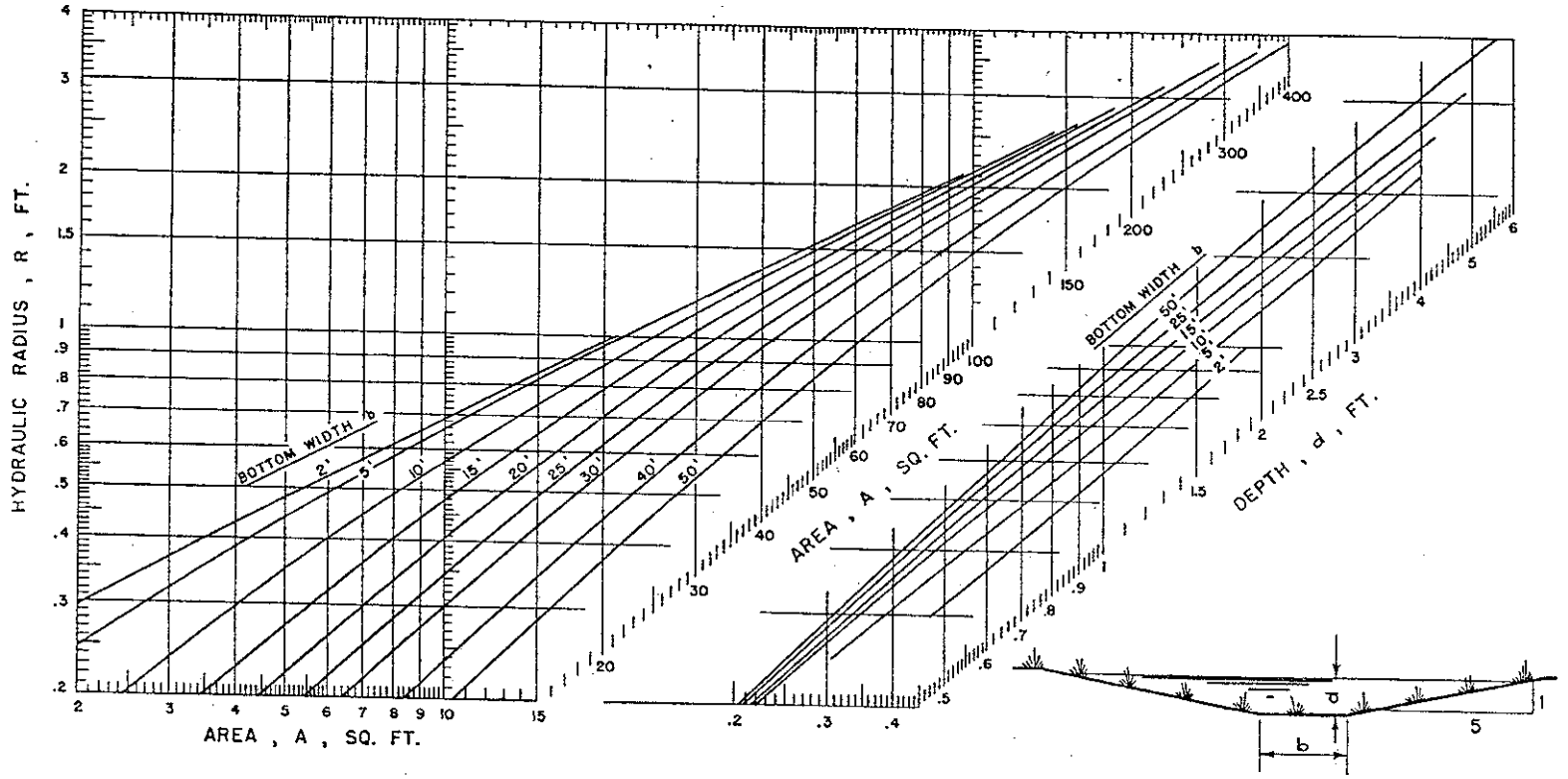


Figure 8.--Dimensions of trapezoidal channels with 5 to 1 side slopes.

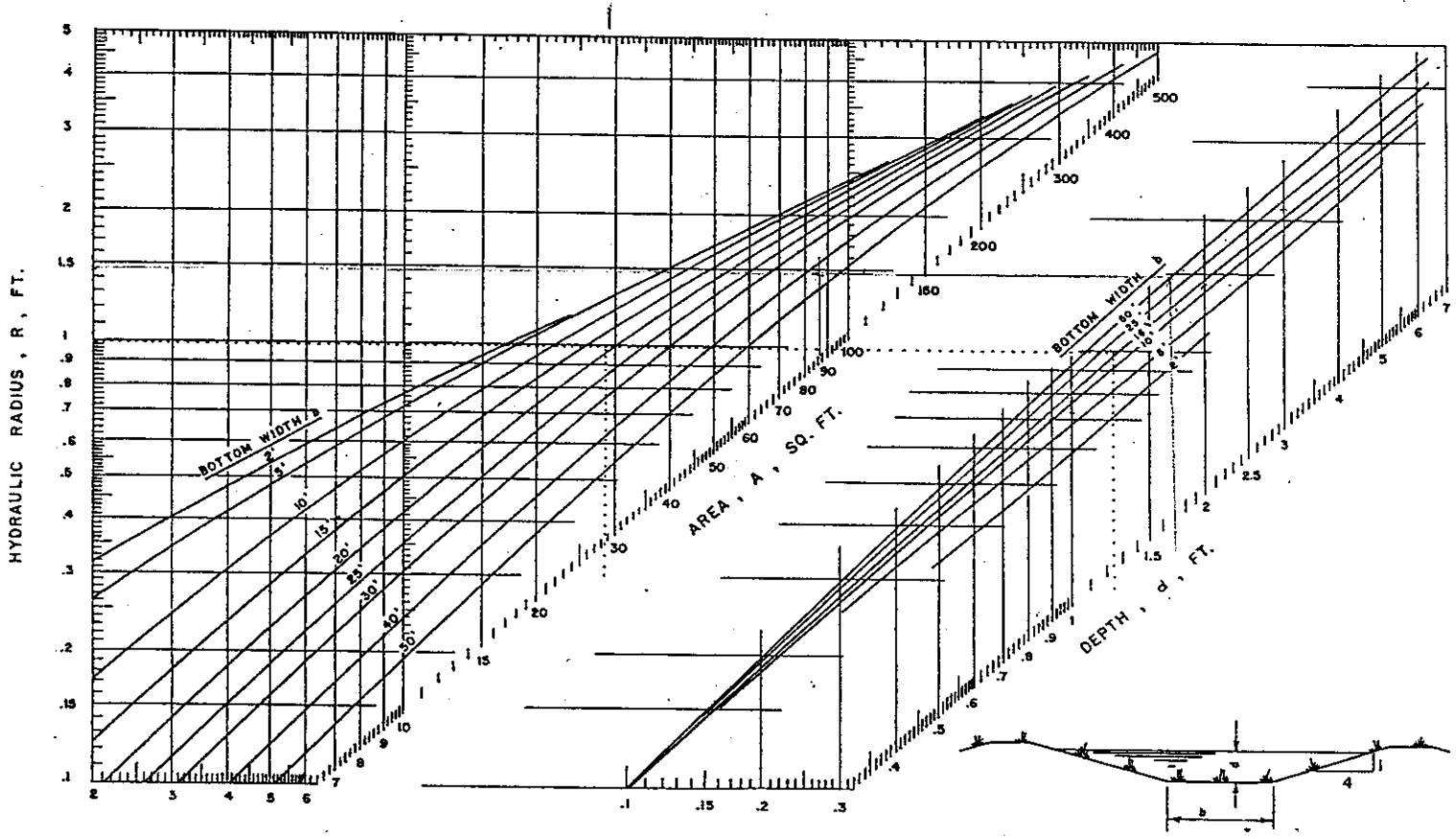


Figure 7.--Dimensions of trapezoidal channels with 4 to 1 side slopes.

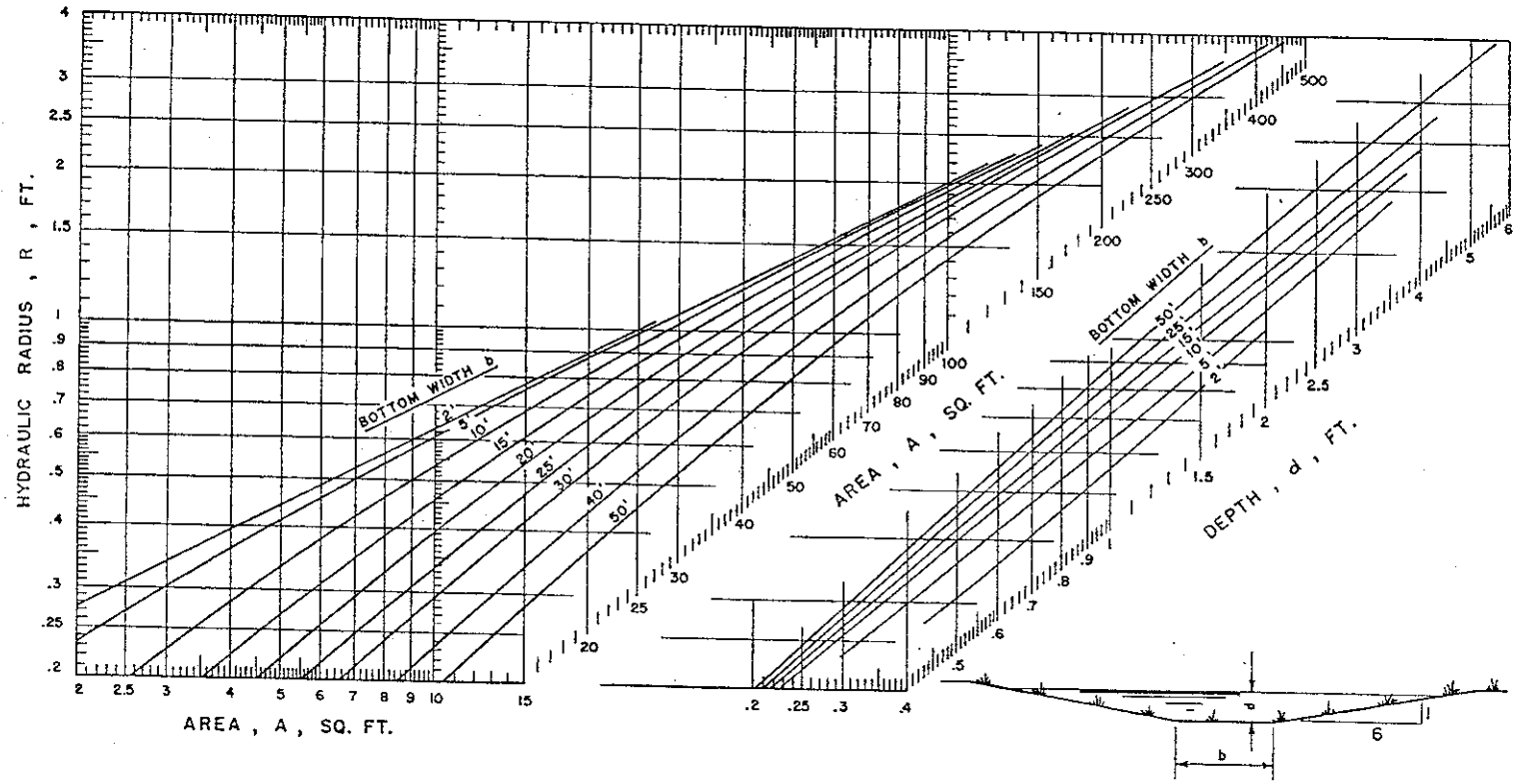


Figure 9.--Dimensions of trapezoidal channels with 6 to 1 side slopes.

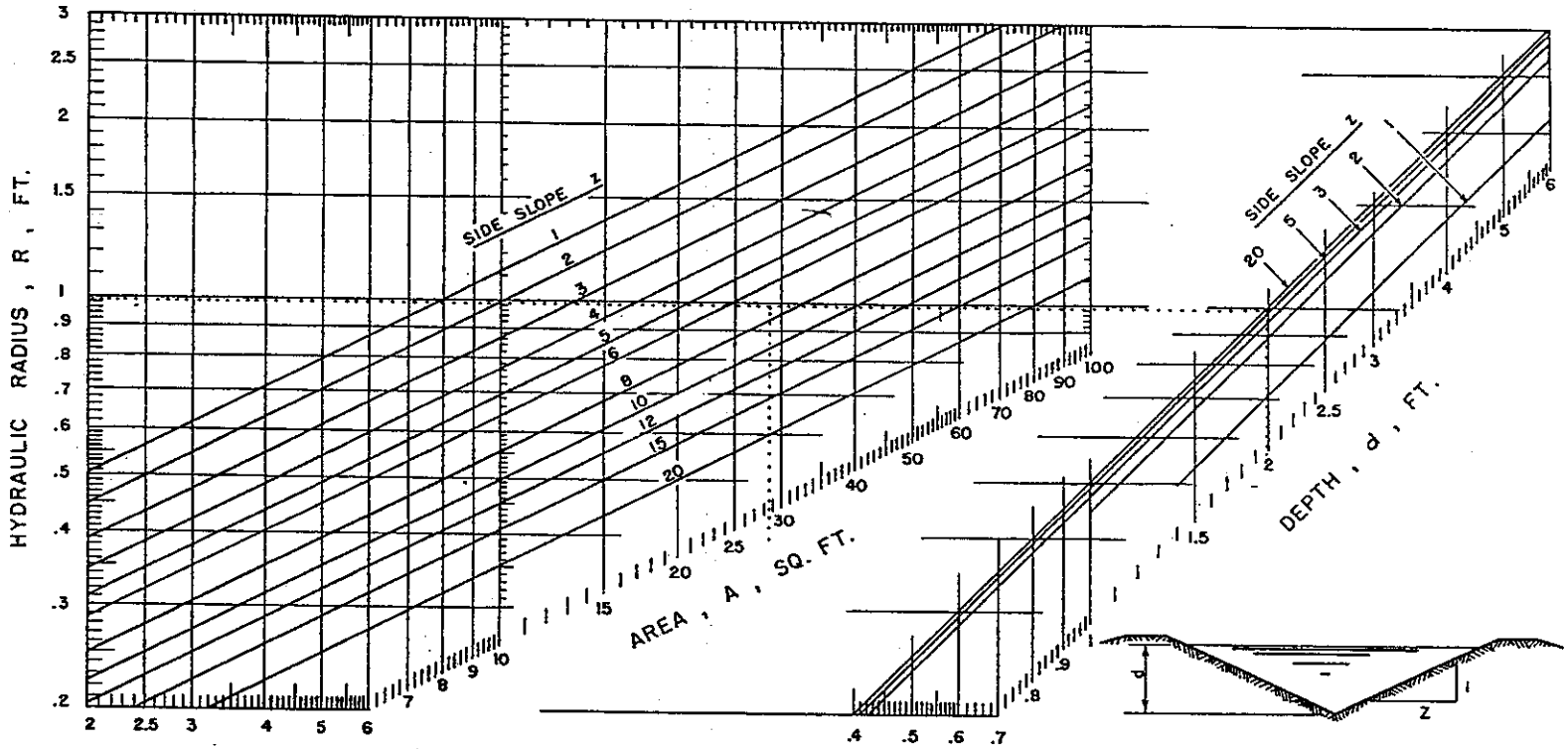


Figure 10.--Dimensions of triangular channels.

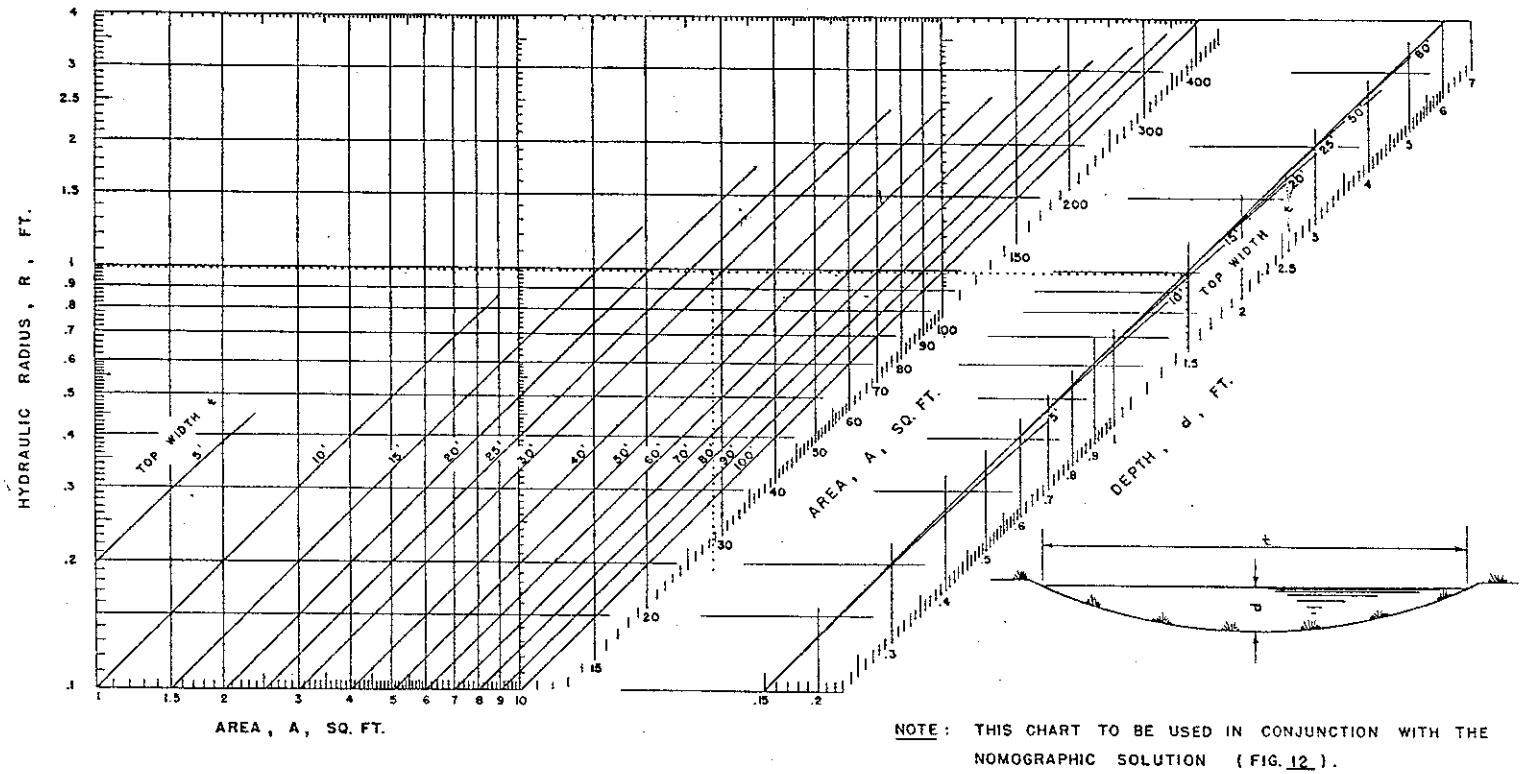
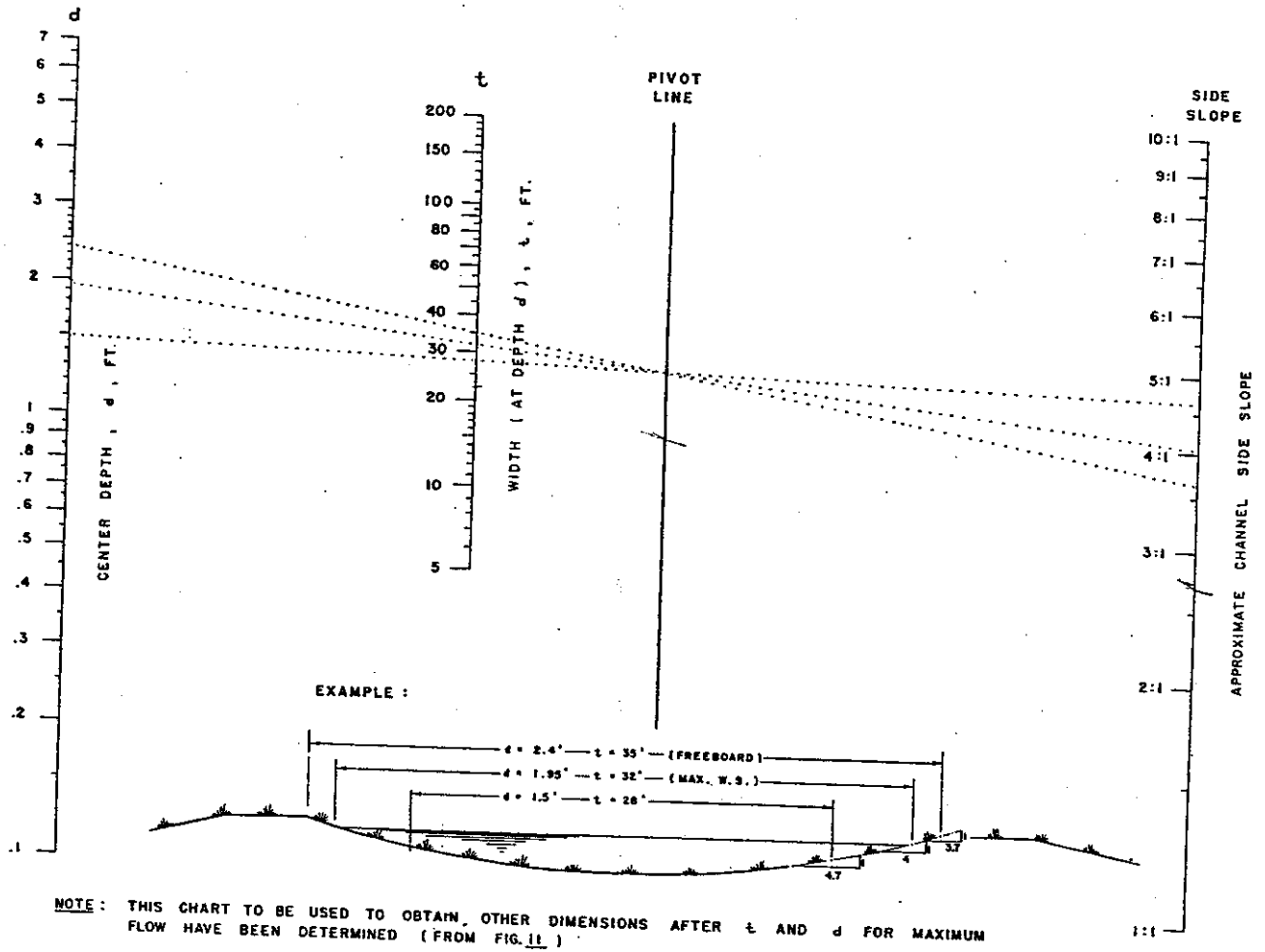


Figure 11.--Dimensions of parabolic channels.



NOTE : THIS CHART TO BE USED TO OBTAIN OTHER DIMENSIONS AFTER t AND d FOR MAXIMUM FLOW HAVE BEEN DETERMINED (FROM FIG. 11)

Figure 12.--Solution for dimensions of parabolic channels.

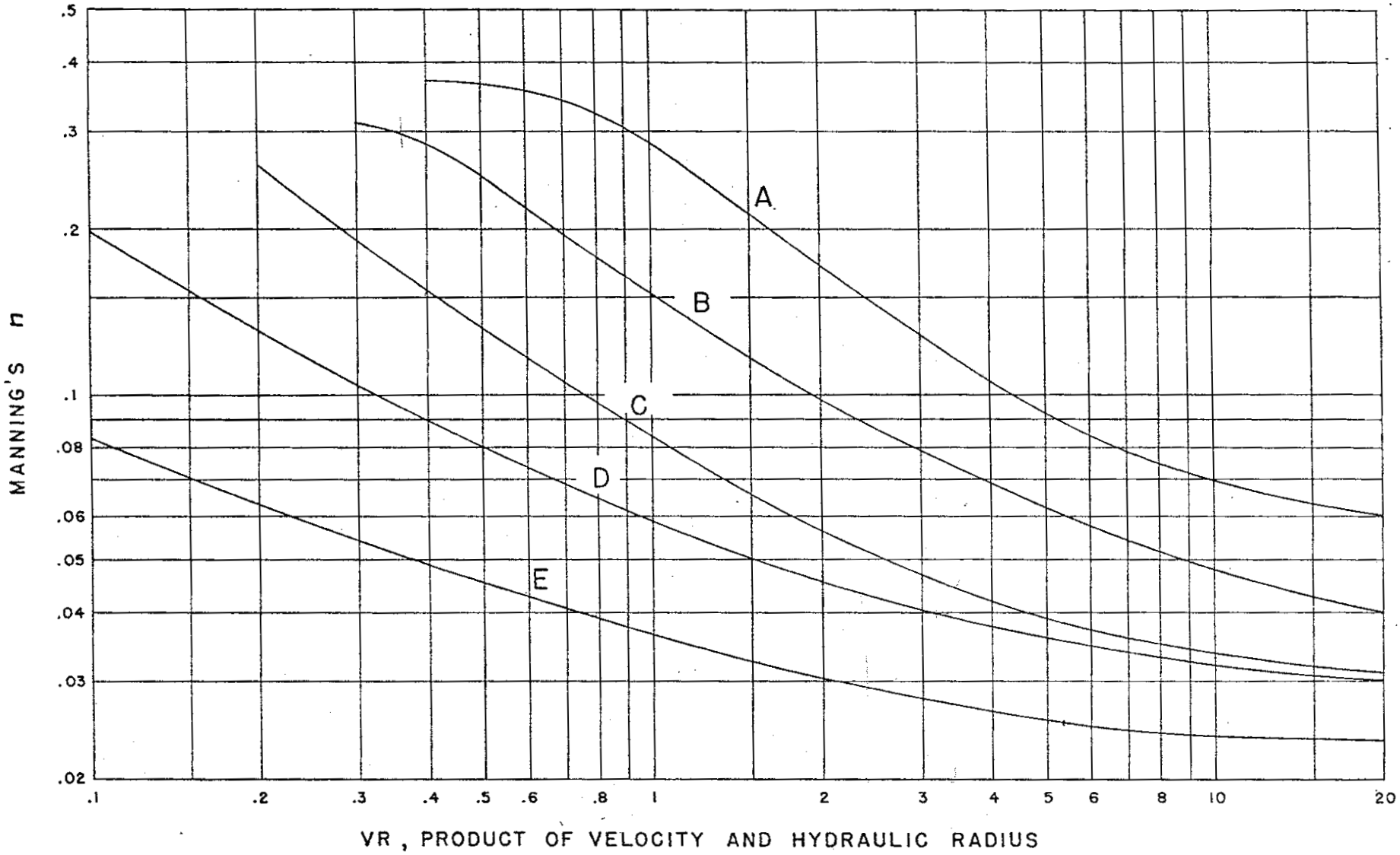


Figure 13.--Degrees of vegetal retardance for which graphical solutions of the Manning formula have been prepared.

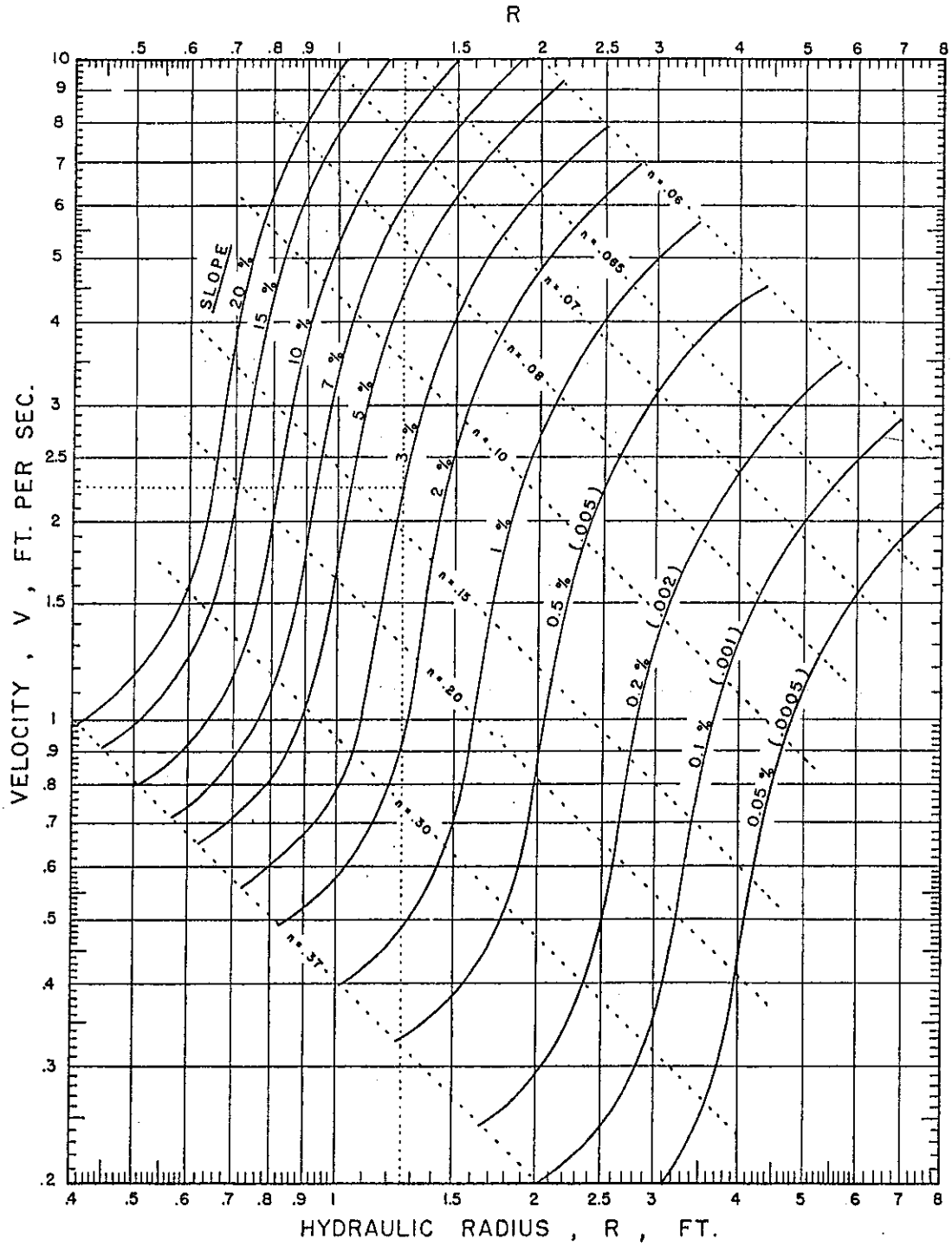


Figure 14.--Solution of the Manning formula for retardance A (very high vegetal retardance).

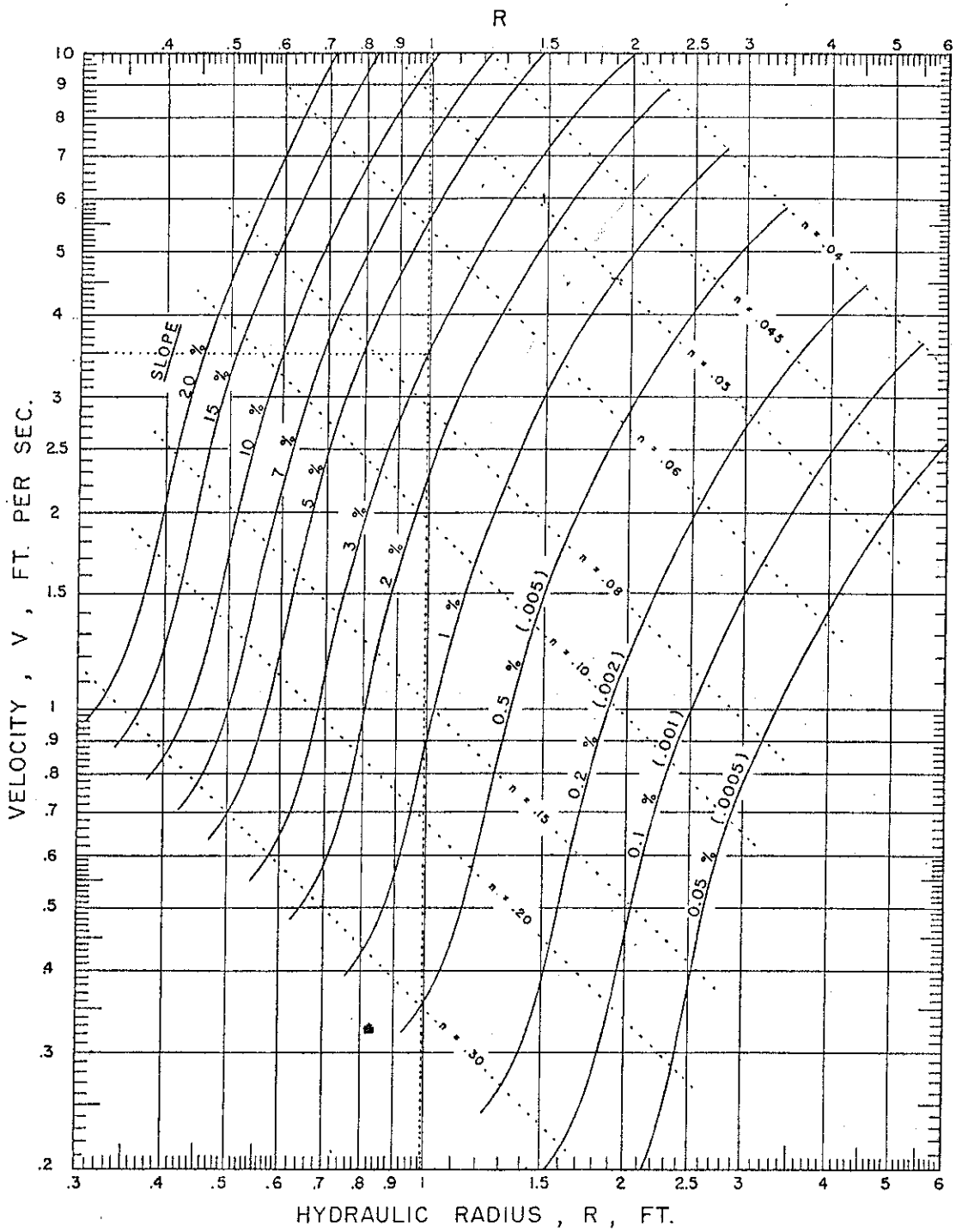


Figure 15.--Solution of the Manning formula for retardance B (high vegetal retardance).

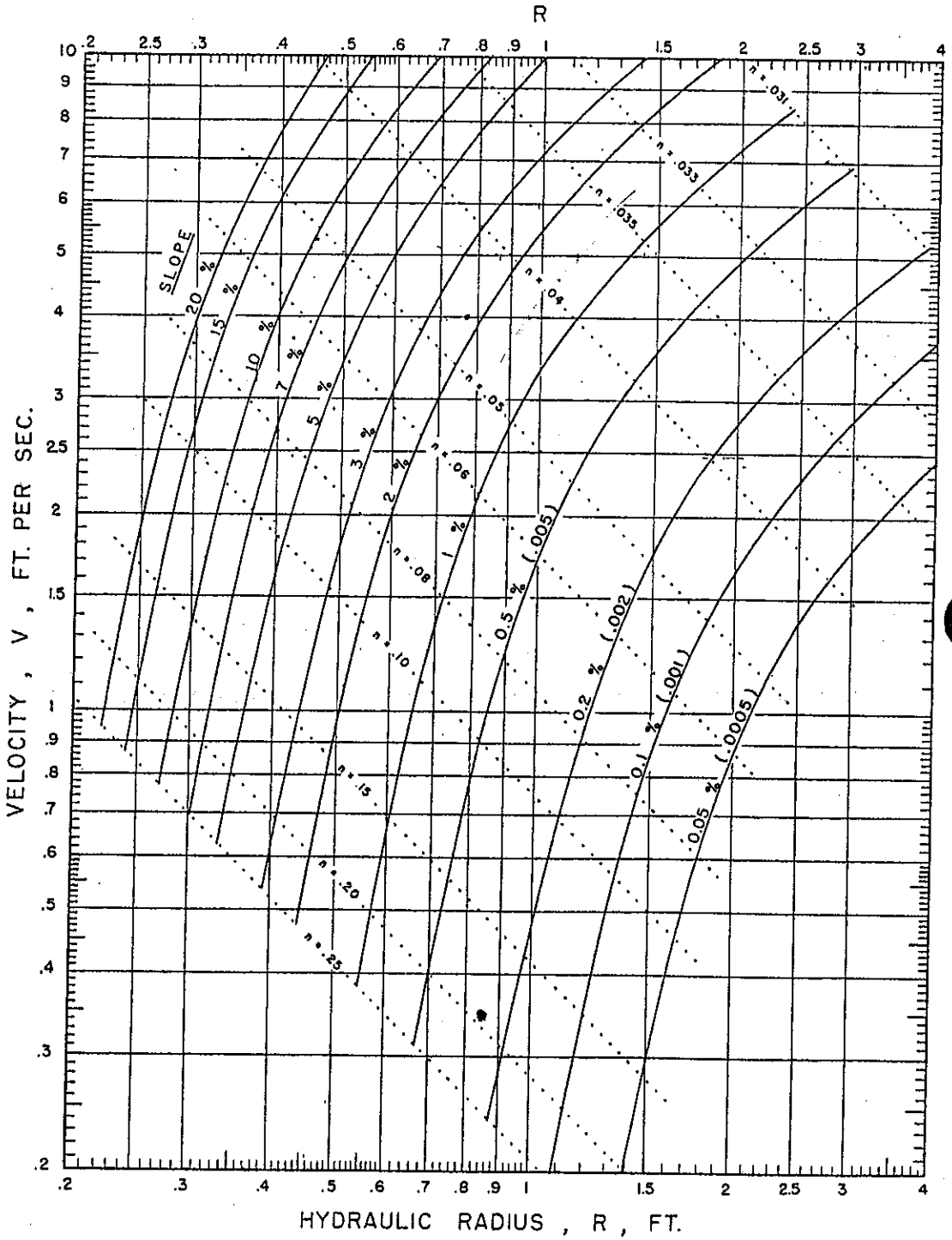


Figure 16.--Solution of the Manning formula for retardance C (moderate vegetal retardance).

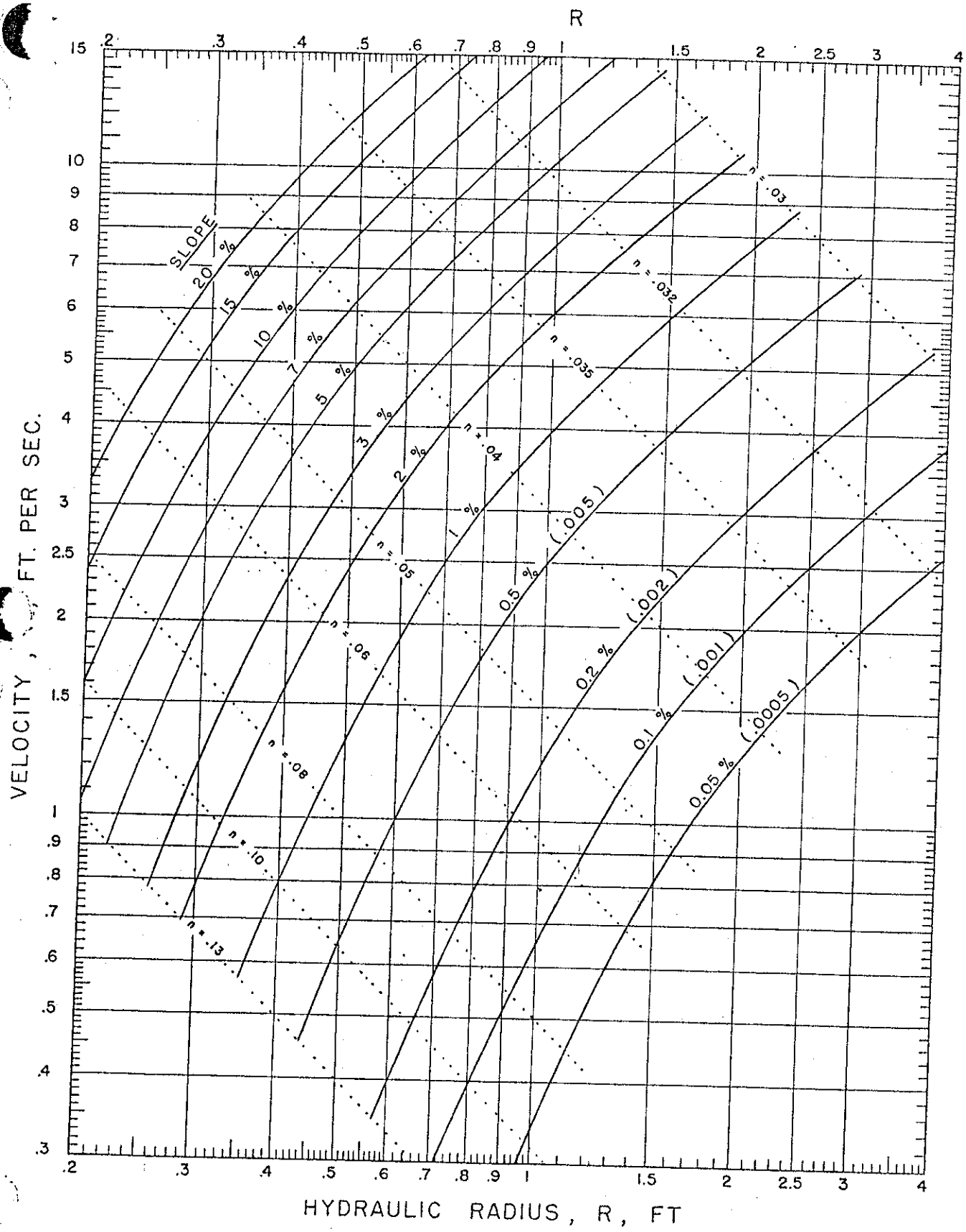


Figure 17.--Solution of the Manning formula for retardance D (low vegetal retardance).

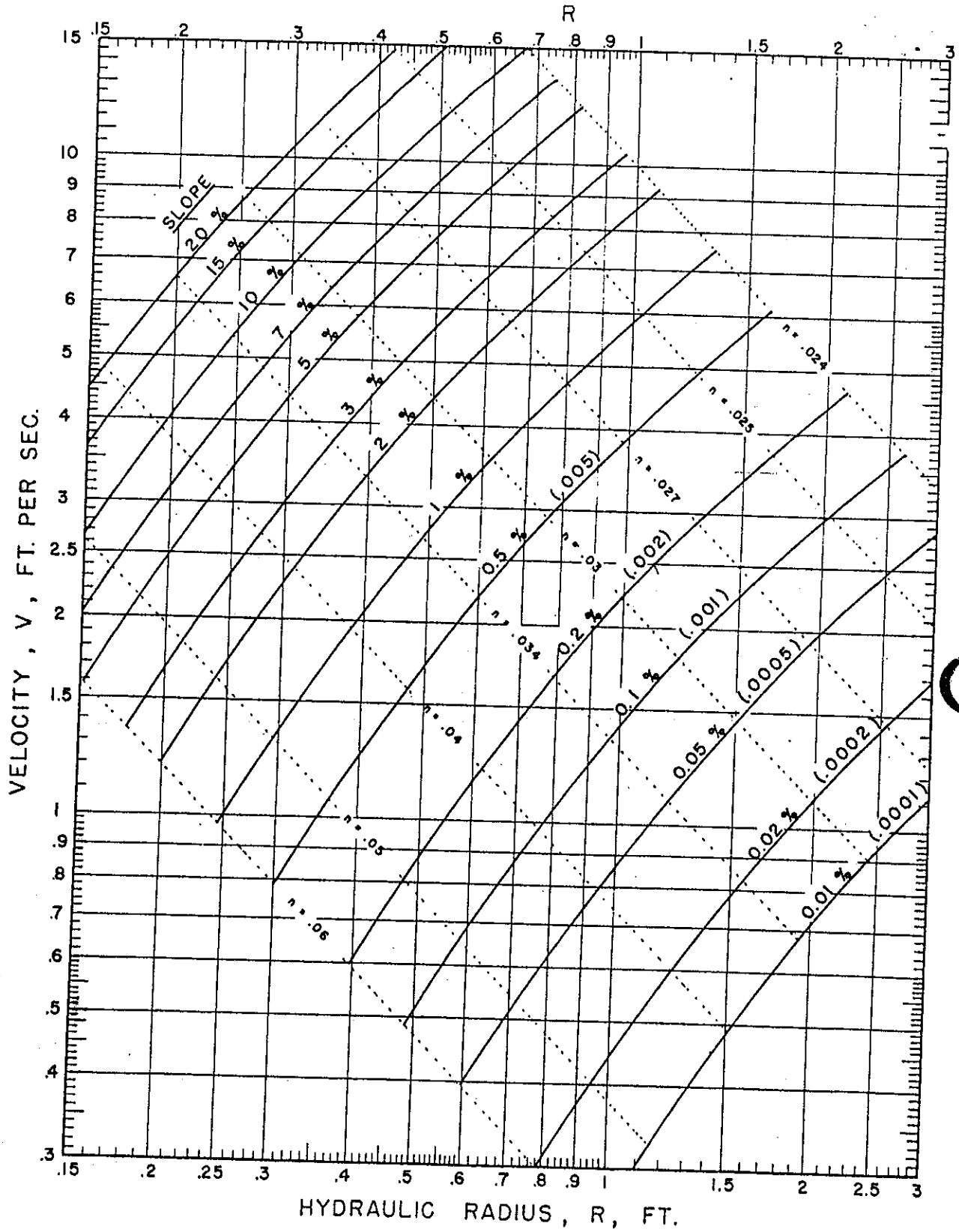


Figure 18.--Solution of the Manning formula for retardance E (very low vegetal retardance).

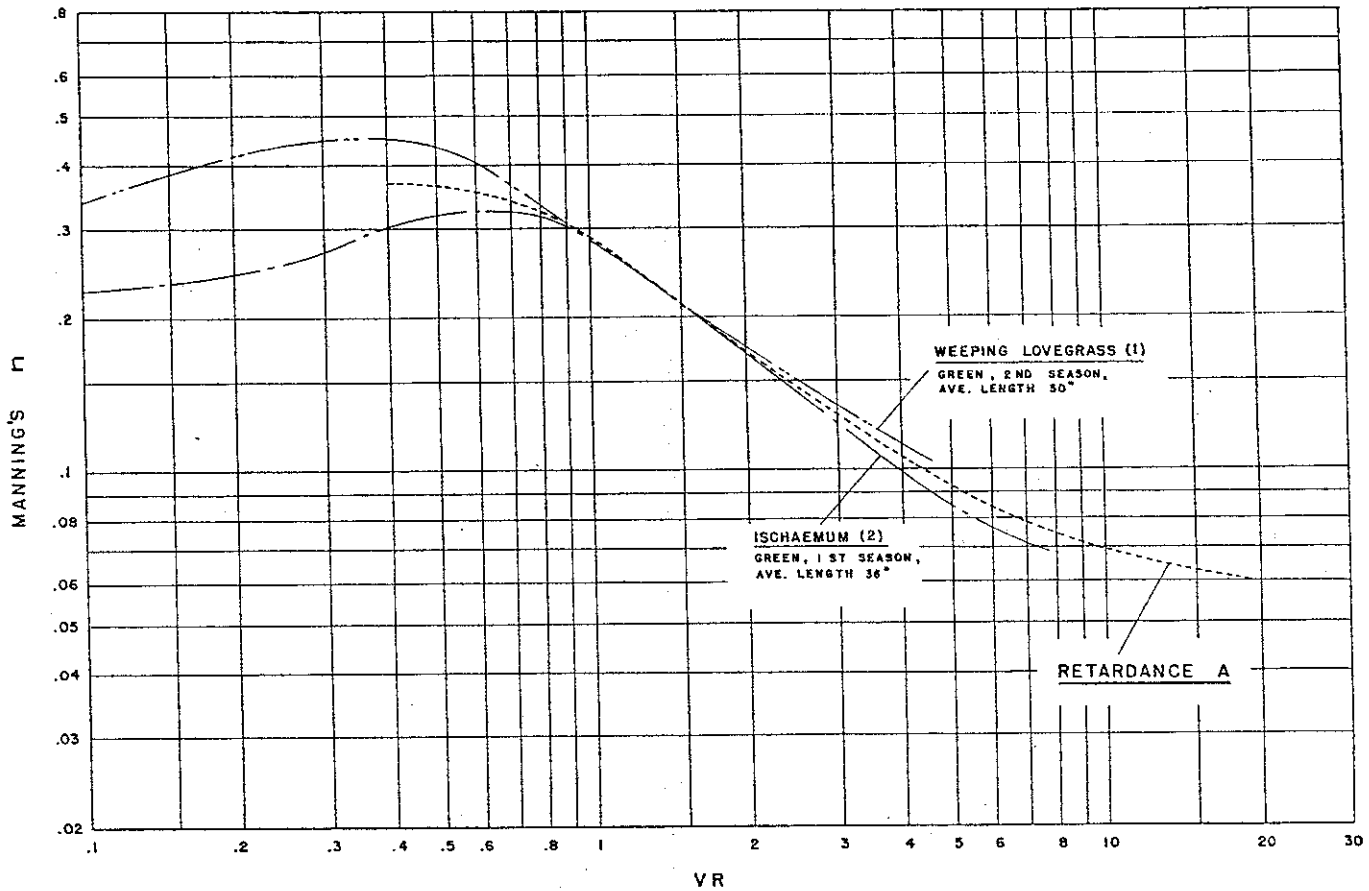


Figure 19.--Experimental results related to retardance A (very high vegetal retardance).

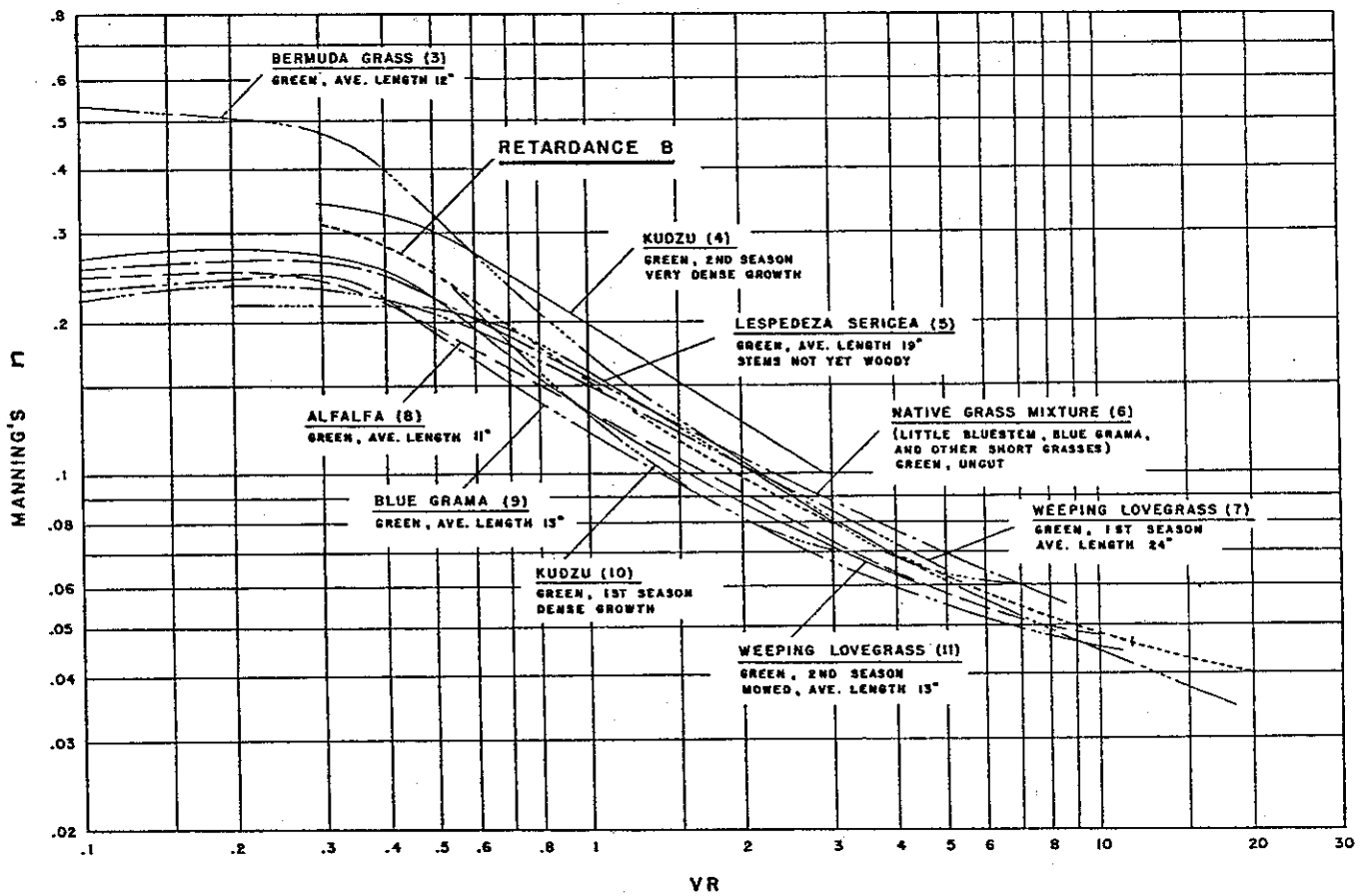


Figure 20.--Experimental results related to retardance B (high vegetal retardance).

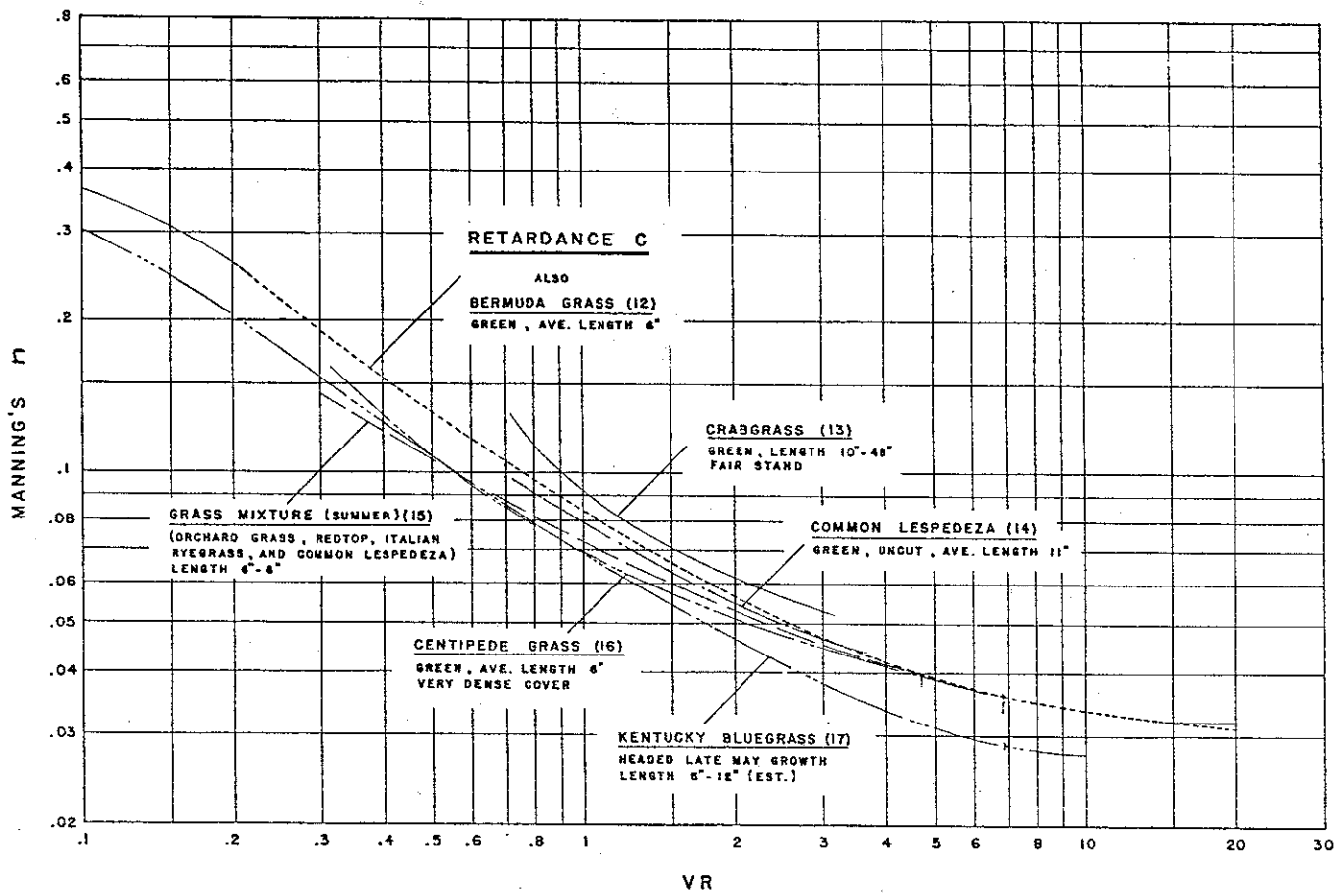


Figure 21.--Experimental results related to retardance C (moderate vegetal retardance).

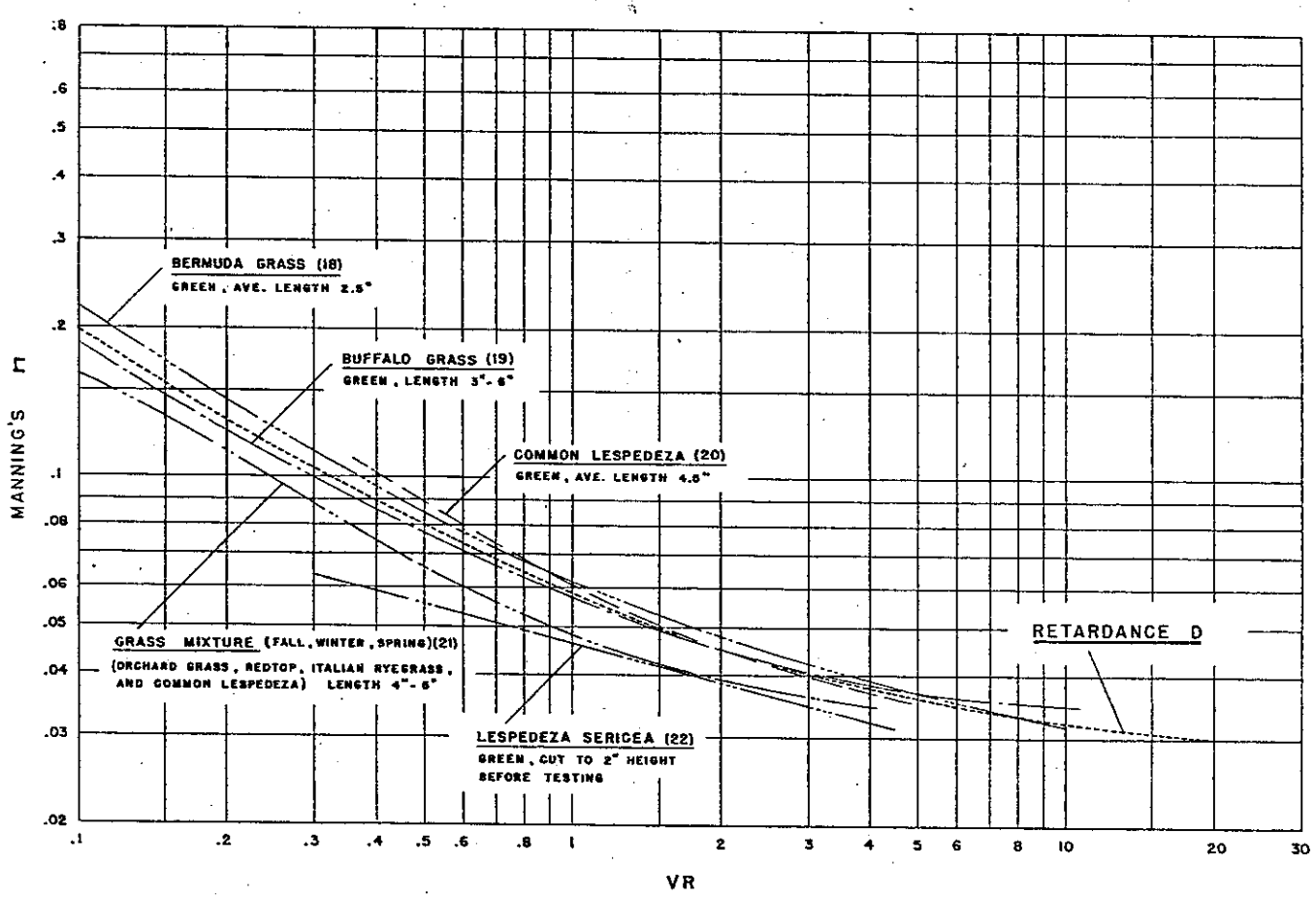


Figure 22.--Experimental results related to retardance D (low vegetal retardance).

TABLE #1

Adapted Species	Sandy Soils	Loam Soils	Clayey Soils	Wet Soils	Saline Soils	Seeding Rate 1/	Seeding Date
MLRA D-30, 31							
Bermudagrass	X	X	X		X	6	June-July
MLRA D-35, 36, 39							
Creeping meadow foxtail				X		8	Aug.-Sept. 5
Kentucky bluegrass		X	X			10	" "
Pubescent wheatgrass	X	X				16	" "
Reed canarygrass				X		10	" "
Streambank wheatgrass	X	X				12	" "
Tall fescue		X	X		X	16	" "
Western wheatgrass		X	X	X	X	16	" "
MLRA D-38							
Creeping meadow foxtail				X		8	Aug.-Sept. 5
Lehman lovegrass	X	X				2	Dec. thru July
Tall fescue		X	X		X	16	Sept.
Western wheatgrass		X	X	X	X	16	June or Oct.
MLRA D-40							
Bermudagrass	X	X	X		X	6	Jan.-June
Lehman lovegrass	X	X				2	Dec. thru July
MLRA D-41, 42							
Bermudagrass	X	X	X		X	6	June
Lehman lovegrass	X	X				2	Dec. thru July
Tall fescue		X	X		X	16	Sept.

1/ Pounds pure live seed per acre

ENGINEERING WORKSHOP
FEBRUARY 5-7, 1990
PHOENIX, AZ.

Dave Seery
Area I Biologist
Flagstaff, Az.

Nature of work determines they.

ENVIRONMENTAL CONCERNS

do first — I.1. ENVIRONMENTAL EVALUATION (E.E.) (SCS does this)

E.A. Environmental Assessment, public meetings etc. (BIA does a lot of this)

E.I.S. Environmental Impact Statement (BLA uses this)

E.E. is first line of protection. It may dictate where and what. Checklist,

Is it done? Who did it?

Be able to justify effects.

2. WETLANDS — very sensitive area, renewed interest in wetlands. 87% wetlands on private. President Bush said that there will be no loss of wetlands. Fish & wildlife Service is making study.

[D] hydric soils
hydrophilic plants (vegetation) } may become wetland in 7 days, no man made
3 days

3. ENDANGERED SPECIES — see red book, Federal List of endangered species

4. RIPARIAN AREAS — floodplains (wetlands)
Controlled flow has stopped flooding, which also stopped the spread of Cottonwood trees.

5. ARCHAEOLIGICAL, [CULTURAL AND HISTORICAL] - pottery, burials, sites etc. 30 pieces of pottery in 50 ft circle.

[D] historical > 50 years old.

II. 1. LANDSCAPE ARCHITECTURE - give some thought to these:
Less obtrusive installations.

2. EQUIPMENT YARDS AND PARKING LOTS - budget in
Disc and seed

3. TEMPORARY WATER POLLUTION - SEDIMENT, AND CHEMICAL

Fix, truck in washing in each cow bell lake

ENVIRONMENTAL EVALUATION

Cooperator/Sponsor/Land User/Farm/Field # _____

Measure/Practice/RMS and Component Practices: _____

Office (AO, FO, SO, etc.) _____

Prepared by: _____ Title: _____ Date: _____

Concurred: _____, District Conservationist Date: _____

Code the following items: + = beneficial effects: 0 = no effects
- = adverse effects: NA = does not apply

ENVIRONMENTAL FACTORS	EFFECT 1/ Short Term	EFFECT 1/ Long Term	NOTES Explain All Codes (+, -, 0, NA)
1. EROSION AND SEDIMENTATION			
2. WATER			
a) Water table alterations			
b) Changes in water quality			
c) Changes in water quantity			
d) Changes in stream courses			
e) Increase or abatement of water erosion			
3. CHANGES IN LAND USE			
a) Prime & unique farmland losses			
b) Any other land-use changes			
4. CHANGES IN AIR QUALITY			
a) Includes construction dust & smoke			
b) Includes induced wind erosion or abatement			
5. WILDLIFE HABITAT			
a) Stream courses			
b) Upland habitat components			
c) Wetland habitat components 3/			
d) Aquatic habitat components			
6. FLOODPLAIN 2/			
7. THREATENED AND ENDANGERED SPECIES			
a) Plants			
b) Animals			
8. CULTURAL RESOURCES			
a) Prehistoric site impacts			
b) Historic site impacts			
c) National Register site impacts			
9. LANDSCAPES			
a) Changes due to activities			
10. OTHER			

1/ Short Term (installation period) Long Term (project or practice life)
2/ See: General Manual 190, Sec 410.25
3/ See: General Manual 190, Sec 410.26

(2) The net drop does not exceed 30 feet.

(3) The unit discharge per foot of width does not exceed 100 cfs.

(4) The maximum inlet capacity does not exceed 4,000 cfs.

D. Grouted rock riprap chutes of standard layout with straight inlets and subject to seasonal flows only where:

(1) The outlet channel grade is stable against degradation at designed flow.

(2) The net drop does not exceed 10 feet.

(3) The unit discharge per foot of width does not exceed 40 cfs.
(4) The maximum inlet capacity does not exceed 4,000 cfs.

III. Channels in earth or systems of such channels with or without associated levees, dikes, or spoil banks and their appurtenances where:

A. Bankfull discharge capacity does not exceed 6,000 cfs, except that the bankfull discharge capacity shall not exceed 4,000 cfs when grade stabilization structures are used in the system.

B. The flow is subcritical and the mean velocity at the maximum possible stage is equal to or less than 1.7 times the square root of the mean depth at the same stage (elevation of the water surface) for the as-built condition. (Lowest reasonable value of "n.")

C. The channel or system of channels has a free unobstructed outlet.

IV. Streambank protection (mechanical)~~—riprap~~, including associated grading, excavation, fill, and filters necessary.

A. Channel bankfull discharge capacity (low bank) does not exceed 15,000 cfs.

B. Watershed drainage area does not exceed 400 square miles.

V. Culverts.

Precast pipe, singular or multiple conduit with a maximum inside diameter of 8 feet or the equivalent in cases of metal pipe arches.

ENGINEERING JOB APPROVAL CHART
SOIL CONSERVATION SERVICE
ARIZONA

EXHIBIT B

Practice Code	Job Type	Controlling Factors	Units	Job Class I	II	III	IV	V
		On-site Water Supply or Sewage Treatment	Daily Design Capacity (People)	---	---	50	100	200
		Off-site Public Water Supply and Sewage Treatment	Daily Design Capacity (People)	25	50	100	200	400
566	Recreation Land Grading and Shaping	Area Treated	Acres	5	10	10	ALL	ALL
568	Recreation Trail and Walkway	Length	Miles	1	2	5	10	ALL
554	Regulating Water in a Drainage System	Area Drained	Acres	40	80	320	640	ALL
555	Rock Barrier	Height of Barrier	Feet	---	4	6	8	ALL
557	Row Arrangement	Area	Acres	20	40	100	160	ALL
570	Runoff Management System	Area Benefited	Acres	40	80	160	320	ALL
572	Spoilbank Spreading	Length	Feet	3000	5500	ALL	ALL	ALL
574	Spring Development	Quantity of Flow	gpm	25	50	100	500	ALL
580	Streambank Protection	Beaches and Shoreline Revetments, bulkheads and groins - height above mean tide or mean high water	Feet	---	---	---	2	3
		Streambanks, Vegetation	Mi.^{2}	---	50	100	ALL	ALL
		Streambanks, Mechanical Protection [redacted] Capacity (Bank Full)	cfs	100	500	1000	1500	2000
		Drainage Area	Mi.^{2}	10	25	50	75	100
		Channel Depth (Low Bank)	Feet	3	4	6	8	10
584	Stream Channel Stabilization	Design Capacity	cfs	50	100	200	300	500
606	Subsurface Drain	[illegible]	[illegible]	[illegible]	80	320	640	ALL
607	Surface Drainage Field Ditch	Design Capacity	cfs	5	10	ALL	ALL	ALL
609	Surface Drain, Main or Lateral	Design Capacity	cfs	10	100	200	300	500
600	Terrace	Total Area in System	Acres	500	100	2000	ALL	ALL
		Drainage Area per Terrace	Acres	40	80	160	ALL	ALL

(AZ210-500-545), February 1985

Thursday
November 13, 1986

Federal Register

Part II

Department of Defense

Corps of Engineers, Department of the Army

33 CFR Parts 320 through 330

Regulatory Programs of the Corps of Engineers; Final Rule

Note 1.—The term "he" and its derivatives used in these regulations are generic and should be considered as applying to both male and female.

List of Subjects

33 CFR Part 320

Environmental protection, Intergovernmental relations, Navigation, Water pollution control, Waterways.

33 CFR Part 321

Dams, Intergovernmental relations, Navigation, Waterways.

33 CFR Part 322

Continental shelf, Electric power, Navigation, Water pollution control, Waterways.

33 CFR Part 323

Navigation, Water pollution control, Waterways.

33 CFR Part 324

Water pollution control.

33 CFR Part 325

Administrative practice and procedure, Intergovernmental relations, Environmental protection, Navigation, Water pollution control, Waterways.

33 CFR Part 326

Investigations, Intergovernmental relations, Law enforcement, Navigation, Water pollution control, Waterways.

33 CFR Part 327

Administrative practice and procedure, Navigation, Water pollution control, Waterways.

33 CFR Part 328

Navigation, Water pollution control, Waterways.

33 CFR Part 329

Waterways.

33 CFR Part 330

Navigation, Water pollution control, Waterways.

Dated: November 4, 1986.

Robert K. Dawson,

Assistant Secretary of the Army (Civil Works).

Accordingly, the Department of the Army is revising 33 CFR Parts 320, 321, 322, 323, 324, 325, 326, 327, 329, and 330 and adding Part 328 to read as follows:

PART 320—GENERAL REGULATORY POLICIES

Sec.
320.1 Purpose and scope.
320.2 Authorities to issue permits.
320.3 Related laws.
320.4 General policies for evaluating permit applications.

Authority: 33 U.S.C. 401 *et seq.*; 33 U.S.C. 1344; 33 U.S.C. 1413.

§ 320.1 Purpose and scope.

(a) *Regulatory approach of the Corps of Engineers.* (1) The U.S. Army Corps of Engineers has been involved in regulating certain activities in the nation's waters since 1890. Until 1968, the primary thrust of the Corps' regulatory program was the protection of navigation. As a result of several new laws and judicial decisions, the program has evolved to one involving the consideration of the full public interest by balancing the favorable impacts against the detrimental impacts. This is known as the "public interest review." The program is one which reflects the national concerns for both the protection and utilization of important resources.

(2) The Corps is a highly decentralized organization. Most of the authority for administering the regulatory program has been delegated to the thirty-six district engineers and eleven division engineers. If a district or division engineer makes a final decision on a permit application in accordance with the procedures and authorities contained in these regulations (33 CFR Parts 320–330), there is no administrative appeal of that decision.

(3) The Corps seeks to avoid unnecessary regulatory controls. The general permit program described in 33 CFR Parts 325 and 330 is the primary method of eliminating unnecessary federal control over activities which do not justify individual control or which are adequately regulated by another agency.

(4) The Corps is neither a proponent nor opponent of any permit proposal. However, the Corps believes that applicants are due a timely decision. Reducing unnecessary paperwork and delays is a continuing Corps goal.

(5) The Corps believes that state and federal regulatory programs should complement rather than duplicate one another. The Corps uses general permits, joint processing procedures, interagency review, coordination, and authority transfers (where authorized by law) to reduce duplication.

(6) The Corps has authorized its district engineers to issue formal determinations concerning the applicability of the Clean Water Act or the Rivers and Harbors Act of 1899 to activities or tracts of land and the applicability of general permits or statutory exemptions to proposed activities. A determination pursuant to this authorization shall constitute a Corps final agency action. Nothing contained in this section is intended to affect any authority EPA has under the Clean Water Act.

(b) *Types of activities regulated.* This Part and the Parts that follow (33 CFR Parts 321–330) prescribe the statutory authorities, and general and special policies and procedures applicable to the review of applications for Department of the Army (DA) permits for controlling certain activities in waters of the United States or the oceans. This part identifies the various federal statutes which require that DA permits be issued before these activities can be lawfully undertaken; and related Federal laws and the general policies applicable to the review of those activities. Parts 321–324 and 330 address special policies and procedures applicable to the following specific classes of activities:

(1) Dams or dikes in navigable waters of the United States (Part 321);

(2) Other structures or work including excavation, dredging, and/or disposal activities, in navigable waters of the United States (Part 322);

(3) Activities that alter or modify the course, condition, location, or capacity of a navigable water of the United States (Part 322);

(4) Construction of artificial islands, installations, and other devices on the outer continental shelf (Part 322);

(5) Discharges of dredged or fill material into waters of the United States (Part 323);

(6) Activities involving the transportation of dredged material for the purpose of disposal in ocean waters (Part 324); and

(7) Nationwide general permits for certain categories of activities (Part 330).

(c) *Forms of authorization.* DA permits for the above described activities are issued under various forms of authorization. These include individual permits that are issued following a review of individual applications and general permits that authorize a category or categories of activities in specific geographical regions or nationwide. The term "general permit" as used in these regulations (33 CFR Parts 320–330) refers to both those regional permits issued by district or division engineers on a regional basis and to nationwide permits which are issued by the Chief of Engineers through publication in the Federal Register and are applicable throughout the nation. The nationwide permits are found in 33 CFR Part 330. If an activity is covered by a general permit, an application for a DA permit

does not have to be made. In such cases, a person must only comply with the conditions contained in the general permit to satisfy requirements of law for a DA permit. In certain cases pre-notification may be required before initiating construction. (See 33 CFR 330.7)

(d) *General Instructions.* General policies for evaluating permit applications are found in this part. Special policies that relate to particular activities are found in Parts 321 through 324. The procedures for processing individual permits and general permits are contained in 33 CFR Part 325. The terms "navigable waters of the United States" and "waters of the United States" are used frequently throughout these regulations, and it is important from the outset that the reader understand the difference between the two. "Navigable waters of the United States" are defined in 33 CFR Part 329. These are waters that are navigable in the traditional sense where permits are required for certain work or structures pursuant to Sections 9 and 10 of the Rivers and Harbors Act of 1899. "Waters of the United States" are defined in 33 CFR Part 328. These waters include more than navigable waters of the United States and are the waters where permits are required for the discharge of dredged or fill material pursuant to Section 404 of the Clean Water Act.

§ 320.2 Authorities to issue permits.

(a) Section 9 of the Rivers and Harbors Act, approved March 3, 1899 (33 U.S.C. 401) (hereinafter referred to as section 9), prohibits the construction of any dam or dike across any navigable water of the United States in the absence of Congressional consent and approval of the plans by the Chief of Engineers and the Secretary of the Army. Where the navigable portions of the waterbody lie wholly within the limits of a single state, the structure may be built under authority of the legislature of that state if the location and plans or any modification thereof are approved by the Chief of Engineers and by the Secretary of the Army. The instrument of authorization is designated a permit (See 33 CFR Part 321.) Section 9 also pertains to bridges and causeways but the authority of the Secretary of the Army and Chief of Engineers with respect to bridges and causeways was transferred to the Secretary of Transportation under the Department of Transportation Act of October 15, 1966 (49 U.S.C. 1155(g)(A)). A DA permit pursuant to section 404 of the Clean Water Act is required for the discharge of dredged or fill material into waters of the United States associated with bridges and causeways. (See 33 CFR Part 323.)

(b) Section 10 of the Rivers and Harbors Act approved March 3, 1899, (33 U.S.C. 403) (hereinafter referred to as section 10), prohibits the unauthorized obstruction or alteration of any navigable water of the United States. The construction of any structure in or over any navigable water of the United States, the excavating from or depositing of material in such waters, or the accomplishment of any other work affecting the course, location, condition, or capacity of such waters is unlawful unless the work has been recommended by the Chief of Engineers and authorized by the Secretary of the Army. The instrument of authorization is designated a permit. The authority of the Secretary of the Army to prevent obstructions to navigation in navigable waters of the United States was extended to artificial islands, installations, and other devices located on the seabed, to the seaward limit of the outer continental shelf, by section 4(f) of the Outer Continental Shelf Lands Act of 1953 as amended (43 U.S.C. 1333(e)). (See 33 CFR Part 322.)

(c) Section 11 of the Rivers and Harbors Act approved March 3, 1899, (33 U.S.C. 404), authorizes the Secretary of the Army to establish harbor lines channelward of which no piers, wharves, bulkheads, or other works may be extended or deposits made without approval of the Secretary of the Army. Effective May 27, 1970, permits for work shoreward of those lines must be obtained in accordance with section 10 and, if applicable, section 404 of the Clean Water Act (see § 320.4(o) of this Part).

(d) Section 13 of the Rivers and Harbors Act approved March 3, 1899, (33 U.S.C. 407), provides that the Secretary of the Army, whenever the Chief of Engineers determines that anchorage and navigation will not be injured thereby, may permit the discharge of refuse into navigable waters. In the absence of a permit, such discharge of refuse is prohibited. While the prohibition of this section, known as the Refuse Act, is still in effect, the permit authority of the Secretary of the Army has been superseded by the permit authority provided the Administrator, Environmental Protection Agency (EPA), and the states under sections 402 and 405 of the Clean Water Act, (33 U.S.C. 1342 and 1345). (See 40 CFR Parts 124 and 125.)

(e) Section 14 of the Rivers and Harbors Act approved March 3, 1899, (33 U.S.C. 408), provides that the Secretary of the Army, on the recommendation of the Chief of Engineers, may grant permission for the temporary occupation or use of any sea wall, bulkhead, jetty, dike, levee, wharf, pier, or other work built by the United States. This permission will be granted by an appropriate real estate instrument in accordance with existing real estate regulations.

(f) Section 404 of the Clean Water Act (33 U.S.C. 1344) (hereinafter referred to as section 404) authorizes the Secretary of the Army, acting through the Chief of Engineers, to issue permits, after notice and opportunity for public hearing, for the discharge of dredged or fill material into the waters of the United States at specified disposal sites. (See 33 CFR Part 323.) The selection and use of disposal sites will be in accordance with guidelines developed by the Administrator of EPA in conjunction with the Secretary of the Army and published in 40 CFR Part 230. If these guidelines prohibit the selection or use of a disposal site, the Chief of Engineers shall consider the economic impact on navigation and anchorage of such a prohibition in reaching his decision. Furthermore, the Administrator can deny, prohibit, restrict or withdraw the use of any defined area as a disposal site whenever he determines, after notice and opportunity for public hearing and after consultation with the Secretary of the Army, that the discharge of such materials into such areas will have an unacceptable adverse effect on municipal water supplies, shellfish beds and fishery areas, wildlife, or recreational areas. (See 40 CFR Part 225).

(g) Section 103 of the Marine Protection, Research and Sanctuaries Act of 1972, as amended (33 U.S.C. 1413) (hereinafter referred to as section 103), authorizes the Secretary of the Army, acting through the Chief of Engineers, to issue permits, after notice and opportunity for public hearing, for the transportation of dredged material for the purpose of disposal in the ocean where it is determined that the disposal will not unreasonably degrade or endanger human health, welfare, or amenities, or the marine environment, ecological systems, or economic potentialities. The selection of disposal sites will be in accordance with criteria developed by the Administrator of the EPA in consultation with the Secretary of the Army and published in 40 CFR Parts 220–229. However, similar to the EPA Administrator's limiting authority cited in paragraph (f) of this section, the Administrator can prevent the issuance of a permit under this authority if he

finds that the disposal of the material will result in an unacceptable adverse impact on municipal water supplies, shellfish beds, wildlife, fisheries, or recreational areas. (See 33 CFR Part 324).

§ 220.3 Related laws.

(a) Section 401 of the Clean Water Act (33 U.S.C. 1341) requires any applicant for a federal license or permit to conduct any activity that may result in a discharge of a pollutant into waters of the United States to obtain a certification from the State in which the discharge originates or would originate, or, if appropriate, from the interstate water pollution control agency having jurisdiction over the affected waters at the point where the discharge originates or would originate, that the discharge will comply with the applicable effluent limitations and water quality standards. A certification obtained for the construction of any facility must also pertain to the subsequent operation of the facility.

(b) Section 307(c) of the Coastal Zone Management Act of 1972, as amended (16 U.S.C. 1456(c)), requires federal agencies conducting activities, including development projects, directly affecting a state's coastal zone, to comply to the maximum extent practicable with an approved state coastal zone management program. Indian tribes doing work on federal lands will be treated as a federal agency for the purpose of the Coastal Zone Management Act. The Act also requires any non-federal applicant for a federal license or permit to conduct an activity affecting land or water uses in the state's coastal zone to furnish a certification that the proposed activity will comply with the state's coastal zone management program. Generally, no permit will be issued until the state has concurred with the non-federal applicant's certification. This provision becomes effective upon approval by the Secretary of Commerce of the state's coastal zone management program. (See 15 CFR Part 930.)

(c) Section 302 of the Marine Protection, Research and Sanctuaries Act of 1972, as amended (16 U.S.C. 1432), authorizes the Secretary of Commerce, after consultation with other interested federal agencies and with the approval of the President, to designate as marine sanctuaries those areas of the ocean waters, of the Great Lakes and their connecting waters, or of other coastal waters which he determines necessary for the purpose of preserving or restoring such areas for their conservation, recreational, ecological, or aesthetic values. After designating such an area, the Secretary of Commerce shall issue regulations to control any activities within the area. Activities in the sanctuary authorized under other authorities are valid only if the Secretary of Commerce certifies that the activities are consistent with the purposes of Title III of the Act and can be carried out within the regulations for the sanctuary.

(d) The National Environmental Policy Act of 1969 (42 U.S.C. 4321–4347) declares the national policy to encourage a productive and enjoyable harmony between man and his environment. Section 102 of that Act directs that "to the fullest extent possible: (1) The policies, regulations, and public laws of the United States shall be interpreted and administered in accordance with the policies set forth in this Act, and (2) all agencies of the Federal Government shall * * * insure that presently unquantified environmental amenities and values may be given appropriate consideration in decision-making along with economic and technical considerations * * *". (See Appendix B of 33 CFR Part 325.)

(e) The Fish and Wildlife Act of 1956 (16 U.S.C. 742a, *et seq.*), the Migratory Marine Game Fish Act (16 U.S.C. 760c–760g), the Fish and Wildlife Coordination Act (16 U.S.C. 661–666c) and other acts express the will of Congress to protect the quality of the aquatic environment as it affects the conservation, improvement and enjoyment of fish and wildlife resources. Reorganization Plan No. 4 of 1970 transferred certain functions, including certain fish and wildlife-water resources coordination responsibilities, from the Secretary of the Interior to the Secretary of Commerce. Under the Fish and Wildlife Coordination Act and Reorganization Plan No. 4, any federal agency that proposes to control or modify any body of water must first consult with the United States Fish and Wildlife Service or the National Marine Fisheries Service, as appropriate, and with the head of the appropriate state agency exercising administration over the wildlife resources of the affected state.

(f) The Federal Power Act of 1920 (16 U.S.C. 791a *et seq.*), as amended, authorizes the Federal Energy Regulatory Agency (FERC) to issue licenses for the construction and the operation and maintenance of dams, water conduits, reservoirs, power houses, transmission lines, and other physical structures of a hydro-power project. However, where such structures will affect the navigable capacity of any navigable water of the United States (as defined in 16 U.S.C. 796), the plans for the dam or other physical structures affecting navigation must be approved by the Chief of Engineers and the Secretary of the Army. In such cases the interests of navigation should normally be protected by a DA recommendation to FERC for the inclusion of appropriate provisions in the FERC license rather than the issuance of a separate DA permit under 33 U.S.C. 401 *et seq.* As to any other activities in navigable waters not constituting construction and the operation and maintenance of physical structures licensed by FERC under the Federal Power Act of 1920, as amended, the provisions of 33 U.S.C. 401 *et seq.* remain fully applicable. In all cases involving the discharge of dredged or fill material into waters of the United States or the transportation of dredged material for the purpose of disposal in ocean waters, section 404 or section 103 will be applicable.

(g) The National Historic Preservation Act of 1966 (16 U.S.C. 470) created the Advisory Council on Historic Preservation to advise the President and Congress on matters involving historic preservation. In performing its function the Council is authorized to review and comment upon activities licensed by the Federal Government which will have an effect upon properties listed in the National Register of Historic Places, or eligible for such listing. The concern of Congress for the preservation of significant historical sites is also expressed in the Preservation of Historical and Archeological Data Act of 1974 (16 U.S.C. 469 *et seq.*), which amends the Act of June 27, 1960. By this Act, whenever a federal construction project or federally licensed project, activity, or program alters any terrain such that significant historical or archeological data is threatened, the Secretary of the Interior may take action necessary to recover and preserve the data prior to the commencement of the project.

(h) The Interstate Land Sales Full Disclosure Act (15 U.S.C. 1701 *et seq.*) prohibits any developer or agent from selling or leasing any lot in a subdivision (as defined in 15 U.S.C. 1701(3)) unless the purchaser is furnished in advance a printed property report containing information which the Secretary of Housing and Urban Development may, by rules or regulations, require for the protection of purchasers. In the event the loan question is part of a project that requires DA authorization, the property report is required by Housing and Urban Development regulation to state whether

or not a permit for the development has been applied for, issued, or denied by the Corps of Engineers under section 10 or section 404. The property report is also required to state whether or not any enforcement action has been taken as a consequence of non-application for or denial of such permit.

(i) The Endangered Species Act (16 U.S.C. 1531 *et seq.*) declares the intention of the Congress to conserve threatened and endangered species and the ecosystems on which those species depend. The Act requires that federal agencies, in consultation with the U.S. Fish and Wildlife Service and the National Marine Fisheries Service, use their authorities in furtherance of its purposes by carrying out programs for the conservation of endangered or threatened species, and by taking such action necessary to insure that any action authorized, funded, or carried out by the Agency is not likely to jeopardize the continued existence of such endangered or threatened species or result in the destruction or adverse modification of habitat of such species which is determined by the Secretary of the Interior or Commerce, as appropriate, to be critical. (See 50 CFR Part 17 and 50 CFR Part 402.)

(j) The Deepwater Port Act of 1974 (33 U.S.C. 1501 *et seq.*) prohibits the ownership, construction, or operation of a deepwater port beyond the territorial seas without a license issued by the Secretary of Transportation. The Secretary of Transportation may issue such a license to an applicant if he determines, among other things, that the construction and operation of the deepwater port is in the national interest and consistent with national security and other national policy goals and objectives. An application for a deepwater port license constitutes an application for all federal authorizations required for the ownership, construction, and operation of a deepwater port, including applications for section 10, section 404 and section 103 permits which may also be required pursuant to the authorities listed in section 320.2 and the policies specified in section 320.4 of this Part.

(k) The Marine Mammal Protection Act of 1972 (16 U.S.C. 1361 *et seq.*) expresses the intent of Congress that marine mammals be protected and encouraged to develop in order to maintain the health and stability of the marine ecosystem. The Act imposes a perpetual moratorium on the harassment, hunting, capturing, or killing of marine mammals and on the importation of marine mammals and marine mammal products without a permit from either the Secretary of the Interior or the Secretary of Commerce, depending upon the species of marine mammal involved. Such permits may be issued only for purposes of scientific research and for public display if the purpose is consistent with the policies of the Act. The appropriate Secretary is also empowered in certain restricted circumstances to waive the requirements of the Act.

(l) Section 7(a) of the Wild and Scenic Rivers Act (16 U.S.C. 1278 *et seq.*) provides that no department or agency of the United States shall assist by loan, grant, license, or otherwise in the construction of any water resources project that would have a direct and adverse effect on the values for which such river was established, as determined by the Secretary charged with its administration.

(m) The Ocean Thermal Energy Conversion Act of 1980, (42 U.S.C. section 9101 *et seq.*) establishes a licensing regime administered by the Administrator of NOAA for the ownership, construction, location, and operation of ocean thermal energy conversion (OTEC) facilities and plantships. An application for an OTEC license filed with the Administrator constitutes an application for all federal authorizations required for ownership, construction, location, and operation of an OTEC facility or plantship, except for certain activities within the jurisdiction of the Coast Guard. This includes applications for section 10, section 404, section 103 and other DA authorizations which may be required.

(n) Section 402 of the Clean Water Act authorizes EPA to issue permits under procedures established to implement the National Pollutant Discharge Elimination System (NPDES) program. The administration of this program can be, and in most cases has been, delegated to individual states. Section 402(b)(6) states that no NPDES permit will be issued if the Chief of Engineers, acting for the Secretary of the Army and after consulting with the U.S. Coast Guard, determines that navigation and anchorage in any navigable water will be substantially impaired as a result of a proposed activity.

(o) The National Fishing Enhancement Act of 1984 (Pub. L. 98-623) provides for the development of a National Artificial Reef Plan to promote and facilitate responsible and effective efforts to establish artificial reefs. The Act establishes procedures to be followed by the Corps in issuing DA permits for artificial reefs. The Act also establishes the liability of the permittee and the United States. The Act further creates a civil penalty for violation of any provision of a permit issued for an artificial reef.

§ 320.4 General policies for evaluating permit applications.

The following policies shall be applicable to the review of all applications for DA permits. Additional policies specifically applicable to certain types of activities are identified in 33 CFR Parts 321-324.

(a) *Public Interest Review.* (1) The decision whether to issue a permit will be based on an evaluation of the probable impacts, including cumulative impacts, of the proposed activity and its intended use on the public interest. Evaluation of the probable impact which the proposed activity may have on the public interest requires a careful weighing of all those factors which become relevant in each particular case. The benefits which reasonably may be expected to accrue from the proposal must be balanced against its reasonably foreseeable detriments. The decision whether to authorize a proposal, and if so, the conditions under which it will be allowed to occur, are therefore determined by the outcome of this general balancing process. That decision should reflect the national concern for both protection and utilization of important resources. All factors which may be relevant to the proposal must be considered including the cumulative effects thereof: among those are conservation, economics, aesthetics, general environmental concerns, wetlands, historic properties, fish and wildlife values, flood hazards, floodplain values, land use, navigation, shore erosion and accretion, recreation, water supply and conservation, water quality, energy needs, safety, food and fiber production, mineral needs, considerations of property ownership and, in general, the needs and welfare of the people. For activities involving 404 discharges, a permit will be denied if the discharge that would be authorized by such permit would not comply with the Environmental Protection Agency's 404(b)(1) guidelines. Subject to the preceding sentence and any other applicable guidelines and criteria (see §§ 320.2 and 320.3), a permit will be granted unless the district engineer determines that it would be contrary to the public interest.

(2) The following general criteria will be considered in the evaluation of every application:

(i) The relative extent of the public and private need for the proposed structure or work:

(ii) Where there are unresolved conflicts as to resource use, the practicability of using reasonable alternative locations and methods to accomplish the objective of the proposed structure or work; and

(iii) The extent and permanence of the beneficial and/or detrimental effects which the proposed structure or work is likely to have on the public and private uses to which the area is suited.

(3) The specific weight of each factor is determined by its importance and relevance to the particular proposal. Accordingly, how important a factor is and how much consideration it deserves will vary with each proposal. A specific factor may be given great weight on one proposal, while it may not be present or as important on another. However, full consideration and appropriate weight will be given to all comments, including those of federal, state, and local agencies, and other experts on matters within their expertise.

(b) *Effect on wetlands.* (1) Most wetlands constitute a productive and valuable public resource, the unnecessary alteration or destruction of which should be discouraged as contrary to the public interest. For projects to be undertaken or partially or entirely funded by a federal, state, or local agency, additional requirements on wetlands considerations are stated in Executive Order 11990, dated 24 May 1977.

(2) Wetlands considered to perform functions important to the public interest include:

(i) Wetlands which serve significant natural biological functions, including food chain production, general habitat and nesting, spawning, rearing and resting sites for aquatic or land species;

(ii) Wetlands set aside for study of the aquatic environment or as sanctuaries or refuges;

(iii) Wetlands the destruction or alteration of which would affect detrimentally natural drainage characteristics, sedimentation patterns, salinity distribution, flushing characteristics, current patterns, or other environmental characteristics;

(iv) Wetlands which are significant in shielding other areas from wave action, erosion, or storm damage. Such wetlands are often associated with barrier beaches, islands, reefs and bars;

(v) Wetlands which serve as valuable storage areas for storm and flood waters;

(vi) Wetlands which are ground water discharge areas that maintain minimum baseflows important to aquatic resources and those which are prime natural recharge areas;

(vii) Wetlands which serve significant water purification functions; and

(viii) Wetlands which are unique in nature or scarce in quantity to the region or local area.

(3) Although a particular alteration of a wetland may constitute a minor change, the cumulative effect of numerous piecemeal changes can result in a major impairment of wetland resources. Thus, the particular wetland site for which an application is made will be evaluated with the recognition that it may be part of a complete and interrelated wetland area. In addition, the district engineer may undertake, where appropriate, reviews of particular wetland areas in consultation with the Regional Director of the U. S. Fish and Wildlife Service, the Regional Director of the National Marine Fisheries Service of the National Oceanic and Atmospheric Administration, the Regional Administrator of the Environmental Protection Agency, the local representative of the Soil Conservation Service of the Department of Agriculture, and the head of the appropriate state agency to assess the cumulative effect of activities in such areas.

(4) No permit will be granted which involves the alteration of wetlands identified as important by paragraph (b)(2) of this section or because of provisions of paragraph (b)(3), of this section unless the district engineer concludes, on the basis of the analysis required in paragraph (a) of this section, that the benefits of the proposed alteration outweigh the damage to the wetlands resource. In evaluating whether a particular discharge activity should be permitted, the district engineer shall apply the section 404(b)(1) guidelines (40 CFR Part 230. 10(a) (1), (2), (3)).

(5) In addition to the policies expressed in this subpart, the Congressional policy expressed in the Estuary Protection Act, Pub. L. 90-454, and state regulatory laws or programs for classification and protection of wetlands will be considered.

(c) *Fish and wildlife.* In accordance with the Fish and Wildlife Coordination Act (paragraph 320.3(e) of this section) district engineers will consult with the Regional Director, U.S. Fish and Wildlife Service, the Regional Director, National Marine Fisheries Service, and the head of the agency responsible for fish and wildlife for the state in which work is to be performed, with a view to the conservation of wildlife resources by prevention of their direct and indirect loss and damage due to the activity proposed in a permit application. The Army will give full consideration to the views of those agencies on fish and wildlife matters in deciding on the issuance, denial, or conditioning of individual or general permits.

(d) *Water quality.* Applications for permits for activities which may adversely affect the quality of waters of the United States will be evaluated for compliance with applicable effluent limitations and water quality standards, during the construction and subsequent operation of the proposed activity. The evaluation should include the consideration of both point and non-point sources of pollution. It should be noted, however, that the Clean Water Act assigns responsibility for control of non-point sources of pollution to the states. Certification of compliance with applicable effluent limitations and water quality standards required under provisions of section 401 of the Clean Water Act will be considered conclusive with respect to water quality considerations unless the Regional Administrator, Environmental Protection Agency (EPA), advises of other water quality aspects to be taken into consideration.

(e) *Historic, cultural, scenic, and recreational values.* Applications for DA permits may involve areas which possess recognized historic, cultural, scenic, conservation, recreational or similar values. Full evaluation of the general public interest requires that due consideration be given to the effect which the proposed structure or activity may have on values such as those associated with wild and scenic rivers, historic properties and National Landmarks, National Rivers, National Wilderness Areas, National Seashores, National Recreation Areas, National Lakeshores, National Parks, National Monuments, estuarine and marine sanctuaries, archeological resources, including Indian religious or cultural sites, and such other areas as may be established under federal or state law for similar and related purposes. Recognition of those values is often reflected by state, regional, or local land use classifications, or by similar federal controls or policies. Action on permit applications should, insofar as possible, be consistent with, and avoid significant adverse effects on the values or purposes for which those classifications, controls, or policies were established.

(f) *Effects on limits of the territorial sea.* Structures or work affecting coastal waters may modify the coast line or base line from which the territorial sea is measured for purposes of the Submerged Lands Act and international law. Generally, the coast line or base line is the line of ordinary low water on

the mainland; however, there are exceptions where there are islands or lowtide elevations offshore (the Submerged Lands Act, 43 U.S.C. 1301(a) and *United States v. California*, 381 U.S.C. 139 (1965), 382 U.S. 448 (1966)). Applications for structures or work affecting coastal waters will therefore be reviewed specifically to determine whether the coast line or base line might be altered. If it is determined that such a change might occur, coordination with the Attorney General and the Solicitor of the Department of the Interior is required before final action is taken. The district engineer will submit a description of the proposed work and a copy of the plans to the Solicitor, Department of the Interior, Washington, DC 20240, and request his comments concerning the effects of the proposed work on the outer continental rights of the United States. These comments will be included in the administrative record of the application. After completion of standard processing procedures, the record will be forwarded to the Chief of Engineers. The decision on the application will be made by the Secretary of the Army after coordination with the Attorney General.

(g) *Consideration of property ownership.* Authorization of work or structures by DA does not convey a property right, nor authorize any injury to property or invasion of other rights.

(1) An inherent aspect of property ownership is a right to reasonable private use. However, this right is subject to the rights and interests of the public in the navigable and other waters of the United States, including the federal navigation servitude and federal regulation for environmental protection.

(2) Because a landowner has the general right to protect property from erosion, applications to erect protective structures will usually receive favorable consideration. However, if the protective structure may cause damage to the property of others, adversely affect public health and safety, adversely impact floodplain or wetland values, or otherwise appears contrary to the public interest, the district engineer will so advise the applicant and inform him of possible alternative methods of protecting his property. Such advice will be given in terms of general guidance only so as not to compete with private engineering firms nor require undue use of government resources.

(3) A riparian landowner's general right of access to navigable waters of the United States is subject to the similar rights of access held by nearby riparian landowners and to the general public's right of navigation on the water surface. In the case of proposals which create undue interference with access to, or use of, navigable waters, the authorization will generally be denied.

(4) Where it is found that the work for which a permit is desired is in navigable waters of the United States (see 33 CFR Part 329) and may interfere with an authorized federal project, the applicant should be apprised in writing of the fact and of the possibility that a federal project which may be constructed in the vicinity of the proposed work might necessitate its removal or reconstruction. The applicant should also be informed that the United States will in no case be liable for any damage or injury to the structures or work authorized by Sections 9 or 10 of the Rivers and Harbors Act of 1899 or by section 404 of the Clean Water Act which may be caused by, or result from, future operations undertaken by the Government for the conservation or improvement of navigation or for other purposes, and no claims or right to compensation will accrue from any such damage.

(5) Proposed activities in the area of a federal project which exists or is under construction will be evaluated to insure that they are compatible with the purposes of the project.

(6) A DA permit does not convey any property rights, either in real estate or material, or any exclusive privileges. Furthermore, a DA permit does not authorize any injury to property or invasion of rights or any infringement of Federal, state or local laws or regulations. The applicant's signature on an application is an affirmation that the applicant possesses or will possess the requisite property interest to undertake the activity proposed in the application. The district engineer will not enter into disputes but will remind the applicant of the above. The dispute over property ownership will not be a factor in the Corps public interest decision.

(h) *Activities affecting coastal zones.* Applications for DA permits for activities affecting the coastal zones of those states having a coastal zone management program approved by the Secretary of Commerce will be evaluated with respect to compliance with that program. No permit will be issued to a non-federal applicant until certification has been provided that the proposed activity complies with the coastal zone management program and the appropriate state agency has concurred with the certification or has waived its right to do so. However, a permit may be issued to a non-federal applicant if the Secretary of Commerce, on his own initiative or upon appeal by the applicant, finds that the proposed activity is consistent with the objectives of the Coastal Zone Management Act of 1972 or is otherwise necessary in the interest of national security. Federal agency and Indian tribe applicants for DA permits are responsible for complying with the Coastal Zone Management Act's directives for assuring that their activities directly affecting the coastal zone are consistent, to the maximum extent practicable, with approved state coastal zone management programs.

(i) *Activities in marine sanctuaries.* Applications for DA authorization for activities in a marine sanctuary established by the Secretary of Commerce under authority of section 302 of the Marine Protection, Research and Sanctuaries Act of 1972, as amended, will be evaluated for impact on the marine sanctuary. No permit will be issued until the applicant provides a certification from the Secretary of Commerce that the proposed activity is consistent with the purposes of Title III of the Marine Protection, Research and Sanctuaries Act of 1972, as amended, and can be carried out within the regulations promulgated by the Secretary of Commerce to control activities within the marine sanctuary.

(j) *Other Federal, state, or local requirements.* (1) Processing of an application for a DA permit normally will proceed concurrently with the processing of other required Federal, state, and/or local authorizations or certifications. Final action on the DA permit will normally not be delayed pending action by another Federal, state or local agency (See 33 CFR 325.2 (d)(4)). However, where the required Federal, state and/or local authorization and/or certification has been denied for activities which also require a Department of the Army permit before final action has been taken on the Army permit application, the district engineer will, after considering the likelihood of subsequent approval of the other authorization and/or certification and the time and effort remaining to complete processing the Army permit application, either immediately deny the Army permit without prejudice or continue processing the application to a conclusion. If the district engineer continues processing the application, he will conclude by either denying the permit as contrary to the public interest, or denying it without prejudice indicating that except for the other Federal, state or local denial the Army permit could, under appropriate conditions, be issued. Denial without prejudice means that there is no prejudice to the right of the applicant to reinstate processing of the Army permit

application if subsequent approval is received from the appropriate Federal, state and/or local agency on a previously denied authorization and/or certification. Even if official certification and/or authorization is not required by state or federal law, but a state, regional, or local agency having jurisdiction or interest over the particular activity comments on the application, due consideration shall be given to those official views as a reflection of local factors of the public interest.

(2) The primary responsibility for determining zoning and land use matters rests with state, local and tribal governments. The district engineer will normally accept decisions by such governments on those matters unless there are significant issues of overriding national importance. Such issues would include but are not necessarily limited to national security, navigation, national economic development, water quality, preservation of special aquatic areas, including wetlands, with significant interstate importance, and national energy needs. Whether a factor has overriding importance will depend on the degree of impact in an individual case.

(3) A proposed activity may result in conflicting comments from several agencies within the same state. Where a state has not designated a single responsible coordinating agency, district engineers will ask the Governor to express his views or to designate one state agency to represent the official state position in the particular case.

(4) In the absence of overriding national factors of the public interest that may be revealed during the evaluation of the permit application, a permit will generally be issued following receipt of a favorable state determination provided the concerns, policies, goals, and requirements as expressed in 33 CFR Parts 320–324, and the applicable statutes have been considered and followed: e.g., the National Environmental Policy Act; the Fish and Wildlife Coordination Act; the Historical and Archeological Preservation Act; the National Historic Preservation Act; the Endangered Species Act; the Coastal Zone Management Act; the Marine Protection, Research and Sanctuaries Act of 1972, as amended; the Clean Water Act, the Archeological Resources Act, and the American Indian Religious Freedom Act. Similarly, a permit will generally be issued for Federal and Federally-authorized activities; another federal agency's determination to proceed is entitled to substantial consideration in the Corps' public interest review.

(5) Where general permits to avoid duplication are not practical, district engineers shall develop joint procedures with those local, state, and other Federal agencies having ongoing permit programs for activities also regulated by the Department of the Army. In such cases, applications for DA permits may be processed jointly with the state or other federal applications to an independent conclusion and decision by the district engineer and the appropriate Federal or state agency. (See 33 CFR 325.2(e).)

(6) The district engineer shall develop operating procedures for establishing official communications with Indian Tribes within the district. The procedures shall provide for appointment of a tribal representative who will receive all pertinent public notices, and respond to such notices with the official tribal position on the proposed activity. This procedure shall apply only to those tribes which accept this option. Any adopted operating procedures shall be distributed by public notice to inform the tribes of this option.

(k) *Safety of impoundment structures.* To insure that all impoundment structures are designed for safety, non-Federal applicants may be required to demonstrate that the structures comply with established state dam safety criteria or have been designed by qualified persons and, in appropriate cases, that the design has been independently reviewed (and modified as the review would indicate) by similarly qualified persons.

(l) Floodplain management. (1) Floodplains possess significant natural values and carry out numerous functions important to the public interest. These include:

(i) Water resources values (natural moderation of floods, water quality maintenance, and groundwater recharge);

(ii) Living resource values (fish, wildlife, and plant resources);

(iii) Cultural resource values (open space, natural beauty, scientific study, outdoor education, and recreation); and

(iv) Cultivated resource values (agriculture, aquaculture, and forestry).

(2) Although a particular alteration to a floodplain may constitute a minor change, the cumulative impact of such changes may result in a significant degradation of floodplain values and functions and in increased potential for harm to upstream and downstream activities. In accordance with the requirements of Executive Order 11988, district engineers, as part of their public interest review, should avoid to the extent practicable, long and short term significant adverse impacts associated with the occupancy and modification of floodplains, as well as the direct and indirect support of floodplain development whenever there is a practicable alternative. For those activities which in the public interest must occur in or impact upon floodplains, the district engineer shall ensure, to the maximum extent practicable, that the impacts of potential flooding on human health, safety, and welfare are minimized, the risks of flood losses are minimized, and, whenever practicable the natural and beneficial values served by floodplains are restored and preserved.

(3) In accordance with Executive Order 11988, the district engineer should avoid authorizing floodplain developments whenever practicable alternatives exist outside the floodplain. If there are no such practicable alternatives, the district engineer shall consider, as a means of mitigation, alternatives within the floodplain which will lessen any significant adverse impact to the floodplain.

(m) *Water supply and conservation.* Water is an essential resource, basic to human survival, economic growth, and the natural environment. Water conservation requires the efficient use of water resources in all actions which involve the significant use of water or that significantly affect the availability of water for alternative uses including opportunities to reduce demand and improve efficiency in order to minimize new supply requirements. Actions affecting water quantities are subject to Congressional policy as stated in section 101(g) of the Clean Water Act which provides that the authority of states to allocate water quantities shall not be superseded, abrogated, or otherwise impaired.

(n) *Energy conservation and development.* Energy conservation and development are major national objectives. District engineers will give high priority to the processing of permit actions involving energy projects.

(o) *Navigation.* (1) Section 11 of the Rivers and Harbors Act of 1899 authorized establishment of harbor lines shoreward of which no individual permits were required. Because harbor lines were established on the basis of navigation impacts only, the Corps of Engineers published a regulation on 27 May 1970 (33 CFR 209.150) which declared that permits would thereafter be required for activities shoreward of the harbor lines. Review of applications

would be based on a full public interest evaluation and harbor lines would serve as guidance for assessing navigation impacts. Accordingly, activities constructed shoreward of harbor lines prior to 27 May 1970 do not require specific authorization.

(2) The policy of considering harbor lines as guidance for assessing impacts on navigation continues.

(3) Protection of navigation in all navigable waters of the United States continues to be a primary concern of the federal government.

(4) District engineers should protect navigational and anchorage interests in connection with the NPDES program by recommending to EPA or to the state, if the program has been delegated, that a permit be denied unless appropriate conditions can be included to avoid any substantial impairment of navigation and anchorage.

(p) *Environmental benefits.* Some activities that require Department of the Army permits result in beneficial effects to the quality of the environment. The district engineer will weigh these benefits as well as environmental detriments along with other factors of the public interest.

(q) *Economics.* When private enterprise makes application for a permit, it will generally be assumed that appropriate economic evaluations have been completed, the proposal is economically viable, and is needed in the market place. However, the district engineer in appropriate cases, may make an independent review of the need for the project from the perspective of the overall public interest. The economic benefits of many projects are important to the local community and contribute to needed improvements in the local economic base, affecting such factors as employment, tax revenues, community cohesion, community services, and property values. Many projects also contribute to the National Economic Development (NED), (i.e., the increase in the net value of the national output of goods and services).

(r) *Mitigation.*[1] (1) Mitigation is an important aspect of the review and balancing process on many Department of the Army permit applications. Consideration of mitigation will occur throughout the permit application review process and includes avoiding, minimizing, rectifying, reducing, or compensating for resource losses. Losses will be avoided to the extent practicable. Compensation may occur on-site or at an off-site location. Mitigation requirements generally fall into three categories.

(i) Project modifications to minimize adverse project impacts should be discussed with the applicant at pre-application meetings and during application processing. As a result of these discussions and as the district engineer's evaluation proceeds, the district engineer may require minor project modifications. Minor project modifications are those that are considered feasible (cost, constructability, etc.) to the applicant and that, if adopted, will result in a project that generally meets the applicant's purpose and need. Such modifications can include reductions in scope and size; changes in construction methods, materials or timing; and operation and maintenance practices or other similar modifications that reflect a sensitivity to environmental quality within the context of the work proposed. For example, erosion control features could be required on a fill project to reduce sedimentation impacts or a pier could be reoriented to minimize navigational problems even though those projects may satisfy all legal requirements (paragraph (r)(1)(ii) of this section) and the public interest review test (paragraph (r)(1)(iii) of this section) without such modifications.

(ii) Further mitigation measures may be required to satisfy legal requirements. For Section 404 applications, mitigation shall be required to ensure that the project complies with the 404(b)(1) Guidelines. Some mitigation measures are enumerated at 40 CFR 230.70 through 40 CFR 230.77 (Subpart H of the 404(b)(1) Guidelines).

(iii) Mitigation measures in addition to those under paragraphs (r)(1) (i) and (ii) of this section may be required as a result of the public interest review process. (See 33 CFR 325.4(a).) Mitigation should be developed and incorporated within the public interest review process to the extent that the mitigation is found by the district engineer to be reasonable and justified. Only those measures required to ensure that the project is not contrary to the public interest may be required under this subparagraph.

(2) All compensatory mitigation will be for significant resource losses which are specifically identifiable, reasonably likely to occur, and of importance to the human or aquatic environment. Also, all mitigation will be directly related to the impacts of the proposal, appropriate to the scope and degree of those impacts, and reasonably enforceable. District engineers will require all forms of mitigation, including compensatory mitigation, only as provided in paragraphs (r)(1) (i) through (iii) of this section. Additional mitigation may be added at the applicants' request.

[1] This is a general statement of mitigation policy which applies to all Corps of Engineers regulatory authorities covered by these regulations (33 CFR Parts 320-330). It is not a substitute for the mitigation requirements necessary to ensure that a permit action under section 404 of the Clean Water Act complies with the section 404(b)(1) Guidelines. There is currently an Interagency Working Group formed to develop guidance on implementing mitigation requirements of the Guidelines.

PART 321—PERMITS FOR DAMS AND DIKES IN NAVIGABLE WATERS OF THE UNITED STATES

Sec.
321.1 General.
321.2 Definitions.
321.3 Special policies and procedures.

Authority: 33 U.S.C. 401.

§ 321.1 General.

This regulation prescribes, in addition to the general policies of 33 CFR Part 320 and procedures of 33 CFR Part 325, those special policies, practices, and procedures to be followed by the Corps of Engineers in connection with the review of applications for Department of the Army (DA) permits to authorize the construction of a dike or dam in a navigable water of the United States pursuant to section 9 of the Rivers and Harbors Act of 1899 (33 U.S.C. 401). See 33 CFR 320.2(a). Dams and dikes in navigable waters of the United States also require DA permits under section 404 of the Clean Water Act, as amended (33 U.S.C. 1344). Applicants for DA permits under this Part should also refer to 33 CFR Part 323 to satisfy the requirements of section 404.

§ 321.2 Definitions.

For the purpose of this regulation, the following terms are defined:

(a) The term "navigable waters of the United States" means those waters of the United States that are subject to the ebb and flow of the tide shoreward to the mean high water mark and/or are presently used, or have been used in the past, or may be susceptible to use to transport interstate or foreign commerce. See 33 CFR Part 329 for a more complete definition of this term.

(b) The term "dike or dam" means, for the purposes of section 9, any impoundment structure that completely spans a navigable water of the United States and that may obstruct interstate waterborne commerce. The term does not include a weir. Weirs are regulated pursuant to section 10 of the Rivers and Harbors Act of 1899. (See 33 CFR Part 322.)

§ 321.3 Special policies and procedures.

The following additional special policies and procedures shall be applicable to the evaluation of permit applications under this regulation:

(a) The Assistant Secretary of the Army (Civil Works) will decide whether DA authorization for a dam or dike in an interstate navigable water of the United States will be issued, since this authority has not been delegated to the Chief of Engineers. The conditions to be imposed in any instrument of authorization will be recommended by the district engineer when forwarding the report to the Assistant Secretary of the Army (Civil Works), through the Chief of Engineers.

(b) District engineers are authorized to decide whether DA authorization for a dam or dike in an intrastate navigable water of the United States will be issued (see 33 CFR 325.8).

(c) Processing a DA application under section 9 will not be completed until the approval of the United States Congress has been obtained if the navigable water of the United States is an interstate waterbody, or until the approval of the appropriate state legislature has been obtained if the vigable water of the United States is intrastate waterbody (i.e., the navigable portion of the navigable water of the United States is solely within the boundaries of one state). The district engineer, upon receipt of such an application, will notify the applicant that the consent of Congress or the state legislature must be obtained before a permit can be issued.

PART 322—PERMITS FOR STRUCTURES OR WORK IN OR AFFECTING NAVIGABLE WATERS OF THE UNITED STATES

Sec.
322.1 General.
322.2 Definitions.
322.3 Activities requiring permits.
322.4 Activities not requiring permits.
322.5 Special policies.

Authority: 33 U.S.C. 403.

§ 322.1 General.

This regulation prescribes, in addition to the general policies of 33 CFR Part 320 and procedures of 33 CFR Part 325, those special policies, practices, and procedures to be followed by the Corps of Engineers in connection with the view of applications for Department of Army (DA) permits to authorize certain structures or work in or affecting navigable waters of the United States pursuant to section 10 of the Rivers and Harbors Act of 1899 (33 U.S.C. 403) (hereinafter referred to as section 10). See 33 CFR 320.2(b). Certain structures or work in or affecting navigable waters of the United States are also regulated under other authorities of the DA. These include discharges of dredged or fill material into waters of the United States, including the territorial seas, pursuant to section 404 of the Clean Water Act (33 U.S.C. 1344; see 33 CFR Part 323) and the transportation of dredged material by vessel for purposes of dumping in ocean waters, including the territorial seas, pursuant to section 103 of the Marine Protection, Research and Sanctuaries Act of 1972, as amended (33 U.S.C. 1413; see 33 CFR Part 324). A DA permit will also be required under these additional authorities if they are applicable to structures or work in or affecting navigable waters of the United States. Applicants for DA permits under this part should refer to the other cited authorities and implementing regulations for these additional permit requirements to determine whether they also are applicable to their proposed activities.

§ 322.2 Definitions.

For the purpose of this regulation, the following terms are defined:

(a) The term "navigable waters of the United States" and all other terms relating to the geographic scope of jurisdiction are defined at 33 CFR Part 329. Generally, they are those waters of the United States that are subject to the ebb and flow of the tide shoreward to the mean high water mark, and/or are presently used, or have been used in the past, or may be susceptible to use to transport interstate or foreign commerce.

(b) The term "structure" shall include, without limitation, any pier, boat dock, boat ramp, wharf, dolphin, weir, boom, breakwater, bulkhead, revetment, riprap, jetty, artificial island, artificial reef, permanent mooring structure, power transmission line, permanently moored floating vessel, piling, aid to navigation, or any other obstacle or obstruction.

(c) The term "work" shall include, without limitation, any dredging or disposal of dredged material, excavation, filling, or other modification of a navigable water of the United States.

(d) The term "letter of permission" means a type of individual permit issued in accordance with the abbreviated procedures of 33 CFR 325.2(e).

(e) The term "individual permit" means a DA authorization that is issued following a case-by-case evaluation of a specific structure or work in accordance with the procedures of this regulation and 33 CFR Part 325, and a determination that the proposed structure or work is in the public interest pursuant to 33 CFR Part 320.

(f) The term "general permit" means a DA authorization that is issued on a nationwide or regional basis for a category or categories of activities when:

(1) Those activities are substantially similar in nature and cause only minimal individual and cumulative environmental impacts; or

(2) The general permit would result in avoiding unnecessary duplication of the regulatory control exercised by another Federal, state, or local agency provided it has been determined that the environmental consequences of the action are individually and cumulatively minimal. (See 33 CFR 325.2(e) and 33 CFR Part 330.)

(g) The term "artificial reef" means a structure which is constructed or placed in the navigable waters of the United States or in the waters overlying the outer continental shelf for the purpose of enhancing fishery resources and commercial and recreational fishing opportunities. The term does not include activities or structures such as wing deflectors, bank stabilization, grade stabilization structures, or low flow key ways, all of which may be useful to enhance fisheries resources.

§ 322.3 Activities requiring permits.

(a) *General.* DA permits are required under section 10 for structures and/or work in or affecting navigable waters of the United States except as otherwise provided in § 322.4 below. Certain activities specified in 33 CFR Part 330 are permitted by that regulation ("nationwide general permits"). Other activities may be authorized by district or division engineers on a regional basis ("regional general permits"). If an activity is not exempted by section 322.4 of this part or authorized by a general permit, an individual section 10 permit will be required for the proposed activity. Structures or work are in navigable waters of the United States if they are within limits defined in 33 CFR Part 329. Structures or work outside these limits are subject to the provisions of law cited in paragraph (a) of this section, if these structures or work affect the course, location, or condition of the waterbody in such a manner as to impact on its navigable capacity. For purposes of a section 10 permit, a tunnel or other structure or work under or over a navigable water of the United States is considered to have an impact on the navigable capacity of the waterbody.

(b) *Outer continental shelf.* DA permits are required for the construction

be shown and the extension is otherwise justified.

(ii) Drilling rigs must be at least 500 feet from any fairway boundary or whatever distance necessary to insure that minimum clearance over an anchor line within a fairway will be 125 feet.

(iii) No anchor buoys or floats or related rigging will be allowed on the surface of the water or to a depth of 125 feet from the surface, within the fairway.

(iv) Drilling rigs may not be placed closer than 2 nautical miles of any other drilling rig situated along a fairway boundary, and not closer than 3 nautical miles to any drilling rig located on the opposite side of the fairway.

(v) The permittee must notify the district engineer, Bureau of Land Management, Mineral Management Service, U.S. Coast Guard, National Oceanic and Atmospheric Administration and the U.S. Navy Hydrographic Office of the approximate dates (commencenent and completion) the anchors will be in place to insure maximum notification to mariners.

(vi) Navigation aids or danger markings must be installed as required by the U.S. Coast Guard.

(2) District engineers may grant permits for the erection of structures within an area designated as an anchorage area, but the number of structures will be limited by spacing, as follows: The center of a structure to be erected shall be not less than two (2) nautical miles from the center of any existing structure. In a drilling or production complex, associated structures shall be as close together as practicable having due consideration for the safety factors involved. A complex of associated structures, when connected by walkways, shall be considered one structure for the purpose of spacing. A vessel fixed in place by moorings and used in conjunction with the associated structures of a drilling or production complex, shall be considered an attendant vessel and its extent shall include its moorings. When a drilling or production complex includes an attendant vessel and the complex extends more than five hundred (500) yards from the center or the complex, a structure to be erected shall be not closer than two (2) nautical miles from the near outer limit of the complex. An underwater completion installation in and anchorage area shall be considered a structure and shall be marked with a lighted buoy as approved by the United States Coast Guard.

PART 323—PERMITS FOR DISCHARGES OF DREDGED OR FILL MATERIAL INTO WATERS OF THE UNITED STATES

Sec.
323.1 General.
323.2 Definitions.
323.3 Discharges requiring permits.
323.4 Discharges not requiring permits.
323.5 Program transfer to states.
323.6 Special policies and procedures.

Authority: 33 U.S.C. 1344.

§ 323.1 General.

This regulation prescribes, in addition to the general policies of 33 CFR Part 320 and procedures of 33 CFR Part 325, those special policies, practices, and procedures to be followed by the Corps of Engineers in connection with the review of applications for DA permits to authorize the discharge of dredged or fill material into waters of the United States pursuant to section 404 of the Clean Water Act (CWA) (33 U.S.C. 1344) (hereinafter referred to as section 404). (See 33 CFR 320.2(g).) Certain discharges of dredged or fill material into waters of the United States are also regulated under other authorities of the Department of the Army. These include dams and dikes in navigable waters of the United States pursuant to section 9 of the Rivers and Harbors Act of 1899 (33 U.S.C. 401; see 33 CFR Part 321) and certain structures or work in or affecting navigable waters of the United States pursuant to section 10 of the Rivers and Harbors Act of 1899 (33 U.S.C. 403; see 33 CFR Part 322). A DA permit will also be required under these additional authorities if they are applicable to activities involving discharges of dredged or fill material into waters of the United States. Applicants for DA permits under this part should refer to the other cited authorities and implementing regulations for these additional permit requirements to determine whether they also are applicable to their proposed activities.

§ 323.2 Definitions.

For the purpose of this part, the following terms are defined:

(a) The term "waters of the United States" and all other terms relating to the geographic scope of jurisdiction are defined at 33 CFR Part 328.

(b) The term "lake" means a standing body of open water that occurs in a natural depression fed by one or more streams from which a stream may flow, that occurs due to the widening or natural blockage or cutoff of a river or stream, or that occurs in an isolated natural depression that is not a part of a surface river or stream. The term also includes a standing body of open water created by artificially blocking or restricting the flow of a river, stream, or tidal area. As used in this regulation, the term does not include artificial lakes or ponds created by excavating and/or diking dry land to collect and retain water for such purposes as stock watering, irrigation, settling basins, cooling, or rice growing.

(c) The term "dredged material" means material that is excavated or dredged from waters of the United States.

(d) The term "discharge of dredged material" means any addition of dredged material into the waters of the United States. The term includes, without limitation, the addition of dredged material to a specified discharge site located in waters of the United States and the runoff or overflow from a contained land or water disposal area. Discharges of pollutants into waters of the United States resulting from the onshore subsequent processing of dredged material that is extracted for any commercial use (other than fill) are not included within this term and are subject to section 402 of the Clean Water Act even though the extraction and deposit of such material may require a permit from the Corps of Engineers. The term does not include plowing, cultivating, seeding and harvesting for the production of food, fiber, and forest products (See § 323.4 for the definition of these terms). The term does not include *de minimis*, incidental soil movement occurring during normal dredging operations.

(e) The term "fill material" means any material used for the primary purpose of replacing an aquatic area with dry land or of changing the bottom elevation of an waterbody. The term does not include any pollutant discharged into the water primarily to dispose of waste, as that activity is regulated under section 402 of the Clean Water Act.

(f) The term "discharge of fill material" means the addition of fill material into waters of the United States. The term generally includes, without limitation, the following activities: Placement of fill that is necessary for the construction of any structure in a water of the United States; the building of any structure or impoundment requiring rock, sand, dirt, or other material for its construction; site-development fills for recreational, industrial, commercial, residential, and other uses; causeways or road fills; dams and dikes; artificial islands; property protection and/or reclamation devices such as riprap, groins, seawalls, breakwaters, and revetments; beach nourishment; levees; fill for structures such as sewage treatment facilities,

intake and outfall pipes associated with power plants and subaqueous utility lines; and artificial reefs. The term does not include plowing, cultivating, seeding and harvesting for the production of food, fiber, and forest products (See § 323.4 for the definition of these terms).

(g) The term "individual permit" means a Department of the Army authorization that is issued following a case-by-case evaluation of a specific project involving the proposed discharge(s) in accordance with the procedures of this part and 33 CFR Part 325 and a determination that the proposed discharge is in the public interest pursuant to 33 CFR Part 320.

(h) The term "general permit" means a Department of the Army authorization that is issued on a nationwide or regional basis for a category or categories of activities when:

(1) Those activities are substantially similar in nature and cause only minimal individual and cumulative environmental impacts; or

(2) The general permit would result in avoiding unnecessary duplication of regulatory control exercised by another Federal, state, or local agency provided it has been determined that the environmental consequences of the action are individually and cumulatively minimal. (See 33 CFR 325.2(e) and 33 CFR Part 330.)

§ 323.3 Discharges requiring permits.

(a) *General.* Except as provided in § 323.4 of this Part, DA permits will be required for the discharge of dredged or fill material into waters of the United States. Certain discharges specified in 33 CFR Part 330 are permitted by that regulation ("nationwide permits"). Other discharges may be authorized by district or division engineers on a regional basis ("regional permits"). If a discharge of dredged or fill material is not exempted by § 323.4 of this Part or permitted by 33 CFR Part 330, an individual or regional section 404 permit will be required for the discharge of dredged or fill material into waters of the United States.

(b) *Activities of Federal agencies.* Discharges of dredged or fill material into waters of the United States done by or on behalf of any Federal agency, other than the Corps of Engineers (see 33 CFR Part 209.145), are subject to the authorization procedures of these regulations. Agreement for construction or engineering services performed for other agencies by the Corps of Engineers does not constitute authorization under the regulations. Division and district engineers will therefore advise Federal agencies and instrumentalities accordingly and cooperate to the fullest extent in expediting the processing of their applications.

§ 323.4 Discharges not requiring permits.

(a) *General.* Except as specified in paragraphs (b) and (c) of this section, any discharge of dredged or fill material that may result from any of the following activities is not prohibited by or otherwise subject to regulation under section 404:

(1)(i) Normal farming, silviculture and ranching activities such as plowing, seeding, cultivating, minor drainage, and harvesting for the production of food, fiber, and forest products, or upland soil and water conservation practices, as defined in paragraph (a)(1)(iii) of this section.

(ii) To fall under this exemption, the activities specified in paragraph (a)(1)(i) of this section must be part of an established (i.e., on-going) farming, silviculture, or ranching operation and must be in accordance with definitions in § 323.4(a)(1)(iii). Activities on areas lying fallow as part of a conventional rotational cycle are part of an established operation. Activities which bring an area into farming, silviculture, or ranching use are not part of an established operation. An operation ceases to be established when the area on which it was conducted has been coverted to another use or has lain idle so long that modifications to the hydrological regime are necessary to resume operations. If an activity takes place outside the waters of the United States, or if it does not involve a discharge, it does not need a section 404 permit, whether or not it is part of an established farming, silviculture, or ranching operation.

(iii) (A) Cultivating means physical methods of soil treatment employed within established farming, ranching and silviculture lands on farm, ranch, or forest crops to aid and improve their growth, quality or yield.

(B) Harvesting means physical measures employed directly upon farm, forest, or ranch crops within established agricultural and silvicultural lands to bring about their removal from farm, forest, or ranch land, but does not include the construction of farm, forest, or ranch roads.

(C)(*1*) Minor Drainage means:

(*i*) The discharge of dredged or fill material incidental to connecting upland drainage facilities to waters of the United States, adequate to effect the removal of excess soil moisture from upland croplands. (Construction and maintenance of upland (dryland) facilities, such as ditching and tiling, incidental to the planting, cultivating, protecting, or harvesting of crops, involve no discharge of dredged or fill material into waters of the United States, and as such never require a section 404 permit.);

(*ii*) The discharge of dredged or fill material for the purpose of installing ditching or other such water control facilities incidental to planting, cultivating, protecting, or harvesting of rice, cranberries or other wetland crop species, where these activities and the discharge occur in waters of the United States which are in established use for such agricultural and silvicultural wetland crop production;

(*iii*) The discharge of dredged or fill material for the purpose of manipulating the water levels of, or regulating the flow or distribution of water within, existing impoundments which have been constructed in accordance with applicable requirements of CWA, and which are in established use for the production of rice, cranberries, or other wetland crop species. (The provisions of paragraphs (a)(1)(iii)(C)(*1*) (*ii*) and (*iii*) of this section apply to areas that are in established use exclusively for wetland crop production as well as areas in established use for conventional wetland/non-wetland crop rotation (e.g., the rotations of rice and soybeans) where such rotation results in the cyclical or intermittent temporary dewatering of such areas.)

(*iv*) The discharges of dredged or fill material incidental to the emergency removal of sandbars, gravel bars, or other similar blockages which are formed during flood flows or other events, where such blockages close or constrict previously existing drainageways and, if not promptly removed, would result in damage to or loss of existing crops or would impair or prevent the plowing, seeding, harvesting or cultivating of crops on land in established use for crop production. Such removal does not include enlarging or extending the dimensions of, or changing the bottom elevations of, the affected drainageway as it existed prior to the formation of the blockage. Removal must be accomplished within one year of discovery of such blockages in order to be eligible for exemption.

(*2*) Minor drainage in waters of the U.S. is limited to drainage within areas that are part of an established farming or silviculture operation. It does not include drainage associated with the immediate or gradual conversion of a wetland to a non-wetland (e.g., wetland species to upland species not typically adapted to life in saturated soil conditions), or conversion from one wetland use to another (for example, silviculture to farming). In addition,

minor drainage does not include the construction of any canal, ditch, dike or other waterway or structure which drains or otherwise significantly modifies a stream, lake, swamp, bog or any other wetland or aquatic area constituting waters of the United States. Any discharge of dredged or fill material into the waters of the United States incidental to the construction of any such structure or waterway requires a permit.

(D) Plowing means all forms of primary tillage, including moldboard, chisel, or wide-blade plowing, discing, harrowing and similar physical means utilized on farm, forest or ranch land for the breaking up, cutting, turning over, or stirring of soil to prepare it for the planting of crops. The term does not include the redistribution of soil, rock, sand, or other surficial materials in a manner which changes any area of the waters of the United States to dry land. For example, the redistribution of surface materials by blading, grading, or other means to fill in wetland areas is not plowing. Rock crushing activities which result in the loss of natural drainage characteristics, the reduction of water storage and recharge capabilities, or the overburden of natural water filtration capacities do not constitute plowing. Plowing as described above will never involve a discharge of dredged or fill material.

(E) Seeding means the sowing of seed and placement of seedlings to produce farm, ranch, or forest crops and includes the placement of soil beds for seeds or seedlings on established farm and forest lands.

(2) Maintenance, including emergency reconstruction of recently damaged parts, of currently serviceable structures such as dikes, dams, levees, groins, riprap, breakwaters, causeways, bridge abutments or approaches, and transportation structures. Maintenance does not include any modification that changes the character, scope, or size of the original fill design. Emergency reconstruction must occur within a reasonable period of time after damage occurs in order to qualify for this exemption.

(3) Construction or maintenance of farm or stock ponds or irrigation ditches, or the maintenance (but not construction) of drainage ditches. Discharges associated with siphons, pumps, headgates, wingwalls, weirs, diversion structures, and such other facilities as are appurtenant and functionally related to irrigation ditches are included in this exemption.

(4) Construction of temporary sedimentation basins on a construction site which does not include placement of fill material into waters of the U.S. The term "construction site" refers to any site involving the erection of buildings, roads, and other discrete structures and the installation of support facilities necessary for construction and utilization of such structures. The term also includes any other land areas which involve land-disturbing excavation activities, including quarrying or other mining activities, where an increase in the runoff of sediment is controlled through the use of temporary sedimentation basins.

(5) Any activity with respect to which a state has an approved program under section 208(b)(4) of the CWA which meets the requirements of sections 208(b)(4) (B) and (C).

(6) Construction or maintenance of farm roads, forest roads, or temporary roads for moving mining equipment, where such roads are constructed and maintained in accordance with best management practices (BMPs) to assure that flow and circulation patterns and chemical and biological characteristics of waters of the United States are not impaired, that the reach of the waters of the United States is not reduced, and that any adverse effect on the aquatic environment will be otherwise minimized. These BMPs which must be applied to satisfy this provision shall include those detailed BMPs described in the state's approved program description pursuant to the requirements of 40 CFR Part 233.22(i), and shall also include the following baseline provisions:

(i) Permanent roads (for farming or forestry activities), temporary access roads (for mining, forestry, or farm purposes) and skid trails (for logging) in waters of the U.S. shall be held to the minimum feasible number, width, and total length consistent with the purpose of specific farming, silvicultural or mining operations, and local topographic and climatic conditions;

(ii) All roads, temporary or permanent, shall be located sufficiently far from streams or other water bodies (except for portions of such roads which must cross water bodies) to minimize discharges of dredged or fill material into waters of the U.S.;

(iii) The road fill shall be bridged, culverted, or otherwise designed to prevent the restriction of expected flood flows;

(iv) The fill shall be properly stabilized and maintained during and following construction to prevent erosion;

(v) Discharges of dredged or fill material into waters of the United States to construct a road fill shall be made in a manner that minimizes the encroachment of trucks, tractors, bulldozers, or other heavy equipment within waters of the United States (including adjacent wetlands) that lie outside the lateral boundaries of the fill itself;

(vi) In designing, constructing, and maintaining roads, vegetative disturbance in the waters of the U.S. shall be kept to a minimum;

(vii) The design, construction and maintenance of the road crossing shall not disrupt the migration or other movement of those species of aquatic life inhabiting the water body;

(viii) Borrow material shall be taken from upland sources whenever feasible;

(ix) The discharge shall not take, or jeopardize the continued existence of, a threatened or endangered species as defined under the Endangered Species Act, or adversely modify or destroy the critical habitat of such species;

(x) Discharges into breeding and nesting areas for migratory waterfowl, spawning areas, and wetlands shall be avoided if practical alternatives exist;

(xi) The discharge shall not be located in the proximity of a public water supply intake;

(xii) The discharge shall not occur in areas of concentrated shellfish production;

(xiii) The discharge shall not occur in a component of the National Wild and Scenic River System;

(xiv) The discharge of material shall consist of suitable material free from toxic pollutants in toxic amounts; and

(xv) All temporary fills shall be removed in their entirety and the area restored to its original elevation.

(b) If any discharge of dredged or fill material resulting from the activities listed in paragraphs (a) (1)–(6) of this section contains any toxic pollutant listed under section 307 of the CWA such discharge shall be subject to any applicable toxic effluent standard or prohibition, and shall require a Section 404 permit.

(c) Any discharge of dredged or fill material into waters of the United States incidental to any of the activities identified in paragraphs (a) (1)–(6) of this section must have a permit if it is part of an activity whose purpose is to convert an area of the waters of the United States into a use to which it was not previously subject, where the flow or circulation of waters of the United States may be impaired or the reach of such waters reduced. Where the proposed discharge will result in significant discernible alterations to flow or circulation, the presumption is that flow or circulation may be impaired by such alteration. For example, a

permit will be required for the conversion of a cypress swamp to some other use or the conversion of a wetland from silvicultural to agricultural use when there is a discharge of dredged or fill material into waters of the United States in conjunction with construction of dikes, drainage ditches or other works or structures used to effect such conversion. A conversion of a Section 404 wetland to a non-wetland is a change in use of an area of waters of the United States. A discharge which elevates the bottom of waters of the United States without converting it to dry land does not thereby reduce the reach of, but may alter the flow or circulation of, waters of the United States.

(d) Federal projects which qualify under the criteria contained in section 404(r) of the CWA are exempt from section 404 permit requirements, but may be subject to other state or Federal requirements.

§ 323.5 Program transfer to states.

Section 404(h) of the CWA allows the Administrator of the Environmental Protection Agency (EPA) to transfer administration of the section 404 permit program for discharges into certain waters of the United States to qualified states. (The program cannot be transferred for those waters which are presently used, or are susceptible to use in their natural condition or by reasonable improvement as a means to transport interstate or foreign commerce shoreward to their ordinary high water mark, including all waters which are subject to the ebb and flow of the tide shoreward to the high tide line, including wetlands adjacent thereto). See 40 CFR Parts 233 and 124 for procedural regulations for transferring Section 404 programs to states. Once a state's 404 program is approved and in effect, the Corps of Engineers will suspend processing of section 404 applications in the applicable waters and will transfer pending applications to the state agency responsible for administering the program. District engineers will assist EPA and the states in any way practicable to effect transfer and will develop appropriate procedures to ensure orderly and expeditious transfer.

§ 323.6 Special policies and procedures.

(a) The Secretary of the Army has delegated to the Chief of Engineers the authority to issue or deny section 404 permits. The district engineer will review applications for permits for the discharge of dredged or fill material into waters of the United States in accordance with guidelines promulgated by the Administrator, EPA, under authority of section 404(b)(1) of the CWA. (see 40 CFR Part 230.) Subject to consideration of any economic impact on navigation and anchorage pursuant to section 404(b)(2), a permit will be denied if the discharge that would be authorized by such a permit would not comply with the 404(b)(1) guidelines. If the district engineer determines that the proposed discharge would comply with the 404(b)(1) guidelines, he will grant the permit unless issuance would be contrary to the public interest.

(b) The Corps will not issue a permit where the regional administrator of EPA has notified the district engineer and applicant in writing pursuant to 40 CFR 231.3(a)(1) that he intends to issue a public notice of a proposed determination to prohibit or withdraw the specification, or to deny, restrict or withdraw the use for specification, of any defined area as a disposal site in accordance with section 404(c) of the Clean Water Act. However the Corps will continue to complete the administrative processing of the application while the section 404(c) procedures are underway including completion of final coordination with EPA under 33 CFR Part 325.

PART 324—PERMITS FOR OCEAN DUMPING OF DREDGED MATERIAL

Sec.
324.1 General.
324.2 Definitions.
324.3 Activities requiring permits.
324.4 Special procedures.

Authority: 33 U.S.C. 1413.

§ 324.1 General.

This regulation prescribes in addition to the general policies of 33 CFR Part 320 and procedures of 33 CFR Part 325, those special policies, practices and procedures to be followed by the Corps of Engineers in connection with the review of applications for Department of the Army (DA) permits to authorize the transportation of dredged material by vessel or other vehicle for the purpose of dumping it in ocean waters at dumping sites designated under 40 CFR Part 228 pursuant to section 103 of the Marine Protection, Research and Sanctuaries Act of 1972, as amended (33 U.S.C. 1413) (hereinafter referred to as section 103). See 33 CFR 320.2(h). Activities involving the transportation of dredged material for the purpose of dumping in the ocean waters also require DA permits under Section 10 of the Rivers and Harbors Act of 1899 (33 U.S.C. 403) for the dredging in navigable waters of the United States. Applicants for DA permits under this Part should also refer to 33 CFR Part 322 to satisfy the requirements of Section 10.

§ 324.2 Definitions.

For the purpose of this regulation, the following terms are defined:

(a) The term "ocean waters" means those waters of the open seas lying seaward of the base line from which the territorial sea is measured, as provided for in the Convention on the Territorial Sea and the Contiguous Zone (15 UST 1606; TIAS 5639).

(b) The term "dredged material" means any material excavated or dredged from navigable waters of the United States.

(c) The term "transport" or "transportation" refers to the conveyance and related handling of dredged material by a vessel or other vehicle.

§ 324.3 Activities requiring permits.

(a) *General.* DA permits are required for the transportation of dredged material for the purpose of dumping it in ocean waters.

(b) *Activities of Federal agencies.* (1) The transportation of dredged material for the purpose of disposal in ocean waters done by or on behalf of any Federal agency other than the activities of the Corps of Engineers is subject to the procedures of this regulation. Agreement for construction or engineering services performed for other agencies by the Corps of Engineers does not constitute authorization under these regulations. Division and district engineers will therefore advise Federal agencies accordingly and cooperate to the fullest extent in the expeditious processing of their applications. The activities of the Corps of Engineers that involve the transportation of dredged material for disposal in ocean waters are regulated by 33 CFR 209.145.

(2) The policy provisions set out in 33 CFR 320.4(j) relating to state or local authorizations do not apply to work or structures undertaken by Federal agencies, except where compliance with non-Federal authorization is required by Federal law or Executive policy. Federal agencies are responsible for conformance with such laws and policies. (See EO 12088, October 18, 1978.) Federal agencies are not required to obtain and provide certification of compliance with effluent limitations and water quality standards from state or interstate water pollution control agencies in connection with activities involving the transport of dredged material for dumping into ocean waters beyond the territorial sea.

§ 324.4 Special procedures.

The Secretary of the Army has delegated to the Chief of Engineers the authority to issue or deny section 103 permits. The following additional procedures shall also be applicable under this regulation.

(a) *Public notice.* For all applications for section 103 permits, the district engineer will issue a public notice which shall contain the information specified in 33 CFR 325.3.

(b) *Evaluation.* Applications for permits for the transportation of dredged material for the purpose of dumping it in ocean waters will be evaluated to determine whether the proposed dumping will unreasonably degrade or endanger human health, welfare, amenities, or the marine environment, ecological systems or economic potentialities. District engineers will apply the criteria established by the Administrator of EPA pursuant to section 102 of the Marine Protection, Research and Sanctuaries Act of 1972 in making this evaluation. (See 40 CFR Parts 220-229) Where ocean dumping is determined to be necessary, the district engineer will, to the extent feasible, specify disposal sites using the recommendations of the Administrator pursuant to section 102(c) of the Act.

(c) *EPA review.* When the Regional Administrator, EPA, in accordance with 40 CFR 225.2(b), advises the district engineer, in writing, that the proposed dumping will comply with the criteria, the district engineer will complete his evaluation of the application under this part and 33 CFR Parts 320 and 325. If, however, the Regional Administrator advises the district engineer, in writing, that the proposed dumping does not comply with the criteria, the district engineer will proceed as follows:

(1) The district engineer will determine whether there is an economically feasible alternative method or site available other than the proposed ocean disposal site. If there are other feasible alternative methods or sites available, the district engineer will evaluate them in accordance with 33 CFR Parts 320, 322, 323, and 325 and this Part, as appropriate.

(2) If the district engineer determines that there is no economically feasible alternative method or site available, and the proposed project is otherwise found to be not contrary to the public interest, he will so advise the Regional Administrator setting forth his reasons for such determination. If the Regional Administrator has not removed his objection within 15 days, the district engineer will submit a report of his determination to the Chief of Engineers for further coordination with the Administrator, EPA, and decision. The report forwarding the case will contain the analysis of whether there are other economically feasible methods or sites available to dispose of the dredged material.

(d) *Chief of Engineers review.* The Chief of Engineers shall evaluate the permit application and make a decision to deny the permit or recommend its issuance. If the decision of the Chief of Engineers is that ocean dumping at the proposed disposal site is required because of the unavailability of economically feasible alternatives, he shall so certify and request that the Secretary of the Army seek a waiver from the Administrator, EPA, of the criteria or of the critical site designation in accordance with 40 CFR 225.4.

PART 325—PROCESSING OF DEPARTMENT OF THE ARMY PERMITS

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Authority: 33 U.S.C. 401 et seq.; 33 U.S.C. 1344; 33 USC 1413.

§ 325.1 Applications for permits.

(a) *General.* The processing procedures of this Part apply to any Department of the Army (DA) permit. Special procedures and additional information are contained in 33 CFR Parts 320 through 324, 327 and Part 330. This Part is arranged in the basic timing sequence used by the Corps of Engineers in processing applications for DA permits.

(b) *Pre-application consultation for major applications.* The district staff element having responsibility for administering, processing, and enforcing federal laws and regulations relating to the Corps of Engineers regulatory program shall be available to advise potential applicants of studies or other information foreseeably required for later federal action. The district engineer will establish local procedures and policies including appropriate publicity programs which will allow potential applicants to contact the district engineer or the regulatory staff element to request pre-application consultation. Upon receipt of such request, the district engineer will assure the conduct of an orderly process which may involve other staff elements and affected agencies (Federal, state, or local) and the public. This early process should be brief but thorough so that the potential applicant may begin to assess the viability of some of the more obvious potential alternatives in the application. The district engineer will endeavor, at this stage, to provide the potential applicant with all helpful information necessary in pursuing the application, including factors which the Corps must consider in its permit decision making process. Whenever the district engineer becomes aware of planning for work which may require a DA permit and which may involve the preparation of an environmental document, he shall contact the principals involved to advise them of the requirement for the permit(s) and the attendant public interest review including the development of an environmental document. Whenever a potential applicant indicates the intent to submit an application for work which may require the preparation of an environmental document, a single point of contact shall be designated within the district's regulatory staff to effectively coordinate the regulatory process, including the National Environmental Policy Act (NEPA) procedures and all attendant reviews, meetings, hearings, and other actions, including the scoping process if appropriate, leading to a decision by the district engineer. Effort devoted to this process should be commensurate with the likelihood of a permit application actually being submitted to the Corps. The regulatory staff coordinator shall maintain an open relationship with each potential applicant or his consultants so as to assure that the potential applicant is fully aware of the substance (both quantitative and qualitative) of the data required by the district engineer for use in preparing an environmental assessment or an environmental impact statement (EIS) in accordance with 33 CFR Part 230, Appendix B.

(c) *Application form.* Applicants for all individual DA permits must use the standard application form (ENG Form 4345, OMB Approval No. OMB 49-R0420). Local variations of the application form for purposes of facilitating coordination with federal, state and local agencies may be used. The appropriate form may be obtained from the district office having

jurisdiction over the waters in which the activity is proposed to be located. Certain activities have been authorized general permits and do not require submission of an application form but may require a separate notification.

(d) *Content of application.* (1) The application must include a complete description of the proposed activity including necessary drawings, sketches, or plans sufficient for public notice (detailed engineering plans and specifications are not required); the location, purpose and need for the proposed activity; scheduling of the activity; the names and addresses of adjoining property owners; the location and dimensions of adjacent structures; and a list of authorizations required by other federal, interstate, state, or local agencies for the work, including all approvals received or denials already made. See § 325.3 for information required to be in public notices. District and division engineers are not authorized to develop additional information forms but may request specific information on a case-by-case basis. (See § 325.1(e)).

(2) All activities which the applicant plans to undertake which are reasonably related to the same project and for which a DA permit would be required should be included in the same permit application. District engineers should reject, as incomplete, any permit application which fails to comply with this requirement. For example, a permit application for a marina will include dredging required for access as well as any fill associated with construction of the marina.

(3) If the activity would involve dredging in navigable waters of the United States, the application must include a description of the type, composition and quantity of the material to be dredged, the method of dredging, and the site and plans for disposal of the dredged material.

(4) If the activity would include the discharge of dredged or fill material into the waters of the United States or the transportation of dredged material for the purpose of disposing of it in ocean waters the application must include the source of the material; the purpose of the discharge, a description of the type, composition and quantity of the material; the method of transportation and disposal of the material; and the location of the disposal site. Certification under section 401 of the Clean Water Act is required for such discharges into waters of the United States.

(5) If the activity would include the construction of a filled area or pile or float-supported platform the project description must include the use of, and specific structures to be erected on, the fill or platform.

(6) If the activity would involve the construction of an impoundment structure, the applicant may be required to demonstrate that the structure complies with established state dam safety criteria or that the structure has been designed by qualified persons and, in appropriate cases, independently reviewed (and modified as the review would indicate) by similiarly qualified persons. No specific design criteria are to be prescribed nor is an independent detailed engineering review to be made by the district engineer.

(7) *Signature on application.* The application must be signed by the person who desires to undertake the proposed activity (i.e. the applicant) or by a duly authorized agent. When the applicant is represented by an agent, that information will be included in the space provided on the application or by a separate written statement. The signature of the applicant or the agent will be an affirmation that the applicant possesses or will possess the requisite property interest to undertake the activity proposed in the application, except where the lands are under the control of the Corps of Engineers, in which cases the district engineer will coordinate the transfer of the real estate and the permit action. An application may include the activity of more than one owner provided the character of the activity of each owner is similar and in the same general area and each owner submits a statement designating the same agent.

(8) If the activity would involve the construction or placement of an artificial reef, as defined in 33 CFR 322.2(g), in the navigable waters of the United States or in the waters overlying the outer continental shelf, the application must include provisions for siting, constructing, monitoring, and managing the artificial reef.

(9) *Complete application.* An application will be determined to be complete when sufficient information is received to issue a public notice (See 33 CFR 325.1(d) and 325.3(a).) The issuance of a public notice will not be delayed to obtain information necessary to evaluate an application.

(e) *Additional information.* In addition to the information indicated in paragraph (d) of this section, the applicant will be required to furnish only such additional information as the district engineer deems essential to make a public interest determination including, where applicable, a determination of compliance with the section 404(b)(1) guidelines or ocean dumping criteria. Such additional information may include environmental data and information on alternate methods and sites as may be necessary for the preparation of the required environmental documentation.

(f) *Fees.* Fees are required for permits under section 404 of the Clean Water Act, section 103 of the Marine Protection, Research and Sanctuaries Act of 1972, as amended, and sections 9 and 10 of the Rivers and Harbors Act of 1899. A fee of $100.00 will be charged when the planned or ultimate purpose of the project is commercial or industrial in nature and is in support of operations that charge for the production, distribution or sale of goods or services. A $10.00 fee will be charged for permit applications when the proposed work is non-commercial in nature and would provide personal benefits that have no connection with a commercial enterprise. The final decision as to the basis for a fee (commercial vs. non-commercial) shall be solely the responsibility of the district engineer. No fee will be charged if the applicant withdraws the application at any time prior to issuance of the permit or if the permit is denied. Collection of the fee will be deferred until the proposed activity has been determined to be not contrary to the public interest. Multiple fees are not to be charged if more than one law is applicable. Any modification significant enough to require publication of a public notice will also require a fee. No fee will be assessed when a permit is transferred from one property owner to another. No fees will be charged for time extensions, general permits or letters of permission. Agencies or instrumentalities of federal, state or local governments will not be required to pay any fee in connection with permits.

§ 325.2 Processing of applications.

(a) *Standard procedures.* (1) When an application for a permit is received the district engineer shall immediately assign it a number for identification, acknowledge receipt thereof, and advise the applicant of the number assigned to it. He shall review the application for completeness, and if the application is incomplete, request from the applicant within 15 days of receipt of the application any additional information necessary for further processing.

(2) Within 15 days of receipt of an application the district engineer will either determine that the application is complete (see 33 CFR 325.1(d)(9) and issue a public notice as described in § 325.3 of this Part, unless specifically exempted by other provisions of this

regulation or that it is incomplete and notify the applicant of the information necessary for a complete application. The district engineer will issue a supplemental, revised, or corrected public notice if in his view there is a change in the application data that would affect the public's review of the proposal.

(3) The district engineer will consider all comments received in response to the public notice in his subsequent actions on the permit application. Receipt of the comments will be acknowledged, if appropriate, and they will be made a part of the administrative record of the application. Comments received as form letters or petitions may be acknowledged as a group to the person or organization responsible for the form letter or petition. If comments relate to matters within the special expertise of another federal agency, the district engineer may seek the advice of that agency. If the district engineer determines, based on comments received, that he must have the views of the applicant on a particular issue to make a public interest determination, he applicant will be given the opportunity to furnish his views on such issue to the district engineer (see § 325.2(d)(5)). At the earliest practicable time other substantive comments will be furnished to the applicant for his information and any views he may wish to offer. A summary of the comments, the actual letters or portions thereof, or representative comment letters may be furnished to the applicant. The applicant may voluntarily elect to contact objectors in an attempt to resolve objections but will not be required to do so. District engineers will ensure that all parties are informed that the Corps alone is responsible for reaching a decision on the merits of any application. The district engineer may also offer Corps regulatory staff to be present at meetings between applicants and objectors, where appropriate, to provide information on the process, to mediate differences, or to gather information to aid in the decision process. The district engineer should not delay processing of the application unless the applicant requests a reasonable delay, normally not to exceed 30 days, to provide additional information or comments.

(4) The district engineer will follow Appendix B of 33 CFR Part 230 for environmental procedures and documentation required by the National Environmental Policy Act of 1969. A decision on a permit application will require either an environmental assessment or an environmental impact statement unless it is included within a categorical exclusion.

(5) The district engineer will also evaluate the application to determine the need for a public hearing pursuant to 33 CFR Part 327.

(6) After all above actions have been completed, the district engineer will determine in accordance with the record and applicable regulations whether or not the permit should be issued. He shall prepare a statement of findings (SOF) or, where an EIS has been prepared, a record of decision (ROD), on all permit decisions. The SOF or ROD shall include the district engineer's views on the probable effect of the proposed work on the public interest including conformity with the guidelines published for the discharge of dredged or fill material into waters of the United States (40 CFR Part 230) or with the criteria for dumping of dredged material in ocean waters (40 CFR Parts 220 to 229), if applicable, and the conclusions of the district engineer. The SOF or ROD shall be dated, signed, and included in the record prior to final action on the application. Where the district engineer has delegated authority to sign permits for and in his behalf, he may similarly delegate the signing of the SOF or ROD. If a district engineer makes a decision on a permit application which is contrary to state or local decisions (33 CFR 320.4(j) (2) & (4)), the district engineer will include in the decision document the significant national issues and explain how they are overriding in importance. If a permit is warranted, the district engineer will determine the special conditions, if any, and duration which should be incorporated into the permit. In accordance with the authorities specified in Section 325.8 of this Part, the district engineer will take final action or forward the application with all pertinent comments, records, and studies, including the final EIS or environmental assessment, through channels to the official authorized to make the final decision. The report forwarding the application for decision will be in a format prescribed by the Chief of Engineers. District and division engineers will notify the applicant and interested federal and state agencies that the application has been forwarded to higher headquarters. The district or division engineer may, at his option, disclose his recommendation to the news media and other interested parties, with the caution that it is only a recommendation and not a final decision. Such disclosure is encouraged in permit cases which have become controversial and have been the subject of stories in the media or have generated strong public interest. In those cases where the application is forwarded for decision in the format prescribed by the Chief of Engineers, the report will serve as the SOF or ROD. District engineers will generally combine the SOF, environmental assessment, and findings of no significant impact (FONSI), 404(b)(1) guideline analysis, and/or the criteria for dumping of dredged material in ocean waters into a single document.

(7) If the final decision is to deny the permit, the applicant will be advised in writing of the reason(s) for denial. If the final decision is to issue the permit and a standard individual permit form will be used, the issuing official will forward the permit to the applicant for signature accepting the conditions of the permit. The permit is not valid until signed by the issuing official. Letters of permission require only the signature of the issuing official. Final action on the permit application is the signature on the letter notifying the applicant of the denial of the permit or signature of the issuing official on the authorizing document.

(8) The district engineer will publish monthly a list of permits issued or denied during the previous month. The list will identify each action by public notice number, name of applicant, and brief description of activity involved. It will also note that relevant environmental documents and the SOF's or ROD's are available upon written request and, where applicable, upon the payment of administrative fees. This list will be distributed to all persons who may have an interest in any of the public notices listed.

(9) Copies of permits will be furnished to other agencies in appropriate cases as follows:

(i) If the activity involves the construction of artificial islands, installations or other devices on the outer continental shelf, to the Director, Defense Mapping Agency, Hydrographic Center, Washington, DC 20390 Attention, Code NS12, and to the Charting and Geodetic Services, N/CG222, National Ocean Service NOAA, Rockville, Maryland 20852.

(ii) If the activity involves the construction of structures to enhance fish propagation (e.g., fishing reefs) along the coasts of the United States, to the Defense Mapping Agency, Hydrographic Center and National Ocean Service as in paragraph (a)(9)(i) of this section and to the Director, Office of Marine Recreational Fisheries, National Marine Fisheries Service, Washington, DC 20235.

(iii) If the activity involves the erection of an aerial transmission line, submerged cable, or submerged pipeline

roes a navigable water of the United States, to the Charting and Geodetic Services N/CG222, National Ocean Service NOAA, Rockville, Maryland 20852.

(iv) If the activity is listed in paragraphs (a)(9) (i), (ii), or (iii) of this section, or involves the transportation of dredged material for the purpose of dumping it in ocean waters, to the appropriate District Commander, U.S. Coast Guard.

(b) *Procedures for particular types of permit situations.*—(1) *Section 401 Water Quality Certification.* If the district engineer determines that water quality certification for the proposed activity is necessary under the provisions of section 401 of the Clean Water Act, he shall so notify the applicant and obtain from him or the certifying agency a copy of such certification.

(i) The public notice for such activity, which will contain a statement on certification requirements (see § 325.3(a)(8)), will serve as the notification to the Administrator of the Environmental Protection Agency (EPA) pursuant to section 401(a)(2) of the Clean Water Act. If EPA determines that the proposed discharge may affect the quality of the waters of any state other than the state in which the discharge will originate, it will so notify such other state, the district engineer, and the applicant. If such notice or a request for supplemental information is not received within 30 days of issuance of the public notice, the district engineer will assume EPA has made a negative determination with respect to section 401(a)(2). If EPA determines another state's waters may be affected, such state has 60 days from receipt of EPA's notice to determine if the proposed discharge will affect the quality of its waters so as to violate any water quality requirement in such state, to notify EPA and the district engineer in writing of its objection to permit issuance, and to request a public hearing. If such occurs, the district engineer will hold a public hearing in the objecting state. Except as stated below, the hearing will be conducted in accordance with 33 CFR Part 327. The issues to be considered at the public hearing will be limited to water quality impacts. EPA will submit its evaluation and recommendations at the hearing with respect to the state's objection to permit issuance. Based upon the recommendations of the objecting state, EPA, and any additional evidence presented at the hearing, the district engineer will condition the permit, if issued, in such a manner as may be necessary to insure compliance with applicable water quality requirements. If the imposition of conditions cannot, in the district engineer's opinion, insure such compliance, he will deny the permit.

(ii) No permit will be granted until required certification has been obtained or has been waived. A waiver may be explicit or will be deemed to occur if the certifying agency fails or refuses to act on a request for certification within sixty days after receipt of such a request unless the district engineer determines a shorter or longer period is reasonable for the state to act. In determining whether or not a waiver period has commenced or waiver has occurred, the district engineer will verify that the certifying agency has received a valid request for certification. If, however, special circumstances identified by the district engineer require that action on an application be taken within a more limited period of time, the district engineer shall determine a reasonable lesser period of time, advise the certifying agency of the need for action by a particular date, and that, if certification is not received by that date, it will be considered that the requirement for certification has been waived. Similarly, if it appears that circumstances may reasonably require a period of time longer than sixty days, the district engineer, based on information provided by the certifying agency, will determine a longer reasonable period of time, not to exceed one year, at which time a waiver will be deemed to occur.

(2) *Coastal Zone Management Consistency.* If the proposed activity is to be undertaken in a state operating under a coastal zone management program approved by the Secretary of Commerce pursuant to the Coastal Zone Management (CZM) Act (see 33 CFR 320.3(b)), the district engineer shall proceed as follows:

(i) If the applicant is a federal agency, and the application involves a federal activity in or affecting the coastal zone, the district engineer shall forward a copy of the public notice to the agency of the state responsible for reviewing the consistency of federal activities. The federal agency applicant shall be responsible for complying with the CZM Act's directive for ensuring that federal agency activities are undertaken in a manner which is consistent, to the maximum extent practicable, with approved CZM Programs. (See 15 CFR Part 930.) If the state coastal zone agency objects to the proposed federal activity on the basis of its inconsistency with the state's approved CZM Program, the district engineer shall not make a final decision on the application until the disagreeing parties have had an opportunity to utilize the procedures specified by the CZM Act for resolving such disagreements.

(ii) If the applicant is not a federal agency and the application involves an activity affecting the coastal zone, the district engineer shall obtain from the applicant a certification that his proposed activity complies with and will be conducted in a manner that is consistent with the approved state CZM Program. Upon receipt of the certification, the district engineer will forward a copy of the public notice (which will include the applicant's certification statement) to the state coastal zone agency and request its concurrence or objection. If the state agency objects to the certification or issues a decision indicating that the proposed activity requires further review, the district engineer shall not issue the permit until the state concurs with the certification statement or the Secretary of Commerce determines that the proposed activity is consistent with the purposes of the CZM Act or is necessary in the interest of national security. If the state agency fails to concur or object to a certification statement within six months of the state agency's receipt of the certification statement, state agency concurrence with the certification statement shall be conclusively presumed. District engineers will seek agreements with state CZM agencies that the agency's failure to provide comments during the public notice comment period will be considered as a concurrence with the certification or waiver of the right to concur or non-concur.

(iii) If the applicant is requesting a permit for work on Indian reservation lands which are in the coastal zone, the district engineer shall treat the application in the same manner as prescribed for a Federal applicant in paragraph (b)(2)(i) of this section. However, if the applicant is requesting a permit on non-trust Indian lands, and the state CZM agency has decided to assert jurisdiction over such lands, the district engineer shall treat the application in the same manner as prescribed for a non-Federal applicant in paragraph (b)(2)(ii) of this section.

(3) *Historic Properties.* If the proposed activity would involve any property listed or eligible for listing in the National Register of Historic Places, the district engineer will proceed in accordance with Corps National Historic Preservation Act implementing regulations.

(4) *Activities Associated with Federal Projects.* If the proposed activity would consist of the dredging of an access channel and/or berthing facility associated with an authorized federal navigation project, the activity will be included in the planning and coordination of the construction or maintenance of the federal project to the maximum extent feasible. Separate notice, hearing, and environmental documentation will not be required for activities so included and coordinated, and the public notice issued by the district engineer for these federal and associated non-federal activities will be the notice of intent to issue permits for those included non-federal dredging activities. The decision whether to issue or deny such a permit will be consistent with the decision on the federal project unless special considerations applicable to the proposed activity are identified. (See § 322.5(c).)

(5) *Endangered Species.* Applications will be reviewed for the potential impact on threatened or endangered species pursuant to section 7 of the Endangered Species Act as amended. The district engineer will include a statement in the public notice of his current knowledge of endangered species based on his initial review of the application (see 33 CFR 325.2(a)(2)). If the district engineer determines that the proposed activity would not affect listed species or their critical habitat, he will include a statement to this effect in the public notice. If he finds the proposed activity may affect an endangered or threatened species or their critical habitat, he will initiate formal consultation procedures with the U.S. Fish and Wildlife Service or National Marine Fisheries Service. Public notices forwarded to the U.S. Fish and Wildlife Service or National Marine Fisheries Service will serve as the request for information on whether any listed or proposed to be listed endangered or threatened species may be present in the area which would be affected by the proposed activity, pursuant to section 7(c) of the Act. References, definitions, and consultation procedures are found in 50 CFR Part 402.

(c) [Reserved]

(d) *Timing of processing of applications.* The district engineer will be guided by the following time limits for the indicated steps in the evaluation process:

(1) The public notice will be issued within 15 days of receipt of all information required to be submitted by the applicant in accordance with paragraph 325.1.(d) of this Part.

(2) The comment period on the public notice should be for a reasonable period of time within which interested parties may express their views concerning the permit. The comment period should not be more than 30 days nor less than 15 days from the date of the notice. Before designating comment periods less than 30 days, the district engineer will consider: (i) Whether the proposal is routine or noncontroversial, (ii) mail time and need for comments from remote areas, (iii) comments from similar proposals, and (iv) the need for a site visit. After considering the length of the original comment period, paragraphs (a)(2) (i) through (iv) of this section, and other pertinent factors, the district engineer may extend the comment period up to an additional 30 days if warranted.

(3) District engineers will decide on all applications not later than 60 days after receipt of a complete application, unless (i) precluded as a matter of law or procedures required by law (see below), (ii) the case must be referred to higher authority (see § 325.8 of this Part), (iii) the comment period is extended, (iv) a timely submittal of information or comments is not received from the applicant, (v) the processing is suspended at the request of the applicant, or (vi) information needed by the district engineer for a decision on the application cannot reasonably be obtained within the 60-day period. Once the cause for preventing the decision from being made within the normal 60-day period has been satisfied or eliminated, the 60-day clock will start running again from where it was suspended. For example, if the comment period is extended by 30 days, the district engineer will, absent other restraints, decide on the application within 90 days of receipt of a complete application. Certain laws (e.g., the Clean Water Act, the CZM Act, the National Environmental Policy Act, the National Historic Preservation Act, the Preservation of Historical and Archeological Data Act, the Endangered Species Act, the Wild and Scenic Rivers Act, and the Marine Protection, Research and Sanctuaries Act) require procedures such as state or other federal agency certifications, public hearings, environmental impact statements, consultation, special studies, and testing which may prevent district engineers from being able to decide certain applications within 60 days.

(4) Once the district engineer has sufficient information to make his public interest determination, he should decide the permit application even though other agencies which may have regulatory jurisdiction have not yet granted their authorizations, except where such authorizations are, by federal law, a prerequisite to making a decision on the DA permit application. Permits granted prior to other (non-prerequisite) authorizations by other agencies should, where appropriate, be conditioned in such manner as to give those other authorities an opportunity to undertake their review without the applicant biasing such review by making substantial resource commitments on the basis of the DA permit. In unusual cases the district engineer may decide that due to the nature or scope of a specific proposal, it would be prudent to defer taking final action until another agency has acted on its authorization. In such cases, he may advise the other agency of his position on the DA permit while deferring his final decision.

(5) The applicant will be given a reasonable time, not to exceed 30 days, to respond to requests of the district engineer. The district engineer may make such requests by certified letter and clearly inform the applicant that if he does not respond with the requested information or a justification why additional time is necessary, then his application will be considered withdrawn or a final decision will be made, whichever is appropriate. If additional time is requested, the district engineer will either grant the time, make a final decision, or consider the application as withdrawn.

(6) The time requirements in these regulations are in terms of calendar days rather than in terms of working days.

(e) *Alternative procedures.* Division and district engineers are authorized to use alternative procedures as follows:

(1) *Letters of permission.* Letters of permission are a type of permit issued through an abbreviated processing procedure which includes coordination with Federal and state fish and wildlife agencies, as required by the Fish and Wildlife Coordination Act, and a public interest evaluation, but without the publishing of an individual public notice. The letter of permission will not be used to authorize the transportation of dredged material for the purpose of dumping it in ocean waters. Letters of permission may be used:

(i) In those cases subject to section 10 of the Rivers and Harbors Act of 1899 when, in the opinion of the district engineer, the proposed work would be minor, would not have significant individual or cumulative impacts on environmental values, and should encounter no appreciable opposition.

(ii) In those cases subject to section 404 of the Clean Water Act after:

(A) The district engineer, through consultation with Federal and state fish and wildlife agencies, the Regional

Administrator, Environmental Protection Agency, the state water quality certifying agency, and, if appropriate, the state Coastal Zone Management Agency, develops a list of categories of activities proposed for authorization under LOP procedures;

(B) The district engineer issues a public notice advertising the proposed list and the LOP procedures, requesting comments and offering an opportunity for public hearing; and

(C) A 401 certification has been issued or waived and, if appropriate, CZM consistency concurrence obtained or presumed either on a generic or individual basis.

(2) *Regional permits.* Regional permits are a type of general permit as defined in 33 CFR 322.2(f) and 33 CFR 323.2(n). They may be issued by a division or district engineer after compliance with the other procedures of this regulation. After a regional permit has been issued, individual activities falling within those categories that are authorized by such regional permits do not have to be further authorized by the procedures of this regulation. The issuing authority will determine and add appropriate conditions to protect the public interest. When the issuing authority determines on a case-by-case basis that the concerns for the aquatic environment so indicate, he may exercise discretionary authority to override the regional permit and require an individual application and review. A regional permit may be revoked by the issuing authority if it is determined that it is contrary to the public interest provided the procedures of § 325.7 of this Part are followed. Following revocation, applications for future activities in areas covered by the regional permit shall be processed as applications for individual permits. No regional permit shall be issued for a period of more than five years.

(3) *Joint procedures.* Division and district engineers are authorized and encouraged to develop joint procedures with states and other Federal agencies with ongoing permit programs for activities also regulated by the Department of the Army. Such procedures may be substituted for the procedures in paragraphs (a)(1) through (a)(5) of this section provided that the substantive requirements of those sections are maintained. Division and district engineers are also encouraged to develop management techniques such as joint agency review meetings to expedite the decision-making process. However, in doing so, the applicant's rights to a full public interest review and independent decision by the district or division engineer must be strictly observed.

(4) *Emergency procedures.* Division engineers are authorized to approve special processing procedures in emergency situations. An "emergency" is a situation which would result in an unacceptable hazard to life, a significant loss of property, or an immediate, unforeseen, and significant economic hardship if corrective action requiring a permit is not undertaken within a time period less than the normal time needed to process the application under standard procedures. In emergency situations, the district engineer will explain the circumstances and recommend special procedures to the division engineer who will instruct the district engineer as to further processing of the application. Even in an emergency situation, reasonable efforts will be made to receive comments from interested Federal, state, and local agencies and the affected public. Also, notice of any special procedures authorized and their rationale is to be appropriately published as soon as practicable.

§ 325.3 Public notice.

(a) *General.* The public notice is the primary method of advising all interested parties of the proposed activity for which a permit is sought and of soliciting comments and information necessary to evaluate the probable impact on the public interest. The notice must, therefore, include sufficient information to give a clear understanding of the nature and magnitude of the activity to generate meaningful comment. The notice should include the following items of information:

(1) Applicable statutory authority or authorities;

(2) The name and address of the applicant;

(3) The name or title, address and telephone number of the Corps employee from whom additional information concerning the application may be obtained;

(4) The location of the proposed activity;

(5) A brief description of the proposed activity, its purpose and intended use, so as to provide sufficient information concerning the nature of the activity to generate meaningful comments, including a description of the type of structures, if any, to be erected on fills or pile or float-supported platforms, and a description of the type, composition, and quantity of materials to be discharged or disposed of in the ocean;

(6) A plan and elevation drawing showing the general and specific site location and character of all proposed activities, including the size relationship of the proposed structures to the size of the impacted waterway and depth of water in the area;

(7) If the proposed activity would occur in the territorial seas or ocean waters, a description of the activity's relationship to the baseline from which the territorial sea is measured;

(8) A list of other government authorizations obtained or requested by the applicant, including required certifications relative to water quality, coastal zone management, or marine sanctuaries;

(9) If appropriate, a statement that the activity is a categorical exclusion for purposes of NEPA (see paragraph 7 of Appendix B to 33 CFR Part 230);

(10) A statement of the district engineer's current knowledge on historic properties;

(11) A statement of the district engineer's current knowledge on endangered species (see § 325.2(b)(5));

(12) A statement(s) on evaluation factors (see § 325.3(c));

(13) Any other available information which may assist interested parties in evaluating the likely impact of the proposed activity, if any, on factors affecting the public interest;

(14) The comment period based on § 325.2(d)(2);

(15) A statement that any person may request, in writing, within the comment period specified in the notice, that a public hearing be held to consider the application. Requests for public hearings shall state, with particularity, the reasons for holding a public hearing;

(16) For non-federal applications in states with an approved CZM Plan, a statement on compliance with the approved Plan; and

(17) In addition, for section 103 (ocean dumping) activities:

(i) The specific location of the proposed disposal site and its physical boundaries;

(ii) A statement as to whether the proposed disposal site has been designated for use by the Administrator, EPA, pursuant to section 102(c) of the Act;

(iii) If the proposed disposal site has not been designated by the Administrator, EPA, a description of the characteristics of the proposed disposal site and an explanation as to why no previously designated disposal site is feasible;

(iv) A brief description of known dredged material discharges at the proposed disposal site;

(v) Existence and documented effects of other authorized disposals that have been made in the disposal area (e.g.,

heavy metal background reading and organic carbon content);

(vi) An estimate of the length of time during which disposal would continue at the proposed site; and

(vii) Information on the characteristics and composition of the dredged material.

(b) *Public notice for general permits.* District engineers will publish a public notice for all proposed regional general permits and for significant modifications to, or reissuance of, existing regional permits within their area of jurisdiction. Public notices for statewide regional permits may be issued jointly by the affected Corps districts. The notice will include all applicable information necessary to provide a clear understanding of the proposal. In addition, the notice will state the availability of information at the district office which reveals the Corps' provisional determination that the proposed activities comply with the requirements for issuance of general permits. District engineers will publish a public notice for nationwide permits in accordance with 33 CFR 330.4.

(c) *Evaluation factors.* A paragraph describing the various evaluation factors on which decisions are based shall be included in every public notice.

(1) Except as provided in paragraph (c)(3) of this section, the following will be included:

"The decision whether to issue a permit will be based on an evaluation of the probable impact including cumulative impacts of the proposed activity on the public interest. That decision will reflect the national concern for both protection and utilization of important resources. The benefit which reasonably may be expected to accrue from the proposal must be balanced against its reasonably foreseeable detriments. All factors which may be relevant to the proposal will be considered including the cumulative effects thereof; among those are conservation, economics, aesthetics, general environmental concerns, wetlands, historic properties, fish and wildlife values, flood hazards, floodplain values, land use, navigation, shoreline erosion and accretion, recreation, water supply and conservation, water quality, energy needs, safety, food and fiber production, mineral needs, considerations of property ownership and, in general, the needs and welfare of the people."

(2) If the activity would involve the discharge of dredged or fill material into the waters of the United States or the transportation of dredged material for the purpose of disposing of it in ocean waters, the public notice shall also indicate that the evaluation of the impact of the activity on the public interest will include application of the guidelines promulgated by the Administrator, EPA, (40 CFR Part 230) or of the criteria established under authority of section 102(a) of the Marine Protection, Research and Sanctuaries Act of 1972, as amended (40 CFR Parts 220 to 229), as appropriate. (See 33 CFR Parts 323 and 324).

(3) In cases involving construction of artificial islands, installations and other devices on outer continental shelf lands which are under mineral lease from the Department of the Interior, the notice will contain the following statement: "The decision as to whether a permit will be issued will be based on an evaluation of the impact of the proposed work on navigation and national security."

(d) *Distribution of public notices.* (1) Public notices will be distributed for posting in post offices or other appropriate public places in the vicinity of the site of the proposed work and will be sent to the applicant, to appropriate city and county officials, to adjoining property owners, to appropriate state agencies, to appropriate Indian Tribes or tribal representatives, to concerned Federal agencies, to local, regional and national shipping and other concerned business and conservation organizations, to appropriate River Basin Commissions, to appropriate state and areawide clearing houses as prescribed by OMB Circular A-95, to local news media and to any other interested party. Copies of public notices will be sent to all parties who have specifically requested copies of public notices, to the U.S. Senators and Representatives for the area where the work is to be performed, the field representative of the Secretary of the Interior, the Regional Director of the Fish and Wildlife Service, the Regional Director of the National Park Service, the Regional Administrator of the Environmental Protection Agency (EPA), the Regional Director of the National Marine Fisheries Service of the National Oceanic and Atmospheric Administration (NOAA), the head of the state agency responsible for fish and wildlife resources, the State Historic Preservation Officer, and the District Commander, U.S. Coast Guard.

(2) In addition to the general distribution of public notices cited above, notices will be sent to other addressees in appropriate cases as follows:

(i) If the activity would involve structures or dredging along the shores of the seas or Great Lakes, to the Coastal Engineering Research Center, Washington, DC 20016.

(ii) If the activity would involve construction of fixed structures or artificial islands on the outer continental shelf or in the territorial seas, to the Assistant Secretary of Defense (Manpower, Installations, and Logistics (ASD(MI&L)), Washington, DC 20310; the Director, Defense Mapping Agency (Hydrographic Center) Washington, DC 20390, Attention, Code NS12; and the Charting and Geodetic Services, N/CG222, National Ocean Service NOAA, Rockville, Maryland 20852, and to affected military installations and activities.

(iii) If the activity involves the construction of structures to enhance fish propagation (e.g., fishing reefs) along the coasts of the United States, to the Director, Office of Marine Recreational Fisheries, National Marine Fisheries Service, Washington, DC 20235.

(iv) If the activity involves the construction of structures which may affect aircraft operations or for purposes associated with seaplane operations, to the Regional Director of the Federal Aviation Administration.

(v) If the activity would be in connection with a foreign-trade zone, to the Executive Secretary, Foreign-Trade Zones Board, Department of Commerce, Washington, DC 20230 and to the appropriate District Director of Customs as Resident Representative, Foreign-Trade Zones Board.

(3) It is presumed that all interested parties and agencies will wish to respond to public notices; therefore, a lack of response will be interpreted as meaning that there is no objection to the proposed project. A copy of the public notice with the list of the addressees to whom the notice was sent will be included in the record. If a question develops with respect to an activity for which another agency has responsibility and that other agency has not responded to the public notice, the district engineer may request its comments. Whenever a response to a public notice has been received from a member of Congress, either in behalf of a constituent or himself, the district engineer will inform the member of Congress of the final decision.

(4) District engineers will update public notice mailing lists at least once every two years.

§ 325.4. Conditioning of permits.

(a) District engineers will add special conditions to Department of the Army permits when such conditions are necessary to satisfy legal requirements or to otherwise satisfy the public interest requirement. Permit conditions will be directly related to the impacts of the proposal, appropriate to the scope and degree of those impacts, and reasonably enforceable.

(1) Legal requirements which may be satisfied by means of Corps permit conditions include compliance with the 404(b)(1) guidelines, the EPA ocean dumping criteria, the Endangered Species Act, and requirements imposed by conditions on state section 401 water quality certifications.

(2) Where appropriate, the district engineer may take into account the existence of controls imposed under other federal, state, or local programs which would achieve the objective of the desired condition, or the existence of an enforceable agreement between the applicant and another party concerned with the resource in question, in determining whether a proposal complies with the 404(b)(1) guidelines, ocean dumping criteria, and other applicable statutes, and is not contrary to the public interest. In such cases, the Department of the Army permit will be conditioned to state that material changes in, or a failure to implement and enforce such program or agreement, will be grounds for modifying, suspending, or revoking the permit.

(3) Such conditions may be accomplished on-site, or may be accomplished off-site for mitigation of significant losses which are specifically identifiable, reasonably likely to occur, and of importance to the human or aquatic environment.

(b) District engineers are authorized to add special conditions, exclusive of paragraph (a) of this section, at the applicant's request or to clarify the permit application.

(c) If the district engineer determines that special conditions are necessary to insure the proposal will not be contrary to the public interest, but those conditions would not be reasonably implementable or enforceable, he will deny the permit.

(d) *Bonds.* If the district engineer has reason to consider that the permittee might be prevented from completing work which is necessary to protect the public interest, he may require the permittee to post a bond of sufficient amount to indemnify the government against any loss as a result of corrective action it might take.

§ 325.5 Forms of permits.

(a) *General discussion.* (1) DA permits under this regulation will be in the form of individual permits or general permits. The basic format shall be ENG Form 1721, DA Permit (Appendix A).

(2) The general conditions included in ENG Form 1721 are normally applicable to all permits; however, some conditions may not apply to certain permits and may be deleted by the issuing officer. Special conditions applicable to the specific activity will be included in the permit as necessary to protect the public interest in accordance with Section 325.4 of this Part.

(b) *Individual permits*—(1) *Standard permits.* A standard permit is one which has been processed through the public interest review procedures, including public notice and receipt of comments, described throughout this Part. The standard individual permit shall be issued using ENG Form 1721.

(2) *Letters of permission.* A letter of permission will be issued where procedures of paragraph 325.2(e)(1) have been followed. It will be in letter form and will identify the permittee, the authorized work and location of the work, the statutory authority, any limitations on the work, a construction time limit and a requirement for a report of completed work. A copy of the relevant general conditions from ENG Form 1721 will be attached and will be incorporated by reference into the letter of permission.

(c) *General permits*—(1) *Regional permits.* Regional permits are a type of general permit. They may be issued by a division or district engineer after compliance with the other procedures of this regulation. If the public interest so requires, the issuing authority may condition the regional permit to require a case-by-case reporting and acknowledgment system. However, no separate applications or other authorization documents will be required.

(2) *Nationwide permits.* Nationwide permits are a type of general permit and represent DA authorizations that have been issued by the regulation (33 CFR Part 330) for certain specified activities nationwide. If certain conditions are met, the specified activities can take place without the need for an individual or regional permit.

(3) *Programmatic permits.* Programmatic permits are a type of general permit founded on an existing state, local or other Federal agency program and designed to avoid duplication with that program.

(d) *Section 9 permits.* Permits for structures in interstate navigable waters of the United States under section 9 of the Rivers and Harbors Act of 1899 will be drafted at DA level.

§ 325.6 Duration of permits.

(a) *General.* DA permits may authorize both the work and the resulting use. Permits continue in effect until they automatically expire or are modified, suspended, or revoked.

(b) *Structures.* Permits for the existence of a structure or other activity of a permanent nature are usually for an indefinite duration with no expiration date cited. However, where a temporary structure is authorized, or where restoration of a waterway is contemplated, the permit will be of limited duration with a definite expiration date.

(c) *Works.* Permits for construction work, discharge of dredged or fill material, or other activity and any construction period for a structure with a permit of indefinite duration under paragraph (b) of this section will specify time limits for completing the work or activity. The permit may also specify a date by which the work must be started, normally within one year from the date of issuance. The date will be established by the issuing official and will provide reasonable times based on the scope and nature of the work involved. Permits issued for the transport of dredged material for the purpose of disposing of it in ocean waters will specify a completion date for the disposal not to exceed three years from the date of permit issuance.

(d) *Extensions of time.* An authorization or construction period will automatically expire if the permittee fails to request and receive an extension of time. Extensions of time may be granted by the district engineer. The permittee must request the extension and explain the basis of the request, which will be granted unless the district engineer determines that an extension would be contrary to the public interest. Requests for extensions will be processed in accordance with the regular procedures of § 325.2 of this Part, including issuance of a public notice, except that such processing is not required where the district engineer determines that there have been no significant changes in the attendant circumstances since the authorization was issued.

(e) *Maintenance dredging.* If the authorized work includes periodic maintenance dredging, an expiration date for the authorization of that maintenance dredging will be included in the permit. The expiration date, which in no event is to exceed ten years from the date of issuance of the permit, will be established by the issuing official after evaluation of the proposed method of dredging and disposal of the dredged material in accordance with the requirements of 33 CFR Parts 320 to 225. In such cases, the district engineer shall require notification of the maintenance dredging prior to actual performance to insure continued compliance with the requirements of this regulation and 33 CFR Parts 320 to 324. If the permittee desires to continue maintenance

dredging beyond the expiration date, he must request a new permit. The permittee should be advised to apply for the new permit six months prior to the time he wishes to do the maintenance work.

§ 325.7 Modification, suspension, or revocation of permits.

(a) *General.* The district engineer may reevaluate the circumstances and conditions of any permit, including regional permits, either on his own motion, at the request of the permittee, or a third party, or as the result of periodic progress inspections, and initiate action to modify, suspend, or revoke a permit as may be made necessary by considerations of the public interest. In the case of regional permits, this reevaluation may cover individual activities, categories of activities, or geographic areas. Among the factors to be considered are the extent of the permittee's compliance with the terms and conditions of the permit; whether or not circumstances relating to the authorized activity have changed since the permit was issued or extended, and the continuing adequacy of or need for the permit conditions; any significant objections to the authorized activity which were not earlier considered; revisions to applicable statutory and/or regulatory authorities; and the extent to which modification, suspension, or other action would adversely affect plans, investments and actions the permittee has reasonably made or taken in reliance on the permit. Significant increases in scope of a permitted activity will be processed as new applications for permits in accordance with § 325.2 of this Part, and not as modifications under this section.

(b) *Modification.* Upon request by the permittee or, as a result of reevaluation of the circumstances and conditions of a permit, the district engineer may determine that the public interest requires a modification of the terms or conditions of the permit. In such cases, the district engineer will hold informal consultations with the permittee to ascertain whether the terms and conditions can be modified by mutual agreement. If a mutual agreement is reached on modification of the terms and conditions of the permit, the district engineer will give the permittee written notice of the modification, which will then become effective on such date as the district engineer may establish. In the event a mutual agreement cannot be reached by the district engineer and the permittee, the district engineer will proceed in accordance with paragraph (c) of this section if immediate suspension is warranted. In cases where immediate suspension is not warranted but the district engineer determines that the permit should be modified, he will notify the permittee of the proposed modification and reasons therefor, and that he may request a meeting with the district engineer and/or a public hearing. The modification will become effective on the date set by the district engineer which shall be at least ten days after receipt of the notice by the permittee unless a hearing or meeting is requested within that period. If the permittee fails or refuses to comply with the modification, the district engineer will proceed in accordance with 33 CFR Part 326. The district engineer shall consult with resource agencies before modifying any permit terms or conditions, that would result in greater impacts, for a project about which that agency expressed a significant interest in the term, condition, or feature being modified prior to permit issuance.

(c) *Suspension.* The district engineer may suspend a permit after preparing a written determination and finding that immediate suspension would be in the public interest. The district engineer will notify the permittee in writing by the most expeditious means available that the permit has been suspended with the reasons therefor, and order the permittee to stop those activities previously authorized by the suspended permit. The permittee will also be advised that following this suspension a decision will be made to either reinstate, modify, or revoke the permit, and that he may within 10 days of receipt of notice of the suspension, request a meeting with the district engineer and/or a public hearing to present information in this matter. If a hearing is requested, the procedures prescribed in 33 CFR Part 327 will be followed. After the completion of the meeting or hearing (or within a reasonable period of time after issuance of the notice to the permittee that the permit has been suspended if no hearing or meeting is requested), the district engineer will take action to reinstate, modify, or revoke the permit.

(d) *Revocation.* Following completion of the suspension procedures in paragraph (c) of this section, if revocation of the permit is found to be in the public interest, the authority who made the decision on the original permit may revoke it. The permittee will be advised in writing of the final decision.

(e) *Regional permits.* The issuing official may, by following the procedures of this section, revoke regional permits for individual activities, categories of activities, or geographic areas. Where groups of permittees are involved, such as for categories of activities or geographic areas, the informal discussions provided in paragraph (b) of this section may be waived and any written notification may be made through the general public notice procedures of this regulation. If a regional permit is revoked, any permittee may then apply for an individual permit which shall be processed in accordance with these regulations.

§ 325.8 Authority to issue or deny permits.

(a) *General.* Except as otherwise provided in this regulation, the Secretary of the Army, subject to such conditions as he or his authorized representative may from time to time impose, has authorized the Chief of Engineers and his authorized representatives to issue or deny permits for dams or dikes in intrastate waters of the United States pursuant to section 9 of the Rivers and Harbors Act of 1899; for construction or other work in or affecting navigable waters of the United States pursuant to section 10 of the Rivers and Harbors Act of 1899; for the discharge of dredged or fill material into waters of the United States pursuant to section 404 of the Clean Water Act; or for the transportation of dredged material for the purpose of disposing of it into ocean waters pursuant to section 103 of the Marine Protection, Research and Sanctuaries Act of 1972, as amended. The authority to issue or deny permits in interstate navigable waters of the United States pursuant to section 9 of the Rivers and Harbors Act of March 3, 1899 has not been delegated to the Chief of Engineers or his authorized representatives.

(b) *District engineer's authority.* District engineers are authorized to issue or deny permits in accordance with these regulations pursuant to sections 9 and 10 of the Rivers and Harbors Act of 1899; section 404 of the Clean Water Act; and section 103 of the Marine Protection, Research and Sanctuaries Act of 1972, as amended, in all cases not required to be referred to higher authority (see below). It is essential to the legality of a permit that it contain the name of the district engineer as the issuing officer. However, the permit need not be signed by the district engineer in person but may be signed for and in behalf of him by whomever he designates. In cases where permits are denied for reasons other than navigation or failure to obtain required local, state, or other federal approvals or certifications, the Statement of Findings must conclusively justify a denial decision. District

engineers are authorized to deny permits without issuing a public notice or taking other procedural steps where required local, state, or other federal permits for the proposed activity have been denied or where he determines that the activity will clearly interfere with navigation except in all cases required to be referred to higher authority (see below). District engineers are also authorized to add, modify, or delete special conditions in permits in accordance with § 325.4 of this Part, except for those conditions which may have been imposed by higher authority, and to modify, suspend and revoke permits according to the procedures of § 325.7 of this Part. District engineers will refer the following applications to the division engineer for resolution:

(1) When a referral is required by a written agreement between the head of a Federal agency and the Secretary of the Army;

(2) When the recommended decision is contrary to the written position of the Governor of the state in which the work would be performed;

(3) When there is substantial doubt as to authority, law, regulations, or policies applicable to the proposed activity;

(4) When higher authority requests the application be forwarded for decision; or

(5) When the district engineer is precluded by law or procedures required by law from taking final action on the application (e.g. section 9 of the Rivers and Harbors Act of 1899, or territorial sea baseline changes).

(c) *Division engineer's authority.* Division engineers will review and evaluate all permit applications referred by district engineers. Division engineers may authorize the issuance or denial of permits pursuant to section 10 of the Rivers and Harbors Act of 1899; section 404 of the Clean Water Act; and section 103 of the Marine Protection, Research and Sanctuaries Act of 1972, as amended; and the inclusion of conditions in accordance with § 325.4 of this Part in all cases not required to be referred to the Chief of Engineers. Division engineers will refer the following applications to the Chief of Engineers for resolution:

(1) When a referral is required by a written agreement between the head of a Federal agency and the Secretary of the Army;

(2) When there is substantial doubt as to authority, law, regulations, or policies applicable to the proposed activity;

(3) When higher authority requests the application be forwarded for decision; or

(4) When the division engineer is precluded by law or procedures required by law from taking final action on the application.

§ 325.9 Authority to determine jurisdiction.

District engineers are authorized to determine the area defined by the terms "navigable waters of the United States" and "waters of the United States" except:

(a) When a determination of navigability is made pursuant to 33 CFR 329.14 (division engineers have this authority); or

(b) When EPA makes a section 404 jurisdiction determination under its authority.

§ 325.10 Publicity.

The district engineer will establish and maintain a program to assure that potential applicants for permits are informed of the requirements of this regulation and of the steps required to obtain permits for activities in waters of the United States or ocean waters. Whenever the district engineer becomes aware of plans being developed by either private or public entities which might require permits for implementation, he should advise the potential applicant in writing of the statutory requirements and the provisions of this regulation. Whenever the district engineer is aware of changes in Corps of Engineers regulatory jurisdiction, he will issue appropriate public notices.

Appendix A—Permit Form and Special Conditions

A. Permit Form

Department of the Army Permit

Permittee _____
Permit No. _____
Issuing Office _____

Note.—The term "you" and its derivatives, as used in this permit, means the permittee or any future transferee. The term "this office" refers to the appropriate district or division office of the Corps of Engineers having jurisdiction over the permitted activity or the appropriate official of that office acting under the authority of the commanding officer.

You are authorized to perform work in accordance with the terms and conditions specified below.

Project Description: (Describe the permitted activity and its intended use with references to any attached plans or drawings that are considered to be a part of the project description. Include a description of the types and quantities of dredged or fill materials to be discharged in jurisdictional waters.)

Project Location: (Where appropriate, provide the names of and the locations on the waters where the permitted activity and any off-site disposals will take place. Also, using name, distance, and direction, locate the permitted activity in reference to a nearby landmark such as a town or city.)

Permit Conditions:

General Conditions:

1. The time limit for completing the work authorized ends on _____. If you find that you need more time to complete the authorized activity, submit your request for a time extension to this office for consideration at least one month before the above date is reached.

2. You must maintain the activity authorized by this permit in good condition and in conformance with the terms and conditions of this permit. You are not relieved of this requirement if you abandon the permitted activity, although you may make a good faith transfer to a third party in compliance with General Condition 4 below. Should you wish to cease to maintain the authorized activity or should you desire to abandon it without a good faith transfer, you must obtain a modification of this permit from this office, which may require restoration of the area.

3. If you discover any previously unknown historic or archeological remains while accomplishing the activity authorized by this permit, you must immediately notify this office of what you have found. We will initiate the Federal and state coordination required to determine if the remains warrant a recovery effort or if the site is eligible for listing in the National Register of Historic Places.

4. If you sell the property associated with this permit, you must obtain the signature of the new owner in the space provided and forward a copy of the permit to this office to validate the transfer of this authorization.

5. If a conditioned water quality certification has been issued for your project, you must comply with the conditions specified in the certification as special conditions to this permit. For your convenience, a copy of the certification is attached if it contains such conditions.

6. You must allow representatives from this office to inspect the authorized activity at any time deemed necessary to ensure that it is being or has been accomplished in accordance with the terms and conditions of your permit.

Special Conditions: (Add special conditions as required in this space with reference to a continuation sheet if necessary.)

Further Information:

1. Congressional Authorities: You have been authorized to undertake the activity described above pursuant to:

( ) Section 10 of the Rivers and Harbors Act of 1899 (33 U.S.C. 403).

( ) Section 404 of the Clean Water Act (33 U.S.C. 1344).

( ) Section 103 of the Marine Protection, Research and Sanctuaries Act of 1972 (33 U.S.C. 1413).

2. Limits of this authorization.

a. This permit does not obviate the need to obtain other Federal, state, or local authorizations required by law.

b. This permit does not grant any property rights or exclusive privileges.

c. This permit does not authorize any injury to the property or rights of others.

d. This permit does not authorize interference with any existing or proposed Federal project.

3. Limits of Federal Liability. In issuing this permit, the Federal Government does not assume any liability for the following:

a. Damages to the permitted project or uses thereof as a result of other permitted or unpermitted activities or from natural causes.

b. Damages to the permitted project or uses thereof as a result of current or future activities undertaken by or on behalf of the United States in the public interest.

c. Damages to persons, property, or to other permitted or unpermitted activities or structures caused by the activity authorized by this permit.

d. Design or construction deficiencies associated with the permitted work.

e. Damage claims associated with any future modification, suspension, or revocation of this permit.

4. Reliance on Applicant's Data: The determination of this office that issuance of this permit is not contrary to the public interest was made in reliance on the information you provided.

5. Reevaluation of Permit Decision. This office may reevaluate its decision on this permit at any time the circumstances warrant. Circumstances that could require a reevaluation include, but are not limited to, the following:

a. You fail to comply with the terms and conditions of this permit.

b. The information provided by you in support of your permit application proves to have been false, incomplete, or inaccurate (See 4 above).

c. Significant new information surfaces which this office did not consider in reaching the original public interest decision.

Such a reevaluation may result in a determination that it is appropriate to use the suspension, modification, and revocation procedures contained in 33 CFR 325.7 or enforcement procedures such as those contained in 33 CFR 326.4 and 326.5. The referenced enforcement procedures provide for the issuance of an administrative order requiring you to comply with the terms and conditions of your permit and for the initiation of legal action where appropriate. You will be required to pay for any corrective measures ordered by this office, and if you fail to comply with such directive, this office may in certain situations (such as those specified in 33 CFR 209.170) accomplish the corrective measures by contract or otherwise and bill you for the cost.

6. Extensions. General condition 1 establishes a time limit for the completion of the activity authorized by this permit. Unless there are circumstances requiring either a prompt completion of the authorized activity or a reevaluation of the public interest decision, the Corps will normally give favorable consideration to a request for an extension of this time limit.

Your signature below, as permittee, indicates that you accept and agree to comply with the terms and conditions of this permit.

(Permittee)

(Date)

This permit becomes effective when the Federal official, designated to act for the Secretary of the Army, has signed below.

(District Engineer)

(Date)

When the structures or work authorized by this permit are still in existence at the time the property is transferred, the terms and conditions of this permit will continue to be binding on the new owner(s) of the property. To validate the transfer of this permit and the associated liabilities associated with compliance with its terms and conditions, have the transferee sign and date below.

(Transferee)

(Date)

B. Special Conditions. No special conditions will be preprinted on the permit form. The following and other special conditions should be added, as appropriate, in the space provided after the general conditions or on a referenced continuation sheet:

1. Your use of the permitted activity must not interfere with the public's right to free navigation on all navigable waters of the United States.

2. You must have a copy of this permit available on the vessel used for the authorized transportation and disposal of dredged material.

3. You must advise this office in writing, at least two weeks before you start maintenance dredging activities under the authority of this permit.

4. You must install and maintain, at your expense, any safety lights and signals prescribed by the United States Coast Guard (USCG), through regulations or otherwise, on your authorized facilities. The USCG may be reached at the following address and telephone number:

5. The condition below will be used when a Corps permit authorizes an artificial reef, an aerial transmission line, a submerged cable or pipeline, or a structure on the outer continental shelf.

National Ocean Service (NOS) has been notified of this authorization. You must notify NOS and this office in writing, at least two weeks before you begin work and upon completion of the activity authorized by this permit. Your notification of completion must include a drawing which certifies the location and configuration of the completed activity (a certified permit drawing may be used). Notifications to NOS will be sent to the following address: The Director, National Ocean Service (N/CG 222), Rockville, Maryland 20852.

6. The following condition should be used for every permit where legal recordation of the permit would be reasonably practicable and recordation could put a subsequent purchaser or owner of property on notice of permit conditions.

You must take the actions required to record this permit with the Registrar of Deeds or other appropriate official charged with the responsibility for maintaining records of title to or interest in real property.

Appendix B—[Reserved] (For Future NEPA Regulation)

Appendix C—[Reserved] (For Historic Properties Regulation)

PART 326—ENFORCEMENT

Sec.
326.1 Purpose.
326.2 Policy.
326.3 Unauthorized activities.
326.4 Supervision of authorized activities.
326.5 Legal action.

Authority: 33 U.S.C. 401 et seq.; 33 U.S.C. 1344; 33 U.S.C. 1413.

§ 326.1 Purpose.

This Part prescribes enforcement policies (§ 326.2) and procedures applicable to activities performed without required Department of the Army permits (§ 326.3) and to activities not in compliance with the terms and conditions of issued Department of the Army permits (§ 326.4). Procedures for initiating legal actions are prescribed in § 326.5. Nothing contained in this Part shall establish a non-discretionary duty on the part of district engineers nor shall deviation from these precedures give rise to a private right of action against a district engineer.

§ 326.2 Policy.

Enforcement, as part of the overall regulatory program of the Corps, is based on a policy of regulating the waters of the United States by discouraging activities that have not been properly authorized and by requiring corrective measures, where appropriate, to ensure those waters are not misused and to maintain the integrity of the program. There are several methods discussed in the remainder of this part which can be used either singly or in combination to implement this policy, while making the most effective use of the enforcement resources available. As EPA has independent enforcement authority under the Clean Water Act for unauthorized discharges, the district engineer should normally coordinate with EPA to determine the most effective and efficient manner by which resolution of a section 404 violation can be achieved.

§ 326.3 Unauthorized activities.

(a) *Surveillance.* To detect unauthorized activities requiring permits, district engineers should make the best use of all available resources.

purchased by any person from the Corps of Engineers or the reporter of such hearing. A copy will be available for public inspection at the office of the appropriate district engineer.

(f) All written statements, charts, tabulations, and similar data offered in evidence at the hearing shall, subject to exclusion by the presiding officer for reasons of redundancy, be received in evidence and shall constitute a part of the record.

(g) The presiding officer shall allow a period of not less than 10 days after the close of the public hearing for submission of written comments.

(h) In appropriate cases, the district engineer may participate in joint public hearings with other Federal or state agencies, provided the procedures of those hearings meet the requirements of this regulation. In those cases in which the other Federal or state agency allows a cross-examination in its public hearing, the district engineer may still participate in the joint public hearing but shall not require cross examination as a part of his participation.

§ 327.9 Filing of the transcript of the public hearing.

Where the presiding officer is the initial action authority, the transcript of the public hearing, together with all evidence introduced at the public hearing, shall be made a part of the administrative record of the permit action or Federal project. The initial action authority shall fully consider the matters discussed at the public hearing in arriving at his initial decision or recommendation and shall address, in his decision or recommendation, all substantial and valid issues presented at the hearing. Where a person other than the initial action authority serves as presiding officer, such person shall forward the transcript of the public hearing and all evidence received in connection therewith to the initial action authority together with a report summarizing the issues covered at the hearing. The report of the presiding officer and the transcript of the public hearing and evidence submitted thereat shall in such cases be fully considered by the initial action authority in making his decision or recommendation to higher authority as to such permit action or Federal project.

§ 327.10 Authority of the presiding officer.

Presiding officers shall have the following authority:

(a) To regulate the course of the hearing including the order of all sessions and the scheduling thereof, after any initial session, and the recessing, reconvening, and adjournment thereof; and

(b) To take any other action necessary or appropriate to the discharge of the duties vested in them, consistent with the statutory or other authority under which the Chief of Engineers functions, and with the policies and directives of the Chief of Engineers and the Secretary of the Army.

§ 327.11 Public notice.

(a) Public notice shall be given of any public hearing to be held pursuant to this regulation. Such notice should normally provide for a period of not less than 30 days following the date of public notice during which time interested parties may prepare themselves for the hearing. Notice shall also be given to all Federal agencies affected by the proposed action, and to state and local agencies and other parties having an interest in the subject matter of the hearing. Notice shall be sent to all persons requesting a hearing and shall be posted in appropriate government buildings and provided to newspapers of general circulation for publication. Comments received as form letters or petitions may be acknowledged as a group to the person or organization responsible for the form letter or petition.

(b) The notice shall contain time, place, and nature of hearing; the legal authority and jurisdiction under which the hearing is held; and location of and availability of the draft environmental impact statement or environmental assessment.

PART 328—DEFINITION OF WATERS OF THE UNITED STATES

Sec.
328.1 Purpose.
328.2 General scope.
328.3 Definitions.
328.4 Limits of jurisdiction.
328.5 Changes in limits of waters of the United States.

Authority: 33 U.S.C. 1344.

§ 328.1 Purpose.

This section defines the term "waters of the United States" as it applies to the jurisdictional limits of the authority of the Corps of Engineers under the Clean Water Act. It prescribes the policy, practice, and procedures to be used in determining the extent of jurisdiction of the Corps of Engineers concerning "waters of the United States." The terminology used by section 404 of the Clean Water Act includes "navigable waters" which is defined at section 502(7) of the Act as "waters of the United States including the territorial seas." To provide clarity and to avoid confusion with other Corps of Engineer regulatory programs, the term "waters of the United States" is used throughout 33 CFR Parts 320-330. This section does not apply to authorities under the Rivers and Harbors Act of 1899 except that some of the same waters may be regulated under both statutes (see 33 CFR Parts 322 and 329).

§ 328.2 General scope.

Waters of the United States include those waters listed in § 328.3(a). The lateral limits of jurisdiction in those waters may be divided into three categories. The categories include the territorial seas, tidal waters, and non-tidal waters (see 33 CFR 328.4 (a), (b), and (c), respectively).

§ 328.3 Definitions.

For the purpose of this regulation these terms are defined as follows:

(a) The term "waters of the United States" means

(1) All waters which are currently used, or were used in the past, or may be susceptible to use in interstate or foreign commerce, including all waters which are subject to the ebb and flow of the tide;

(2) All interstate waters including interstate wetlands;

(3) All other waters such as intrastate lakes, rivers, streams (including intermittent streams), mudflats, sandflats, wetlands, sloughs, prairie potholes, wet meadows, playa lakes, or natural ponds, the use, degradation or destruction of which could affect interstate or foreign commerce including any such waters:

(i) Which are or could be used by interstate or foreign travelers for recreational or other purposes; or

(ii) From which fish or shellfish are or could be taken and sold in interstate or foreign commerce; or

(iii) Which are used or could be used for industrial purpose by industries in interstate commerce;

(4) All impoundments of waters otherwise defined as waters of the United States under the definition;

(5) Tributaries of waters identified in paragraphs (a) (1)-(4) of this section;

(6) The territorial seas;

(7) Wetlands adjacent to waters (other than waters that are themselves wetlands) identified in paragraphs (a) (1)-(6) of this section.

Waste treatment systems, including treatment ponds or lagoons designed to meet the requirements of CWA (other than cooling ponds as defined in 40 CFR 123.11(m) which also meet the criteria of this definition) are not waters of the United States.

(b) The term "wetlands" means those areas that are inundated or saturated by surface or ground water at a frequency and duration sufficient to support, and that under normal circumstances do support, a prevalence of vegetation typically adapted for life in saturated soil conditions. Wetlands generally include swamps, marshes, bogs, and similar areas.

(c) The term "adjacent" means bordering, contiguous, or neighboring. Wetlands separated from other waters of the United States by man-made dikes or barriers, natural river berms, beach dunes and the like are "adjacent wetlands."

(d) The term "high tide line" means the line of intersection of the land with the water's surface at the maximum height reached by a rising tide. The high tide line may be determined, in the absence of actual data, by a line of oil or scum along shore objects, a more or less continuous deposit of fine shell or debris on the foreshore or berm, other physical markings or characteristics, vegetation lines, tidal gages, or other suitable means that delineate the general height reached by a rising tide. The line encompasses spring high tides and other high tides that occur with periodic frequency but does not include storm surges in which there is a departure from the normal or predicted reach of the tide due to the piling up of water against a coast by strong winds such as those accompanying a hurricane or other intense storm.

(e) The term "ordinary high water mark" means that line on the shore established by the fluctuations of water and indicated by physical characteristics such as clear, natural line impressed on the bank, shelving, changes in the character of soil, destruction of terrestrial vegetation, the presence of litter and debris, or other appropriate means that consider the characteristics of the surrounding areas.

(f) The term "tidal waters" means those waters that rise and fall in a predictable and measurable rhythm or cycle due to the gravitational pulls of the moon and sun. Tidal waters end where the rise and fall of the water surface can no longer be practically measured in a predictable rhythm due to masking by hydrologic, wind, or other effects.

§ 328.4 Limits of jurisdiction.

(a) *Territorial Seas.* The limit of jurisdiction in the territorial seas is measured from the baseline in a seaward direction a distance of three nautical miles. (See 33 CFR 329.12)

(b) *Tidal Waters of the United States.* The landward limits of jurisdiction in tidal waters:

(1) Extends to the high tide line, or

(2) When adjacent non-tidal waters of the United States are present, the jurisdiction extends to the limits identified in paragraph (c) of this section.

(c) *Non-Tidal Waters of the United States.* The limits of jurisdiction in non-tidal waters:

(1) In the absence of adjacent wetlands, the jurisdiction extends to the ordinary high water mark, or

(2) When adjacent wetlands are present, the jurisdiction extends beyond the ordinary high water mark to the limit of the adjacent wetlands.

(3) When the water of the United States consists only of wetlands the jurisdiction extends to the limit of the wetland.

§ 328.5 Changes in limits of waters of the United States.

Permanent changes of the shoreline configuration result in similar alterations of the boundaries of waters of the United States. Gradual changes which are due to natural causes and are perceptible only over some period of time constitute changes in the bed of a waterway which also change the boundaries of the waters of the United States. For example, changing sea levels or subsidence of land may cause some areas to become waters of the United States while siltation or a change in drainage may remove an area from waters of the United States. Man-made changes may affect the limits of waters of the United States; however, permanent changes should not be presumed until the particular circumstances have been examined and verified by the district engineer. Verification of changes to the lateral limits of jurisdiction may be obtained from the district engineer.

PART 329—DEFINITION OF NAVIGABLE WATERS OF THE UNITED STATES

Sec.

329.1 Purpose.
329.2 Applicability.
329.3 General policies.
329.4 General definitions.
329.5 General scope of determination.
329.6 Interstate or foreign commerce.
329.7 Intrastate or interstate nature of waterway.
329.8 Improved or natural conditions of the waterbody.
329.9 Time at which commerce exists or determination is made.
329.10 Existence of obstructions.
329.11 Geographic and jurisdictional limits of rivers and lakes.
329.12 Geographic and jurisdictional limits of oceanic and tidal waters.
329.13 Geographic limits: shifting boundaries.
329.14 Determination of navigability.
329.15 Inquiries regarding determinations.
329.16 Use and maintenance of lists of determinations.

Authority: 33 U.S.C. 401 *et seq.*

§ 329.1 Purpose.

This regulation defines the term "navigable waters of the United States" as it is used to define authorities of the Corps of Engineers. It also prescribes the policy, practice and procedure to be used in determining the extent of the jurisdiction of the Corps of Engineers and in answering inquiries concerning "navigable waters of the United States." This definition does not apply to authorities under the Clean Water Act which definitions are described under 33 CFR Parts 323 and 328.

§ 329.2 Applicability.

This regulation is applicable to all Corps of Engineers districts and divisions having civil works responsibilities.

§ 329.3 General policies.

Precise definitions of "navigable waters of the United States" or "navigability" are ultimately dependent on judicial interpretation and cannot be made conclusively by administrative agencies. However, the policies and criteria contained in this regulation are in close conformance with the tests used by Federal courts and determinations made under this regulation are considered binding in regard to the activities of the Corps of Engineers.

§ 329.4 General definition.

Navigable waters of the United States are those waters that are subject to the ebb and flow of the tide and/or are presently used, or have been used in the past, or may be susceptible for use to transport interstate or foreign commerce. A determination of navigability, once made, applies laterally over the entire surface of the waterbody, and is not extinguished by later actions or events which impede or destroy navigable capacity.

§ 329.5 General scope of determination.

The several factors which must be examined when making a determination whether a waterbody is a navigable water of the United States are discussed in detail below. Generally, the following conditions must be satisfied:

(vii) Description of improvements to navigation not listed in paragraph (c)(5) of this section:

(4) Nature and location of significant obstructions to navigation in portions of the waterbody used or potentially capable of use in interstate commerce:

(5) Authorized projects:

(i) Nature, condition and location of any improvements made under projects authorized by Congress:

(ii) Description of projects authorized but not constructed:

(iii) List of known survey documents or reports describing the waterbody:

(6) Past or present interstate commerce:

(i) General types, extent, and period in time:

(ii) Documentation if necessary:

(7) Potential use for interstate commerce, if applicable:

(i) If in natural condition:

(ii) If improved:

(8) Nature of jurisdiction known to have been exercised by Federal agencies if any:

(9) State or Federal court decisions relating to navigability of the waterbody, if any:

(10) Remarks:

(11) Finding of navigability (with date) and recommendation for determination:

§ 329.15 Inquiries regarding determinations.

(a) Findings and determinations should be made whenever a question arises regarding the navigability of a waterbody. Where no determination has been made, a report of findings will be prepared and forwarded to the division engineer, as described above. Inquiries may be answered by an interim reply which indicates that a final agency determination must be made by the division engineer. If a need develops for an energency determination, district engineers may act in reliance on a finding prepared as in Section 329.14 of this Part. The report of findings should then be forwarded to the division engineer on an expedited basis.

(b) Where determinations have been made by the division engineer, inquiries regarding the *navigability* of specific portions of waterbodies covered by these determinations may be answered as follows:

This Department, in the administration of the laws enacted by Congress for the protection and preservation of the navigable waters of the United States, has determined that _____ (River) (Bay) (Lake, etc.) is a navigable water of the United States from _____ to _____. Actions which modify or otherwise affect those waters are subject to the jurisdiction of this Department, whether such actions occur within or outside the navigable areas.

(c) Specific inquiries regarding the *jurisdiction* of the Corps of Engineers can be answered only after a determination whether (1) the waters are navigable waters of the United States or (2) if not navigable, whether the proposed type of activity may nevertheless so affect the navigable waters of the United States that the assertion of regulatory jurisdiction is deemed necessary.

§ 329.16 Use and maintenance of lists of determinations.

(a) Tabulated lists of final determinations of navigability are to be maintained in each district office, and be updated as necessitated by court decisions, jurisdictional inquiries, or other changed conditions.

(b) It should be noted that the lists represent only those waterbodies for which determinations have been made; absence from that list should not be taken as an indication that the waterbody is not navigable.

(c) Deletions from the list are not authorized. If a change in status of a waterbody from navigable to non-navigable is deemed necessary, an updated finding should be forwarded to the division engineer; changes are not considered final until a determination has been made by the division engineer.

PART 330—NATIONWIDE PERMITS

Sec.

330.1 General.
330.2 Definitions.
330.3 Activities occuring before certain dates.
330.4 Public notice.
330.5 Nationwide permits.
330.6 Management practices.
330.7 Notification procedures.
330.8 Discretionary Authority.
330.9 State water quality certification.
330.10 Coastal Zone Management consistency determination.
330.11 Nationwide permit verification.
330.12 Expiration of nationwide permits.

Authority: 33 U.S.C. 401 et seq.; 33 U.S.C. 1344; 33 U.S.C. 1413.

§ 330.1 General.

The purpose of this regulation is to describe the Department of the Army's (DA) nationwide permit program and to list all current nationwide permits which have been issued by publication herein. A nationwide permit is a form of general permit which may authorize activities throughout the nation. (Another type of general permit is a "regional permit" and is issued by division or district engineers on a regional basis in accordance with 33 CFR Part 325). Copies of regional conditions and modifications, if any, to the nationwide permits can be obtained from the appropriate district engineer. Nationwide permits are designed to allow certain activities to occur with little, if any, delay or paperwork. Nationwide permits are valid only if the conditions applicable to the nationwide permits are met. Failure to comply with a condition does not necessarily mean the activity cannot be authorized but rather that the activity can only be authorized by an individual or regional permit. Several of the nationwide permits require notification to the district engineer prior to commencement of the authorized activity. The procedures for this notification are located at § 330.7 of this Part. Nationwide permits can be issued to satisfy the requirements of section 10 of the Rivers and Harbors Act of 1899, section 404 of the Clean Water Act, and/or section 103 of the Marine Protection, Research and Sanctuaries Act. The applicable authority is indicated at the end of each nationwide permit.

§ 330.2 Definitions.

(a) The definitions of 33 CFR Parts 321–329 are applicable to the terms used in this Part.

(b) The term "headwaters" means the point on a non-tidal stream above which the average annual flow is less than five cubic feet per second. The district engineer may estimate this point from available data by using the mean annual area precipitation, area drainage basin maps, and the average runoff coefficient, or by similar means. For streams that are dry for long periods of the year, district engineers may establish the "headwaters" as that point on the stream where a flow of five cubic feet per second is equaled or exceeded 50 percent of the time. And all wetlands

(c) Discretionary authority means the authority delegated to division engineers in § 330.8 of this part to override provisions of nationwide permits, to add regional conditions, or to require individual permit application.

§ 330.3 Activities occurring before certain dates.

The following activities were permitted by nationwide permits issued on July 19, 1977, and unless modified do not require further permitting:

(a) Discharges of dredged or fill material into waters of the United States outside the limits of navigable waters of the United States that occurred before the phase-in dates which began July 25, 1975, and extended section 404 jurisdiction to all waters of the United

States. (These phase-in dates are: After July 25, 1975, discharges into navigable waters of the United States and adjacent wetlands; after September 1, 1976, discharges into navigable waters of the United States and their primary tributaries, including adjacent wetlands, and into natural lakes, greater than 5 acres in surface area; and after July 1, 1977, discharges into all waters of the United States.) (Section 404)

(b) Structures or work completed before December 18, 1968, or in waterbodies over which the district engineer had not asserted jurisdiction at the time the activity occurred provided, in both instances, there is no interference with navigation. (Section 10)

§ 330.4 Public notice.

(a) *Chief of Engineers.* Upon proposed issuance of new nationwide permits, modification to, or reissuance of, existing nationwide permits, the Chief of Engineers will publish a notice in the **Federal Register** seeking public comments and including the opportunity for a public hearing. This notice will state the availability of information at the Office of the Chief of Engineers and at all district offices which reveals the Corps' provisional determination that the proposed activities comply with the requirements for issuance under general permit authority. The Chief of Engineers will prepare this information which will be supplemented, if appropriate, by division engineers.

(b) *District engineers.* Concurrent with publication in the **Federal Register** of proposed, new, or reissued nationwide permits by the Chief of Engineers, district engineers will so notify the known interested public by an appropriate notice. The notice will include regional conditions, if any, developed by the division engineer.

§ 330.5 Nationwide permits.

(a) *Authorized activities.* The following activities are hereby permitted provided they meet the conditions listed in paragraph (b) of this section and, where required, comply with the notification procedures, of § 330.7.

(1) The placement of aids to navigation and regulatory markers which are approved by and installed in accordance with the requirements of the U.S. Coast Guard (33 CFR Part 66, Subchapter C). (Section 10)

(2) Structures constructed in artificial canals within principally residential developments where the connection of the canal to a navigable water of the United States has been previously authorized (see 33 CFR Part 322.5(g)). (Section 10)

(3) The repair, rehabilitation, or replacement of any previously authorized, currently serviceable, structure or fill, or of any currently serviceable structure or fill constructed prior to the requirement for authorization, provided such repair, rehabilitation, or replacement does not result in a deviation from the plans of the original structure or fill, and further provided that the structure or fill has not been put to uses differing from uses specified for it in any permit authorizing its original construction. Minor deviations due to changes in materials or construction techniques and which are necessary to make repair, rehabilitation, or replacement are permitted. Maintenance dredging and beach restoration are not authorized by this nationwide permit. (Section 10 and 404)

(4) Fish and wildlife harvesting devices and activities such as pound nets, crab traps, eel pots, lobster traps, duck blinds, and clam and oyster digging. (Section 10)

(5) Staff gages, tide gages, water recording devices, water quality testing and improvement devices, and similar scientific structures. (Section 10)

(6) Survey activities including core sampling, seismic exploratory operations, and plugging of seismic shot holes and other exploratory-type bore holes. Drilling of exploration-type bore holes for oil and gas exploration is not authorized by this nationwide permit; the plugging of such holes is authorized. (Sections 10 and 404).

(7) Outfall structures and associated intake structures where the effluent from that outfall has been permitted under the National Pollutant Discharge Elimination System program (Section 402 of the Clean Water Act) (see 40 CFR Part 122) provided that the district or division engineer makes a determination that the individual and cumulative adverse environmental effects of the structure itself are minimal in accordance with § 330.7 (c)(2) and (d). Intake structures per se are not included—only those directly associated with an outfall structure are covered by this nationwide permit. This permit includes minor excavation, filling and other work associated with installation of the intake and outfall structures. (Sections 10 and 404)

(8) Structures for the exploration, production, and transportation of oil, gas, and minerals on the outer continental shelf within areas leased for such purposes by the Department of Interior, Mineral Management Service, provided those structures are not placed within the limits of any designated shipping safety fairway or traffic separation scheme (where such limits have not been designated or where changes are anticipated, district engineers will consider recommending the discretionary authority provided by 330.8 of this Part, and further subject to the provisions of the fairway regulations in 33 CFR 322.5(1) (Section 10).

(9) Structures placed within anchorage or fleeting areas to facilitate moorage of vessels where such areas have been established for that purpose by the U.S. Coast Guard. (Section 10)

(10) Non-commercial, single-boat, mooring buoys. (Section 10)

(11) Temporary buoys and markers placed for recreational use such as water skiing and boat racing provided that the buoy or marker is removed within 30 days after its use has been discontinued. At Corps of Engineers reservoirs, the reservoir manager must approve each buoy or marker individually. (Section 10)

(12) Discharge of material for backfill or bedding for utility lines, including outfall and intake structures, provided there is no change in preconstruction bottom contours (excess material must be removed to an upland disposal area). A "utility line" is defined as any pipe or pipeline for the transportation of any gaseous, liquid, liquifiable, or slurry substance, for any purpose, and any cable, line, or wire for the transmission for any purpose of electrical energy, telephone and telegraph messages, and radio and television communication. (The utility line and outfall and intake structures will require a Section 10 permit if in navigable waters of the United States. See 33 CFR Part 322. See also paragraph (a)(7) of this section). (Section 404)

(13) Bank stabilization activities provided:

(i) The bank stabilization activity is less than 500 feet in length;

(ii) The activity is necessary for erosion prevention;

(iii) The activity is limited to less than an average of one cubic yard per running foot placed along the bank within waters of the United States;

(iv) No material is placed in excess of the minimum needed for erosion protection;

(v) No material is placed in any wetland area;

(vi) No material is placed in any location or in any manner so as to impair surface water flow into or out of any wetland area;

(vii) Only clean material free of waste metal products, organic materials, unsightly debris, etc. is used; and

(viii) The activity is a single and complete project. (Sections 10 and 404)

(14) Minor road crossing fills including all attendant features, both temporary and permanent, that are part of a single and complete project for crossing of a non-tidal waterbody, provided that the crossing is culverted, bridged or otherwise designed to prevent the restriction of, and to withstand, expected high flows and provided further that discharges into any wetlands adjacent to the waterbody do not extend beyond 100 feet on either side of the ordinary high water mark of that waterbody. A "minor road crossing fill" is defined as a crossing that involves the discharge of less than 200 cubic yards of fill material below the plane of ordinary high water. The crossing may require a permit from the US Coast Guard if located in navigable waters of the United States. Some road fills may be eligible for an exemption from the need for a Section 404 permit altogether (see 33 CFR 323.4). District engineers are authorized, where local circumstances indicate the need, to define the term "expected high flows" for the purpose of establishing applicability of this nationwide permit. (Sections 10 and 404)

(15) Discharges of dredged or fill material incidental to the construction of bridges across navigable waters of the United States, including cofferdams, abutments, foundation seals, piers, and temporary construction and access fills provided such discharge has been authorized by the US Coast Guard as part of the bridge permit. Causeways and approach fills are not included in this nationwide permit and will require an individual or regional Section 404 permit. (Section 404)

(16) Return water from an upland, contained dredged material disposal area (see 33 CFR 323.2(d)) provided the state has issued a site specific or generic certification under section 401 of the Clean Water Act (see also 33 CFR 325.2(b)(1)). The dredging itself requires a Section 10 permit if located in navigable waters of the United States. The return water or runoff from a contained disposal area is administratively defined as a discharge of dredged material by 33 CFR 323.2(d) even though the disposal itself occurs on the upland and thus does not require a section 404 permit. This nationwide permit satisfies the technical requirement for a section 404 permit for the return water where the quality of the return water is controlled by the state through the section 401 certification procedures. (Section 404)

(17) Fills associated with small hydropower projects at existing reservoirs where the project which includes the fill is licensed by the Federal Energy Regulatory Commission (FERC) under the Federal Power Act of 1920, as amended; has a total generating capacity of not more than 1500 kw (2,000 horsepower); qualifies for the short-form licensing procedures of the FERC (see 18 CFR 4.61); and the district or division engineer makes a determination that the individual and cumulative adverse effects on the environment are minimal in accordance with § 330.7 (c)(2) and (d). (Section 404)

(18) Discharges of dredged or fill material into all waters of the United States other than wetlands that do not exceed ten cubic yards as part of a single and complete project provided the material is not placed for the purpose of stream diversion. (Sections 10 and 404)

(19) Dredging of no more than ten cubic yards from navigable waters of the United States as part of a single and complete project. This permit does not authorize the connection of canals or other artificial waterways to navigable waters of the United States (see Section 33 CFR 322.5(g)). (Section 10)

(20) Structures, work, and discharges for the containment and cleanup of oil and hazardous substances which are subject to the National Oil and Hazardous Substances Pollution Contingency Plan, (40 CFR Part 300), provided the Regional Response Team which is activated under the Plan concurs with the proposed containment and cleanup action. (Sections 10 and 404)

(21) Structures, work, discharges associated with surface coal mining activities provided they were authorized by the Department of the Interior, Office of Surface Mining, or by states with approved programs under Title V of the Surface Mining Control and Reclamation Act of 1977; the appropriate district engineer is given the opportunity to review the Title V permit application and all relevant Office of Surface Mining or state (as the case may be) documentation prior to any decision on that application; and the district or division engineer makes a determination that the individual and cumulative adverse effects on the environment from such structures, work, or discharges are minimal in accordance with §§ 330.7 (c) (2) and (3) and (d). (Sections 10 and 404)

(22) Minor work, fills, or temporary structures required for the removal of wrecked, abandoned, or disabled vessels, or the removal of man-made obstructions to navigation. This permit does not authorize maintenance dredging, shoal removal, or river bank snagging. (Sections 10 and 404)

(23) Activities, work, and discharges undertaken, assisted, authorized, regulated, funded, or financed, in whole or in part, by another federal agency or department where that agency or department has determined, pursuant to the CEQ Regulation for Implementing the Procedural Provisions of the National Environmental Policy Act (40 CFR Part 1500 et seq.), that the activity, work, or discharge is categorically excluded from environmental documentation because it is included within a category of actions which neither individually nor cumulatively have a significant effect on the human environment, and the Office of the Chief of Engineers (ATTN: DAEN-CWO-N) has been furnished notice of the agency's or department's application for the categorical exclusion and concurs with that determination. Prior to approval for purposes of this nationwide permit of any agency's categorical exlcusions, the Chief of Engineers will solicit comments through publication in the Federal Register. (Sections 10 and 404)

(24) Any activity permitted by a state administering its own Section 404 permit program for the discharge of dredged or fill material authorized at 33 U.S.C. 1344(g)-(l) is permitted pursuant to section 10 of the Rivers and Harbors Act of 1899. Those activities which do not involve a section 404 state permit are not included in this nationwide permit but many will be exempted by section 154 of Pub. L. 94-587. (See 33 CFR 322.3(a)(2)). (Section 10)

(25) Discharge of concrete into tightly sealed forms or cells where the concrete is used as a structural member which would not otherwise be subject to Clean Water Act jurisdiction. (Section 404)

(26) Discharges of dredged or fill material into the waters listed in paragraphs (a)(26) (i) and (ii) of this section except those which cause the loss or substantial adverse modification of 10 acres or more of such waters of the United States, including wetlands. For discharges which cause the loss or substantial adverse modification of 1 to 10 acres of such waters, including wetlands, notification to the district engineer is required in accordance with section 330.7 of this section. (Section 404).

(i) Non-tidal rivers, streams, and their lakes and impoundments, including adjacent wetlands, that are located above the headwaters.

(ii) Other non-tidal waters of the United States, including adjacent wetlands, that are not part of a surface tributary system to interstate waters or

navigable waters of the United States (i.e., isolated waters).

(b) *Conditions.* The following special conditions must be followed in order for the nationwide permits identified in paragraph (a) of this section to be valid:

(1) That any discharge of dredged or fill material will not occur in the proximity of a public water supply intake.

(2) That any discharge of dredged or fill material will not occur in areas of concentrated shellfish production unless the discharge is directly related to a shellfish harvesting activity authorized by paragraph (a)(4) of this section.

(3) That the activity will not jeopardize a threatened or endangered species as identified under the Endangered Species Act (ESA), or destroy or adversely modify the critical habitat of such species. In the case of federal agencies, it is the agencies' responsibility to comply with the requirements of the ESA. If the activity may adversely affect any listed species or critical habitat, the district engineer must initiate Section 7 consultation in accordance with the ESA. In such cases, the district engineer may:

(i) Initiate section 7 consultation and then, upon completion, authorize the activity under the nationwide permit by adding, if appropriate, activity specific conditions, or

(ii) Prior to or concurrent with section 7 consultation he may recommend discretionary authority (See section 330.8) or use modification, suspension, or revocation procedures (See 33 CFR 325.7).

(4) That the activity shall not significantly disrupt the movement of those species of aquatic life indigenous to the waterbody (unless the primary purpose of the fill is to impound water);

(5) That any discharge of dredged or fill material shall consist of suitable material free from toxic pollutants (see section 307 of the Clean Water Act) in toxic amounts;

(6) That any structure or fill authorized shall be properly maintained.

(7) That the activity will not occur in a component of the National Wild and Scenic River System; nor in a river officially designated by Congress as a "study river" for possible inclusion in the system, while the river is in an official study status;

(8) That the activity shall not cause an unacceptable interference with navigation;

(9) That, if the activity may adversely affect historic properties which the National Park Service has listed on, or determined eligible for listing on, the National Register of Historic Places, the permittee will notify the district engineer. If the district engineer determines that such historic properties may be adversely affected, he will provide the Advisory Council on Historic Preservation an opportunity to comment on the effects on such historic properties or he will consider modification, suspension, or revocation in accordance with 33 CFR 325.7. Furthermore, that, if the permittee before or during prosecution of the work authorized, encounters a historic property that has not been listed or determined eligible for listing on the National Register, but which may be eligible for listing in the National Register, he shall immediately notify the district engineer;

(10) That the construction or operation of the activity will not impair reserved tribal rights, including, but not limited to, reserved water rights and treaty fishing and hunting rights;

(11) That in certain states, an individual state water quality certification must be obtained or waived (See § 330.9);

(12) That in certain states, an individual state coastal zone management consistency concurrence must be obtained or waived (See § 330.10);

(13) That the activity will comply with regional conditions which may have been added by the division engineer (See § 330.8(a)); and

(14) That the management practices listed in § 330.6 of this part shall be followed to the maximum extent practicable.

(c) *Further information.* (1) District engineers are authorized to determine if an activity complies with the terms and conditions of a nationwide permit unless that decision must be made by the division engineer in accordance with § 330.7.

(2) Nationwide permits do not obviate the need to obtain other Federal, state or local authorizations required by law.

(3) Nationwide permits do not grant any property rights or exclusive privileges.

(4) Nationwide permits do not authorize any injury to the property or rights of others.

(5) Nationwide permits do not authorize interference with any existing or proposed Federal project.

(d) *Modification, Suspension or Revocation of Nationwide Permits.* The Chief of Engineers may modify, suspend, or revoke nationwide permits in accordance with the relevant procedures of 33 CFR 325.7. Such authority includes, but is not limited to: adding individual, regional, or nationwide conditions; revoking authorization for a category of activities or a category of waters by requiring individual or regional permits; or revoking an authorization on a case-by-case basis. This authority is not limited to concerns for the aquatic environment as is the discretionary authority in § 330.8.

§ 330.6 Management practices.

(a) In addition to the conditions specified in § 330.5 of this Part, the following management practices shall be followed, to the maximum extent practicable, in order to minimize the adverse effects of these discharges on the aquatic environment. Failure to comply with these practices may be cause for the district engineer to recommend, or the division engineer to take, discretionary authority to regulate the activity on an individual or regional basis pursuant to § 330.8 of this Part.

(1) Discharges of dredged or fill material into waters of the United States shall be avoided or minimized through the use of other practical alternatives.

(2) Discharges in spawning areas during spawning seasons shall be avoided.

(3) Discharges shall not restrict or impede the movement of aquatic species indigenous to the waters or the passage of normal or expected high flows or cause the relocation of the water (unless the primary purpose of the fill is to impound waters).

(4) If the discharge creates an impoundment of water, adverse impacts on the aquatic system caused by the accelerated passage of water and/or the restriction of its flow shall be minimized.

(5) Discharge in wetlands areas shall be avoided.

(6) Heavy equipment working in wetlands shall be placed on mats.

(7) Discharges into breeding areas for migratory waterfowl shall be avoided.

(8) All temporary fills shall be removed in their entirety.

§ 330.7 Notification procedures.

(a) The general permittee shall not begin discharges requiring pre-discharge notification pursuant to the nationwide permit at § 330.5(a)(26):

(1) Until notified by the district engineer that the work may proceed under the nationwide permit with any special conditions imposed by the district or division engineer; or

(2) If notified by the district or division engineer that an individual permit may be required; or

(3) Unless 20 days have passed from receipt of the notification by the district engineer and no notice has been

received from the district or division engineer.

(b) Notification pursuant to the nationwide permit at § 330.5(a)(26) must be in writing and include the information listed below. Notification is not an admission that the proposed work would result in more than minimal impacts to waters of the United States; it simply allows the district or division engineer to evaluate specific activities for compliance with general permit criteria.

(1) Name, address, and phone number of the general permittee;

(2) Location of the planned work;

(3) Brief description of the proposed work, its purpose, and the approximate size of the waters, including wetlands, which would be lost or substantially adversely modified as a result of the work; and

(4) Any specific information required by the nationwide permit and any other information that the permittee believes is appropriate.

(c) *District engineer review of notification.* Upon receipt of notification, the district engineer will promptly review the general permittee's notification to determine which of the following procedures should be followed:

(1) If the nationwide permit at § 330.5(a)(26) is involved and the district engineer determines either, (i) the proposed activity falls within a class of discharges or will occur in a category of waters which has been previously identified by the Regional Administrator, Environmental Protection Agency; the Regional Director, Fish and Wildlife Service; the Regional Director, National Marine Fisheries Service; or the heads of the appropriate state natural resource agencies as being of particular interest to those agencies; or (ii) the particular discharge has not been previously identified but he believes it may be of importance to those agencies, he will promptly forward the notification to the division engineer and the head and appropriate staff officials of those agencies to afford those agencies an adequate opportunity before such discharge occurs to consider such notification and express their views, if any, to the district engineer concerning whether individual permits should be required.

(2) If the nationwide permits at § 330.5(a) (7), (17), or (21) are involved and the Environmental Protection Agency, the Fish and Wildlife Service, the National Marine Fisheries Service or the appropriate state natural resource or water quality agencies forward concerns to the district engineer, he will forward those concerns to the division engineer together with a statement of the factors pertinent to a determination of the environmental effects of the proposed discharges, including those set forth in the 404(b)(1) guidelines, and his views on the specific points raised by those agencies.

(3) If the nationwide permit at § 330.5(a)(21) is involved the district engineer will give notice to the Environmental Protection Agency and the appropriate state water quality agency. This notice will include as a minimum the information required by paragraph (b) of this section.

(d) *Division engineer review of notification.* The division engineer will review all notifications referred to him in accordance with paragraph (c)(1) or (c)(2) of this section. The division engineer will require an individual permit when he determines that an activity does not comply with the terms or conditions of a nationwide permit or does not meet the definition of a general permit (see 33 CFR 322.2(f) and 323.2(n)) including discharges under the nationwide permit at § 330.5(a)(26) which have more than minimal adverse environmental effects on the aquatic environment when viewed either cumulatively or separately. In reaching his decision, he will review factors pertinent to a determination of the environmental effects of the proposed discharge, including those set forth in the 404(b)(1) guidelines, and will give full consideration to the views, if any, of the federal and state natural resource agencies identified in paragraph (c) of this section. If the division engineer decides that an individual permit is not required, and a federal or appropriate state natural resource agency has indicated in writing that an activity may result in more than minimal adverse environmental impacts, he will prepare a written statement, available to the public on request, which sets forth his response to the specific points raised by the commenting agency. When the division engineer reaches his decision he will notify the district engineer, who will immediately notify the general permittee of the division engineer's decision.

§ 330.8 Discretionary authority.

Except as provided in paragraphs (c)(2) and (d) of this section, division engineers on their own initiative or upon recommendation of a district engineer are authorized to modify nationwide permits by adding regional conditions or to override nationwide permits by requiring individual permit applications on a case-by-case basis, for a category of activities, or in specific geographic areas. Discretionary authority will be based on concerns for the aquatic environment as expressed in the guidelines published by EPA pursuant to section 404(b)(1). (40 CFR Part 230)

(a) *Activity Specific conditions.* Division engineers are authorized to modify nationwide permits by adding individual conditions on a case-by-case basis applicable to certain activities within their division. Activity specific conditions may be added by the District Engineer in instances where there is mutual agreement between the district engineer and the permittee. Furthermore, district engineers will condition NWPs with conditions which have been imposed on a state section 401 water quality certification issued pursuant to § 330.9 of this Part.

(b) *Regional conditions.* Division engineers are authorized to modify nationwide permits by adding conditions on a generic basis applicable to certain activities or specific geographic areas within their divisions. In developing regional conditions, division and district engineers will follow standard permit processing procedures as prescribed in 33 CFR Part 325 applying the evaluation criteria of 33 CFR Part 320 and appropriate parts of 33 CFR Parts 321, 322, 323, and 324. Division and district engineers will take appropriate measures to inform the public of the additional conditions.

(c) *Individual permits*—(1) *Case-by-Case.* In nationwide permit cases where additional individual or regional conditioning may not be sufficient to address concerns for the aquatic environment or where there is not sufficient time to develop such conditions under paragraphs (a) or (b) of this section, the division engineer may suspend use of the nationwide permit and require an individual permit application on a case-by-case basis. The district engineer will evaluate the application and will either issue or deny a permit. However, if at any time the reason for taking discretionary authority is satisfied, then the division engineer may remove the suspension, reactivating authority under the nationwide permit. Where time is of the essence, the district engineer may telephonically recommend that the division engineer assert discretionary authority to require an individual permit application for a specific activity. If the division engineer concurs, he may orally authorize the district engineer to implement that authority. Oral authorization should be followed by written confirmation.

(2) *Category.* Additionally, after notice and opportunity for public hearing, division engineers may decide that individual permit applications

should be required for categories of activities, or in specific geographic areas. However, only the Chief of Engineers may modify, suspend, or revoke nationwide permits on a statewide or nationwide basis. The division engineer will announce the decision to persons affected by the action. The district engineer will then regulate the activity or activities by processing an application(s) for an individual permit(s) pursuant to 33 CFR Part 325.

(d) For the nationwide permit found at § 330.5(a)(26), after the applicable provisions of § 330.7(a) (1) and (3) have been satisfied, the permittee's right to proceed under the general permit may be modified, suspended, or revoked only in accordance with the procedure set forth in 33 CFR 325.7.

(e) A copy of all modifications or revocations of activities covered by nationwide permits will be forwarded to the Office of the Chief of Engineers, ATTN: DAEN-CWO-N.

§ 330.9 State water quality certification.

(a) State water quality certification is required for nationwide permits which may result in any discharge into waters of the United States. If a state issues a water quality certification which includes special conditions, the district engineer will add these conditions as conditions of the nationwide permit in that state. However, if such conditions do not comply with the provisions of 33 CFR 325.4 or if a state denies a required 401 certification for a particular nationwide permit, authorization for all discharges covered by the nationwide permit within the state is denied without prejudice until the state issues an individual or generic water quality certification or waives its right to do so. A district engineer will not process an individual permit application for an activity for which authorization has been denied without prejudice under the nationwide permit program. However, if the division engineer determines that it would otherwise be appropriate to exercise his discretionary authority, pursuant to § 330.8, to override the nationwide permit or permits in question, he may do so, and the district engineer may proceed with the processing of individual permit applications. In instances where a state has denied the 401 water quality certification for discharges under a particular nationwide permit, applicants must furnish the district engineer with an individual or generic 401 certification or a copy of the application to the state for the certification. If a state fails to act within a reasonable period of time (see § 325.2(b)(1)(ii)), a waiver will be presumed. Upon receipt of an individual or generic certification or a waiver of certification, the proposed work is authorized under the nationwide permit. If a state issues a conditioned individual certification, the district engineer will include those conditions that comply with 33 CFR 325.4 as special conditions of the nationwide permit (see 33 CFR Part 330.8(a)) and notify the applicant that the work is authorized under the nationwide permit provided all conditions are met.

(b) Certification requirements for nationwide permits fall into the following general categories:

(1) *No certification required.* Nationwide permits numbered 1, 2, 4, 5, 8, 9, 10, 11, and 19 do not involve activities which may result in a discharge and therefore 401 certification is not applicable.

(2) *Certification sometimes required.* Nationwide permits numbered 3, 6, 7, 13, 20, 21, 22, and 23 each involve various activities, some of which may result in a discharge and require certification, and others of which do not. State denial of certification for any specific nationwide permit in this category affects only those activities involving discharges. Those not involving discharges remain in effect.

(3) *Certification required.* Nationwide permits numbered 12, 14, 15, 16, 17, 18, 24, 25, and 26 involve activities which would result in discharges and therefore 401 certification is required.

(c) District engineers will take appropriate measures to inform the public of which waterbodies or regions within the state, and for which nationwide permits, an individual 401 water quality certification is required.

§ 330.10 Coastal zone management consistency determination.

In instances where a state has not concurred that a particular nationwide permit is consistent with an approved coastal zone management plan, authorization for all activities subject to such nationwide permit within or affecting the state coastal zone agency's area of authority is denied without prejudice until the applicant has furnished to the district engineer a coastal zone management consistency determination pursuant to section 307 of the Coastal Zone Management Act and the state has concurred in it. If a state does not act on an applicant's consistency statement within six months after receipt by the state, consistency shall be presumed. District engineers will take appropriate measures to inform the public of which waterbodies or regions within the state, and for which nationwide permits, such individual consistency determination is required. District engineers will not process any permit application for an activity which has been denied without prejudice under the nationwide permit program. However, if the division engineer determines that it would otherwise be appropriate to exercise his discretionary authority, pursuant to § 330.8, to override the nationwide permit or permits in question, he may do so, and the district engineer may proceed with the processing of individual permit applications.

§ 330.11 Nationwide permit verification.

(a) General permittees may, and in some cases must, request from a district engineer confirmation that an activity complies with the terms and conditions of a nationwide permit. District engineers will respond promptly to such requests. The response will state that the verification is valid for a period of no more than two years or a lesser period of time if deemed appropriate. Section 330.12 takes precedence over this section, therefore, it is incumbent upon the permittee to remain informed of changes to nationwide permits.

(b) If the district engineer decides that an activity does not comply with the terms or conditions of a nationwide permit, he will so notify the person desiring to do the work and indicate that an individual permit is required (unless covered by a regional permit).

(c) If the district engineer decides that an activity does comply with the terms and conditions of a nationwide permit he will so notify the general permittee. In such cases, as with any activity which qualifies under a nationwide permit, the general permittee's right to proceed with the activities under the nationwide permit may be modified, suspended, or revoked only in accordance with the procedures of 33 CFR 325.7.

§ 330.12 Expiration of nationwide permits.

The Chief of Engineers will review nationwide permits on a continual basis, and will decide to either modify, reissue (extend) or revoke the permits at least every five years. If a nationwide permit is not modified or reissued within five years of publication in the **Federal Register**, it automatically expires and becomes null and void. Authorization of activities which have commenced or are under contract to commence in reliance upon a nationwide permit will remain in effect provided the activity is completed within twelve months of the date a nationwide permit has expired or was revoked unless discretionary permit authority has been exercised in

accordance with § 330.8 of this Part or modification, suspension, or revocation procedures are initiated in accordance with the relevent provisions of 33 CFR 325.7. Activities completed under the authorization of a nationwide permit which was in effect at the time the activity was completed continue to be authorized by that nationwide permit.

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EMERGENCY WATERSHED PROTECTION
(EWP)

Construct under 2 Authorities:

SECTION 216, FLOOD CONTROL ACT OF MAY 17, 1950
(PUBLIC LAW 81-516)

SECTION 403, AGRICULTURAL CREDIT ACT OF 1978
(PUBLIC LAW 95-334)

ARIZONA INTERIM GUIDELINES FOR SECTION 216-403 WORK

Arizona guidelines are supplementary to the rules and regulations for the Emergency Watershed Protection Program dated November 17, 1981 and February 1, 1983 (Attachment 1).

A. Types of Emergencies

1. Exigency - A compelling situation of unusual urgency. This type of emergency exists when the real-term probability of damage to life or high value property is high enough to demand immediate action. Prompt remedial action is to be taken when imminent threat to loss of life exists. The Corps of Engineers shall be contacted by telephone prior to construction on streams requiring 404 permits. Construction may proceed after the State Conservationist has obtained funds, all permits and land rights (construction permits) have been obtained and project agreement is signed. Drawings and written specifications may not be required depending on the urgency of the situation. Consultation with other SCS engineers shall be obtained if needed. Generally, local equipment is rented to do the work. The work is directed by the SCS personnel made responsible for the job. SCS provides 100% of the construction cost. Funds are to be obligated within ten (10) days and construction is to be completed in an additional thirty (30) days. Extensions of time (additional ten (10) and thirty (30) days respectively) are possible if approved by the Chief.

2. Nonexigency - An emergency requiring action because of potential for future hazard to life and high value property, but of less urgency than the compelling situation. This situation exists when the near-term probability of damage to life or high value property is high enough to constitute an emergency but not sufficiently high to be considered an exigency. Funds are to be obligated within 220 days. SCS provides 80% of the construction cost and local sponsors are required to provide the remaining 20% in cash, in-kind services, or a combination of the two as agreed to in the project agreement. Extensions of time are possible if approved by the Chief.

Engineer's Workshop 2/7/90
John Weaver

SUMMARY
1983-84 EWP WORK After Oct 1983 flood
SCS Arizona

Project Exigency	Type	Total $
1. Blue River	Dike & Rock Riprap	$31,629
2. Church of Christ	Rock Riprap	12,254
3. Nelson	Rock Riprap	10,145
4. Cross "F" Trailer Park/Kirkland	Rock Riprap	75,000
5. Maricopa	Rock Riprap	303,290
6. Calabasas School	Rock Riprap	70,671
7. Aravapai (O'Neal)	Rock Riprap	246,563
8. Rillito at 1st Avenue	Soil Cement	931,386
9. Pantano Wash S. Speedway	Soil Cement	742,412
10. Pantano Wash N. Tanque Verde	Soil cement	248,633
11. Houck	Gabions	22,350
12. Santa Cruz River S. Ajo Way	Soil Cement	1,065,420
13. Rillito River at County Club	Soil Cement	582,012
14. Pantano Wash Nr. Golf Links	Soil Cement	391,234
15. Cottonwood Wash	Gabions	23,630

has worked well

Non Exigency		
1. Greene's Reservoir	Dikes	502,901
2. San Simon	Channel Clearance	90,847
3. Tubac School*	Kellner Jacks	76,752
4. Patagonia	Rock Riprap	29,250
5. Marana	Kellner Jacks	141,150
6. Upper Aravapai	Dikes	26,850
7. Skull Valley*	Channel & Dike	13,145
8. Lower Aravapai (Stambaugh)	Dike	27,400
9. Ruth Martin	~~Dike~~ soil cement dike	22,185
10. Red Rock	Dike	88,126
11. South Greene's Wash	Channel and Dikes	88,050
12. Dead Horse Ranch*	Bank Sloping	15,340
13. Graveyard Wash	Channel and Dike	6,390
14. Tubac North*	Kellner Jacks	142,857
15. Gila River Reach 1	Dikes	221,860
16. Gila River Reach 2	Dikes	149,752
17. Gila River Reach 3	Dikes	163,298
18. Gila River Reach 4	Dikes	101,676
19. Gila River Reach 5	Dikes	183,227
20. Gila River Reach 6	Dikes	143,600
21. Cottonwood Wash	Channel Clearance	34,935
22. Firestone Gradens* (Nogales)	Kellner Jacks	26,617
23. Repairs - Marana, Tubac Sites	Kellner Jacks	30,000 Est.
24. Safford Jackstraws	Kellner Jacks	10,000

* vegetation planting

25. Safford 8th Avenue	Kellner Jacks	183,358
26. Graham Canal Jetties*	Railroad Deflecting	225,000
27. Greenlee Co. Dikes	Dikes	42,565
28. Graham Co. Dikes	Dikes	36,032
29. Aha Quin (1986)	Dike and Rock Riprap	
30. Nogales Wash (1986)*	Kellner Jacks	38,950
	Approximate Total	$7,606,130

*These projects and also some of the Gila River dike projects in Graham County had willow and other vegetative plantings included at part of the bank practices. Costs shown do not include costs for vegetative plantings

Engineer's Workshop 2/7/90
John Weaver

SUMMARY
1979-1980 EWP PROJECTS After large flood of 1979
SCS Arizona

1.	Aravapai Creek-Klondike	Rock Riprap	$985,028.49	
2.	Aravapi Creek Eqp. Rental	Dike Repair	157,481.75	
3.	Beaver Dam Wash	Kellner Dikes	98,824.00	
4.	John Beck - Camp Verde	Rock Riprap	386,369.90	
5.	Black Canyon City	Rock Riprap	820,850.00	
6.	Blue River	Gabions and Dike	42,328.69	
7.	Brown - Camp Verde	Kellner Jacks	?	
	Repaired 1982	Kellner Jacks	12,497.00	
8.	Carmen/Tubac School (has failed)	Rock Riprap	323,572.00	
9.	Date Creek	Dike and Kellner Jacks	22,708.00	
10.	Greene's Wash	Channel and Dikes	206,989.88	
11.	Jensen-Oak Creek	Channel and Tree Revement	4,000.00	
	Repair (Diversion Removal)		3,000.00	Est.
12.	Lake Montezuma	Channel Clearance	16,884.21	
13.	Littlefield	Kellner Jacks	47,100.00	
	Repaired 1982	Jack and Rock Riprap	14,000.00	Est.
14.	Marana (jacks spun - swapped)	Kellner Jacks have been replaced	182,025.53	
15.	New River School	Rock Riprap	36,900.00	Est.
16.	Puerco River	Kellner Jacks	27,496.00	
17.	Queen Creek Reach 1	Channel and Dikes	1,294,696.30	
	Queen Creek Reach 2	Channel and Dikes	726,906.00	
18.	Rillito Creek	Rock Riprap & Soil Cement	346,332.17	
19.	Roosevelt Gardens/Houston Creek	Gabions	954,309.39	
	Repaired 1982	Gabions	71,750.92	
20.	Safford/Duncan washed in 1983 flood	Dikes	1,132,270.29	
			363,344.00	
21.	Sycamore/Perkins	Kellner Jacks	19,760.00	Est.
22.	Sycamore Canyon	Rock Riprap	288,357.83	
23.	Mrs. Word	Rock Riprap & Kellner	100,679.20	
		Approximate Total	$8,374,454	

Soil requirement: Sandy or Sandy Loam
Less than 3000 ppm salts
- must water regime (dry)
- dormant season planting
- protected, cows, etc

STREAMBANK PROTECTION PLAN

I. General Information

A. Plant materials will be secured from sites off the designated work areas.

Plants will be Coyote willows, Salix exigua.

II. Items of work to be performed

A. Plant materials - Coyote willows - dormant cuttings

(1) shall be a minimum of one half inch diameter.
(2) shall be 6-8 feet in lenght.
(3) all side branches will be removed.
(4) shall be cut and planted while completely dormant.
(5) dormant cuttings will be transported with bottom ends kept wet.
(6) dormant cuttings will be stored with bottom ends in water until planted. Cuttings should be planted within twenty-one days.

III> General Instructions

A. All dormant cuttings will be dipped in a solution of Rootone F (Indole-3 butyric acid). Mix according to label instructions. Dip cuttings for five minutes just prior to planting. Mix solution in large barrel to facillitate dipping.

B. Take steps to insure that the cuttings are not planted upside down.

C. It is critical that:

(a) the cuttings never dry out
(b) dry season water table is always reached
(c) plantings are protected from livestock, at least two growing seasons.

D. All cuttings will be planted after Nov 15 and before Apr 1.

ENGINEERING WORKSHOP
CHANNEL STABILIZATION AND BANK PROTECTION
February 5-7, 1990

Rock Riprap

Rock riprap is natural rock that is loose (dumped or hand-placed) or grouted in place on a streambank to prevent erosion and minimize scour. Riprap can be used in areas where there is acceptable quality stone and the costs for quarrying, transporting, and placing the stone are economical.

I. Design of Loose Rock Riprap

A. Criteria - Design Considerations

1. Rock Size and Gradation - Angular interlocking rock reasonably well graded is the preferred rock. The shape and size of rock that will be stable in the streamflow can be determined by various methods. Corps of Engineers state that well gradation using D_{50} is most stable. Placing poorly graded riprap induces localized weak spots.

2. Layer Thickness of Riprap - Thickness of the stone blanket should be at least equal to the maximum size stone at the top, 12" - 18" minimum thickness, and three times the size of stone at the base.

D_{100}

3. Filter Layer - A filter layer is generally used unless the bedding material meets the filter requirements. Filter fabric or a 6" minimum filter layer of selected aggregates is used.
4. Toe Protection - A firmly established revetment toe is important to prevent undermining or scouring of the foundation, which is the major cause of failure in rock riprap projects. The toe should be extended below the scour depth. Where excavation for a deep toe is not practical, alternate methods can be used such as anchoring or driving sheet piling (see EFM, Undermining of Revetments, page 16-6). The minimum toe width is 4 ft and trench depth is 3 ft.
5. Bank
   - Need to know the composition of the bedding material to determine if a filter layer is required.
   - Side Slope - Banks should be sloped so that the pressure of the stone is mainly against the bank rather than against the stone in the lower courses and toe. EFM states that side slope should not be steeper than 1 $^1/_2$: 1. Arizona Standard Drawing calls for a minimum of 2:1.
   - Average Bank Height - The upper vertical limit of protection should extend above the high water with a 1 ft minimum freeboard allowance.

6. Streambank Length and Alignment - Upstream and downstream ends should extend into the bank with cutoff end sections beyond the limits of bank erosion. The effects of channel alignment shoud be considered. Straight alignments experience normal side scour. Curved alignments are subject to high velocity currents acting against the bank.
7. Hydrology and Hydraulics Parameters
   - Watershed Drainage Area
   - Channel Bed Composition and Channel Gradient
   - Bankfull Capacity of Stream
   - Velocity at Bankfull
8. Hazard from Debris and/or Ice - May need to increase rock size (1.5 - 2 times) in areas where hazard of minor impact might occur.

B. Methods

1. Engineering Field Manual, Chapter 16, "Streambank Protection"
   Relates the maximum required rock size, D_{100}, to the velocity of the stream based on Isbash curves for stone size and weight in relation to design velocity within 10 ft of the bank. A gradation of riprap table gives the recommended gradation for weight of stone used.

2. Engineering Design Standards - Far West States most used by FWS
   Professor E.W. Lane developed a tractive stress theory that relates bank shear to bed shear and produces the following equation for rock size (D_s) where 75% of the material, by weight, is smaller (D_{75}):

$$D_{75} = \frac{3.5}{CK} wDS \qquad \text{(in.)}$$

C is the relative limiting tractive force and is related to the ratio of curve radius to water surface width. K is the related to the side slope. These relationships are plotted on an easy-to-use nomagraph. The equation was developed for material with specific gravity of 2.56 and must be adjusted for other material.

w = specific weight of water

D = depth of flow

S = channel slope

3. Advisory PO-18, 5/1/74
   Adapted the tractive stress method from the National Cooperative Highway Research Program Report No. 108 (HRP 108) and developed the equation:

$$D_{50} = \left[\frac{1180(S_e)^{13/6}\,\bar{R}}{P}\right]^{0.4} K \qquad \text{(ft)}$$

   where Q is the design discharge (cfs), R is the hydraulic radius at design discharge (ft), P is the wetted perimeter at design discharge (ft), S_e is the channel energy gradient (ft/ft), K is the correction factor for specific gravity, alignment and side slope.

4. National Crushed Stone Association
   NCSA uses an average velocity/rock size relationship, D_{50}, made into tabular form.

No factor of safety?

5. U.S. Army Corps of Engineers, EM 1110-2-1601, ETL 1110-2-120
   This method generally follows the tractive stress approach by considering local boundary shear or average boundary shear design for determining the average size riprap, D_{50}, as follows:

$$\text{(ft)} \qquad D_{50} = \frac{T_c}{C(\gamma_s - \gamma_w)} \qquad \text{and} \qquad \bar{T}_c = \gamma_w RS \quad \text{(average shear)}$$

   where T_c is the critical shear stress, C is a coefficient that varies with roughness (0.04 is the best estimate), γ_s is the unit weight of stone, γ_w is the unit weight of water. Corrections are made for non-uniformity of flow, bend effects and slope. The local boundary shear is an iterative solution.

6. Corps of Engineers Stone Stability Charts
   Relates the rock size, D_{50}, to the velocity of the stream and was developed by the Isbash curves, design chart (ft).

7. TR-59
   Adapts the tractive stress-HRP 108 for riprap gradient control structures.

C. Example Problem - Summary of Results (see attachment for problem solutions)

Design Method	Problem #1 Rock Size	Problem #2 Rock Size
1. EFM	D_{100} = 28 in.	D_{100} = 9 in.
2. FWS most other used	D_{75} = 16 in.	D_{75} = 6 in.
3. Advisory PO-18	D_{50} = 18 in.	D_{50} = 6 in.
4. NCSA	D_{50} = 15 in.	D_{50} = 5 in.
5. CE	D_{50} = 14 in.	D_{50} = 5 in.
6. CE Stone Stability Charts	D_{50} = 15 in.	D_{50} = 5 in.

II. Specifications

A. Construction Specifications
   61 Loose Rock Riprap
   203 Geotextiles
B. Material Specifications
   521 Aggregates for Drainfill and Filters
   523 Rock For Riprap
   303 Geotextiles, Woven and Nonwoven

III. Cost

The price of riprap depends on the availability of suitable rock and the treatment location. For loose rock riprap the average price is $24 per cubic yard.
which include purchase price, hauling, and placement

IV. References

Field Office Technical Guide, Section IV
Practice Standard and Spec 580, "Streambank and Shoreline Protection (ft)"

Engineering Design Standards - Far West States Section VI - Channel System, Hydraulic Deisgn - Channels, "Criteria for Riprap Rock Size", 6-13a

Engineering Field Manual
Chapter 16, "Streambank Protection"
Chapter 17, "Construction Materials"

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

National Engineering Handbook
Chapter 5, "Hydraulics"
Chapter 20, "Specifications for Construction Contracts"

"Rock Riprap Criteria - Streambank and Slope Protection" West States Eng. Mtg. April 25-29, 1983

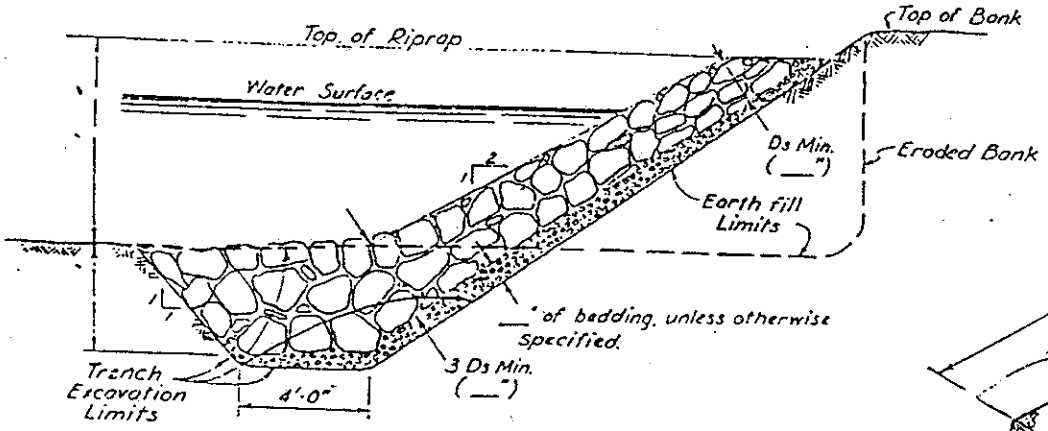
"Rock Riprap Design Procedure" TCS Advisory ENG-PO-18, May 1, 1974

Soil Mechanics Note No. 1 (SMN-1), Guide for Determining the Gradation of Sand and Gravel Filters, January 1986

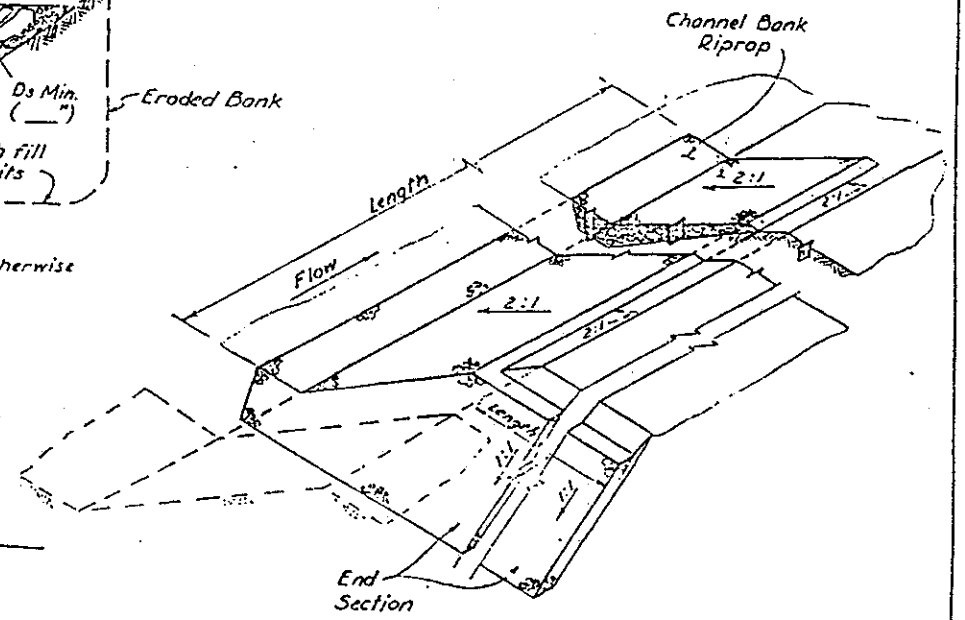
Technical Release No. 59, "Hydraulic Design of Riprap Gradient Control Structures", January 23, 1976, Amend. 1, March 1986

U.S. Army Corps of Engineers. "Streambank Protection Course" Vicksburg, MI, Lecture G-2-b-2, Researcher: Stephen T. Maynard, Waterways Experiment Station

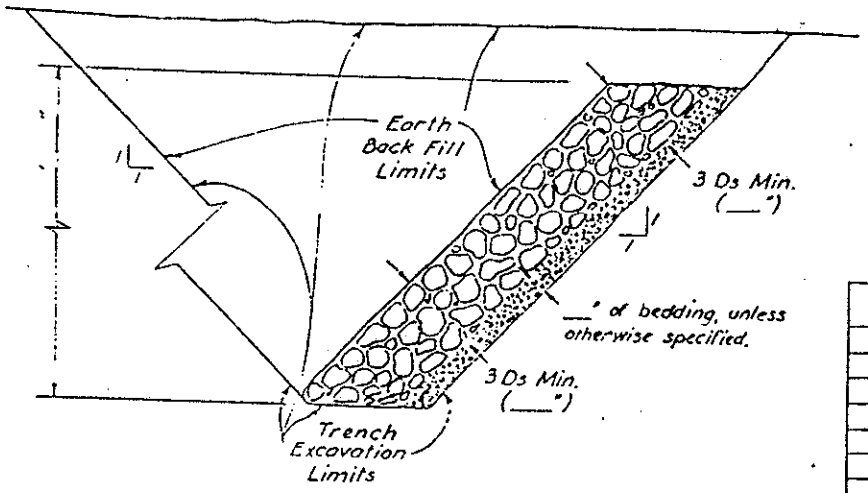
U.S. Department of Transportation. Federal Highway Administration. Bureau of Public Roads. Use of Riprap for Bank Protection Hydraulic Engineering Circular No. 11, June 1967



TYPICAL RIPRAP SECTION
(Not to Scale)



ISOMETRIC VIEW
(Not to Scale)



TYPICAL UPSTREAM END SECTION
(Not to Scale)

Site	Length	Ds	End Section Length

DUMPED ROCK RIPRAP

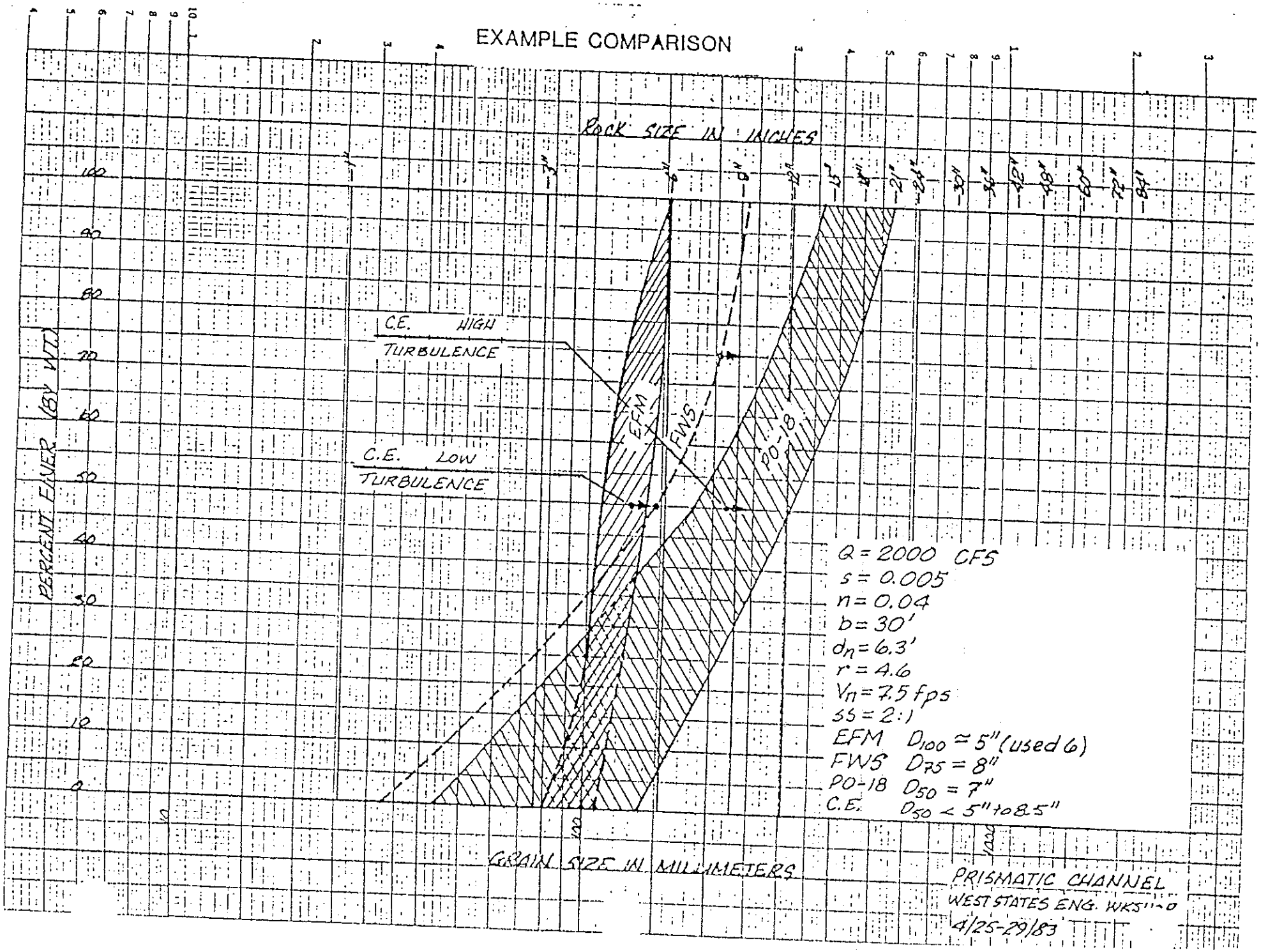
U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

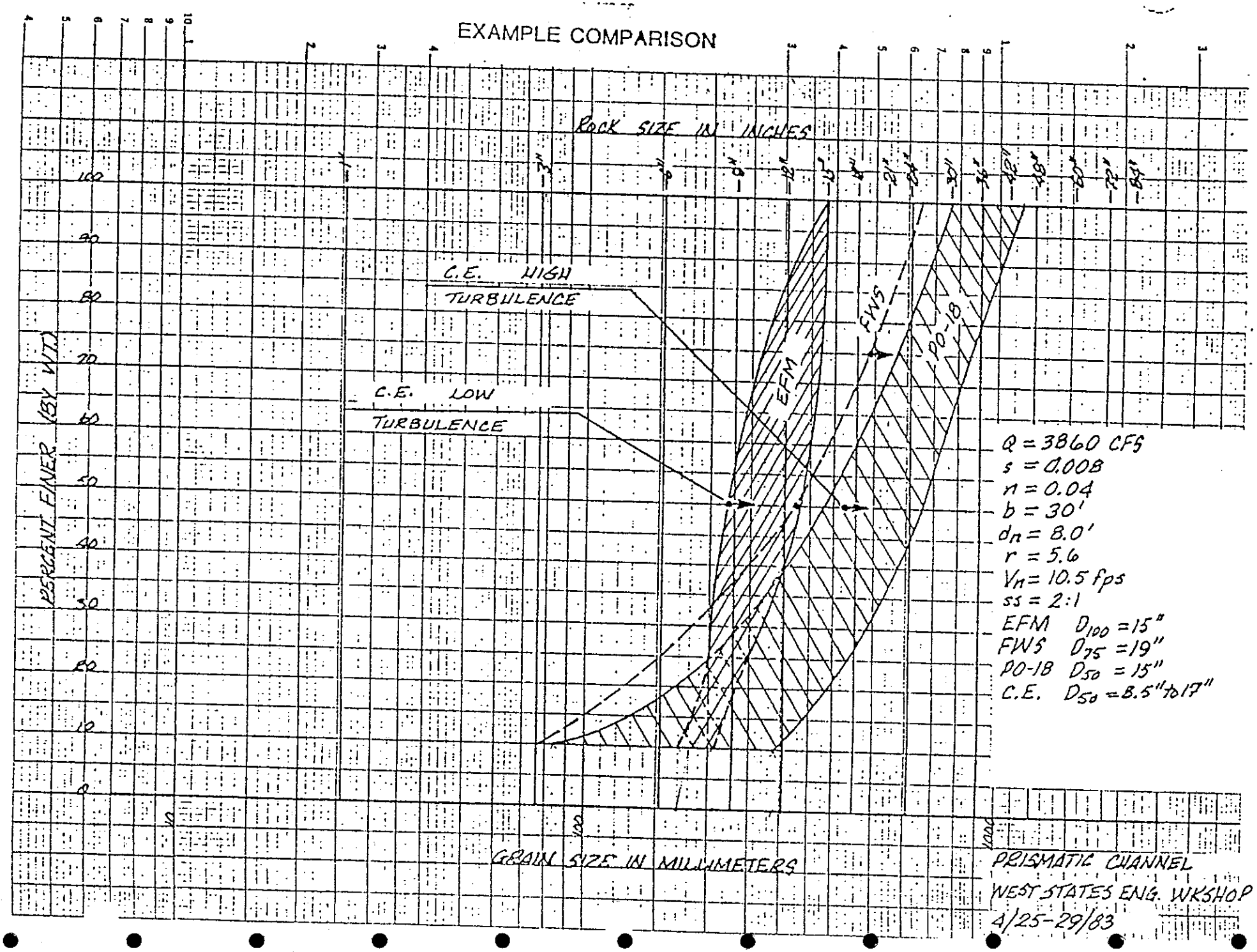
Designed WKA 4-79
Drawn SFS 4-79
Traced
Checked

ROCK RIPRAP CRITERIA

STREAMBANK AND SLOPE PROTECTION

REFERENCE	ROCK SIZE	GRADATION	FILTER
1. ENG. FIELD MAN.	ISBASH CURVE: VELOCITY VERSUS D_{100} SIZE EXHIBIT 16-1	MAXIMUM, MINIMUM, AND D_{75} SIZE FROM EXHIBIT 16-2	SUITABLE MATERIAL, CLEAN SAND-GRAVEL MIN. 6" THICK
2. FAR WEST STATES	TRACTIVE STRESS-LANE $D_{75} = \frac{3.5}{CK} WDS_e$	WELL GRADED THICKNESS: D_{75} TOP OF SLOPE, $3\ D_{75}$ BASE OF SLOPE	SM-NOTE 1 $\frac{D_{15\ RR}}{D_{85\ BASE}} \leq 5$
3. ADVISORY PO-18 5/1/74	TRACTIVE STRESS-HRP108 $D_{50} = \left[\frac{118Q(Se)^{13/6} R}{P}\right]^{0.4} K$	GRADATION LIMITS CURVE DEVELOPED FROM HRP-108	NO SPECIFIC INFO.
4. TR-59 Rip-Rap Gradient Control Structures	TRACTIVE STRESS-HRP-108 PROCEDURE DETAILED IN TR	INDEX $= \left[\frac{D_{85}}{D_{50}} + \frac{D_{50}}{D_{15}}\right] \leq 5.5$	$\frac{D_{15\ RR}}{D_{85\ BASE}} < 5 < \frac{D_{15\ RR}}{D_{15\ BASE}} < 40$ $\frac{D_{50\ RR}}{D_{50\ BASE}} < 40$
5. ROCK RIPRAP FOR SLOPE PROTECTION DRAFT TR Debris on embankment slope use safety factor	WAVE HEIGHT AND EMB. SLOPE, DESIGN CHARTS D_{50} FROM FIG. 8	FIGURE 10 UNIFORM ROCK GRADED ROCK	SM NOTE 1 MINIMUM THICKNESS 9 INCHES
6. STREAMBANK PROTECTION, WA. LETTER 7/20/82	STD. GRADATION FOR $V \leq 10$ fps FWS OR EFM FOR $V > 10$ fps	GRADATION LIMITS RATIO TO D_{75} SIZE	$\frac{D_{15\ RR}}{D_{85\ BASE}} \leq 8$
7. C.E. STONE STABILITY CHARTS	DEVELOPED FROM ISBACH VELOCITY VERSUS D_{50}, LOW AND HIGH TURBULENCE	N.A.	N.A.


EXAMPLE COMPARISON
ROCK SIZE IN INCHES
3"
6"
9"
12"
15"
18"
21"
24"
30"
36"
42"
48"
60"
72"
84"
PERCENT FINER (BY WT.)
100
90
80
70
60
50
40
30
20
10
0
C.E. HIGH TURBULENCE
C.E. LOW TURBULENCE
EFM
FWS
PO-18
Q = 2000 CFS
s = 0.005
n = 0.04
b = 30'
d_n = 6.3'
r = 4.6
V_n = 7.5 fps
SS = 2:1
EFM D_{100} ≃ 5" (used 6)
FWS D_{75} = 8"
PO-18 D_{50} = 7"
C.E. D_{50} < 5" to 8.5"
GRAIN SIZE IN MILLIMETERS
10
100
1000
PRISMATIC CHANNEL
WEST STATES ENG. WKS
4/25-29/83


EXAMPLE COMPARISON
ROCK SIZE IN INCHES
C.E. HIGH TURBULENCE
C.E. LOW TURBULENCE
EFM
FWS
PO-18
PERCENT FINER (BY WT.)
GRAIN SIZE IN MILLIMETERS
Q = 3860 CFS
s = 0.008
n = 0.04
b = 30'
dn = 8.0'
r = 5.6
Vn = 10.5 fps
ss = 2:1
EFM D100 = 15"
FWS D75 = 19"
PO-18 D50 = 15"
C.E. D50 = 8.5" to 17"
PRISMATIC CHANNEL
WEST STATES ENG. WKSHOP
4/25-29/83

B. Methods

1. SCS, EFM, Chapter 16, "Streambank Protection"

Notes:

1. The velocity is the current speed within 10 ft. of the bank.

2. If debris impact is a major factor, double the stone weight.

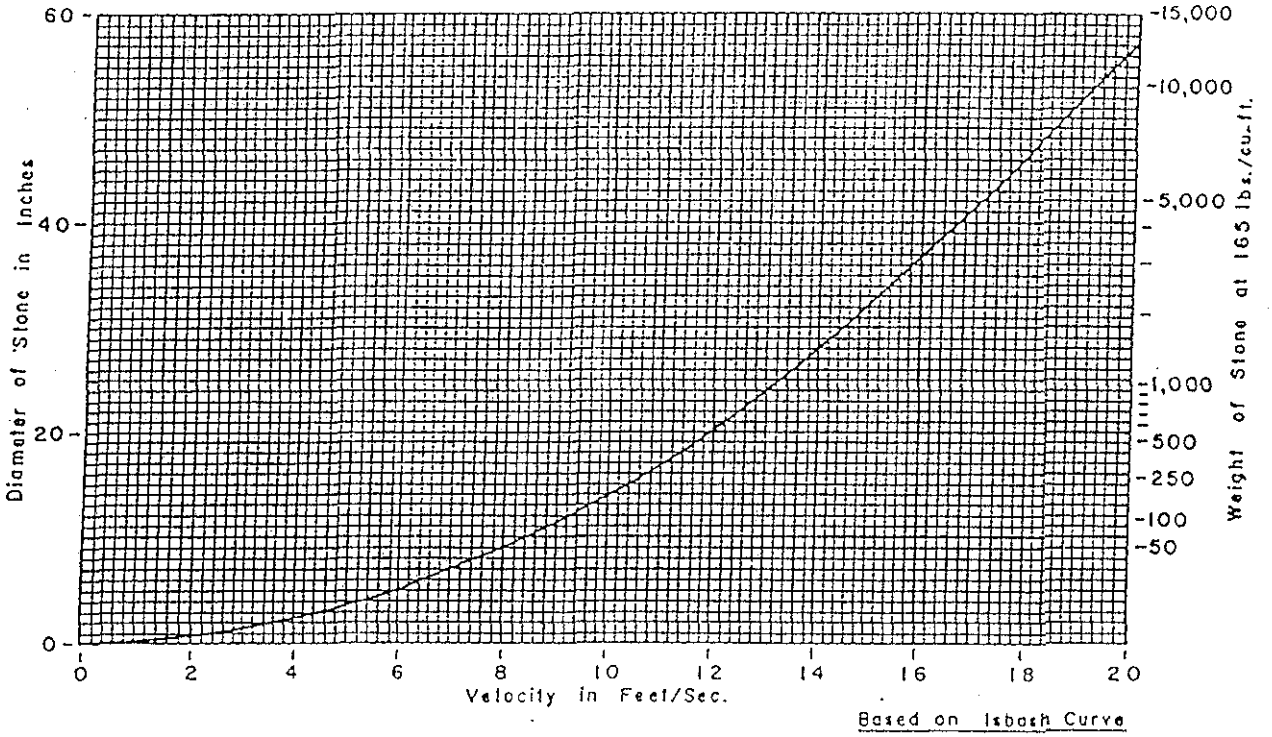


Exhibit 16-1 Maximum stone size for riprap

Maximum weight of stone required	Minimum and maximum range in weight of stones	Weight range of 75 percent of stones
(lbs.)	(lbs.)	(lbs.)
150	25 - 150	50 - 150
200	25 - 200	50 - 200
250	25 - 250	50 - 250
400	25 - 400	100 - 400
600	25 - 600	150 - 600
800	25 - 800	200 - 800
1000	50 -1000	250 -1000
1300	50 -1300	325 -1300
1600	50 -1600	400 -1600
2000	75 -2000	600 -2000
2700	100 -2700	800 -2700

Exhibit 16-2. Gradation of riprap

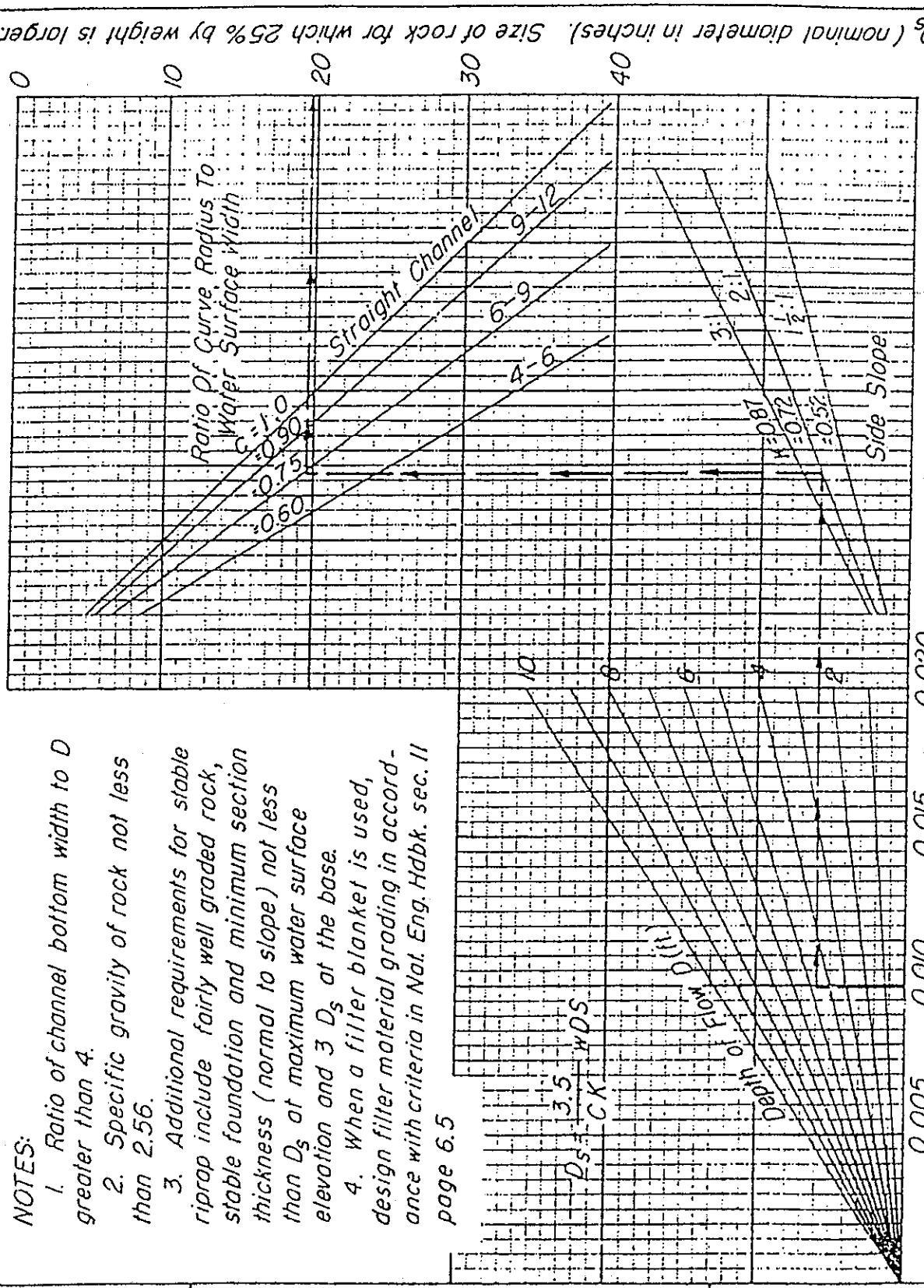
II. Design of [illegible] Riprap

B. Methods

2. Far West States - "Criteria for Riprap Rock Size"

FIG. 6-13d

ROCK SIZE FOR STABLE RIPRAP



Curve radius/water surface width

	C
4 - 6	0.60
6 - 9	0.75
9 - 12	0.90
straight channel	1.00

Values of K for various side slopes are:

Slope	K
1-1/2 - 1	.50
1-3/4 - 1	.63
2 - 1	.72
2-1/2 - 1	.80
3 - 1	.87

$$D_s = \frac{3.5\,WDS}{CK}$$

NOTES:

1. Ratio of channel bottom width to D greater than 4.
2. Specific gravity of rock not less than 2.56.
3. Additional requirements for stable riprap include fairly well graded rock, stable foundation and minimum section thickness (normal to slope) not less than D_s at maximum water surface elevation and 3 D_s at the base.
4. When a filter blanket is used, design filter material grading in accordance with criteria in Nat. Eng. Hdbk. sec. 11 page 6.5

REFERENCE	U. S. DEPARTMENT of AGRICULTURE	STANDARD DWG NO.
"Design of Stable Channels" Transactions A.S.C.E. 1955	SOIL CONSERVATION SERVICE	7-L-16043
	ENGINEERING and WATERSHED PLANNING UNIT	SHEET 1 OF 1
	PORTLAND, OREGON	DATE: 7-56

II. Design of Loose Rock Riprap

B. Methods

4. National Crushed Stone Association

SUGGESTED SIZES

Depending on factors involved in the final decision and crushed stone products generally available, the following is offered.

Table 1

Flow Velocity (ft./sec.)	GRADED RIPRAP STONE Size Inches (sq. openings) NCSA No.	Max.	Ave.	Min.	Filter Stone NCSA No.
2.5	R-1	1½	¾	No. 8	FS-1
4.5	R-2	3	1½	1	FS-1
6.5	R-3	6	3	2	FS-1
9.0	R-4	12	6	3	FS-2
11.5	R-5	18	9	5	FS-2
13.0	R-6	24	12	7	FS-3
14.5	R-7	30	15	12	FS-3

Notes:
- See Appendix for more complete information on riprap sizes and filter stone requirements.
- Flow velocities measured at mid-stream or 10 ft. from eroding bank, whichever is closest to bank.
- Stone sizes could be reduced about 30% for side slopes of 6 to 1.

INCL 2

II. Design of Loose Rock Riprap

B. Methods

3. Advisory PO-18, 5/1/74

Excerpt from HRB #108

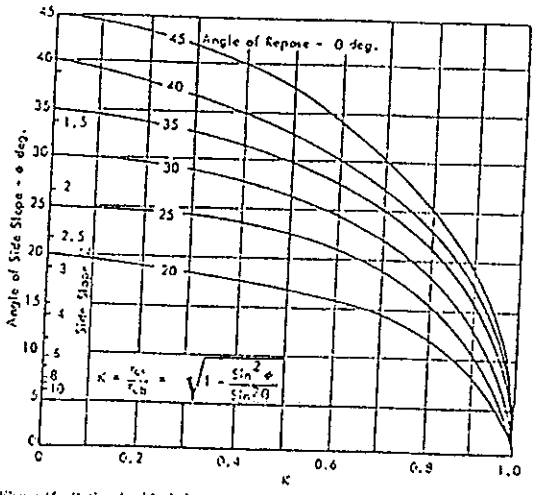


Figure 15. Ratio of critical shear on sides to critical shear on bottom for noncohesive sediment.

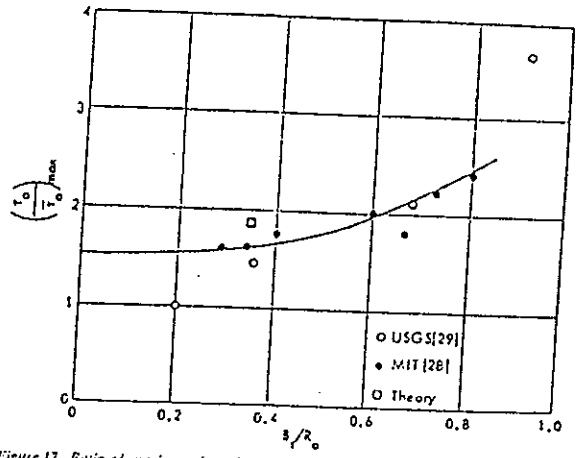


Figure 17. Ratio of maximum boundary shear stress to the mean shear stress for flow in bends.

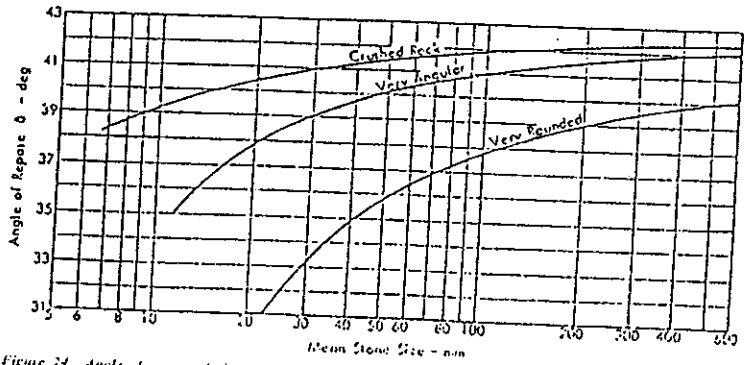


Figure 24. Angle of repose of riprap as a function of mean size and shape of stone.

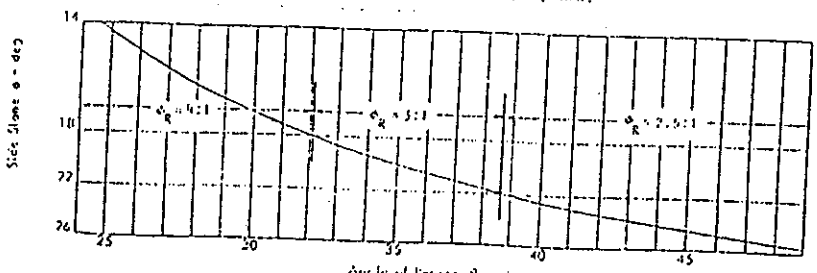
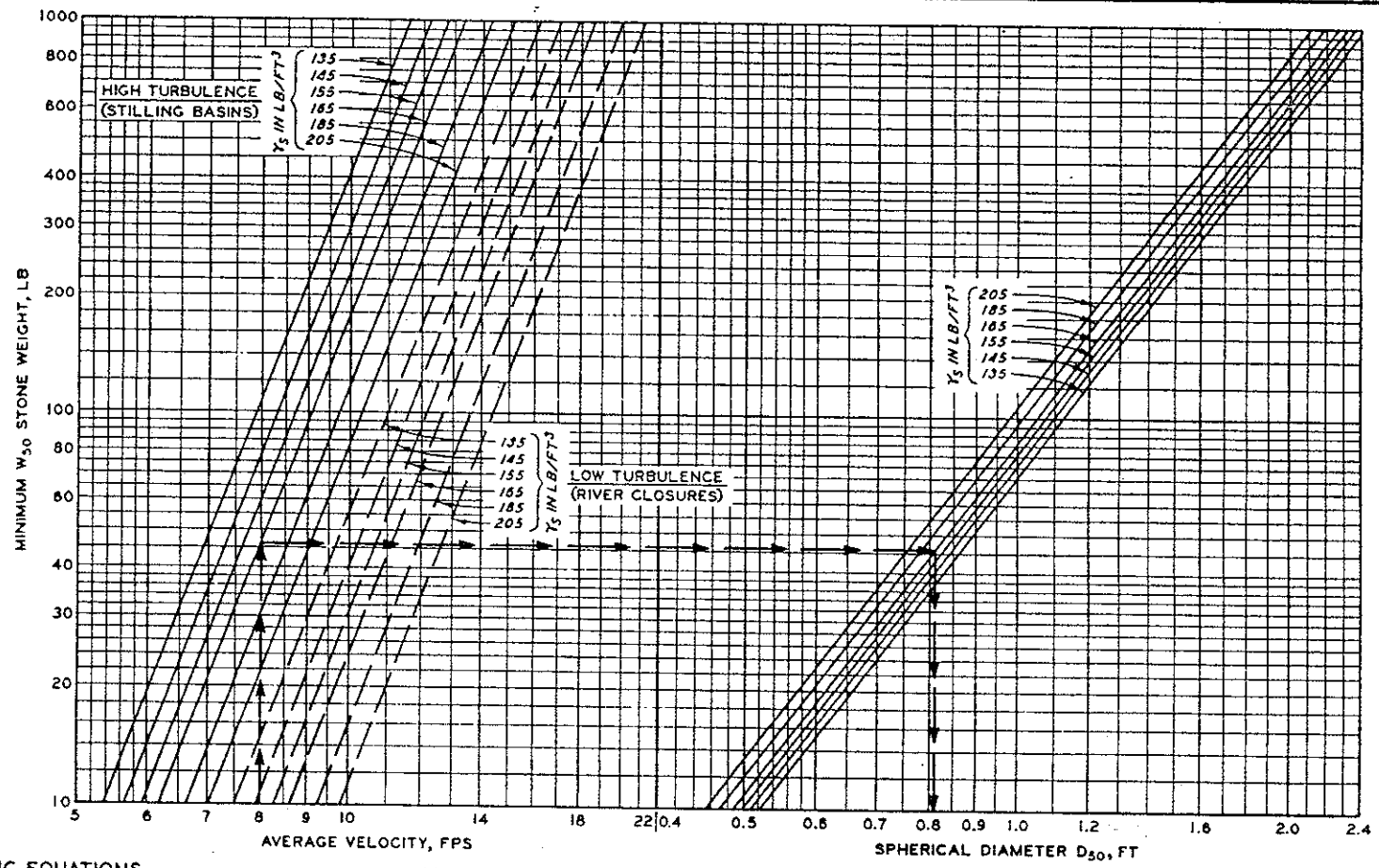


Figure 25. Recommended side slopes of horizontal channels as a function of riprap angle of repose.

II. Design of Loose Rock Riprap
B. Methods
6. CE, Stone Stability

INCL 1



PREPARED BY U. S. ARMY ENGINEER WATERWAYS EXPERIMENT STATION, VICKSBURG, MISSISSIPPI

BASIC EQUATIONS

$$V = C\left[2g\left(\frac{\gamma_S-\gamma_W}{\gamma_W}\right)\right]^{1/2}\left(D_{50}\right)^{1/2}$$

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

WHERE:
- V = VELOCITY, FPS
- γ_S = SPECIFIC STONE WEIGHT, LB/FT³
- γ_W = SPECIFIC WEIGHT OF WATER, 62.5 LB/FT³
- W_{50} = WEIGHT OF STONE. SUBSCRIPT DENOTES PERCENT OF TOTAL WEIGHT OF MATERIAL CONTAINING STONE OF LESS WEIGHT.
- D_{50} = SPHERICAL DIAMETER OF STONE HAVING THE SAME WEIGHT AS W_{50}
- C = ISBASH CONSTANT (0.86 FOR HIGH TURBULENCE LEVEL FLOW AND 1.20 FOR LOW TURBULENCE LEVEL FLOW)
- g = ACCELERATION OF GRAVITY, FT/SEC²

STONE STABILITY
VELOCITY VS STONE DIAMETER

HYDRAULIC DESIGN CHART 712-1
(SHEET 1 OF 2)

REV 8-58, 9-70 WES 6-57

INSTRUCTIONS FOR USE OF CONSTRUCTION SPECIFICATION 61

LOOSE ROCK RIPRAP

1. Applicability.

   Construction Specification 61 is applicable to equipment-placed or hand-placed loose rock riprap, filter layers and bedding.

2. Material Specifications.

   Material Specifications 521 and 523 are complementary to Construction Specification 61.

3. Items to be Included in Contract Specifications and Drawings.

   a. Complete plans and cross-sections of the required riprap.

   b. Type of placement (equipment-or hand-placed).

   c. Pay limits, where applicable.

   d. Class of subgrade fill, where applicable.

   e. Foundation preparation requirements, if any.

   f. Gradation requirements for materials.

   g. Screening, selection or other processing requirements, to insure obtaining rock of the required quality and grading.

   h. Sources of materials, if the sources are to be specified. When sources are designated in the contract, the available quantity and the quality of usable material at each source must be determined beforehand by geologic investigations and (1) adequate sampling and testing or (2) specific case history which establishes the quality by satisfactory performance under comparable conditions of use and exposure.

   i. Methods of measurement and payment if the standard specification includes more than one method.

4. Discussion of Methods.

   Section 8, Measurement and Payment.

   (1) Method 1 is intended for use where filter material or bedding is a minor item and payment for it is to be included in the payment for riprap.

   (2) Methods 2, 3, and 4 provide various means of measurement and are intended for use where filter material or bedding, if any, is to be paid for as a separate item.

(3) When all but one method are deleted for use in a contract specification, delete from the last paragraph "All Methods The following provisions apply to all methods of measurement payment" and "left justify" the remaining two sentences.

CONSTRUCTION SPECIFICATION

61. LOOSE ROCK RIPRAP

1. SCOPE

The work shall consist of the construction of loose rock riprap revetments and blankets, including filter layers or bedding where specified.

2. MATERIALS

Rock for loose rock riprap shall conform to the requirements of Material Specification 523 or, if so specified shall be obtained from designated sources. It shall be free from dirt, clay, sand, rock fines and other materials not meeting the required gradation limits.

At least 30 days prior to delivery of rock from other than designated sources, the Contractor shall designate in writing the source from which he intends to obtain the rock and information satisfactory to the Contracting Officer that the material meets the requirements of the contract. The Contractor shall provide the Engineer free access to the source for the purpose of obtaining samples for testing. The size and grading of the rock shall be as specified in Section 9 of this specification.

Rock from designated sources shall be excavated, selected and processed as necessary to meet the quality and grading requirements in Section 9 of this specification. The rock shall conform to the specified grading limits when installed in the riprap.

Filter or bedding materials when required, shall, unless otherwise specified, conform to the requirements of Material Specification 521.

3. SUBGRADE PREPARATION

The subgrade surfaces on which the riprap or bedding course is to be placed shall be cut or filled and graded to the lines and grades shown on the drawings. When fill to subgrade lines is required, it shall consist of approved materials and shall conform to the requirements of the specified class of fill.

Riprap shall not be placed until the foundation preparation is completed and the subgrade surfaces have been inspected and approved by the Engineer.

4. EQUIPMENT-PLACED ROCK RIPRAP

The rock shall be placed by equipment on the surfaces and to the depths specified. The riprap shall be constructed to the full course thickness in one operation and in such a manner as to avoid

serious displacement of the underlying materials. The rock shall be delivered and placed in a manner that will insure that the riprap in place shall be reasonably homogeneous with the larger rocks uniformly distributed and firmly in contact one to another with the smaller rocks and spalls filling the voids between the larger rocks.

Riprap shall be placed in a manner to prevent damage to structures. Hand placing will be required to the extent necessary to prevent damage to the permanent works.

5. HAND-PLACED RIPRAP

The rock shall be placed by hand on the surfaces and to the depths specified. It shall be securely bedded with the larger rocks firmly in contact one to another. Spaces between the larger rocks shall be filled with smaller rocks and spalls. Smaller rocks shall not be grouped as a substitute for larger rock. Flat slab rock shall be laid on edge.

6. FILTER LAYERS OR BEDDING

When the drawings specify filter layers or bedding beneath riprap, the filter or bedding material shall be spread uniformly on the prepared subgrade surfaces to the depth specified. Compaction of filter layers or bedding will not be required, but the surface of such layers shall be finished reasonably free of mounds, dips or windrows.

7. TESTING

The Engineer will perform such tests as are required to verify that the riprap, filter, and bedding materials and the completed work meet the requirements of the specifications. These tests are not intended to provide the Contractor with the information he needs to assure that the materials and workmanship meet the requirements of the specifications, and their performance will not relieve the Contractor of the responsibility of performing his own tests for that purpose.

8. MEASUREMENT AND PAYMENT

Method 1 For items of work for which specific unit prices are established in the contract, the volume of each type of riprap, including filter layers and bedding, will be measured within the specified limits and computed to the nearest cubic yard by the method of average cross-sectional end areas. Payment for each type of riprap, including filter layers and bedding, will be made at the contract unit price for that type of riprap. Such payment will be considered full compensation for all labor, materials, equipment and all other items necessary and incidental to the completion of the riprap, filter layers and bedding.

Method 2 For items of work for which specific unit prices are established in the contract, the volume of each type of riprap and the volume of each type of filter layer or bedding will be measured within the specified limits and computed to the nearest cubic yard by the method of average cross-sectional end areas. Payment for each type of riprap will be made at the contract unit price for that type of riprap. Payment for each type of filter or bedding will be made at the contract unit price for that type of filter or bedding. Such payment will be considered full compensation for all labor, materials, equipment and all other items necessary and incidental to the completion of the riprap, filter layers and bedding.

Method 3 For items of work for which specific unit prices are established in the contract, the quantity of each type of riprap placed within the specified limits will be measured to the nearest ton by actual weight, and the volume of each type of filter layer or bedding will be measured within the specified limits and computed to the nearest cubic year by the method of average cross-sectional end areas. For each load of rock placed as specified, the Contractor shall furnish to the Engineer a statement-of-delivery ticket showing the weight, to the nearest 0.1 ton, of rock in the load.

Payment for each type of riprap will be made at the contract unit price for that type of riprap. Payment for each type of filter or bedding will be made at the contract unit price for that type of filter or bedding. Such payment will be considered full compensation for all labor, materials, equipment and all other items necessary and incidental to the completion of the riprap, filter layers and bedding.

Method 4 For items of work for which specific unit prices are established in the contract, the quantity of each type of riprap placed within the specified limits will be measured to the nearest ton by actual weight, and the volume of each type of filter material or bedding delivered and placed within the specified limits will be measured to the nearest cubic yard by measurement of the hauling equipment. For each load of material placed as specified, the Contractor shall furnish to the Engineer a statement-of-delivery ticket showing the weight, to the nearest 0.1 ton, of rock in the load; or the volume, to the nearest 0.1 cubic yard, of filter material or bedding in the load.

Payment for each type of riprap will be made at the contract unit price for that type of riprap. Payment for each type of filter or bedding will be made at the contract unit price for that type of filter or bedding. Such payment will be considered full compensation for all labor, materials, equipment and all other items necessary and incidental to completion of the riprap, filter layers and bedding.

All Methods The following provisions apply to all methods of measurement and payment. Compensation for any item of work described in the contract but not listed in the bid schedule will be included in the payment for the item of work to which it is made subsidiary. Such items and the items to which they are made subsidiary are identified in Section 9 of this specification.

INSTRUCTIONS FOR USE OF CONSTRUCTION SPECIFICATION 203

GEOTEXTILES

1. APPLICABILITY

This construction specification is applicable in the installation of both woven and nonwoven geotextile fabric under riprap, stone filling, precast concrete blocks, gabions, or other similar applications. This specification is generally used where geotextile fabric is placed on natural granular soil materials or fine-grained material surfaces. Geotextile fabric, if properly designed and installed, may eliminate or substantially reduce the need for multiple zones of sand or gravel filter material that might otherwise be required where filtration and drainage are necessary.

The specification is also applicable in the installation of nonwoven geotextile for subsurface drainage, such as underdrains, interceptor drains, and relief wells. The geotextile may eliminate the use of a fine-grained filter or transition material. It also allows the granular or coarse drainage material to be selected from available sources with less concern on specific gradation.

Finally, the specification is applicable in the installation of access roads, recreation roads, and other light to medium use roads. The geotextile allows stabilization of the road subgrade with less depth of gravel or select backfill. Future maintenance is also reduced.

For more detailed discussion on functional, design, and construction aspects of geotextiles, see Technical Note W-26, "Guide for the Use of Geotextiles".

2. MATERIAL SPECIFICATIONS

Material Specification 303 is complementary to Construction Specification 203.

3. ITEMS TO BE INCLUDED IN THE CONTRACT SPECIFICATIONS AND DRAWINGS

a. Material selected for use, Class I, II, III or IV and woven or nonwoven, if appropriate. For definitions of Class I-IV, see "Guide for the Use of Geotextiles".

b. Requirements, as applicable, for Apparent Opening Size (AOS) and Percent Open Area.

c. When specified, size and configuration of panels for payment, including required anchorage and overlap. Details of panel placement and orientation where necessary.

d. The required roll width, when appropriate.

e. More specific details on lap, securing pins, size, spacing, and placement of rolled geotextile.

f. Requirements, as appropriate, for sewing of laps and splices.

g. Requirements and details for toe protection and anchorage at the perimeters of the fabric need to be shown. Good details and notes on various types of anchorage should be provided.

h. Requirements of cushioning or bedding material used to cover the geotextile to protect against damage during riprap placement.

i. Modified or additional requirements for the geotextile fabric properties as applicable when design requirements exceed the minimum values indicated in Table 1 or 2.

j. Special installation requirements.

4. DISCUSSION OF METHODS

Section 5, Measurement and Payment

1) Methods 1 and 2 are intended for use when the quantity of geotextile is measured for payment.

2) Method 3 is intended for use when the quantity will not be measured for payment and there is not likely to be any change in quantity from that estimated.

3) When all but one method are deleted for use in a contract specification, delete from the last paragraph "All Methods" and left justify the remaining text.

203. GEOTEXTILES

1. SCOPE

This work shall consist of furnishing all materials, equipment, and labor necessary for the installation of geotextiles.

2. MATERIALS

Geotextile fabric shall conform to the requirements of Material Specification 303 and Section 7 of this specification.

3. STORAGE

Prior to use, the geotextile shall be stored in a clean dry place, out of direct sunlight, not subject to extremes of either hot or cold, and with the manufacturer's protective cover in place.

4. SURFACE PREPARATION

The surface on which the geotextile is to be placed shall be graded to the neat lines and grades as shown on the drawings. The surface shall be reasonably smooth, free of holes, depressions, and projections. Excessively wet, muddy conditions shall be dewatered where geotextiles are to be placed against an excavation or prepared slope. The surface preparation will be inspected and approved by the Engineer prior to placing the geotextile.

5. PLACEMENT

A. General

The geotextile shall be placed on the approved prepared surface at the locations and in accordance with the details shown on the drawings and specified in Section 7. The geotextile shall be unrolled along the placement area and loosely laid (not stretched) in such a manner that it will conform to the surface irregularities when the stone or other material is placed on or against it. No cuts or punctures will be permitted in the geotextile unless otherwise specified. The geotextile may be folded and overlapped to permit proper placement in the designated area.

When specified in Section 7, the geotextile shall be provided in panels and the seams of the geotextile shall be machine sewn with thread of a material meeting the chemical requirements given for plastic yarn. The panels shall be sewn together at the factory or another approved location to form panels of the specified size.

The orientation of the width and length of the geotextile panels shall be as shown on the drawings. (Note: Alternate panel sizes may be submitted for approval.) The length shall be placed parallel to the direction of water flow unless otherwise indicated on the drawings. The end roll details shall be as shown on the drawings or as specified in Section 7. The minimum overlap shall be 18 inches unless otherwise specified.

Securing pins, provided by the geotextile manufacturer, shall be placed along the edge of the geotextile to adequately secure it during placement. Pins shall be steel or fiberglass formed as a "U", "L", or "T" share or contain "ears" to prevent total penetration. Steel washers shall be provided on all but the "U"-shaped pins. At vertical laps, securing pins shall be inserted through both layers along a line through the approximate midpoint of the overlap. At horizontal laps and laps across slopes, securing pins shall be inserted through the bottom layer only. Securing pins shall be placed along a line approximately 2 inches in from the edge of the outer limits of the placed geotextile at intervals not greater than 12 feet unless otherwise specified. Additional pins shall be installed as necessary to prevent any slippage of the fabric, regardless of locations. The use of securing pins will be held to the minimum necessary. Pins shall be left in place unless otherwise specified. The fabric may be secured with other methods when specified.

B. Slope Protection

The fabric shall not be placed until it can be anchored and protected with the intended covering within 48 hours. If this is not practical, temporary cover, for protection from ultraviolet light, may be used as approved by the Engineer.

C. Subsurface Drains

The geotextile shall not be placed until drainfill or other material can be used to cover it within the same working day. Drainfill material shall be placed in a manner that prevents damage to the geotextile. In no case will material be dropped on uncovered fabric from a height greater than 5 feet.

D. Road Stabilization

The geotextile shall be unrolled parallel to the roadway centerline, and loosely laid (not stretched) in such a manner that it will conform to the surface irregularities when the gravel or other fill material is placed on it. Unless otherwise specified, the minimum overlap of geotextile shall be 24 inches for unsewn fabric.

The overlap can be reduced by field sewing the geotextile with 2 rows of stitching located at least 3 inches from the fabric edge. The fabric will be sewn with thread of a material meeting the chemical requirements given for plastic yarn.

Securing pins, provided by the geotextile manufacturer, may be used when necessary, to insure temporary anchoring of the geotextile during the installation process. The pins shall be removed prior to permanent backfilling.

The gravel or select backfill should be dumped and spread in a uniform thickness generally not to exceed 9 inches after compaction. The first 2 or 3 lifts may be used to cover the geotextile and receive only light compaction. Overstressing the soil and severe rutting should be avoided by utilizing equipment in spreading and dumping that exerts only moderate pressures on the soil. Backfill depths may have to be increased and equipment loads reduced to control soil stress. Ruts developed during spreading and compacting should be filled with additional backfill material so that backfill thickness can be maintained.

6. MEASUREMENT AND PAYMENT

Method 1. For items of work for which specific unit prices are established in the contract, the quantity of geotextile placed within the specified limits will be determined to the nearest unit area by computing the area covered. Payment for the geotextile will be made at the contract unit price. Such payment will be considered full compensation for all labor, materials, equipment, (including specified overlaps and anchorages, as shown on the drawings), and other incidental items necessary for the completion of the work.

Method 2. For items of work described in Section 7 of this specification for which specific unit prices are established in the contract, the quantity of geotextile placed within the specified limits will be determined to the nearest unit area by computing the area of the actual roll size or partial roll. The roll measurement includes allowances for overlaps and end anchorage as shown on the drawings. Payment for the geotextile will be made at the contract unit price. Such payment will be considered full compensation for all labor, materials, equipment, and other incidental items necessary to the completion of the work.

Method 3. For items of work for which specific lump sum prices are established in the contract, payment for geotextiles will be made at the contract lump sum price. Such payment will constitute full compensation for furnishing, transporting, and installing the geotextile including furnishing all labor, equipment, tools, materials, and other incidental items necessary for the completion of the work.

All Methods. Compensation for any item of work described in the contract but not listed in the bid schedule will be included in the payment for the item of work to which it is made subsidiary. Such items and the items to which they are made subsidiary are identified in Section 7 of this specification.

MATERIAL SPECIFICATION

521. *AGGREGATES FOR DRAINFILL AND FILTERS*

1. SCOPE

This specification covers the quality of mineral aggregates for the construction of drainfill and filters.

2. QUALITY

Drainfill and filter aggregates shall be sand, gravel or crushed stone or mixtures thereof. They shall be composed of clean, hard, durable mineral particles free from organic matter, clayballs, soft particles or other substances that would interfere with their free-draining properties.

Aggregates of crushed limbstone shall be thoroughly washed and screened. Course aggregate containing crushed limestone shall have not more than 3 percent by weight of particles finer than the No. 4 sieve. Crushed limestone shall not be used for fine aggregates except in combination with other materials such that not more than 5 percent of the portion finer than the No. 4 sieve shall be crushed limestone.

Aggregates shall be tested for soundness according to ASTM Method C 88, and shall have a weighted average loss in five cycles of not more than 12 percent when sodium sulfate is used or 18 percent when magnesium sulfate is used.

3. GRADING

Drainfill and filter aggregates shall conform to the specified grading limits after being placed in the work, and after being compacted if compaction is specified. Grading shall be determined by ASTM Method C 136.

4. STORING AND HANDLING

Drainfill and filter aggregates shall be stored and handled by methods that prevent segregation of particle sizes or contamination by mixing with other materials.

Rock that fails to meet the requirements stated in a, b, and c above, may be accepted only if similar rock from the same source has been demonstrated to be sound after 5 years or more of service under conditions of weather, wetting and drying, and erosive forces similar to those anticipated for the rock to be installed under this specification.

3. GRADING

The rock shall conform to the specified grading limits after it has been placed in the riprap.

MATERIAL SPECIFICATION

523. ROCK FOR RIPRAP

1. SCOPE

This specification covers the quality of rock to be used in the construction of rock riprap.

2. QUALITY

Individual rock fragments shall be dense, sound and free from cracks, seams and other defects conducive to accelerated weathering. The rock fragments shall be angular to subrounded in shape. The least dimension of an individual rock fragment shall be not less than one-third the greatest dimension of the fragment.

Except as provided below, the rock shall have the following properties:

a. Bulk specific gravity (saturated surface-dry basis) not less than 2.5.

b. Absorption not more than 2 percent.

c. Soundness: Weight loss in 5 cycles not more than 10 percent when sodium sulfate is used or 15 percent when magnesium sulfate is used.

The bulk specific gravity and absorption shall be determined by ASTM Method C 127. The test for soundness shall be performed by ASTM Method C 88 for coarse aggregate modified as follows:

The test sample shall not be separated into fractions. It shall consist of 5000 ± 300 grams of rock fragments, reasonably uniform in size and shape and weighing approximately 100 grams each, obtained by breaking the rock and selecting fragments of the required size.

After the sample has been dried, following completion of the final test cycle and washing to remove the sodium sulfate or magnesium sulfate, the loss of weight shall be determined by subtracting from the original weight of the sample the final weight of all fragments which have not broken into three or more pieces.

The report shall show the percentage loss of weight and the results of the qualitative examination.

303. GEOTEXTILES, WOVEN AND NONWOVEN

1. SCOPE

This specification is applicable to the quality control of both woven and nonwoven geotextiles.

2. QUALITY

Geotextiles shall be manufactured from synthetic long chain or polymeric filaments or yarns such as polypropylene, polyethylene, polyester, polyamide, or polyvinylidene-chloride. The geotextile shall be formed into a stable network of filaments or yarn that retain their relative position to each other, are inert to commonly encountered chemicals, and are resistant to ultraviolet light, heat, hydrocarbons, mildew, rodents, and insects. The geotextile shall be free of any chemical treatment or coating that might significantly reduce its permeability and shall have no flaws or defects that significantly alter its physical properties. The geotextile shall be exposed to ultraviolet radiation (sunlight) for no more than 15 days total in the period of time following manufacture until the fabric is covered with soil, rock, concrete, etc. Unless otherwise specified, the requirements for materials are as follows:

a. Woven Geotextile

Woven geotextile shall conform to the physical properties listed in Table 1. The woven geotextile shall be manufactured from monofilament yarn that is woven into a uniform pattern with distinct and measurable openings. The fabric shall be manufactured so that the yarns will retain their relative position with regard to each other. Yarn composition shall be at least 85 percent by weight of propylene, ethylene, or vinylidene-chloride, and shall contain stabilizers and/or inhibitors to enhance its resistance to ultraviolet or heat exposure. The edges of the material shall be selvaged or otherwise finished to prevent the outer yarn from unraveling.

b. Nonwoven Geotextile

Nonwoven geotextile shall conform to the physical properties listed in Table 2. Nonwoven geotextile shall be manufactured from randomly oriented fibers bonded together by the needle-punched process. In addition, one side may be slightly heat bonded.

(Note: Non-needle punched, heat bonded nonwoven fabric may be used for Class IV in Table 2.)

c. Shipping, Product Identification, Certification, and Test Data

The geotextile shall be shipped in rolls wrapped with a protective covering to keep out mud, dirt, dust, debris and direct sunlight. Each roll of geotextile shall be clearly marked to identify the brand, type and the individual production run. (See ASTM D 4873, Standard Guide for Identification, Storage, and Handling of Geotextiles.)

The geotextile shall meet the specified requirements (Table 1 or 2) for the product style or type shown on the label. Product properties as listed in the "Specifiers Guide" (latest issue), Geotechnical Fabrics Report, Industrial Fabrics Association International, 345 Cedar Bldg., Suite 450, St. Paul, MN., 55105, and that represent minimum average roll values, will be acceptable documentation that the product style or type meets these specified requirements.

For products that do not appear in the above directory, or do not have minimum average roll velues listed, typical test data from the identified production run of the fabric will be required for each of the specified tests, Table 1 or 2, as appropriate.

All geotextile materials will be subject to sampling and testing by qualified testing laboratories at any time from initial site delivery until final installation.

TABLE 1. REQUIREMENTS FOR WOVEN GEOTEXTILES

Property	Test Method	Class I	Class II & III	Class IV
Tensile Strength lbs 1/	ASTM D 4632 Grab Test Method	200 min. in any principal direction	120 min. in any principal direction	150 min. in any principal direction
Bursting Strength psi 1/	ASTM D 3786 Diaphragm Tester	450 min	300 min.	250 min.
Elongation - % 1/ Failure	ASTM D 4632	35 max.	35 max.	35 max.
Puncture - lbs. 1/	ASTM D 3787	100	55	60
Ultraviolet Light Resistance	ASTM D 4355 500 hrs. exposure	70% min. tensile strength retained	70% min. tensile strength retained	70% min. tensile strength retained
Apparent Opening Size (AOS)	ASTM D 4751	As specified with a min. size ≥ #100 2/	As specified with a min. size ≥ #100 2/	As specified with a min. size ≥ #100 2/
Percent Open Area	CW-02215	4.0 % min.	4.0% min.	1.0% min.

1/ Average roll minimum value (weakest principal direction).

2/ U.S. standard sieve size.

TABLE 2. REQUIREMENTS FOR NONWOVEN GEOTEXTILES

Property	Test Method	Class I	Class II	Class III	Class IV 3/
Tensile Strength lbs 1/	ASTM D 4632 Grab Test Method	180 min.	120 min.	90 min.	120 min.
Bursting Strength psi 1/	ASTM D 3786 Diaphragm Tester	320 min.	210 min.	180 min.	210 min.
Elongation - % 1/ Failure	ASTM D 4632	100 max.	100 max.	100 max.	100 max.
Puncture - lbs. 1/	ASTM D 3787	80	40	35	50
Ultraviolet Light Resistance	ASTM D 4355 500 hrs.	70% tensile strength retained	70% tensile strength retained	70% tensile strength retained	70% tensile strength retained
Apparent Opening Size (AOS)	ASTM D 4751	Size ≤ #40 max. 2/	Size ≤ #40 max. 2/	Size ≤ #40 max. 2/	Size ≤ #40 max. 2/
Permittivity $sec.^{-1}$	ASTM D 4491	0.70	0.70	0.70	0.30

1/ Average roll minimum value (weakest principal direction).

2/ U.S. standard sieve size.

3/ A heat-bonded (non-needle punched) nonwoven fabric may be used for this class.

GIVEN:

1. Average Stream Channel Slope = 0.01 ft/ft
2. Depth = 5 ft
3. Side Slopes = 2:1
4. Area at Bankfull = 950 ft^2
5. Wetted Perimeter = 205 ft
6. Hydraulic Radius = 4.63 ft
7. Velocity = 14 ft/sec
8. Design Discharge = 13,110 cfs
9. Manning's n = 0.030
10. Length of Treatment = 400 ft
11. Straight Alignment

REQUIRED:

Rock riprap size for the following methods:
1. Engineering Field Manual (EFM)
2. Far West States (FWS)
3. Advisory PO-18 (HRP 108 – adaptation)
4. National Crushed Stone Association
5. Corps of Engineers, EM 1110-2-1601, ETL 1110-2-120
6. Corps of Engineers, Stone Stability Charts

SOLUTION:

1. EFM

Ref: Chapter 16, "Streambank Protection", p. 16-18

Velocity = 14 ft/sec

Exhibit 16-1 p. 16-20

D_{100} = 28in.

Weight around 1600 lbs

Exhibit 16-2 p. 16-21

Minimum and maximum sizes of stone in riprap may be 50 to 1600 lbs. Weight of 75% of stones ranges from 400 to 1600 lbs

SOLUTION (Cont'):

2. FWS

Ref: "Criteria for Riprap Rock Size", p. 6-13a to 13d

Channel Slope = 0.01 ft/ft
Depth = 5.0 ft
Side Slope = 2:1
Straight Alignment

FWS p. 16-13b p. 16-13c

$K = 0.72$
$C = 1.0$

Nomagraph p. 16-13d

$D_{75} = 15.5$ in.

3. Advisory PO-18 (Adaptation to HRP-108)

Ref: "Rock Riprap Design Procedure", May 1, 1974

Design Discharge, Q = 13,110 cfs
Channel Slope, S_c = 0.010 ft/ft
Hydraulic Radius, R = 4.63 ft
Wetted Perimeter, P = 205 ft

Correction Factor, K

$$K = K_{SG} K_A K_S$$

Specific Gravity $K_{SG} = 1.0$ if in freshwater and SG = 2.65

$$K_{SG} = \frac{1.706}{[SG-1]^{16/15}} = \frac{1.706}{[2.56-1]^{16/15}} = 1.06$$

$$K_A = \frac{(\tau_o/\bar{\tau})_{max}}{1.5} = 1.0 \quad \text{(straight channel alignment)}$$

Side-Slope For 2:1 $\phi = 26.56°$

$$K_S = \frac{0.816}{\sqrt{1-\left(\frac{\sin\phi}{0.643}\right)^2}} = \frac{0.816}{\sqrt{1-\left(\frac{\sin 26.56°}{0.643}\right)^2}} = 1.14$$

$$K = 1.06(1.0)1.14 = 1.21$$

SOLUTION (cont'):

3. Advisory PO-18 (cont')

$$D_{50} = \left[\frac{118\,Q\,(S_e)^{13/6}\,R}{\rho}\right]^{0.4} K$$

$$D_{50} = \left[\frac{118(13{,}110)(0.01)^{13/6}(4.63)}{205}\right]^{0.4} 1.21$$

$D_{50} = 1.47$ ft $= 17.6$ in.

4. NCSA

Ref: Corps of Engineers, "Stream bank Protection Course", Lecture 6-2-b-2, Incl 2

Velocity = 14 ft/sec

$D_{100} = 30$ in. $D_{min} = 12$ in.

$D_{50} = 15$ in.

5. Corps of Engineers, EM 1110-2-1601, ETL 1110-2-120

Ref: Corps of Engineers, "Streambank Protection Course", Lecture 6-2-b-2

Channel Slope, S = 0.01 ft/ft
Hydraulic Radius, R = 4.63 ft
Unit wt of Stone, γ_s = 165 #/ft³

$\tau_o = \gamma R S = (62.4)(4.63)(0.01) = 2.89$ #/ft²

Average Boundary Shear

Corrections

Non-uniformity $\tau_o = 1.5 \times 2.89 = 4.33$ #/ft²

Bend $\tau_o = 1.0 \times 4.33 = 4.33$ #/ft²

Side Slope $\tau_o = 0.72 \times 4.33 = 3.12$ #/ft²

Plate 3C in EM

SOLUTION (cont'):

5. CE (cont')

$C = 0.04$
$FS = 1.5$

$$D_{50} = \frac{T}{C(\gamma_s - \gamma_w)} = \frac{3.12(1.5)}{0.04(165 - 62.4)} = 1.14 \text{ ft}$$

$D_{50} = 14$ in.

6. CE, Stone Stability Charts

Ref: Corps of Engineers, "Structural Protection Course", Lecture G-2-b-2

Velocity, $V = 14$ ft/sec
Unit wt of stone, $\gamma_s = 165$ #/ft^3

$D_{50} = 1.26$ ft $= 15$ in. - Low Turbulence

$D_{50} = 19$ in. - High Turbulence (by equation)

GIVEN:

Q	=	2950 cfs	(Design Discharge)
V	=	8 ft/sec	(Velocity at Bankfull)
A	=	368 ft^2	(Cross-Sectional Area)
P	=	102 ft	(Wetted Perimeter)
R	=	3.61 ft	(Hydraulic Radius)
S	=	0.005 ft/ft	(Channel Slope)
d	=	4 ft	(Depth at Bankfull)
z	=	2:1	(Side Slope)
n	=	0.030	(Manning's n)
L	=	400 ft	(Length of Treatment with Straight Alignment)

REQUIRED:

Rock riprap size for the following methods:

1. Engineering Field Manual (EFM)
2. Far West States (FWS)
3. SCS Advisory PO-18 (HRP-108 adaptation)
4. National Crushed Stone Association (NCSA)
5. Corps of Engineers, EM 1110-2-1601, ETL 1110-2-120
6. Corps of Engineers, Stone Stability Charts

SOLUTION:

1. EFM

Ref: Chapter 16, "Streambank Protection", p. 16-16

$V = 8$ ft/sec

EFM Exhibit 16-1 p. 16-20

$D_{100} = 9$ in.

Weight of stone at 165 lbs/cu ft = 50 lbs

2. FWS

Ref: "Criteria for Riprap Rock Size", p. 6-13a to 13d

$S = 0.005$ ft/ft
$d = 4$ ft
$z = 2:1$ $\Rightarrow$ $K = 0.72$
Straight Channel $\Rightarrow$ $C = 1.0$

p. 16-13b
p. 16-13c

FWS Nomograph p. 16-13d

$D_{75} = 6$ in.

3. SCS Advisory PO-18

Ref: "Rock Riprap Design Procedure", 5/1/74

$Q = 2950$ cfs
$S = 0.005$ ft/ft
$R = 3.61$ ft
$P = 102$ ft

$$D_{50} = \left[\frac{118\, Q\, (S)^{13/6}\, R}{P}\right]^{0.4} K$$

SOLUTION (cont'):

3. SCS Advisory PO-18 (cont')

Correction Factor, $K = K_{SG} K_A K_S$

Specific Gravity
$K_{SG} = 1$ if $SG = 2.65$ and fresh water

$$K_{SG} = \frac{1.706}{[SG - 1]^{16/15}} = \frac{1.706}{[2.56 - 1]^{16/15}} = 1.06$$

Alignment

$$K_A = \frac{(\tau_o - \bar{\tau})_{max}}{1.5} = 1.0 \text{ (straight channel)}$$

Side Slope
For 2:1
$\phi = 26.56°$

$$K_S = \frac{0.816}{\sqrt{1 - \left(\frac{\sin\phi}{0.643}\right)^2}} = \frac{0.816}{\sqrt{1 - \left(\frac{\sin 26.56°}{0.643}\right)^2}} = 1.14$$

$$K = (1.06)(1.0)(1.14) = 1.21$$

$$D_{50} = \left[\frac{118\,(2950)\,(0.005)^{13/6}\,(3.61)}{102}\right]^{0.4} (1.21)$$

$$D_{50} = 0.53 \text{ ft} = 6.4 \text{ in.}$$

4. NCSA

Ref: CE, "Streambank Protection Course", Lecture 6-2-b-2

$V = 8$ ft/sec

By interpolation:

$D_{100} = 9.6$ in.

$D_{50} = 4.8$ in.

$D_{min} = 2.6$ in.

SOLUTION (cont'):

5. CE, EM 1110-2-1601, ETL 1110-2-120

Ref: CE, "Streambank Protection Course", Lecture 6-2-b-2

$\gamma_s = 165 \#/ft^3$

$S = 0.005$ ft/ft
$R = 3.61$ ft

Avg. Boundary Shear, $\bar{\tau}_o$

$\bar{\tau}_o = \gamma_s R S = (62.4)(3.61)(0.005) = 1.13 \#/ft^2$

Corrections

Non-Uniformity $\tau_o = (1.5)(1.13) = 1.69 \#/ft^2$
Bend $\tau_o = (1.0)(1.69) = 1.69 \#/ft^2$
Side-Slope $\tau_o = (0.72)(1.69) = 1.22 \#/ft^2$

FS = 1.5
C = 0.04

$$D_{50} = \frac{\tau}{C(\gamma_s - \gamma_w)} = \frac{1.22\ (1.5)}{0.04(165 - 62.4)} = 0.44 \text{ ft}$$

$D_{50} = 5.3$ in.

6. CE, Stone Stability Charts

Ref: CE, "Streambank Protection Course", Lecture 6-2-b-2

$V = 8$ ft/sec
$\gamma_s = 165 \#/ft^3$

High Turbulence:

$D_{50} = 0.82$ ft $= 9.8$ in.

Low Turbulence: (by equation)

$D_{50} = 0.42$ ft $= 5.0$ in.

PILING REVETMENT WITH WIRE FACING

Continuous piling revetment with a facing of woven wire is a common type of protection. It is particularly adapted to streams where the depth of water next to the bank is in excess of 3 to 4 feet. It has an advantage over riprap and brush mat construction in deep water as it is more economical and eliminates the problems involved in building a stable foundation under water for holding the riprap in place. This type of protection is easily damaged by ice flows or heavy flood debris and should not be used where these problems occur.

The piles are spaced from 6 to 8 feet on centers. Timber piles should have a diameter sufficiently large to permit driving to the required depth. Railroad rails or pipe may be used when available. The pilings are driven to a depth of approximately one-half length below the point of maximum scour. If the streambed is firm and not subject to appreciable scour, the pile should be driven to refusal or to a depth of at least one-half the length. The pilings are carried to the height required to protect the bank, which may be the full height or a lesser height if it is practical to use vegetation or other types above a certain elevation.

A heavy grade of woven wire, which conforms to State Standards and Specifications, is fastened to the stream side of the pile. Its purpose is to collect debris and trash and form a permeable wall which serves to reduce the velocity on the bank side. If the streambed is subject to scour, the woven wire is extended horizontally toward the streambed for a distance at least equal to the anticipated depth of scour. Concrete blocks or other weights are attached to the bottom at regular intervals. During floods when scour occurs the weights cause the wire to settle in a vertical position along the face of the pile. The piling can be strengthened considerably by connecting the tops with waling or by installing a system of cross bracing. Placing brush and debris back of the piling will increase the effectiveness. Where piling revetment extends for several hundred feet, a few permeable dams of brush and rock placed at right angles to the revetment will tend to stop currents developing between the bank and the piling. Typical details of a piling revetment are shown in Figure 16-3.

A more expensive type of construction, which gives more protection, is built by using two rows of piling having rock and brush in between. The rock and brush are placed in wire baskets, which must be set in a trench excavated to at least one-half or more of the depth of the anticipated scour. If the baskets settle, the wire may be extended by adding baskets of material to keep the brush and rock level with the top of the pile.

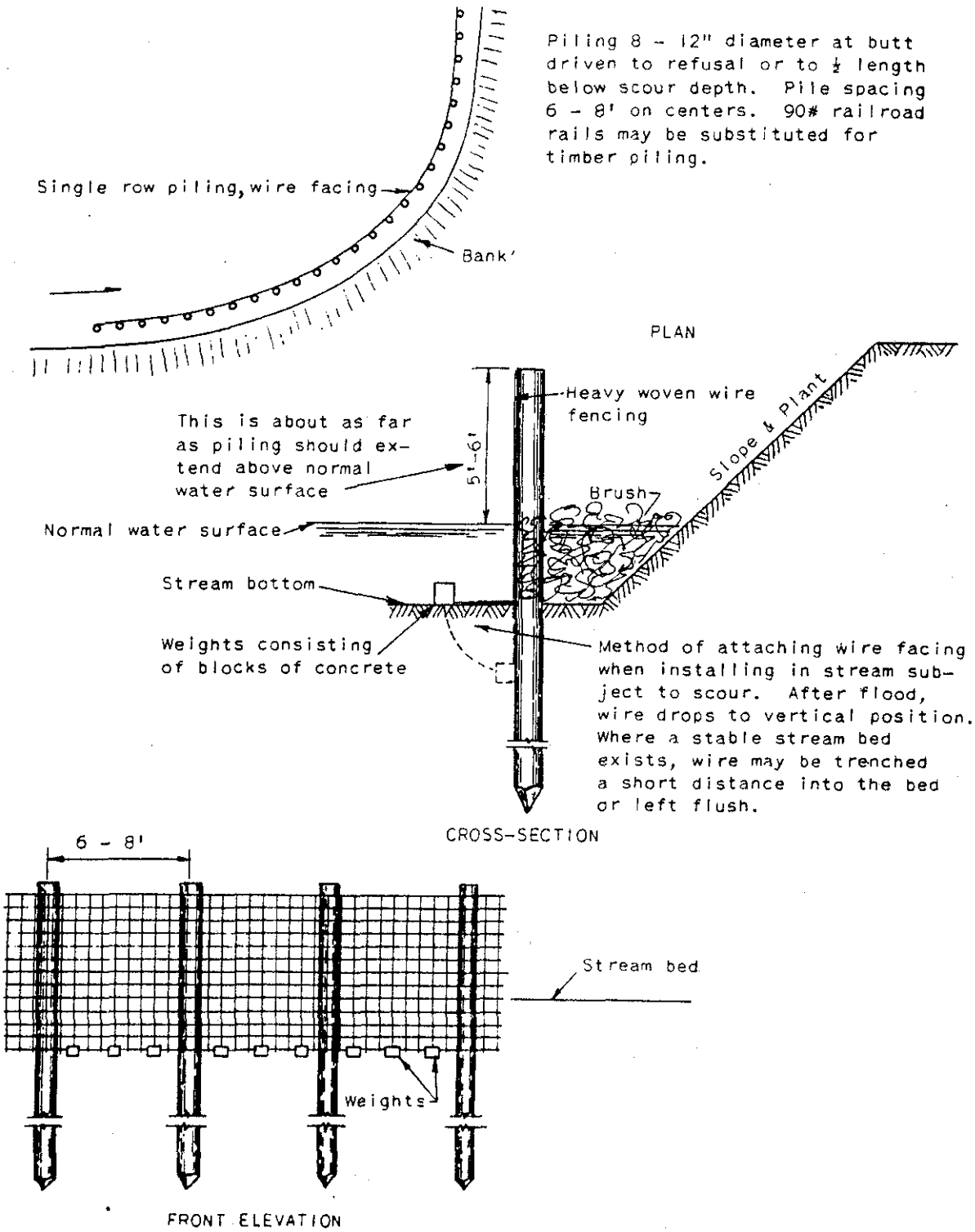


Figure 16-3 Piling revetment with wire facing

AZ. HUACHUCA CITY RC&D.

J.M.E. 3/10/88

CHANNEL HYDRAULICS 1 4

TOP OF WIRE

$$V = \frac{1.486}{n} R^{2/3} S_o^{1/2}$$

USE: $n = 0.037$

$A = 667.2 \square'$

$R = \frac{667.2 \square'}{92.9'} = 7.2$

$W.P. = 92.9'$

$S_o = 0.008 \frac{'}{'}$

$$V = \frac{1.486}{0.037} (7.2)^{2/3} (0.008 \tfrac{'}{'})^{1/2}$$

$(7.2)^{2/3} = 3.7$

$(0.008 \tfrac{'}{'})^{1/2} = 0.09$

$$V = \frac{1.486}{0.037} (3.7)(0.09)$$

$V = 13.4 \ \frac{FT}{SEC}$

BANK FULL

$n = 0.037$

$R = \frac{1201.6 \square'}{128.2'} = 9.4$

$A = 1,201.6 \square'$

$W.P. = 128.2'$

$$V = \frac{1.486}{0.037} (9.4)^{2/3} (0.008)^{1/2}$$

$S_o = 0.008 \frac{'}{'}$

$(9.4)^{2/3} = 4.5$

$$V = \frac{1.486}{0.037} (4.5)(0.09)$$

$V = 16.3 \ FT/SEC.$

HUACHUCA CITY RCFD

2 4

AREA = 210 SQ. MILES ±.

Q_{100} FROM COUNTY = 23,000 C.F.S.

ROBIN McARTHUR COULD NOT GET OTHER

Q = FROM COUNTY. ALSO THIS WAS AT BRIDGE

2± MILES UPSTREAM OF THIS SITE.

ROBIN MADE ESTIMATE FROM U.S.G.S

EQUATION = Q_{100} = 13,400 C.F.S. Q_{10} = 6,000 C.F.S.

$$Q = VA$$

TOP OF WIRE:

$$Q = 13.4 \frac{1}{sec} (667.2 \square')$$

$$Q = 8,941 \text{ C.F.S.}$$

BANK FULL:

$$Q = (16.3 \frac{FT}{sec})(1,201.6 \square')$$

$$Q = 19,586 \text{ C.F.S.}$$

AZ. HUACHUCA CITY R.C.&D.

A. ELDER 4/19/88

DIKE DESIGN 3 4

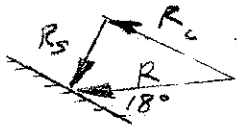
8' HIGH WOVEN WIRE FENCE WILL PROTECT TOE OF BANK WHERE EROSION IS TAKING PLACE. PLANTING TO BE DONE BEHIND FENCE.

COMPUTE LOADS BY MOMENTUM EQ. $\Sigma F = \rho Q V$

USING 18° ∠ OF ATTACK

& V = 13 FT/SEC.

INCREASE 25% FOR IMPACT.



FROM GRAHAM CANAL DESIGN

KINGS HANDBOOK P. 300

$$R = ma = \frac{W}{g}\frac{(V_1 - V_2)^2}{\text{UNIT}} = \frac{W}{g} V^2$$

$$R_s = R \sin 18°$$

$$F = R_s = \frac{W}{g} V^2 \sin 18° = \frac{62.4}{32.2}(13)^2 \sin 18°$$

$$F = 101.2 \ \#/FT^2$$

& ON 8' HIGH FENCE =

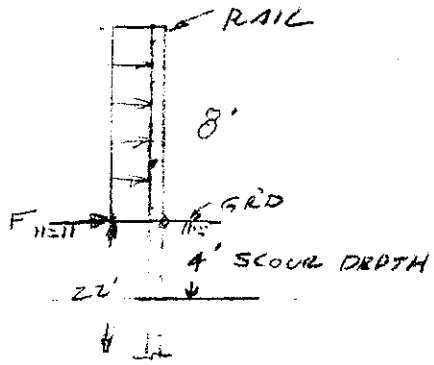
$$F = 101.2 \ \#/FT^2 \times 8' = 809.6 \ \#/FT.$$

WITH IMPACT $F = (809.6 \frac{\#}{FT})(1.25)$ $\underline{1{,}012}$ $\frac{\#}{FT}$

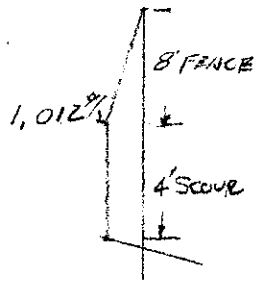
PILE DESIGN

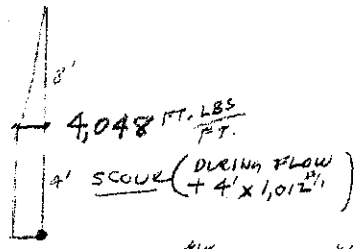
4 4

FROM GRAHAM CANAL DESIGN



SHEAR @ 8' = 1,012 #/FT.





$M = \frac{1}{2}(8')(1{,}012\ \#/FT) + (4)(1{,}012\ \#/FT)$

$M = 8{,}096\ \frac{FT\text{-}\#}{FT.}$

RAIL #/40	Sx (in³)	ALLOWABLE M / PILE	DEPTH (d)"
85	12.2	38.6 K'	5 3/16
105	14.3	45.3	5 3/16
132	27.6	87.4	7 1/8
175	23.3	72.8	6

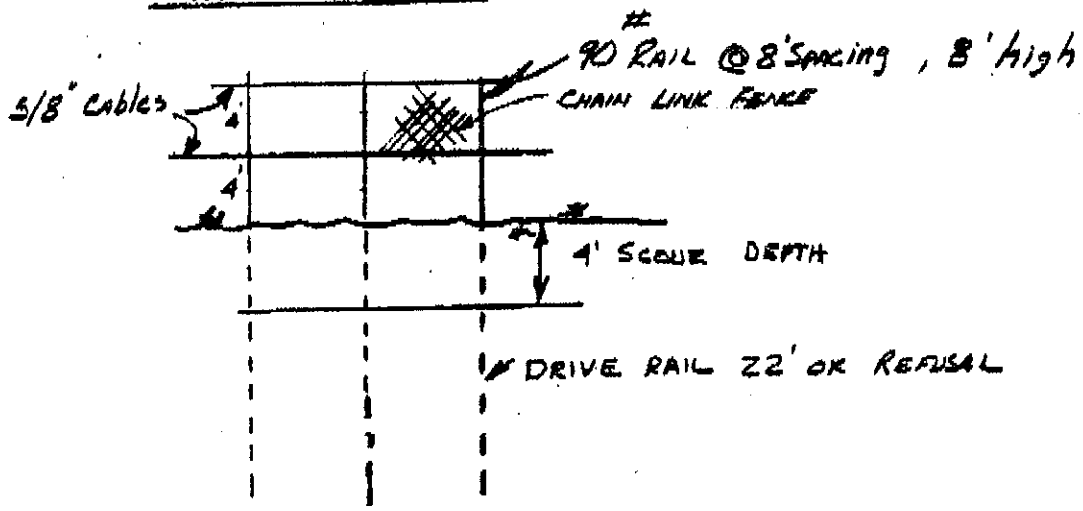
ARIZ HUACHUCA CITY RC&D

JW 6/1/89

1 5

SPONSORS SAY RAIL IS TOO EXPENSIVE. WANT TO SUBSTITUTE USED STEEL GAS PIPELINE PIPE, PILES TO BE INSTALLED WITH DRILL.

PRESENT DESIGN



FROM DESIGN REPORT 4/19/88

LOAD = 101.2 #/FT²

+ 25% IMPACT

LOAD ON 8' FENCE (101.2)(8)(1.25) = 1012 #/FT

MOMENT = 8,096 FT-# PER FT OF FENCE

W/ 8' SPACING

M = (8096)(8) = 64,768 FT-#

2 5

ALLOWABLE STEEL STRESS ?

CHAVEZ SAYS $F_y = 63{,}000$ psi FOR 0.60% CARBON

@ 50% $f_c = (63000)(1/2) = 31{,}500$

USE 30,000 psi for bending

REQD. $S = \frac{(64\,768)(12)}{f_c}$ WHERE S = SECT. MODULUS
f_c = allowable
= 30,000

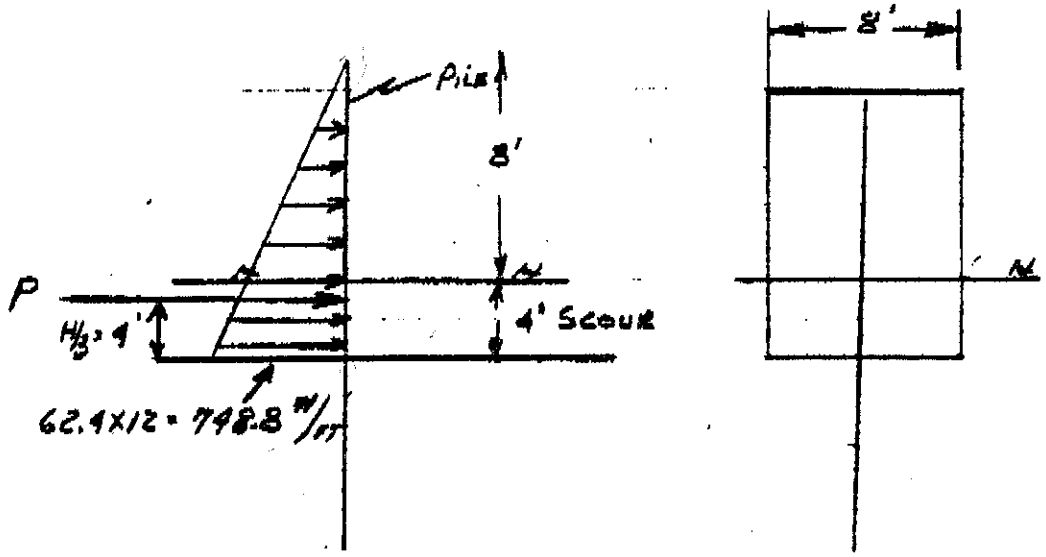
$S = 25.90 \text{ in}^3$

TOO HIGH, MOMENTUM WITH 13 ft/sec AND ASSUMED SOLID WALL NOT REALISTIC. REDUCE BY 50% ∴ REQD. $S = \frac{25.90}{2} = 12.95$

90# RAIL $S = 12.6$ @ 15.2 in^3 OK

8' spacing for 90# rail acceptable

CHECK DESIGN BY ASSUMING STATIC WATER LOAD



HYDROSTATIC LOAD (ASSUMING SOLID WALL)

$P = (748.8)(1/2)(12) = 4492$ #

MOMENT PER FT WIDTH

$(4492)(H/3) = 17,968$ FT-#/FT

8' PILE SPACING

MOMENT

$(17,968)(8) = 143,744$ FT-# High

FENCE WILL NOT ACT AS A SOLID WALL, NOR CAN IT SUSTAIN LOADS OF THIS MAGNITUDE.

USE ONLY 25% OF COMPUTED SOLID WALL LOADS

$\therefore M = (143,744)(.25) = 35,936$ FT/#

SECTION MODULUS $S = \frac{M}{f}$ where M = MOMENT

f = allowable steel stress RAIL = 30,000 psi
PIPE = 20,000 psi
(see below)

RAIL REQD $S = \frac{35,936}{30,000} \times 12 = 14.37 \text{ in}^3 \approx 90^{\#}$ RAIL

PIPE REQD $S = \frac{35,936}{20,000} \times 12 = 21.56 \text{ in}^3$

WHAT PIPE HAS REQD. S ?

DID NOT LOCATE TABLE SHOWING SECTION MODULUS OF PIPES. COMPUTE.

$$S = 0.0982 \frac{(d^4 - d_1^4)}{d}$$

COMPUTE FOR VARIOUS THICKNESSES of 6" PIPE

O.D. (d)"	t	I.D. (d_1)	S
6	.135	5.865	1.85
6	.188	5.812	5.55
6	0.219	~~5.781~~ 5.562	~~3.55~~ in³ NEED 21.56 in³

ALLOWABLE UNIT STRESS f = ?

AWWA "STEEL PIPE MANUAL" SAYS 50% OF YIELD
TABLE 6.2. ONLY SHOWS 15,000 psi
FOR 6" PIPE.

ASSUME f = 20,000 psi

5 5

TRY OTHER PIPE SIZES

O.D. d	t	I.D. d_1	S
12	.250	11.50 ~~11.75~~	26.56 ~~13.705~~ ~~A-to 30# RAIL~~

DRILL ROD:

4.5	0.344	3.812	4.34 (NEED 21.56 IN^3)

$f = 30{,}000\ \#/\square''$, $S = \frac{M}{f}$ OR $M = \frac{(4.34)(30{,}000)}{12} = 10{,}850^{\#'}$

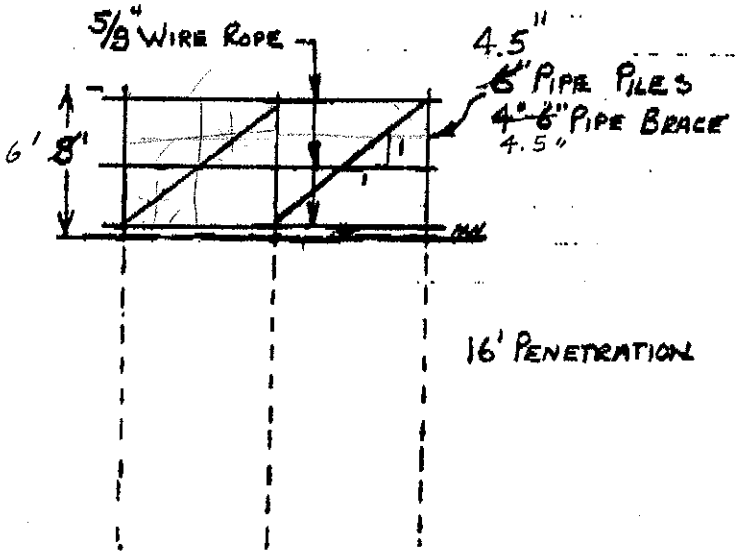
$10{,}850^{\#'} < 35{,}936^{\#'}$ NEEDED

SO USE CONFIGURATION WITH BACK & SIDE BRACES.

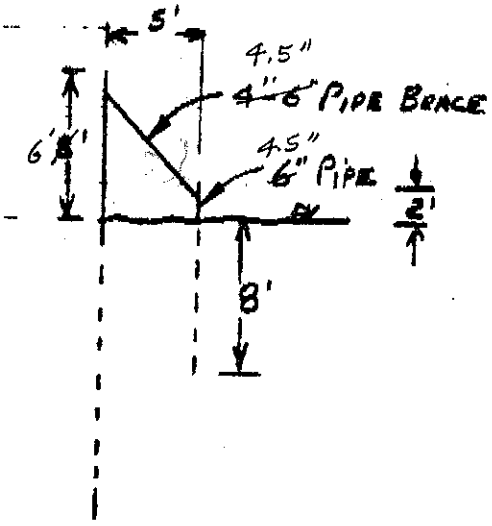
ARIZ
J. Weaver 6/2/89

HUACHUCA CITY RC&D

ALTERNATE DESIGN - SINGLE ROW ~~STEEL~~ DRILL ROD PIPE AND WIRE



ELEVATION

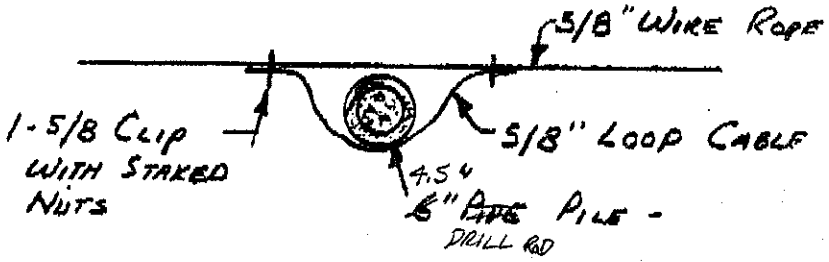


SECTION

ALL CONNECTIONS WELDED.

PILES TO BE DRIVEN OR INSTALLED IN DRILL HOLES.

WIRE ROPE CONNECTION



PLAN

Kellner Jacks 45-50 $/lin. ft. installed

Bank Stabilization — reduce velocity adjacent to bank structural, means may not always be appropriate (in high rainfall areas, N + S.E. Ariz.)

Riprap — well-graded rock very important

Grouted riprap — 6" min size, 9" slump, place conc with pump. exposed 60% rock then smooth vibrate. Regains more conc

KELLNER JACKS

PATENTED 1920

KELLNER JETTIES COMPANY

TOPEKA, KANSAS

KELLNER JACKS

PRESENTATION OUTLINE

1. SLIDES OF COMPLETED SCS JOBS

2. DISCUSS DESIGN
   - Objectives
   - Costs
   - LOCATION / GENERAL COMMENTS
   - DESIGN LOADS & CABLE SIZE
   - ANCHORS
   - STANDARD DRAWING A2 85020

3. REFERENCES FOR DESIGN

4. SPECIFICATIONS

5. O & M GUIDELINES

6. VIDEO AS TIME PERMITS

KELLNER JACKS

BASIC OBJECTIVE IS REDUCTION OF FLOW VELOCITIES WHICH CAUSES PARTIAL DEPOSITION OF SUSPENDED SEDIMENT. VEGETAL GROWTH UPON SEDIMENTS, EITHER PLANTED OR VOLUNTEER, FURTHER RETARD FLOWS AND STABILIZES DEPOSITED MATERIAL. KELLNER JACKS ARE MOST EFFECTIVE IN MODERATELY SINUOUS AND SEDIMENT-LADEN STREAMS. DO NOT INSTALL KELLNER JACKS ON HIGH VELOCITY DEBRIS LADEN STREAMS.

KELLNER JACKS

WHERE APPLICABLE, PROBABLY THE BEST TYPE OF PROTECTION TO USE UNLESS COST IS NOT A FACTOR. KELLNER JACKS PROVIDE EXCELLENT PROTECTION AT MINIMAL COST.

SINGLE LINE $45 - $50 PER LINEAL FOOT INSTALLED

CAN BE INSTALLED EITHER AS:

1. DEFLECTING JETTIES

2. SINGLE OR DOUBLE LINE

3. JETTIE FIELD WITH TIE-BACK OR LATERAL LINES.

KELLNER JACKS CAN BE CONSTRUCTED FROM:

1. WOOD

   EXAMPLE - TELEPHONE POLES, ROADRUNNER RANCH, CAMP VERDE

2. STEEL

3. CONCRETE (MOST OF THE JACKS CONSTRUCTED IN THE SOUTHERN STATES ARE CONCRETE).

GENERAL COMMENTS - KELLNER JACKS

1. JACKS ARE MOST EFFECTIVE ON SINUOUS SEDIMENT-LADEN STREAMS.

2. JACK STRING SHOULD ALWAYS BE SUPPLEMENTED WITH PLANTED VEGETATION IF POSSIBLE.

3. EXTEND BANK PROTECTION FAR ENOUGH TO PREVENT BEING OUT FLANKED. TIE TO HARDPOINT AT U/S END. START AT STABLE SECTION.

4. OVER THE BANK ANCHORS ARE PREFERRED. USE STREAMBED ANCHORS AS A LAST RESORT. IF USED, INSTALL STREAMBED ANCHORS BELOW ANTICIPATED SCOUR DEPTH. CONSIDER PILE ANCHORS IF STREAMBED ANCHORS ARE REQUIRED.

5. CONTINUOUS STRING ALONG THE BANK IS USUALLY PREFERABLE INSTEAD OF DEFLECTING JETTIES BECAUSE THE D/S ANCHOR ON THE END OF THE JETTIE IS USUALLY DIFFICULT TO CONSTRUCT TO DESIGN DEPTH AND IS SUBJECT TO DISPLACEMENT FROM CHANNEL SCOUR.

6. USUALLY BEST TO HOLD THE BANK AND NOT ATTEMPT TO "TRAIN" OR RE-DIRECT THE PATH OF FLOW.

7. MEANDER PATTERN OF STREAMS CONSTANTLY CHANGING. SHORT PIECES OF BANK PROTECTION PUT IN FOR SET OF CONDITIONS MAY NOT WORK FOR LONG.

8. LONG RADIUS, EASY CURVES ARE DESIRABLE. THE EFFECTS OF SHARP ANGLES TENDS TO TRAVEL DOWNSTREAM, WHICH IS ANOTHER REASON FOR EASY CURVES.

KELLNER JACKS

ADVANTAGES:

1. CAN BE FABRICATED IN YARD OR SHOP AND PLACED IN STREAM QUICKLY. VERY EFFECTIVE METHOD OF EMERGENCY TREATMENT.

2. LONG LIFE (STEEL MEMBER).

3. INEXPENSIVE IN COMPARISON WITH OTHER STRUCTURAL BANK PROTECTION.

4. ADAPTABLE TO WIDE RANGE OF CONDITIONS AND ALIGNMENT, FLEXIBLE, EXAMPLE - TUBAC SCHOOL, WHERE ROCK RIP RAP FAILED, KELLNER JACKS HAVE PERFORMED WELL.

DISADVANTAGES:

1. MAY SINK IN SOFT STREAMBED, EXAMPLE - PUERCO RIVER SITE.

2. LEGS MAY BE DAMAGED BY FLOATING TREES OR OTHER DEBRIS.

INSTALLATIONS IN ARIZONA (PARTIAL LISTING)

SCS-EWP

TUBAC (2 SITES), NOGALES (2 SITES), MARANA, SAFFORD, LITTLEFIELD, CAMP VERDE (2-3 SITES), DATE CREEK, SYCAMORE CANYON, PUERCO RIVER EAST OF HOLBROOK.

SCS-CCC

CHINLE WASH AT CHINLE.

OTHER

USBR - AT BUCKEYE, ON GILA RIVER AT ARIZONA GAME AND FISH DEPARTMENT FARM.

DESIGN

STANDARD DRAWING
SCS-ARIZ AZ-85020 (2 sheets)

DESIGN LOADS & CABLE SIZE

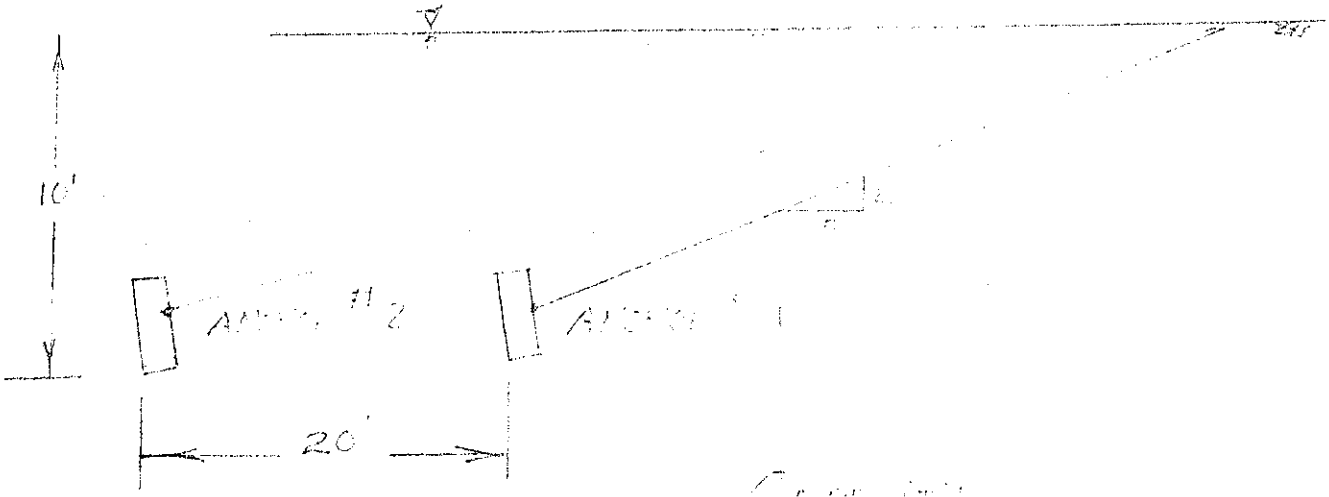
METHOD 1 "PARSONS METHOD"
USES MOMENTUM EQUATION $F = MV$ TO COMPUTE FORCES ON A SINGLE JACK

METHOD 2 DRAG FORCE $F = C_D \rho A \frac{\Delta V^2}{2}$

METHOD 3 USE MINIMUM OF 2-3/4" dia. cables

ANCHORS

THE ANCHOR SHOWN ON AR2. STANDARD DRAWING A2-85000 IS A 8'X2'X1'-7" CONCRETE BLOCK. THIS ANCHOR WAS DESIGNED ASSUMING 10' DEPTH IN SATURATED SOIL. HOLDING CAPACITY FOR 2 ANCHORS @ 10' DEPTH SPACED 20' APART IS:



CAPACITY

ANCHOR #1 - 22 T
ANCHOR #2 - 28 T
TOTAL 50 T

DESIGN NOTES (SEE STANDARD DRAWING)

- 3/8" dia. Leg cable to prevent spinning
- Use welded or bolted connections
- Legs cut from 40' length steel angles produces 2-16' & 1-8'
- Used cable OK but assume strength = 1/2 new cable, galvanized
- angles are 4X4X1/4

REFERENCES

1. "BANK AND SHORE PROTECTION IN CALIFORNIA HIGHWAY PRACTICE", NOVEMBER 1970

2. U.S. CORPS OF ENGINEERS "CIVIL WORKS INVESTIGATION NO. 858 - USE OF STEEL JETTIES ON ALLUVIAL STREAMS", AUGUST 1962

SPECIFICATIONS

SPECIFICATIONS HAVE BEEN PREPARED FOR ALL OF THE SCS EWP JOBS. SAMPLE SPECIFICATION ATTACHED (FOR NOGALES WASH EWP 1986).

MAINTENANCE

O&M GUIDELINES FOR KELLNER JACKS AND CHANNEL VEGETATION ARE ATTACHED.

J. WEAVER

FIRESTONE GARDEN
EMF PROJECT

EMPIRICAL [illegible] ON KELLNER JACKS
[illegible] EQUATION

16

① MAIN LINE (CONT.)
PARSONS METHOD

ASSUMPTIONS

1. WATER DEPTH 6'
2. VELOCITY - STREAM SIDE = 11.5'/s
   - JACK SIDE (1/3) = 3.8
3. SPACING - 12 1/2'
4. WATER HITS @ 30%
5. 3/10 FLOW THROUGH
   7/10 FLOW DIVERTED
6. WATER DEPTH = 6'

SOLUTION

1. EFFECTIVE AREA PER JACK
   6' X 12.5' = 75 sq. ft.

2. FRICTION LOAD PER JACK
   $\Delta V = 11.5 - 3.8 = 7.7'/s$

3. FRICTION FORCE (F)

$$F = \frac{w A \Delta V}{g} = \frac{(62.4)(75 ft^2)(3/10)(7.7'/s)}{32.2}$$

$F = \cancel{340}$
$= 372$

① MAIN LINE (CONT)

4. LOAD OF DIVERTED FLOW

$F = M \frac{\Delta V}{\Delta t}$ STREAM SIDE V = 11.5

$M = \frac{7/10 (62.4)(75)(11.5)}{32.2} = 1170$

$\Delta V = \sqrt{2V^2 - 2V^2 \cos 30°}$

$= \sqrt{2(11.5^2) - 2(11.5^2).87}$

$= \sqrt{264 - 230}$

$= \sqrt{34} = 5.83$ '/s

$F_x = (1170)(5.83)(1 - \cos 30°)$

$= (1170)(5.83)(1 - .87)$

$= (1170)(5.83)(.134)$

$F_x = 914$ #

$F_y = F_x \Delta v (.5)$

$= (914)(5.83)(.5)$

$F_y = 2664$ #

5. UNIT STREAM FLOW PER JACK

WATER DEPTH = 6'
JACK SPACING = 12.5'
STREAMSIDE V = 11.5

$\varphi = (6)(12.5)(11.5) = 863$ cfs

WHEN $\angle = 30°$ DEFLECTED $\varphi_2 = 1/2\ \varphi_1$

EACH JACK DEFLECTS $7/10\ \varphi_2 = (7/10)(1/2\ \varphi_1)$

THEN RESULTANT FORCE ON JACKS = $1/2\ F_x$, $1/2\ F_y$

& $1/2\ F$ (RESULTANT)

$F = MV\sqrt{2(1-\cos 30°}$

$= (1170)(11.5)\sqrt{2(.13)}$

$= 6860$ #

$1/2\ F = 3430$ #

DEFLECTED LOAD ON CABLE

$F_D = 1/2\ F \cos(60° + 15°)$

$= 1/2\ (6860)(\cos 75°)$ .2589

$= 890$ #/JACK

6. IMPACT LOAD PER JACK

ASSUME HIT WITH 12" DIA. LOG - 10' LONG

$V_1 = 11.5$ '/s $\quad V_2 = 0$ '/s

LET $\Delta t = 0.1$ SEC.

DISTANCE TO STOP $\frac{V_1 - V_2}{2}(\Delta t) = \left(\frac{11.5-0}{2}\right)(.1) = 0.58'$

USE 0.6'

$F = ma \qquad a = \frac{\Delta V}{\Delta t} = \frac{11.5}{.1} = 115$ FT/SEC

$$M = \frac{(\pi)(d^2)(L)(\omega)}{4g}$$

$$= \frac{(3.14)(1.0^2)(10)(45^{\#}/FT^2)}{(4)(32.2)}$$

$$M = 10.97$$

$F = Ma = (10.97)(115) = 1260^{\#}$

MAX. LOAD ON JACK = FRICTION + DIVERTED FLOW + IMPACT

= STEP 3 + STEP 5 + STEP 6

$= 340 + 890 + 1260 \doteq 2490^{\#}$

MAX LOAD ON CABLE WITH IMPACT LOAD ON 1 JACK

CABLE L = 250' SPACING 12.5'

250/12.5 = 20 JACKS

LOAD ON 250' STRING

L = (20 JACKS)(FRICTION + DIVERTED) + IMPACT

= 20(340 + 890) + 1260

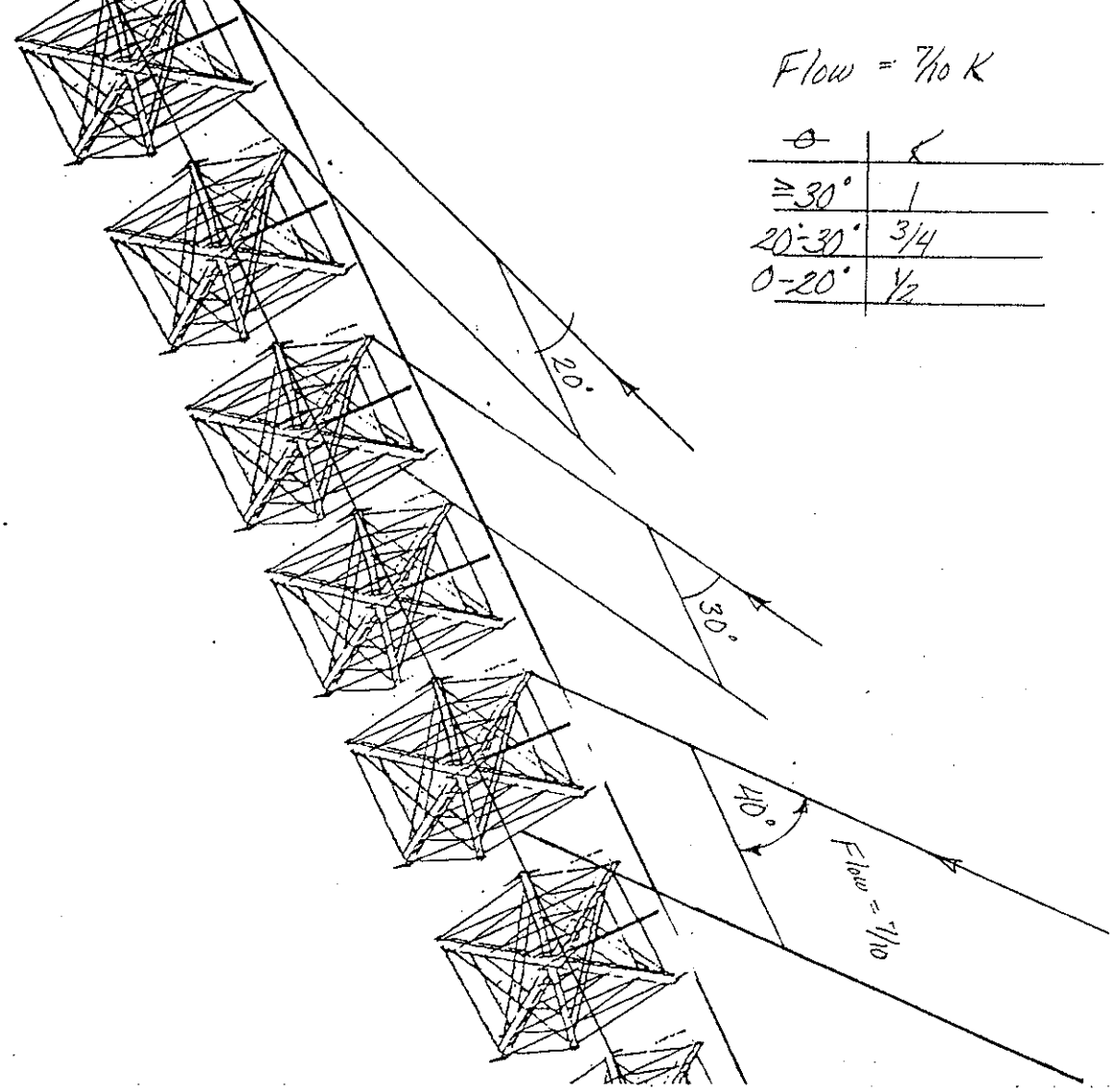
= 25,860 # = 12.93 TON

BREAKING STRENGTH OF VARIOUS WIRE ROPE
REFERENCE TIGER BRAND WIRE ROPE
ENGINEERING HAND BOOK U S STEEL
1945

6X12, 7 FIBER CORE	PLOW STEEL	FACTOR SAFETY SINGLE LINE
3/4	11.4 TON	
13/16	13.3 TON	1.02
7/8	15.4 TON	1.19
1"	20.0 TON	1.55

6X24, 7 FIBER CORE	PLOW STEEL	
3/4	16.2 TON	1.25
13/16	19.0 TON	1.47
7/8	21.9 TON	1.69
1"	28.5 TON	2.20

NOTE: SPEC USED FOR TUBAC & MANANA SAWS
6X24, 7 FIBER CORE, 16.2 TON ON 3/4". THESE
SPECS. OBVIOUSLY WRITTEN FROM TIGER BRAND
ENG. REFERENCE.



Flow = 7/10 K

θ	K
≥ 30°	1
20-30°	3/4
0-20°	1/2

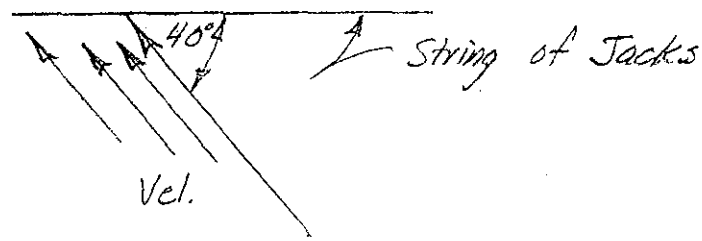
12 Safford Bridge (EWD)

ICC 2-11-85

1

~ Determine the Force on each Jack, given the following:

1.)



2.) Surface area on the plane of a Jack. = 12.5' plane x 6' depth.

3.) Assume Velocity on jacks = 7.5 fps

4.) @ 40° - 60°F ⇒ $\rho = 1.94$ slugs/ft³ → $\nu = 1.0$ ft²/sec

Ref:
1.) SCHAUM'S OUTLINE SERIES; Fluid Mechanics & Hydraulics.
2.) Elem. Fluid Mechanics, John K. Vennard Third Ed., pg. 359

5.) Each Jack leg = 16'

In this design, it will be assumed that the jacks will be covered with debris and essentially act as a sphere displacing the flow. This is assumed due to the Jacks being placed in the river initially.

Big debris (logs, tree branches, etc...) will be assumed to displace or move the jacks, and not impact the jacks as a rigged structure would have to absorb.

It will be assumed that 3/10 of flow will be passed through and 7/10 of the flow will be diverted.

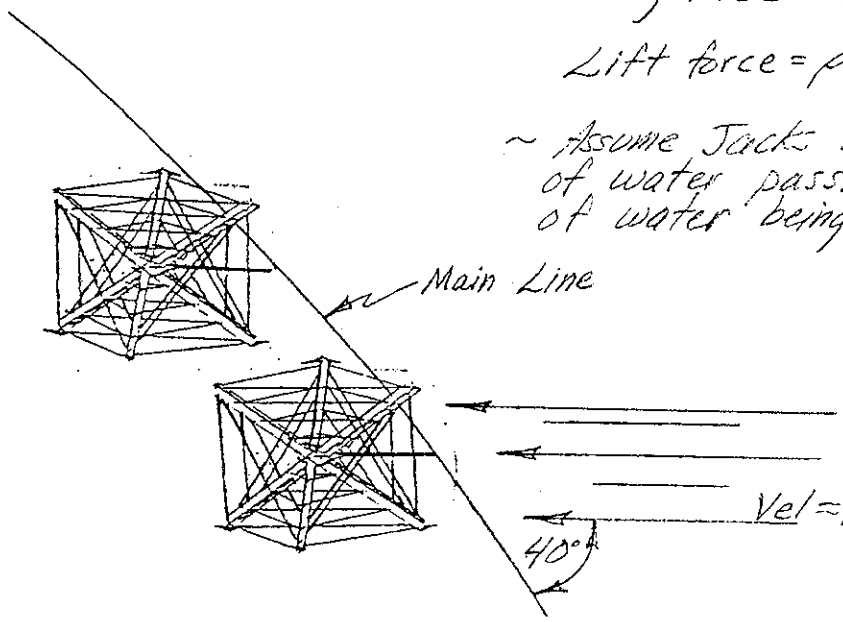
AZ
ICC 2-12-85

Safford Bridge (EWP)

Drag force $= C_D \rho A \frac{\Delta V^2}{2}$; (Ref. 1)

Lift force $= \rho V (2\pi v r)$; (Ref. 2)

~ Assume Jacks as spheres with 3/10 of water passing through and 7/10 of water being diverted



$\Delta V = 10.7 - 3/10(10.7)$
$= 7.5$ fps

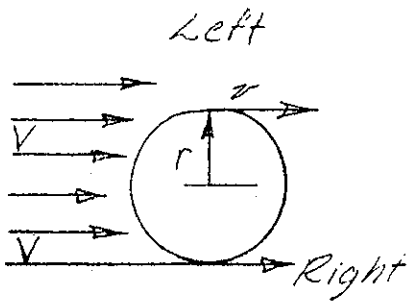
Drag Force

Reynolds Number (RE) $= \frac{\Delta V D}{\nu} = \frac{7.5(16')}{1} = 120$

From Ref. 1, pg. 263 (enclosed) $\Rightarrow C_D = 1.0$ for RE = 320

$\rightarrow \quad F_D = 1.0(1.94)\,12.5'(6')\left(\frac{7.5^2}{2}\right) = 4{,}092^{\#} + 25\%$ IMPACT
$= 5{,}115^{\#}$ PER JACK

Lift Force



$F_L = 2\pi r_e \rho \Delta V^2 = 0$, This is due to the geometry of the sphere.
The transverse force (Lift) is negated when flow is symmetrical about an object.

The flow is evenly distributed and slow enough to consider $V = v$.
($v = 2V\sin\theta$, $\sin 90° = 1$)

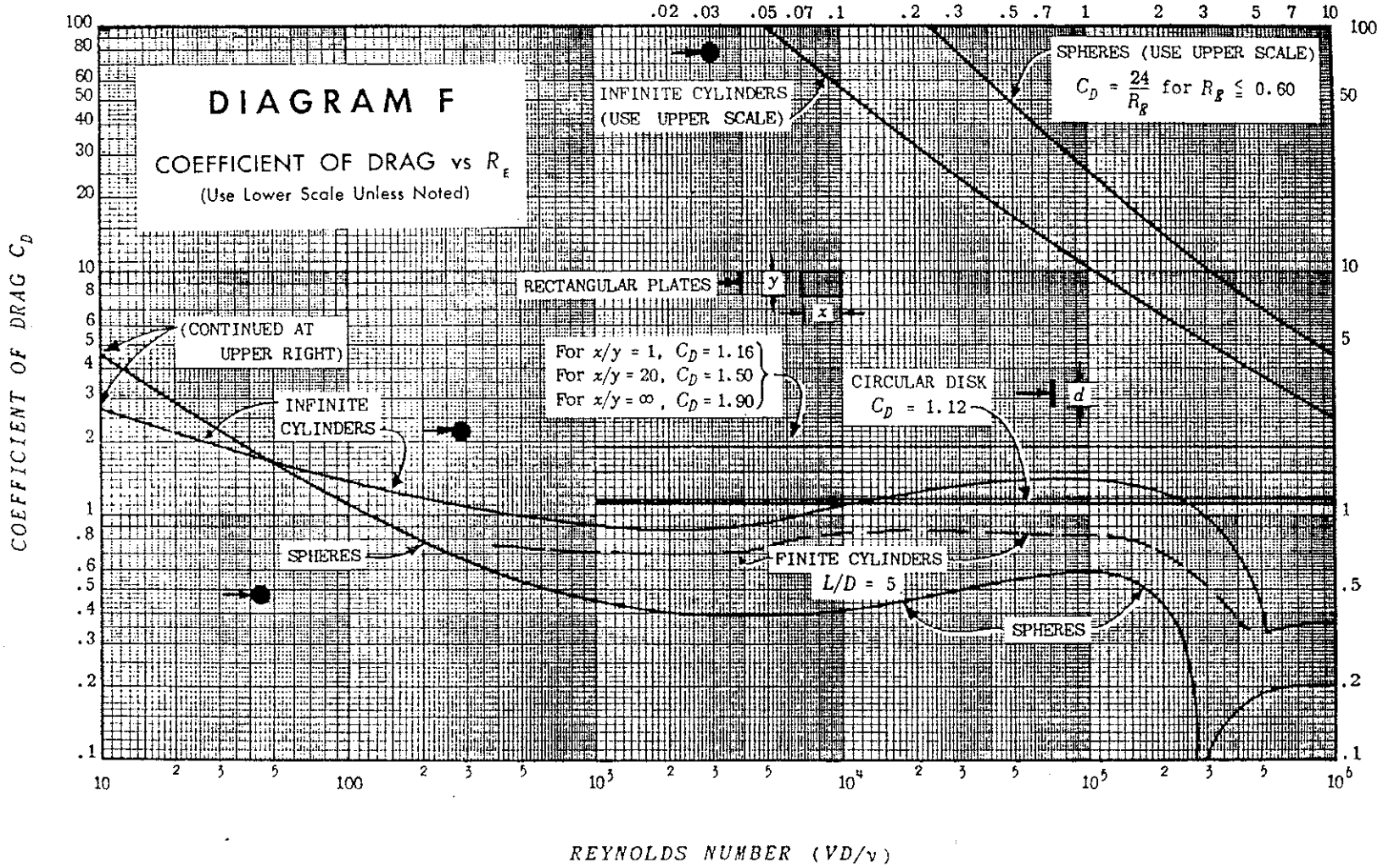
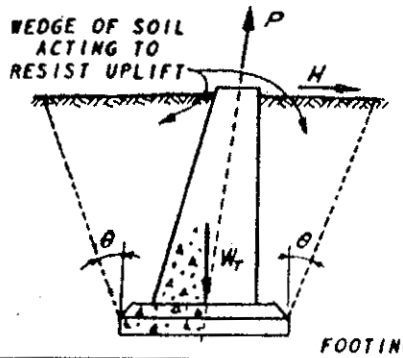


DIAGRAM F
COEFFICIENT OF DRAG vs R_E
(Use Lower Scale Unless Noted)
INFINITE CYLINDERS
(USE UPPER SCALE)
SPHERES (USE UPPER SCALE)
$C_D = \frac{24}{R_g}$ for $R_g \leq 0.60$
RECTANGULAR PLATES
y
x
(CONTINUED AT
UPPER RIGHT)
For $x/y = 1$, $C_D = 1.16$
For $x/y = 20$, $C_D = 1.50$
For $x/y = \infty$, $C_D = 1.90$
CIRCULAR DISK
$C_D = 1.12$
d
INFINITE
CYLINDERS
SPHERES
FINITE CYLINDERS
$L/D = 5$
SPHERES
COEFFICIENT OF DRAG C_D
REYNOLDS NUMBER (VD/ν)

13

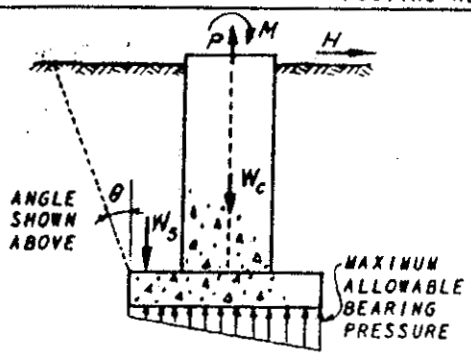


ANGLE θ = 30° FOR COHESIVE SOIL, 20° FOR COHESIONLESS SOIL.

W_T = WEIGHT OF FOOTING PLUS WEDGE OF SOIL ACTING TO RESIST UPLIFT.

SAFETY FACTOR = $\frac{W_T}{P}$, SAFETY FACTOR SHOULD BE NO LESS THAN 1.5 WHERE TRANSIENT LOADS ARE APPLIED.

FOOTING RESISTING LARGE UPLIFT AND SMALL HORIZONTAL LOAD.



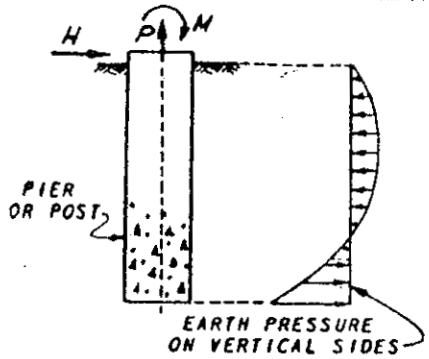
W_s = WEIGHT OF WEDGE OF SOIL ON SIDE OF FOOTING TENDING TO MOVE UPWARD.

W_c = WEIGHT OF FOOTING.

ANALYSIS OF STABILITY AND SOIL PRESSURES SAME AS IN FIG. 10-8. MAXIMUM SOIL PRESSURE ON BASE OF FOOTING IS OBTAINED BY COMBINING W_s W_c, APPLIED LOAD AND MOMENT.

REQUIRED SAFETY FACTOR AGAINST OVERTURNING ≧ 1.5, WHERE TRANSIENT LOADS ARE APPLIED.

FOOTING RESISTING LARGE MOMENT AND SMALL UPLIFT AND HORIZONTAL LOAD.

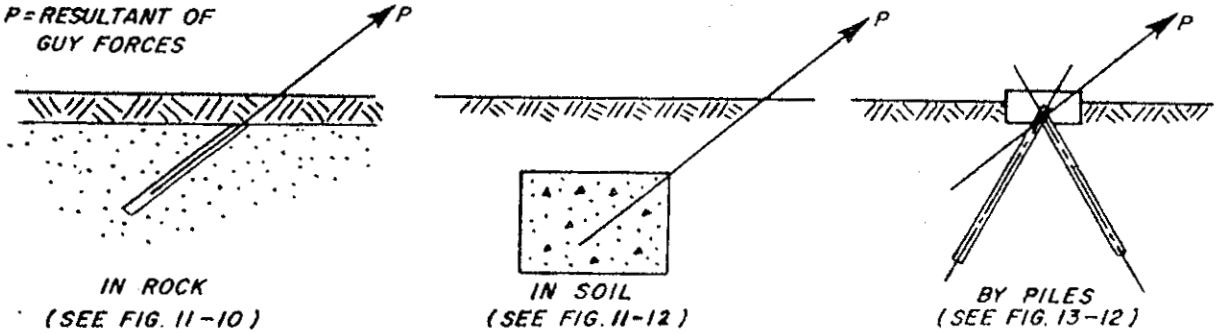


MOMENT IS RESISTED BY EARTH PRESSURE ON SIDES OF PIER OR POST.

FOR ANALYSIS OF STRESS OR DEFLECTION, SEE CASE I, FIG. 13-3.

ALLOWABLE MOMENT ORDINARILY IS LIMITED BY THE TOLERABLE MOVEMENT OF THE FOUNDATION.

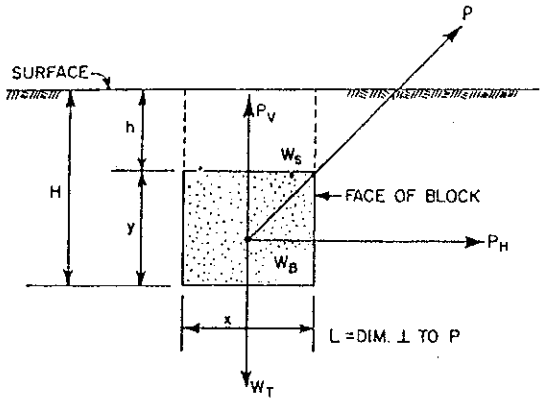
PIER OR POST RESISTING LARGE MOMENT AND SMALL UPLIFT AND HORIZONTAL LOAD.



TOWER GUY ANCHORAGE (SEE SECTION 5)

FIGURE 11-11
Resistance of Footings and Anchorages to Combined Transient Loads

Dept. Navy Design Manual



P = RESULTANT OF MAXIMUM GUY FORCES
P_V, P_H = COMPONENTS OF P
W_T = WEIGHT OF BLOCK + SOIL ON BLOCK
W_B W_S
x, y, L = BLOCK DIMENSIONS
γ = UNIT WEIGHT OF SOIL, p.c.f.
$W_S = x \cdot L \cdot h \cdot \gamma$
P_p = TOTAL PASSIVE PRESSURE LBS/L.F.
ϕ = ANGLE OF INTERNAL FRICTION
C = COHESION, p.s.f.

1. RESISTANCE TO VERTICAL FORCE

SAFETY FACTORS IN VERTICAL DIRECTION :
[USE TOTAL UNIT WEIGHTS ABOVE WATER TABLE,
" BOUYANT " " BELOW " "]

$$\frac{W_T}{P_V} \geq 1.5 \qquad \frac{W_B}{P_V} \geq 1.0$$

2. RESISTANCE TO HORIZONTAL FORCE

SAFETY FACTOR IN HORIZONTAL DIRECTION :

a. SEE SECTION 4, CHAPTER 5 FOR P_p COMPUTATIONS
b. PASSIVE RESISTANCE CONSIDERED ON FACE OF BLOCK (AREA y x L) ONLY.

$$\frac{P_P}{P_H} = 1.5$$

NOTES: BACKFILL SHALL BE COMPACTED AS SPECIFIED IN TABLE 9-3

EXAMPLE:

$\phi = 30°$; C = 0
WATER TABLE AT 5' DEPTH
$P = 40^K$; $P_V = 27^K$; $P_H = 30^K$
γ = 110 pcf, DRY
TRY BLOCK x, y, L = 6', 5', 8'
h = 2'
KEEP P_H AT 1/2 (0.5) TO 2/3 (0.7) BLOCK DEPTH (H) BY VARYING x & y

VERT: W_B (ABOVE W.T.) = 6' x 8' x 3' x 150 pcf = 21,600#
W_B (BELOW W.T.) = 6' x 8' x 2' x 87.5 pcf = 8,400
W_B = 30,000
W_S = 6' x 8' x 2' x 110 pcf W_S = 10,500
W_T = 40,500#

S.F. CHECK $\frac{W_T}{P_V} = \frac{40.5^K}{27^K} = 1.5$ $\frac{W_B}{P_V} = \frac{30^K}{27} = 1.1$

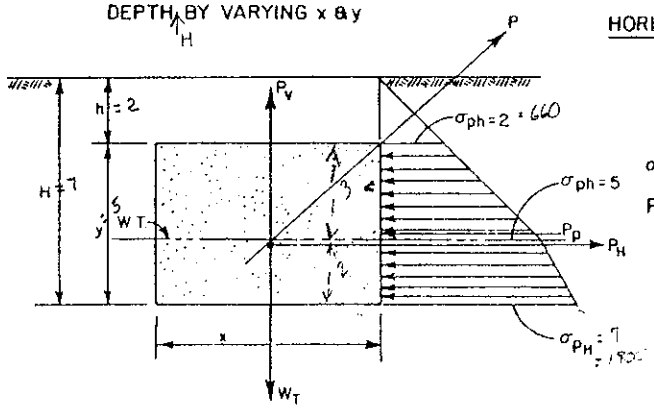
∴ OK VERT.

HORIZ: FROM FIG. 5-12, WITH $\phi = 30°$, $\beta = 0°$: $K_p = 3.0$
FROM FIG. 5-10:

$\sigma_{ph=2} = K_p \gamma h = 3.0 \times 110 \times 2 = 660$
$\sigma_{ph=5} = K_p \gamma h = 3.0 \times 110 \times 5 = 1650$
$\sigma_{PH} = K_p \gamma_{SUB} H + \gamma h_{W.T.} = 3.0 \times 60 \times 7 + 550 = 1800$
$P_p = 1/2 \cdot 3 (660 + 1650) L + 1/2 \cdot 2 (1650 + 1800) L$
$= 3465 L + 3450 L = 6915 \times 8 = 55{,}200$#

$$\frac{P_p}{P_H} = \frac{57^K}{30^K} = 1.83 > 1.5 \text{ S.F.}$$

∴ OK HORIZ.



MAKE ADDITIONAL TRIALS VARYING h, x, y, L

FIGURE 11-12
Tower Guy Anchorage in Soil by Concrete Deadman

Dept. Navy Design Manual

balance larger hol lower pressures can produce more reliable anchors and more rapidly. Additional details on prestressed anchors may be found in PCI (1974); a practical application in Ware et al. (1973).

While proof testing the assembly ensures wall stability, it is still necessary to predict in advance the location of the anchor and its principal dimensions. In general, the slope should be from 15 to not more than 30° from the horizontal so that the vertical component of anchor force on the piling is not significant. A modest slope makes for easier drilling and installation of the anchor. It is common to place the anchor portion outside of the "active" pressure zone (Fig. 13-12) with the remainder of the cable free (but protected against corrosion). This allows the cable to stretch under the prestress load so that when the cable is released from the jack and anchored the resulting prestress losses are not so great that there is no remaining anchor force. These losses from change in elongation Δe are ($\Delta P = AE\,\Delta e/L$) such the remaining force in other than high-strength steel would be quite low and, after further changes in elongation from soil creep, would be unacceptable for anchorage. In permanent installations it might be advisable to fill the remainder of the anchor length with grout for long-term protection after the prestressing has been done.

The tieback anchor resistance can be estimated using Eqs. (4-15) and (4-17) when the base of the anchor is belled. When the base is not belled or the shaft is somewhat variable in diameter as obtained by placing the grout/concrete under pressure, the anchor resistance can be computed (refer to Fig. 13-14*d*) as

$$F_{a.r.} = \pi D \gamma d_2 LK \tan\phi + c_a \pi DL \qquad (13\text{-}5)$$

where c_a is the adhesion of about 0.7 to 0.9c. Use the average shaft diameter for D. L = length of grout and d_2 = average depth of grouted length L. The use of $K = K_o$ can be justified if the concrete (or grout) is placed under pressure, which often the case; otherwise use $K = K_a$. Values of $K > K_o$ are not recommended because of soil creep. This equation is similar to those of Littlejohn (1970); see also Oosterbaan and Gifford (1972).

Deadman Anchors

If the anchor is a deadman and is a long member with a length L considerably greater than the depth and if located so that the depth d_1 (refer to Fig. 13-15 for terms) is less than 0.5 to 0.7d_2, the allowable anchor pull may be computed as

$$F_{a.r.} = \frac{(P_p - P_a)L}{F} \qquad (13\text{-}6)$$

where F is the factor of safety, say 1.25 to 1.5: P_p and P_a are the passive- and active-earth-pressure forces per unit of anchor width computed as $\frac{1}{2}\gamma d_2^2 K_i$. This equation takes into account that as the member increases in length the principal resistance is passive pressure and that the shear on the two ends of the deadman is negligible. It is self-evident that the anchor must be outside the Rankine active-pressure zone. It should also be far enough out that the passive anchor zone does not intersect the Rankine active zone.

If the deadman consists in essentially a short block of $L \le 1.5H$ and embedded in granular soil, the anchor resistance may be computed as

$$F_{a.r.} = \frac{C\gamma d_2^2 LK_p}{F} \qquad (13\text{-}7)$$

with terms also identified in Fig. 13-15. The coefficient C may be taken as 0.5 to 0.65, and K_p is the Rankine value (Table 11-4). Full-scale anchor-block tests by Smith (1957) indicated that $C = 0.65$ is reasonable to account for both passive resistance and end and bottom shear, which on short concrete blocks is considerable. One should use $C = 0.5$ for vertical steel plates. Tests reported by Tschebotarioff (1962) also indicate that $C = 0.65$ is reasonable.

Neely et al. (1973) performed a series of model anchor-plate tests using plates with the H of Fig. 13-15 taken as 2 in and varying the length L. Results were presented in terms of a shape-factor coefficient and a force coefficient. Comparison of that method with Eq. (13-7) by the author for a $\phi = 35°$ and a 2-ft square plate gives 6.1 kips for the Neely method and 6.8 kips from Eq. (13-7). With this small difference and the fact that Neely et al. used models on a single sand of $\phi = f(\gamma)$, Eq. (13-7) is recommended for general use as being simpler.

For cohesive soil ($\phi = 0$) the anchor force can be computed as for a deep footing where the limit of bearing is approximately 9c and guided somewhat by tests of Mackenzie (1955) to obtain the following equation:

$$F_{a.r.} = McHL \qquad (13\text{-}8)$$

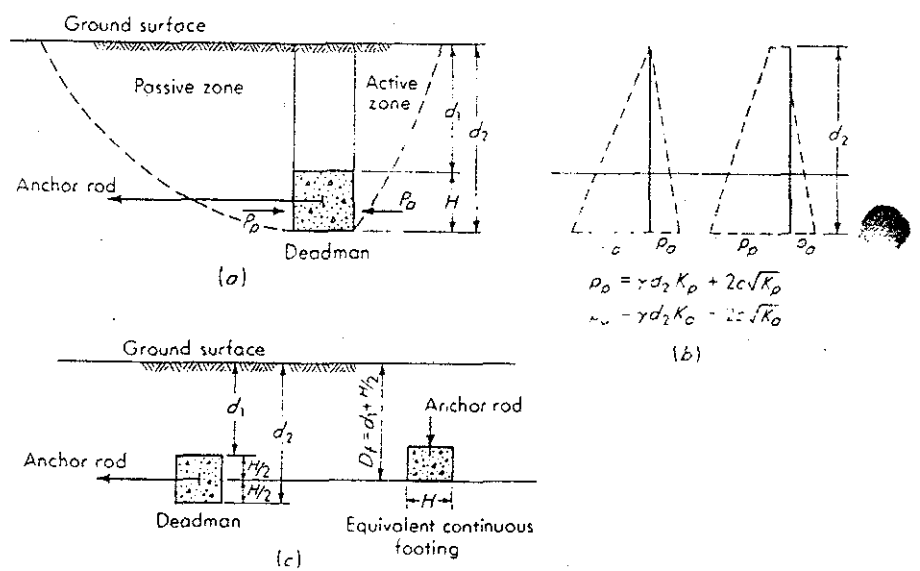


Figure 13-15 Anchor capacity of deadmen. (*a*) Deadman located near surface and $d_1 < 0.5$ to $0.7d_2$; (*b*) earth-pressure diagrams for granular or cohesive soils; (*c*) deadman located at H much smaller than d_2. For this case treat deadman as a continuous footing with D_f as shown. The footing must resist a V = anchor-rod force; use equations given in Chap. 4.

SOIL CONSERVATION SERVICE
ARIZONA

CONSTRUCTION SPECIFICATION

216-12. JACK

1. SCOPE

The work shall consist of the construction and installation of jack revetment for streambank protection.

2. MATERIALS

A. Wooden Jack Legs and Anchors shall be reasonably straight sections from trees found in the local area and approved by the engineer, and shall be the diameter shown on the drawings.

B. Metal Jack Legs shall be structural steel conforming to or exceeding the requirements of ASTM A 36, and shall be the size and shape as shown on the drawings.

C. Wire shall be 6 gage galvanized steel wire as specified or as shown on the drawings. Wires shall have a minimum tensile strength of 58000 pounds per square inch. Wire shall have Class 3 zinc coating as specified in ASTM A 641.

D. Bolts and Clamps shall be galvanized steel.

E. Welding Electrodes shall conform to the requirements of American Welding Society AWS A5.1, "Specification for Mild Steel Covered Arch-Welding Electrodes."

F. Cable shall be new galvanized cable or good condition used cable of type, quality and size as shown on the drawings or as specified in Section 5.

3. JACK INSTALLATION

Jacks, and appurtenances shall be installed in a workmanlike manner to the lines and at the location shown on the drawings or staked in the field.

4. MEASUREMENT AND PAYMENT

For items of work for which specific unit prices are established in the contract, payment for jack revetment will be made at the contract unit price for each jack installed. Such payment will constitute full compensation for all labor, materials, equipment and all other items incidental and necessary to the completion of the work.

5. ITEMS OF WORK AND CONSTRUCTION DETAILS

Items of work to be performed in conformance with this specification and the construction details are:

a. Bid Item 2, Jacks Revetments

(1) This item shall consist of furnishing all materials and labor necessary to install jacks. Materials include the angle iron and wire for the jack, the 1/2" diameter loop cable, and the cable clips to secure the jack and the 3/8" anti-spin leg cable and 3/8" clips.

(2) The 1/2 inch diameter loop cable shall be new galvanized 6 or 7 strand steel with a steel core having a breaking strength not lesss than 9 tons in conformance with the latest revision of Federal Specification RR-W-410.

(3) The 3/8 inch diameter leg cable shall be new galvanized 6 or 7 strand steel with a steel core having a breaking strength not less than 5.5 tons in conformance with the latest revision of Federal Specification RR-W-410.

CONSTRUCTION SPECIFICATION

216-4 MAINLINE CABLE

1. SCOPE

This work shall consist of Laying and installing cable for streambank protection.

2. MATERIALS

A. Cable shall be new galvanized cable of type, quality and size as specified in Section 5.

B. Rope Clips shall be galvanized steel.

C. Thimbles shall be galvanized steel, light duty.

3. CABLE INSTALLATION

Cable, cable clamps, and appurtenances shall be installed with the specified jacks and anchors in a workman-like manner to the lines and location shown on the drawings or staked in the field.

4. MEASUREMENT AND PAYMENT

The length of the 1" diameter cable will be measured to the nearest 1.0-foot along the jack strings. Splices will not be included in measurement and payment. Payment will be made at the contract unit price for the specified length of cable. Such payment will constitute full compensation for all labor, materials, equipment, and all other items necessary and incidental to the completion of work, including rope clips and thimbles.

5. ITEMS OF WORK AND CONSTRUCTION DETAILS

Items of work to be performed in accordance with this specification and construction detail are:

A. Bid Item 3. Mainline Cable

(1) This item shall consist of furnishing and laying of the mainline cable, cable to anchors, clips, and all anchor cable loop attachments, and splices.

(2) The 1" diameter cable shall be new galvanized 6x19 classification extra improved plow steel rope, and with an independent wire rope core having a breaking strength of not less than 45-tons in conformance with the latest revision of Federal Specification RR-W-410.

(3) Both the mainline and cable to anchor require 2 parallel cables.

(4) Limit splices of mainline cable to one (1) in 250-feet.

(5) All cable will be installed with tight loops.

(6) Slack shall be limited to the minimum required for normal construction and tag or dead ends will not be included in measurement and payment.

(7) Pigtail with thimble splices will be allowed at the concrete anchors for ease of construction but extra cable, clips, or thimbles required for pigtails shall not be included in measurement and payment.

CONSTRUCTION SPECIFICATION

216-3 ANCHORS

1. SCOPE

The work shall consist of laying and installing cable for steambank protection.

2. MATERIALS

A. Concrete anchors shall be constructed with Type II or Type V cement, shall contain a minimum of 6 bags of cement per cubic yard of concrete, water-cement ratio of 0.5 or less, and a 3/4" maximum nominal size of coarse aggregate. Mixes that include fly ash shall be based on absolute volume with a maximum substitution of 20 percent. Fly ash shall be Class F. The source of fly ash shall be identified by the contractor and approved by the engineer.

3. REINFORCEMENT

Before reinforcement is placed, the surfaces of the bars and metal supports shall be cleaned to remove any loose mill scale, oil grease, or foreign substances. Reinforcement shall be accurately placed and secured in position in manner that will prevent its displacement during the placement of the concrete.

4. EXCAVATION

Trench excavation for anchor cable and anchor blocks shall be to the lines and grades shown on the drawings and as staked in the field. Trenches deeper than 5 feet shall be sloped on a 1 to 1 above the 5-foot level.

5. BACKFILL

The trench for anchor cable and anchor blocks shall be backfilled with suitable material obtained from the excavation. Fill shall be compacted to a minimum of three feet above the anchor cable and the anchor block. Moisture content of fill material shall be sufficient to obtain the required density. Fill shall be compacted to a density equivalent to that of the material in the trench side walls by means of hand tamping, manually directed power tampers, or plate vibrators. The remainder of the trench shall be backfilled with random backfill from the excavation, or from other sources approved by the Engineer. Mounding shall be as required to approximate natural ground surface after settling.

6. INSTALLATION

Anchors and appurtenances shall be installed in a workmanlike manner to the lines and at the location shown on the drawings or staked in the field.

7. MEASUREMENT AND PAYMENT

For items of work for which specific unit prices are established in the contract, payment for anchors will be made at the contract unit price for each anchor installed. Such payment will constitute full compensation for all labor, materials, equipment and all other items incidental and necessary to the completion of the work, including, anchors, excavation and backfill, cable, clips and thimbles.

Compensation for any item of work described in the contract but not listed in the bid schedule will be included in the payment for the item of work to which it is made subsidiary. Such items and the items to which they are made subsidiary are identified in Section 8 of this specification.

8. ITEMS OF WORK AND CONSTRUCTION DETAILS

Items of work to be performed in conformance with this specification and the construction details are:

a. Bid Item 4,- Anchors

(1) This item shall consist of furnishing all materials and labor necessary to construct and install concrete anchors. Materials for each anchor includes the concrete, reinforcement steel, 1" diameter cable, 1" cable clips, and 1" thimbles.

(2) The 1" cable shall be new galvanized 6x19 classification improved plow steel, with an independent wire rope core, having a breaking strength of not less than 40 tons in conformance with the latest revision of Federal Specification RR-W-410.

(3) Cable clips and thimbles shall be as specified in Specification No. 216-4.

(4) Concrete anchors shall be moist cured or covered with a liquid curing compound for 7 days before placement. Anchors shall be lifted and placed with slings. Concrete anchors shall be covered with a liquid curing compound before being buried.

(5) The 1" diameter cable within the concrete anchor shall be cleaned of oil, grease, or other contaminants before the cable is installed within the concrete.

(6) Any required diversion of the river or dewatering for construction shall be included in the contract price for concrete anchors. There will not be a separate payment for River Diversion or Dewatering.

U.S. DEPARTMENT OF AGRICULTURE

SOIL CONSERVATION SERVICE

ARIZONA

OPERATION AND MAINTENANCE GUIDELINES

580 STREAMBANK PROTECTION - KELLNER JACKS

For: Job _____

Sponsor/Cooperator _____

County _____

Location _____

Introduction

The success or failure of installed works to fulfill their function over time is dependent upon the thoroughness and timeliness of maintenance. Even small problems or conditions that are left unattended may become serious resulting in hazardous and/or expensive failures.

MAINTENANCE

1. Inspect annually and after every major storm. Check for any signs of failure.

2. Steel legged Kellner Jacks are relatively maintenance free. Usually, the greatest need for maintenance is for the vegetation within the jetty field. However, vandals can remove steel angle iron, cable, and fastners. Impact from massive floating debris can bend jack legs and break wires. This is not usually a serious matter, but, the effect of a loose jetty line caused by broken cables or displaced anchors can be a very serious matter. Salvaging such a failure can be costly. Replacement may be the proper action in many instances.

3. Inspect individual jacks for bent legs or broken lacing wires. Replace wires as necessary.

4. Inspect for broken cables or dislodged anchors. Repair cables, reset anchors, or install new jetty lines as necessary.

5. Inspect the riverbank, particulary at the upstream end of the line and at all anchors to determine if the jetty line is in danger of becoming ineffective from bank erosion or, if anchors are in danger of being washed out. Take corrective action as required.

U.S. DEPARTMENT OF AGRICULTURE

SOIL CONSERVATION SERVICE

ARIZONA

OPERATION AND MAINTENANCE GUIDELINES

322 CHANNEL VEGETATION

For: Job _____

Sponsor/Cooperator _____

County _____

Location _____

Introduction

The success or failure of installed works to fulfill their function over time is dependent upon the thoroughness and timeliness of maintenance. Even small problems or conditions that are left unattended may become serious resulting in hazardous and/or expensive failures.

MAINTENANCE

1. Inspect annually and after every major storm. Check for dead, stressed, or damaged trees.

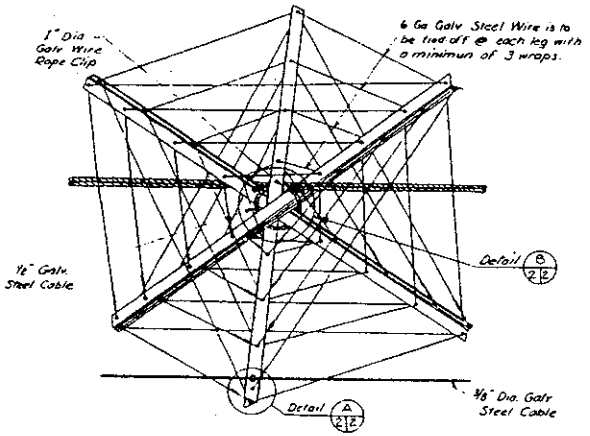
2. Protect the plantings by controlling rodents, beavers, livestock, fire, and human access until plantings are well established.

3. Control competitive weeds as needed.

4. Reseed or replant, along with the use of fertilizers and/or soil amendments and irrigation, as needed.

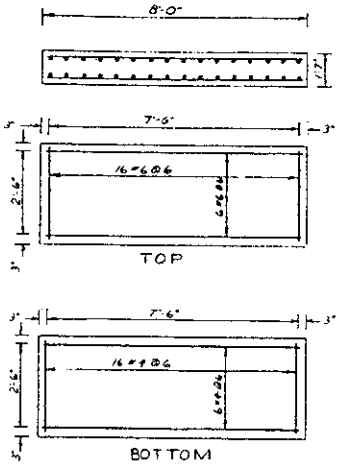
5. Repair appurtenances and fences.

3/8" Cable for Heavy Jacks. Lace through 3 Legs and secure with one 3/8" Clip. Requires 16-3/8" Clips per Jack.

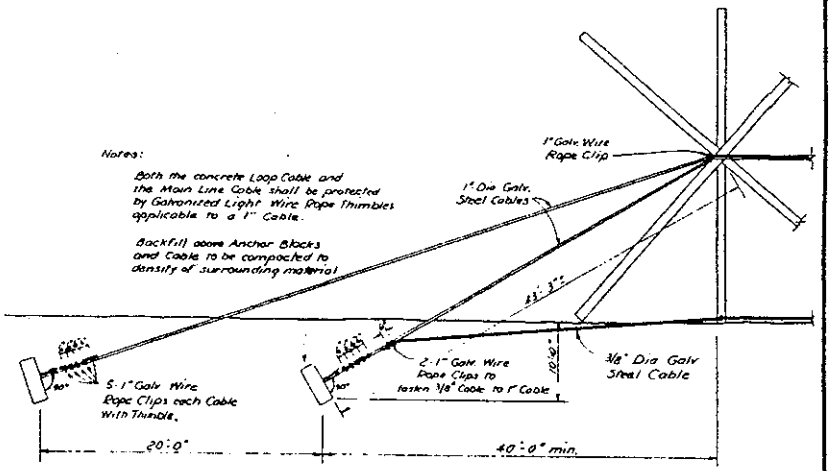


KELLNER JACK

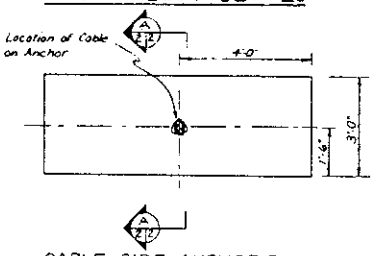
1. Heavy Jacks - Unit is 16'x4"x4"x1/4" Angle Iron with Welded or Bolted Connection, laced with No. 6 Galvanized Wire (3/8" Cable for Outer Lace).
2. Light Jacks - Unit is 16'x3"x3"x1/4" Angle Iron with welded or bolted Connection, laced with No. 6 Galvanized Wire
3. Jacks are spaced on 12½' centers
4. Jacks placed on the Main Line and Side String shall alternate between cables.
5. All Main Line Cables, including Side Strings, shall be clipped together between every fourth and fifth Jack



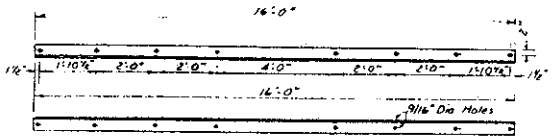
ANCHOR BLOCK DETAILS



ANCHOR PLACEMENT DETAILS

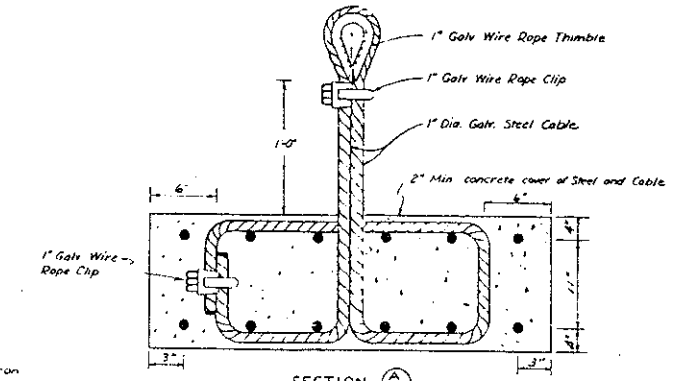


CABLE SIDE ANCHOR BLOCK



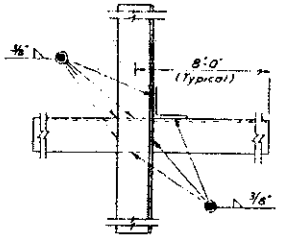
DETAILS TYPICAL KELLNER JACK LEG

(3 REQUIRED EACH JACK)

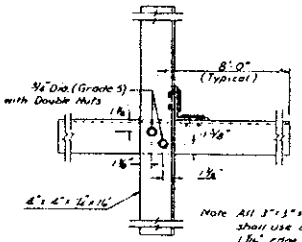


SECTION A 2/2

CABLE TO ANCHOR BLOCK DETAIL

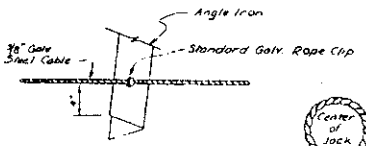


DETAILS WELDED JACK LEG ASSEMBLY

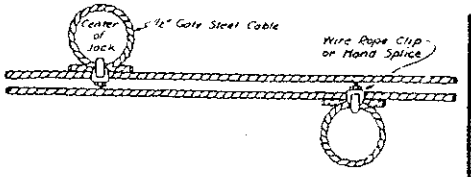


Note All 3"x3"x1/4"x1/16" Angle irons shall use a 1/2" dia bolt with a 1 3/16" edge distance to the center of the bolt from two edges.

DETAILS BOLTED JACK LEG ASSEMBLY



DETAIL A 2/2



DETAIL B 2/2

JACK DETAILS
SHEET 2 OF 2 SHEETS

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

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Title:
Date:
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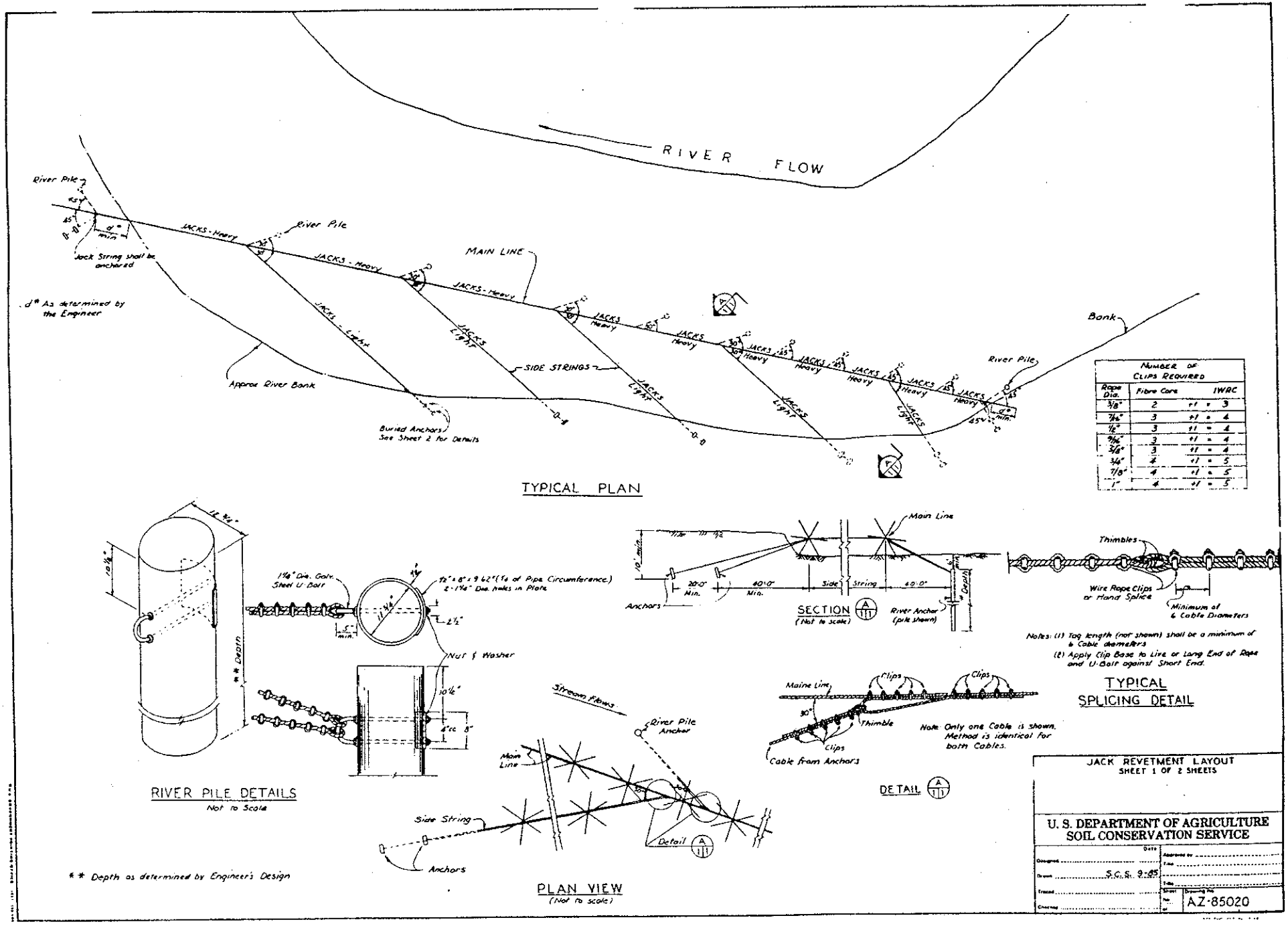
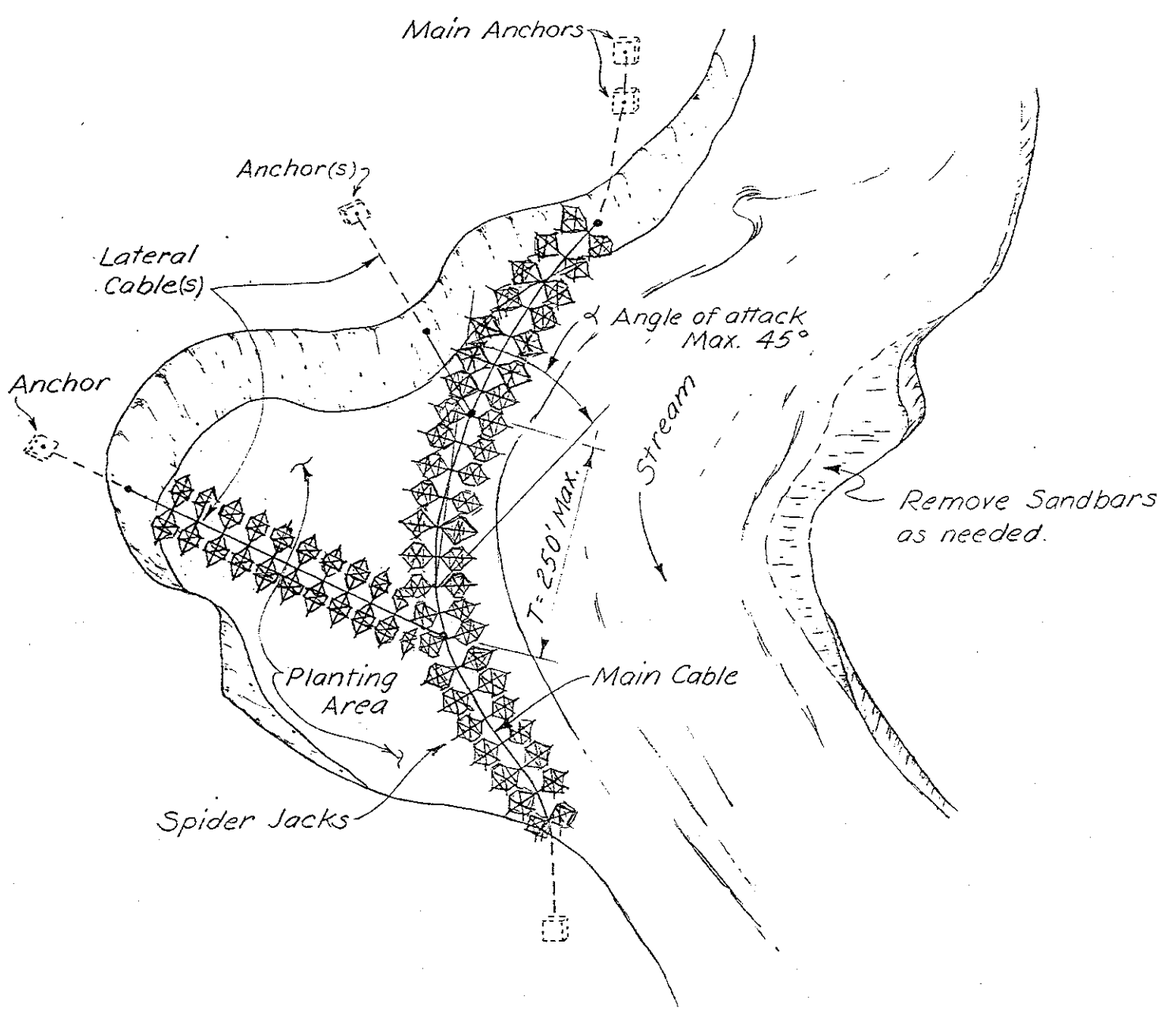

RIVER FLOW
River Pile
Jack String shall be anchored
d* As determined by the Engineer
JACKS - Heavy
JACKS - Light
MAIN LINE
SIDE STRINGS
Approx River Bank
Buried Anchors See Sheet 2 for Details
Bank
TYPICAL PLAN
NUMBER OF CLIPS REQUIRED
Rope Dia. | Fibre Core | IWRC
3/8" | 2 | +1 = 3
7/16" | 3 | +1 = 4
1/2" | 3 | +1 = 4
9/16" | 3 | +1 = 4
5/8" | 3 | +1 = 4
3/4" | 4 | +1 = 5
7/8" | 4 | +1 = 5
1" | 4 | +1 = 5
1 1/4" Dia. Galv. Steel U-Bolt
Nut & Washer
** Depth
RIVER PILE DETAILS
Not to Scale
Main Line
Anchors
20'-0" Min.
40'-0" Min.
Side String
40'-0"
Depth
SECTION
(Not to scale)
River Anchor (pile shown)
Thimbles
Wire Rope Clips or Hand Splice
Minimum of 6 Cable Diameters
Notes: (1) Tag length (not shown) shall be a minimum of 6 Cable diameters
(2) Apply Clip Base to Live or Long End of Rope and U-Bolt against Short End.
TYPICAL SPLICING DETAIL
Stream Flows
River Pile Anchor
Side String
Detail
PLAN VIEW
(Not to scale)
Maine Line
Clips
Thimble
Cable from Anchors
Note: Only one Cable is shown. Method is identical for both Cables.
DETAIL
** Depth as determined by Engineer's Design
JACK REVETMENT LAYOUT
SHEET 1 OF 2 SHEETS
U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
S.C.S. 9-85
AZ-85020



PLAN VIEW
(Not to Scale)

Note:
Tree plantings shall be willow, cottonwood or other species as specified. Plantings shall be fenced if livestock is present.

Velocity < 10 fps prefer < 5 fps
depth < 5 ft.
low impact area (debris)
Anchors
must have vegetation
best in wide channels

ESTIMATED QUANTITIES

Spider Jacks	_____	No.
Main Cable*	_____	dia.
Main Cable*	_____	length
Lateral Cable*	_____	dia.
Lateral Cable*	_____	length
Main Anchors	_____	No.
Lateral Anchors	_____	No.
"T" Distance	_____	ft.
Clips	_____	No.
Fence	_____	length

* Used cable may be installed if approved by the Engineer. Strength of lateral cables shall not be less than 50% of the strength of the main cable.

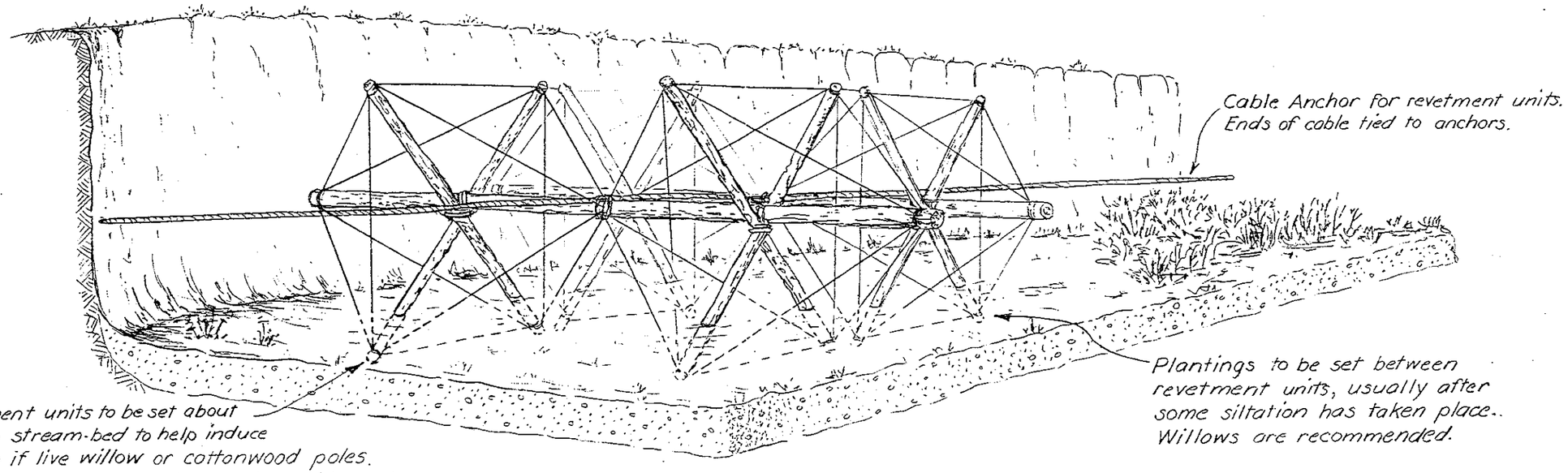
Cooperator _____ Date _____

SPIDER JACKS
(MULTIPLE UNITS)

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

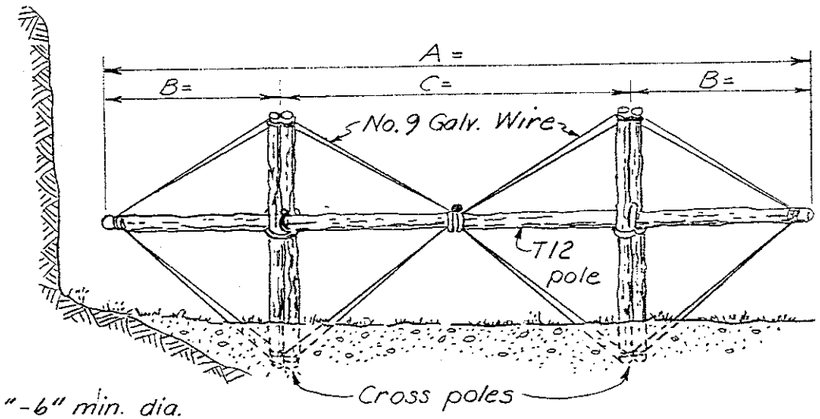
Designed W.K.A., J.W., S.J.	Date 10-86	Approved John Weaver	
Drawn EFS	10-86	Title Asst. State Cons. Eng. Nov., 1986	
Traced		Title	
Checked		Sheet No. 1 of 3	Drawing No. 87005-AZ

SCS-ENG 313A REV. 8-75



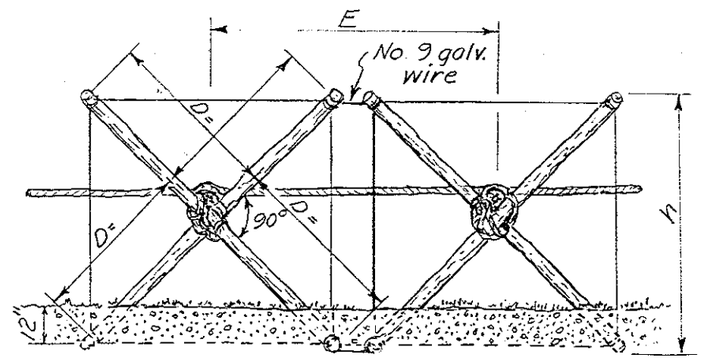
ISOMETRIC

_____ Lineal feet of No. 9 galv. wire required per unit.



Cross Poles 4"-6" min. dia.
T12 Poles 6"-8" min. dia.

SIDE VIEW



END VIEW

NOTE:
These Sketches show Live Jacks. Juniper or Oak Jacks are to be set on Ground Line.

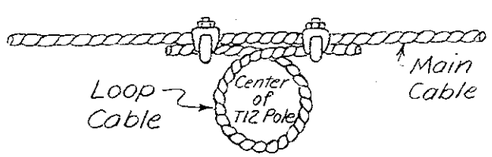
$E = 1.414\,(h+1)$

$D = \frac{1.414\,(h+1)}{2}$

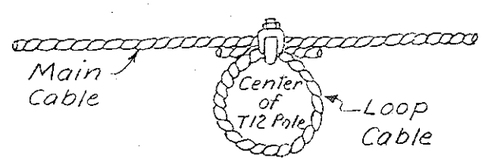
$B = D$
$C = 2D$
$A = 4D$

ALTERNATE METHODS OF SECURING T12 POLE TO MAIN CABLE

METHOD No. 1 - Loop Cable & Main Cable are the same diameter.



METHOD No. 2 - Loop Cable is 1/2 the diameter of the Main Cable.



Cooperator _____ Date _____

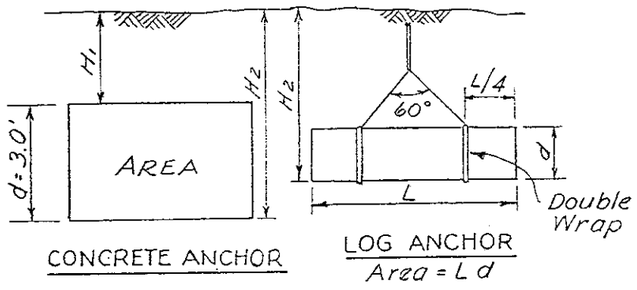
SPIDER JACKS
(MULTIPLE UNITS)

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

Designed W.K.A., J.W., S.J.	Date 10-86	Approved by	
Drawn E.F.S.	10-86	Title	
Traced		Title	
Checked		Sheet No. 2 of 3	Drawing No. 87005-AZ

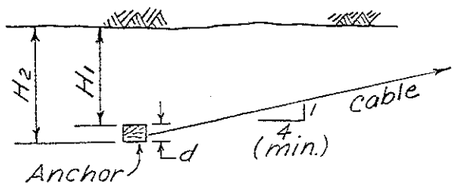
SCS-ENG 313A REV 8 75

ANCHORS



CONCRETE ANCHOR

LOG ANCHOR
Area = L d



ANCHOR RESISTANCE PR (TONS)
$P_R = 0.0734\ (2H_2 - d)(d)(L)$

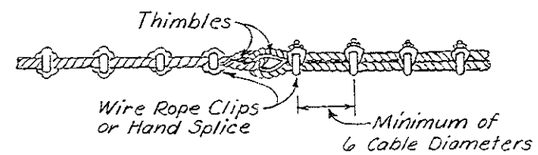
1. Size anchors to match breaking strength of cable.
2. Formula for computing anchor resistance assumes dry, non-cohesive soil, 4:1 or flatter angle of pull. Increase anchor requirements if anchor installed is below water table.
3. Locate anchor a minimum of 20ft. away from bank edge and 20ft. minimum distance between anchors.
4. Concrete shall be 6 bag mix, Type II or V Cement.
5. Backfill above anchors and cables shall be compacted to the same density as the surrounding ground.

BREAKING STRENGTH - NEW CABLE

STANDARD FLEXIBLE HOISTING ROPE
6 x 19 Classification
Fiber Core or IWRC 1/

Improved Plow Steel Fiber Core Dia.	Improved Plow Steel Fiber Core Tons	Improved Plow Steel IWRC Tons	Extra Improved Plow Steel IWRC Tons
3/16"	1.55	1.60	
1/4"	2.74	2.94	3.40
5/16"	4.26	4.50	5.27
3/8"	6.10	6.56	7.55
7/16"	8.27	8.89	10.20
1/2"	10.70	11.50	13.30
9/16"	13.50	14.50	16.80
5/8"	16.70	17.90	20.60
3/4"	23.80	25.60	29.40
7/8"	32.20	34.60	39.80
1"	41.80	44.90	51.70
1 1/8"	52.60	56.50	65.00
1 1/4"	64.60	69.40	79.90

1/ IWRC = Independent Wire Rope Core
Reduce Strengths 10% for Galvanizing
Reduce Strengths 20% for Thimble Splices
All Cable, Wire, Thimbles & Clips shall be galvanized.



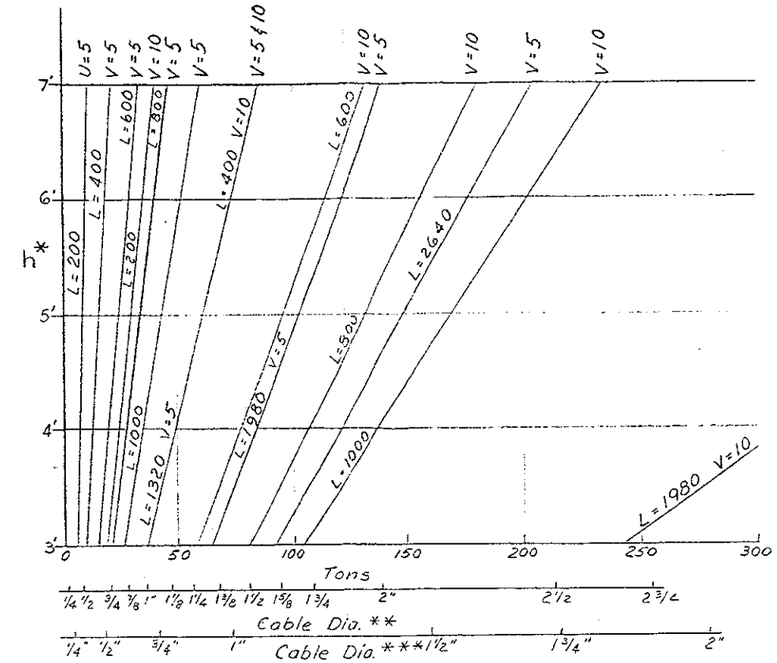
Notes:
1. Top length (not shown) shall be a minimum of 6 Cable Diameters
2. Apply Clip Base to Live or Long End of Rope and U-Bolt against Short End.

SIZE OF CABLE	1 1/4"	1 1/8"	1"	7/8"	3/4"	5/8"	1/2"	3/8"
No. OF CLIPS	5	5	4	4	3	3	3	2

CABLE CLIPS

One additional clip should be attached when using a rope having an IWRC or strand core.

TYPICAL SPLICING DETAIL

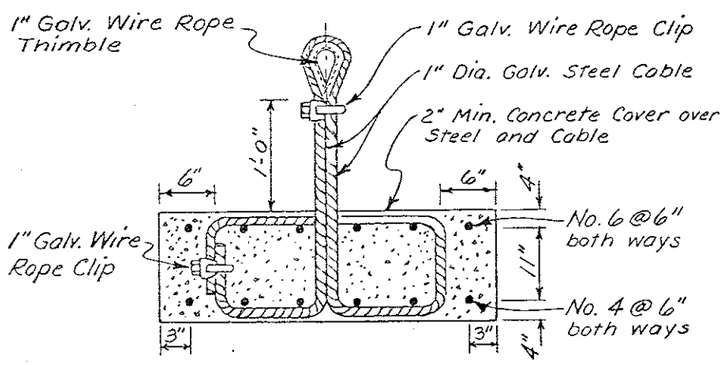


MAINLINE CABLE SIZE

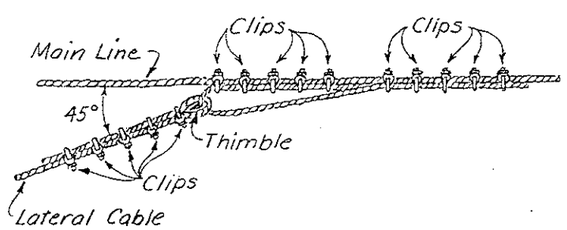
* Water depth
** Single rope, 6x19 Std. Hoist, plow steel.
*** Two ropes, 6x19 Std. Hoist, plow steel.

DESIGN DATA

Drainage Area _____
Bankfull Discharge _____
Channel Depth (Low Bank) _____
Job Class _____
Velocity (V) _____
Length of Protection (L) _____
Cable Size: Main _____
Lateral _____



CONCRETE ANCHOR DETAIL



LATERAL CONNECTION DETAIL

Cooperator _____ Date _____

DESIGN CRITERIA
SPIDER JACKS
(MULTPLE UNITS)

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

Designed W.K.A., J.W., S.J.	Date 10-86	Approved by	
Drawn E.F.S.	10-86	Title	
Traced		Title	
Checked		Sheet No. 3 of 3	Drawing No. 87005-AZ

SCS-ENG-21 Rev. 7-76

UNITED STATES DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
ARIZONA

SPECIFICATION

STREAMBANK PROTECTION
(Spider Jacks)

1. Scope

The work shall consist of furnishing and installing materials as required to construct spider jack revetment for bank protection.

2. General

Measures and construction methods that enhance fish and wildlife values shall be incorporated as needed and practical. Special attention shall be given to protecting and maintaining key shade, food, den trees and visual resources and to stabilizing disturbed areas.

3. Materials

a. Wood Jack Legs and Anchors shall be reasonably straight sections from trees found in the local area and approved by the engineer, and shall be the diameter shown on the drawings.

b. Wire shall be 9 gage medium temper galvanized zinc coating steel wire as specified in ASTM A 641. Wire shall have Class 3 zinc coating as specified in ASTM A 641.

c. Cable shall be new galvanized cable or good condition used cable of type, quality and size as shown on the drawings. The engineer shall approve all used cable.

d. Rope clips and thimbles shall be galvanized steel.

e. Concrete anchors shall be constructed with Type II or Type V cement, shall contain a minimum of 6 bags of cement per cubic yard of concrete, water-cement ratio of 0.5 or less, and a 3/4" maximum nominal size of coarse aggregate. Mixes that include fly ash shall be based on absolute volume with a maximum substitution of 20 percent. Fly ash shall be Class F. The source of fly ash shall be identified by the contractor and approved by the engineer.

4. Reinforcement

Before reinforcement is placed, the surfaces of the bars and metal supports shall be cleaned to remove any loose mill scale, oil, grease, or

2/7/90

RAILROAD CAR DEFLECTING JETTIES

SCS INSTALLED GRAHAM CANAL DEFLECTING JETTIES PROJECT IN 1986 ON THE GILA RIVER AT SAFFORD, ARIZONA

PROJECT CONSISTS OF 26 JETTIE UNITS PROTECTING 2200 ft. of BANK.

TOTAL MATERIALS INSTALLED:

ITEM		NO
RAILS (PILES)	39' Length 131#	388
PIGGYBACK CARS	89.5' Length	20
FLATCARS	x 40' Length	4
ORE CARS	x 34' Length	8

RAILS & CARS WERE PURCHASED ON MATERIAL CONTRACT, SPONSORS DONATED RAILROAD CARS FOR THEIR 20% SHARE OF TOTAL PROJECT COSTS. TOTAL COSTS = $225,604 WHICH IS ABOUT $101.63 PER FT. OF BANK PROTECTED.

(SEE COST SUMMARY, NEXT PAGE)

DESIGN REPORT

Graham Canal - City of Safford

Deflecting Jetties

GENERAL

This job proposes the use of locally available railroad cars and rail to construct deflecting jetties. It is a copy of a highly successful deflecting jettie job (Wastewater Pond) which is located about 1 1/2 miles downstream of the Graham Canal site.

LOCATION

The proposed job is located on the north bank of the Gila River about one mile upstream of the Safford 8th Avenue Bridge. The previously WNTC reviewed Safford 8th Avenue Kellner Jack job is located just upstream of the Bridge but on the south bank. The downstream end of the jetties is about even, but on the opposite bank, with the upstream end of the Kellner Jacks. The installed Wastewater Pond Jetties are about 1/4 miles further downstream of the bridge on the south bank.

Aerial Photograph

Aerials obtained and reviewed for this job are:

Date	Scale	Comment
6/17/73	1" = 660	
2/13/79	1" = 1,000	After 1979 flood
4/29/82	1" = 1,320	
11/29/83	1" = 500	After October 83 flood 140,000 cfs
2/8/85	1" = 1,000	After December 84 flood 50,000 cfs

Partial History

The 6/17/73 photography shows the river on a straight alignment with about 500' of bank between the canal and the river. The 2/13/79 photography shows massive removal of bank material with the bank adjacent to the canal. It is this writers understanding that the canal was moved northward by the owners about 20-30 feet because of bank erosion. In 1979 or 1980, the Canal Company installed a driven rail deflector jettie with wire and rock at the mouth of Lone Star Wash, which is just upstream of the Project Site. The October 1983 flood damaged this jettie; all of the rock was lost out of the bottom by scour, but the rails are still in place and catch debris.

Bank erosion by the large flood of October 1983 is not known by this writer at this time. The post flood photography of 11/29/83 shows the bank to be intact. The smaller flood of December 1984 caused substantial bank erosion. The bank is now within about 6' of the canal and a Canal Company

well located further downstream is in the river bed. In the spring of 1985, the Canal Company partially constructed another jettie about 400' d/s of Lone Star Wash by driving about 30 rails in a straight line from the bank extending about 450' out into the river. Willows were planted behind these rails. The rails are presently bare; no cable, whalers, or wire (see photos 1-6).

Literature Review

The literature reviewed contain some reference to deflecting jetties but none of the type proposed for installation at this site. References reviwed and pertinent extracts are included in the attached material.

Wastewater Pond Deflecting Jettie Project

In 1980, following the 1979 flood, the City of Safford installed 6 railroad flatcar jetties downstream of the Safford Bridge. The cars were about 60' long with 50' spacing between the cars. Rails were driven 29-30 feet. Rails used were 119# and 131#, spaced at 6-7 feet on both sides of the jettie. This was local construction, without formal plans or analysis of forces.

This job has been successful. It withstood the large 140,000 cfs flow of October 1983 without damage or bank erosion. Some of the cars have settled 4-5 feet. The area behind the cars has completed silted in and supports dense stands of trees. (See Photos 7-12).

Soil Logs at Project Site

Boring were not taken. Soil conditions are assumed to be the same as for the Safford 8th Avenue Kellner Jack job.

Design

Loads were computed by the momentum equation with a 15° angle of attack and 10.0 ft./s velocity, and increased 25% for impact.

Pile resistance was computed using the same method as for the piles for the Safford 8th Avenue Bridge job assuming an 8' high car and 6' of scour. The design calls for 9'-10' car, however, scour will not be a factor.

Type of Cars & Placement

There may be some adjustment required in the type of cars actually installed since the Sponsors do not own all of the necessary cars, although there are sufficient cars available in the Safford Valley.

Factors of Safety

The computed factor of safety for bending of the piles in moment is 2:07:1, which compares with a computed factor of safety of 2.22:1 for the installed Wastewater Jetties Project assuming the same load conditions for each. The Graham Canal Project will use stronger piles but at a greater spacing. The factor of safety for soil resistance on the piles is 5.9:1.

Summary

The jetties, at a cost of about $85.00 per L.F., are cost competitive with other types of structural bank protection. The Kellner Jacks at Safford 8th Avenue Bridge cost $113.00 per L.F. A similar installation near the project site has performed well.

John E. Weaver
Assistant State Conservation Engineer

DESIGN CRITERIA

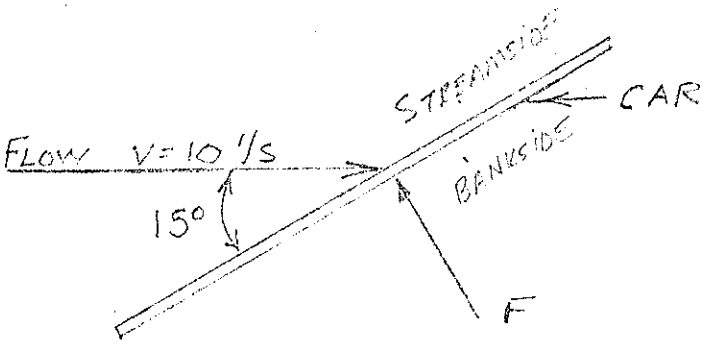
1. ALIGNMENT ON EASY CURVE
2. JETTIES AT 15° ANGLE WITH STREAMFLOW
3. 50' SPACING BETWEEN JETTIES
4. JETTIES SHALL BE LOOSELY INSTALLED BETWEEN THE PILES SO THAT THE JETTIES CAN SETTLE IF THEY ARE UNDERCUT BY CHANNEL SCOUR
5. MINIMUM JETTIE LENGTH ≈ 50 ft.

TYPE OF RAILCAR	LENGTH
PIGGYBACKS	89.5'
FLATCARS	50.6 & 53.7
BOX CARS	57
ORE CARS	33.5 & 34.5

DESIGN LOAD - STREAMSIDE

ASSUME

1. VELOCITY 10.0 FT. SEC.
2. ANGLE OF ATTACK 15°
3. NEGLECT FRICTION ON FLAT CAR SURFACE
4. CAR 8' HEIGHT
5. IMPACT LOAD + 25%



ASSUME FIXED VANE. FOR THE MOMENTUM EQUATION NORMAL TO THE PLATE.

$$F = \frac{\gamma}{g} V^2 \sin\theta$$

$$F = \frac{62.4}{32.2}(10^2)(\sin 15°)$$

$$F = 50.1 \text{ psf} \times 8' \text{ ht} = 401.25 \text{ plf}$$

+ 25% IMPACT

$$F = 501.56 \text{ plf}$$

COMPUTATION SHEET
SCS-ENG-522 REV. 3-69

U.S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

STATE		PROJECT GRAHAM CANAL		
BY	DATE	CHECKED BY	DATE	JOB NO.
SUBJECT PILES - BANKSIDE				SHEET ____ OF ____

ASSUME BEAM LOADING

STEEL - YIELD STRENGTH = 72,000 PSI

F.S. = 2.1

USE $f = 38{,}000$ PSI = 38 KSI

$M = \frac{fI}{c}$ where M = BENDING MOMENT

f = NORMAL UNIT STRESS

I = MOMENT OF INERTIA

C = DISTANCE FROM EXTREME FIBER TO NEUTRAL AXIS

SECTION MODULUS $S_x = \frac{I}{c}$

$\therefore M = \frac{f S_x c}{c} = \frac{f S_x}{12}$

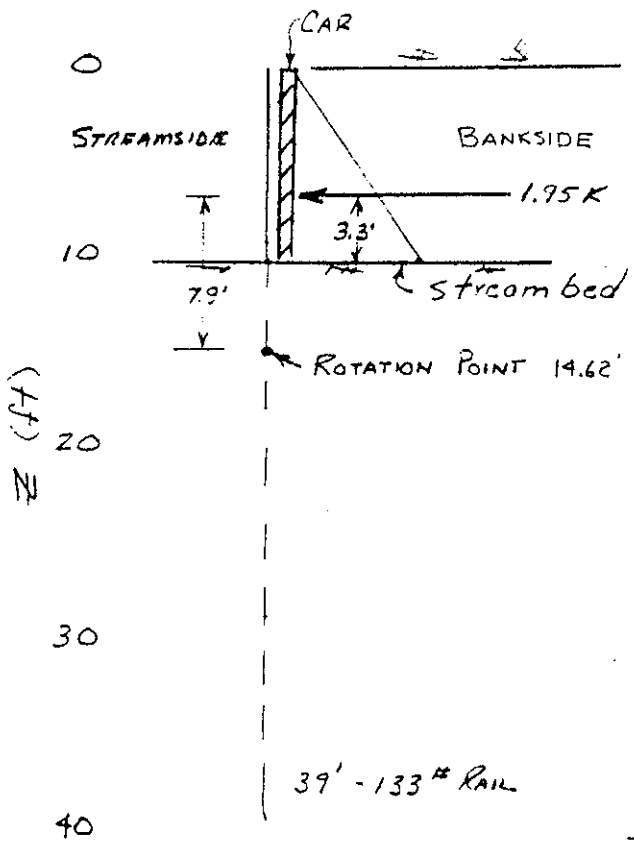
$M = \frac{38 S_x}{12}$

COMPUTATION SHEET
SCS-ENG-523 Rev. 8-69

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

STATE		PROJECT		
ARIZ		GRAHAM CANAL		
BY J.W.	DATE 8-13-85	CHECKED BY	DATE	JOB NO.
SUBJECT PILES STREAMSIDE				SHEET 11 OF

FIND THE NUMBER OF RAILS (PILES) NEEDED ON THE STREAMSIDE TO HOLD SOIL PRESSURE.



Assumptions

1. River is dry, no water loads
2. 10' ht. of soil on bankside
3. Silty sand, medium density
4. Moist Soil

$\gamma = 130\#/ft^3$ TABLE 2.1 FAR WEST

$K_a = 0.3$ TABLE 2.1 FAR WEST

$$P = \frac{K_a \gamma h^2}{2}$$

$$P = \frac{(0.3)(130)(10^2)}{2}$$

$P = 1{,}950$ # PER FT. OF CAR

MOMENTS

@ Z = 10' (bottom of car)

$M = (1{,}950\#)(3.3') = 6{,}435$ # per FT OF CAR

@ Z = 14.2' (ROTATION POINT)

$M = (1950)(7.9') = 15.4$ K-FT PER FT OF CAR

ASSUME 119# RAIL
EACH PILE WILL RESIST 72.5K-FT (WITH 6' OF SCOUR)

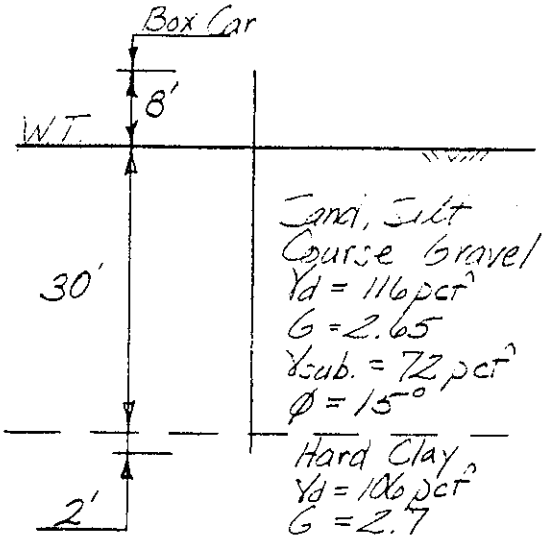
No. OF PILES NEEDED $= \frac{15.4}{72.5} = 0.2$ PILES Use 3 each car

STATE		PROJECT		
AZ		Safford Graham Canal		
BY	DATE	CHECKED BY	DATE	JOB NO.
TCC	7-12-85			
SUBJECT				SHEET 2 OF
Design Soil Profile				

Ref.

1.) SCHAUM'S OUTLINE SERIES; Fluid Mechanics & Hydraulics.
2.) "Foundation Design and Construction", MJ Tomlinson, Fourth Edition
3.) Mc Graw-Hill Civil Eng. Series, Third Ed.
4.) American Institute of Steel Construction → Eleventh Ed.

?



$$\gamma_{sub.} = \gamma_d - \frac{\gamma_d}{G} =$$

$$= 116 - \frac{116}{2.65} = 72 \text{ pcf}$$

~ Assume Clay is saturated

$$\gamma_{sat.} = \gamma_d (1 + w)$$
$$= 106(1 + 0.3) = 138 \text{ pcf}$$
$$\gamma_{sub.} = 138 - 62.4 = 75.6 \text{ pcf}$$

$$C_u = \left(\frac{75}{100}\right) \frac{\text{gf}}{\text{cm}^2} \times \frac{2 \text{ Ksf}}{1 \text{ kgf/cm}^2} = \underline{1.5 \text{ Ksf}}$$ (Ref. ", pg 97)

COMPUTATION SHEET
SCS-ENG-523 Rev. 8-69

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

STATE		PROJECT		
AZ		Safford Graham Canal		
BY ICC	DATE 7-11-85	CHECKED BY	DATE	JOB NO.
SUBJECT Find Max. Allowable Lateral Load on Pile				SHEET 6 OF

Ref. Foundation Design (pg 427-434)

$$P_z = P_{oz} K_{\phi z} + C_u K_{cz}$$

Note: At the end of the computations, a variety of pile sizes will be investigated for suitability. The smallest pile that will be investigated will be a 85# rail (B=5.5"). The minimum lateral load for the different piles investigated will given as that for the 85# rail.
A 6-foot scour will be assumed.

	Z		Z/B	$K_{\phi z}$	K_{cz}	$P_{oz} = \gamma_s Z$	$P_{oz} K_{\phi z}$	$C_u K_{cz}$	P_z	
		1								
		3								
		5								
W.T. ▽		7								
		9	2.2	—	—	—	—	—	—	▲ Neglect due to Scour
		11	6.5	—	—	—	—	—	—	
	Z_1	13	10.9	—	—	—	—	—	—	
	1	15	15.3	3.5	17	504 psf	1764	0	1764	
	3	17	19.6	3.5	17.6	648 psf	2268		2268	Sand & Gravel
	5	19	24.0		17.6	792	2772		2772	
	7	21	28.4			936	3276		3276	
	9	23	32.7			1080	3780		3780	
	11	25	37.1			1224	4284		4284	
	13	27	41.5			1368	4788		4788	
	15	29	45.8			1512	5292		5292	
	17	31	50.2			1656	5796		5796	
	19	32	54.5			1800	6300		6300	
	21	33	58.9			1944	6804		6804	
	23	35	63.3			2088	7308		7308	
	25	37	67.6	9.8	36	2344	22,971	27,000	49,971	Stiff Clay
	27	39	72.0	15.0	65	2495	37,425	48750	86,175	

$$\begin{cases} d = 4n = 2' \\ Z = \text{depth} \\ B = \text{pile width} = 5.5'' \end{cases}$$

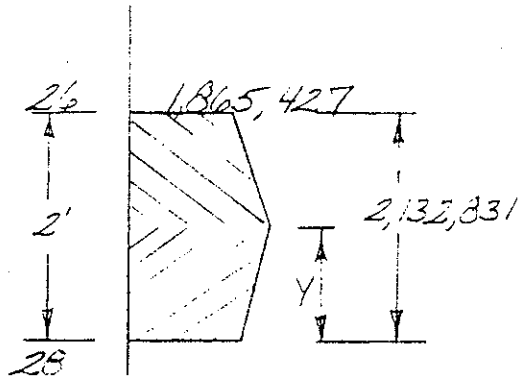
COMPUTATION SHEET
SCS-ENG-523 Rev. 8-69

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

STATE		PROJECT		
AZ		Safford Graham Canal		
BY	DATE	CHECKED BY	DATE	JOB NO.
ICC	7-11-85			
SUBJECT				SHEET 7 OF

Find x by $\Sigma M = \sum_{z=0}^{z=x} M - \sum_{z=x}^{z=L} M$ (Ref. pg. 432, Tomlinson)

Z,	P_z	$P_z dz, B$
1	1764	1617
3	2268	6237
5	2772	12,705
7	3276	21,021
9	3780	31,185
11	4284	43,197
13	4788	57,057
15	5292	72,765
17	5796	90,321
19	6300	109,725
21	6804	130,977
23	7308	143,451
25	49971	1,145,169
27	86,175	2,132,831



1.) $(1-y)(2,132,831) + 1,865,427$
2.) $y(2,132,831)$

For $y = 0.935$ (percentage of $y = 2'$)

1.) 2,004,061.0
2.) 1,994,197.0
} 0.49% error OK

$y = 0.935(2') = 1.87'$

Upper Layer = 138,634.0 + 1,865,427 = 2,004,061.0 '-#

Lower Layer = 1,994,197 '-#

$x = 26 + 2 - 1.87 = \underline{26.13'}$

COMPUTATION SHEET
SCS-ENG-523 Rev. 8-69

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

STATE AZ		PROJECT Safford Graham Canal		
BY ICC	DATE 7-11-85	CHECKED BY	DATE	JOB NO.
SUBJECT				SHEET 8 OF

Solve eq. 7.24 (Tomlinson)

$$\sum_{0}^{x} P_z (4/n) B (x-z) \quad , \text{for } x = 26.13'$$

z_1	$(x-z_1)$	$P_z(4/n)B$	$P_z(4/n)B(x-z_1)$
1	25.13	1617	40,635
3	23.13	2079	48,087
5	21.13	2541	53,671
7	19.13	3003	57,447
9	17.13	3465	59,355
11	15.13	3927	59,416
13	13.13	4389	57,628
15	11.13	4851	53,992
17	9.13	5313	48,508
19	7.13	5775	41,176
21	5.13	6237	31,996
23	3.13	6699	20,968
25	1.13	45,807	51,762
26.07	0.07	5,135	359
			625,020

$$\sum_{x}^{x+L} P_z (4/n) B (z-x)$$

z	$(z-x)$	$P_z(4/n)B$	$P_z(4/n)B(x-z_1)$
27.07	0.94	73,859	69,428

$$H_u(x) = 625{,}020 + 69{,}428$$

$$H_u = \frac{694{,}448}{26.13} = 26{,}577^{\#} = \underline{\underline{26.6^{k}}}$$

J. Weaver
1/16/89

GRAHAM CANAL DEFLECTING JETTIES
APPROX. TOTAL COST

MATERIALS

RAILS 370 @ $135.88 (CONTRACT) = $50,275.60
18 @ $135.88 (CONTRACT) = 2,430.00

CARS

10 Piggyback @ $3,800 (CONTRACT) = 38,000.00
10 Piggyback
4 FLATCARS } SPONSORS 20% ≈ 42,920 est. 20%
8 ORECARS

INSTALLATION = 80,978
$214,604

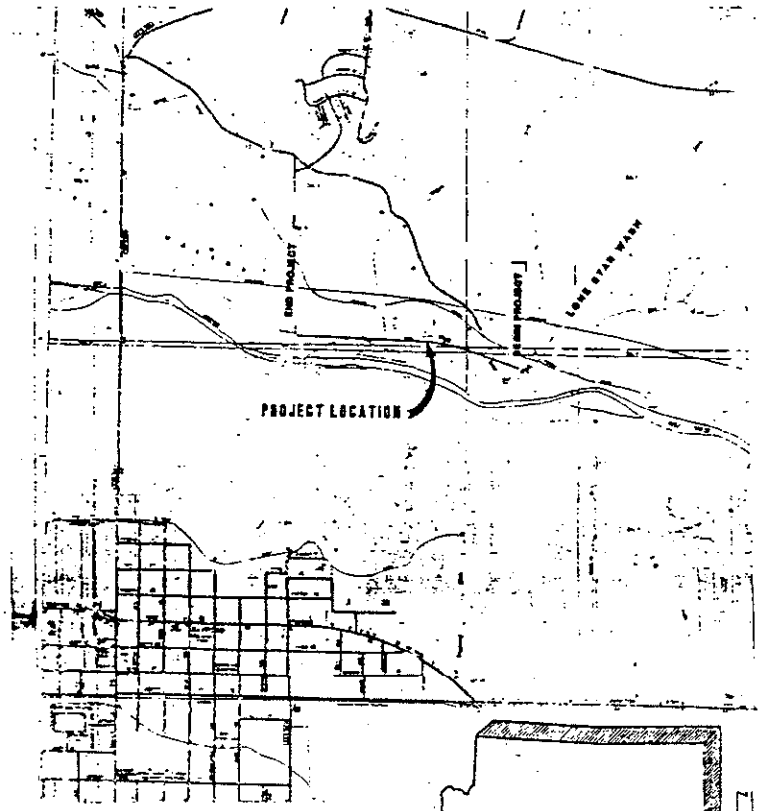
TOTAL LENGTH = 2,220'
= $96.67 PER FT.

OTHER
WASTEWAY CHANNEL $11,000.00

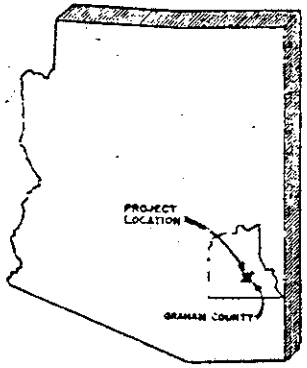
TOTAL JOB = $225,604

≈ $101.63 PER FT.

GRAHAM CANAL - CITY OF SAFFORD
DEFLECTING JETTIES PROJECT
EMERGENCY WATERSHED PROTECTION PROGRAM
GRAHAM COUNTY, ARIZONA



PROJECT MAP
SAFFORD, AZ



ARIZONA

INDEX

1. PROJECT LOCATION, & INDEX
2. PLAN, PROFILE, & JETTIE DETAILS

DEFLECTING JETTIES

PRELIMINARY LAYOUT

JETTIE #	TYPE	LENGTH (FT)	LENGTH ON ALIGNMENT (FT)	SPACING (FT)	ALIGNMENT STATIONS BEGIN	ALIGNMENT STATIONS END
1	Piggyback	89.5	84.5	–	11+60±	
2				50		
3						
4						
5						
6						
7						
8						
9						
10						
11						
12						
13						
14						
15						
16						
17						
18						
19						
20						
21	Flat (2)	80.0	77.3			
22	Flat (2)	80.0	77.3			
23	Ore (2)	68.0	65.7			
24	Ore (2)	68.0	65.7			
25	Ore (2)	68.0	65.7			45+57.4±
26	Ore (2)	68.0	65.7			
	Totals	2,222.0	2,147.4	1,250.0		

TOTAL NUMBER OF RAILS (PILES) REQUIRED

TYPE OF JETTIE	LENGTH (FT)	RAILS PER JETTIE BANKSIDE	RAILS PER JETTIE STREAMSIDE	NO. OF JETTIES	TOTAL RAILS BANKSIDE	TOTAL RAILS STREAMSIDE	TOTAL
Piggyback	89.5	10	5	20	200	100	300
Flat Cars (2)	80.0	10	6	2	20	12	32
Ore Cars (2)	68.0	8	6	4	32	24	56
					252	136	388

CONTRACT NO 50-9457-6-00510
GOVT. REP. Bobby Kilgrease
INSPECTOR Bill T. Cutter
CONTRACTOR Smith & Bell
COMPLETION DATE May 2, 1986

AS BUILT

GILA VALLEY IRRIGATION DISTRICT	GRAHAM CANAL COMPANY	CITY OF SAFFORD				
APPROVED	APPROVED	APPROVED				
DATE 10/-- 1985	DATE 10-22-85	DATE 10-22-85				

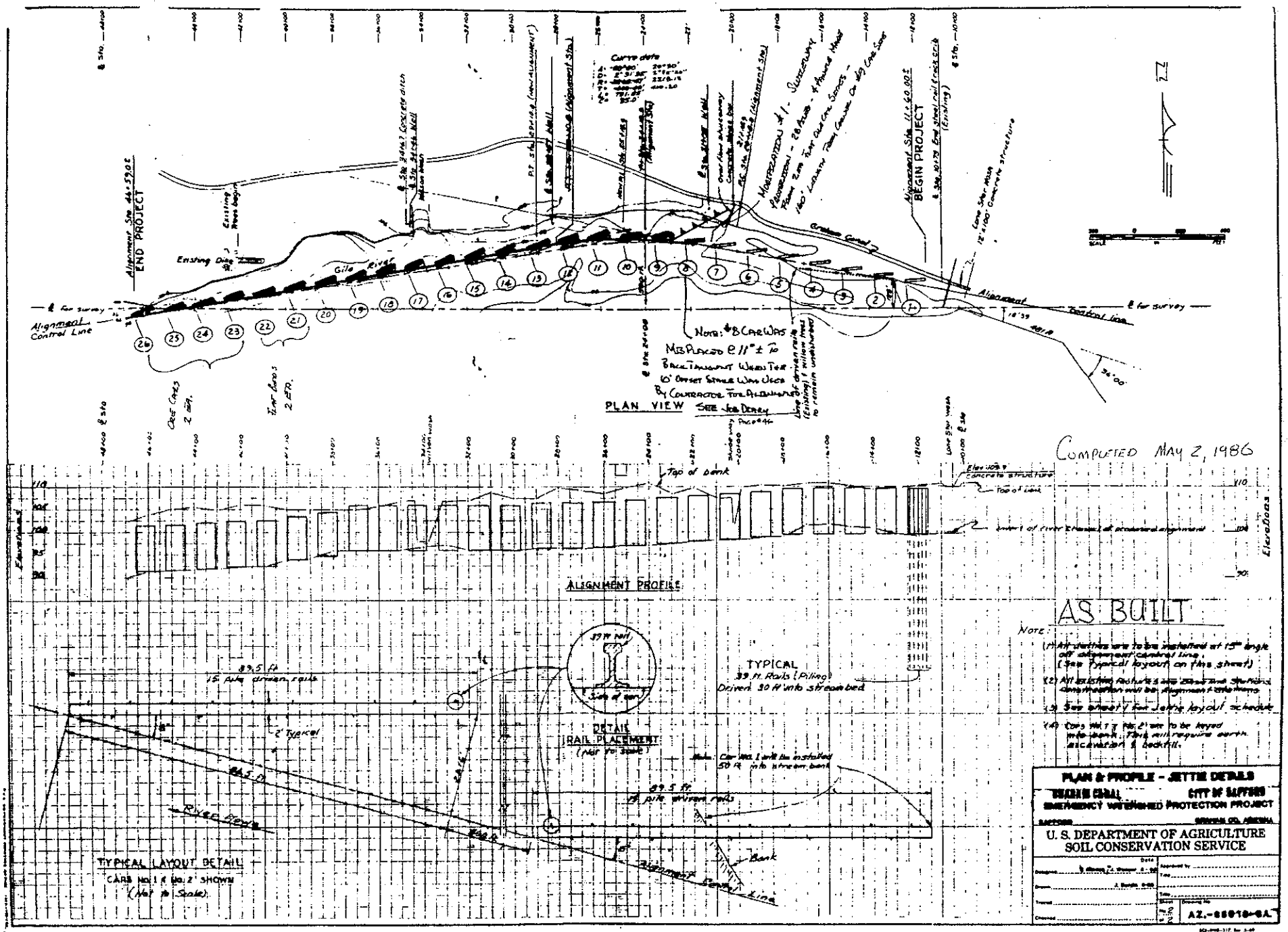
PROJECT LOCATION, & INDEX
GRAHAM CANAL - CITY OF SAFFORD
EMERGENCY WATERSHED PROTECTION PROGRAM
GRAHAM COUNTY ARIZONA

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

STATE CONSERVATION ENGINEER
STATE ENGINEERING TECH.

AZ. - 85818 - SA.

— As Built —


Alignment Sta 46+52.0±
END PROJECT
Existing Dike
Gila River
Alignment Control Line
℄ for survey
Alignment Sta 11+60.00±
BEGIN PROJECT
Grahm Canal
Alignment
Control Line
PLAN VIEW
Note: #8 Cable Was Replaced @ 11" ± To Back Tangent When For 60' Offset Single Wire Used By Contractor For Alignment
Computed May 2, 1986
Top of bank
Elevations
Elevations
ALIGNMENT PROFILE
AS BUILT
Note:
(1) All jetties are to be installed at 15° angle off alignment centerline. (See typical layout on this sheet)
(2) All existing features are shown and structure construction will be alignment stationing.
(3) See sheet 7 for jetty layout schedule
(4) Cons No. 1 & No. 2 are to be keyed into bank. This will require earth excavation & backfill.
TYPICAL
39 ft. Rails (Piling)
Driven 30 ft into streambed
DETAIL
RAIL PLACEMENT
(Not to Scale)
89.5 ft
15 pile driven rails
2' Typical
River Flow
Alignment Centerline
Bank
TYPICAL LAYOUT DETAIL
CABS No. 1 & No. 2 SHOWN
(Not to Scale)
PLAN & PROFILE - JETTIE DETAILS
CITY OF SAFFORD
EMERGENCY WATERSHED PROTECTION PROJECT
U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
AZ.-65010-GA.

— As Built —

Oct. 1985

GRAHAM CANAL

DEFLECTING JETTIES

MATERIAL SPECIFICATIONS

SPONSORS MATERIALS

RAILS

Rails shall be new or used 39 foot railroad rail with a minimum new nominal weight of 131 pounds per yard and a minimum new Base Section Modulus of 27.0 cubic inches. The rails shall be straight. Splices will not be permitted. The cross-section of each rail shall be constant throughout its length. The maximum allowable wear for used rail shall be 5 percent of the new nominal height (see table below) except that 10 percent wear will be allowed for a one foot length on each end. At least 90 percent of the total number of rails supplied shall be 39 feet in length; the remaining 10 percent shall be at least 38 feet in length. Any rail that appears to be purposely shortened to meet the length provisions of this paragraph will be rejected.

A partial listing of acceptable rail is shown below. The minimum height required for used rail (5% wear factor) is shown in the last column.

Rail	Nominal Weight (Lbs/Yd.)	Base Section Modulus (Cu. In.)	New Height (In.)	Minimum[1]/ Height Reqd. (In.)
RE 131	131	27.65	7.125	6.77
RE 132	132	27.60	7.125	6.77
RE 133	133	27.0	7.0625	6.71
RE 136	136	28.3	7.3125	6.95
NYC 136	136	28.1	7.281	6.92
RE 140	140	28.7	7.3125	6.95

1/ 5 percent wear

Oct. 1985

GRAHAM CANAL

DEFLECTING JETTIES

MATERIAL SPECIFICATIONS

SPONSORS MATERIALS

RAILROAD CAR JETTIES

General

Jetties will be constructed from the beds of steel railroad cars. Beds shall have a minimum width of 9 feet. Beds shall be straight and not bent, twisted, or warped, except a bent piggyback car(s) might be accepted for use as a bank tie-in jetty at the head end of the project. Any bent piggyback will be evaluated by the Government on a case-be-case basis. No more than two, if any, bent piggyback cars will be accepted.

The wheel, axles, tongues, tanks, pipes, compressors, and other extraneous appurtenances shall be removed from the cars.

Piggybacks

Piggyback cars with steel beds shall be provided as is. One piggyback car will constitute on jettie unit.

Bottom Dump Ore Cars

Bottom dump ore cars shall be provided as is. Two bottom dump ore cars shall constitute one jettie unit.

Side Dump Ore Cars

The sides shall be removed from Side Dump Ore Cars. The ends can either be removed or left on. Two side dump ore cars shall consititute one jettie unit.

Flatcars

Steel bed flatcars shall be provided as is. Wooden bed flatcars shall be plated on the underside of the bed with steel or concrete (see Sketch No. 1). One flatcar shall contitute one jettie unit.

Refrigerator Box Cars

The ends, tops, and sides of refrigerator box cars shall be removed. The sides and ends shall be cut about one foot above the floor, but not less than the height of the perforated steel channels in the floor that support the removable wooden slat flooring (see Sketch No. 2). These steel channels shall not be removed. The bed shall be plated with steel or concrete (see Sketch No. 2). One refrigerator box car shall constitute one jettie unit.

BID ESTIMATE

GRAHAM CANAL - CITY OF SAFFORD

DEFLECTING JETTIES

INSTALLATION CONTRACT

Item No.	Spec. No.	Supplies Services	Quantity	Unit	Unit Price	Amount
1	216-1	Mobilization	1	Job	$	$
2	216-2	Jetties	26	Each	$	$
3	216-3	Piles	390	Each	$	$
					Subtotal	$

CONSTRUCTION SPECIFICATION

216-2 JETTIES

1. SCOPE

The work shall consist of installing steel jetties for streambank protection.

2. MATERIALS

Jetties shall be constructed from the beds of steel railroad cars. All Jetties will be furnished by the Government at the job site.

3. INTALLATION

The jetties (railroad car beds) shall be installed in the vertical plane between driven railroad rail piles. The jetties will be installed at a 15° angle with the alignment as staked in the field.

The bottom edge of the jetties will rest on the streambed for the full length of the jettie. The streambed at the location of each jettie shall be bladed smooth or otherwise treated to remove brush, large rocks, earth mounds, or other obstructions prior to installation of the jetties.

Jetties shall be loosely installed between the piles so that the jetties can settle if they are undercut by channel scour. Excess steel or other fixtures on a jettie that will hang up on a pile and restrict free settlement shall be removed prior to installation of the jettie.

4. MEASUREMENT AND PAYMENT

Payment will be at the contract unit price for each jettie installed. Such payment will consititute full compensation for all labor, materials, equipment, and all other items necessary and incidental to the completion of the work.

Compensation for any item of work described in the contract but not listed in the Bid Schedule will be included in the payment for the item of work to which it is made subsidiary. Such items and the items to which they are made subsidiary are identified in Section 5 of this specification.

NSN 7540-01-152-8067
PREVIOUS EDITION USABLE
26-109
STANDARD FORM 26 (REV. 10-83)
Prescribed by GSA
FAR (48 CFR) 53.111

5. ITEMS OF WORK AND CONSTRUCTION DETAILS

Items of work to be performed in conformance with this specification and the construction details are:

a. Bid Item 2, Jetties

(1) This item shall consist of furnishing all labor, and equipment necessary for installing steel jetties constructed from the beds of railroad cars, including any excavation backfill, site preparation, and other items necessary to install the jetties.

(2) Jettie (car) No. 1 shall be a minimum of 89.0 feet in length and installed at streambed elevation at least 50.0 feet into the riverbank as staked in the field. The riverbank will be trench excavated to the minimum width necessary to prepare and install the jetties. Excavates areas within a six foot distance of the Graham Canal. Other excavated areas shall be backfilled without compaction to the original ground elevation.

(3) Payment will be made in accordance with Section 4 of this specification and will include compensation for Subsidiary Item, Removal of Water.

b. Subsidiary Item, Removal of Water

(1) No separate payment will be made for removal of water or river diversion. Compensation for removal of water will be included in the payment for Jetties, Bid Item No. 2.

SN 7540-01-152-8087
PREVIOUS EDITION USABLE
36-109
STANDARD FORM 36 (REV. 10-83)
Prescribed by GSA
FAR (48 CFR) 53.111

CONSTRUCTION SPECIFICATION

216-3 PILES

1. SCOPE

The work shall consist of installing steel piles for streambank protection.

2. MATERIALS

All piles will be furnished by the Government at the job site. The piles are 39 feet railroad rails with a minimum weight of 131 pounds per year.

3. INSTALLATION OF STEEL PILES

A. Equipment:

1. Hammers for Steel Piles:

Steel piles shall be driven with a steam or air or diesel hammer.

2. Leads:

Pile driver leads shall be constructed in such a manner as to afford freedom of movement to the hammer and they shall be blocked or braced so that the initial driving of the pile can be done without rotation or shifting of the pile and to assure concentric hits on the piles.

3. Followers:

Driving of piles with followers will not be permitted.

B. Driving Piles:

1. General:

All piles shall be driven to a minimum depth of 30.0 feet from average streambed elevation at the jettie location. All piles shall be driven to at least the required depth using a hammer that is capable of overcoming the soil resistance to reach that depth without causing damage to the pile.

NSN 7540-01-152-8067
PREVIOUS EDITION USABLE
36-109
STANDARD FORM 36 (REV. 10-83)
Prescribed by GSA
FAR (48 CFR) 53.111

Piles shall be driven vertically insofar as practicable. The driving of each individual pile shall be a continuous operation.

Piles and appurtenances shall be installed in a workmanlike manner to the lines and at location shown on the drawings or staked in the field.

3. Defective Piles:

The methods used in driving piles shall not subject them to excessive and undue abuse producing deformation of the steel. Excessive manipulation of piles to force them into proper position will not be permitted.

4. MEASUREMENT AND PAYMENT

Payment for each steel pile will be made at the contract unit price for steel pile installed. Such payment will include full compensation for all labor, materials, equipment and all other items necessary to the completion of the work.

Compensation for any item of work described in the contract but not listed in the bid schedule will be included in the payment for the item of work to which it is made subsidiary. Such items and the items to which they are made subsidiary are identified in Section 5 of this specification.

NSN 7540-01-152-8067
PREVIOUS EDITION USABLE
36-109
STANDARD FORM 36 (REV. 10-83)
Prescribed by GSA
FAR (48 CFR) 53.111

5. ITEMS OF WORK AND CONSTRUCTION DETAILS

Items of work to be performed in conformance with this specification and the construction details are:

a. Bid Item 3, Piles

(1) This item shall consist of furnishing all, labor, and equipment necessary for installing steel piles including any excavation, backfill, and any earth pads, mats, or other systems necessary to support pile driving equipment and all appurtenances as shown on the drawings and staked in the field.

CONSTRUCTION SPECIFICATION

11. REMOVAL OF WATER

1. SCOPE

The work shall consist of the removal of surface water and ground water as needed to perform the required construction in accordance with the specifications. It shall include (1) building and maintaining all necessary temporary impounding works, channels, and diversions, (2) furnishing, installing and operating all necessary pumps, piping and other facilities and equipment, and (3) removing all such temporary works and equipment after they have served their purposes.

2. DIVERTING SURFACE WATER

The Contractor shall build, maintain, and operate all cofferdams, channels, flumes, sumps, and other temporary diversion and protective works needed to divert streamflow and other surface water through or around the construction site and away from the construction work while construction is in progress. Unless otherwise specified, a diversion must discharge into the same natural drainageway in which its headworks are located.

Unless otherwise specified, the Contractor shall furnish to the Engineer, in writing, his plan for diverting surface water before beginning the construction work for which the diversion is required. Acceptance of this plan will not relieve the Contractor of responsibility for completing the work as specified.

3. DEWATERING THE CONSTRUCTION SITE

Foundations, cutoff trenches and other parts of the construction site shall be dewatered and kept free of standing water or excessively muddy conditions as needed for proper execution of the construction work. The Contractor shall furnish, install, operate and maintain all drains, sumps, pumps, casings, wellpoints, and other equipment needed to perform the dewatering as specified. Dewatering methods that cause a loss of fines from foundation areas will not be permitted.

NSN 7540-01-152-8067
PREVIOUS EDITION USABLE
36-109
STANDARD FORM 36 (REV. 10-83)
Prescribed by GSA
FAR (48 CFR) 53.111

Unless otherwise specified, the Contractor shall furnish to the Engineer, in writing, his plan for dewatering before beginning the construction work for which the dewatering is required. Acceptance of this plan will not relieve the Contractor of responsibility for completing the work as specified.

4. DEWATERING BORROW AREAS

Unless otherwise specified in Section 8, the Contractor shall maintain the borrow areas in drainable condition or otherwise provide for timely and effective removal of surface and ground waters that accumulate within the borrow areas from any source. Borrow material shall be processed as necessary to achieve proper and uniform moisture content for placement.

If pumping to dewater borrow areas is included as an item of work in the bid schedule, each pump used for this purpose shall be equipped with a water meter in the discharge line. Accuracy of the meters shall be such that the measured quantity of water is within 3 percent, plus or minus, of the true quantity. Means shall be provided by the Contractor to check the accuracy of the water meters when requested by the Engineer.

5. EROSION AND POLLUTION CONTROL

Removal of water from the construction site, including the borrow areas shall be accomplished in such a manner that erosion and the transmission of sediment and other pollutants are minimized.

6. REMOVAL OF TEMPORARY WORKS

After the temporary works have served their purposes, the Contractor shall remove them or level and grade them to the extent required to present a sightly appearance and to prevent any obstruction of the flow of water or any other interference with the operation of or access to the permanent works.

Except as otherwise specified, pipes and casings shall be removed from temporary wells and the wells shall be filled to ground level with gravel or other materials approved by the Engineer.

7. MEASUREMENT AND PAYMENT

(Method 1) - Items of work listed in the bid schedule for removal of water, diverting surface water, dewatering construction sites, and dewatering borrow areas will be paid for at the contract lump sum

NSN 7540-01-152-8067
PREVIOUS EDITION USABLE
26-109
STANDARD FORM 36 (REV. 10-83)
Prescribed by GSA
FAR (48 CFR) 53.111

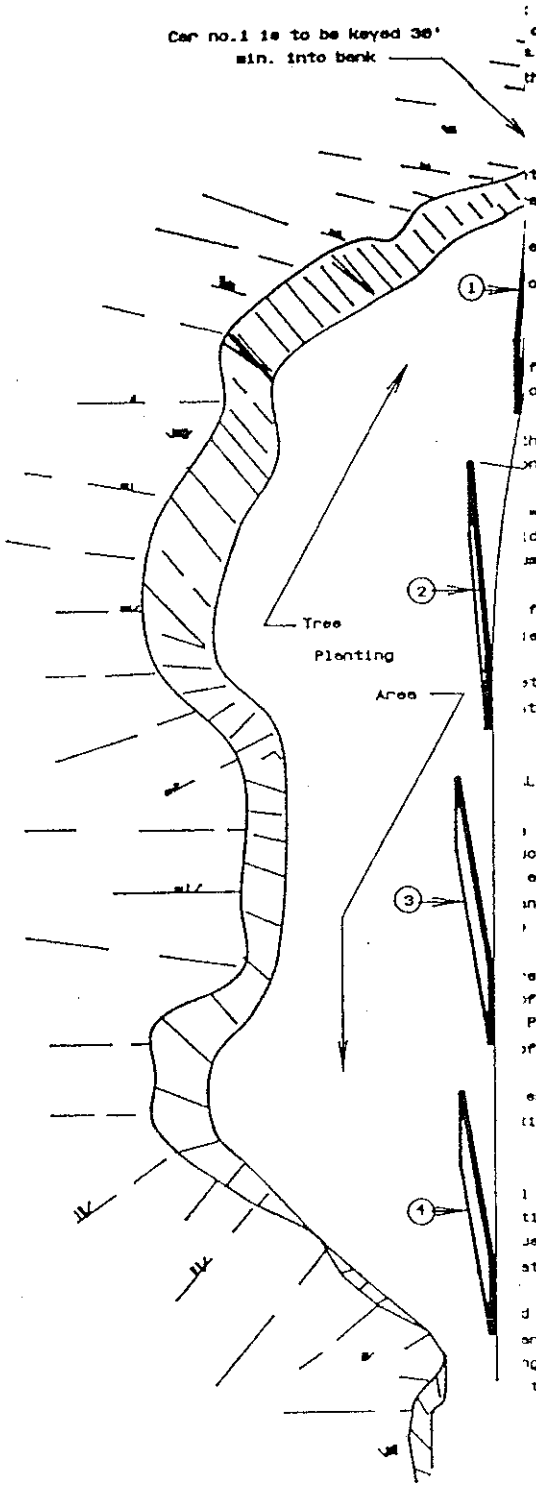
prices. Such payment will constitute full compensation for all labor, equipment, tools, and all other items necessary and incidental to the completion of the work.

Method 2) - Items of work listed in the bid schedule for removal of water, diverting sruface water, dewatering construction sites, and dewatering borrow areas will be paid for at the contract lump sum prices. Such payment will constitute full compensation for furnishing, installing, operating, and maintaining the necessary trenches, drains, sumps, pumps, and piping, and for all labor, equipment, tools, and all other items necessary and incidental to the completion of the work except that additional payment for pumping to dewater borrow areas will be made as described in the following paragraph.

If pumping to dewater borrow areas is listed as an item of work in the bid schedule, payment will be made at the contract unit price which shall be the price per 1,000 gallons shown in the bid schedule. Such payment will consititute full compensation for pumping only. Compensation for equipment and preparation and for other costs associated with pumping will be included in the lump sum payment for removal of water or the lump sum payment for dewatering borrow areas. Payment will be made only for pumping that is necessary to dewater borrow areas that cannot be effectively drained by gravity or that must have the water table lowered to be usable. Pumping for other purposes will not be included for payment in this item.

All Methods - The following provisions apply to all methods of measurement and payment. Compensation for any item of work described in the contract but not listed in the bid schedule will be included in the payment for the item of work to which it is made subsidiary. Such items and the items to which they are made subsidiary are identified in Section 8 of this specification.

SN 7540-01-152-8067
PREVIOUS EDITION USABLE
96-109
STANDARD FORM 36 (REV. 10-83)
Prescribed by GSA
FAR (48 CFR) 53.111



: railroad rail. The rail shall be straight.
orose - section of each rail shall be constant
a allowable wear for used rail shall be 5 percent
that 10 percent wear will be allowed for one foot

steel railroad cars. Cars shall have a minimum
aight and not bent, twisted, or warped ,except
for use as a bank tie in jetty at the ends of
evaluated on a case - by - case basis.

compressors, and other extraneous appurtenances

feet).
one jettie unit.

h = 34.5 feet).
onstitute one jettie unit.

=34 feet)
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mp ore cars shall constitute one jettie unit

ft.)
jettie unit.

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ttie unit. The sides, top, and ends of

LLATION

steam or air diesel hammer.
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end shall be blocked or braced so that
n be done without rotation or
concentric hits on the piles.

s will not be permitted. All piles
f 30 feet from average streambed
Piles shall be driven vertically
f each individual pile shall be a continous

shall not subject them to excessive
tion of the steel.

l rest on the full length of the
tion of each jettie shall be blended
ish, large rocks, earth mounds,or other
ation of the jetties

d between the piles so that the jetties can
annel scour. Excess steel or other
ng up on a pile and restrict free
to installation of the jettie.

ISOMETRIC PLAN VIE

Approved By: Kim Wilmore
Title: Asst. State Conv. Eng.
Date: 6/89

		DATE
Designed.	J. VEAVER	5/89
Drawn	B. LLOYD	5/89
Traced		
Checked.		

BANK PROTECTION
RAILROAD CAR DEFLECTING JETTIES

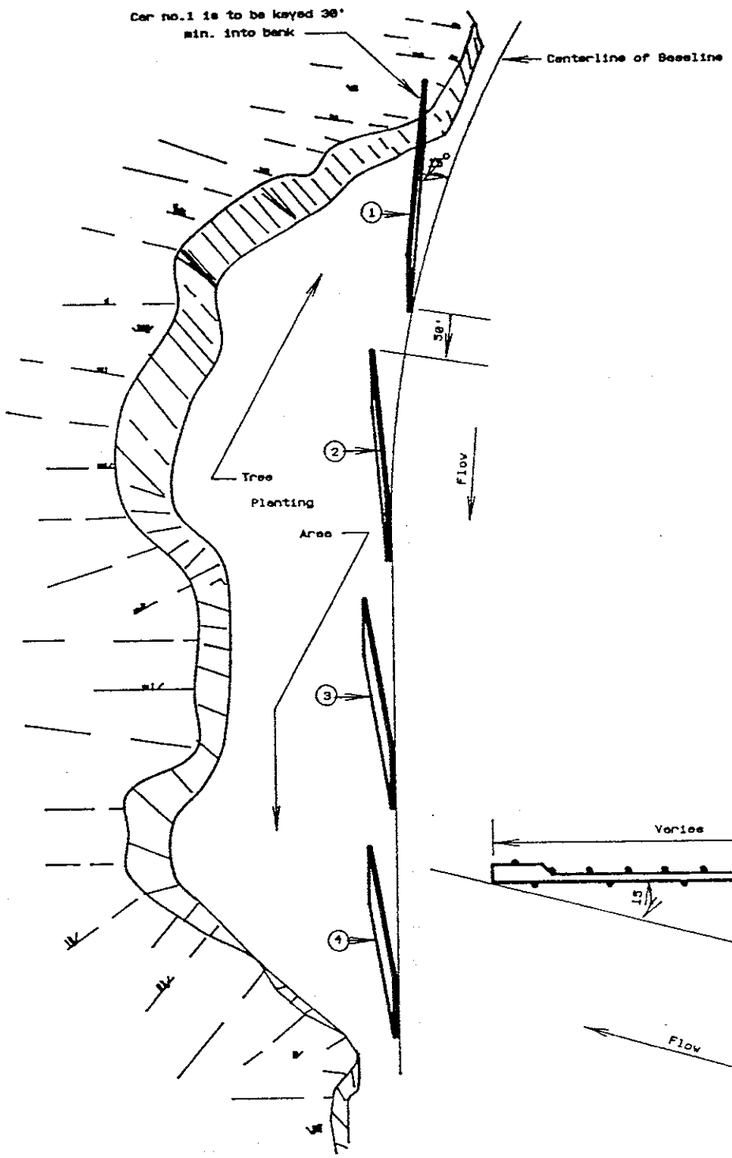
U.S. DEPARTMENT OF AGRICULTURE SOIL CONSERVATION SERVICE

CAD FILE NO.

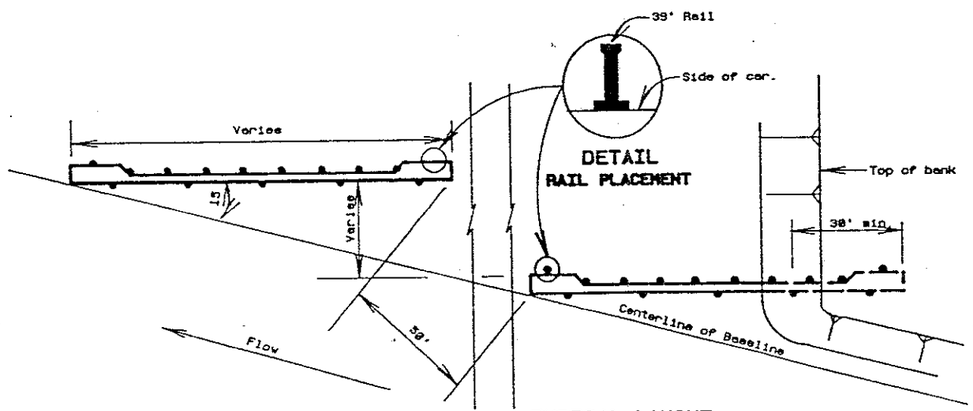
DRAWING NO.
89082 - AZ. - SA.

SHEET NO. 1 OF 1

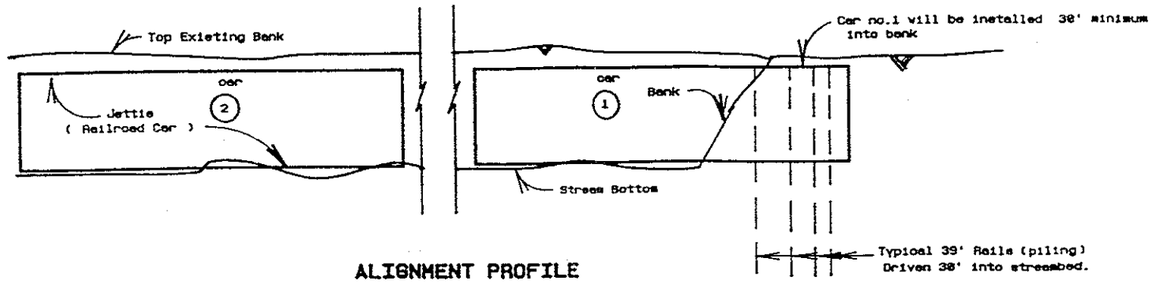
REVISIONS		
DATE	APPROVED	TITLE



ISOMETRIC PLAN VIEW



TYPICAL LAYOUT DETAIL



ALIGNMENT PROFILE

NOT TO SCALE

DESIGN ASSUMPTIONS

Stream velocity——— =16.0 ft./ sec.
Scour depth——————— = 6.0 ft.
Initial angle of attack =15.0 degrees
Car height——————— = 8-18 ft.
Length of rail ——— =39.0 ft.
Driven length of pile - =31.0 ft.

Soils——— 0 - 30 ft. sand,silt,coarse gravel.
30 - 31 ft. dense saturated clay

Computed max. moment = 5170 ft. # per. ft.length of car
Computed max. shear = 582 ft. # per. ft. length of car
Impact load = +15 percent

MAXIMUM PILE SPACING

RAILS		BASE Sx (in)	MAXIMUM SPACING BANKSIDE (FT)	MAXIMUM SPACING STREAMSIDE
ARA -A	90	15.2	9	Min. 3 piles per car
ASCE	90	13.5	8	"
AREA	100	17.8	11	"
US	150	14.3	9	"
AREA	115	22.0	13.5	"
CF & I	119	22.9	14	"
CB	122	22.9	14	"
BE	131.1	27.55	14	"
AREA	132	27.6	17	"
RE	136	28.3	17	"
NYC	136	28.1	17	"
RE	140	28.7	17	"

Designer should compute actual pile resistance and determine type of pile and spacing if flow conditions exceed the values shown above or if soils in profile provide less resistance than the soils shown above.

MATERIALS

RAILS

Rails shall be new or used 39 foot railroad rail. The rail shall be straight. Splices will not be permitted. The cross-- section of each rail shall be constant throughout its length. The maximum allowable wear for used rail shall be 5 percent of the new nominal height except that 18 percent wear will be allowed For one foot lenght on each end.

JETTIES

Jetties will be constructed from steel railroad cars. Cars shall have a minimum width of 8 feet. Cars shall be straight and not bent, twisted, or warped ,except a bent car (s) might be accepted for use as a bank tie in jetty at the ends of the project. Any bent car will be evaluated on a case - by - case basis.

The wheel, axles, tongues, pipes, compressors, and other extraneous appurtenances shall be removed from the cars.

PIGGYBACKS (normal length = 85.5 feet).
One piggyback car will constitute one jettie unit.

BOTTOM DUMP ORE CARS (normal length = 34.5 feet).
Two bottom dumps ore cars shall constitute one jettie unit.

SIDE DUMP ORE CARS (normal length =34 feet)
The sides shall be removed from side dump ore cars. The ends can either be removed or left on. Two side dump ore cars shall constitute one jettie unit

FLATCARS (normal length = 50 - 54 ft.)
One flatcar shall constitute one jettie unit.

BOX CARS (Normal length = 57.8 feet)
One boxcar shall constitute one jettie unit. The sides, top, and ends of the car shall be removed.

INSTALLATION

PILES

Steel piles shall be driven with a steam or air diesel hammer.
Pile driver leads shall be constructed in such a manner as to afford freedom of movement to the hammer and shall be blocked or braced so that the initial driving of the pile can be done without rotation or shifting of the pile and to assure concentric hits on the piles.

Driving of the piles with followers will not be permitted. All piles shall be driven to minimum depth of 30 feet from average streambed elevation at the jettie location. Piles shall be driven vertically insofar as possible. The driving of each individual pile shall be a continous operation.
The methods used in driving piles shall not subject them to excessive and undue abuse producing deformation of the steel.

JETTIES

The bottom edge of the jettie will rest on the full length of the jettie. The streambed at the location of each jettie shall be bladed or otherwise treated to remove brush, large rocks, earth mounds,or other obstructions prior to the installation of the jetties

Jetties shall be loosely installed between the piles so that the jetties can settle if they are undercut by channel scour. Excess steel or other fixtures on a jettie that will hang up on a pile and restrict free settlement shall be removed prior to installation of the jettie.

REVISIONS		
DATE	APPROVED	TITLE

Designed J. WEAVER 5/89
Drawn B. LLOYD 5/89
Traced
Checked
Approved By John Wilcox
Title Asst. State Conv. Engr.
Title

U.S. DEPARTMENT OF AGRICULTURE SOIL CONSERVATION SERVICE

BANK PROTECTION
RAILROAD CAR DEFLECTING JETTIES

CAD FILE NO.
DRAWING NO. 89082 - AZ. - SA.
SHEET NO. 1 OF 1

GABIONS
Engineer Workshop
February 7, 1990

Leland Hardy

A. General:

Gabion erosion control structures consist of modular wire baskets filled with 4"-12" durable rock; stacked or joined together to form the desired mass or shape.

Gabions are used world wide for:

- Retaining wall structures
- Bank stabilization structures
- Energy dissipation (drop) structures
- Bridge abutment protection/stabilization
- Channel lining
- Slope stabilization (sloughing) wire only or mattress

Gabions are first class structures, often used where other materials cannot be used or for asthetic reasons are undesirable. Advantages are:

- Structure may be shaped around curves to fit the site.
- Baskets can be modified to fit around obstacles.
- Utilization of local construction materials and labor.
- Durability (galvanized wire may be plastic coated).
- Versatile -- Asthetics to energy dissipation.
- Can be installed with only hand labor.
- Flexible -- Structure tends to be self-healing.

B. Slides of Arizona Gabion projects

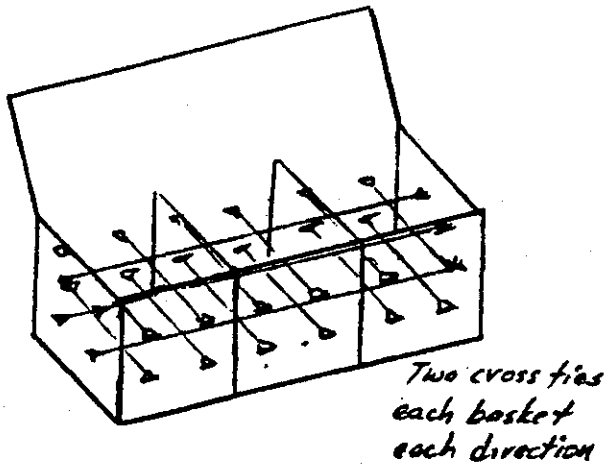
C. Designing gabion structures for bank stabilization Design principles are the same as for other construction materials. Design for: undercutting, sliding, overturning, bank mining, and where necessary foundation bearing.

1. Undercutting:
   a. Place lower limit of structure below anticipated scour of channel.
   b. Use 1' x 6' baskets extended perpendicular to bank in the first layer (tend to be self-healing).
   c. Concentrate horizontal water force against side of structure.

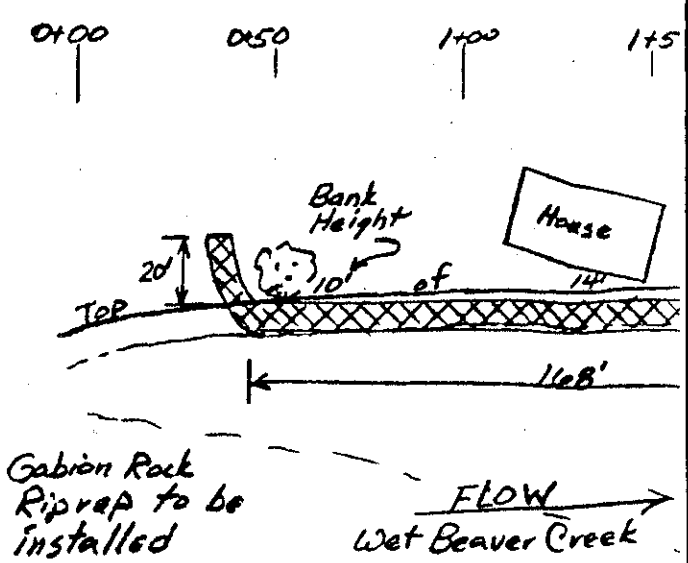
2. Sliding:
   a. Broaden base for increased friction to resist horizontal movement.
   b. Slope foundation toward bank.
   c. On high or saturated soil banks check for foundation shear failure. Stabilize shear plane with structure shape and weight.

3. Overturning:
   a. Place gabion baskets so structure cross-section center of gravity tends to rotate structure against banks.
   b. Slope exposed face of structure into the bank.

(over)



TYPICAL BASKET
(Cross ties)



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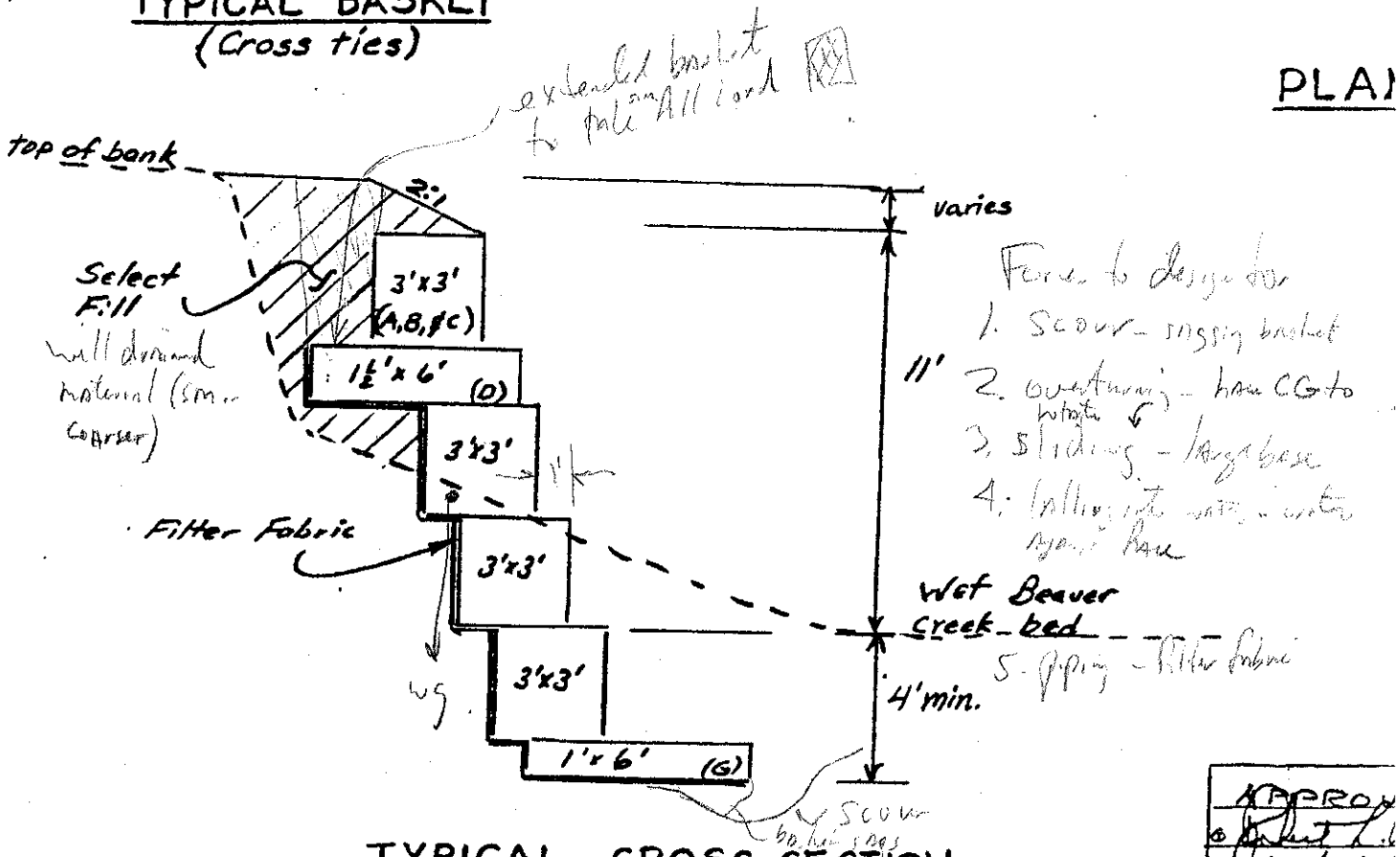

TYPICAL CROSS SECTION

APPRO

STREAMBANK PROTECTION COURSE

US Army Engineer Waterways Experiment Station
Vicksburg, MS 39180-0631

2-6 December 1985

SCHEDULE

Time	Subject	Speaker*
Monday		
0800-0830	Registration and Administrative Details	Louis Renaud, LMVD
0830-0845	Welcome	Frank Herrmann
0845-0930	Orientation and WES Multi-Screen Presentation	Bob McCarley
0930-1000	(A) Introduction and Overview of Section 32 Program	Gene Stallings, OCE
1000-1015	Break	
1015-1145	(B) River Mechanics and Geomorphology	Tim Ethridge, LMVD
1145-1230	(C) Streambed Degradation and Aggradation	David Biedenharn, LMK
1230-1330	Lunch (including group photograph)	
	(D) Streambank Erosion and Failure	
1330-1400	1. Causes of Erosion	Randy Oswalt
	2. Erosion of Soils	Ed Perry
1400-1430	3. Streambank Failure	Ed Perry
1430- 1445	Break	
	(E) General Requirements for Streambank and Streambed Protection	
1445-1515	1. General Requirements	Steve Maynard
1515-1545	2. Need for Filters (granular and filter fabric)	Malcolm Keown
1545-1625	(F) Nonstructural Streambank Erosion Protection Measures	
	1. Control of Surface Drainage	Mike Spoor, ORH
	2. Regulation of Water Surface Elevation and River Traffic	Mike Spoor and George Kincaid, ORH
Tuesday		
0800-0830	(F) Nonstructural Streambank Erosion Protection Measures (continued)	
	3. Planted Vegetation	Hollis Allen
	4. Management of Native Vegetation	Hollis Allen

* WES staff unless indicated.

Time	Subject	Speaker*
Tuesday (cont'd)		
	(G) Structural Streambank Erosion Protection Measures	
0830-0845	1. Relocation of Channel	Laszlo Varga, ORD
0845-0900	2. Single Component Revetment	
	a. General Requirements	Steve Maynard
	b. Stone Revetments and Longitudinal Peaked Stone Dike	
0900-0930	1. Stone Quality and Performance	Tom Wilkinson, NCB
0930-1000	2. Riprap Blanket	Steve Maynard
1000-1015	Break	
1015-1045	3. Windrow, Reinforced, and Composite Revetment	Doug Plack, MRO
1045-1115	4. Longitudinal Peaked Stone Dike	Joe Montz, LMN
1115-1130	c. Locally Available Products	
	1. Steel Furnace Slag	Laszlo Varga, ORD
	2. Low Quality Rock (including quarry-run as opposed to graded riprap)	Walt Linder, MRK
1145-1200	d. Cellular Blocks	Walt Linder, MRK
1200-1230	e. Sack Revetment	Charles Elliott, LMVD
1230-1330	Lunch	
	3. Mattress, Tubing, Gabions, and Clay Blanket	
1330-1345	a. Articulated Concrete Mattress	Steve Ellis, LMVD
1345-1400	b. Used Auto Tire Mattress	Bobby Littlejohn, LMM
1400-1430	c. Gabions	Tasso Schmidgall, SWD
1430-1500	d. Synthetic Mattress, Tubing, and Clay Blanket	Walt Linder, MRK
1500-1515	Break	
	4. Soil Stabilization	
1515-1545	a. Cement	Doug Plack, MRO
1545-1600	b. Lime	Bobby Littlejohn, LMM
	5. River Training Structures	
1600-1615	a. Dikes (Groin, Spur, and Jetty)	Doug Plack, MRO
1615-1630	b. Kellner Jacks	Tasso Schmidgall, SWD

* WES staff unless indicated.

Time	Subject	Speaker*
Tuesday (cont'd)		
1630-1715	Preview of Field Trip	Bob McCarley and Jim Hines, LMK
Wednesday		
0700-1000	Travel to Grenada, MS	Jim Hines, LMK
1000-1200	Inspect Vicksburg District and Soil Conservation Service Sites	
1200-1300	Lunch	
1300-1700	Continue Inspection	
1700-1900	Travel to Vicksburg, MS	
Thursday		
	(G) Structural Streambank Erosion Protection Measures (continued)	
	5. River Training Structures (continued)	
0800-0825	c. Retards (including fences)	Charles Elliott, LMVD
0825-0845	d. Hard Points	Warren Mellema, MRD
	(H) Structural Streambed Degradation Protection Measures	
0845-0945	1. Grade Control Structures	Jim Hines, LMK
0945-1000	Break	
	(I) Structural Streambank Failure Protection Measures	
1000-1030	1. Drainage	Laszlo Varga, ORD
1030-1100	2. Slope Flattening and Benching	Joe Keithley, LMM
	3. Restraint Structures	
1100-1130	a. Stone Buttress	Joe Keithley, LMM
1130-1200	b. Gabion and Crib Walls	Laszlo Varga, ORD
	(J) Site Investigation and Environmental Considerations	
1200-1230	1. Hydrology and Hydraulics	Randy Oswalt
1230-1300	Lunch (including questions for panel discussion)	
1330-1415	2. Geology and Soils	Frank Coppinger, NAD
1415-1445	3. Environmental Considerations	Jim Henderson
1445-1500	Break	
1500-1600	(K) Design Procedure	Ted Albrecht
1600-1625	(L) Legislation, Permits, and Legal Problems	Gaylan McGregor, LMK

* WES staff unless indicated.

Time	Subject	Speaker*
Friday		
0800-0845	(M) Instrumentation and Monitoring	Warren Mellema, MRD
0845-0930	(N) Inspection and Maintenance	Charles Elliott, LMVD
0930-0945	Break (including Course Critique)	
0945-1015	(O) Pamphlet, Report to Congress (including Performance Matrices)	Malcolm Keown
1015-1230	Panel Discussion on Practical Experience in Streambank Protection and Questions from Audience	Moderator: Gene Stallings, OCE Panelists: Ted Albrecht Max Lamb, LMVD Warren Mellema, MRD Tasso Schmidgall, SWD Laszlo Varga, ORD SCS Representative
1230-1330	Lunch	
1330-1415	Panel Discussion (continued)	
1415-1430	Closing	Bob McCarley

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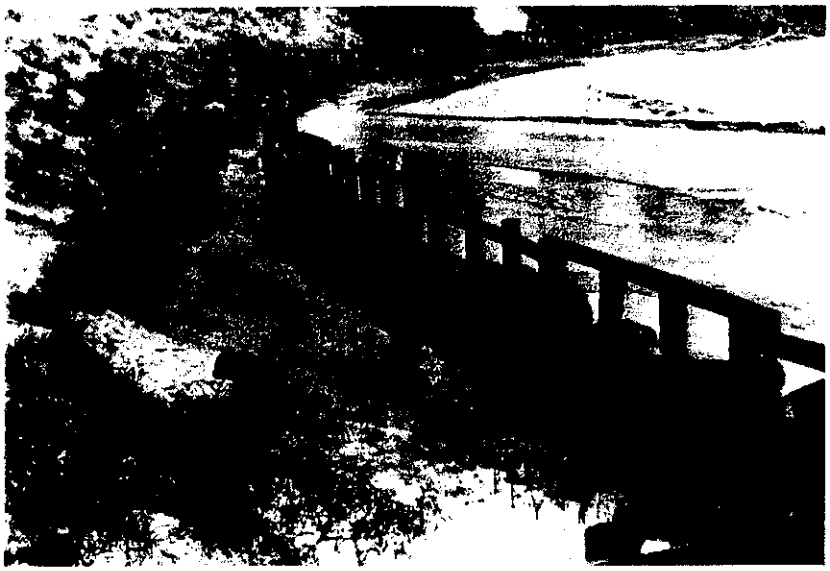
STREAMBANK PROTECTION COURSE

US ARMY ENGINEER
WATERWAYS EXPERIMENT
STATION

VICKSBURG, MISSISSIPPI

2-6 DECEMBER 1985



FIELD TRIP BROCHURE

STREAMBANK PROTECTION COURSE FIELD TRIP

4 December 1985

Location	Site No.*	Reach No.	Type Work	When Built
Canton (Ellison Creek)		--	S.C.S. Grade Control Structure	1976
Grenada				
Perry Creek	1		Longitudinal Stone Dike w/ Transverse stone dikes	1985
Batupan Bogue	6		Transverse Stone Dikes (DEC)	1985
	4	11	Peaked Stone Dike	1977
	4	11	Peaked Stone Dike (Failed)	1977
	4	11	Transverse Dike Tiebacks	1978
	4	11	" " " (Failed)	1978
	4	11	" " " (Repaired)	1985
Perry Creek	2	6B2	Grade Control Structure	1978
	2	28	Longitudinal Stone Dike/Vegetation	1978
	2	28A	" " "/Stone Paving	1981
	3	22A	Board Fence Retard (Failed)	1976
	3	22A	Tire Revetment w/ Stone Toe	1981
	3	--	Stone Bank Paving (SCS)	1984
	5	6B1	Grade Control Structure	1978
	5	7	Tire Post Retards	1978
Charleston				
Tillatoba Creek		--	S.C.S. Grade Control Structure	1978
		1	Stone Dikes w/ Bank Paving	1983
Greenwood Yazoo River	Front Street	1	Longitudinal Stone Dike w/ Gabions	1982
		2	Longitudinal Stone Dike w/ Full Bank Paving	1983

* Site numbers refer to Grenada area map only.

TYPES OF PROTECTION INSTALLED AT THE YAZOO RIVER BASIN DEMONSTRATION PROJECTS

General Descriptions

Site No.

The general physical descriptions of the protective structures used in the Yazoo River Basin are:

Longitudinal stone dikes consist of stone riprap placed parallel to the toe of the streambank and are used to deflect the streamflow and provide toe protection. Areas of the upper bank not covered by stone are sometimes protected by various vegetative treatment methods. — 1, 2

Peaked stone dikes have stone riprap placed parallel to the toe of the streambank and allowed to slope to natural angle of repose. They are used to control the erosion of the toe and induce sedimentation behind the stone. — 4

Board fences with peaked stone consist of a peaked stone dike along the toe of streambank and reinforced with a treated wood piling and timber fence constructed near top bank height. This type of protection deflects and separates the direct flows on the bank and induces sedimentation between fence and bank.

Transverse stone dikes are stone-riprap structures protruding from streambank as "hard points," providing some degree of flow deflection, and used as a tieback for longitudinal stone dikes to prevent flanking of structure and provide areas of sedimentation. — 4, 6

Transverse board fence dikes are composed of treated wood piling and timber fence protruding from bank to deflect flows and provide areas of sediment deposition.

Transverse cable fence dikes have concrete piles with 3/8-in. cables strung between the piling to catch debris and thereby deflect the flows and induce sediment deposition.

Tire revetment is made of used tires tied together with steel banding to form a flexible mattress. The mattress protects the bank from scouring forces of direct flows. — 3

Sand-cement bag revetment consists of biodegradable bags filled with a sand-cement mixture and placed in layers to various bank heights to protect the bank against direct flows.

Wire crib retards, tire-filled, are built of light piling and wire fence cribs placed parallel to the bank and filled with used tires. They deflect flows away from bank and induce sedimentation between the cribs and banks. Some have a stone or sand-cement bag toe protection.

Wire crib retards, hay-filled, are similar to tire cribs but filled with hay bales.

Tire post retards have light piling driven through the center of a stack of used tires, capped with a board railing, and placed parallel to toe of streambank. They are used to divert flows from the bank and induce deposition. — 5

Vegetative treatment consists of seeding with various types of grasses and mulch and/or planting of woody vegetation. — 1, 2, 4, 5, 6

Grade-control structures consist of a sheet-pile weir, with an energy-dissipating baffle and a naturally occurring or preformed, riprap-lined scour hole. The purpose of these structures is to halt the migration of head-cuts. Structures built early in the program have scour holes lined with various sizes of graded riprap; but later structures have only the sheet-pile weir and baffle, allowing the forces of the stream to develop the scour hole. The Goodwin Creek Watershed grade-control structures were specifically designed to measure total sediment load and discharge, concurrent with the control of bed degradation. — 2, 5

Following are a few remarks regarding the descriptions on the previous page:

(1) Longitudinal stone dikes and peaked stone dikes are very similar. A peaked stone dike is merely a specialized type of longitudinal dike, the difference being that peaked stone dikes were constructed without bank grading or transverse tiebacks, and were generally used where erosion potential was low and/or where vegetation was or could be established for upper bank protection.

(2) Often two or more types of protection are combined at a site. Examples are longitudinal stone dikes on the lower bank combined with tire revetment and/or vegetation on the upper bank, longitudinal stone dikes used with transverse stone dikes as tiebacks, various structural methods combined with vegetative treatment, etc. Therefore, the classification of type of work at a given site may be somewhat less straightforward than the listing indicates.

(3) Even though not listed, full-bank stone paving was used in lieu of more experimental slope treatments at a few sites where consequences of failure were severe; for instance, in the vicinity of some grade control structures.

ELLISON CREEK GRADE CONTROL STRUCTURE

This structure was designed and built by the Soil Conservation Service in 1976 at a cost of $239,033. Channel improvement work including several reservoirs, channel straightening, and enlargement was completed in 1962. Subsequent degradation led to the construction of several trapezoidal weirs in 1963. These structures remained effective until the abnormally wet spring of 1973. The present structure replaced one of the original weirs which failed during this period.

Known as a straight drop spillway stilling basin, Type C, this structure has several advantages, with low cost and ease of construction being primary. The structure can be built with relatively little excavation and dewatering, and the use of sheet piles precludes many of the weather-related problems associated with concrete.

STRUCTURE DATA

Vertical drop - 9.5 feet

Weir crest length - 30 feet

Stilling basin length - 82 feet

Uncontrolled drainage area - 13.7 square miles

Design flow - 2300 c.f.s. (bank-full flow)

Construction - heavy sheet pile with concrete floor and baffle blocks

General Upstream View
September 1983

Closeup Upstream View
September 1983

General Downstream View
September 1983

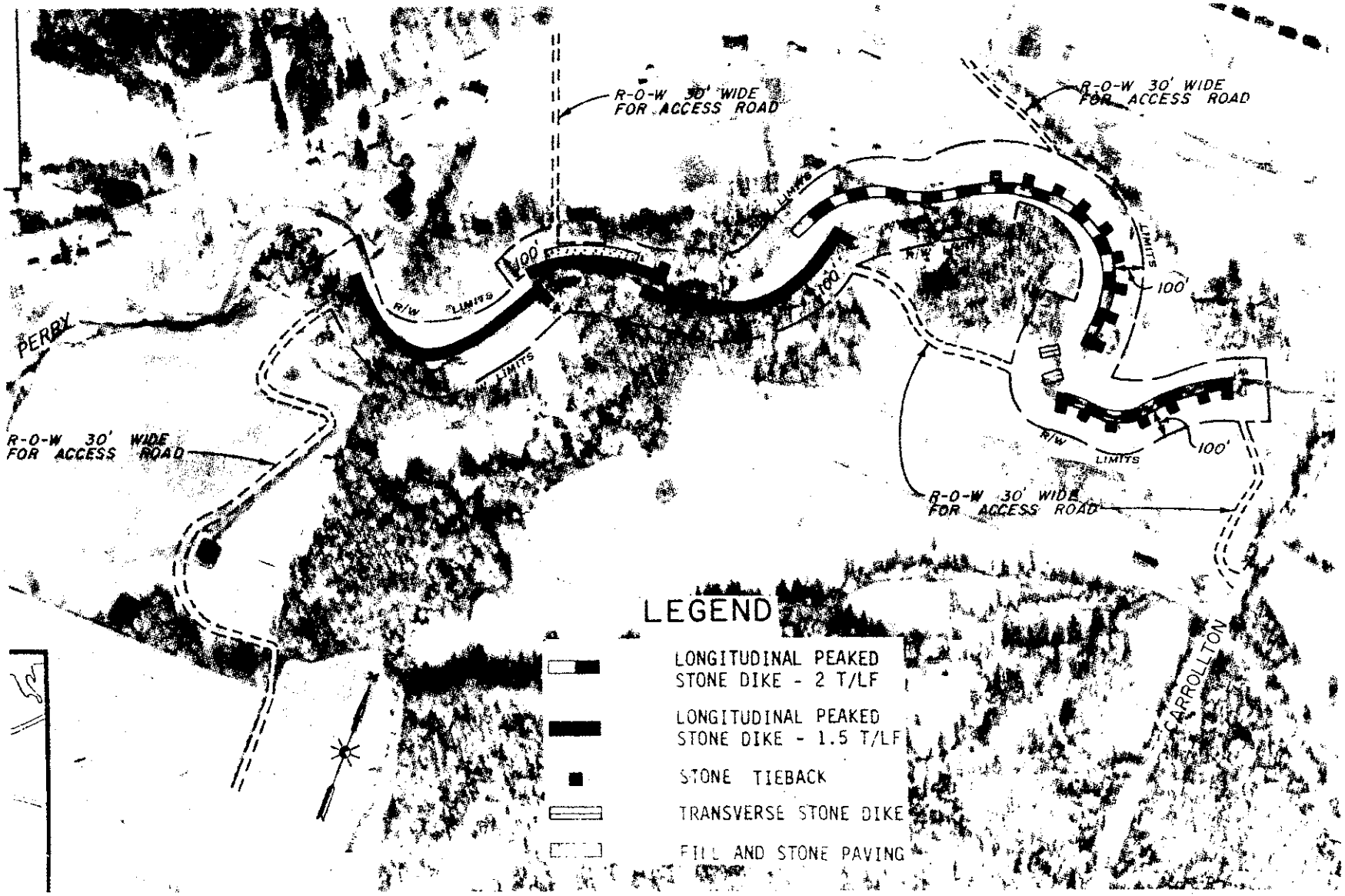
SITE 1

Perry Creek

Perry Creek as suffered through a series of headcuts for the past 35 years. These were the result of base level lowering at the mouth, where Perry Creek joins Batupan Bogue. As in most cases of base level lowering, the immediate result was bed degradation, which, in turn, led to bank instability. This site illustrates the final stages of this process: a channel greatly oversized for the hydrologic events which the basin experiences.

These problems at this particular site were made even more severe by the poor alignment of the reach. The bank caving which began with the bed degradation was aggravated by the two 90-degree bends in the reach. These cause the bank to receive the full brunt of the flow with little natural curvature to ease the flow through the turn.

The bank protection in place was completed in 1985 under the Demonstration of Erosion Control Program, a joint effort of the Corps of Engineers and the Soil Conservation Service. Although named as a "demonstration" project, as was the Section 32 Project, the structures built under this program are not intended to be experimental. The structure consists of two parts: bank paving to eliminate bank erosion from the direct flow impact, and dikes to create a better approach by directing the flow away from the bank. The dikes extend downstream through the bend to move the maximum scouring forces away from the toe of the slope. Willows are planted between the two upstream dikes. When established, the willows slow the flow velocity, thus encouraging deposition.


R-O-W 30' WIDE FOR ACCESS ROAD
R-O-W 30' WIDE FOR ACCESS ROAD
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R-O-W 30' WIDE FOR ACCESS ROAD
R/W LIMITS
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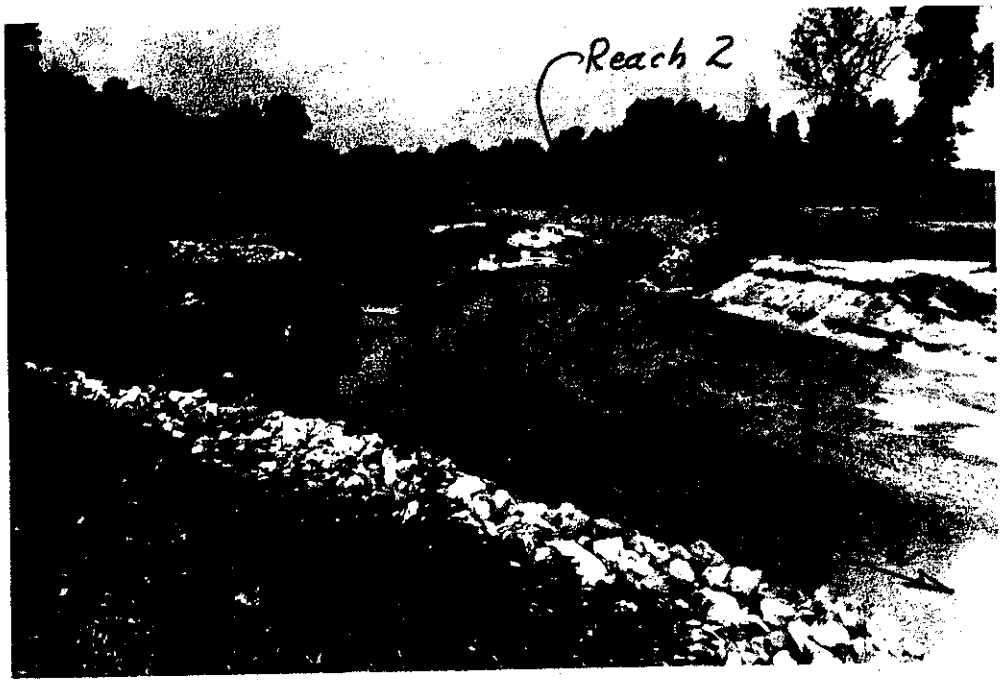
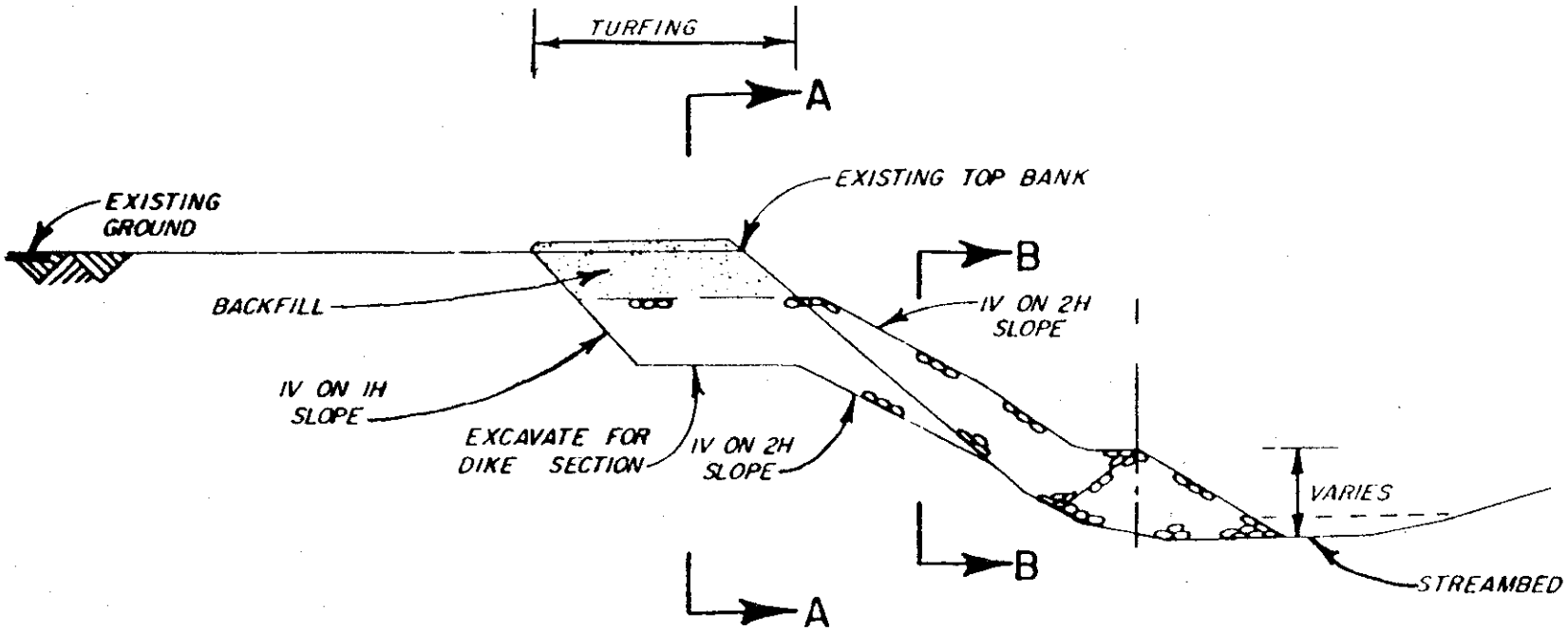
100'
PERRY
CARROLLTON
LEGEND
LONGITUDINAL PEAKED STONE DIKE - 2 T/LF
LONGITUDINAL PEAKED STONE DIKE - 1.5 T/LF
STONE TIEBACK
TRANSVERSE STONE DIKE
FILL AND STONE PAVING

Perry Creek D.E.C., Reach 1, May 1985

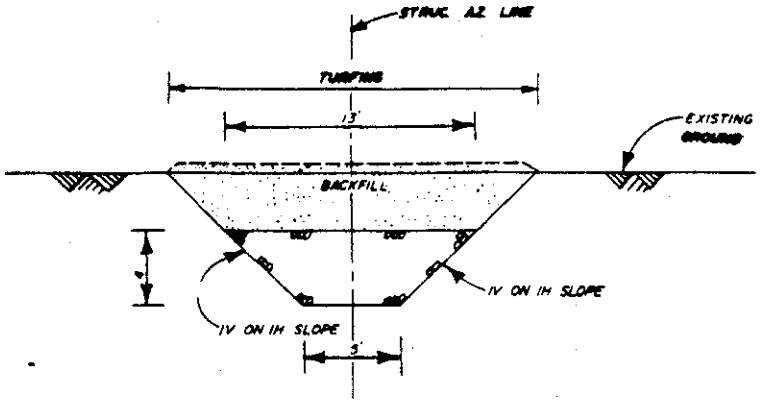


Perry Creek D.E.C., Reach 2, May 1985



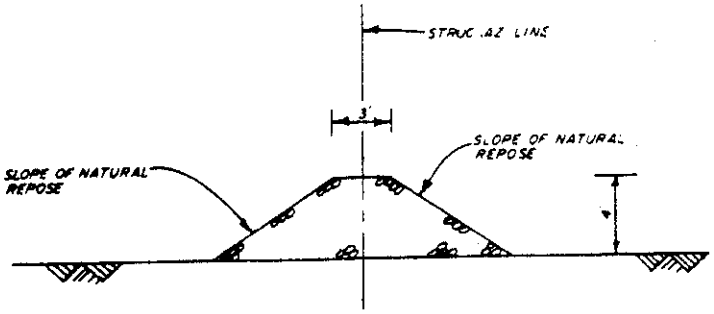
PROFILE

TYPICAL STONE TIEBACK



SECTION A-A

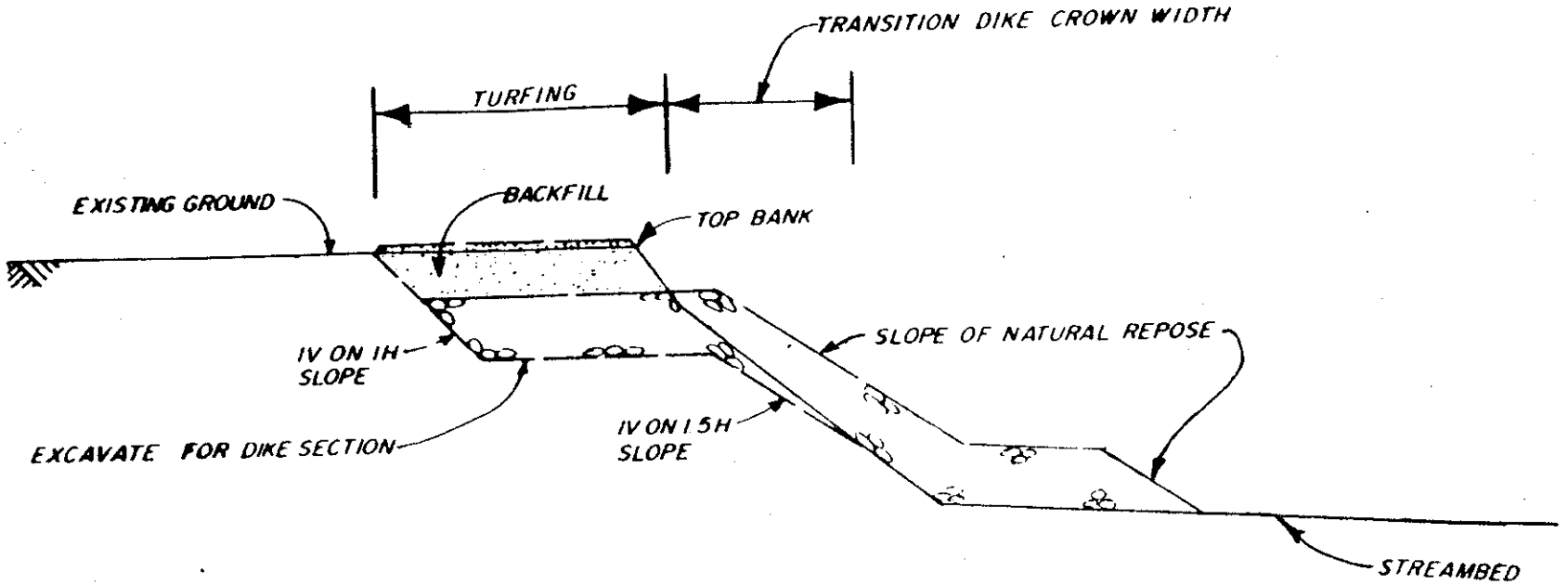
WHERE TIEBACK TIES INTO LOW TOP BANK, TOP OF STONE WILL BE FLUSH WITH EXISTING GROUND AND NO BACKFILL WILL BE REQUIRED



SECTION B-B

TYPICAL SECTIONS FOR STONE TIEBACK AND TRANSVERSE STONE DIKE

NO SCALE



TYPICAL TRANSVERSE STONE DIKE PROFILES

SITE 2 - PERRY CREEK
Grade Control Structure 6B2

Overall View Looking Upstream

Closeup Just After Construction

PERRY CREEK GRADE CONTROL STRUCTURES

PROBLEM: Degradation of Batupan Bogue lowered base level of Perry Creek, initiating a series of headcuts. These headcuts proceeded upstream until they reached the box culvert under Interstate Hwy 55, which suffered severe damage. Grade control structures were built to restrict the migration of headcuts and to generally improve bank stability.

TYPE OF STRUCTURE: Sheet pile weir with riprap-lined stilling basin (preformed scour hole)

CONSTRUCTION MATERIAL: Steel sheet pile and rock

DRAINAGE BASIN AREA: 18.7 square miles

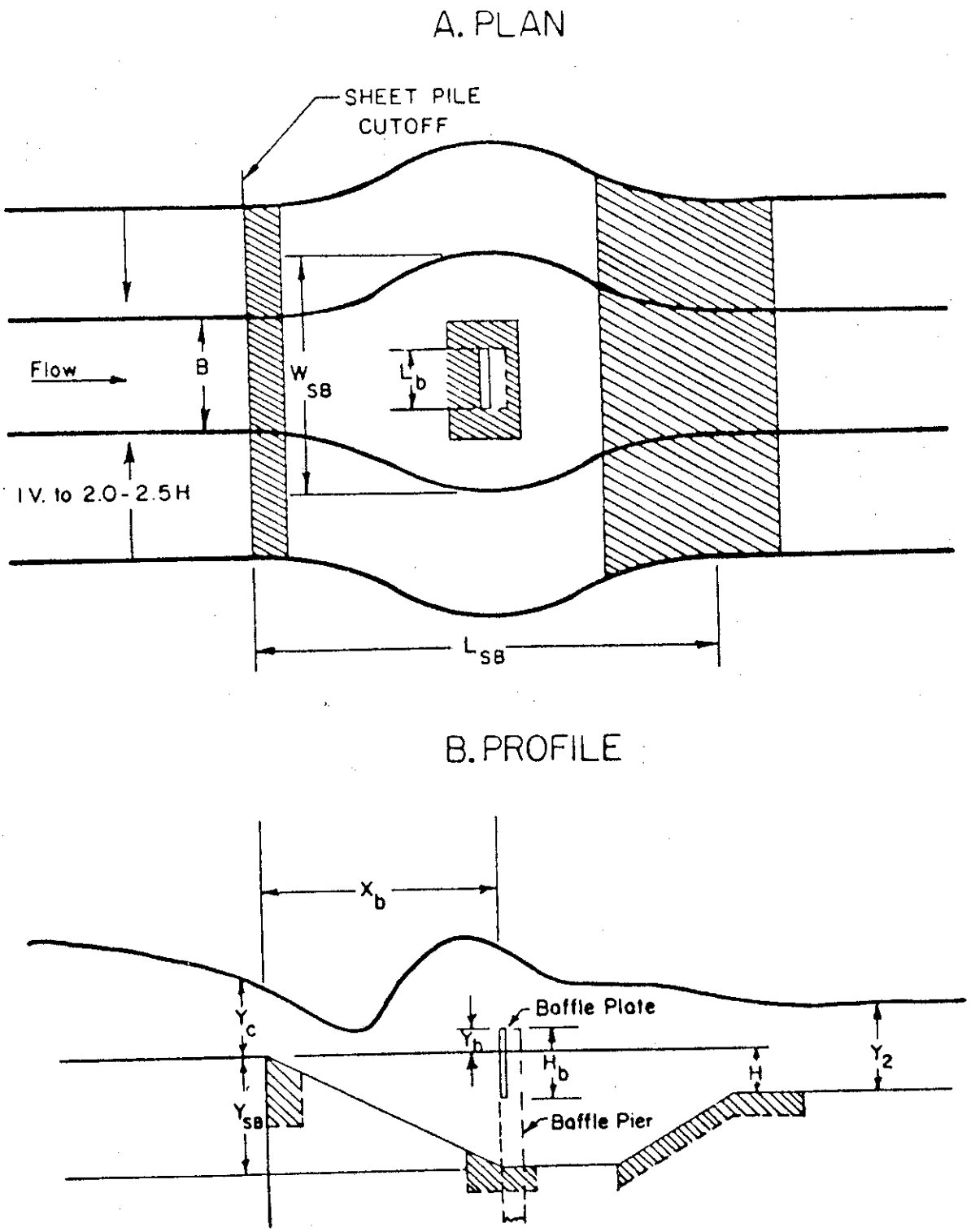
DESIGN DISCHARGE: 4,400 cfs

STRUCTURE DATA:

	6B1	6B2
Drop Height	2 ft	2 ft
Inlet Channel Bottom Width	75 ft	50 ft
Inlet Channel Depth	10 ft	12 ft
Exit Channel Bottom Width	75 ft	60 ft
Exit Channel Depth	12 ft	18 ft
Length of Weir	280 ft	95 ft
Type of Baffle	Sheet pile cell filled with grouted riprap	Sheet pile baffle plate attached to H piles
Width of Baffle	40 ft	26 ft
Cost	$597,000	$105,000

Design of these structures was developed by the Agricultural Research Service (ARS) Laboratory in Oxford, Mississippi, in conjunction with the Vicksburg District, U.S. Army Corps of Engineers. Model studies were performed to develop the criteria for placement of the baffle. The initial weir size is determined by the design discharge. Examples of the criteria follow on subsequent pages.

Following construction, at least two headcuts reached the structure, which effectively stopped their movement. Evidence may be seen of some stone launching at the downstream limit of the bed protection.



Basin Dimensions and Riprap Placement (Shaded Areas)

TENTATIVE DESIGN CRITERIA

Results of the previously described tests were used to establish guidelines for the design of low drop grade control structures. These are summarized below. Reference should be made to the preceding figure.

STILLING BASIN DIMENSIONS

Stilling Basin Width

The stilling basin width, W_{SB}, is:

$$W_{SB} = 2B$$

were W_{SB} is the maximum width of stilling basin at the elevation of the weir crest and B is length of weir crest.

Stilling Basin Length

The stilling basin length, L_{SB}, is:

$$L_{SB} = 2\ X_b$$

where L_{SB} is the length of stilling basin, measured from the weir to the beginning of the downstream channel. X_b is defined as:

$$\frac{X_b}{Y_c} = 3.54 + 4.26\ \left(\frac{H}{Y_c}\right)$$

where H is absolute drop height and Y_c is critical depth.

Stilling Basin Depth

The depth of the stilling basin, Y_{SB}, is:

$$Y_{SB} = 1.0\ Y_c + H$$

where Y_{SB} is the depth of stilling basin, measured from the crest of the weir and H and Y_c are as previously defined.

Stilling Basin Side Slopes

The side slopes of the basin, S_B, are:

$$1{:}2.0 < S_B \leqq 1{:}2.5$$

where S_B is the side slope (vertical to horizontal) of the stilling basin.

BAFFLE DIMENSIONS

Baffle Length

The baffle length, L_b, is:

$$L_b = B/2$$

and is centered in the basin, where L_b is the length of baffle pier or plate.

Baffle Height

The baffle height is pertinent only to the baffle plate since the baffle pier extends into the bottom of the basin. The baffle plate height is given by

$$H_b = Y_c$$

where Y_c is as previously defined.

Height of Baffle Above Weir Crest

The height of the baffle above the weir crest, Y_b, is:

$$Y_c/4 \leq Y_b \leq Y_c/3$$

where Y_c is as previously defined.

RIPRAP SIZE AND PLACEMENT

The size of riprap required for stability of the stilling basin was determined using criteria developed by Anderson, et al. (1970), and later modified by Blaisdell (1973). The critical depth, Y_c, and velocity, V_c, were used as the characteristic parameters to calculate minimum riprap size needed to protect the stilling basin. Based on observations of the model tests, these criteria seemed adequate. However, no specific model tests were conducted. Areas within the stilling basin that are the most vulnerable to attack are indicated by the hatched area in Figures A and B.

SITE 2 - PERRY CREEK
Reach 28 (looking downstream)
Marginally Successful Longitudinal Stone Dike with
Upper Bank Sloped, Seeded, and Mulched

Before
Construction
March 1977

After
Construction
January 1979
(some upper bank
erosion occurring)

September 1982

SITE 2

Reach 28

This reach demonstrates the use of vegetative treatment on upper bank slopes in lieu of more expensive structural protection. The performance here is at neither extreme or total failure or complete success. Unfortunately, the effectiveness of vegetative treatment is subject to factors over which the designer has limited control: severity of flows before the vegetation is established, season of the year that planting is done, and soil conditions.

The concept of selective maintenance is also illustrated. Although erosion has occurred and the original seeding was not very effective, little top bank recession resulted and volunteer vegetation is slowly becoming established. Therefore, no maintenance has been performed, although close surveillance should be continued until vegetation is completely established.

The cross section drawing shows a cut and fill bank grading technique. This is economical construction, eliminating the problem of disposal of graded material. However, unless the fill on the lower bank is completely protected by the stone toe, it is vulnerable to erosion before vegetation becomes established.

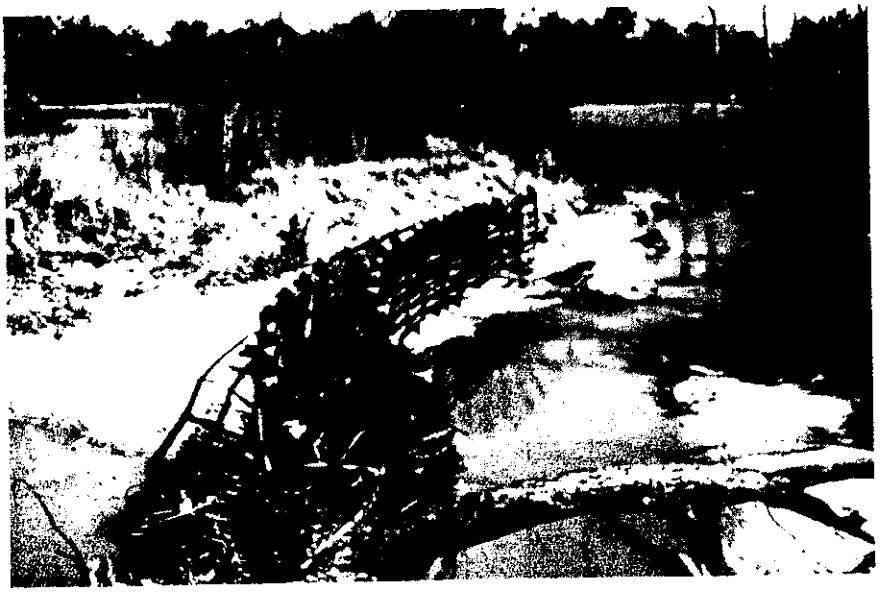
SITE 2 - PERRY CREEK
Reach 28-A (looking upstream)
Successful Longitudinal Stone Dike with Full-Bank Stone Paving

Bank Caving Threatening Approach to Structure
March 1981

Extension of Stone Bank Paving, Using
Longitudinal Stone Dike Toe Protection
September 1982

SITE 3 - PERRY CREEK
Reach 22-A (looking downstream)
Unsuccessful Board Fence Retard (without stone toe or tiebacks)
Constructed in 1976 by SCS
Successfully Replaced with Tire Revetment with Stone Toe and Peaked Stone Dike

Board Fence Retard
Being Flanked
March 1980



Completely
Flanked
and
Ineffective
July 1980

After Replacement
with Tire Revetment
with Stone Toe
September 1983

SITE 3

Reach 22-A

The importance of secure terminations at the upstream and downstream ends of bank protection cannot be over emphasized. The original board-fence retard was flanked by bank caving. The lack of transverse tiebacks to the bank allowed rapid channel development behind the structure, which was soon completely across the channel and partially buried by the encroaching point bar. The remnants were removed during construction of the present revetment in 1981.

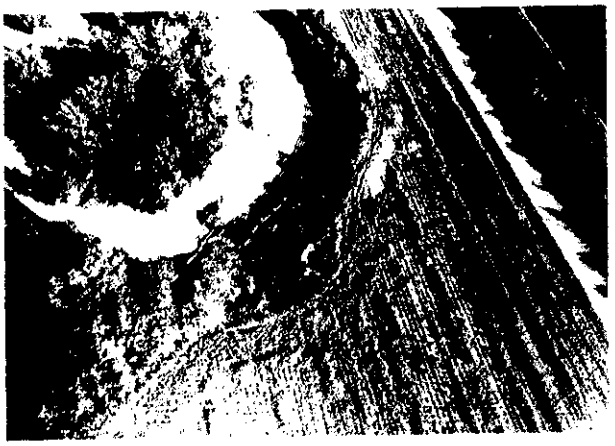
The revetment which was constructed in 1981 illustrates the use of a combination of techniques for economy and effectiveness. A longitudinal stone dike was used for maximum toe protection with less expensive used tire matting on the upper bank slope.

A more unfortunate situation is also illustrated by this revetment. Apparently settlement of fill material in the vicinity of overbank drainage ditches led to a separation at the junction of the stone ditch paving and used tire slope paving, which was then probably aggravated by eddy currents during high flows because of the bankline irregularity at this point and by flow in the ditch. Careful attention to overbank drainage ditches during design and construction is worthwhile, even though the overall success of the revetment may not be at stake.

This revetment design has thus far been successful in every case. At one site, used tire revetment was successful without stone toe protection. However, at two other sites without a stone toe, partial or total failure resulted when high flows occurred soon after installation.

The upstream end of the revetment is a peaked stone dike, selected because the high steep bluff bank made grading the bank for tire revetment impractical. The existing bluff bank material is fairly erosion-resistant, and protecting the lower part of the bank with stone should be sufficient.

SITE 3 - PERRY CREEK
Downstream of Reach 22-A
Board Fence Retard (without stone toe of tiebacks)
Constructed in 1976 by SCS



General Aerial View
January 1983

Upstream View
September 1983

Overbank Drainage
September 1983

Downstream End
September 1983

SITE 3

Board Fence Retard

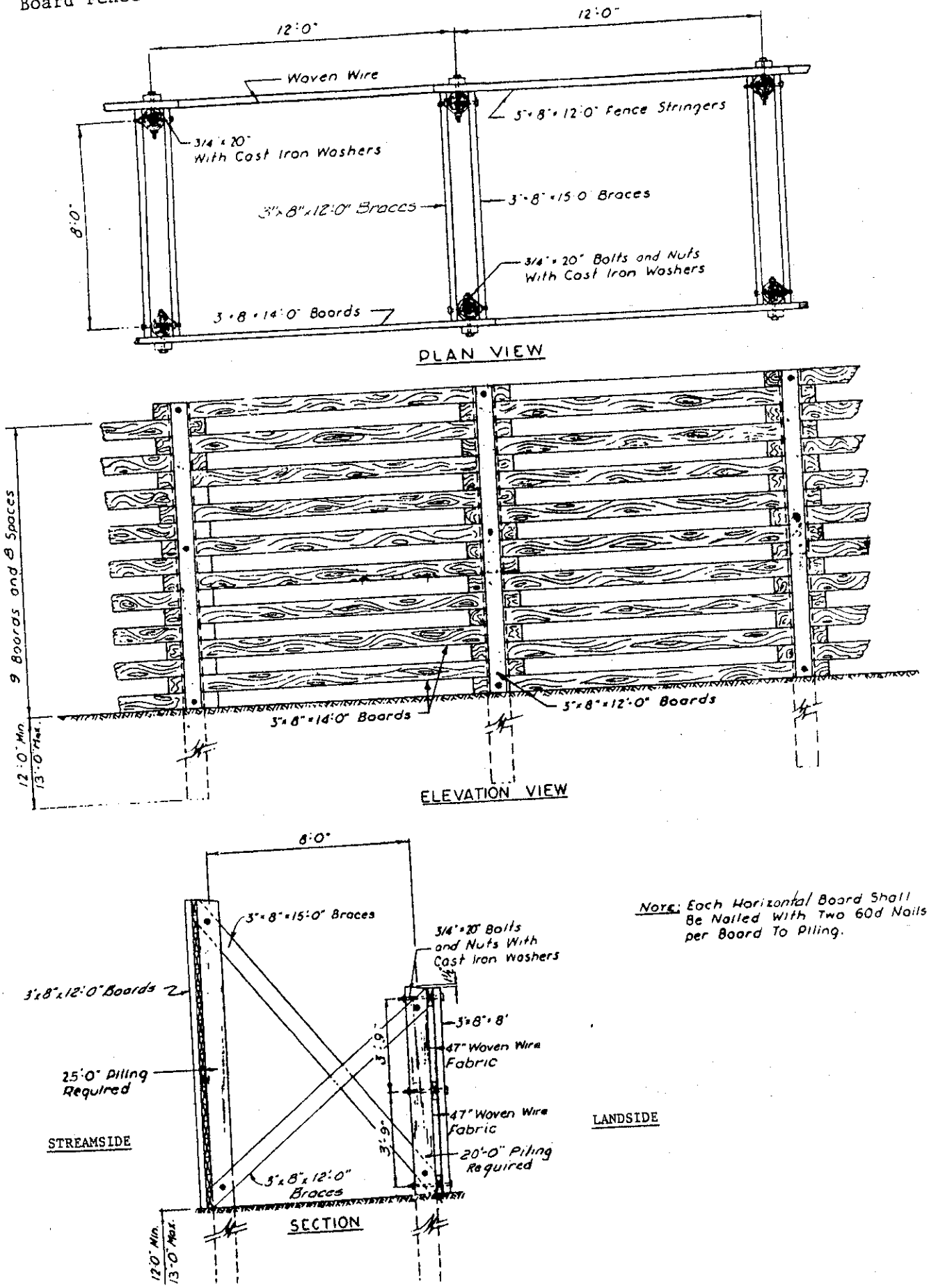
Downstream of Reach 22-A

This board fence retard design has been widely used by the Soil Conservation Service in the Yazoo Basin and elsewhere with considerable success. However, enough problems similar to those at Reach 22-A have been encountered that the present trend is to use stone revetments or concrete jacks in areas of severe attack.

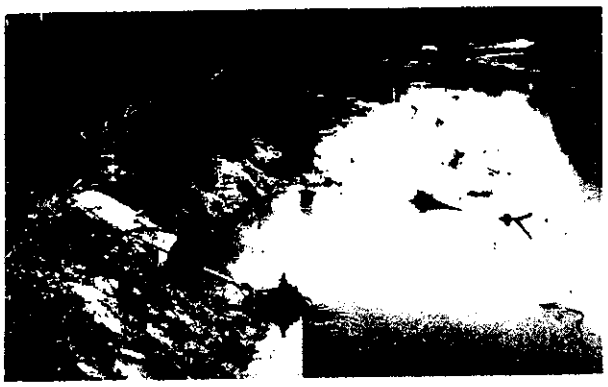
The importance of continued surveillance is illustrated here. This structure was successful for six years, but high flows in late 1982 and early 1983 have initiated severe caving at the upstream end, aggravated by overland flow returning to the channel. This is the beginning of a flanking action similar to that which occurred just upstream. Due to the proximity of a public road, rehabilitation/reinforcement should be considered prior to further deterioration.

SITE 3 - PERRY CREEK

Board Fence Retard Without Stone Toe, Design by Soil Conservation Service



SITE 4 - BATUPAN BOGUE
Reach 12 (looking upstream)
Unsuccessful Peaked Stone Dike
Successfully Reconstructed with Transverse Stone Dikes (Tiebacks) and Willow Plantings



Before Construction
January 1977

After Construction
Fall 1977

After Flood Damage
July 1978

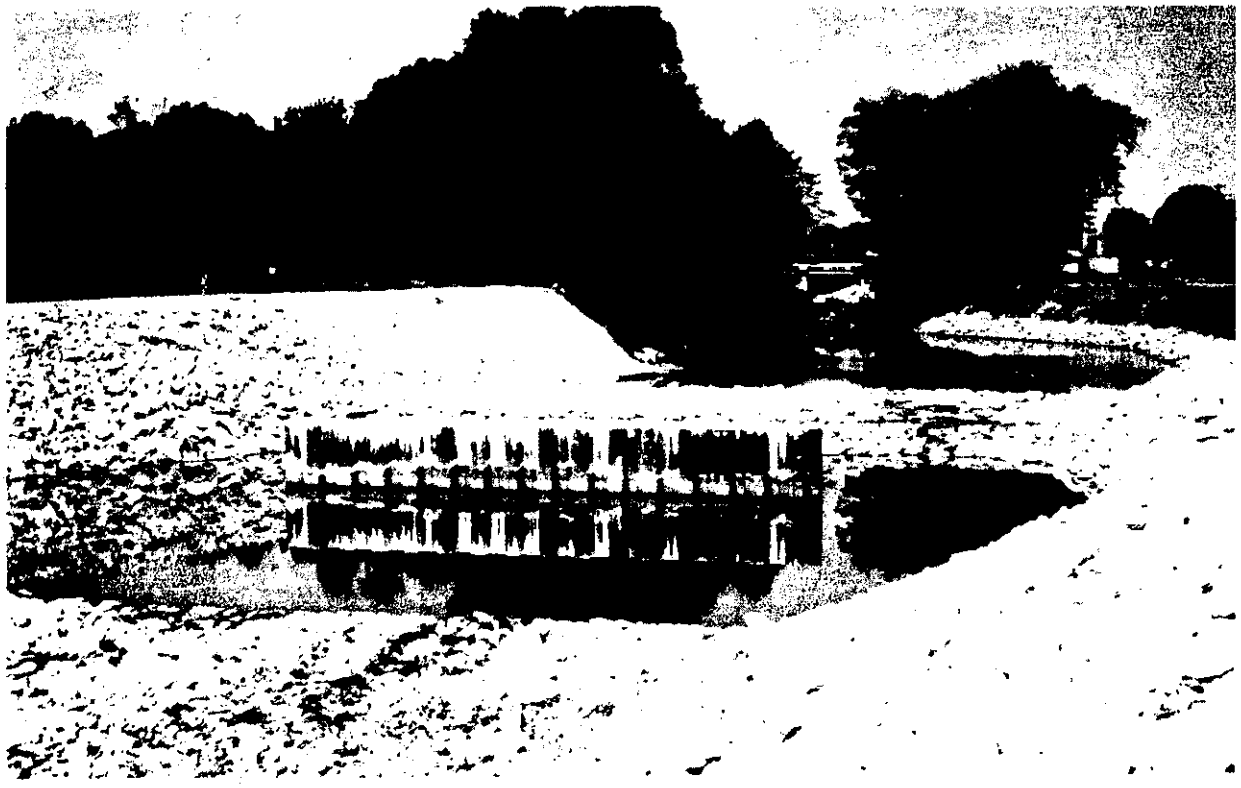
After Reconstruction
October 1978

After High Flows
January 1979

September 1982

SITE 5 - PERRY CREEK
Grade Control Structure 6B1

Note: This aerial view is of Structure 3-A on North Fork Tillatoba Creek, but is typical of Structure 6B1 on Perry Creek.



6B1 Looking Upstream

SITE 5 - PERRY CREEK
Reach 7 (looking downstream)
Successful Tire Post Retards

Before
Construction
March 1978

After
Construction
June 1978

September 1982

SOUTH FORK TILLATOBA CREEK
S.C.S. GRADE CONTROL STRUCTURE

In the 1960's, Middle Fork Tillatoba Creek was determined to be in need of stabilization, primarily as a result of the degradation of South Fork Tillatoba, into which it flows. This is a classic example of sequential base level lowering progressing throughout an entire system by the process known as headcutting. Designs were prepared for several types of grade control structures by the Soil Conservation Service and were modeled to determine the most appropriate. The two-level structure was designed to allow the lower structure to pass all in-bank flows (one-year frequency, or less). The auxiliary (high) structure is set at an elevation which will become operational for flows having a frequency of from two to fifty years. The stilling basin floor of the high-level structure is set at an elevation which will allow some degradation of the exit channel. Two additional structures have been considered for installation upstream of this site.

Following completion of the structure in 1979, various channel changes which were somewhat unexpected have taken place: the channel upstream of the structure has more than doubled in mean width and an active cycle of degradation with increased meandering is occurring. A geomorphic study of the stream was commissioned in April 1982 to determine the effectiveness of the structure in meeting its objectives of (1) preventing the upstream migration of headcuts, (2) providing flow control which would allow the rehabilitation of the reach downstream of the structure, and (3) rehabilitating the reach upstream of the structure. The study found that the first two objectives had apparently been met, at least for the present. However, the third objective has not been met since the channel is still deteriorating. Several contributing factors were noted in the report as follows: (1) The slope at which deposition will occur (0.0014) is less than the pre-construction slope (0.0018), but is still twice the design slope (0.0007). The depositional slope is a function of the shortening of the channel caused by construction and a meander bend cutoff. The only way the channel slope can be reduced is by lengthening of the channel by meandering. (2) The storage function of the structure for flows having a frequency of between one and fifty years causes backwater conditions which lead to bank saturation and subsequent failure as the stage recedes. (3) Excessive base flows and channel slope remove the failed bank material, thus leading to further widening, rather than an effective decrease in bank slope. The main conclusion of the report was that "unless strict attention is paid to maintaining the pre-existing channel and valley slope and length ratios (i.e., sinuosities), emplacement of the Type "C" structure has two mutually exclusive results. The storage provided by the structure is beneficial to the downstream reaches but detrimental to the upstream reaches. If pre-existing channel length cannot be maintained, then it appears that the system would be better served by replacing the Type "C" structure with a flow-control reservoir." [1/]

[1/] "Geomorphic Study of Middle Fork, Tillatoba Creek, Mississippi", Soil Conservation Service (Mississippi) Contract No. 53-44423-1-221, Harvey, M.A., Watson, C.C., Schumm, S.A., Earth Resource Department, Colorado State University, Fort Collins, CO, July 1983.

STRUCTURE DATA

Type of Structure - Separated 2 stage, straight drop spillway stilling basin, Type C

Construction material - Reinforced concrete (approximately 3,130 cubic yards)

Drainage area uncontrolled - 29.1 square miles

Drainage area controlled - 10 square miles

Low stage structure design discharge - 3,750 c.f.s.

Total capacity, both structures - 14,000 c.f.s.

Low stage structure dimensions:

13' vertical drop
34' weir crest
120' long stilling basin

High stage structure dimensions:

8.5' vertical drop
114' long weir crest
38' long stilling basin

Emergency spillway length - 250'

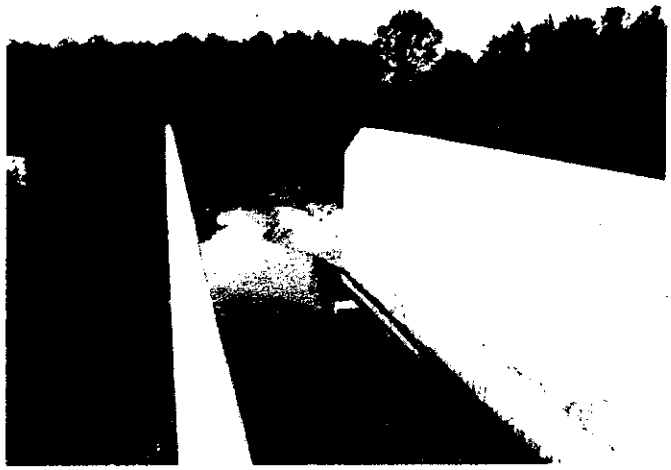
Maximum design discharge - 18,130 c.f.s. (100-year frequency)

Total construction cost - $1,492,788.94

General View
January 1983

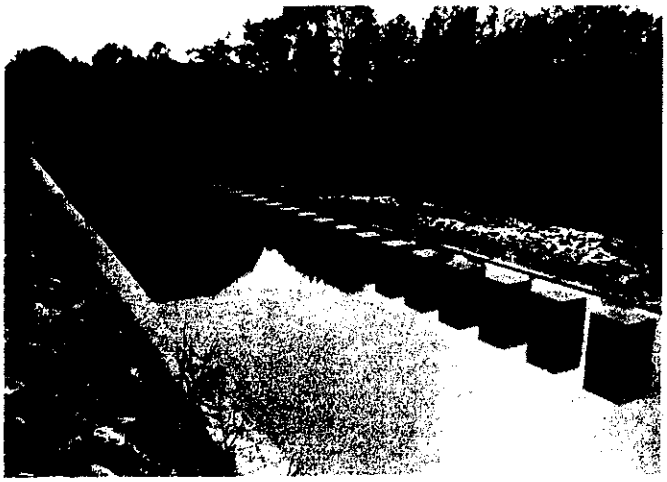
Upstream View of Low Stage Structure
September 1983

General Upstream View
January 1983

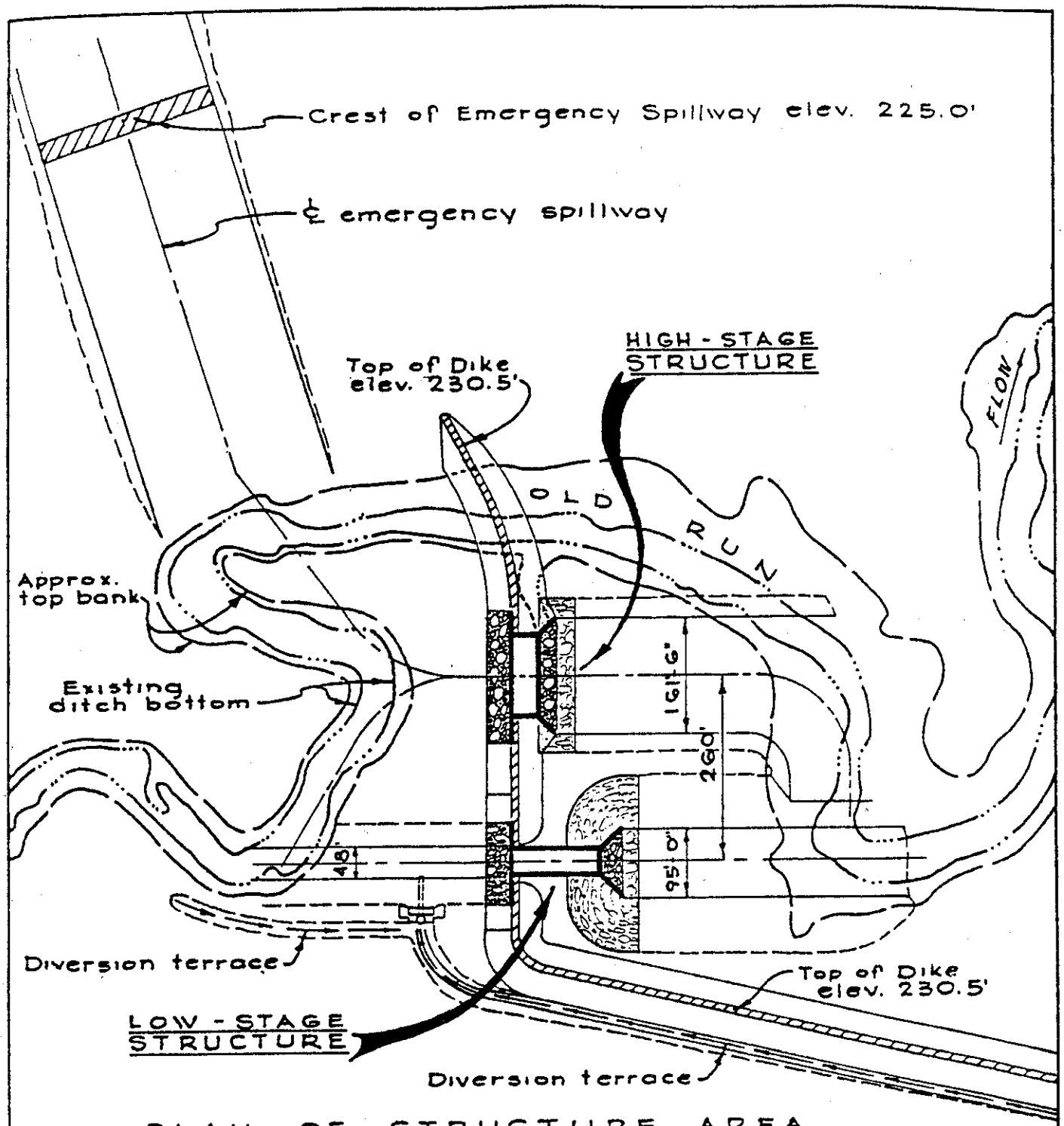


Downstream View of Low Stage Structure
September 1983

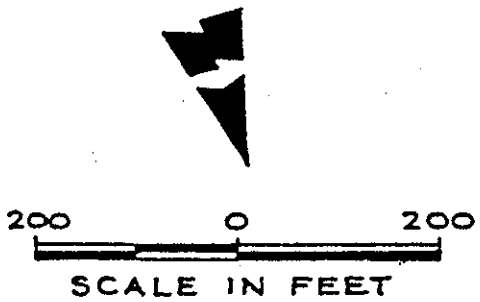
General Upstream View from Low Stage Structure
September 1983



Downstream View of High Stage Structure
September 1983



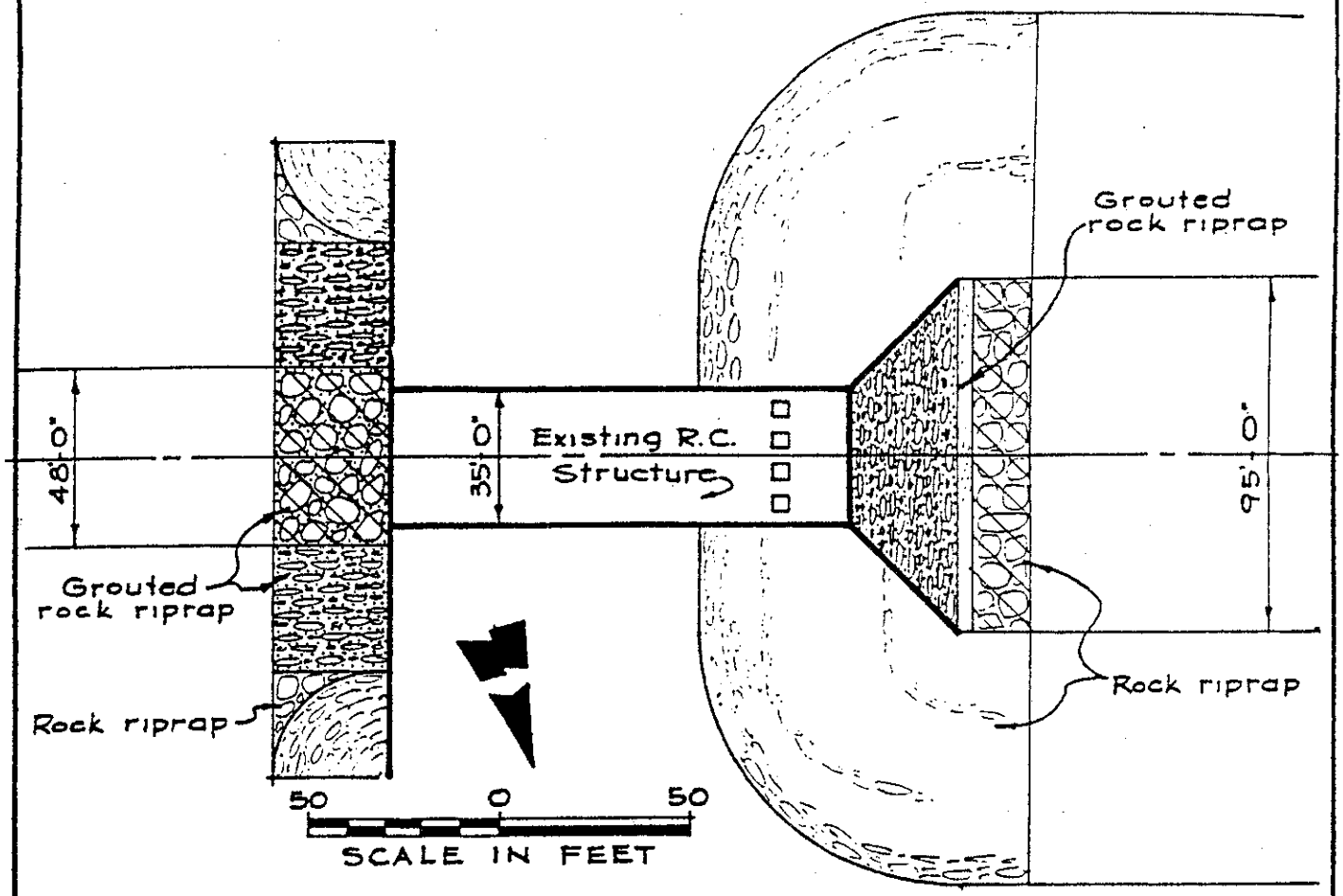
PLAN OF STRUCTURE AREA



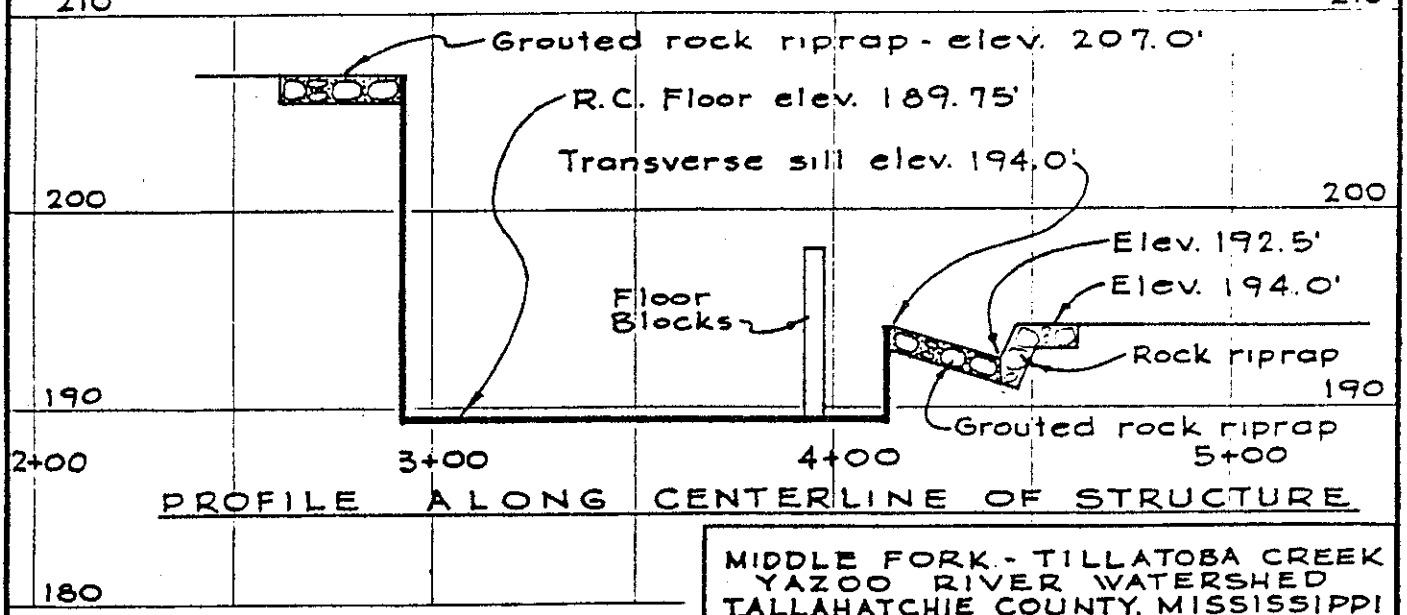
MIDDLE FORK - TILLATOBA CREEK
YAZOO RIVER WATERSHED
TALLAHATCHIE COUNTY, MISSISSIPPI
GRADE CONTROL STRUCTURE - SITE MAP

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

Designed SMITH & SANDERS	Date 8-76	Approved by
Drawn B.L. AUSTIN	9-78	Title
Traced		Date
Checked K.M. HAYWARD	9-78	Sheet No. of / Drawing No.



PLAN - LOW STAGE STRUCTURE



PROFILE ALONG CENTERLINE OF STRUCTURE

MIDDLE FORK - TILLATOBA CREEK
YAZOO RIVER WATERSHED
TALLAHATCHIE COUNTY, MISSISSIPPI
LOW STAGE STRUCTURE - PLAN & PROFILE

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

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Drawn: B. L. AUSTIN	9-78	Title:	
Traced: K. M. HAYWARD	9-78	Sheet No. of	Drawing No.
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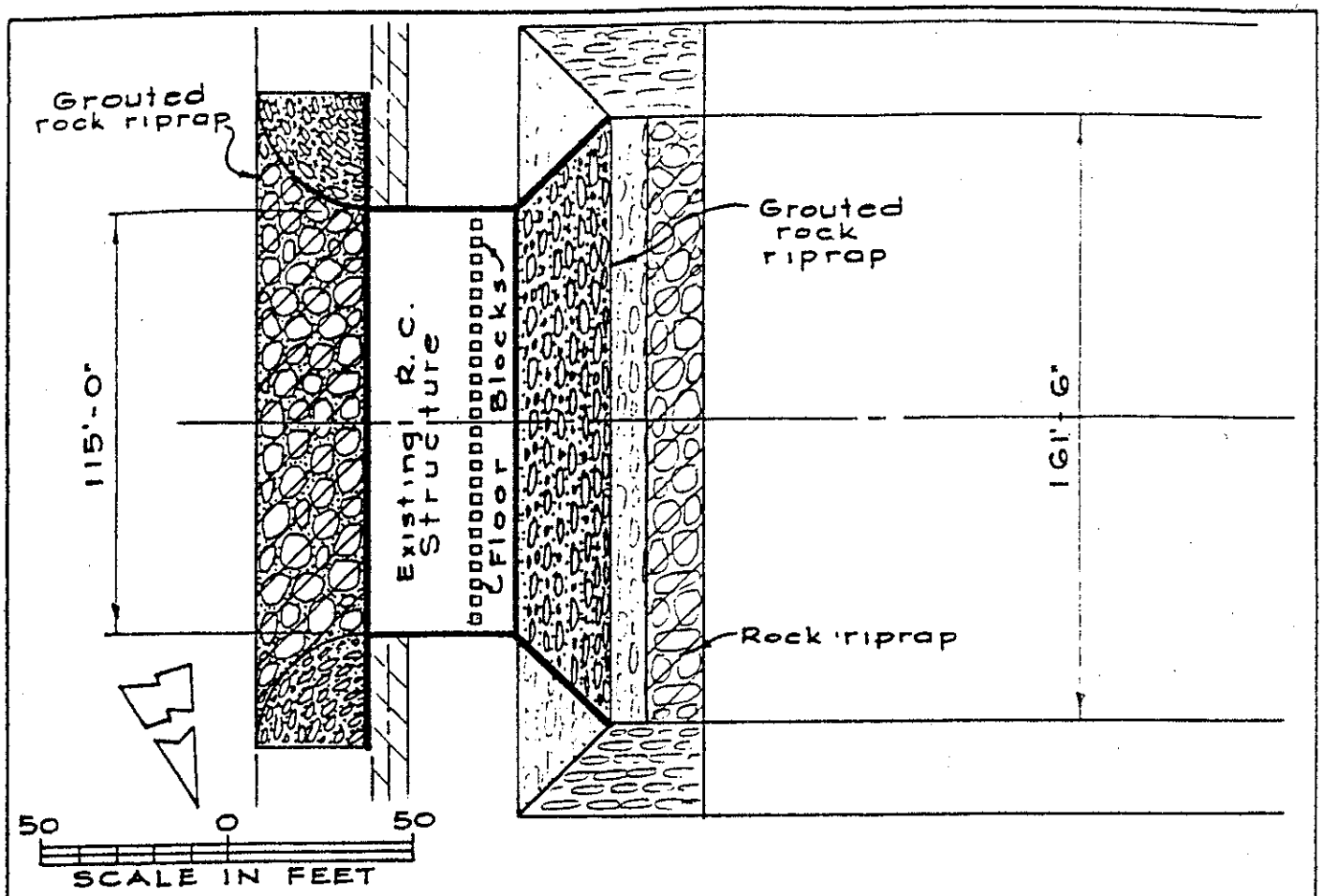
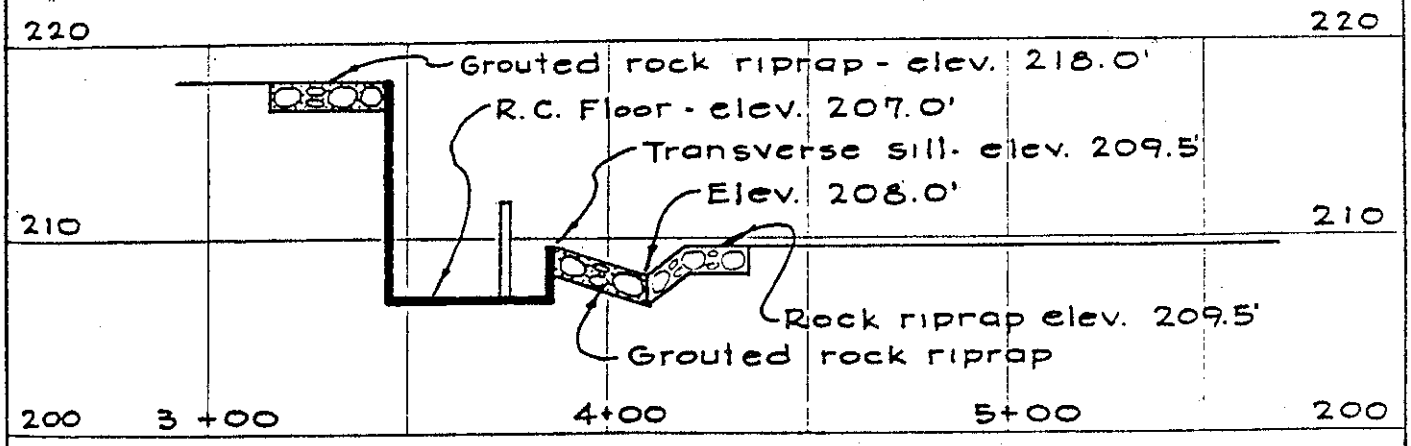


PLAN - HIGH STAGE STRUCTURE



PROFILE ALONG CENTERLINE OF STRUCTURE

MIDDLE FORK - TILLATOBA CREEK
YAZOO RIVER WATERSHED
TALLAHATCHIE COUNTY, MISSISSIPPI
HIGH STAGE STRUCTURE - PLAN & PROFILE

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

Designed: SMITH & SANDERS	Date 8-76	Approved by:	
Drawn: B.L. AUSTIN	9-78	Title:	
Traced:		Title:	
Checked: K.M. HAYWARD	9-78	Sheet No. of	Drawing No.

YAZOO RIVER
Greenwood, Mississippi
Front Street

Before
Construction
April 1982

After Construction
November 1982

Downstream View - After Construction
September 1983

Closeup View - After Construction
November 1982

SITE 1 - PERRY CREEK
Reach 33 (looking downstream)
Rehabilitated with Tire Post Retards with Stone Toe Protection, May or May Not Be Successful

Before Construction March 1977

After Flood Damage January 1979

After Reconstruction February 1982 (most of stone toe already launched)

September 1983

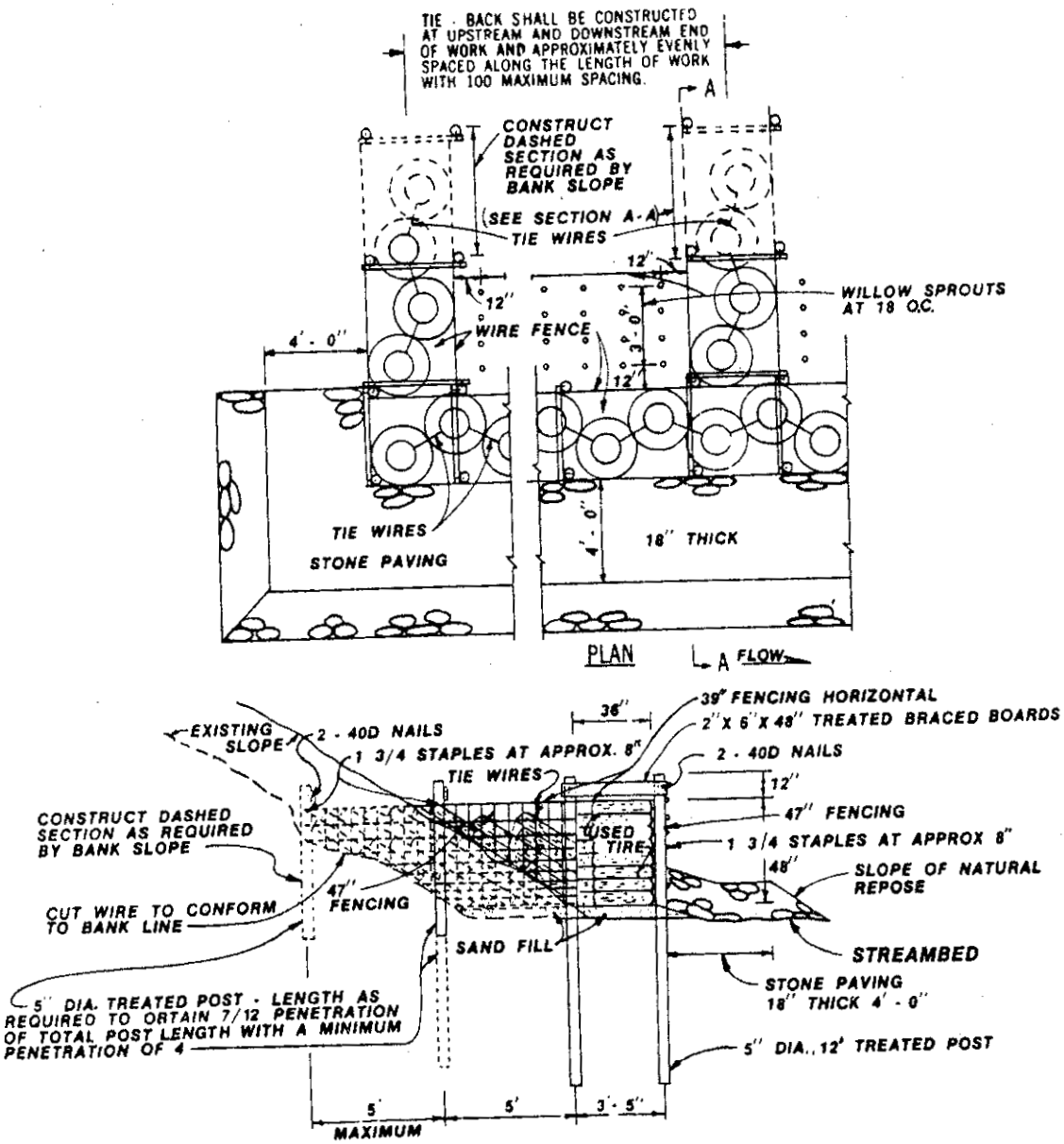
SITE 1 - PERRY CREEK
Reach 35 (looking upstream)
Successful Wire Crib Retards

Before
Construction
March 1977

After
Construction
January 1979

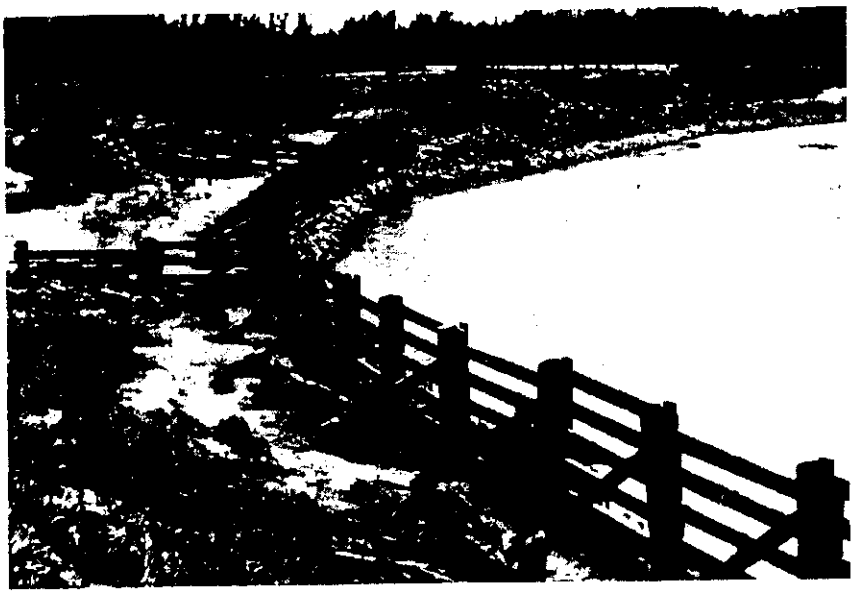
Present
September 1983

SITE 1 - PERRY CREEK
Reach 35



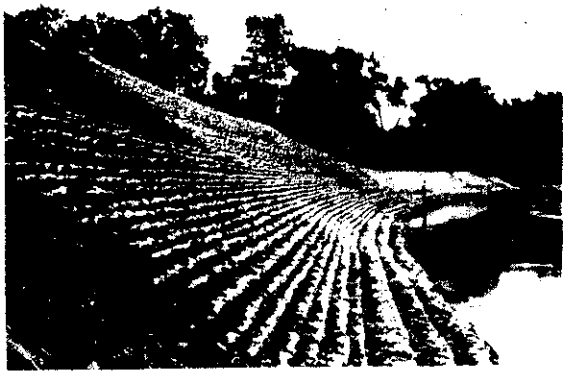
WIRE CRIB RETARD FILLED WITH USED TIRES

OTHER YAZOO BASIN WORK



Board Fence Retard with Tiebacks and Stone Toe

Cable Fence Dikes

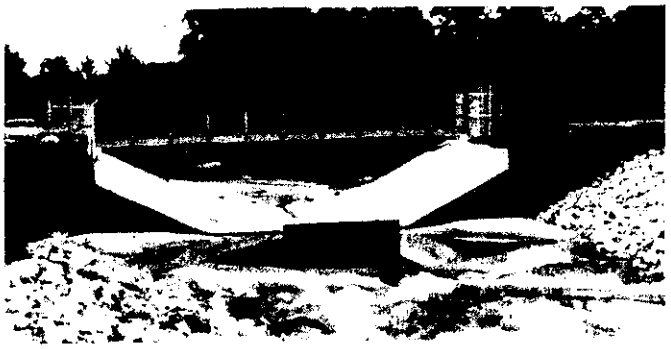


Sand-Cement Bag Revetment

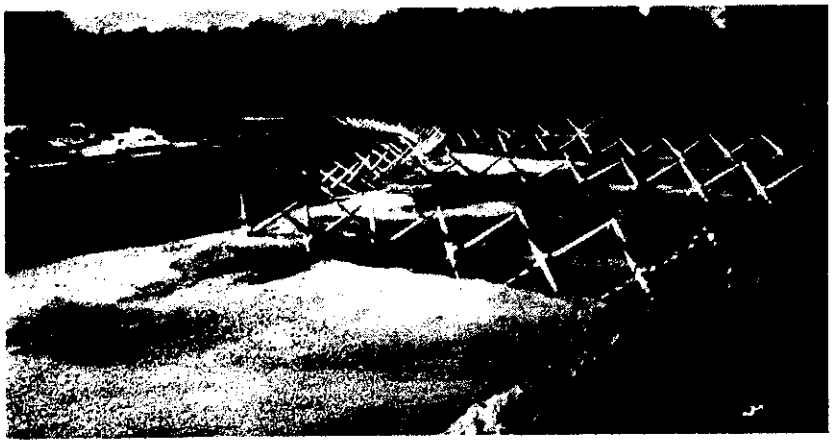
Board Fence Dikes

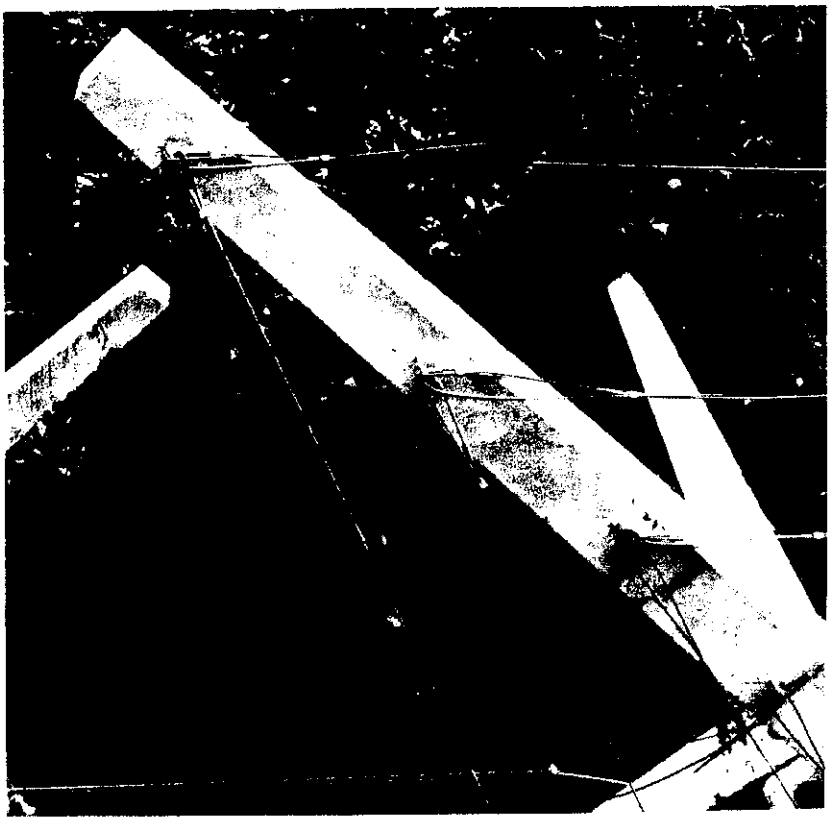
OTHER YAZOO BASIN WORK

Sheet Pile Weir with Baffle



Multipurpose Flume





Concrete Jacks (Soil Conservation Service)

COSTS

(Excerpted from Main Report, Final Report to Congress)

YAZOO RIVER BASIN DEMONSTRATION PROJECTS

Relative Construction Costs

Dollar per Linear Foot of Bank Protected

(Includes Engineering and Design, and Supervision and Administration)

Type of Work / Project / Item No.	Batupan Bogue FY 74	Batupan Bogue 4A	Batupan Bogue 4A-1	Batupan Bogue 4A-2	Hired Labor FY 80	Hunter Creek 1A	Johnson Creek 9	Perry Creek 6A	Tillatoba & Hunter Cr 1	Tillatoba Creek, NF 2	Tillatoba Creek, SF FY 72	Tillatoba Creek, SF FY 73	Tillatoba Creek, SF 5A	Tillatoba Creek, SF 5B	Tillatoba Creek, SF 5C
Transverse stone dike	30		43			36			31	31	38	48			
Longitudinal stone dike w/tieback	142	50							53	55	101	101			
Longitudinal stone dike w/2 tiebacks									72	56					
Longitudinal stone dike w/more tiebacks									86	58					
Type I stone dike									45						
Type I longitudinal stone dike w/1 type I tieback						24		25	26						
Type I longitudinal stone dike w/more than 1 type I tieback						32			41						
Type II longitudinal stone dike w/1 tieback			43			56		37	63						51
Type III longitudinal stone dike w/1 tieback			72												85
Longitudinal peaked stone dikes II		39	43				57	37							68
Longitudinal peaked stone dikes III			86				86								102
Stone paving			50					31		27					
Used tire revetment		36	24										43		43
Sand-cement bag revetment		81											129		
Tire post retards								15							
Wire crib retards								29*						33** 33*	
Board fence transverse dike	34											49			
Board fence longitudinal revetment	150														
Cable fence dikes												58			
Vegetative treatment or willow planting				12	4										
Longitudinal peaked stone dike I							14								

* Tire.
** Hay.

Note: Costs can vary significantly with time and location.

(Excerpted from Main Report, Final Report to Congress)

PART XI: YAZOO RIVER BASIN DEMONSTRATION PROJECTS

Summary and Range of Streambank (Geotechnical) Characteristics

A wide variation in stream pattern characteristics occurs in the eight Yazoo River Basin hill-line streams that were selected for study under the Section 32 Program. The major reason for these differences is the varying geologic formations the streams flow through. The erosion resistance varies considerably between the formations, as well as within the individual units themselves. Therefore, it is difficult to quantitatively describe the erodibility of the formations; however, a relative comparison among them is possible. The following is a list and brief description of major geologic formations found in the Yazoo River Basin study area, arranged in order of increasing resistance to erosion:

Formation	Description
Loess	Unconsolidated silt
Alluvium	Low-strength sands, gravels, and clays
Kosciusko	Loose sands with traces of silt and clay
Tallahatta	Loose sands interbedded with clays and shales
Citronelle	Mixed sand and clay with gravel lenses
Young paleosol	Semiconsolidated clays
Old paleosol	Dense, consolidated silty, clayey sands
Winona	Consolidated clay, shales, and chalks
Zilpha	Dense clay and clay shales

Summary and Range of Flow (Hydraulic) Characteristics

The drainage areas of the eight watersheds selected for study range from 8 to 230 square miles, and there is a wide variation in the channel geometry and hydraulic characteristics. Rainfall data collected at various locations in the study area indicate an average annual rainfall of approximately 53 in. with a range of 35 to 78 in. Limited stage or discharge data were available for the study areas; however, discharges were estimated using relations between precipitation and runoff.

Causes of Erosion and Failure

The major causes of bank erosion and the failure of protective measures in the Yazoo River Basin have been:

- Bed degradation due to a variety of reasons (channelization, cutoffs, loss of geologic control, flood-control activities, and changes in base level) and the subsequent channel widening.
- Natural meander tendencies of alluvial rivers.
- Bank failures caused by hydrostatic pressure.
- Overbank drainage.
- Man's activities.
- Instability in the streambed and streambanks due to localized geology.
- Extreme storm events.
- Structure alignment problems (previous revetment and dikes).

Significant Problems Encountered During the Program

- Local cooperation was essential to the success of the Section 32 Program. A majority of the landowners supported efforts to protect their property; however, a few landowners refused and work was deleted from those ownerships.
- Excessive time between initial concept and construction, particularly on the actively degrading streams, resulted in the need for redesign and relocation of planned work.
- Some streams were in transition states and had a geometry that upset sediment movement; this significantly limited the effectiveness of the stabilization efforts.
- During the extremely high flows in November 1977, seven of the bank stabilization structures on Batupan Bogue (Item 4A) were damaged to the extent that reconstruction measures were needed. This event occurred immediately after placement of the structures; however, upper bank protection was incomplete. Bed degradation (especially in the form of local scour), absence of upper bank protection, and the unusually large flows (100- to 500-year frequency event) contributed to the excessive bank failures.
- The funding sequence was not readily compatible with the construction schedule. Late construction starts minimized time available for adequate monitoring at some demonstration sites.
- Limited geologic and soil stratigraphy information was available to adequately incorporate into designs.

Significant Observations

- A variety of vegetative treatment measures were applied. In cases where existing vegetation was left undisturbed, the effectiveness of bank protection was increased. This was especially true in cases where woody vegetation existed. In some cases it was necessary to use vegetative treatment to help stabilize banks that had been graded or banks that were void of vegetation. The effectiveness of these vegetative treatments generally depended on whether or not the vegetation had been given time to take hold prior to the high-water season. The use of woody vegetation such as willow seemed to provide a more effective bank protection than nonwoody vegetation; however, in time, woody vegetation could impede flows unless its growth is controlled. It is important to note that in some cases the soils (paleosols, Zilpha, etc.) are not conducive to vegetative growth.

  Since bed degradation is very widespread in the Yazoo hill streams, the protection of the bank toe appears to be an important factor in the design of bank stabilization measures. The longitudinal stone dikes that provided effective toe protection were the most successful bank stabilization measures studied, particularly in degrading streams. The success of the longitudinal stone dikes is exemplified on Hunter Creek where the structures have effectively protected the bed and banks and thereby established a stable system. In many cases the absence of toe protection (tire post retards on Perry Creek) contributed to the bank failures. Bank stabilization measures without toe protection were successful in some instances; however, if bed degradation is apparent or anticipated, or in bends having more than slight curvature, then toe protection is needed or structures must be designed to accommodate expected channel deepening.

- Numerous usable geologic controls (consolidated clays in the streambed, outcrops of ironstone, quartzite, etc.) were encountered during the study. In general, the presence of geologic controls has a stabilizing effect on the stream by halting the upstream migration of head-cuts; however, there are times when a well-consolidated streambed may actually contribute to the erosion of the upper banks. This was the case on Batupan Bogue (Item 4A) where the streambed consisted of a well-consolidated clay. During the extreme flow in November 1977, the stream "skated" over the resistant clay bed and scoured the upper banks to over twice their original width.

- The grade-control structures have proven quite effective in halting streambed degradation although there have been some minor erosional problems encountered at a few sites. In some instances (Perry Creek upper structure), it appears that the degradation has been halted by the riprap key downstream of the preformed scour pocket. At other locations in the study area, degradation has been temporarily halted by box culverts and the placement of concrete rubble in the streambed by the local landowner; however, it should be emphasized that these techniques are only temporary and should not be considered as a method of halting bed degradation permanently.

- Even under the best design conditions, the effectiveness of the stabilization measure may be nullified due to construction delays. During the study, several instances of changes in stream patterns and bed elevation occurred from the time of design survey to construction. This was the case on North Fork Tillatoba Creek (Item 1) where the low-water thalweg pattern reversed in a sine-cosine fashion. On Johnson Creek a head-cut had already progressed upstream of the site of the proposed grade-control structure, thereby severely limiting the effectiveness of the structure.

- Monolithic-type bank stabilization structures prevent the seepage of water through the bank, thereby creating hydrostatic pressures in the banks. If pressure release is not provided, mass bank and structural failures may occur. This was the case of the slip failure that occurred on South Fork Tillatoba Creek where sand-cement bag revetment was installed. After placement of the sand-cement bags, the structure acquired the characteristics of a monolithic structure and a slip failure of the bank occurred, possibly due to the buildup of hydrostatic pressure following a bank-full event.

- Scour pockets were observed at the downstream edge of some structures as a result of the eddy action of the water flowing over the downstream stone tiebacks. Similar scour was observed at the point of transition from complete upper bank paving to longitudinal stone toe protection. Careful consideration during design is needed to minimize this scouring action where stabilization measures are terminated (at both upstream and downstream ends).

- It is noteworthy that none of the bank protection works in the Yazoo River Basin, except at grade-control structures, included a filter layer or filter fabric. With a few exceptions, this lack of a filter did not significantly affect the performance of the works. These few exceptions, of course, occurred where strata in the bank material were relatively impermeable, the groundwater level was high, and/or the revetment structure tended to act as a monolith (sacked sand-cement). The significance of this observation is that the designer of bank protection works should not automatically specify expensive filter material if the risk and consequences of loss of minor amounts of bank material through the protective covering are small. Where the bank material is pervious, groundwater level is low, the duration of high stages is short, and the revetment material is flexible and pervious (i.e. riprap), a filter may not be cost-effective.

Arizona Conditions may use filter fabric or bedding material

- Throughout the program it became apparent that some streams responded more effectively to bank stabilization measures than others. This phenomenon may be attributed to the rapid rate of change of the morphologic parameters (width, depth, etc.) in the unstable streams. Streams that have been significantly altered due to bed degradation undergo a rapid rate of change in width, depth, and other channel parameters. After a certain period of time, this rate of change decreases as the stream begins to adjust to a new state of relative equilibrium. It is during this period of adjustment toward a new equilibrium state that bank stabilization measures have the greatest chance of success. Construction to stabilize the stream during the rapid transition state will require more massive and costly protective works to compensate for the increased threat.

Recommendations

- The alignment of the structures is critical. During periods of high and low flows, the location of the major point of attack will vary. It is therefore necessary to define the limits for this point of attack in order to provide adequate bank protection for both high and low flows. The design should provide the highest degree of protection within the limits of the point of attack of high flows with a reduced level of protection upstream and downstream. The most common oversight in design is to extend bank protection too far upstream and not far enough downstream. Since bends migrate downvalley, the downstream end of the protection is more critical; therefore, care should be taken in establishing the lower limits of protection.

- The alignment of the structures should provide a smooth transition from bendway to bendway. Both the high-water and low-water paths should be considered in alignment design for development of an orderly transition between bendways, thereby preventing the structures from creating an obstruction to flow. In instances where the structure alignment was not compatible with the high-water flows (upper seven structures on Item 1, North Fork Tillatoba Creek), the structures were subjected to erosive forces that resulted in upper bank failure. On the other hand, the minimum protection on Johnson Creek was designed to create a smooth transition between bendways for both high and low flows; and the structures have effectively stabilized the banks.

- In streams where the streambed consists of a well-consolidated clay or other geologic control, the stabilization measures should be designed to provide complete upper bank protection with only minimum toe protection. This differs from the design on actively degrading streams such as Perry Creek where toe protection is essential to the success of the structure.

- All possible measures should be taken to expedite the time between design and construction. An effective way to do this is to advertise and award the contract based on generalized design, site layouts, and approximate quantities, then furnish detailed site layouts and cross sections immediately prior to construction.

- Close coordination between design and construction personnel while work is under way is very important. This is exemplified on Johnson Creek (Item 9A) where a protruding clay plug in the bank was encountered during placement of the stone toe protection. Rather than removing the obstruction and aligning the structure as designed, the stone was placed along the protruding bank line, thereby creating a point of discontinuity in the structure. Onsite changes by construction personnel are sometimes necessary but should be closely coordinated with the designers.

- Where possible, natural levees along top bank should be left undisturbed by construction activities. Man-made replacements may not be adequate and may alter the overbank drainage patterns. Engineering techniques to control overbank drainage are necessary to prevent damage to the structure.

- Before any stabilization measures are planned, as much data as are available should be analyzed. This may be accomplished through a research of old plan maps, surveys, topographic maps, aerial photographs, field investigations, discussions with local residents, and historical documentation of the area. This will assist designers to understand how the system has responded to changes in the past and how it may respond in the future.

- It should be emphasized that since this project was somewhat experimental, failures were expected. Some structures were designed with marginal strength in an effort to determine

the minimum amount of protection required to demonstrate that using inexpensive measures can in some cases be false economy. There is a most important difference between "inexpensive" work and "cost-effective" work; "cheap" solutions to significant erosion problems are not possible. However, an understanding of the stream's behavior allows the most cost-effective use of resources, even though the cost may be significant or even prohibitive.

SIGNIFICANT PARTICIPATION BY OTHER ORGANIZATIONS

Federal

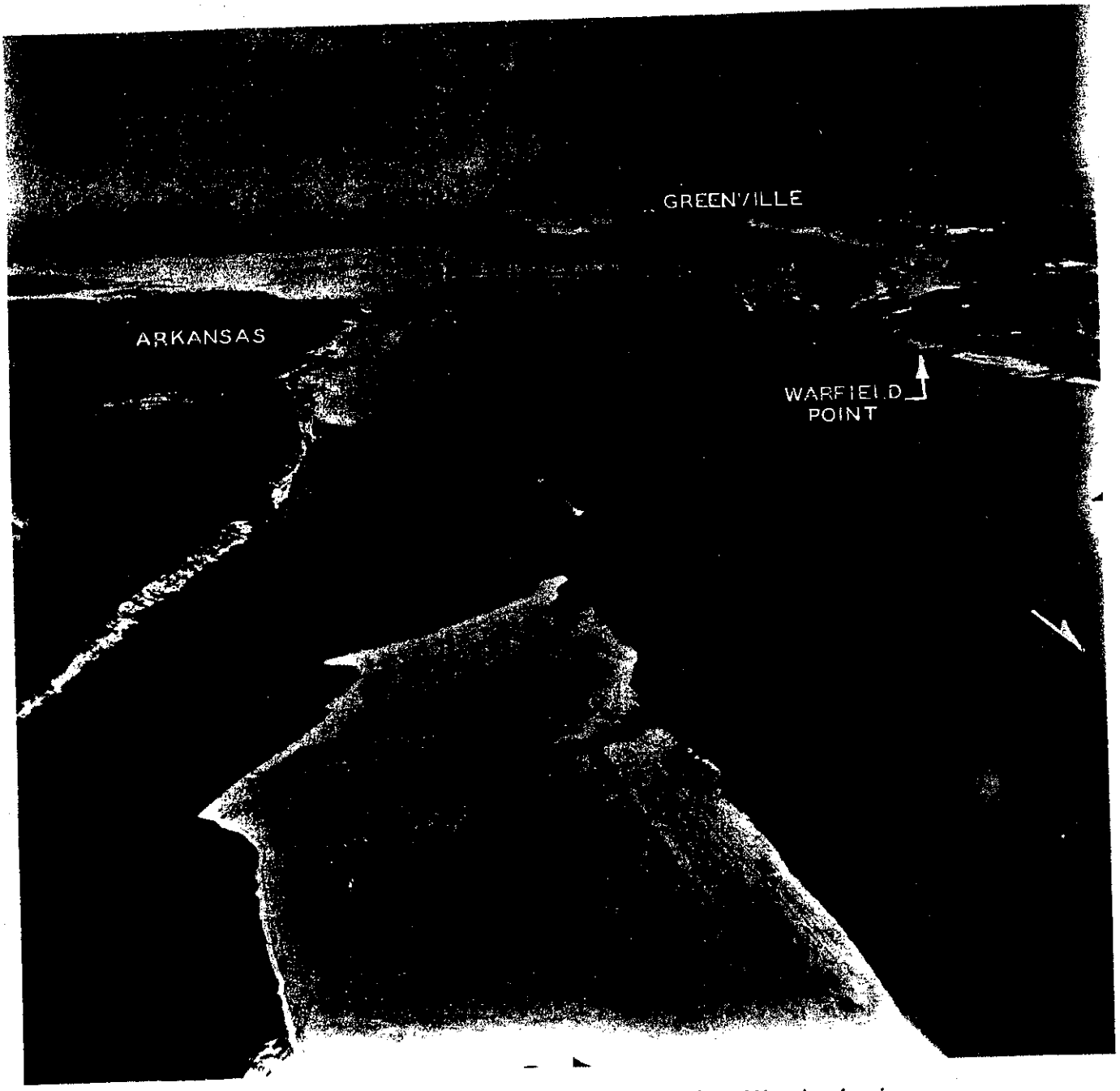
A joint venture was undertaken with the USDA Science and Education Administration Sedimentation Laboratory (SEA) at Oxford, Mississippi, to define and monitor amounts, sources, direction, and time of travel of sediments. Research included complete analyses of the drainage basin morphology, geology, soils, land use, vegetation, basin stratigraphy, hydrology, climatology, and stream hydraulics. Particular emphasis was given to the Goodwin Creek Basin, and the results will be used to determine the performance of selected channel stabilization methods and to determine influence of grade-control structures on channel stability.

The SEA and the USDA Soil Conservation Service (SCS) have cooperated in a program testing the effectiveness of a wide variety of vegetation controls on streambank stability.

The SCS inventory and evaluation of bank stabilization measures in North Mississippi lists 326 sites that were stabilized. Of these sites, SCS lists only 11 percent as having failed to some degree; however, photographs and surveys furnished with this report indicate a much higher percent with problems. Prior to using this SCS report to substantiate erosion problems and stabilization measures in the Yazoo River Basin, a more thorough analysis of the SCS work is needed.

Local

Local sponsors for the demonstration projects in the Yazoo River Basin were Grenada County; Tallahatchie County and North Tillatoba Drainage District; Long Creek Drainage District; and Hotophia Creek Drainage District.



Mississippi River at Greenville, Mississippi

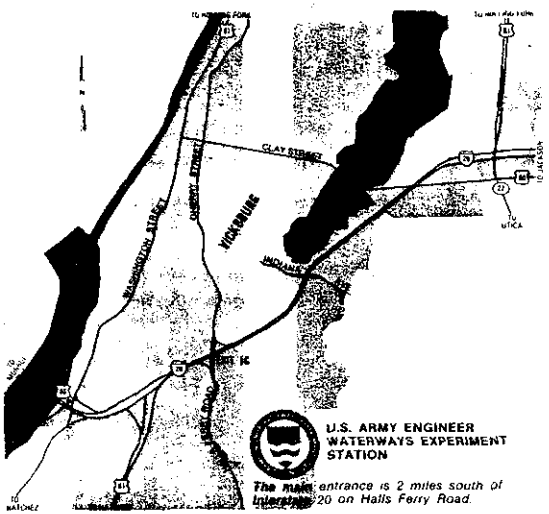
unit have a well developed polygonal cracking. This produces a weakness which, when exposed, results in a block-type failure. The remaining portions of the unit do afford a temporary restraint to bed degradation, but when subject to the extensive degradation which has occurred in this area, it is quickly removed.

At the bottom of the "old paleosol" unit is a layer of organics ranging in thickness from 6 to 48 inches. It has little or no strength, is easily eroded, and probably originated from vegetation killed during the initial rise of the lakes, which could have been formed when the tributary mouths were plugged. When this portion is exposed to stream action, bank instability increases rapidly. It is probable that this entire unit is the sediment that settled in these low energy systems.

The "young paleosol" is the second unit of deposition. It is basically a channel fill deposit in abandoned stream courses within the "old paleosol." This unit is a sandy silt of a lighter color brown than the "old paleosol." It has no internal structure and is easily eroded. Due to extensive sand in the Perry Creek area, this unit has more sand and less erosion resistance than the paleosols in other study areas. When the channel slope is too steep and this unit is exposed by stream action, bank widening and deepening advances upstream.

Overlaying this older valley fill is a layer designated the post settlement alluvium. This is the result of poor farming practices by the early settlers. The unit has no erosion resistance and is a source of large amounts of sediment in the stream.

The bank and bed of Perry Creek are very unstable except for intermittent outcrops of clay. In the lower 6 miles of the stream, the clay outcrops are probably of the "old paleosol" formation and offer some resistance to degradation only if the bed slope is stable. The clay outcrops in the upper portion of the valley are probably of the Zilpha formation and are more resistant to erosion. Every break in the profile slope has clay exposed in the bed. The best method for stabilization of this stream is to regain a stable slope by properly placed grade control structures and then to allow vegetation to restabilize the banks.



The main entrance is 2 miles south of Interstate 20 on Halls Ferry Road.

The Waterways Experiment Station is part of the Corps of Engineers, a major Army Command, whose primary mission is to provide combat support to our fighting Army. The Corps also plans, designs, and supervises construction and maintenance of military hospitals, barracks, commissaries, family housing, and other support complexes.

As the Nation's principal water resources development agency, the Corps plans and executes navigation, flood control, water supply, hydroelectric power, recreation, and water conservation programs.

The Corps is constantly changing to meet the changing needs of the American people in times of peace and war.

US Army Corps
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Waterways Experiment
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WES

Waterways Experiment Station
Vicksburg, Mississippi

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1. VISITORS FACILITY - Display of WES activities ɔstairs. Picnic tables, rest rooms, water fountain, and ·ink machine downstairs.

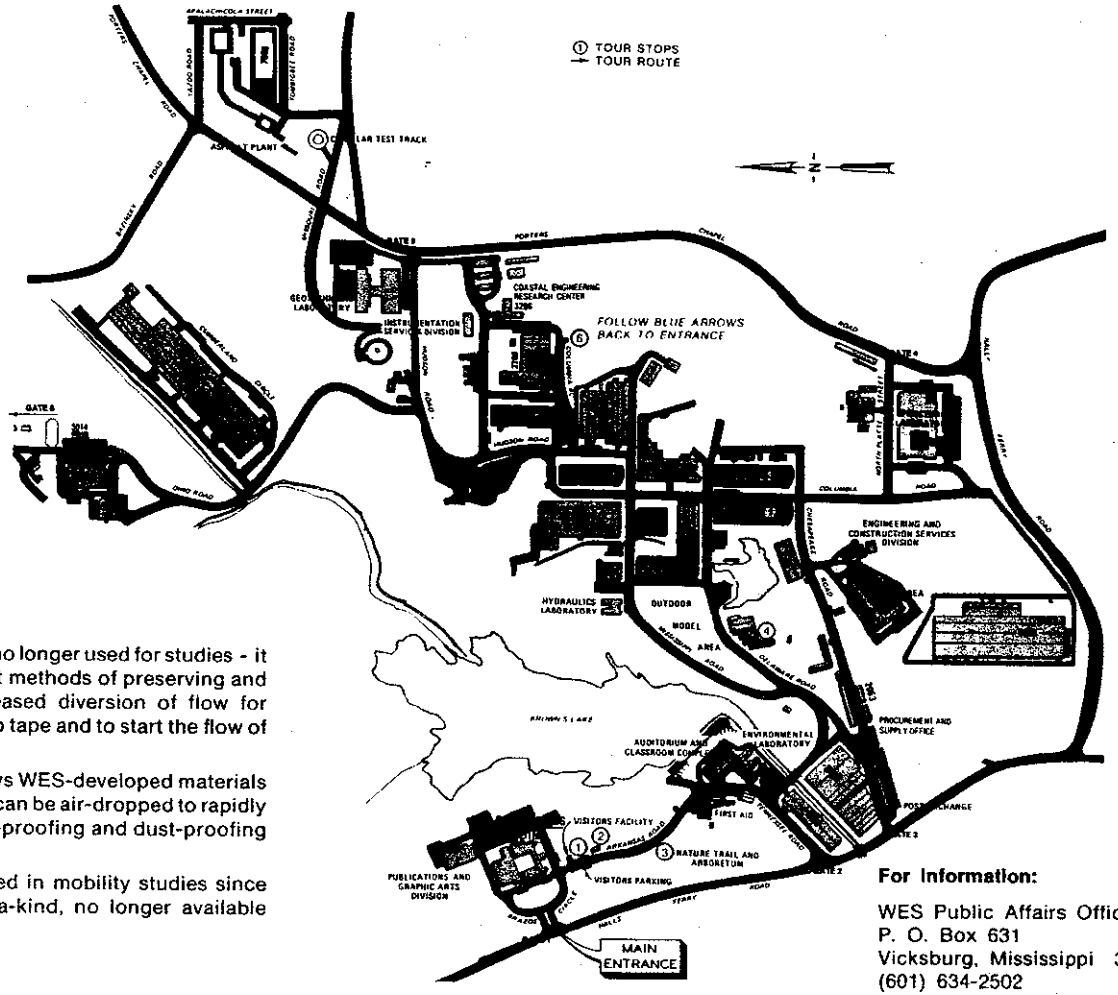
2. WATER RESOURCES DEMONSTRATION MODEL - his automated model simulates both the natural and con-olled flow of water from its origin in the mountains to its estination in the ocean. (See photograph on front of ·ochure.)

3. NATURE TRAIL - The garden clubs of Mississippi ıve designated WES as an official arboretum. The 1/2-ile-long nature trail includes information on plants native the loess bluff area of Mississippi.

4. NIAGARA RIVER AND FALLS MODEL - This model is no longer used for studies - it maintained for our visitors. Originally, it was used to test methods of preserving and ıhancing the beauty of the falls while providing increased diversion of flow for ·droelectric power. Use power switch at entrance for audio tape and to start the flow of ater over the falls.

5. LANDING MAT AND MEMBRANE - This display shows WES-developed materials r Army use for expedient surfacings in remote areas. Mats can be air-dropped to rapidly ɔnstruct landing strips. Membrane can be used for water-proofing and dust-proofing nding surfaces.

6. EXPERIMENTAL VEHICLES - WES has been involved in mobility studies since orld War II. Most of these tested vehicles are one-of-a-kind, no longer available ıywhere.



For Information:

WES Public Affairs Office
P. O. Box 631
Vicksburg, Mississippi 39180
(601) 634-2502

THE WATERWAYS EXPERIMENT STATION

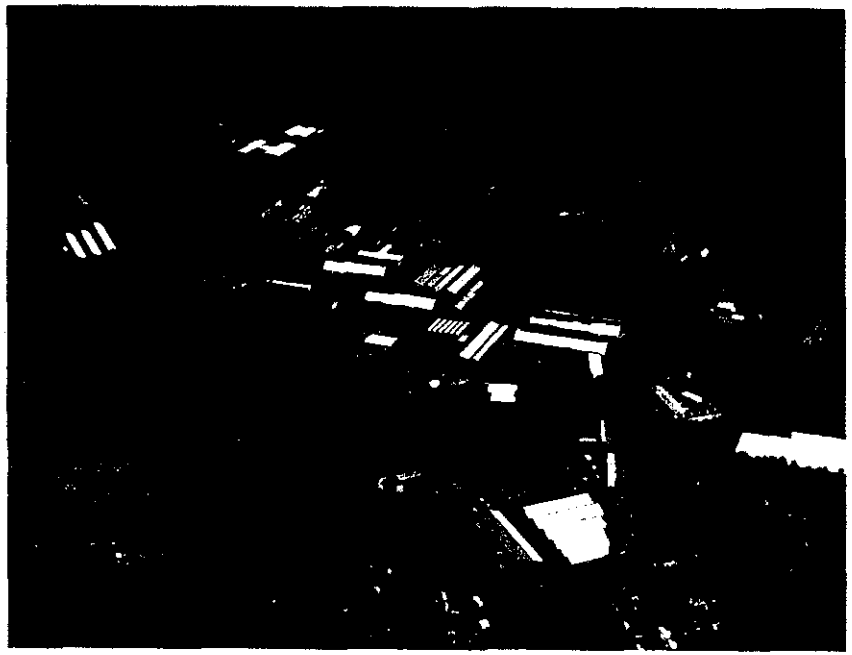
HISTORY AND MISSION

The Waterways Experiment Station (WES) was established in 1929 as a hydraulics laboratory to help control floods in the lower Mississippi Valley. Today, WES is a research, development, and testing complex of five internationally recognized laboratories supporting the civil and military missions of the Corps of Engineers and the Nation.

WES covers nearly 700 acres, employs over 1500 people, and has a 100 million dollar work program for 120 sponsors who pay all project costs on a reimbursable basis. Sponsors include the Corps of Engineers, other Federal agencies, and, under special conditions, private industry and foreign countries.

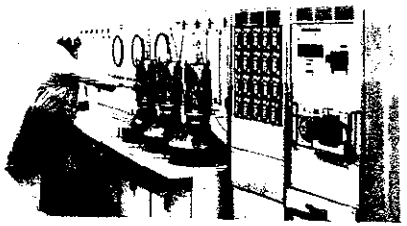
HYDRAULICS LABORATORY

The **Hydraulics Laboratory** studies flood control, navigation, and sedimentation problems in waterways and harbors. Physical and numerical model studies produce precise scientific results of tremendous value to the Nation's development and economy.



GEOTECHNICAL LABORATORY

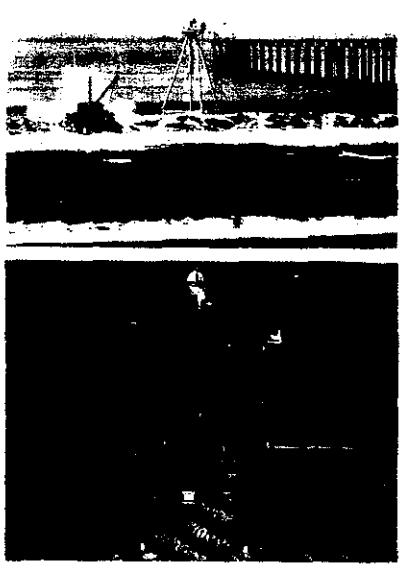
The **Geotechnical Laboratory** conducts virtually any type of soils test for design and investigation projects. Other studies include pavements, off-road mobility of military vehicles, earthquake effects, and engineering geology.



The **Structures Laboratory** studies the response of structures to various kinds of loadings. High explosives simulate nuclear blast and shock effects while vibratory procedures simulate earthquake loadings. Dynamic properties of earth and related materials are investigated. Concrete and cement research make these materials stronger and more economical.

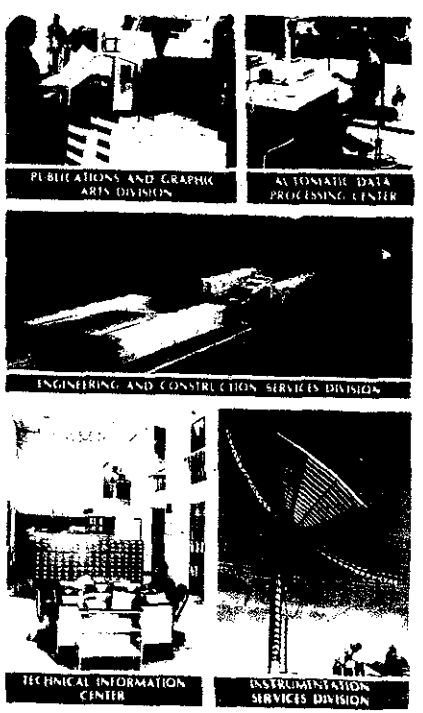
The **Environmental Laboratory** investigates the effects of water resources development projects on the environment, and the interaction between military activities and the environment. Studies include all aspects of research pertinent to environmental protection policy, legislation, and regulation.

The **Coastal Engineering Research Center (CERC)** is the focus of national expertise in coastal engineering. CERC uses physical and numerical models and field investigations to study waves, tides, currents, coastal processes, and materials. These studies help solve navigation, coastal flooding, beach erosion, storm protection, and marine construction problems.



Technical Support Elements

The WES technical support elements assist the laboratories in accomplishing the multiple mission of research, testing, development, and technology transfer.



HERBERT D. VOGEL BUILDING

HERBERT D. VOGEL BUILDING

Design Report
3-1-88

Job:	Formless Chute
Application:	Standard design and drawing
Location:	California
Authority:	PL - 46 (01)
Phase:	Final design and preliminary drawings

Summary: The formless chute has been used extensively in the Marin and Sonoma County areas of California over the last 30 years. Generally these structures have been installed for grade control but occasionaly they have also been used as spillway structures for small dams. The formless chute is traditionally constructed with a trapezoidal section consisting of inlet, chute and flat apron outlet. No specific design information was available for design of these structures. Because of the success and low installation cost of the formless chute the SCS State Engineer requested WNTC assistance in 1986 to evaluate and develop a standard design for these structures.

Following a field evaluation of existing structures and mutually agreed to limitations a design has been developed that is very similar to the existing structures. The design includes a constructed plunge pool that is expected to improve the overall structure stability by providing a stable outlet. There will be an increase in cost for rock riprap but this is somewhat offset by reduced length of chute. The reduced chute length is the result of a plunge pool design that incorporates three feet of drop.

The design covers the hydraulic and structural design and layout for formless chutes varying in drop from 12 feet to 25 feet. The drawings are preliminary and will need to be completed by the State Engineering Staff using a layout format as they deem appropriate for the intended use.

The design, notes for construction, and supporting data are discussed below.

Description of Job: The formless chute consists of a reinforced concrete structure with a level inlet, 3:1 chute and a level outlet section. The structure is trapezodial in cross section with 2:1 side slopes. A shallow rock lined plunge pool immediately downstream of the level outlet apron provides a transition to the natural channel.

Purpose of this Design: The design was prepared to develop a standard design and drawing for formless chutes that will be used for grade stabilization practices in California. This standard drawing will help implement conservation

practices by minimizing design and design review time at both the State and NTC level and will assure the landowner of a quality product.

Basis for Design: The following information and references were used in preparing this design.

1. Trip Report, Saele to Wallin, dated 7/3/86
2. Personal notes from field evaluation of formless chutes, June 16 to 19, 1986, L. Saele
3. Drawings and Specifications from R. Dolicine, Grade Stabilization Structure, dated 6/5/85
4. Letters - Gerbaz to Wallin, Subj.- Formless Chutes, dated 9/26/86 and 3/31/87
5. Correspondence with the SNTC with regard to development of a standard drawing for formless chutes, (appendix A)
6. SCS National Engineering Handbooks No. 5, 6, 11 and 14
7. SCS Engineering Field Manual
8. SCS, Design Note 6 and Soil Mechanics Note 1 and 9.

Design Discussion:

I. *Hyraulic Design*

The hydraulic sizing of the structure is based on standard engineering principles and application of concepts in NEH 5 and NEH 14. The overall configuration is similar to the existing formless chutes that have been built in California over the past 15 to 20 years. The plunge pool or outlet basin has been modified to provide for what we believe will be improved energy dissipation.

The following criteria for the hydraulic design was established based on WNTC experience with hydraulic structures, observations of existing formless chutes in the field and by request of the state staff.

1. Total drop of 12 to 25 feet.

2. Trapezodial shape with 2:1 side slopes

3. Bottom width, variable, to match channel capacity

4. Maximum flow velocity, less than 30 fps *

   * Cavitation begins to be a concern when flow velocities exceed about 30 fps. Also the effect of dynamic pressures on surface irregularities can be devastating.

5. Chute section with 3:1 slope

6. plunge pool or other low cost outlet

A. Inlet Section

The inlet section was hydraulicly designed to provide a non-erosive transition from the natural channel to the chute section and by virtue of the inlet flow depth, control the unit discharge of the formless chute.

Water surface profiles were calculated over a range of drops using the standard configuration of a level inlet, 3:1 chute and a level outlet apron to determine chute and exit velocities. It was determined that a unit discharge of 5 cfs/ft. reaches a terminal velocity along the chute of 20+/- fps at a drop (D) of about 12 feet. However, for a unit discharge of 25 cfs/ft., the velocity (35+ fps) is still increasing rapidly at 25 feet of drop. See graph of velocity versus drop, appendix B. The effect of the level apron at the downstream end of the chute is relatively minor, retarding the velocity by 2 to 3 fps. To keep velocity under the 30 fps criteria for the maximum drop of 25 feet, the unit discharge has been limited to 15 cfs/ft. The calculated value of Hp for the unit discharge of 15 cfs/ft is approximately 2.5 feet. This value was increased by 20% for freeboard and than rounded to 3.0 feet to provide an even dimension for construction. On this basis the design and drawings provide for 0.5 foot increments in the inlet depth from 2.0 to 3.0 feet with unit discharge capacity as indicated in item B. below.

The design assumes that approach velocity will be well below critrical. If the Hp value selected for the structure is based on a flow depth in the natural channel, approaching critical, the drawdown at the critical section will be less and overtopping at the inlet or upper chute may occur. Therefore the designer will want to choose a bottom width and Hp value that is compatable with the maximum anticipated capacity. This may also require raising the crest elevation, in situations where the natural channel velocity is high, to retard the entrance velocity.

The length of the inlet was set at 10 feet. This will provide a minimum length of $3(d_c)$ plus 4 feet for all flow conditions. In addition a marginal length of rock riprap (5 feet) is recommended upstream of the inlet to transition from the natural stream to the inlet.

B. Chute Section

The purpose of the chute is to convey the flow from the inlet to the outlet section. No specific hydraulic design was developed for the chute section other than determining the height of side slopes necessary to contain the flow relative to the inlet capacity. To prevent the possibility of overtopping, the chute sidewall height was determined from water surface profiles, assuming that the inlet section was flowing full. The values were also adjusted to provide

even increments as noted for the inlet. The following values are to be used for design.

Unit capacity (q) (cfs/ft)	Inlet (Hp) (ft) *	Chute (Hc) (ft) *
15	3.0	2.0
10	2.5	1.5
5	2.0	1.25

*measured vertically

A vertical curve at the upper break in grade is not believed necessary for these structures. At the lower break-in-grade where the velocity will be high and the formation of a jump would be undesirable, a vertical curve is more important and will be provided by controlled screeding of the concrete to form a rounded rather than sharp change in slope. Some rounding of the upper curve is also likely to occur during the concrete placement.

C. Outlet apron

The function of the outlet apron is to change the direction of flow from the 3:1 slope to a horizontal direction. Ten feet was believed to be a practical minimum value for this purpose. The flow retardance due to change in direction and the flat slope has only a minor effect on the depth of flow. Therefore the height (Hc) of the side slopes remain the same as the chute through this section.

Several different outlet shapes were considered during the early hydraulic analysis including a flip bucket. For the range of flows analyzed there did not seem to be any advantage of these other shapes over the flat apron.

D. Plunge pool

The purpose of the plunge pool is to provide energy dissipation and return the flow to the existing channel at non-erosive velocities. Many of the existing formless chutes observed in the field were constructed without any type of plunge pool or energy dissipation device. These structures tended to develop a pool with time and their performance could be described as adequate when properly maintained. We could not justify a standard design however without regard for the downstream effects.

SCS-Design Note 6 (DN 6) proved to be the most applicable for analyzing flow from a flat apron. Using the flow depth and velocity at the downstream edge of the apron and equating to an equivalent pipe discharge the equations in DN 6 were used directly to determine size of the plunge pool. The final dimensions were selected so that equations in DN 6 were satisfied over the entire discharge range expected for formless chutes with Hp < 3.0 and D = 12 to 25

feet. The plunge pool bottom width will correspond directly with the structure bottom width. The only design variable will be the size of rock.

The rock size is related to the apron discharge velocity and flow depth. A graph on page 28 of appendix C, shows this relationship. This relationship can best be represented on the drawings by use of a table showing the d_{50} rock size to various Hp values as shown on page 30 of appendix C.

The graph on page 28 indicates, that as the drop increases, there is a slight decrease in the d_{50} rock size. This is probably not true in the real world and would tend to indicate an error in correlating flow off an apron to pipe flow. However we believe the results can be used as long as it is recognized that the procedure is experimental and that some adjustment to rock size maybe necessary as we develop more experience.

II. Structural Design

In general the inlet, chute, and outlet section are designed as slabs on grade. Uplift pressures that may develop due to seepage or dynamic effects of flow are assumed to be controlled by the drainage system. Also, settlement is assumed to be negligible, thus reinforcement is provided only to control temperature and shrinkage. The maximum drop provided by the concrete portion of the structure is 22 feet. The remaining 3 feet is provided in the plunge pool design for a maximum total drop of 25 for the structure. The chute section can be varied in length to provide any drop from 12 (or less) to 25 feet. The bottom width is determined by the needed capacity. A width greater than 20 feet is not recommended.

Structural Design Assumptions:

1. Concrete Slab, constructed on grade
2. Foundation settlement is insignificant
3. Structure is not subject to hydrostatic pressures due to water table
4. Warm climate, no frost penetration
5. Chute will not be steeper than 3:1
6. Structure will meet the requirements of SCS Practice Standard 410, Grade Stabilization Structures.
7. Structure is trapezodial with 2:1 side slopes

A. Concrete thickness and reinforcement.

A thickness of 5 1/2 inches was selected for the concrete floor and side slopes. This will allow the use of a standard 2x6 for depth control and will provide some leeway in maintaining 2 inches of cover over the reinforcement.

Welded wire fabric will be shown for reinforcement except at the upper and lower cutoff wall. The size of wire fabric specified will be 6,6,W2.9,W2.9. This is one of the standard sizes used in building practice and available at most building supply centers. This will provide a steel area of A_s = 0.058 sq.in./ft. in each direction. This does not meet even the unexposed surface requirement for major SCS structures but does satisfy the subgrade drag theory for crack control. Heavier wire fabrics are manufactured and could be used but generally are not available without special ordering. Also, two layers of lighter weight fabric could be used but the difficulty in construction is increased accordingly. Satisfactory performance for crack control will likely depend more on proper placement of the wire fabric than on its size. This will need to be stressed on the drawings.

B. Inlet Section

A cutoff collar, extending 3 feet downward from the invert is provided to control seepage at the upstream end. The depth of 3 feet was mostly arbitrary, based on past experience and assuming that soils will be low permeability, plastic silts and clays. Where other conditions exist the designer should size the cutoff accordingly.

The thickness of the cutoff was set at 8 inches. This was selected for ease of construction and has no structural significance. A single mat of reinforcement (#3 or #4 bars) will control temperature and shrinkage cracking.

C. Chute Section

The upper 6 feet (measured vertically) of the chute will be constructed monolithically with the inlet section. This will result in a section approximately 28 feet long. It was believed that lengths exceeding 30 feet would be excessive for controlling shrinkage cracks and the assumption of insignificant settlement, would not be valid over longer reaches.

The structure will consist of two articulated sections for drops less than 15 feet and three sections for drops up to 25 feet. The maximum length of the center section of chute will be 30 feet (measured horizontally) when the structure is designed for the full 25 of drop. A small cutoff is provided on the downstream side of each articulation joint to control seepage under the chute and to prevent sliding.

C. Outlet Section

The lower 6 feet (measured vertically) of chute and the outlet apron are monolithic. A cutoff wall extending 5 feet downward form the invert is provided to support the apron because of the two foot overfall and to protect the outlet from scour and undermining in the event the rock riprap is

not adequate. The concrete thickness for the cutoff wall is set at 8 inches and is reinforced with a single mat of #3 or #4 bars (depending on what is most readily available) at 12 inches.

D. Drainage System

Transverse drains are provided at three critcal locations to intercept seepage. The locations are (1) the upper break-in-grade, the upstream side of chute cutoff(s) and the lower break-in-grade. A perforated pipe will be used to pickup the drain flow and convey to the discharge point. The type of drainfill material can be one of two options. The first option would be to use a filter fabric (non-woven) with a coarse material (D_{85} = or > 1/2"). The perforated drain pipe could then be standard PVC sewer pipe with 1/2 inch diameter perforations. The second option would not use filter fabric but would require a drain fill material in accordance with SCS Soil Mechanics Note 1 and a drain pipe with compatible perforations.

All drains will outlet at elevations lower than the point of pickup. Except for the lower drain which will outlet through the downstream cutoff wall, the drains will outlet through the floor of the chute with the end mitered to fit the floor.

E. Plunge Pool

The layout of the rock lined plunge pool is shown on the preliminary drawings. The rock lining is extended to a top elevation of 1.0 foot above the assumed downstream tailwater elevation (Hp). Rock gradation is based on the d_{50} rock size determined by the hydraulic analysis. Filter fabric or bedding is to be provided under the rock. Minimum gradation and thickness requirements are given below.

$d_{100} = 2.5\ d_{50}$ where d_{50} is the median size rock as determined from the selected Hp value (see I.D.)

$d_{20} = 0.5\ d_{50}$

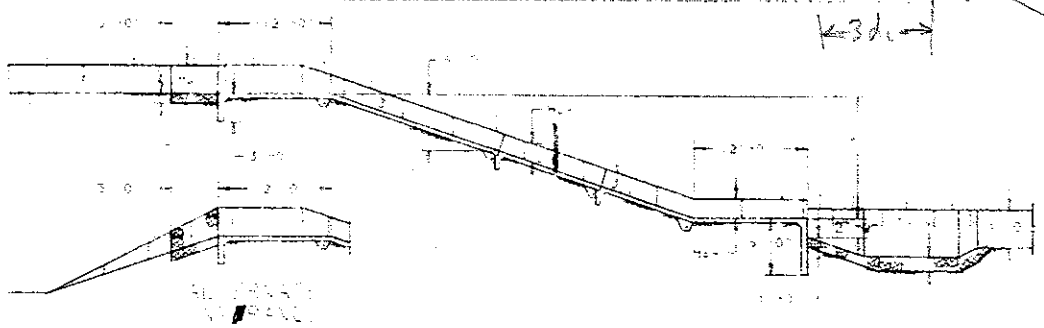
$T = 2\ d_{50}$ (minimum)

III. Drawings

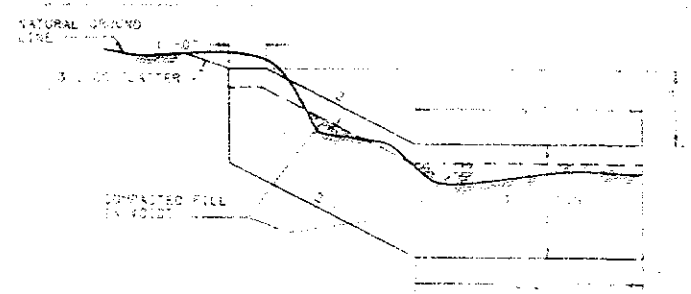
A partially completed drawing for the formless chute with red lined corrections has been prepared.

Prepared by: Leland M. Sacle Date: 3/1/88

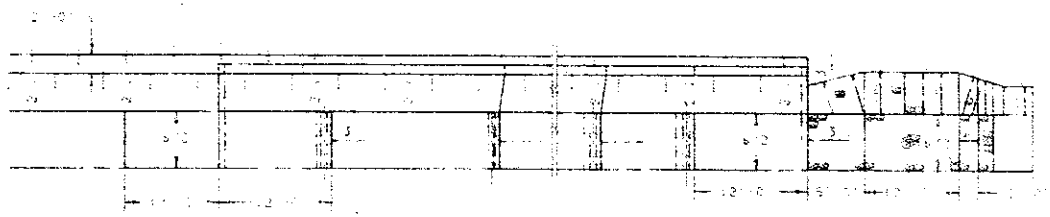
Recommended: Paul J. Morrow



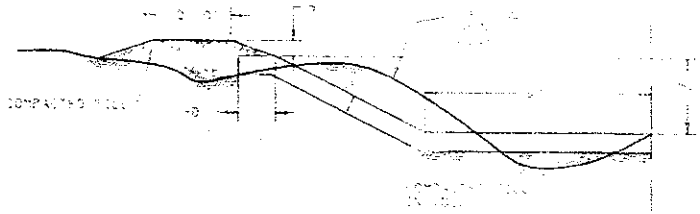
PROFILE OF CHUTE



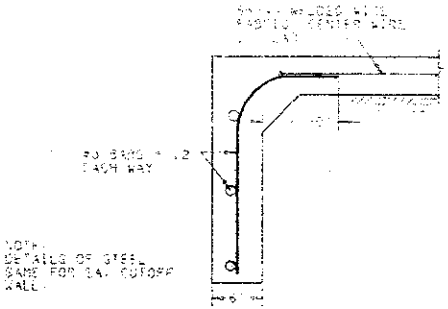
UPSTREAM CUTOFF



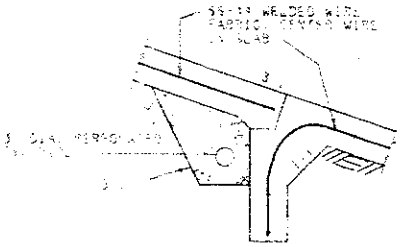
PLAN OF CHUTE



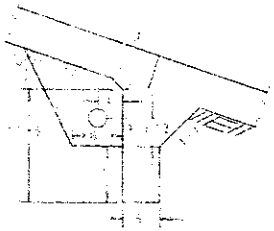
TYPICAL SECTION OF CHUTE

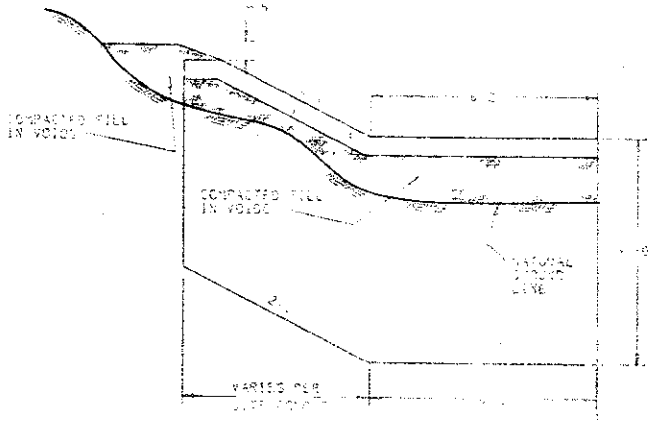


DETAIL OF CUTOFF

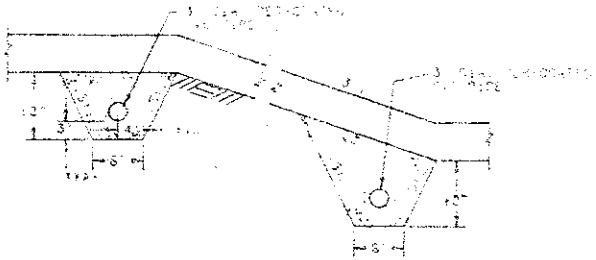


DETAIL OF KEY





DOWNSTREAM CUTOFF



DETAIL OF TRANSVERSE DRAIN

SEE FULL SIZE DRAWING
FOR NUMEROUS CHANGES
3/1/88

VOTE

ERASE THIS NOTE BEFORE PLOTTING
ALL INSTRUMENTS
GREEN 00
RED 1
WHITE 0
YELLOW

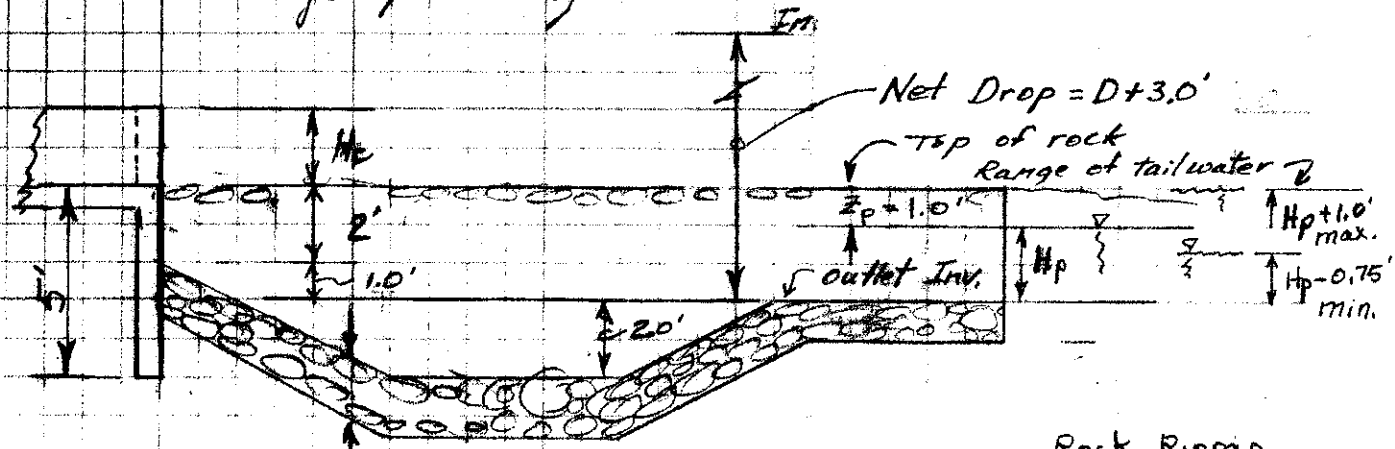
FORMLESS CHUTES

COMPUTATION SHEET
SCS-ENG-523 Rev. 8-69

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

STATE	PROJECT			
CALIF.	FORMLESS CHUTES			
BY: LMS	DATE: 1/88	CHECKED BY	DATE	JOB NO.
SUBJECT: Plunge Pool Layout				SHEET 1 OF 1

Plunge pool layout

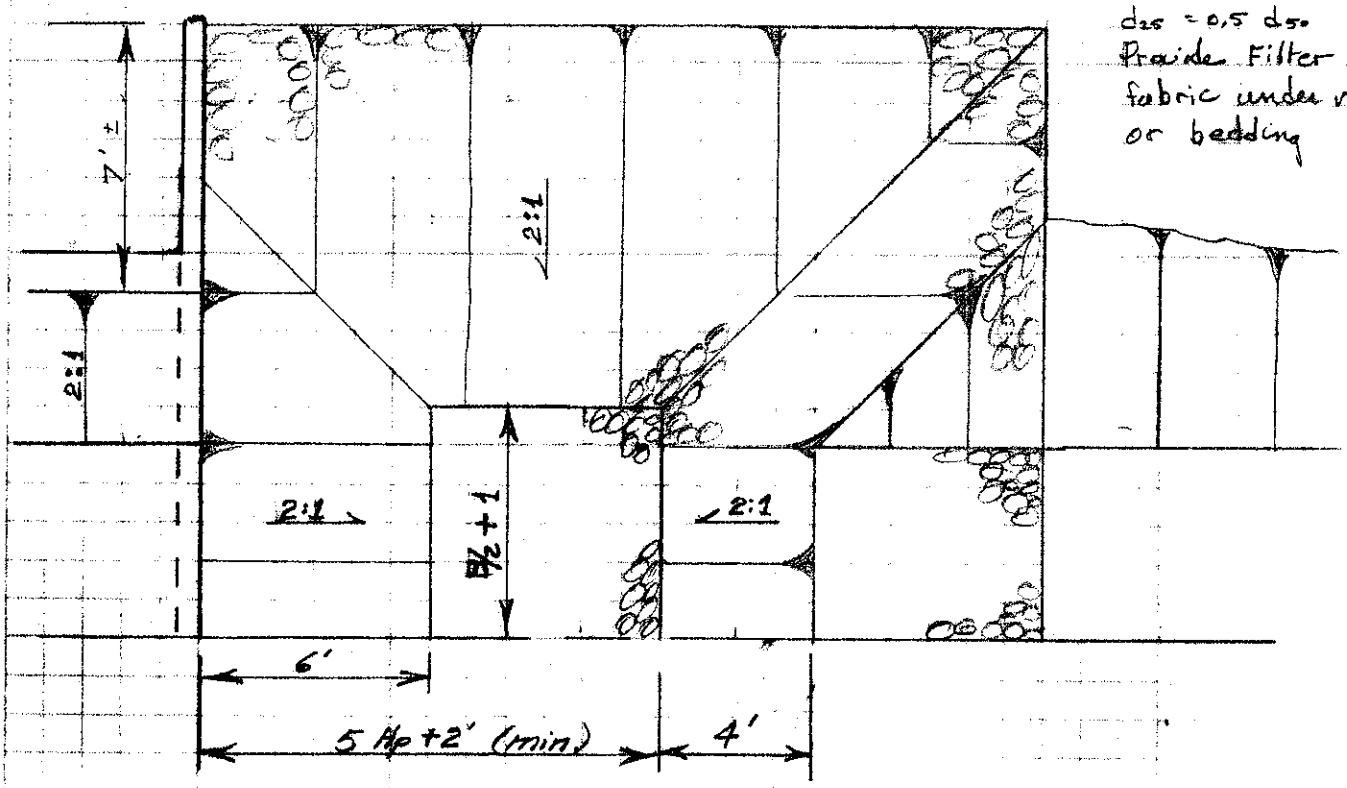


T PROFILE

(5 Hp) min.

Rock Riprap

$d_{50} = 6''$ *
$T = 2 d_{50}$ (min)
$d_{100} = 2.5\ d_{50}$
$d_{25} = 0.5\ d_{50}$
Provide Filter fabric under rock or bedding



PLAN

ROCK SIZE FOR STD. LAYOUT

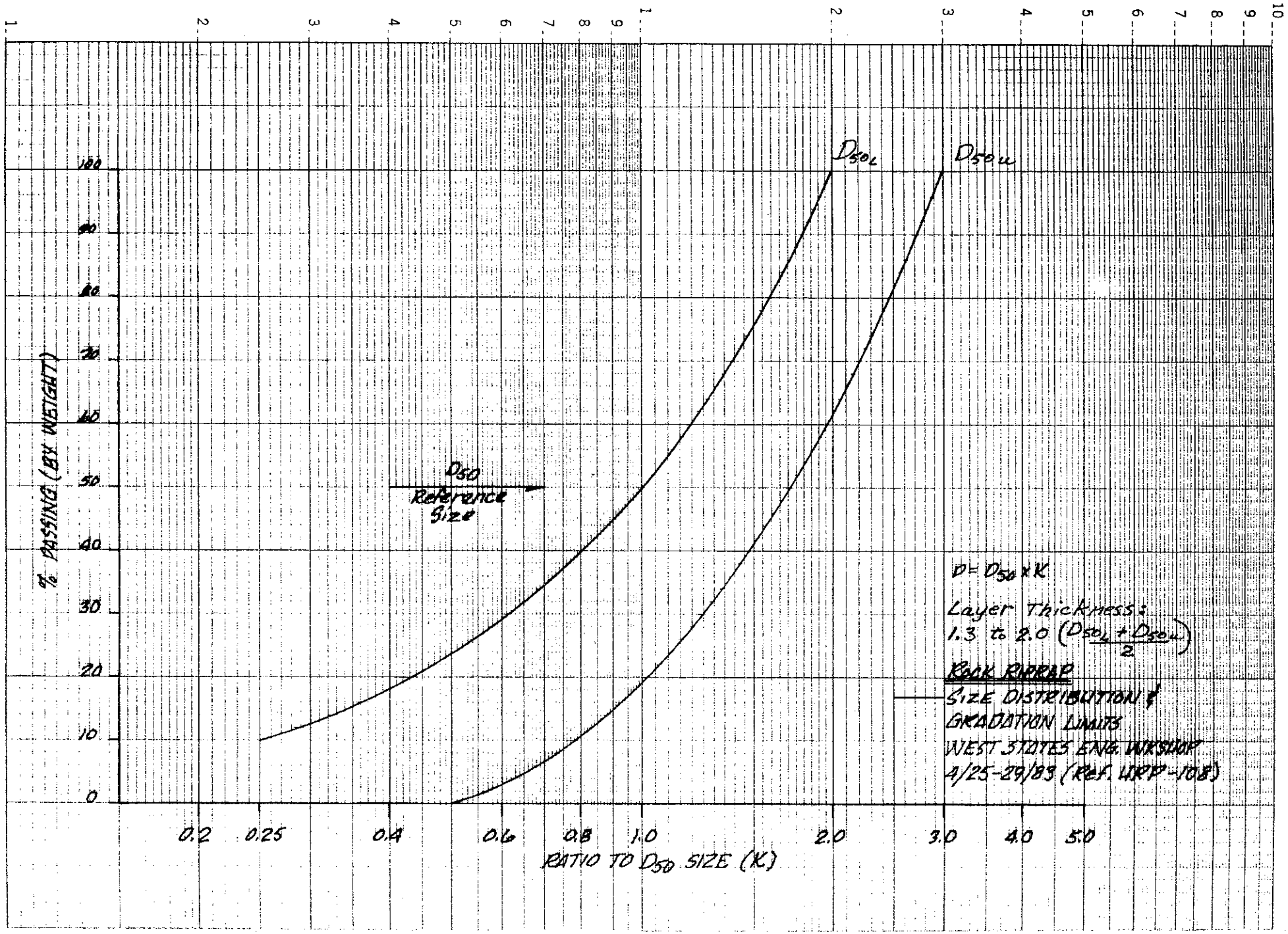
Hp	D_{50}
≤ 2.0'	3"
2.5'	6"
≤ 3.0	6"

ROCK RIPRAP CRITERIA

STREAMBANK AND SLOPE PROTECTION

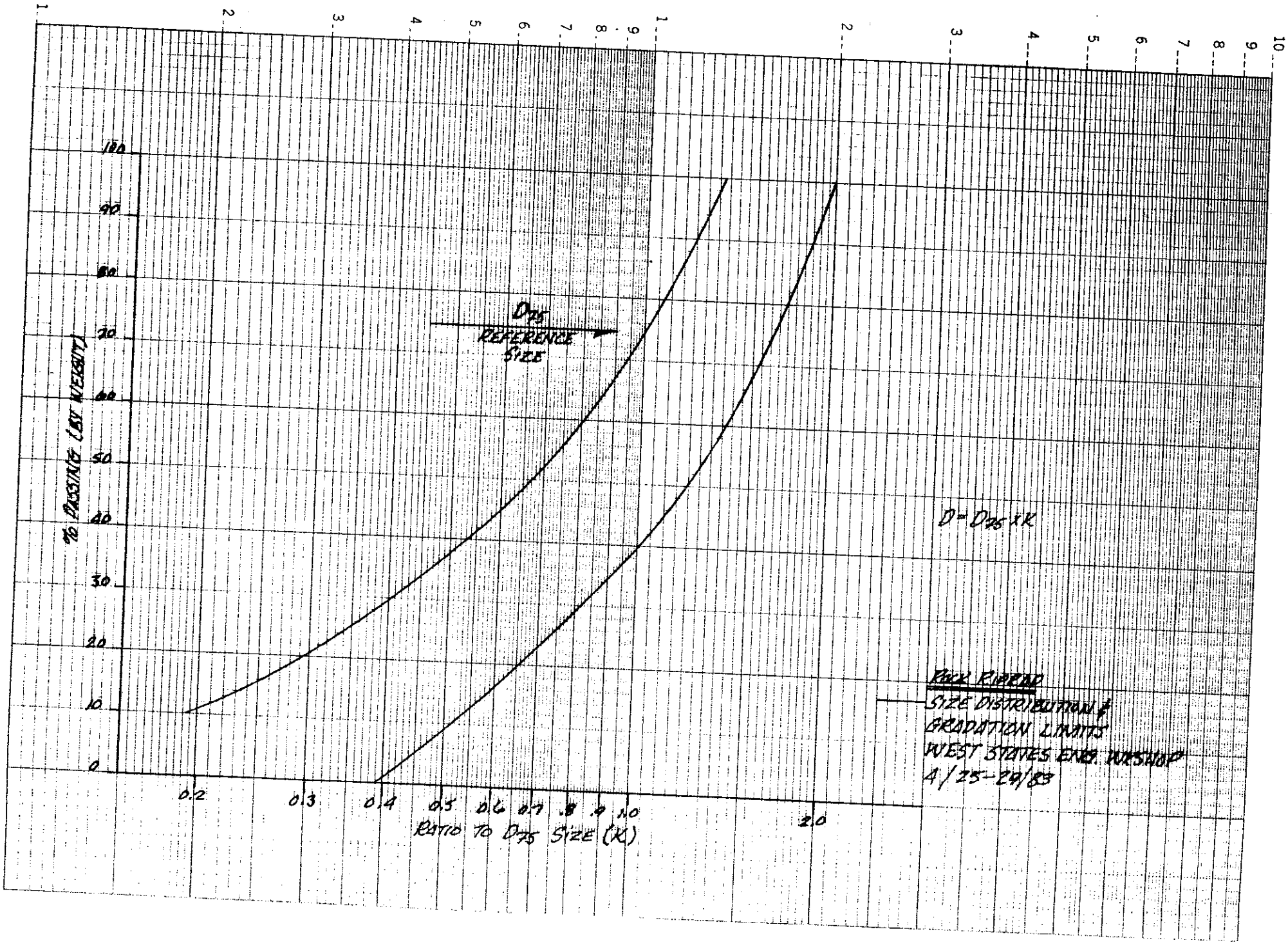
REFERENCE	ROCK SIZE	GRADATION	FILTER
1. ENG. FIELD MAN.	ISBACH CURVE: VELOCITY VERSUS D_{100} SIZE EXHIBIT 16-1 DOUBLE SIZE FOR DEBRIS IMPACT	MAXIMUM, MINIMUM, AND D_{75} SIZE FROM EXHIBIT 16-2	SUITABLE MATERIAL, CLEAN SAND-GRAVEL MIN. 6" THICK
2. FAR WEST STATES	TRACTIVE STRESS-LANE $D_{75} = \frac{3.5}{CK} WDS$	WELL GRADED THICKNESS: D_{75} TOP OF SLOPE, $3\ D_{75}$ BASE OF SLOPE	SM-NOTE 1 $\frac{D_{15\ RR}}{D_{85\ BASE}} \leq 5$
3. ADVISORY PO-18 5/1/74	TRACTIVE STRESS-HRP108 $D_{50} = \left[\frac{118Q(Se)^{13/6} R}{P}\right]^{0.4} K$	GRADATION LIMITS CURVE DEVELOPED FROM HRP-108	NO SPECIFIC INFO.
4. TR-59	TRACTIVE STRESS-HRP-108 PROCEDURE DETAILED IN TR	INDEX $= \left[\frac{D_{85}}{D_{50}} + \frac{D_{50}}{D_{15}}\right] \leq 5.5$	$\frac{D_{15\ RR}}{D_{85\ BASE}} < 5 < \frac{D_{15\ RR}}{D_{15\ BASE}} < 40$ $\frac{D_{50\ RR}}{D_{50\ BASE}} < 40$
5. ROCK RIPRAP FOR SLOPE PROTECTION DRAFT TR	WAVE HEIGHT AND EMB.SLOPE, DESIGN CHARTS D_{50} FROM FIG. 8	FIGURE 10 UNIFORM ROCK GRADED ROCK	SM NOTE 1 MINIMUM THICKNESS 9 INCHES
6. STREAMBANK PROTECTION, WA. LETTER 7/20/82	STD. GRADATION FOR $V \leq 10$ fps FWS OR EFM FOR $V > 10$ fps	GRADATION LIMITS RATIO TO D_{75} SIZE	$\frac{D_{15\ RR}}{D_{85\ BASE}} \leq 8$
7. C.E. STONE STABILITY CHARTS	DEVELOPED FROM ISBACH VELOCITY VERSUS D_{50}, LOW AND HIGH TURBULENCE	N.A.	N.A.

K+E SEMI-LOGARITHMIC 2 CYCLES X 70 DIVISIONS
KEUFFEL & ESSER CO. MADE IN U.S.A.
46 4973

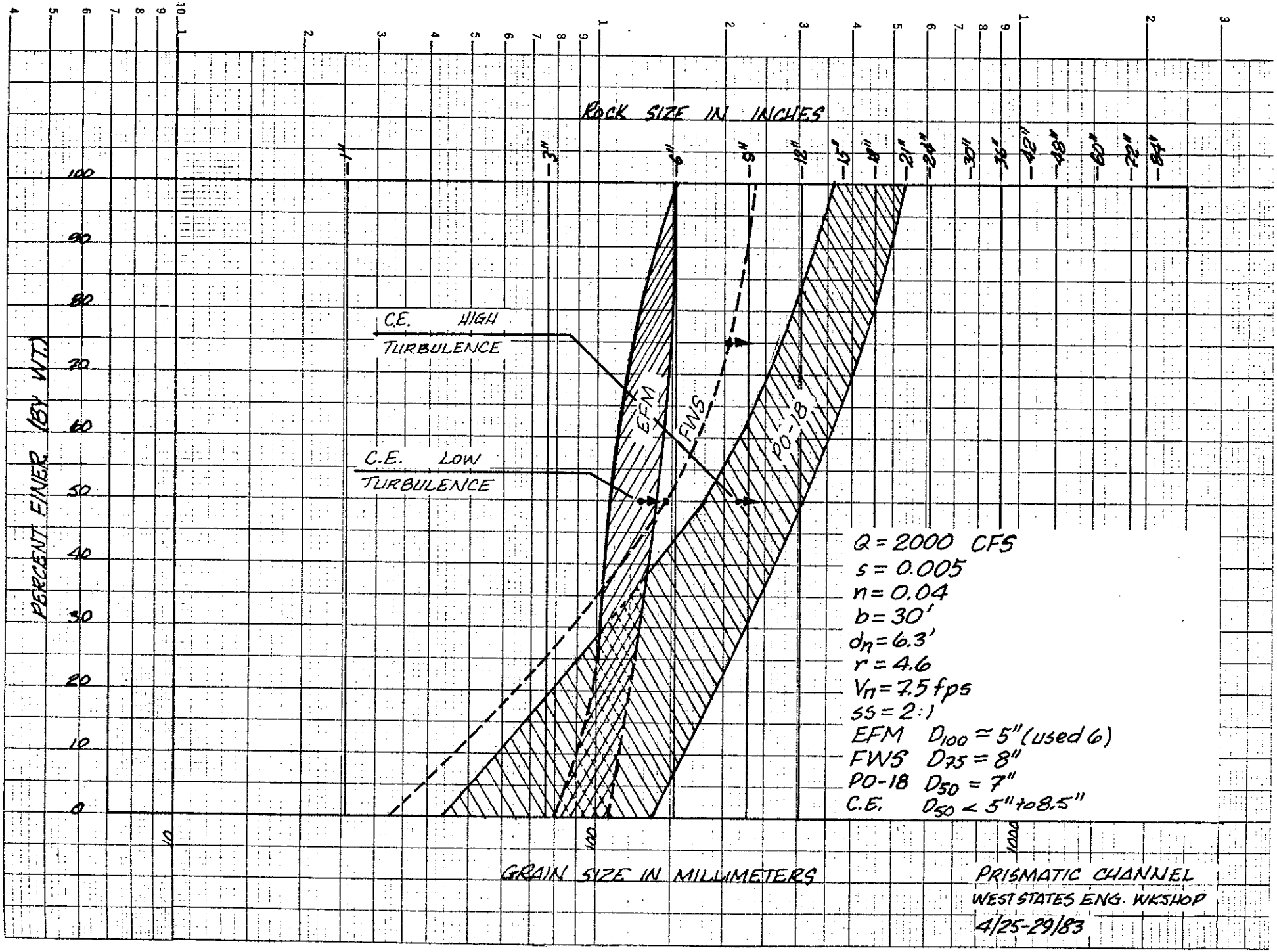


K+E SEMI-LOGARITHMIC • 2 CYCLES X 70 DIVISION
KEUFFEL & ESSER CO. MADE IN U.S.A.

46 4973

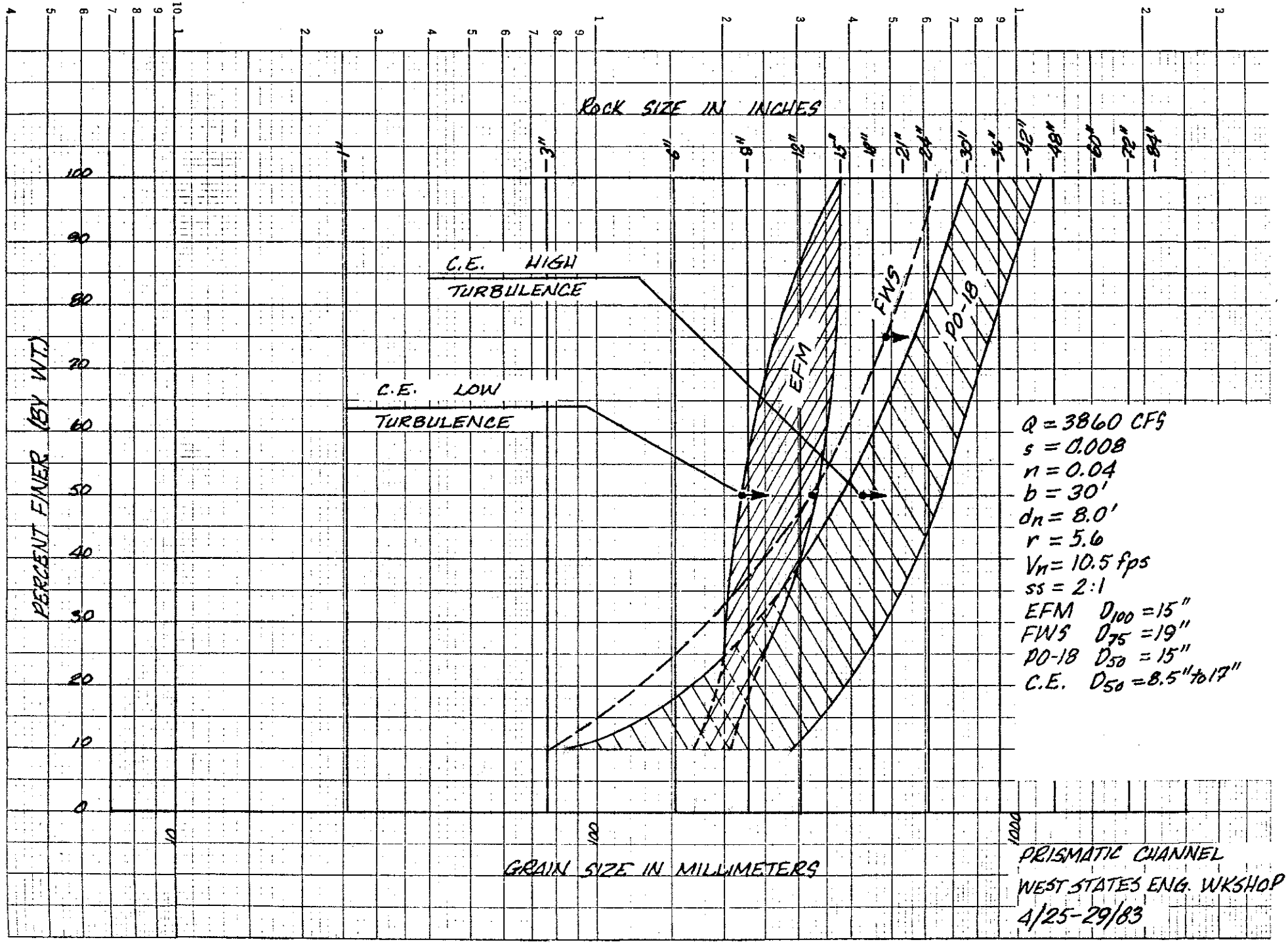


K+E SEMI-LOGARITHMIC 46 5813
5 CYCLES X 140 DIVI MADE IN U.S.A.
KEUFFEL & ESSER CO.



PRISMATIC CHANNEL
WEST STATES ENG. WKSHOP
4/25-29/83

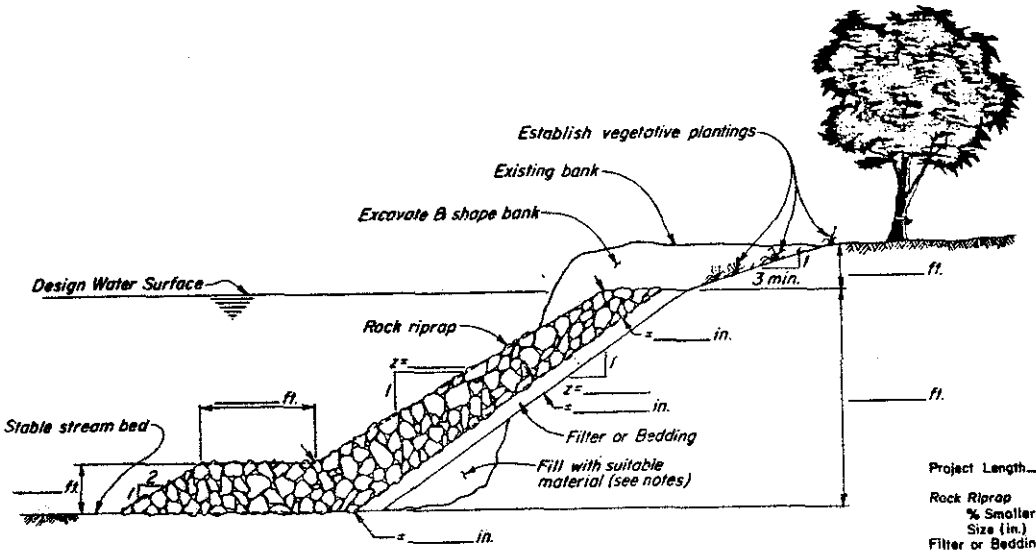
K&E SEMI-LOGARITH 46 5813
3 CYCLES X 140 DIVS
KEUFFEL & ESSER CO.



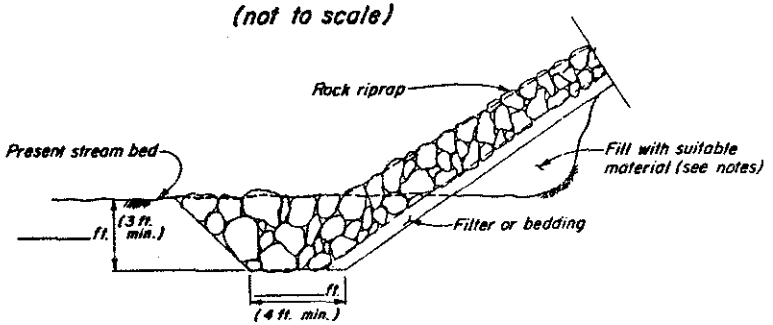
GENERAL NOTES

1. The project shall be constructed to SCS Standard Pro... ...pecification No. 580, Stream and Shoreline Protection. This specification is attached to this drawing and shall be part of this plan.
2. Vegetation within the construction site area that is to be retained will be marked on the site.
3. *The location and alignment shown is approximate.* The exact alignment is to be staked in the field.
4. The fill shall be placed in lifts of 12" or less and shall be compacted by one or more passes of hauling equipment.
5. Rock shall be from an approved source and shall be dense, sound and free from cracks or other defects conducive to accelerated weathering.
6. Rock shall be equipment placed so that larger rocks are uniformly distributed and in contact with one another with smaller rocks filling the voids. No end dumping of rock will be allowed.
7. Riprap will tie into a non-corrosive bank on the downstream end and will be keyed into the bank at the upstream end.
8. Work must be in compliance with the terms and provisions of all applicable permits.

SPECIAL NOTES



TYPICAL CROSS-SECTION
(not to scale)



ALTERNATE TOE CROSS-SECTION
(not to scale)

Project Length ____ ft.

Rock Riprap
% Smaller ____ D_{100} ____ D_{85} ____ D_{50} ____ D_{15}
Size (in.)
Filter or Bedding
Sieve Size ____ D_{100} ____ D_{85} ____ D_{50} ____ D_{15}
% Passing
Vegetative Planting
Mix ____
Seeding Rate, PLS ____ lb./Ac.
Woody Vegetation
Species ____
Planting Rate ____ plants/ft.^{2}
Fertilizer
Mixture ____
Application Rate ____ lb./Ac.

Rock Riprap ____ c.y.
Filter ____ c.y.
Bank Shaping
Excavation ____ c.y.
Fill ____ c.y.
Seeding ____ Ac.
Fertilizer ____ lb.

SPECIAL NOTES FOR DESIGN PURPOSE ONLY

DESIGN PARAMETERS
Drainage Area ____ sq. mi.
Stream Flow ____ c.f.s.
Design ____ c.f.s.
Bankful ____ c.f.s.
Stream Velocity (Design) ____ f.p.s.
Foundation and/or Bank Materials ____
____ U.S.C.S.
D_{100} ____ D_{85} ____ D_{50} ____ D_{15} ____
Estimated ____
Sieved Sample ____
Bank Seepage ____ Yes ____ No

1. Rock riprap should be no steeper than 1.5:1
2. D_{85} identifies rock size of which indicated % (85) is smaller.
3. Use special notes to describe site conditions, restrictions, desired fencing details, source of materials, or other information useful to the contractor.
4. Specify alternate toe cross-section, if applicable.

U. S. DEPARTMENT OF AGRICULTUR
SOIL CONSERVATION SERVICE

Designed ____ Date ____ Approved by ____
Title ____
Drawn ____ Title ____
Traced ____ Sheet ____ No. ____ of ____ Drawing No.
Checked ____

SCS-ENG-313-9 Rev