DRAINAGE AND CHANNEL DESIGN STANDARDS FOR LOCAL DRAINAGE

FOR
FLOOD PLAIN MANAGEMENT WITHIN
PIMA COUNTY, ARIZONA

Prepared by

Pima County Department of Transportation
and Flood Control District

Tucson, Arizona

Adopted by the Board of Supervisors
May 15, 1984

Effective June 1, 1984

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SECTION I

INTRODUCTION

The purpose of these design standards is to provide a guide for the design of open channels and other storm runoff structures. These design standards include procedures for design calculations and design criteria that are applicable for conditions in Pima County which are generally categorized by high velocity flow, and intense, but short duration, flow events. These standards and procedures should only be considered general guidelines and should not be substituted for sound engineering judgement when dealing with specific design problems.

SCOPE

The intent of these standards is to compile design standards and information from various publications and methods practiced by government agencies such as the U.S. Corps of Engineers and the Federal Highway Administration, and to present this information in a comprehensive manner that addresses critical design parameters. Therefore, the scope of these standards is limited to design information. Theoretical details concerning the hydraulic information has not been presented, but may be found in most standard text books and government publications, many of which are referenced herein.

These standards are most applicable to the local drainage problems encountered in the design of subdivisions, roadways, and hydraulic structures. Design information for major improvements such as dams or flood control levees are not included. Such major projects should be designed and reviewed on a case-by-case basis following applicable local, state, and federal regulations.

DEFINITIONS

The following definitions are provided to clarify information contained within the text.

1. Minor Watercourses will refer to any channel with a 100-year peak discharge which is less than 5,000 cfs. The category is further divided into major washes, minor washes, and local or nuisance flow.

   a. Major Wash refers to any stream which has a 100-year peak discharge between 5,000 cfs to 1,000 cfs (1,000 cfs $< Q_{100} <$ 5,000 cfs).

   b. Minor Wash refers to any stream which has a 100-year peak discharge less than 1,000 cfs but greater than 100 cfs (1,000 $> Q_{100} >$ 100 cfs).

   c. Local or nuisance flow is minor flow between adjoining lots, drainage or other flow which is equal to or less than 100 cfs during the 100-year event. (Note: Pima County regulates all drainage with 100-year discharge in excess of 50 cfs).

2. Major Watercourse are rivers which have 100-year peak discharges in excess of 5,000 cfs and includes but is not limited to such rivers as the Santa Cruz River,

Rillito Creek, Tanque Verde Creek, Pantano Wash, Julian Wash, Tucson Arroyo, and the Canada del Oro Wash. Unless noted otherwise, the design standards presented in this text are not generally applicable for the major watercourses and a more detailed analysis of the river mechanics and/or hydraulic analysis may be required.

3. Future Conditions implies the use of the most recent information, as presented in Area Plans, basin management studies or other official documents, to predict future runoff and flood conditions.

4. Rapid Flow includes critical and supercritical flow where the Froude number equals or exceeds one. Because in actual model studies the turbulence normally associated with rapid flow begins at a Froude number of 0.86, for design purposes flow will be assumed rapid if the Froude number exceeds 0.86 in cases where surface disturbances are important, as with freeboard requirements.

5. Tranquil Flow refers to subcritical flow with Froude numbers less than or equal to 0.86.

6. Freeboard refers to height above the design water surface to some critical point. It generally refers to the height from the water surface to the top of the channel. However, where the protection is adequately

tied into the sides of a channel, it may refer to the height from the water surface to the finished floor elevation of adjacent structures.

REFERENCES

The references given below were used in the preparation of these standards and are recommended to the Engineer in applying these design criteria.

1. Arizona Highway Department, Hydrology and Hydraulic Training Session, Section on Scour and Bank Protection; Culvert Outlet Protection.

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7. Henderson, F.M., Open Channel Flow, MacMillan Publishing, New York, 1966.

8. Keeley, J.W., Soil Erosion Studies in Oklahoma, U.S. Bureau of Public Roads.

9. Morris, H.M. and J.M. Wiggert, Applied Hydraulic in Engineering, John Wiley and Sons, New York, 2nd Ed. 1972.

10. Richardson, E.V. and others, Highways in the River Environment and Environmental Design Considerations, Training and Design Manual, Federal Highway Administration, U.S. Department of Transportation, 1975.

11. Sanders, T.G., ed., Hydrology for Transportation Engineers, Federal Highway Administration, U.S. Department of Transportation, 1980

12. Simons, D., R.M. Li, & W. Fullerton, Theoretically-Derived Sediment Transport Equation for Pima County, Arizona,Simons, Li & Associates, Inc., Fort Collins, Colorado, 1981.

13. Simons, D., F. Senturk, Sediment Transport Technology, Water Resource Publications, Fort Collins, Colorado, 1977.

14. Rouse, H. (ed.), Engineering Hydraulics, John Wiley & Sone, New York, 1950.

15. U.S. Bureau of Reclamation, Design of Small Dams, 2nd ed., 1974.

16. U.S. Soil Conservation Service, Design of Open Channels, Technical Release No. 25.

17. U.S. Conservation Service, Hydrology, National Engineering Handbook, Section 4, 1972.

18. U.S. Soil Conservation Service, Urban Hydrology for Small Watersheds, Technical Release No. 55, 1975.

SECTION II

DESIGN CONSIDERATIONS

For the design of drainage structures due consideration must be given to determining the design flood, flow characteristics and other factors defined by the basin and drainage system's hydrologic and hydraulic parameters. Additionally, methods for flood control and stormwater management must be compatible with the applicable regulations governing land use and flood plains.

Within Pima County acceptable methods have been developed for the determination of drainage basin characteristics, the generation of peak flow rates and other hydrologic parameters. These drainage standards have been developed to aid the Engineer in evaluating hydraulic parameters applicable to Pima County. The Pima County Floodplain Management Ordinance establishes and defines appropriate flood plain uses, encroachment standards and methods of stormwater management. The discussion below was prepared to define design parameters and standards, as setforth in the Floodplain Management Ordinance, that must be considered for the design of drainage structures.

DESIGN PARAMETER

Design of any drainage structure must include the determination of the applicable hydrologic and hydraulic parameters for the design flow conditins and the proposed improvements. Design requirements have been developed within these drainage design standards for various sizes of drainage basins and volumes or rates of flow, which because of their different flow characteristics, require separate design criteria. The design standards have been developed primarily for minor and major washes which carry design flows of 100 cfs to 5,000 cfs. Major watercourses with 100-year peak discharges greater than 5,000 cfs must be evaluated in much greater detail with attention paid to river mechanics. If flows are less than 100 cfs then less stringent standards for the design of these structures are generally required.

Hydrologic Parameters

The first step in the design of any structure is to determine the applicable design discharge. In general, for Pima County this implies the 100-year discharge for channel design capacity; however, other discharges such as the 2-year and 10-year discharges are used in determining channel stability, culvert design or other structures. All design discharges should be developed for future conditions using the best information available from land-use area plans and drainage concept reports.

For small urban or rural watersheds it is normally necessary to determine the design discharge from empirical formulas since streamflow records are not generally available. Pima County has developed a method for the determination of peak discharges which has been outlined in the Hydrology Manual for Engineering Design and Flood Plain Management within Pima County, Arizona, September, 1979. This method was developed for the prediction of peak discharges on small semi-arid watersheds with recurrence intervals of 2 to 100 years. As with other methods for small watersheds, this method was developed for a 1-hour storm which occurs uniformly over the basin. When the basin's size increases this assumption becomes less valid; therefore, this method should not be used for watersheds in excess of 10 square miles. For watersheds of 10 square miles or greater it is suggested that the Engineer use the Soil Conservation Service method for peak discharge determination as this method will consider flows from various duration storms and uses an averaged rainfall depth to correct for nonuniformity. Information on the Soil Conservation Service Method for watersheds larger than 10 square miles may be found in SCS's National Engineering Handbook 4 - Supplement A or as modified in Hydrologic Design for Highway Drainage in Arizona, which was prepared by the Arizona Highway Department's Bridge Division.

For most of the major watercourse, regulatory discharge have been developed by the federal government using streamflow data and the Log Pearson Type III distribution, or, where streamflow data was not available, the use of regional equations developed by the U.S. Geological Survey were used. For major watersheds these regulatory 100-year peak discharges will be used for most design purposes.

Hydraulic Parameters

Prior to the preparation of a final design the existing hydraulic conditions and the proposed improvement concepts should be evaluated. Investigations of the existing and future conditions should not be limited to the site but should also include an evaluation of the drainage basin. This does not imply that a detailed study must be conducted; however, a review of the existing and future conditions, including the basin's physical characteristics, will eliminate unnecessary analysis. The Engineer should provide documentation of the necessary design criteria and hydraulic parameters requiring evaluation in any hydrology and/or hydraulic report submitted to Pima County.

Major hydraulic parameters to be considered are channel cross-sectional dimensions, channel slope and velocity. The channel cross-section should be compatible with the upstream and downstream channel conditions. If they are not compatible, additional measures must be incorporated to compensate for variations in flow velocities. Where the Froude number is approximately equal to one, slight changes in energy may result in large changes in the depth of flow. The channel slope and flow velocity have a significant impact on the stability of the channel. If the flow velocity is non-scouring then the channel slope may generally be assumed to be stable and no bank protection would be required unless side slopes of steeper than 3:1 are desired or unusual conditions exist.

With major watercourses, engineering analysis of flow hydraulics and river mechanics must be made in greater detail than for small washes or constructed channels. Flow in the major watercourses during flood events will be unsteady and non-uniform due to variation in channel capacity, channel characteristics and the sudden surge of flood peaks as found in this arid Southwestern Area. Particular hydraulic conditions found in Pima County include river erosion typified by channel meandering and/or degradation of the river bed; perched flow in the overbank areas or along alluvial fans; wave formation or sudden changes in water surface elevation, since many rivers are flowing at critical depth; and localized scour and/or problems caused by sudden channel contractions and encroachments. Detailed hydraulic analysis should be made for any proposed improvement which determines flood limits, flow depths and changes in flood velocities. Where flow is near critical or is supercritical, the sequent depth of the flow should be considered. Because most hydraulic models are one-dimensional rigid bed models, under special conditions an evaluation of the sediment transport capacity or a two dimensional evaluation of the flow's momentum must be made. Since erosion is a major problem in Pima County, the Pima County Flood Control District has had general analyses of sediment transport capacity of the major washes performed. This is adequate in most instances to set <u>general</u> design requirements except where localized conditions are crucial or the intended improvement will cause local scour.

FLOOD PLAIN MANAGEMENT

The Pima County Flood Plain Management Ordinance and related land use plans regulate the use of flood plains and floodways in Pima County. Additionally, the Flood Plain Management Ordinance gives the County Engineer the authority to regulate hydrologic, hydraulic and related flood plain use standards which pertain to engineering decisions. Major provisions of the ordinance pertain to land use suitability for flood plains and floodways, encroachments into the flood plain; subdivisions including building sites, internal drainage, building setbacks, and stormwater management by the use of detention/retention.

The ordinance also sets up procedures for the submittal of Engineering Studies and their review and approval for all flood plain use permits and/or drainage improvements. The drainage standards presented in this text were prepared in order to address the possible design or engineering decision criteria required for the enforcement and administration of the Flood Plain Management Ordinance.

POLICIES

Policies concerning drainage standards have been established by the Pima County Engineer and are listed below. Variance to these policies are only allowed with the written permission of the Pima County Engineer and any request for variance must be supported by acceptable documentation and engineering analysis. The Flood Control District's Design Policies are:

1. All streams with 100-year peak discharges equal to or greater than 50 cfs will be considered in the design of improvements, encroachments or other flood plain activities as regulated by the Pima County Flood Plain Management Ordinance.

2. Flow must not be diverted from one drainage basin into another.

3. Finished floor elevations shall be elevated at a minimum of one foot above the existing regulatory 100-year water surface elevation.

4. For constructed channels, the 100-year peak discharge must be contained in a defined channel cross-section. Defined channel cross-sections may include a constructed low-flow channel with graded overbank areas as long as the 100-year flood limits are contained within the established drainage right-of-way or easement and does not encroach into private lots.

5. For major watercourses, with straight alignment, buildings shall be setback 300 feet for all structures unless bank protection is provided. In some cases, the 300 feet criteria may be increased or decreased by the Pima County Engineer as warranted by the anticipated erosion hazard.

6. All properties adjoining a stream channel or watercourse have a right to an equal degree of encroachment which is measured by the loss of conveyance of flow. To insure that encroachment into the regulatory flood plain does not increase flooding for existing structures or reduce the right of

adjacent properties, encroachment into the flood plain will not be allowed to cause a significant change in flow depth or velocity on adjacent properties. See Section III for definitions and exceptions.

7. All lots within a subdivision shall be provided with all weather access, i.e., a minimum of one route of ingress and egress where the 10-year flow is carried under the roadway, the 25-year flow is one foot deep or less in the dip-section, and 100-year flow is contained within the dip-section. Where flow is less than 100 cfs flow may be carried in a dip-section provided the depth of 100-year flow is one foot or less.

8. Drainage channels may not be constructed adjacent to roadways without written permission from the County Engineer.

9. Dedication of right-of-way or easement is required for all major watercourses with a 100-year peak discharge of 5,000 cfs or greater.

PLAN SUBMITTAL

Prior to the development of any site, certain requirements must be met to assure the proper design and engineering of all associated facilities. This includes a review of existing hydrology and hydraulic characteristics of the site as well as an analysis of all proposed drainage improvements. Requirements and procedures for development plans, or in the case of land subdivision, tentative and final plats, have been outlined by the Subdivision and Development Review Committee. Specifically, the development plan and tentative plat provide detailed information on the existing and proposed drainage improvements and all other improvements to establish their feasibility. The final plat must conform to the approved tentative plat and shall identify and dimension all drainageways/drainage easements and establish appropriate covenants and dedications. A checklist of requirements relating to drainageways, as well as other development requirements, has been included in the Procedures and Requirements for Tentative Plats, Procedures and Requirements for Final Plats, and Development Plan Check List which are available from the Pima County Planning and Zoning Department.

Both development plans and tentative plats require the submittal of a Hydrologic and Hydraulic Drainage Report prepared by a Professional Engineer. This report should include a description of existing and future site conditions; all hydraulic and hydrologic data and computations sheets; a delineation of the flood prone areas, and a detailed description of the proposed manner in which drainage shall be handled. The tentative plat should show existing condition and improvements with the final plat reflecting post development conditions.

Drainage may not be altered, disturbed or obstructed other than as shown on the approved development plan or tentative plat without written approval of the Pima County Flood Plain Board. Additionally, improvement plans for channels, detention basins or other hydraulic structures must be submitted to the

Flood Control District for review and approval prior to construction. These improvement plans must be accompanied by all hydraulic calculation sheets (with references where appropriate), a description of all proposed improvements, and an evaluation of the compatibility of the designed improvements with the concepts and condition described in the Hydraulic and Hydrology Drainage Report.

DEDICATION

Dedicated drainageways or easements shall be provided where necessary to insure proper flow conditions and maintenance. Dedication is required for any major watercourse channel and/or floodway where the 100-year peak discharge is greater than 5,000 cfs. Dedicated drainageways must conform to adopted County policies and the standards outlined within this text. This includes but is not limited to providing access easements, all weather access and the use of proper engineering design standards.

For the dedication of a drainageway or other improvements, the limits of the improvements and/or 100-year flood prone area must be shown in a surveyable manner on the final plat. Any right-of-way or easement shall be clearly labeled on the final plat as private or public. Dedication of any drainage improvements shall be made with the final plat and shall follow the standard format as outlined in Procedures and Requirements for Submitting Final Plats.

HYDROLOGIC AND HYDRAULIC DRAINAGE REPORT

Eacn Hydrologic and Hydraulic Report submitted in conjunction with subdivision plats, development plans, and/or any other specific parcels of land which require drainage analysis, shall contain the following:

(1) A "Cover Sheet", which includes:

(a) Title of Report

(b) Approximate location of project, including Section, Township and Range

(c) Seal and Signature of a Registered Professional Civil Engineer

(2) An "Introduction", which includes:

(a) Legal description of project, along with a Location Plan which shows the physical relationship of the project to nearby properties, as well as major streets and waterways within the immediate vicinity.

(b) A description of existing development within the watersheds affecting the project itself.

(c) A description of future development anticipated within the watersheds affecting the project, including the use(s) planned for the project.

(d) A recent aerial photo of the project area, at a scale no smaller than 1" = 1,000' extending at least 300 feet outside the project boundaries.

(e) Description of any physical features within the project, or contributing watersheds, which might be noteworthy from the standpoint of hydrologic and/or hydraulic considerations, such as ground cover, soil types, etc.

(f) A brief summary of any historical hydrologic and/or hydraulic information known to be available for the project. The source and date of information should be included.

(3) An "Objectives" section, which includes a brief description of the purpose of the report in relationship to development of the project.

(4) A "References" section, which contains all sources and dates of information used to compile the report.

(5) A "Procedure" section, which briefly describes the methodology and assumptions used in preparing the report.

(6) A "Computations" section, which includes:

(a) A watershed map, delineated on a 7.5 minute USGS quadrangle map or on 1" to 200' or 1" to 400' aerial coverage when topographical information is available and included on the aerial. This watershed map should reflect the drainage areas and corresponding points of concentration affecting the project. A 15 minute USGS quadrangle may be acceptable only for larger drainage areas, or when a 7.5 minute USGS qudrangle map is not available for the project area. Aerial photographs without topographic information may only be employed as visual aids unless the drainage system and/or topographic relief is so ill-defined that use of USGS quadrangle maps alone is not possible. In such instances, elevations of the drainage divides, outlet, points of significant slope breaks, as well as any other elevations which may be pertinent to hydrologic and hydraulic considerations, shall be included upon the aerial photographs utilized. Watershed maps prepared on tracing paper are not acceptable.

(b) All hydrologic data sheets prepared while determining the quantities of flow affecting the project, including design rainfall sheets. These sheets shall be identical in form to those provided in the Hydrology Manual for Engineering Design and Flood Plain Management Within Pima County, Arizona.

A data sheet shall be required at each drainage concentration point where a significant watercourse enters and/or exits the project.

(c) All hydraulic data sheets prepared while determining the depth of flow, velocity of flow, Froude number, and aerial extent for each flood prone area contained within the project, clearly identified and labeled. Any sheets containing pertinent cross-sectional data, as well as all rating curves, shall be included. If computer analysis is employed, it should be labeled and included along with cross-section sheets plotted to scale.

(d) All hydraulic sheets prepared in analyzing the influence upon the drainage within and/or adjacent to the project from existing and/or proposed structures to include but not limited to levees, culverts, bridges, and roadways which act as weirs.

(7) A copy of a contour map or, if the project involves subdivision of land or commercial-industrial development, a copy of the tentative plat and/or development plan which includes:

(a) Delineation in a clear and precise manner of all flood prone areas subject to flows exceeding fifty (50) cubic feet per second (cfs).

(b) Clear identification and labeling of each cross-section used in mapping the flood prone areas, so that easy cross-reference to associated hydrologic and hydraulic data sheets is possible.

(c) Labeling of all significant points of drainage concentration which enter and/or leave the project, accompained by the quantities of flow and contributing drainage areas.

(d) Contours clearly plotted at intervals of two (2) feet, unless unusual topographic relief dictates otherwise.

(e) A scale no smaller than 1" = 100', unless unusual physical features dictate otherwise.

(8) A "Results" section, which briefly discusses or displays clearly the findings of the report or calculations. This would include items such as tables labeling data and results of any hydraulic analyses.

(9) A "Conclusion" and/or "Recommendations" section which describes in detail how the drainage affecting the project will be handled in a manner which will allow the development to occur as intended without conflicting with any State and/or County regulations or without adversely affecting adjacent properties and/or the project itself.

SECTION III

CHANNEL DESIGN

The purpose of this section is to discuss the policies and design standards for natural and artifical flood control channels. This section shall outline the requirements for construction along natural channels, standards for encroachments into flood plain fringe areas and design criteria for constructed channels. Discussion of channel alignment and channel stabilization standards have not been included as these topics shall be addressed separately in later sections.

NATURAL CHANNEL REQUIREMENTS

Incised stream channels which have been historically stable should be left in a natural condition if the channel has adequate capacity to convey the expected future 100-year peak discharge as determined from accepted area plans and/or expected land use. For minor watercourses, adequate channel capacity shall be determined by either a backwater analysis or from the following formula which equals the hydraulic depth of flow from a Manning's rating plus a freeboard requirement.

$$\text{Channel depth} = y + \frac{1}{6}\left[y + \frac{v^2}{2g}\right] \qquad \text{eq. 1-III}$$

Where: y = hydraulic depth of flow from a Manning's rating for minor washes or a HEC-II analysis.

v = flow velocity as determined from a Manning's rating for minor washes or a HEC-II analysis.

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With major watercourses, adequate channel capacity shall be determined by a backwater analysis, consideration of the energy grade line and other factors to insure safe design.

For minor watercourses, with a 100-year peak discharge less than 5,000 cfs, a general building setback requirement of 50 feet has been established. The Pima County Flood Control District may determine building setbacks on a case-by-case basis, unless an engineering study, which establishes safe limits, is performed by a registered professional civil engineer and is approved by the County Engineer. When determining building setback requirements, the County Engineer shall consider danger to life and property due to existing flood heights and/or velocities and possible channel meandering.

For major watercourses the following building setbacks shall be required where bank protection is not provided.

(1) Along major watercourses where no unusual conditions exist (e.g., severe meanders, large excavation pits, etc.), a setback of <u>300 feet</u> measured from the nearest top of the channel bank shall be provided at the time of development.

(2) Along major watercourses where unusual conditions do exist, setbacks shall be established on a case-by-case basis after a thorough engineering review.

In lieu of building setbacks, bank protection may be provided in accordance with regulations set forth in Section V which deals with channel stabilization. The building setback requirement may be increased or decreased according to the demonstrated channel stability. All building setbacks shall be shown on the Final Plat as per the streets, alleys, easements and drainage requirements for submittal of Final Plats.

ENCROACHMENT STANDARDS

Whenever encroachment into a flood plain area is proposed, the Pima County Flood Plain Management Section shall consider the danger of life and property due to the existing flood height and/or velocities, or the danger due to the increase in flood height and/or velocities caused by the encroachment. Where encroachments are to be allowed, all property owners have the right to an equal degree of encroachment. The standard used to measure the degree of encroachment shall be the change in the depth of flow and/or the hydraulic efficiency of the channel. An equal degree of encroachment will be determined from the effects upon the flow in the channel rather than equal encroachment distances. Encroachments should not reduce the hydraulic efficiency of the channel nor increase the average water surface elevation by more than one-tenth of a foot.

Pima County's policies and drainage standards for the measurement of encroachment specify that the change in flow characteristics is measured by the average of change throughout the encroachment reach with some variation in the magnitude of the change; i.e., the one-tenth criteria, allowed to ensure sound engineering design and hydraulically smooth flow transitions.

Special situations where the entire flood plain is encompassed by a single landowner or where agreements are obtained from adjacent property owners will be evaluated by the Pima County Engineer on a case-by-case basis considering interim and long term conditions. However, at no time shall encroachment occur in the floodway, affect the river stability, increase downstream flood peaks, or change offsite flow characteristics.

Exception to the encroachment standards may be made on a case-by-case basis only where the entire flood plain remains under development by an individual or corporate ownership. Under these conditions, any modification to the flood plain must not raise the 100-year water surface by more than one foot within the development (Note that the city ordinance standards may differ). In the adjacent upstream and downstream reaches the hydraulic characteristics of the channel must not change nor may the depth of flow be increased by more than 1.0 foot. Where encroachment is adjacent to a major watercourse, the County Engineer will retain the option to request that a study be made of the reduction in overbank storage in order to demonstrate that the encroachment will not significantly increase downstream flood peaks. These studies will be regional in nature and will have their study limits set by the County Engineer.

Encroachment by structures may only occur within the flood plain fringe area. All development structures must be outside the floodway. The engineer is referred to the Pima County Flood Plain Ordinance for definitions of allowed land use within the floodway and flood plain fringe. Whenever the floodway limits have not been defined by either FEMA Flood Boundary and Floodway Maps or by a private engineering study performed by a registered professional civil engineer and accepted by the County Engineer, the floodway limits shall be setback from each side of the channel top of primary bank a distance of 4 times the channel top width as measured from the top of the nearest bank.

Individual Building Site

Individual structures or buildings may be erected within the flood plain provided that the depth of flow and the velocities are not excessive. The depth of flow (d) and the velocity (V) will be considered to have low hazard potential if the depth is less than three (3) feet and the velocities are less than five (5) feet per second.

For all individual building sites, the following standards shall be met:

1. Finished floors (first floor or basement) shall be elevated a minimum of one foot above the 100-year water surface.

2. The building pad will be raised, at a minimum, to the 100-year water surface elevation and shall extend out 25 feet in all directions from the edge of the building unless other approved measures, such as stem walls are utilized.

3. Where necessary, stabilization of the building pad shall be required by the County Engineer (See Section V).

4. Streets shall be elevated to the 50-year water surface elevation where required by the County Engineer.

5. Wherever possible, buildings shall be placed so that the longest exterior dimension is parallel to the direction of the flow or provisions are made to mitigate any blockage of flow.

6. All structures shall be anchored to prevent floatation.

Major Encroachments

Where several raised structures will be built or where development will raise land out of the flood plain, it is necessary that a hydraulic study, to determine the changes in water surface elevation and flow velocities, be conducted by a registered professional civil engineer and approved by the County Engineer. Encroachment shall only be allowed if the average change in the 100-year water surface elevation is 0.1 foot or less and if the increase in the flow velocity is less than 10% of the original flow velocity or less than or equal to 1 fps when the flow velocity is greater than 10 fps. Exceptions to this requirement may only be made if the development meets the criteria set forth under the general encroachment standards or where structural measures are taken to mitigate adverse effects. When conducting an hydraulic analysis of the flow, the change in the flow depth or flow velocity should not vary by more than 10% between each channel cross-section analyzed unless further analysis is provided to the Pima County Department of Transportation and Flood Control District for review.

The following standards shall apply to major encroachments into the flood plains.

1. Finished floor elevation shall be a minimum of one foot above the 100-year water surface elevation whenever the 100-year flood is not contained within a channel which has adequate freeboard, as described in the "Freeboard" portion of this section.

2. The top of the fill will be at or above the 100-year water surface elevation within 25 feet of the building edge.

3. All buildings shall be setback 25 feet from the edge of the fill on the landward side, unless other alternatives, as approved by the Pima County Engineer, are utilized.

4. The fill shall be bank protected against excess velocities and/or channel meandering (See Section V for an explanation of the criteria).

CHANNELIZATION

The purpose of this section is to present design criteria and procedures for design analysis of flood control channels. The formal hydraulic theory and details for each equation have not been included as these may be found in most standard hydraulic textbooks or publications. The engineer may also contact the Pima County Flood Control District for further details and references.

The following discussion outlines the design policies, describes the requirements for channel geometry and right-of-way and gives standard channel cross-sections which will be employed in Pima County.

Channel Geometry

Since the primary function of a flood control channel is to reduce damages and hazards from floods, the channel geometry must take into account the quantity of storm run-off, the freeboard requirement, superelevation of the flow and aggradation and erosion in the channel. Therefore, the design standards that should be applied shall depend upon the channel characteristics and the flow regime.

The following sections describe various cross-sectional elements which may be considered in channel design.

Top Width

Channel top widths for constructed channels should not vary by more than 15% between control points except at the confluence of a major tributary. The intent is to prevent radical changes in the width of flow or the channel slope in the main channel which could cause severe aggradation, degradation or meandering by the channel.

For similar reasons, when the channel that is to be constructed joins the natural upstream channel cross-section, as a general guideline, the discharge per unit width for the bank-full discharge of the natural channel (q_n) should be approximately equal to the discharge per unit width within the constructed channel (q_c) for a flood of the same magnitude. If this guide is followed in designing the channel size, aggradation and degradation within the constructed channel can be minimized.

Depth

While there are no restrictions to the depth of a channel, the channel depth should be kept within reasonable limits. For earthen channels, the channel depth should be limited by the allowable channel velocity for a stable channel (See Section V).

With lined channels erosion is not a problem, however, because of the steeper side slopes associated with concrete channels, it should be recognized that excessive channel depths could create a public hazard. Appropriate measures will be required where necessary.

Freeboard

Freeboard is the vertical distance between the top of the design water surface and the top of bank or critical point. Freeboard is provided to account for variations in the water surface from wave action, debris, accumulation of silt in the channel, or other possible perturbations. The City of Tucson freeboard standards may differ; therefore the engineer should consult with the Tucson City Engineer when designing improvements within the Tucson City limits.

The minimum requirements for freeboard shall vary according to the flow regime within the channel. With sub-critical or tranquil flow, the channel velocities are moderate so that wave action is also minimal. Therefore, the minimum freeboard requirements for subcritical fow in minor watercourses shall be one foot wherever the depth of flow exceeds three feet. When flow is near critical or supercritical, wave action increases and a possible change in the state of the flow and the water surface elevation may occur. For supercritical flow, the water surface elevation may increase due to disturbances within the channel such as anti-dunes, standing waves, and/or hydraulic jumps. Therefore, the depth of freeboard is dependent upon the total energy grade line. Hence, the freeboard of critical and supercritical flow, which is equal to or greater than three feet in depth, shall equal one-sixth of the specific head but not less than one foot in minor and major washes, i.e. $Q_{100} < 5{,}000$ cfs.

FLOW REGIME	FROUDE NUMBER	FREEBOARD	MINIMUM ALLOWED
Tranquil	$F < 0.86$	1'	
Near Critical or Supercritical	$0.86 \leq F$	$\frac{1}{6}\left[y + \frac{V^2}{2g}\right]$	1'

Where: y = depth of flow, ft.

V = velocity, fps

g = acceleration of gravity, 32.2 ft/sec.

Where the depth of flow is less than three feet, the minimum of one foot shall not apply, rather, the freeboard may be determined using the equation for freeboard $1/6\ (y + \frac{V^2}{2g})$.

Freeboard requirements in major watercourses will be decided on a case-by-case basis depending on existing river mechanics, flow regime, overbank conditions and other factors.

Additional requirements for freeboard may be called for in specific cases where aggradation is substantial during a single flow event and/or superelevation must be taken into consideration. See Section V for a discussion on aggradation in channels. For superelevation under supercritical conditions, the following formula is generally used:

$$\Delta y = \frac{V^2 W}{g r_c} \qquad \text{eq. 2-III}$$

Where: Δy = change in the water surface, ft.

V = channel velocity, fps.

W = channel width along the top of the water surface, ft.

r_c = radius of channel center-line curvature, ft.

g = acceleration of gravity, 32.2 ft/sec.

The change in the water surface, Δy, should be added to the freeboard requirement along the outside of the bend.

For rapid flow the disturbance caused by a bend in the channel persists downstream. Therefore, a detailed hydraulic study must be conducted to determine the effects of the channel curvature on the freeboard requirement. (See Section IV for a discussion of channel curvature.)

In cases where the channel is lined, the lining must be carried to the top of the excavated channel, unless the site has been graded to control drainage so that flow will only enter at spillways. Then the lining need only be extended to a height equal to the depth of flow during the 100-year peak discharge providing the top of bank exceeds the freeboard requirement.

Side Slopes

For earthen channels to be stable and non-erodable, the side slopes should be no steeper than 3 to 1. Lined side slopes may vary from vertical to flatter as long as appropriate reinforcement is provided. Soil cement slope paving may be used as an alternative channel lining. For this type of lining, side slopes of 6 to 1 are preferred, but they may be as steep as 3 to 1 where special equipment is used and/or dictated by accepted construction practices.

Right-of-Way

Dedication is required for all major watercourses, for either constructed channels or major natural channels where the 100-year peak discharge is more than 5,000 cfs. Drainage easements may be used in lieu of dedicated right-of-way for natural channels if the flood plain remains as a natural open space. Dedicated right-of-way for a constructed channel for minor and major washes, i.e. $Q_{100} < 5,000$ cfs, shall include the width required for the channel plus a sixteen foot access easement on both sides of the channel.

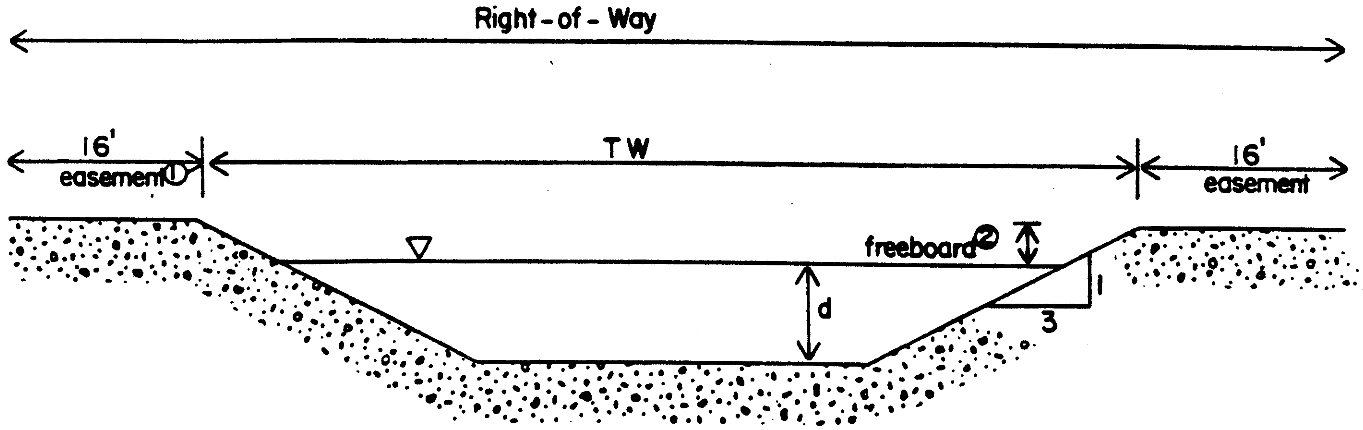
The requirement for an access easement on both sides of a constructed channel may be waived by the County Engineer when the channel bottom width is greater than 20 feet or when both sides of the channel are protected or determined to be stable. In this case, only one access easement, 16 feet wide, will be required. For major watercourses which have long duration flood hydrographs, such as the Rillito, Pantano, and Santa Cruz Rivers, access easements should generally be 50 feet in width on both sides of the channel, but these may vary. These access easements are necessary for adequate channel maintenance.

The exact width of access easement required will be determined by the type and extent of bank protection provided as well as whether adequate turning movement is provided for maintenance vehicles.

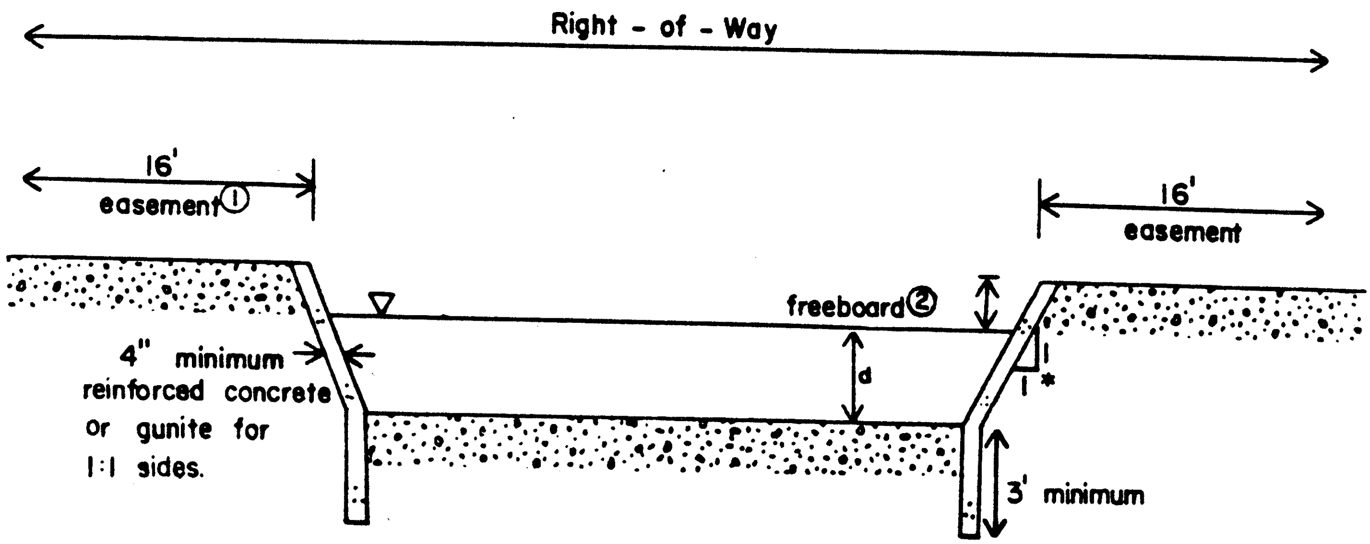
Standard Cross-Sections

The following standard cross-sections are being provided for illustration of the concepts outlined on the previous pages and are applicable to the design of channels for minor watercourses. Subdivision requirements should also be considered in the design of any channel.

CHANNEL CROSS-SECTIONS



Earthen Channel



*may vary from vertical to flatter with appropriate reinforcement.

Protected Sides With Natural Channel Bottom

① Where bottom width >20' only one 16' easement is required.

② $F < 0.86$ Freeboard = 1' minimum

$F \geqslant 0.86$ Freeboard = $\frac{1}{6}\left[d + \frac{v^2}{2g}\right]$; 1' minimum.

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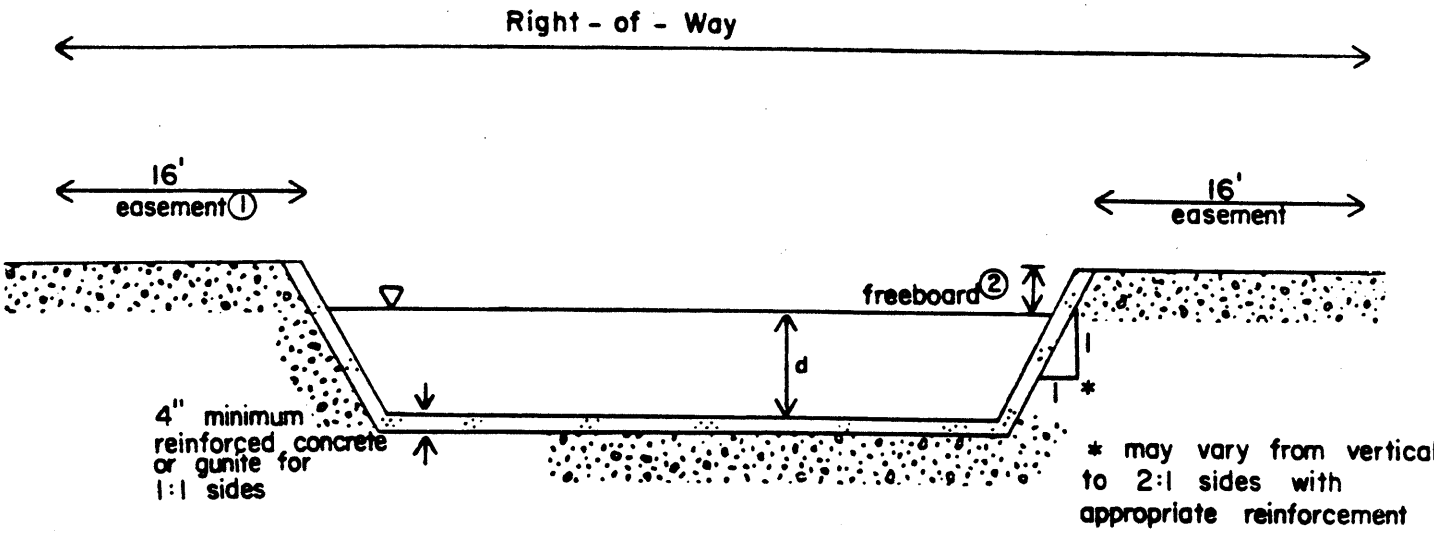
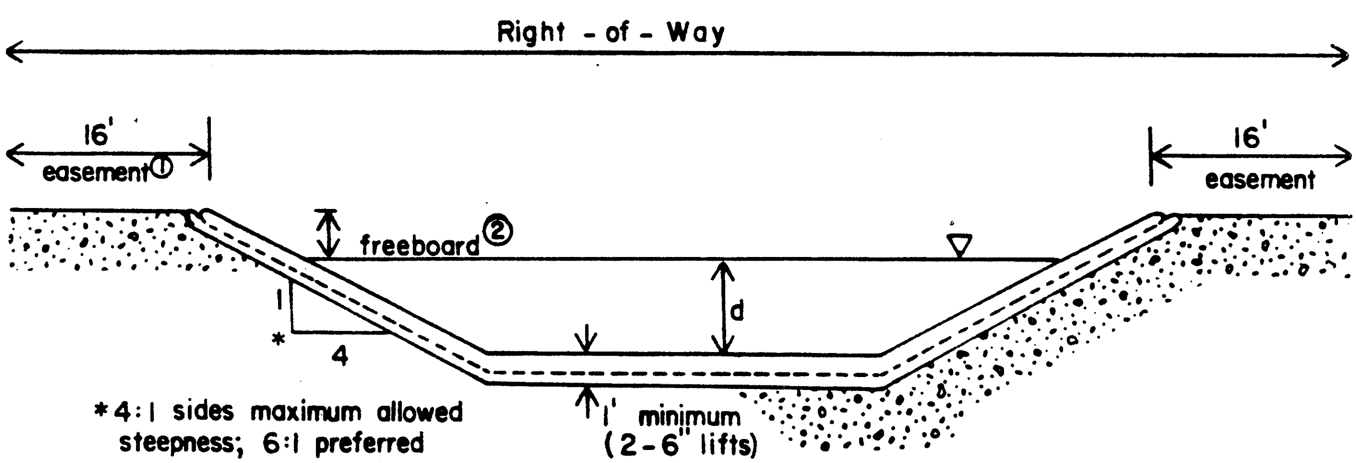
CHANNEL CROSS-SECTIONS



Concrete or Gunite Lined Channel (Excluding Control Structures)



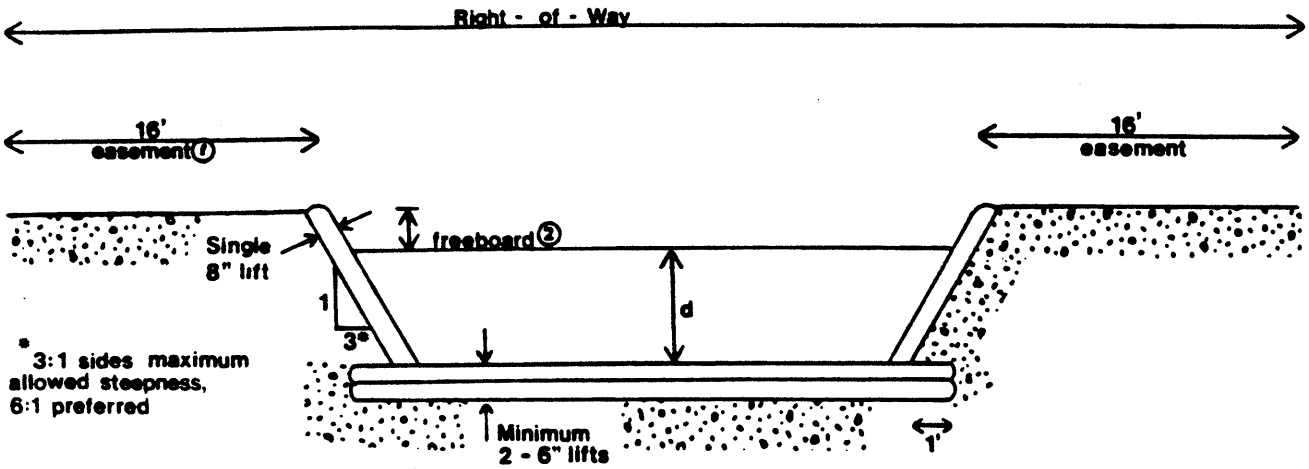
Soil Cement Lined Channel (Excluding Control Structures)

① Where bottom width >20' only one 16' easement is required.

② $F < 0.86$ Freeboard = 1' minimum*

$F \geq 0.86$ Freeboard = $\frac{1}{6}\left(d+\frac{v^2}{2g}\right)$; 1' minimum

CHANNEL CROSS-SECTIONS



Soil Cement Lined Channel
For $v_{100} \leq 15$ fps,
Frequency Of Flow Less Than
5 to 6 Times A Year

NOTE: For drainage easements no longer than one subdivision lot and with $Q_{100} \leq 50$ cfs these standard cross sections do not apply.

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SECTION IV

CHANNEL ALIGNMENT

Channel alignment affects the ultimate efficiency of the channel to convey flow. Unfavorable alignments may increase friction losses, cause siltation or scour in the channel, or create additional adverse effects in the upstream or downstream direction. This section shall outline channel alignment requirements and present technical details which will assist the engineer in designing channel facilities. Areas to be covered shall include general requirements, channel curvatures, transitions, and junctures.

GENERAL REQUIREMENTS

Some basic concepts must be applied if a drainageway is to be functional both within the site boundaries and as part of an area-wide drainage plan. These concepts include allowing for proper downstream and upstream controls, use of collector channels, and maintenance of channel bed slopes.

UPSTREAM AND DOWNSTREAM CONTROLS

Whenever possible, the flow path should be planned so that it may be logically expanded and/or enlarged at a later date in either the upstream or downstream direction. Where a master drainage concept is proposed, dedication will be required similar to highway and street dedication so that a Master Drainage Plan may be developed.

COLLECTOR CHANNELS

Large portions of many areas in Pima County are affected by "sheet flooding" (i.e., where the storm runoff breaks out of a defined channel and spreads out across alluvial fans). Whenever possible, offsite right-of-way should be obtained so that the flow can be collected at a logical upstream point where the flow is confined within a channel. However, this type of collection is not always possible if the distance from the development site to a logical upstream concentration point is too large. Where collector channels must be used, the following are general standards; however actual design will vary on a case-by-case basis to reflect site conditions.

1. The bed slope and cross-section design of the collector channel should be adequate to convey the expected 100-year peak discharge with adequate freeboard provided for aggradation in the channel.
2. Where a collector channel is upstream of residences the finished floor elevation of adjacent residences, i.e., the first tier of lots, shall be raised to a height one foot above either the collector channel bank or levee along the downstream side of the channel. See Figure 1 for the required cross-sectional design.
3. Collector channels, where they are to be placed adjacent to a street right-of-way, should be located on the upstream side of the right-of-way, where possible, to prevent the flow from encroaching onto or crossing the roadway.

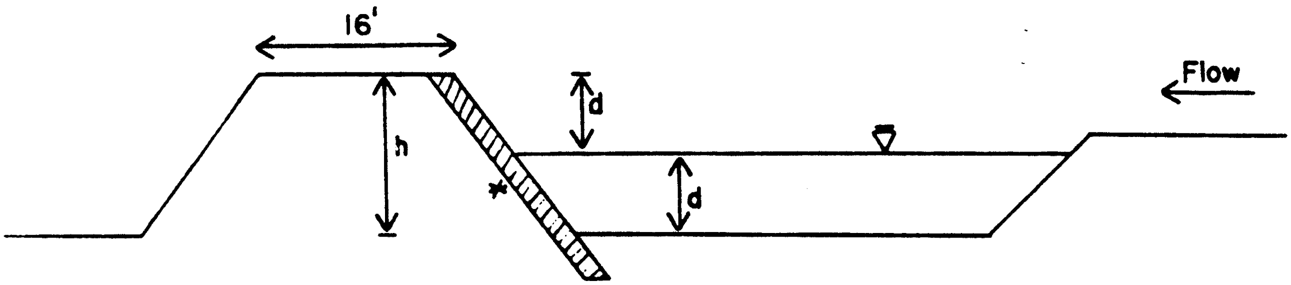

Figure I-IV: Collector Channel Design Cross-Section

h = Height of levee, ft.
d = Average depth of 100-year flow, ft.

* Levee shall be stabilized by soil cement, slope paving or other appropriate methods.

4. The outlet channel for the collector should be located so that when the upstream area is developed, any future channel may join into the downstream channel at a logical location.

CHANNEL BED SLOPE

It should be recognized that channel alignment has a significant effect on the channel bed slope. Quite often the location and elevation of the inlet and/or outlet is fixed, but variations in the overall channel alignment can be made which change the bed slope. Channels should be aligned so that erosion and degradation are minimized. It is equally important, where the available fall is less than 0.5 percent, that the channel alignment allows for the steepest slope possible in order to minimize aggradation within the channel. The engineer is referred to Section V which addresses problems of channel aggradation and degradation.

Whenever possible, abrupt changes in the bed slope should be avoided. Where changes are necessary, the change on the bed slope should not cause the channel top width to vary by more than fifteen percent.

Additionally, when flow within the channel is supercritical, changes in the bed slope may result in a hydraulic jump. It is therefore necessary that an engineering analysis be performed to determine the existing flow regime, and to determine if a hydraulic jump will form. Where a jump may occur, appropriate design measures should be utilized. However, this condition should be avoided.

HYDRAULIC JUMP

A hydraulic jump occurs when flow changes rapidly from supercritical to subcritical. When a hydraulic jump takes place, the lower stage supercritical flow impacts upon the higher stage subcritical flow, and the resulting turbulence and surface waves dissipate the upstream energy. Hydraulic jumps may occur at the following changes in channel alignment:

1. At sudden expansions;
2. Upstream of a sudden contraction;
3. At vertical changes in the channel alignment and abrupt changes in bed slope; and
4. Where a tributary with supercritical flow enters a channel which has a subcritical flow regime.

Where any of these types of abrupt changes in the channel alignment are proposed, it is necessary to determine if a hydraulic jump will occur and, if one does, the characteristics of the jump must be determined.

The approximate jump characteristics of length, height and location should be determined. The exact location of the jump is difficult to pinpoint and may change as the hydraulic jump develops. A jump cannot form unless the upstream Froude number is greater than one. For supercritical flow, a hydraulic jump will form if the flow downstream of the channel disturbance is subcritical. Table I-IV shows the type of jumps that are possible for varying upstream froude numbers.

TABLE I-IV. Types of Hydraulic Jumps

UPSTREAM FROUDE NUMBER	TYPE OF JUMP	CONDITION
$F \leq 1$	None	
$1 < F \leq 1.7$	undular jump	change in flow depth
$1.7 < F \leq 2.5$	weak jump	rollers may form
$2.5 < F \leq 4.5$	oscillating jump	larger irregular waves
$4.5 < F \leq 9$	steady jump	strong smooth jump
$F > 9$	strong jump	large disturbances downstream

As shown by the table, when the flow has a Froude number value of 1.7 or less, then disturbances to the flow or abrupt changes in the channel alignment will have little effect on the depth of flow in the channel, but wave action increases as the Froude number increases to one or greater. For Froude numbers greater than 1.7, significant changes in the flow depth and downstream wave disturbances may result from abrupt changes in the channel alignment. In these cases, the engineer is referred to the appendix for Section IV for a description of jump characteristics, jump height, and locations.

CHANNEL CURVATURE

Curves in a channel cause the maximum flow velocity to shift towards the concave side of the bend. Along the outside of the curve, the depth of flow is at a maximum. This rise in the water surface is referred to as superelevation. The shift in the velocity may cause cross-waves to form, which will persist downstream when the flow is supercritical. Severe erosion, deposition and reduced channel performance result from severe curvatures in channel alignment. To minimize the effect due to channel bends, channel curvature should only be used where topographic or other conditions necessitate their use. If the flow is supercritical, special designs may need to be employed to eliminate the downstream effects.

LIMITING CURVATURE

The centerline radius of curvature should be kept within a maximum allowable range. In design of a channel alignment, the maximum degree of curvature shall be limited according to the following criteria:

1. For subcritical flow, $F < 0.86$, $r_c \geq 3W$. eq. 1-IV
2. For transitional flow going into critical to supercritical flow, $F \geq 0.86$,

   $$r_c \geq 4 \left[\frac{V^2W}{gY}\right] \qquad \text{eq. 2-IV}$$
3. If $r_c \geq 10$ W. the channel alignment may be considered to be straight.

(References, Highways in the River Environment, Hydraulic and Environmental Design Considerations, Federal Highway Administration and Hydraulic Design of Flood Control Channels, Army Corps of Engineers).

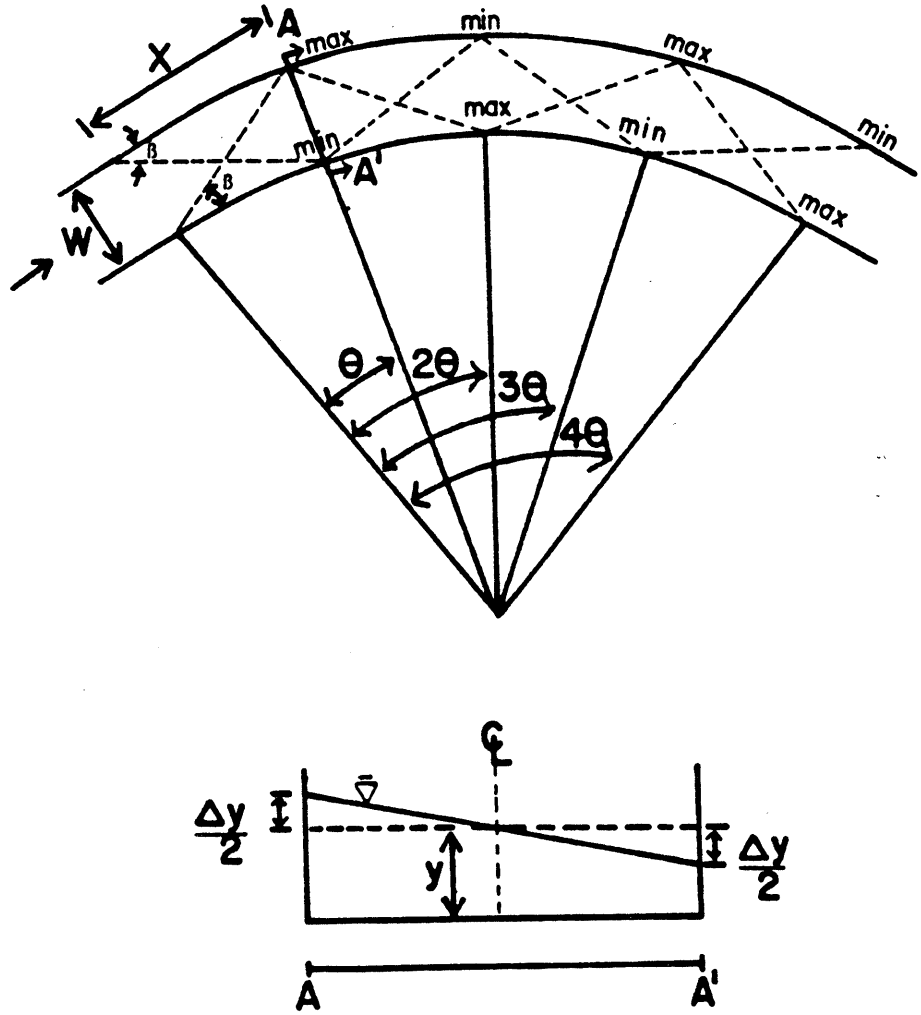

Figure 2-IV: Cross-Wave Pattern For Supercritical Flow In A Curved Channel.

$$\Delta y = \frac{v^2 W}{g\, r_c} \qquad \text{eq. 3-IV.}$$

$$X = \frac{W}{\tan \beta} \qquad \text{eq. 4-IV.}$$

$$\sin \beta = \frac{\sqrt{gy}}{v} = \frac{1}{F} \qquad \text{eq. 5-IV}$$

y = Depth of flow in channel, ft.

W = Channel top width at water surface, ft.

r_c = Centerline radius of curvature, ft.

g = Acceleration due to gravity, 32.2 ft/sec.

v = Average channel velocity, ft/sec.

X = Distance to first superelevation rise, ft.

β = Wave front angle, degrees.

θ = Phase angle, degrees.

F = Froude number.

In the above equations,

r_c = centerline radius of curvature, ft.

W = top width measured at the water surface, ft.

y = depth of flow, ft.

g = acceleration due to gravity, 32.2 ft./sec.

V = flow velocity, fps

SUPERELEVATION

The rise in the water surface, or superelevation, is given by the following formulas shown on Figure 2-IV.

Where: V = velocity of flow, fps

ΔY = rise in the water surface, ft.

W = top width of channel at the water surface, ft.

r_c = centerline radius of curvature, ft.

g = acceleration due to gravity, 32.2 ft./sec.

Note that the formulas are for rapid flow which is frequently the case in Pima County.

DESIGN CURVES

It is necessary with curved alignments that additional attention be directed to the design of the channel. The primary concern is the reduction or elimination of the cross-waves which are produced by the bend. Methods of designing curves include compound curves, banking, multiple vanes and diagonal sills. Of these methods, compound curves or banking are recommended. Use of multiple vanes and diagonal sills are restricted in use because of clogging problems when the flow carries debris. It is suggested that diagonal sills only be used as a remedial solution for existing channels.

1. Compound curves

Compound curves employ upstream and downstream transition curves that are designed to cancel out the effect of the cross-waves produced by the control curve. Compound curves should be used:

a. When the freeboard above the superelevated water-surface is less than one foot.

b. When it is necessary to eliminate cross-waves downstream, as when one channel bend follows another.

c. When the flow is supercritical in trapezoidal channels.

Compound curves should have a turning radius equal to twice the radius of the central curve (See Figure 3-IV). The length of the easement, L_C, is determined by the following equations:

$$L_C = \frac{0.32WV}{\sqrt{y}}, \text{ or } \quad L_C = \frac{W}{\tan B} \qquad \text{eq. 6-IV}$$

All variables are the same as those listed for equation 4-IV.

2. Banking

Channels may be banked to minimize the effect of the curve. Warping the channel bed slope will produce the desired water surface cross-slope needed to reduce the centripetal force. The equation for the channel bed cross-slope is:

$$S_{cs} = \frac{V^2}{gr_c}, \qquad \text{eq. 7-IV}$$

where, S_{cs} = cross-slope, ft/ft with all other variables the same as in equation 3-IV. See Figure 4-IV for

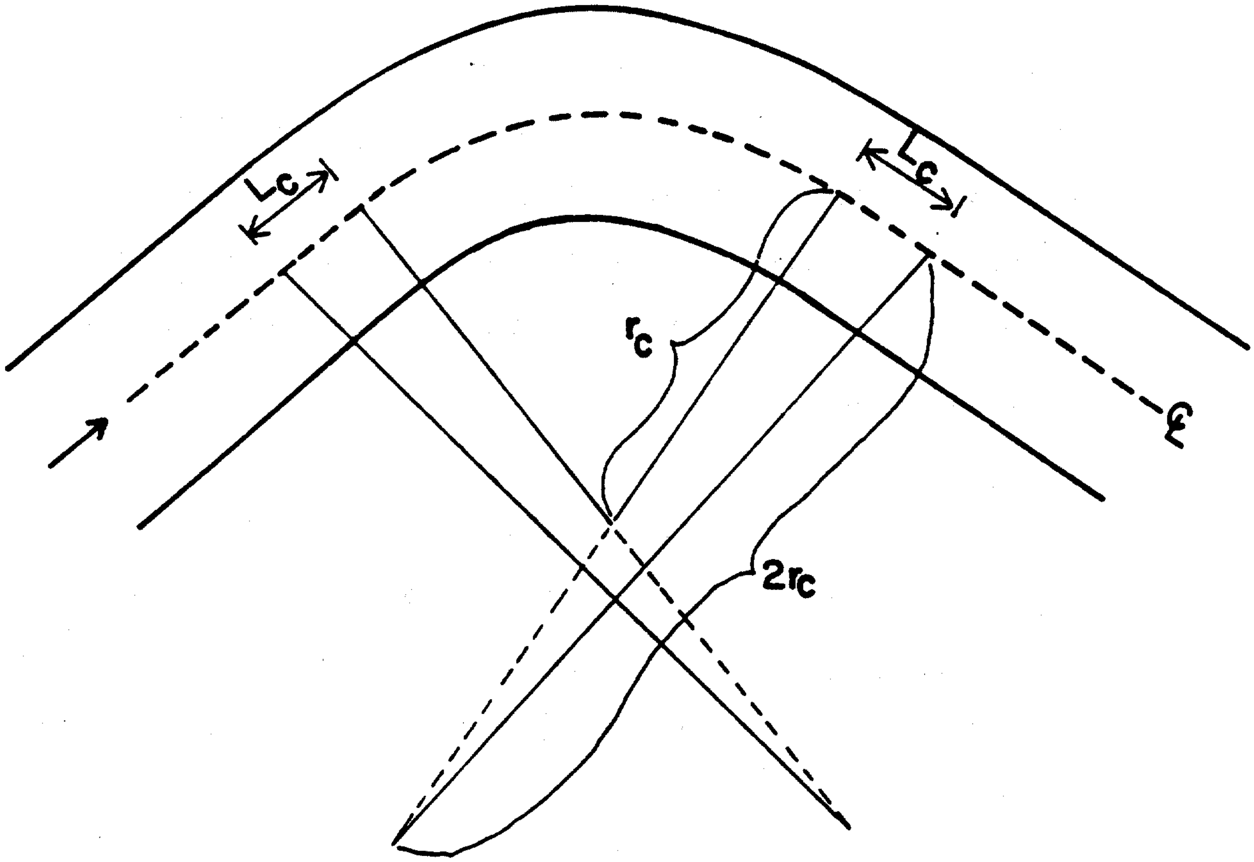

Figure 3-IV: Compound Curve Characteristics.

$L_c = 0.32 \frac{Wv}{\sqrt{y}}$, spiral transition.

$L_c = \frac{W}{\tan\beta}$, simple curve.

L_c = Length of easement curve, ft.

r_c = Centerline radius of curvature, ft.

W = Top width of water surface, ft.

v = Flow velocity, fps.

y = Depth of flow, ft.

$\beta = \sin^{-1}\left(\frac{1}{F}\right)$, F = upstream Froude number.

details for subcritical flow. The engineer is referred to <u>Open Channel Flow</u>, by Henderson, page 255, for a discussion on supercritical banking.

Since it is desirable that the banking have a gradual transition, the warping into the bend should begin at the start of the curve, with the warping then transitioned back to the normal horizontal bed at the end of the curvature.

3. Multiple Vanes

   The effect of the force of the flow going around the curve may be reduced by using concentric curve vanes to divide up the flow. This approach is not generally desired because of problems with debris and high maintenance costs.

4. Diagonal sills

   Diagonal sills reduce the flow velocity around a curve so that the downstream effect from the cross-waves is diminshed. As with the multiple vanes approach, debris is a problem when sills are employed. For this reason, they are not suggested for general use.

TRANSITIONS

Where an abrupt expansion or contraction occurs in a channel, special design considerations must be given to the transition to prevent an increase in the flow depth from backwater effects, excessive velocities, hydraulic jumps or other disturbances.

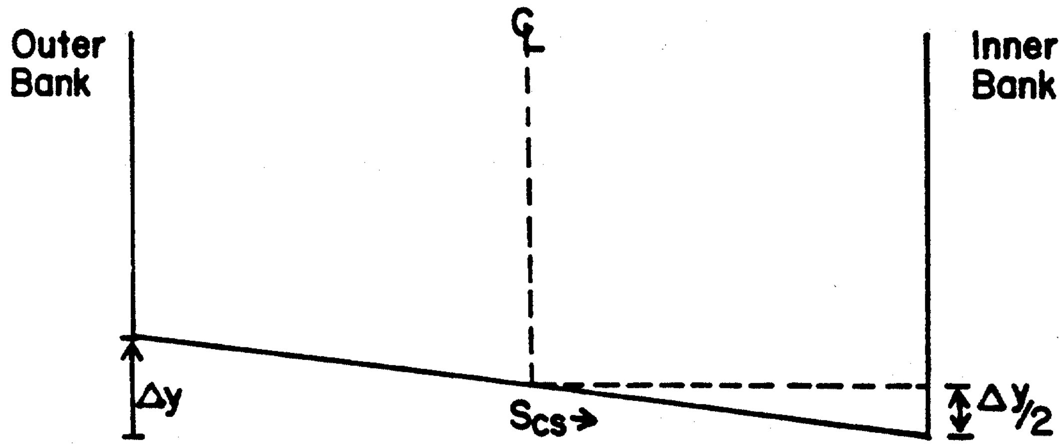

Figure 4-IV: Channel Cross-Slope For Banking Flow.

$$S_{cs} = \frac{v^2}{g r_c}$$

S_{cs} = Channel cross-slope, ft/ft.

v = Channel velocity, fps.

g = Acceleration due to gravity, 32.2 ft/sec.

r_c = Centerline radius of curvature, ft.

Δy = Change in depth of flow.

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Special transition designs are necessary whenever natural flow is entering a constricted channel, or if channelized flow is released into a natural channel, or where there is an abrupt change in the channel geometry. There may be other conditions superimposed on the transition reach that may affect the final design. The following transitions are for the simplest cases with all other features held constant. Conditions may vary so that additional requirements may be called for in the transition design.

ENTRANCE TRANSITIONS

When the incoming flow is in a natural condition, it usually has a much broader floodplain than the downstream constricted channel. In order to collect the flow so that the abrupt contraction does not cause a backwater effect, which increases the upstream flood hazard, a gradual transition should be employed. As a maximum the transition angle between the upstream and downstream flow line in the channel and transitioning levee or channel banks should be equal to:

$$\theta = \tan^{-1}\left[\frac{1}{3.375F}\right] \qquad \text{eq. 8-IV}$$

Where: θ = transition angle, degrees.

F = upstream Froude number.

Or, if flow is supercritical, wall flare transitions of 1:10, For $V < 10$ fps, or 1:15, for $V > 10$fps may be employed. See the appendix, page IV-21.

However, it is desirable that the transition angle θ be no more than 30 degrees. Special transition design techniques may be utilized to transition the flow at an angle greater than 30 degrees only with approval of the Pima County Flood Control District.

Where the effect from backwater would be critical and for all major water courses, it must be demonstrated that the upstream effect of the contraction is insignificant, i.e., the change in the depth of flow is less than 0.1 foot and the velocity does not change by more than 1 fps or 10 percent of the original velocity when it is less than 10 fps. A standard backwater analysis or HEC-2 analysis may be employed to demonstrate that the effect is insignificant.

The effect from backwater will be considered critical if structures are located upstream of the construction and/or the upstream channel does not have adequate freeboard. The analysis must be prepared by a registered professional civil engineer and reviewed and approved by the Pima County Engineer.

As was noted in Section III, the bankfull discharge per unit width in the natural channel (q_n) should be approximately equal to the discharge per unit width in the downstream constructed channel (q_c) for a flood of the same magnitude. This will insure both equal flow and sediment conveyance for the average flow events.

The transition design described above is adequate in most instances where flow is being contracted or where it is expanded into a constructed channel. However, where supercritical flow is being contracted and it is important to minimize downstream wave action, additional requirements for the transition angle, O, and the transition length, L, are necessary. In these cases, the engineer is referred to the accompanying appendix for more detailed information.

EXIT TRANSITIONS

Unless flow leaving a site is entering a constructed drainageway, the flow must be returned, as nearly as is feasible, to the pre-existing conditions, i.e., natural, <u>before</u> it is discharged from the site. This is to prevent adverse downstream effects, such as scour from increased upstream velocities, that would otherwise be created by higher discharges per unit width. To accomplish this purpose, the upstream channel flow must be transitioned with an energy dissipator or the flow may be transitioned over a length, X, where

$$X = 6.65F_1\left[(Z_2 - [0.7] \times [Z_1)]\right] \qquad \text{Eq. 9-IV}$$

Given: X = transition length, ft.

Z_2 = maximum length of a line drawn perpendicular from the projected centerline to the natural flood plain limit, ft.

Z_1 = one-half of the upstream channel top width, ft.,

F_1 = upstream Froude number

See Figure 5-IV for detail

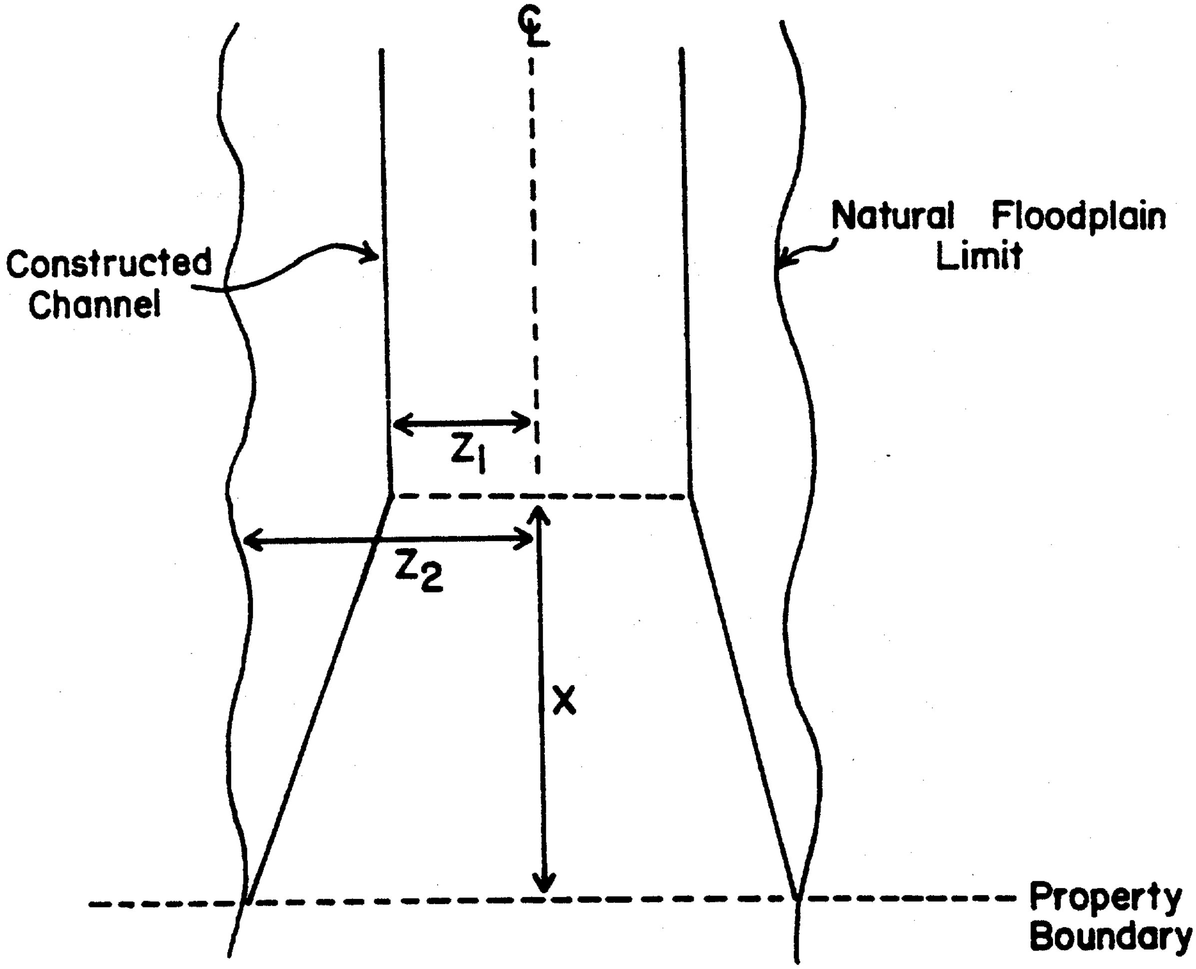

Figure 5-IV: Transition Distance Needed To Allow The Flow To Return To Natural Conditions.

$$X = 6.5\ F_1(Z_2 - 0.7\ Z_1)$$

where X = Transition distance, ft.

Z_2 = Maximum length of line drawn perpendicular from projected centerline to intersection with natural floodplain limit, ft.

Z_1 = One-half of upstream channel top width, ft.

F_1 = Upstream channel Froude number.

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INTERNAL CHANNEL TRANSITIONS

For internal channel transitions, where the flow is to be expanded or contracted, a transition should be designed which minimizes the flow disturbances. Gradual transitions will reduce the amount of head loss and minimize the height of the standing waves.

The angle of divergence of this channel, θ, should be no greater than the value determined using equation 8-IV where

$$\theta = \tan^{-1}\left[\frac{1}{3.375\ F}\right]$$

Again, the desired allowable value of θ is 30 degrees.

The required length for the transition is,

$$L_T = \frac{\Delta W}{2 \tan \theta}, \qquad \text{eq. 10-IV}$$

where, L_T = Transition length, ft.

ΔW = Change in the flow width, ft.

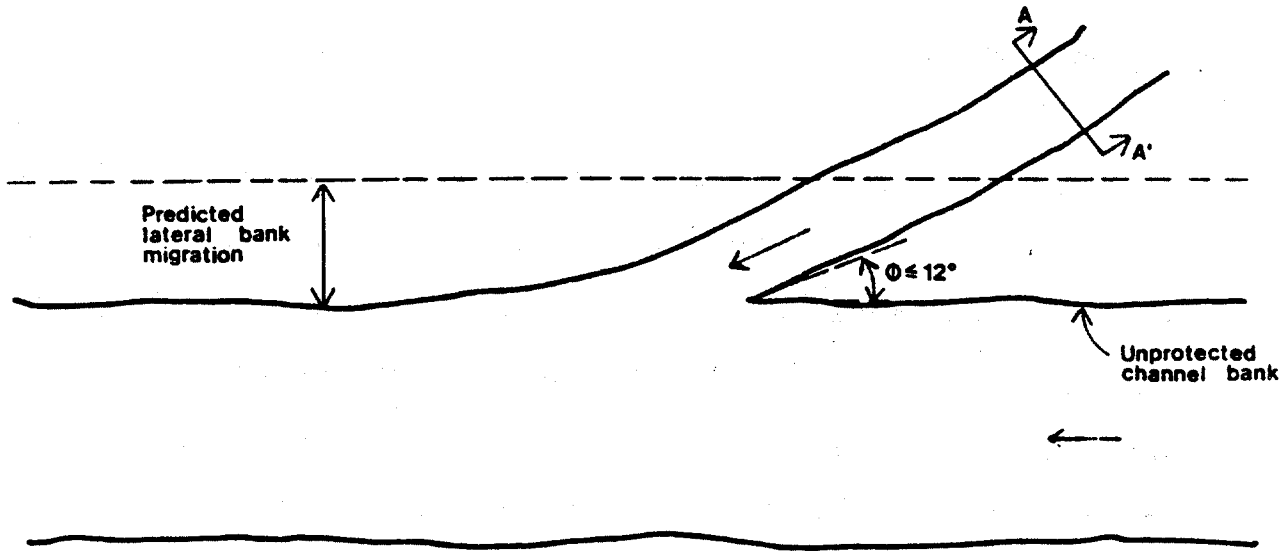
These formulas are adequate for reducing disturbances with a simple linear contraction or expansion. This type of a transition is sufficient where flow is channelized in both the upstream and downstream reach. Where the contraction in a natural stream would cause a jet of water downstream or for a major structure, such as a bridge, a more detailed analysis and transition design will be required. In the accompanying appendix details for a curvilinear transition are given for expanding flow where a downstream jet is anticipated.

CONFLUENCES

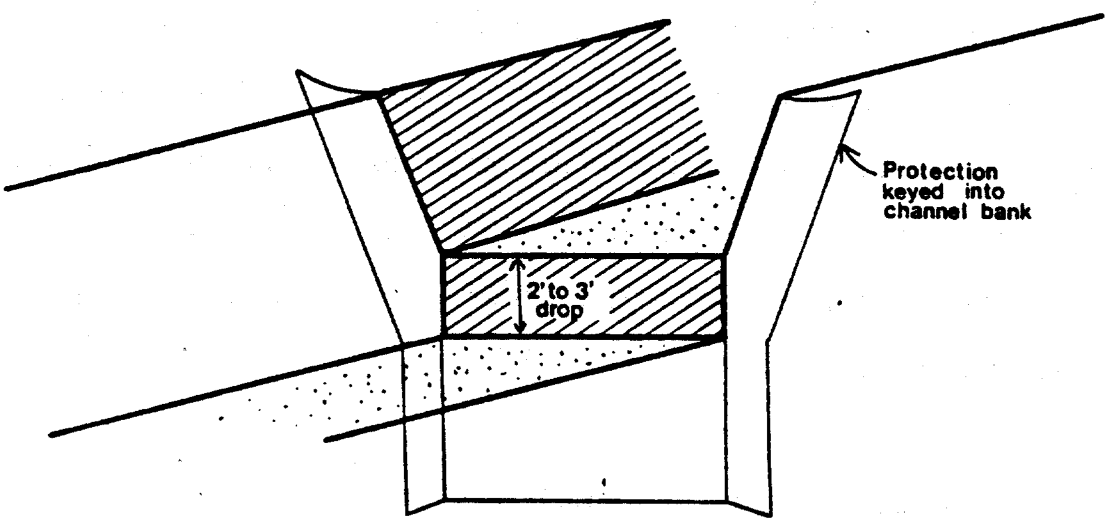
Where two channels join, the tributary flow should be turned so that it merges essentially parallel to the main channel. In any event, the angle of intersection, θ, should not be greater than 12 degress. Under special conditions the angle of intersection may be greater than 12 degrees, but only upon the approval of the Pima County Flood Control District after it has been demonstrated that the tributary and main flow do not influence one another or where other special design measures shall be made to diminish the affect of the juncture on flow conditions.

For minor streams entering a major watercourse, it is preferable that the invert of the tributary be elevated and stabilized 2 to 3 feet above the main channel invert to preclude backwater and sedimentation in the minor stream. Where the main channel is not bank protected, this elevation drop should be located usptream of the confluence at a distance greater than the predicted lateral migration limit of the unlined main channel (See Figure 6-IV).

To design adequate junctions, it should be determined by a hydrologic investigation whether the tributary and the main channel will have hydrographs with times to peak which are sufficiently similar so that both channels may be carrying high flows at the same time. If that is the case, then the channels' juncture should be designed taking into consideration the effect



Plan view of tributary entering major stream channel



Cross-section A-A' stabilized drop upstream
of the predicted lateral migration

Figure 6-IV: Confluence Design Where A Minor Tributary Enters A Major Channel With Unprotected Banks.

of the upstream and downstream flow momentum. A detailed description of the procedure for balancing the upstream and downstream momentum has been included in the appendix accompanying this section.

It should be noted that the design conditions stated for confluences do not apply to small channels which may discharge their flow into a main channel via spillways.

APPENDIX

CHANNEL ALIGNMENT

When channel flow is supercritical, changes in the channel alignment require special design considerations. Among those conditions requiring detailed engineering analysis are: hydraulic jumps; entrance transitions where the flow is supercritical; curvilinear transitions at sudden constrictions, such as, at a bridge, and at the confluence of two tributaries which have hydrographs with similar times to peak.

This appendix has been included in order to present an in-depth discussion and design examples of hydraulic jumps, entrance transitions with rapid flow, existing transitions, and junctures. This appendix contains accepted methods for computing design features; however, there are other methods that may be employed if appropriate applicability can be demonstrated.

HYDRAULIC JUMP

Where supercritical flow enters a hydraulic jump, the kinetic energy term, $v^2/2g$, is the dominant component of the total energy (See Figure 7-IV). The balance of the upstream and downstream hydrodynamic forces and momentum flux, i.e., the specific force, will be equal after the jump. The height of the jump may be determined by using this relationship to set the upstream and downstream specific forces equal to one another.

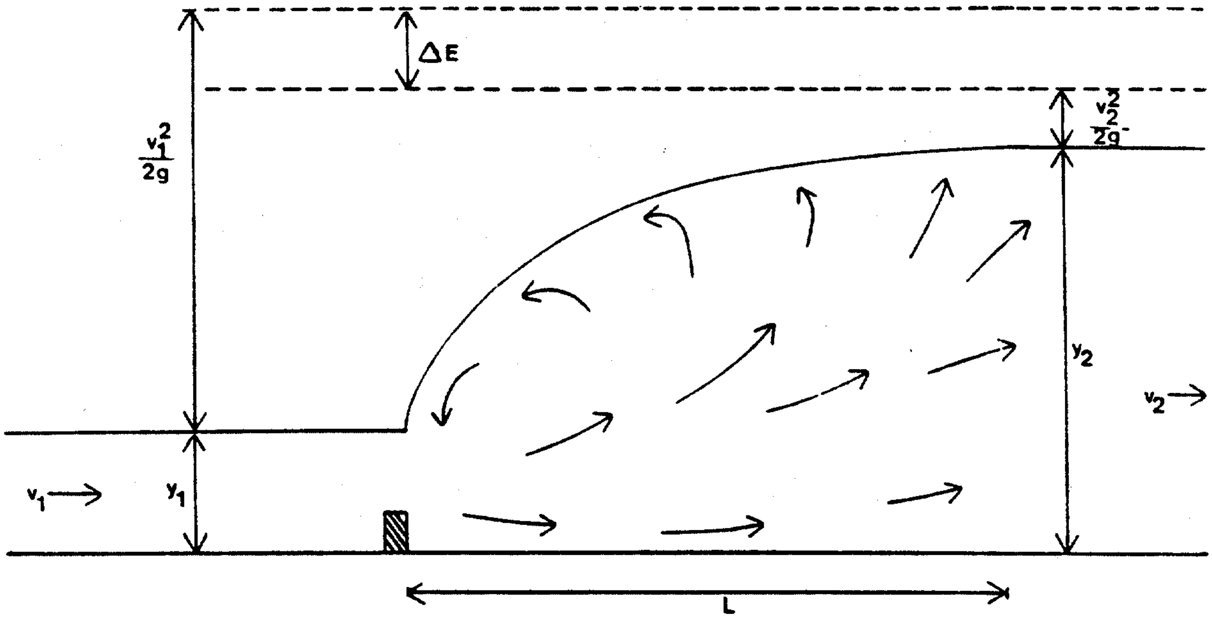


Figure 7-IV: Hydraulic Jump

v_1: Upstream velocity, fps

v_2: Downstream velocity, fps

y_1, y_2: Upstream and downstream sequent depths, ft.

$\frac{v_1^2}{2g}$ = Upstream velocity head

$\frac{v_2^2}{2g}$ = Downstream velocity head

ΔE: Energy lost in the jump

L = Length of jump

2' ⇒ 86.0
1.5' ⇒ 52.6
71.6

A = 46.7 ft²
V = 5.05 T=30

0066

Therefore, $$\frac{Q_1^2}{gA_1} + A_1 \; (ky_1) = \frac{Q_2^2}{gA_2} + A_2 \; (ky_2)$$

Where, Q = discharge, cfs

A = cross-section area, ft.

g = acceleration due to gravity, 32.2 ft/sec.^{2}

Ky = distance to the center of gravity, ft.

y = depth of flow, ft.

K = constant, see table below.

Table 2-IV: Constants to determine the distance to the center of gravity.

Channel Shape	K
rectangular	1/2
triangular	1/3
trapezoidal	$1/6 \; \frac{(2 + b/zy)}{(1 + b/zy)}$ *

*z = side slope ft/ft

The initial upstream condition may be substituted into the specific-force equation II-IV to determine the depth of the flow downstream and the height of the jump, which is the difference between the upstream and downstream flow depths. The upstream and downstream depths of flow are also called sequent depths. The energy loss may be determined using the upstream and downstream depths of flow and the following equation for the specific energy,

$$E = y + \frac{V^2}{2g} \qquad \text{eq. 12-IV}$$

It is easier to solve for both the energy loss and the changes in depth using the graphical method shown in Figure 8-IV. A specific force curve, f, and a specific energy curve, E, can be developed using the design discharge and various flow depths and the corresponding cross-sectional areas. Because the upstream force equals the downstream force after the jump, the points of intersection of a vertical line drawn from the value of the upstream force will locate the upstream and downstream depth of flow. Horizontal lines drawn to the specific energy curve will then yield the upstream and downstream energy.

For a rectangular channel equation 11-IV can be simplified to:

$$Y_2/Y_1 = 1/2 \left[\sqrt{1 + 8F_1^2} - 1\right] \qquad \text{eq. 13-IV}$$

Where F_1 = upstream Froude number

Y_1 = upstream flow depth

Y_2 = downstream flow depth

Figure 9-IV gives a graphical solution for values of F, the Froude number, versus the ratio of Y_2 to Y_1, and Y_2/Y_1.

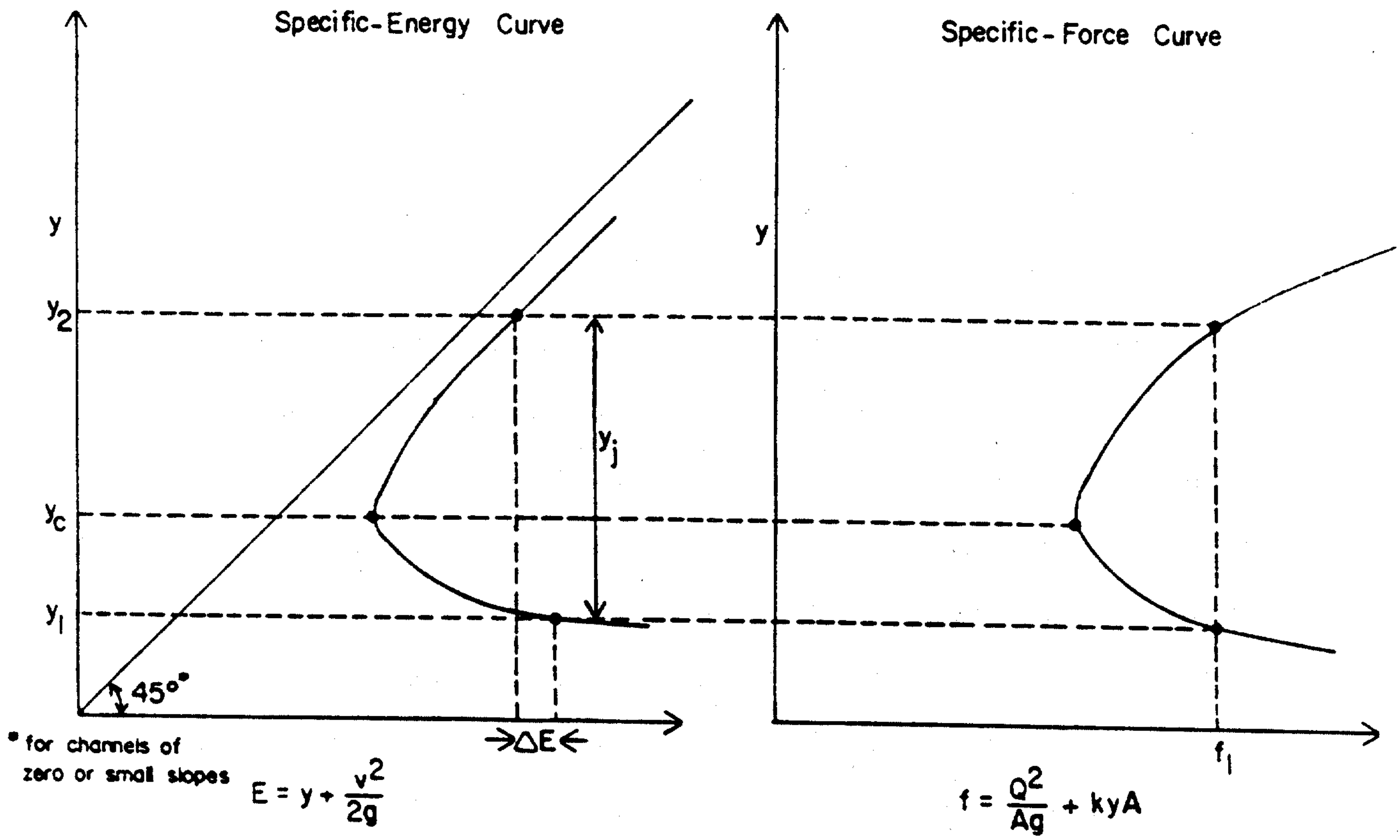


Figure 8-IV : Hydraulic Jump Described With Specific-Energy And Specific-Force Curves.

E = Specific energy, energy per unit weight.

ΔE = Energy loss

f = Specific force, force per unit weight.

Q = Discharge, cfs.

A = Cross-sectional area, ft.

f_1 = Upstream specific force.

y = Depth of flow, ft.

y_1 = Upstream depth, ft.

y_2 = Downstream depth, ft.

y_c = Critical depth, ft.

y_j = Height of jump, ft.

g = Acceleration due to gravity, 32.2 ft/sec.

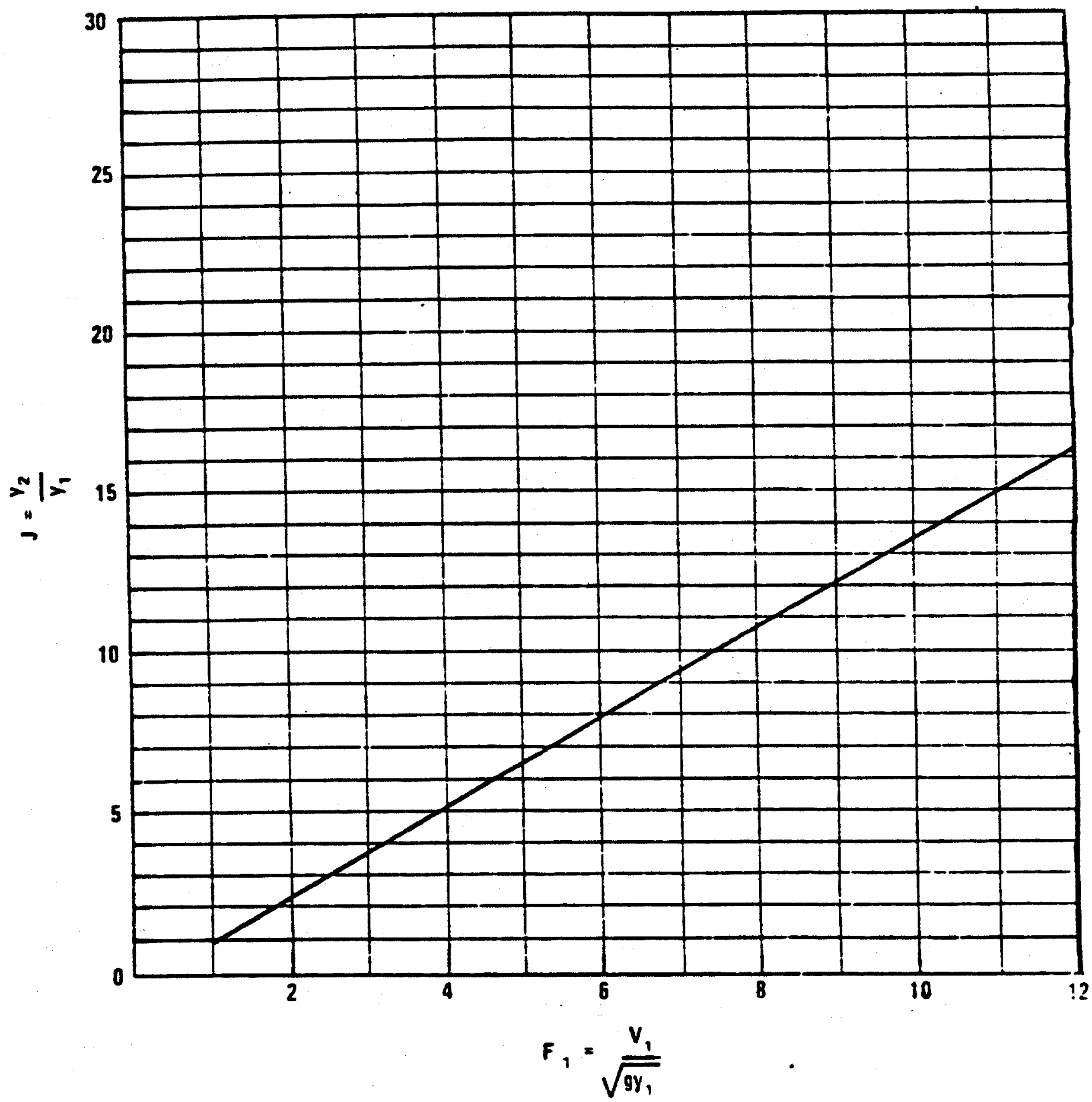


Figure 9-IV: Hydraulic Jump In A Horizontal Rectangular Channel. (H.E.C.-14)

$$\frac{y_2}{y_1} = \frac{1}{2}\left(\sqrt{1+8F_1^2} - 1\right)$$

Where y_2 = Downstream flow depth, ft.
y_1 = Upstream flow depth, ft.
F_1 = Upstream Froude number

0070

JUMP LOCATION

While it is important to know how high the jump will be, the location of the jump should be established so that a channel can be designed which contains the flow along the total channel reach. Hydraulic jumps may be located by using backwater calculation from the downstream control point and step forward calculations from the upstream control point to determine the approximate jump location. By trial and error; the water surface profile must be re-computed until the upstream and downstream flow depths equal those determined by equation 11-IV. It should be recognized by the engineer that determination of channel roughness is not exact, therefore, practical minimum and maximum limits for channel roughness should be used.

The jump length may then be found using Figure 10-IV for rectangular channels or Figure 11-IV for trapezoidal or triangular channels. (These figures were taken from the FHWA Hydraulic Engineering Circular No. 14). Thus the limits of the jump location may be established using the jump length and approximate location derived from the assumed limits for channel roughness. For channels with bed slopes of less than 10 percent, the equations for horizontal channels are still valid and may be used to analyze the hydraulic jump. For a complete discussion of hydraulic jumps in sloping channels, the engineer is referred to Chow's Open Channel Hydraulics and Hydraulic Engineering Circular No. 14: Hydraulic Design of Energy Dissipators for Culverts and Channels.

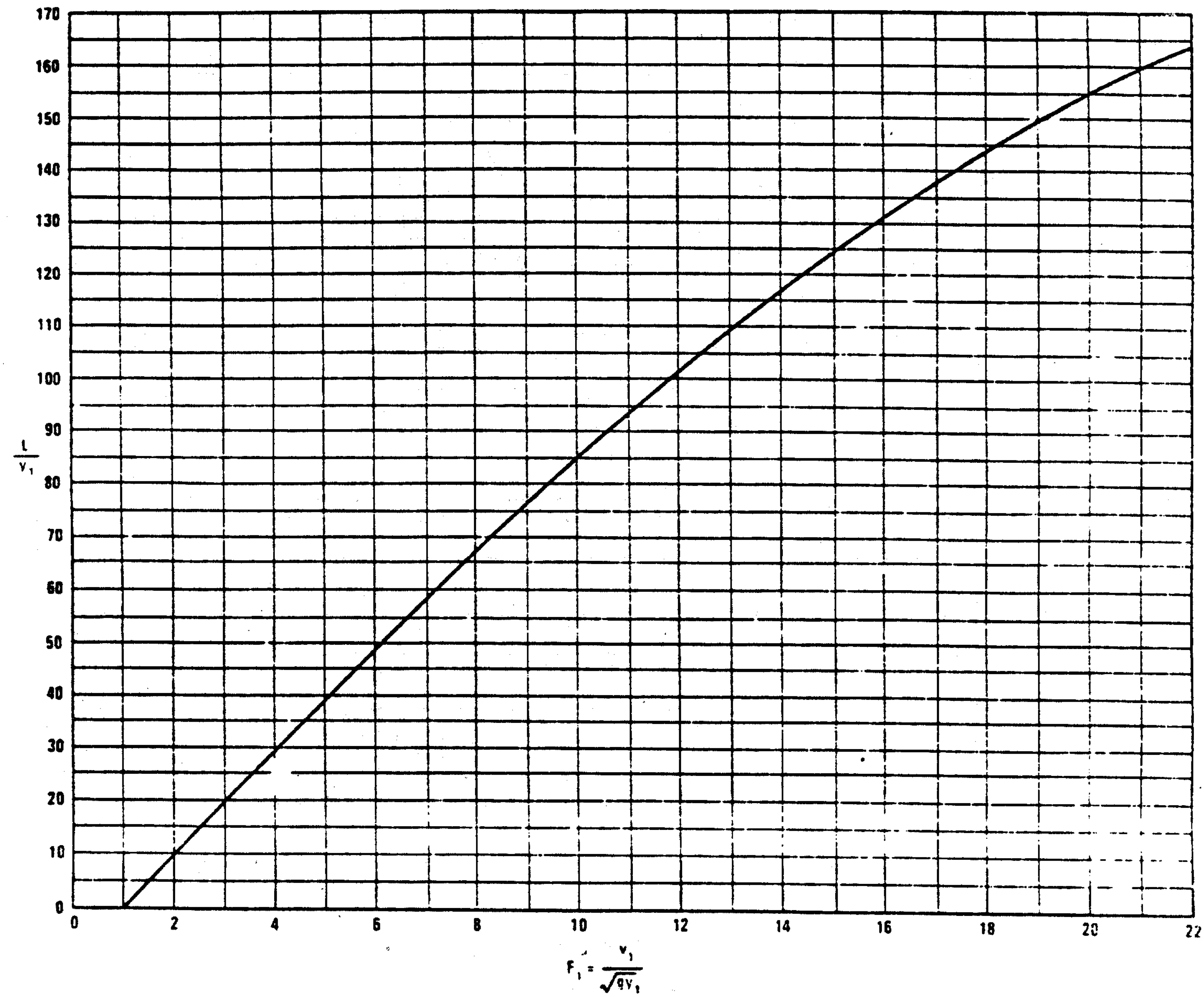


Figure 10- IV : Hydraulic Jump Length For Rectangular Channels [H.E.C. - 14]

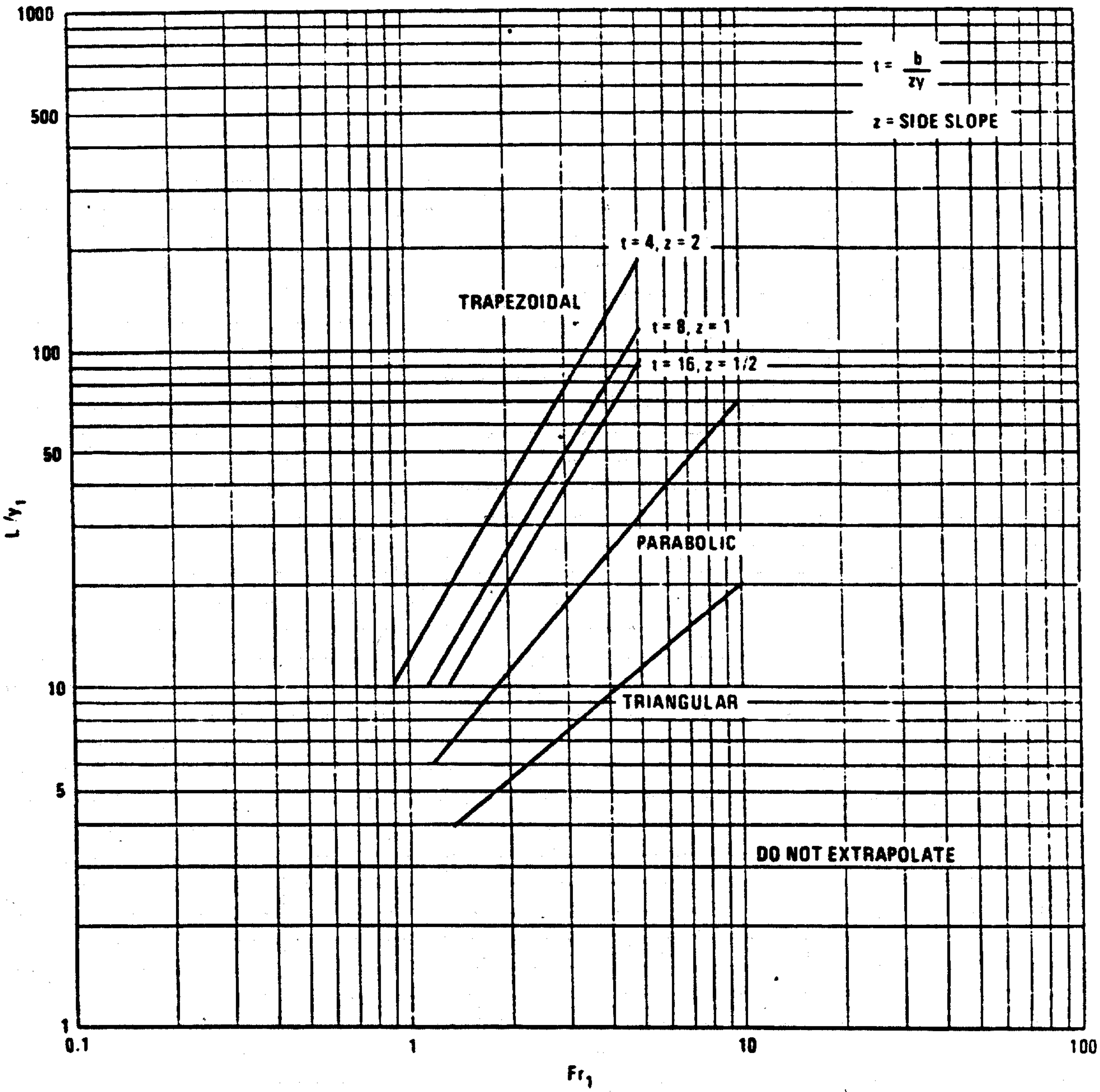


Figure 11-IV: Hydraulic Jump Length For Nonrectangular Channels (H.E.C.-14)

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SPECIAL TRANSITIONS

Special transitions, including curvilinear transitions, are necessary where flow is supercritical or where downstream disturbances must be minimized. While straight line transitions are adequate in most cases, these types of transitions may not apply in all cases. Special transitions should be used to decrease the downstream effects of the positive wave front.

CONTRACTION

Linear

The total length, L, required for a contraction depends on the change in channel width and the wall deflection angle, Θ. Where flow is supercritical and must be transitioned abruptly into a culvert or other contraction, the wall deflection angle should be chosen so that any cross waves that are formed will be cancel- led. In this instance the equation previously given for L,

$$L = \frac{b_1 - b_3}{2\tan\theta} \qquad \text{eq. 14-IV}$$

$$\text{Where } \tan\theta = 1/(3.375F)$$

is still valid; however, Θ is determined such that the following equations are also satisfied so that cross-wave action is minimized.

$$L_1 = \frac{b_1}{2\tan B_1} \qquad \text{eq. 15-IV}$$

$$L_2 = \frac{b_2}{2\tan(B_2-\theta)} \qquad \text{eq. 16-IV}$$

$$L = L_1 + L_2 \qquad \text{eq. 17-IV}$$

See Figure 12-IV for the definition of the variables which are used in equation 14-IV through 17-IV.

With supercritical flow in constructed channels, the following criteria for the design of straight-line transitions are recommended by the Army Corps of Engineers in Hydraulic Design of Flood Control Channels.

Table 3-IV: Recommended Convergence and Divergence Transition Rates

Mean Channel Velocity fps	Wallflare (Horizontal to Longitudinal)
10 - 15	1:10
15 - 30	1:15
30 - 40	1:20

EXPANSION

Expansion of supercritical flow takes place much more gradually than it does at a contraction. For flow which is discharged as a rectangular jet, such as downstream of a bridge, the expansion of the flow can be reasonably described by the following equation:

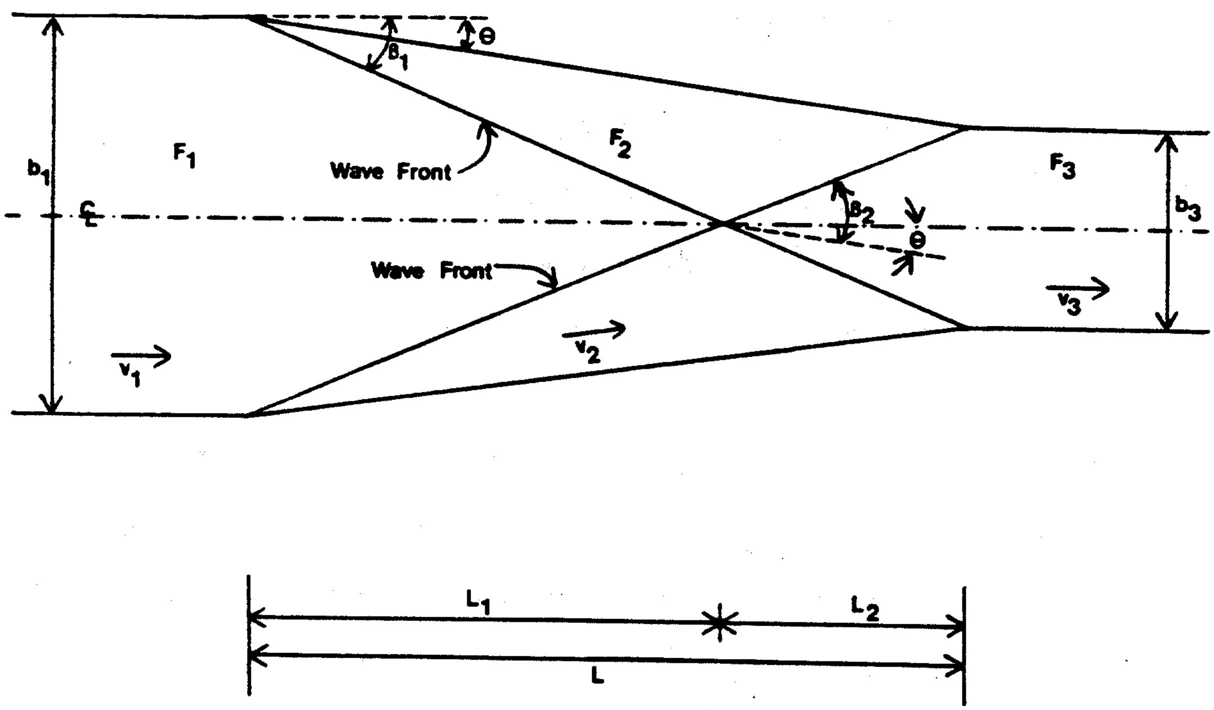

Figure 12-IV: Transition For Channel Contractions In Rapid Flow To Minimize Cross-Wave Action.

$$L = \frac{b_1 - b_3}{2 \tan \Theta}$$

$$L_1 = \frac{b_1}{2 \tan \beta_1} \qquad L_2 = \frac{b_3}{2 \tan (\beta_2 - \Theta)}$$

L = Total transition length, ft.

L_1 = Length to wave front intersection, ft.

L_2 = Length from wave front intersection to end of contraction, ft.

Θ = Wall deflection angle, degrees

β_1, β_2 = Wave front angle from the horizontal, degrees

b_1, b_3 = Channel width, ft.

$$\frac{Z}{b_1} = \frac{1}{2}\left[\frac{x}{b_1 \quad F_1}\right]^{3/2} + \frac{1}{2} \qquad \text{eq. 18-IV}$$

Where, X = distance downstream, ft.

F_1 = upstream Froude number.

b_1 = upstream channel width, ft.

Z = one-half the downstream channel width, $b_2/2$, ft.

Equation 18-IV describes a curvilinear transition which is sufficient to eliminate cross-waves without causing a large change in the depth of flow at a normal cross-section. This equation will, however, result in boundary walls which diverge indefinitely. In order to limit the transition curve so that it gives practical results, but still creates as little downstream disturbance as possible, the generalized transition curve shown on Figure 13-IV may be used to transition expanding flow.

Table 4-IV gives the equations which represent the point of curvature, PC, the point of reverse curvature, PRC, and the point of tangency, PT.

Table 3-IV: Equations Approximating curves for an expanding transition (U.S. Army Corps of Engineers, <u>Hydraulic Design of Flood Control Channels</u>)

POINTS	Z/b_1	$\frac{X}{b_1F_1}$
PC	$\frac{1}{2}$	0
PRC	$\frac{11}{60}\left(\frac{b_2}{b_1}\right) + \frac{19}{60}$	$\left[\frac{22}{15}\left(\frac{b_2}{b_1} - 1\right)\right]^{2/3}$
PT	$\frac{1}{2}\ b_2/b_1$	$\left[\frac{13}{4}\left(\frac{b_2}{b_1}\right)\right] - 9/4$
PC to PRC	$\frac{1}{8}\left(\frac{X}{b_1F_1}\right)^{3/2} + \frac{1}{2}$	0 to $\left(\frac{X}{b_1F_1}\right)_{PRC}$
PRC to PT	$\frac{b_2}{2b_1} - q\left[\left(\frac{X}{b_1F_1}\right) - \frac{X}{b_1F_1}\right]^r$	$\left(\frac{X}{b_1F_1}\right)_{PRC}$ to $\left(\frac{X}{b_1F_1}\right)_{PT}$

Where:

$$q = \frac{\left(\frac{b_2}{2b_1}\right) - \left(\frac{Z}{b_1}\right)_{PRC}}{\left[\left(\frac{X}{b_1F_1}\right)_{PT} - \left(\frac{X}{b_1F_1}\right)_{PRC}\right]^r}$$

$$r = \frac{\left[\left(\frac{X}{b_1F_1}\right)_{PT} - \left(\frac{X}{b_1F_1}\right)_{PRC}\right]\left(3/16\right)\left(\frac{X}{b_1F_1}\right)^{1/2}_{PRC}}{\frac{b_2}{2b_1} - \left(\frac{Z}{b_1}\right)_{PRC}}$$

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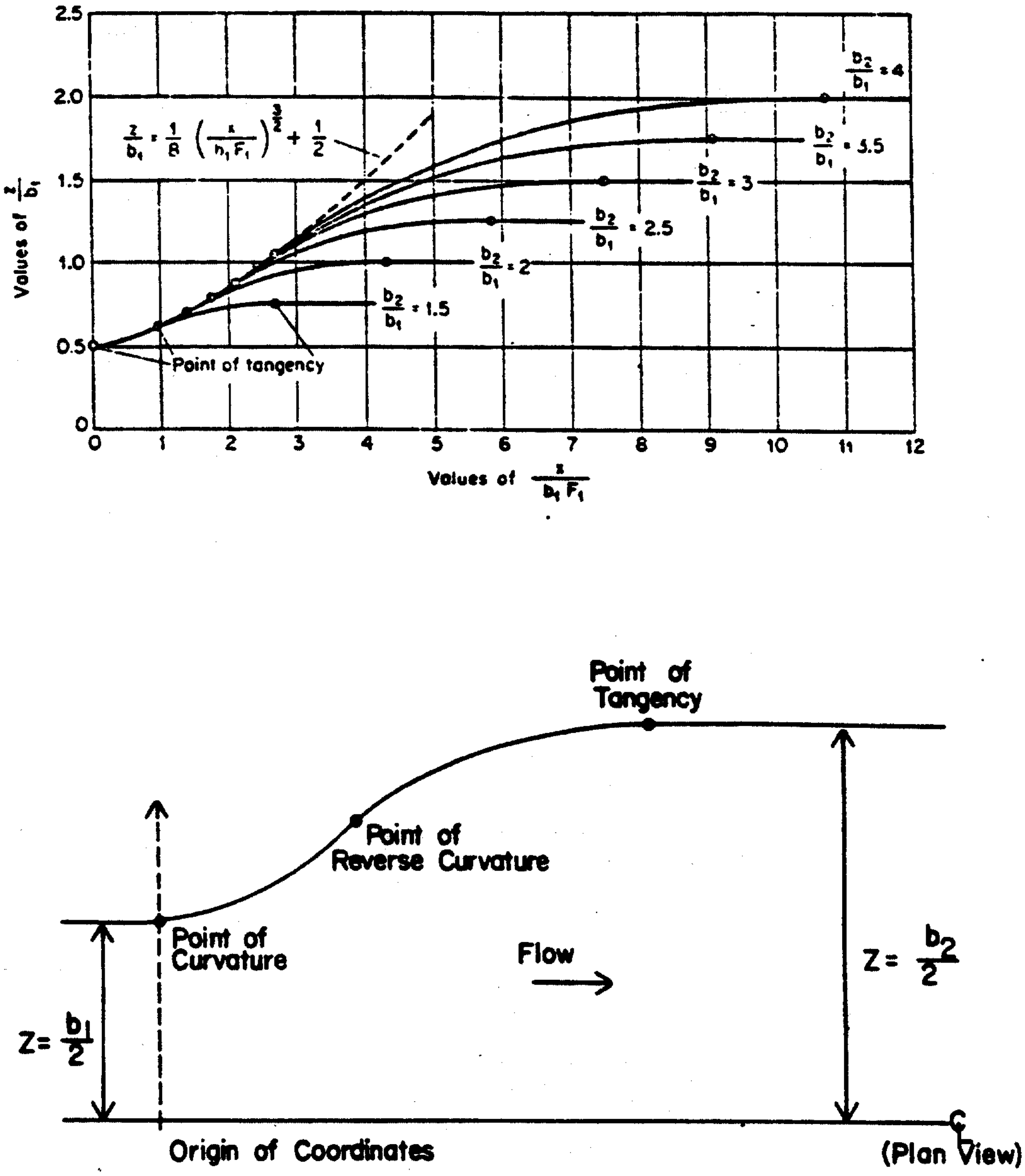

Figure 13-IV : Generalized Boundary Curves For Expanding Transition (Chow, <u>Open Channel Hydraulics</u>)

EXAMPLE

Given a channel where the flow is contracted by a bridge to 100 feet and will expand in the downstream reach to 300 feet, what is the ultimate upstream transition distance X? Compute the transition curve where b_1 = 100 feet, b_2 = 300 feet and F_1 = 1.12.

1. The ratio of b_2/b_1 is 3. The ultimate downstream distance is equal to X at PT.

1. $$\frac{x}{b_1\ f_1} = \frac{13}{4} \left[\frac{b_2}{b_1}\right] - \frac{9}{4}$$

$$X = \left[(200)\ (2)\ \left[13/4\right]\ (3)\right] - 9/4$$

$$X = 834 \text{ feet.}$$

2. The transition curve shown on Figure 14-IV was computed in the following manner.

POINTS	Z/b1	Z	X	X
PC	0.5	50	0	0
PRC	0.867	87	2.048	228
PT	1.5	150	7.5	834
PC to PRC*	0.538	54		50
	0.6065	61		100
	0.8015	80		200
PRC to PT*	1.03	103		300
	1.207	121		400
	1.339	134		500
	1.429	143		600
	1.48	148		700

*Values are chosen for X between the range set by PRC and PT.

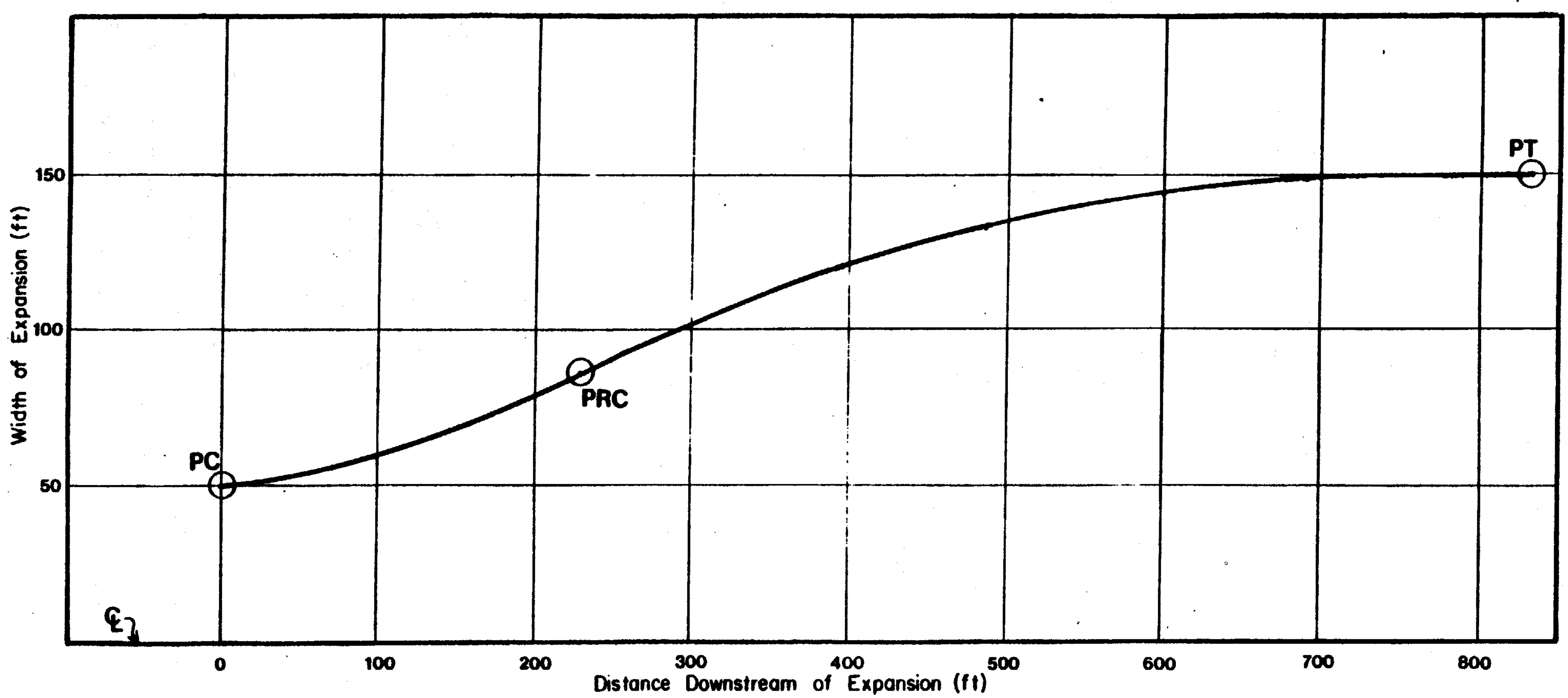

Figure 14-IV: Curvilinear Transition For Example Problem 3.

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CONFLUENCE

At the confluence of two or more channels, the incoming flow from the tributaries and the outgoing flow past the juncture should be balanced to allow for the proper conveyance of the flow. The momentum and continuity equations for the flow in the tributary and main channel must be balanced to insure proper performance of the channels (See Figure 15-IV). The momentum equation for flow in a rectangular channel at a juncture is:

$$\frac{Q_3^2}{gA_3} + \frac{b_3 Y_3^2}{2} = \frac{Q_1^2}{gA_1} + \frac{Q_2^2}{gA_2} \cos \theta + \frac{b_3 Y_1^2}{2} \qquad \text{eq. 19-IV}$$

Where Q_1 = upstream flow in the major tributary, cfs

Q_2 = upstream flow in the minor tributary, cfs

Q_3 = downstream combined flow, cfs

b_1, b_2, b_3 = top widths of the respective flows, ft.

Y_1, Y_2, Y_3 = depth of flow for the respective channels, ft.

θ = angle of the wall deflection

A_1, A_2, A_3 = area of the respective flows in the channels, ft^2.

g = acceleration due to gravity 32.2 ft/sec^2.

For trapezoidal channels, the equation is changed slightly to account for varying channel side slopes and top widths. The equation for a trapezoidal channel is:

$$\frac{Q_3^2}{gA_3} + \left[\frac{b_3}{2} + \frac{ZY_3}{3}\right] Y_3^2 = \frac{Q_1^2}{gA_1} + \frac{Q_2^2}{gA_2} \cos \theta + \left[\frac{b_3}{2} + \frac{ZY_1}{3}\right] Y_1^2 \qquad \text{eq. 20-IV}$$

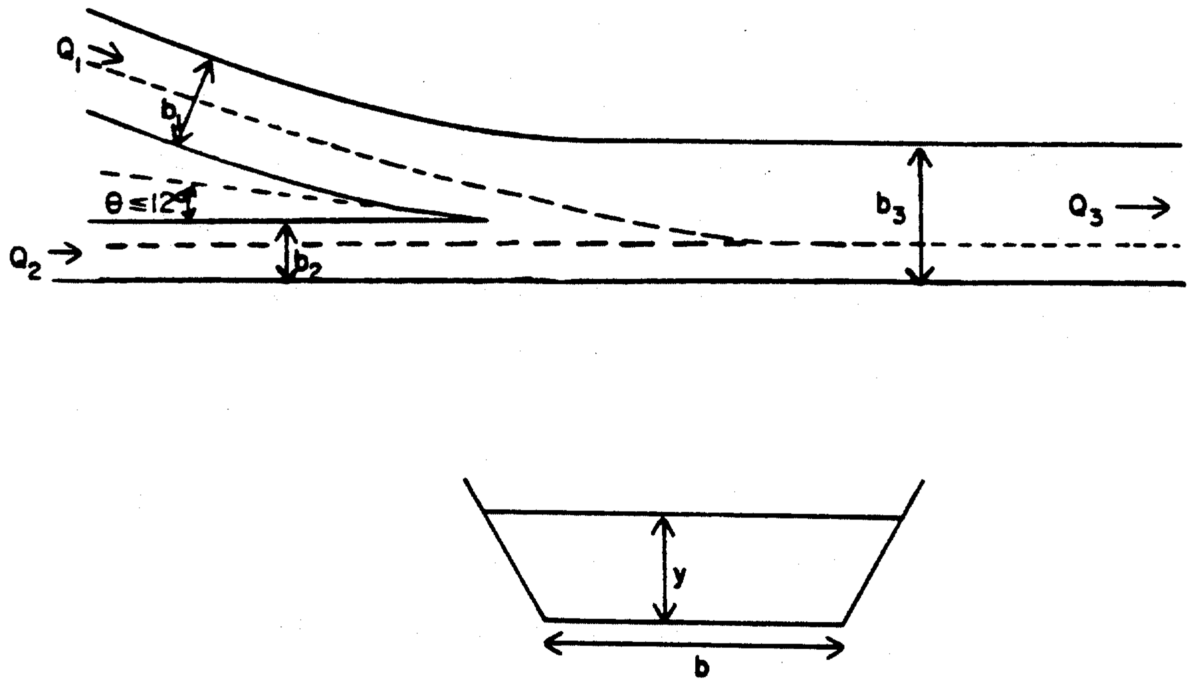


Figure 15-IV: Trapezoidal Channel Junction

Basic Momentum Equation:

$$\frac{Q_3^2}{gA_3} + \left(\frac{b_1}{2} + \frac{z y_3}{2}\right) y_3^2 = \frac{Q_1^2}{gA_1} + \frac{Q_2^2}{gA_2}\cos\theta + \left(\frac{b_1}{2} + \frac{z y_1}{2}\right) y_1^2$$

A = Area, ft^2

y = Flow depth, ft.

b = Bottom width of channel, ft.

Q = Discharge, cfs.

z = Side slope, ft./ft.

θ = Angle of intersection, degrees.

g = Acceleration due to gravity, 32.2 ft/sec.

Where: Z = side slope, horizontal to vertical

b_1, b_2, b_3 = channel bottom widths, ft and all other variables are the same as for equation 19-IV.

The assumptions used with the above equations are:

1. The channel bed slopes are equal with no horizontal drop between the tributaries and main channel.
2. The channel slopes are less than 10 percent. A more detailed analysis is required with steeper bed slopes.
3. The water surface elevations are all equal at the junction. This implies that the depths of flow are the same if assumption number 1 is also met.
4. Flow is uniform at the junction.
5. The channel cross-sections have the same shape.
6. The flows are parallel to the channel walls immediately upstream and downstream of the junction.
7. The angle of the wall deflection, θ, is equal to or less than twelve degrees.

With tranquil flow, all of the above conditions may be met. When the depths of flow in the side and main channels are different, the backwater effect caused by the difference is easily transmitted upstream until the water surface elevations are equal. With supercritical flow, standing waves normally form at the confluence and the backwater can not be transmitted upstream, but in most instances it may be generally assumed that the water surfaces are equal. A hydraulic analysis should be performed in order to determine if a hydraulic jump might occur which would form a standing wave whenever the flow is supercritical.

SECTION V

CHANNEL STABILIZATION

Urbanization in Pima County has resulted in channelization of storm runoff and encroachment into existing flood plains. Where development is either high density residential, industrial, or commercial, the natural watershed and drainage patterns have been radically altered. The general topography for Pima County is relatively steep, especially in the Tucson area. Therefore, when storm runoff, which generally flows in braided streams or as dispersed sheet flow, is concentrated in channels the resulting velocity increase can cause severe erosional problems. In an effort to reduce property damages, some type of bank protection is often required to stabilize man-made channels. Criteria for protection have been developed for urban areas. These methods have been applied in Pima County with modifications being made over time to adjust the criteria to the conditions existing in Pima County. To clarify drainage design standards, the following discussion will outline how the need for channel protection should be determined, what the exceptions or special cases are, and what areas will require further study before an adequate standard can be applied. An appendix has also been included to provide the engineer with technical references and information regarding the design requirements.

This section was written to delineate areas which have significant erosional hazards requiring some form of bank protection and to state the policy of Pima County for reviewing channel designs. The information presented is only meant to be used as a guideline and it should be remembered that not all channels will require stabilization. For design and construction of bank protection, channels and encroachments, it will be necessary for a hydrologic and hydraulic analysis to be performed by a registered professional civil engineer and accepted by the Pima County Engineer.

DETERMINATION OF CHANNEL PROTECTION REQUIREMENTS

In order to define the design requirements for channel stabilization, a complete hydrologic and hydraulic report should be prepared by a registered professional civil engineer and submitted to the Pima County Flood Control District for approval. The following standards are normally applied in the review process for channel designs and should be utilized by the engineer in designing drainageways.

Where stream channels are left in their natural state, bank stabilization in most cases is not required. However, once a stream's natural flow characteristics are altered, a determination must be made as to whether bank protection will be necessary to maintain the channel and prevent excessive erosion. The cases where earthen channels are not appropriate and bank protection shall be required are:

1. When channel side slopes are steeper than 3:1 in non-cohesive soils unless approved soil analysis demonstrates steeper side slopes are stable; or,

2. When channels are constructed in fill slopes; or

3. When the constructed channel velocities for the 10-year flood exceed the force required to erode the particular soil type of the area. Standard charts, by the Army Corps of Engineers, Soil Conservation Service and others, which show the grain size versus the basic velocity at which the particle is moved, should be employed unless natural vegetation is sufficient or is added to maintain the channel configuration. Included in the Appendix are charts to determine allowable velocities. Other factors such as the channel alignment, bank-slope and depth of flow are to be used to modify the basic velocities and determine the allowable velocities for unprotected earthen channels.

For isolated building pads where encroachments into an existing 100-year flood plain utilizing fill are involved, bank protection should be provided. However, as long as it is determined that the stream channel will not meander, encroachments into overbank flow areas or in sheet flow areas do not require protection if it is demonstrated by an engineering analysis, which is accepted by the Pima County Flood Control District, that the depth and velocity of the 100-year flow will not cause significant erosion.

It should be recognized that the items listed above are not necessarily independent of each other when making an evaluation as to the need for stabilization.

ACCEPTABLE STABILIZATION METHODS

The type of stabilization used for a particular channel will depend on many factors including flow conditions such as velocities, depth of flow, soil type, economic factors, availability of materials, and compatibility with existing stabilization measures. A variety of protection methods are acceptable, including the use of vegetation cover, when designing channels which are acceptable for particular stream classification or land use. The types of bank protection and/or channel lining that have been found to be acceptable to Pima County are:

1. Soil cement used to either completely line a channel or to armor the channel sides. In addition, it may be used to protect fill areas which encroach into a floodplain or areas where sheet flow is being transitioned into a channel by a dike.

The degree of protection can be varied by changing the side slopes from 1:1 with horizontal lifts (normally 8-foot-wide lifts, which provides the maximum wall thickness), to 3:1 slope paving for areas where less protection is required, such as encroachment into shallow flooding zones where slope paving with 8 inch lifts will be used. However, if the appropriate type of soil is not available on site, then the cost of importing acceptable material may make this method economically infeasible.

2. Asphalt paving is acceptable only for small drainage easements, less than 15 feet wide, spillways, curbcuts, or other flowage ways, between single lots to carry flow to or from

streets. In Pima County's hot arid climate, asphalt will deteriorate in time and provides only a limited amount of protection. Large volumes of flow and high velocities can erode the asphalt lining causing damage and maintenance problems.

3. Gunite or concrete fully-lined channels should be used where the flow velocities are very high, channel gradients are steep and the soil is non-cohesive. While the cost of this type of bank protection is high, it uses the smallest amount of land and has very low maintenance requirements. Using either this type of protection or soil cement to completely line a channel is the best means of maintaining a fixed bottom slope where the natural gradients are steep and/or degradation of the channel is expected.

4. Gunite or concrete lined sides with natural bottoms are acceptable where an engineering analysis shows that the flow velocities would erode the banks and where channels are built in fill material. If degradation of the channel bottom is expected because of a mildly steep gradient, then grade control structures should be used in conjunction with the lined sides to stabilize the channel bottom. On steeper slopes more grade check structures are required making it less economical than totally lining the channel. This type of protection is usually cheaper than fully lining a channel and it will also keep channel velocities lower than in fully-lined channels so that the downstream erosional problems will be reduced. See page 25 and 29 of the Appendix for a description of the method for determining stable channel bottom slopes and cut-off wall design and placement.

5. Riprap protection includes dumped rock, rock filled gabions and grouted riprap. Normally, riprap is used to protect unstable banks but it can also be used to reduce the flow velocity in channels or at culvert outlets. A good source of rock for riprap does not always exist. Therefore, when adequate rock cannot be obtained, the available rock must be either grouted or enclosed in a gabion; or another form of bank protection must be used.

6. Any other method approved by the Pima County Engineer. Any of the above mentioned forms of bank protection require that they are adequately keyed into the banks at the upstream and downstream ends and that the flow is smoothly transitioned into the channel to prevent a backwater effect and then released at the downstream end in a manner which does not harm the downstream owners. If the method utilized for bank protection is one which leaves the channel bottom unprotected, then the bank protection must be toed down below the channel bottom past the depth for general scour (See Appendix.) In all cases, bank protection is to be provided for the design peak discharge, or to the top of bank where side inflow occurs. (See Section III for freeboard requirements).

SPECIAL CASES REQUIRING STABILIZATION

It is often difficult to set one standard which applies to all cases. There are occasions where channels are constructed in such a manner as to cause special hydraulic problems. The following is a list of cases where additional engineering analysis will be required in order to decide if bank stabilization is to be required.

1. The proximity of a building to a natural channel which has a high erosion potential. Natural channel banks may have a higher erosion potential if they lack vegetation, have side slopes at or steeper than 1:1 and are composed of loose unconsolidated material. Natural channels usually have unstable side slopes where degradation of the channel bottom causes sloughing of the sides or sloughing at natural bends along the streams course. When bank protection is not provided for a major watercourse with straight alignments and where no unusual hydraulic conditions exist, a setback of 300 feet for buildings is required, unless demonstrated to be stable. However, a setback has not been determined for minor channels. Refer to Section III for building setback requirement details.

2. Culvert outlets may require energy dissipators or training dikes to spread and direct the flow. If the outlet velocity is less than 1.5 times the natural stream velocity, then protection is not required. When outlet velocities are greater than 1.5 times but less than 2.5 times the natural velocities, dumped riprap, splash pads, etc., should be used, unless the outlet velocity is also greater than 10 fps, then wire tied or fully grouted riprap is required. In rocky soil, a cutoff wall may be substituted for the dumped riprap in order to allow a natural rock basin to form without endangering the structure.

Whenever the outlet flow velocities exceed 2.5 times the natural flow velocities some type of energy dissipator is needed (See Chapter VI, Culvert Outlet Protection). These standards were developed by Arizona Department of Transportation for drainage basin in excess of 64 acres.

3. When training dikes are used to transition sheet flow into a channel, or for the expansion of flow from a man-made channel, back to natural conditions.

4. At the junction of two channels. In addition, the flows should be turned so that they are, if possible, approximately parallel at the confluence, but at no time should their angle of confluence be greater than 12 degrees. Small local drainage inlets may be constructed at right angles to the main channel, but in all cases adequate stabilization of the spillway is required (See Chapter IV, Channel Alignment).

5. At bends in natural or man-made channels, erosional hazards may exist because of the increased shear stresses and tractive forces on the channel sides and bottom and bank protection may be required for that reach of the channel. However, when the centerline radius of curvature is a minimum of 10 times the channel top width, then the bend can be ignored and the channel treated as if it has a straight alignment at that location. Therefore, if it is determined that the straight portion of the channel is stable, no bank protection shall be required. When the channel does require bank protection, the degree of protection at a curve would not have to be increased beyond what is required for a straight reach, if the ratio of 10:1 is observed.

APPENDIX

STABLE CHANNEL DESIGN

To assist the engineer in applying the design criteria for channel stabilization set forth in Section V, a list of suitable references and technical information is being provided.

REFERENCES:

The following list contains the acceptable references for use in designing stable channels and for calculating sediment transport and scour.

1. Bank and Shore Protection in California Highway Practice, California Department of Public Works, November, 1979.

2. Brater, E.F. and H.W. King, Handbook of Hydraulics, 6th ed., McGraw-Hill, New York, 1976.

3. Chow, V.T., Open Channel Hydraulics, McGraw-Hill, 1959.

4. Chow, V.T. (ed), Handbook of Applied Hydrology, McGraw-Hill, New York, 1964.

5. Design of Small Dams, 2d, ed., U.S. Bureau of Reclamation, 1974.

6. Design of Stable Channels with Flexible Linings, Hydraulic Engineering Circular No. 15, Federal Highway Administration U.S.D.O.T.

7. Hydraulic Design of Energy Dissipators for Culverts, Hydraulic Circular No. 14, Federal Highway Administration, U.S.D.O.T.

8. Henderson, F.M. Open Channel Flow, MacMillan Publishing, New York, 1966.

9. Morris, H.M. and J.M. Wiggert, Applied Hydraulics in Engineering, John Wiley and Sons, New York, 2d, ed, 1972.

10. Richardson, E.V., D.B. Simons, S, Karaki, K. Mahmood and M.A. Stevens, Highways in the River Environment and Environmental Design Considerations, Training and Design Manual, Federal Highway Administration, U.S.D.O.T., May, 1975.

11. Rouse, H. (ed), Engineering Hydraulics, John Wiley and Sons, New York, 1950.

When submitting designs to Pima County for approval, the engineer should site the references and pages used on all design calculation sheets. Table 1 below is being furnished to facilitate the use of the references presented.

Table 1-V

Important Topics Concerning Stable Channel Designs

TOPIC	REFERENCES
General Information	3,4,8,11
Open-Channel Design	3,6,8,9,11
Scour	7,10,11
Energy Dissipators	7,11
Bank Protection	1,10
Channel Transitions and Curves	7,9,11
Sediment Transport	8,9,10,11

TECHNICAL INFORMATION

The equations and technical information that may be employed in Pima County in designing channels with stable slopes and banks are given on the following pages for minor watercourses (less than 5,000 cfs). These have been compiled for use because the necessary information is not readily available. Other methods for determination of stable channels may be used with approval by Pima County. Where possible, references for the material presented shall be noted and an example given. If information regarding the derivation of a formula is desired, it can be obtained from the Pima County Flood Control District.

ALLOWABLE VELOCITY FOR EARTHEN CHANNELS

The allowable velocity for an earthen channel is a function of the channel alignment, channel side slopes, depth of flow, bed particle grain size and water quality. The following charts and standards have been developed by the Soil Conservation Service (SCS) for use with a design flow equal to the 10-year peak discharge. In determining the basic velocity at which sediment movement and erosion begins, a discrete particle size of D_{75} is employed by SCS as the representative grain diameter.

Procedure:

To determine what velocities will cause a given channel to scour, the basic velocity, V_b, is determined using the D_{75} particle size, and the chart shown in Figure 1a-V. In the absence of sieve analysis results for D_{75} the engineer shall use D_{75} equal to 4mm. This value is an average value for D_{75} taken from various sieve analyses in Pima County. The basic velocity is then adjusted using correction factors for channel alignment (A), bank slope (B) and depth of flow (D). These correction factors are shown on Figure 1b-V, Figure 1c-V, and Figure 1d-V respectively. (Correction factor B should only be used where the soil is in discrete particles as it would be in fill, i.e., unconsolidated material).

For a given channel using the design flow parameters, the allowable velocity, V_a, in an unprotected earthen channel can be determined using equation 1-V:

$$V_a = V_b \times D \times A \times B; \quad \text{for } Q_{10} \qquad \text{eq. 1a-V}$$

or

$$V_a = 1.6 \times V_b \times D \times A \times B; \quad \text{for } Q_{100} \qquad \text{eq. 1b-V}$$

Example (1): What is the allowable flow velocity for a channel carrying clear water that is designed for the Q_{10}?

Given: 1) Alignment is straight

2) Depth of flow = 3 feet

3) Side Slopes = 3:1

4) D_{75} = 4 mm; assumed because soil data was not available.

Then: V_b = 2.5 fps

A = 1.0

B = 0.86

D = 1.01

V_a = 2.5 x 1 x 0.86 x 1.01

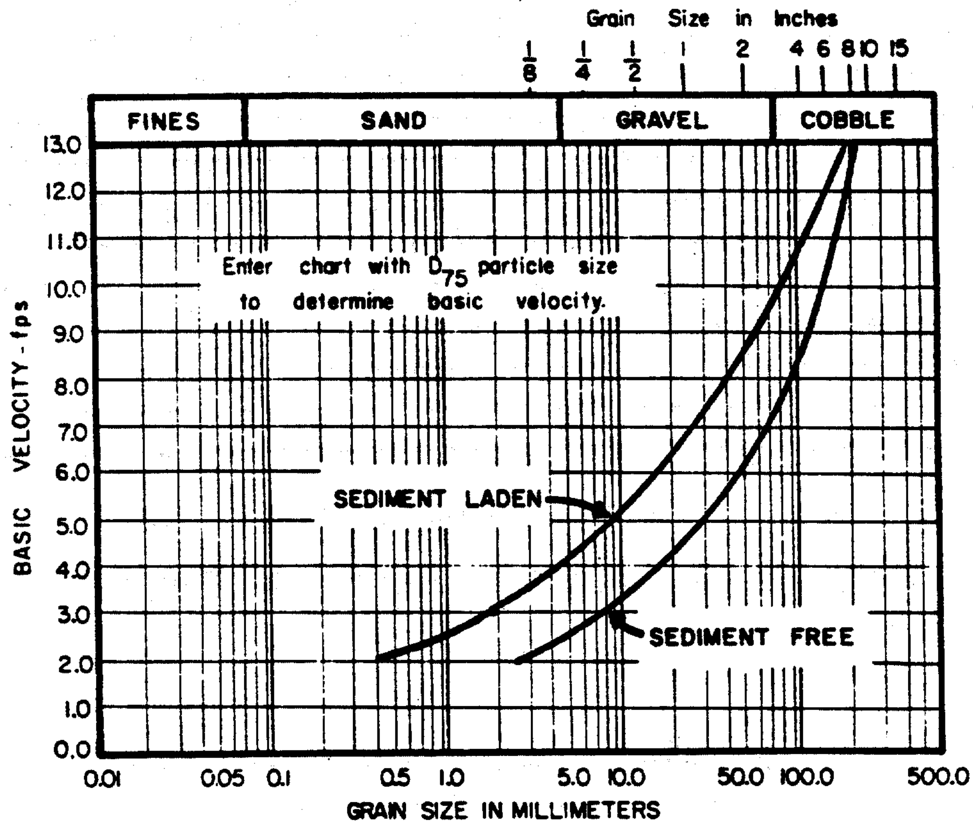
V_a = 2.17 fps

If the flow were sediment laden then,

V_b = 4.0 fps

and,

ALLOWABLE VELOCITIES FOR UNPROTECTED EARTH CHANNELS



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

Figure Ia-V: Basic Velocity Chart

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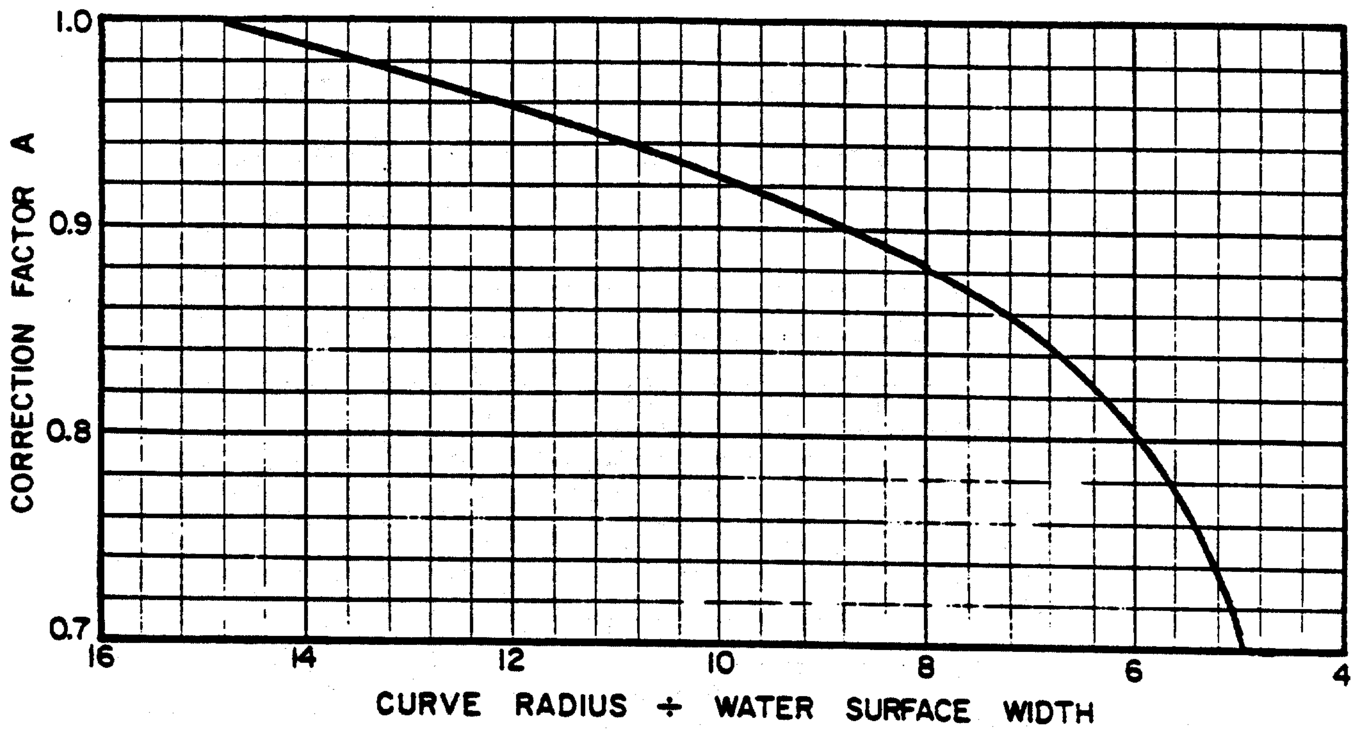

Figure Ib-V: Correction Factor A For Channel Alignment

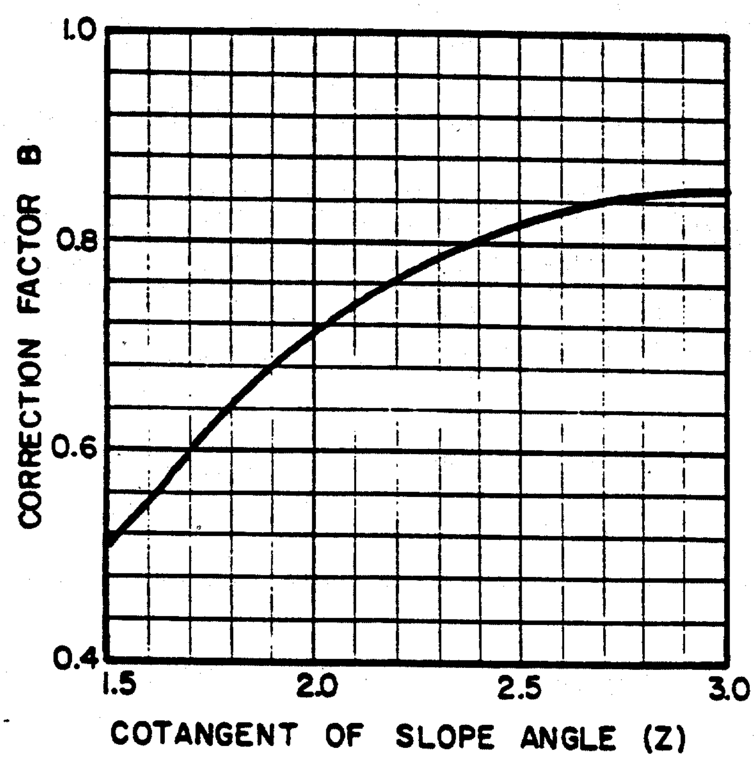

Figure Ic-V: Correction Factor B For Bank Slope

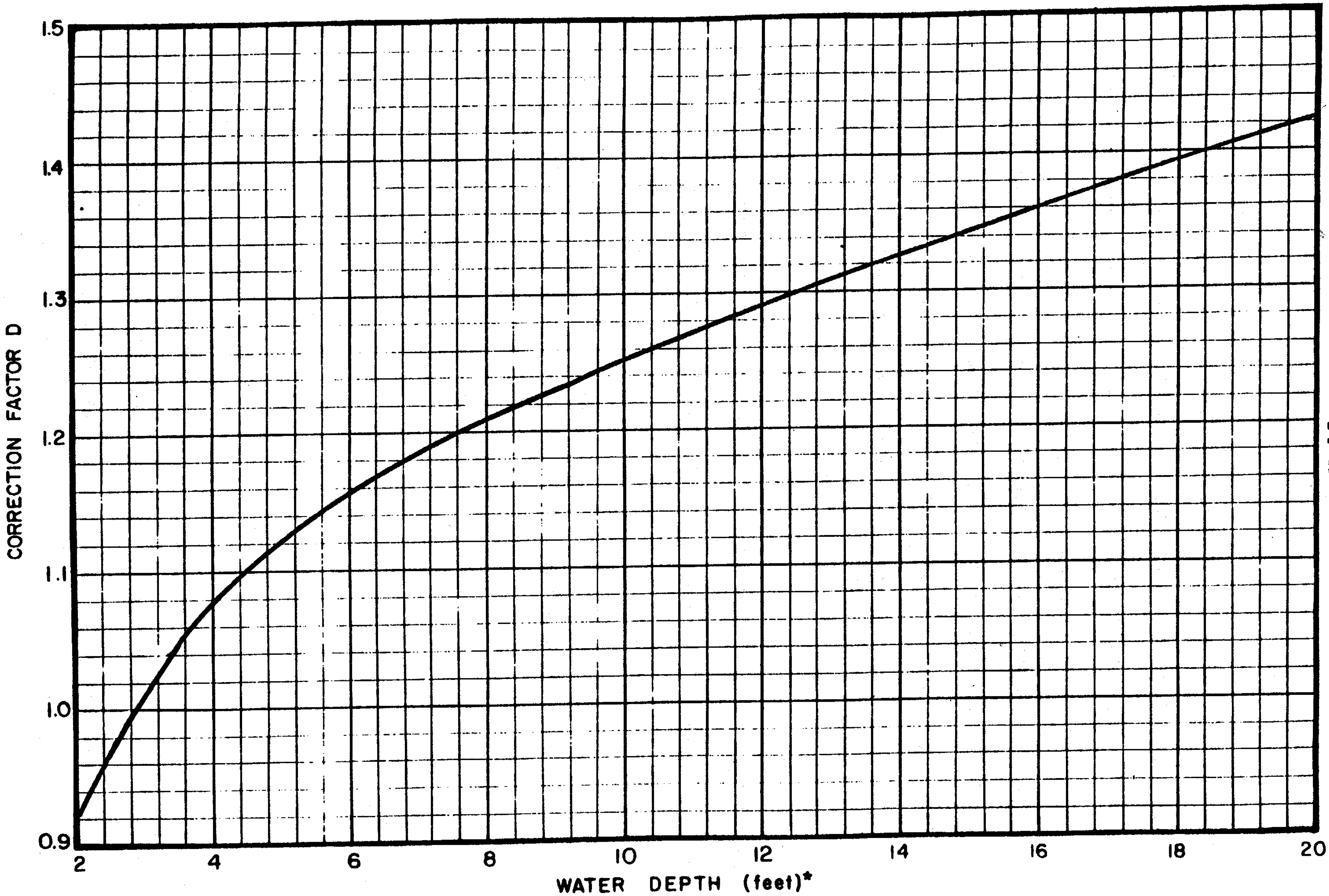

Figure Id-V: Correction Factor D For Depth Of Design Flow

*where the depth of flow is less than 2 feet D = 0.93

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$$V_a = 4.0 \times 1 \times 0.86 \times 1.01$$

$$V_a = 3.48 \text{ fps.}$$

For a quick check to determine the allowable velocities for Q_{10} in channels of non-cohesive coils, such as those found in alluvial channels in Pima County, the following formulas may also be employed.

$$V_a = 2\,Y^{0.5}; \quad Y \quad 2 \text{ ft.} \qquad \text{eq. 2a-V}$$

$$V_a = 2.5Y^{0.2}, \quad Y \quad 2 \text{ ft.} \qquad \text{eq. 2b-V}$$

Where: V_a = allowable velocity, fps

Y = depth of flow, ft

Therefore, if $Y = 3$ feet as in example (1):

$$V_a = 2\,(3)^{0.5}$$

$$V_a = 3.46 \text{ fps}$$

Where channel velocities are expected to exceed the allowable velocity, appropriate measures must be taken to stabilize the channel.

EQUATIONS FOR STABLE SLOPES AND CHANNEL DESIGN

Degradation and aggradation of channel beds are a major problem in Pima County. Where stream velocities are too great, as determined by the allowable velocity (see pages V-12 - V-17), erosion will cause the channel to degrade until, with time, the bed slope is reduced to a stable or "equilibrium" slope. If, however, the channel slopes are too flat to carry the sediment load, deposition will occur and thereby reduce the channel's capacity. It is, therefore, essential that channels are constructed at slopes which will adequately carry the sediment load without causing erosion. The following equations may be utilized to estimate values for the stable channel slope and the necessary channel geometry to prevent stream bed degradation. Where factors limit the design slope to one which is greater than the equilibrium slope, the channel shall be completely lined; or, grade control structures shall be used in conjunction with stabilized banks. These equations are for the design of channels for minor watercourses.

Clear Water Equation:

The following equations should be utilized only in urban areas which generate essentially clear runoff, i.e., land use densities which exceeds 5 residences per acre and where the upstream channel sides and bottom are protected against erosion. These equations do not apply if channels are unlined or are natural.

$$S_c = \left[\frac{1.45\ n}{q^{0.11}}\right]^2 \qquad \text{eq. 3-V}$$

$$V = 2Y^{0.5} \qquad \text{eq. 4-V}$$

$$V = (4q)^{0.33} \qquad \text{eq. 5-V}$$

$$Y = (0.5q)^{0.67} \qquad \text{eq. 6-V}$$

$$q = 2Y^{1.5} \qquad \text{eq. 7-V}$$

Where: q = discharge per unit width, cfs/ft

V = velocity, ft/sec

n = Manning's roughness coefficient

Y = Hydraulic depth, ft

S_c = Channel bed slope, ft/ft

For information concerning the above equations, contact the Pima County Flood Control District.

Procedure:

Because it is the mean annual flood which primarily shapes a channel, the above equations should be utilized with the flow parameters for the <u>2-year peak discharge</u>. Estimates of the 2-year peak discharge can be determined using the approximate ratios for lesser flood magnitude to the 100-year flood given on page 135 of "Hydrology Manual for Engineering Design and Flood Plain Management within Pima County, Arizona."

The engineer should first establish that the velocities exceed the alluvial flow velocities. Equation 4-V is equation 2a-V previously given in the section on allowable flow velocity. This equation can be used with equations 5-V, 6-V and 7-V to calculate the design parameters necessary for a non-scouring channel. The stable design slope can then be arrived at using equation 3-V and the discharge per unit width. Where channel geometry and slope are fixed by other constraints, the stable slope given by the equation will allow the engineer to determine if the design slope is stable or whether the channel must be lined or grade controls provided. Where grade controls are used the stable slope should also be used to determine the necessary number of cutoff walls and their locations.

Example (2): What is the stable channel slope for a channel being designed in an urbanized area where the upstream channel is also completely lined?

Given: 1) Q_{100} = 450 cfs

Q_2 = 90 cfs

Channel slope = 1%

n = 0.021 for concrete sides with a natural bottom

Sideslopes = 1:1

Bottom Width = 20 ft

Using Manning's equation for uniform flow yields;

Y = 0.75 ft.	V = 5.6 fps
W = 21.5 ft.	Q = 90 cfs
A = 16 ft.^{2}	q = 4.3 cfs/ft

Then for a stable slope;

$$S_c = \left[\frac{1.45\ (0.021)}{(4.3)^{0.11}}\right]^2$$

S_c = 0.0007

Since S_{design} = 0.01 S_{stable} = .0007, the channel must be completely lined or designed with lined sides and grade check structures. Or, the channel could be redesigned so that the discharge per unit width is equal to the value determined by equation 7-V.

Stable Slope Equation Considering Sediment Transport

The equilibrium slope equation presented below attempts to balance the sediment discharge between the upstream and downstream channel reaches. The calculated slope will not necessarily define the stable slope, but will only guarantee that the incoming sediment supply equals the outgoing supply. For this reason a comparison should be made of existing stable channel designs within the area. If the equation is applied in an area where existing comparable channels are essentially stable, then the slope which is calculated by the formula below should be similar in order to maintain the same equilibrium which previously existed.

Given:

$$S_u = \left[\frac{N_u}{N_n}\right]^2 \left[\frac{Q_{wu}}{Q_{wn}}\right]^{-1.4} \left[\frac{T_u}{T_n}\right]^{0.5} (1-R)^{0.9}\, S_n \qquad \text{eq. 8-V}$$

Where: S_u = Stable equilibrium slope after urbanization, ft/ft

S_n = natural slope or existing stable slope, ft/ft

N_u = Manning's roughness factor for urbanized conditions.

N_n = Manning's roughness factor for natural conditions.

Q_{wu} = 2-year peak discharge after urbanization, cfs.

Q_{wn} = 2-year peak discharge before urbanization, cfs.

T_u = 2-year top width, urbanized, ft.

T_n = 2-year top width, natural, ft.

R = reduction factor for sediment supply.

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The above equation was developed by equating the sediment transporting capacity in the upstream reach to the sediment discharge capacity under future urbanized conditions.

The engineer must also determine if the channel velocities exceed the allowable erosion velocity, page V-11, and if comparable channels have similar existing slopes. Because urbanization will decrease the available sediment, a sediment reduction factor, R, which varies as a decimal fraction from 0 to 1, is applied to the equation. The reduction factor corresponds approximately to the change in the upstream channel and basin conditions. <u>This factor should only be employed where the percentage of imprevious cover in the upstream basin is high (greater than 50%), or where dams, gravel pits, etc. would reduce the downstream sediment supply in the channel</u>.

The equation remains valid and would be useful in the following types of problems:

1. Where a portion of a drainage basin is to be urbanized even if no channelization occurs.

2. Where the upstream area is no longer natural, but may still be considered to be in equilibrium or stable.

3. Where both the upstream and downstream areas have the same basin cover, but a change in the channel bed slope, either natural or manmade, could make the channel unstable.

4. When encroachment will take place which will change the channel top width.

5. To determine if siltation will take place.

Example (3): It has been proposed that a stream be channelized from the river to an existing channelized section upstream. The upstream channelized reach is stable and has not altered significantly over a period of many years. While the proposed section will have a 2% slope and a narrow channel, the existing upstream section has a 1% slope and a less confined channel cross-section. Additionally, the entire catchment area containing both portions of the channel has been developed for one residence per acre (20% impervious) with earthen channels. Using Figure 2-V and the information listed below, the equilibrium slope is calculated as follows:

	UPSTREAM CHANNEL	DOWNSTREAM CHANNEL
Mean annual flood, Q_2	500 cfs	500 cfs
Channel Shape	trapezoidal	trapezoidal
Channel Slope	0.01	0.02
Channel resistance (n)	0.025	0.025
Channel base width	20 ft	10 ft
Side Slopes	2:1	2:1

Using Mannings equations for uniform flow:

	Y,ft	T,ft	A,ft^2	V,fps	Q,cfs
Upstream	2.3	29.08	55.7	9.1	500
Downstream	2.6	20.44	39.7	12.6	500

Then using equation 8-V:

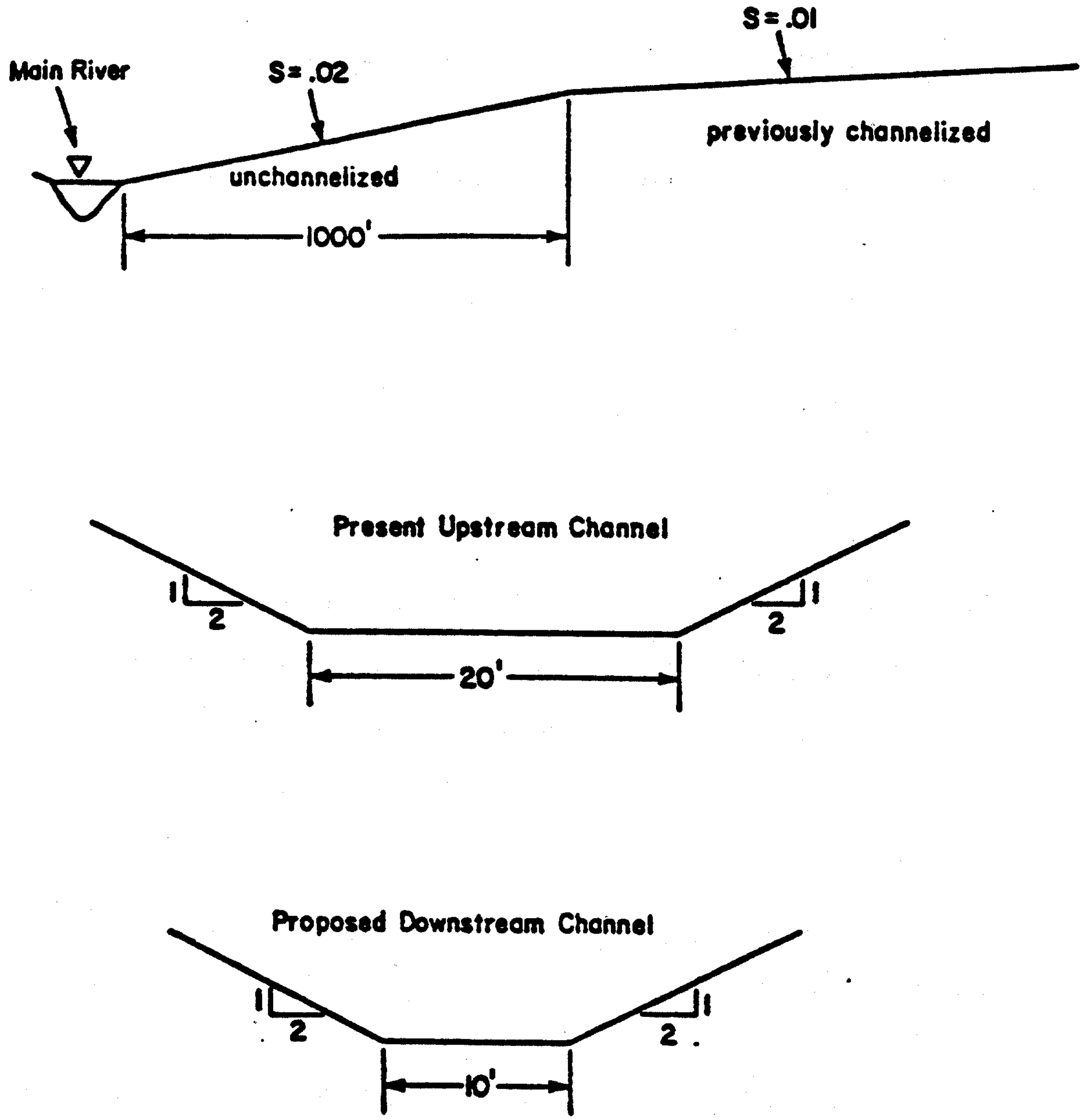

Figure 2-V: Physical Layout Of Design Example

$$S_d = \left[\frac{0.025}{0.025}\right]^2 \left[\frac{500}{500}\right]^{-1.4} \left[\frac{20.44}{29.08}\right]^{0.5} (0.01)$$

$S_d = 0.0084$ Where $S_d = s_2$

Recalculating the downstream depth of flow using S = 0.0084 for Q = 500 cfs:

Y = 3.25 ft. T = 23.00 ft. A= 53.7 ft.^{2} V = 9.2 fps

then, $S_d = \left[\frac{23.00}{29.08}\right]^{0.5} (0.01)$

$S_d = 0.009$

$S_d = 0.01$

Usually, one iteration provides a sufficiently accurate value. Therefore, the downstream channel should be constructed at a 1% slope to prevent erosion unless the channel is fully lined or grade control structures are installed in conjunction with lined banks.

Cut-Off Wall Design

I. Reach Length

Where a channel is located on a slope which is steeper than desirable, cut-off walls allow the stream to naturally degrade to the stable "equilibrium" slope between the walls while maintaining the integrity of the channel provided that the banks are lined. The amount of ultimate drop which will exist downstream of each is controlled by the reach length between each cut-off wall. The top of the cut-off wall should be placed at the existing invert with the placement of the wall determined by the following equation:

$$L_r = \frac{100h}{S_i - S_f} \qquad \text{eq. 9-V}$$

Where: L_r = Reach Length between walls, ft.

h = Height of ultimate drop, ft.

S_i = Initial slope, in percent.

S_f = Stable slope, in percent.

If the initial and final slope values are approximately the same then L_r will become excessively large and grade control <u>may not</u> be required.

The ultimate height of a cut-off wall above the downstream channel invert should not exceed 2 feet, nor should the cut-off walls be spaced closer than 30 feet.

II. Depth of Scour

The depth of scour below the cut-off wall should be determined using the following equation from <u>Design of Small Dams</u>, in conjunction with the 100-year discharge and the channel bottom width:

$$D_s = 1.32 \left[H_t\right]^{0.225} q^{0.54} \qquad \text{eq. 10-V}$$

Where: D_s = Maximum scour measured from the top of the tailwater, ft.

q = Discharge per unit width of channel bottom width, cfs/ft.

H_t = Total head measured from the upstream energy grade line to the downstream energy gradeline, ft (See Figure 3-V).

Equation 10-V was used to develop equation 11-V below which will directly provide the depth of cut-off wall (D_{cw}) required for a given drop (h).

$$D_{cw} = 0.06\, q + 3h - \frac{L_r}{100} + 1.25 \qquad \text{eq. 11-V}$$

Where: D_{cw} = depth of cut-off wall, ft.

q = discharge per unit width of channel bottom width, dfs/ft.

h = ultimate height of drop, ft. This is the desired drop and should vary from 0 to 2 feet.

L_r = Reach length, ft.

If the depth of the cut-off wall calculated is negative, then the reach length between the walls is excessive and grade controls may not be necessary.

PROCEDURE:

The stable channel slope should be calculated using equation 3-V. This is assumed to be the ultimate slope or final slope between the cut-off walls. A drop height (0' - 2') is then

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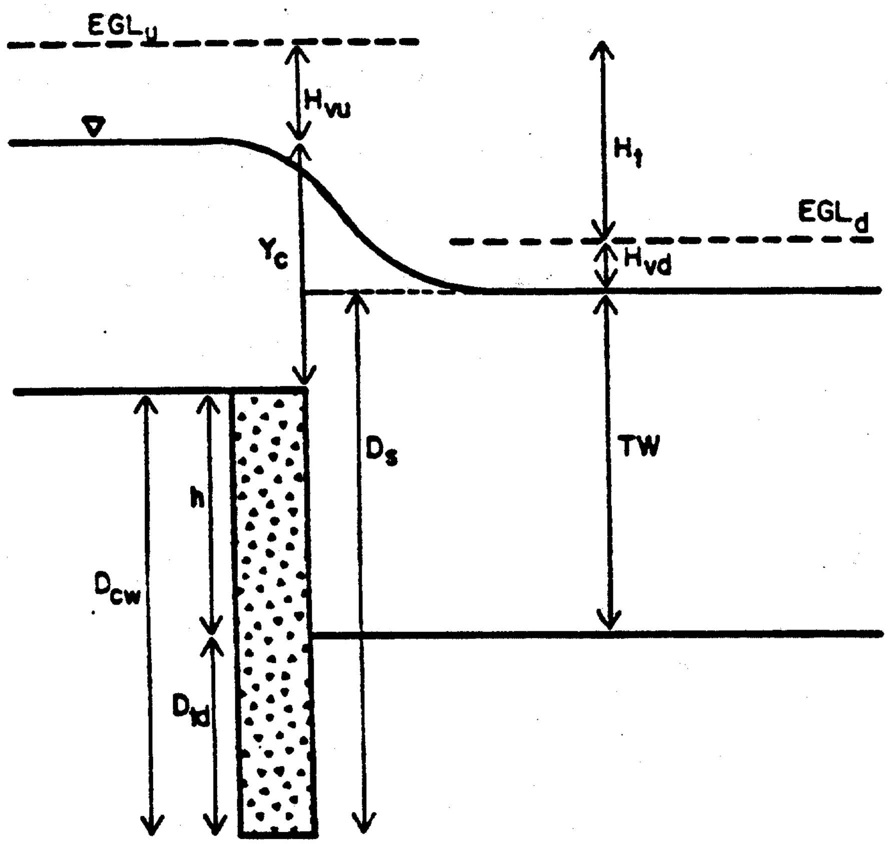

Figure 3-V: Depth Of Scour For Flow Over A Cut-off Wall

EGL_u = Energy grade line upstream

EGL_d = Energy grade line downstream

H_{vu} = Velocity head upstream

H_{vd} = Velocity head downstream

H_t = Total head differental between upstream and downstream sections at cut-off wall

h = Height of drop

TW = Tailwater

Y_c = Critical depth (assumed at brink)

D_s = Depth of Scour

D_{td} = Ultimate length of toe down

D_{cw} = Total depth of cut-off wall

D112

chosen and the cut-off wall spacing or reach length between walls (L_r) is determined using equation 9-V and the existing slope. The discharge per unit width should be calculated using the channel bottom width and the 100-year peak discharge. Then, using equation 11-V, the length of the cut-off wall can be determined.

Example (4): Using the information from example (2), and a height of drop of 2 feet, determine the placement and depth of the cut-off wall.

S_i = 1% h = 2 feet

S_f = 0.07% q = 22.5 cfs/ft

$$\text{Using } L_r = \frac{100(h)}{Si-Sf}$$

$$L_r = \frac{100(2)}{1-0.07} = 215 \text{ feet.}$$

Using equation 10

$$D_{cw} = 0.06(22.5) + 3(2) - \frac{215}{100} + 1.25$$

$$D_{cw} = 6.45 \text{ ft.} \cong 6 \text{ ft.}$$

The value of D_{cw} derived by the equation may be rounded off to the nearest foot. If S_f had equalled 0.8% or greater, then the value of D_{cw} would have been negative. In that case the difference between a 1% initial slope and a 0.8% final slope would not warrant grade controls.

The total length of a cut-off wall shall be a minimum of 3 feet and maximum of 6 feet. If the cut-off wall value is not within this range, modification of the channel design or wall spacing is required.

III. Design Channel Depth

To determine the correct depth of channel required to contain the 100-year peak discharge under the ultimate conditions, or during the initial conditions, the design channel depth, as measured from the top of the cut-off wall, must equal the largest of the following values:

(a) $C_d = 1.25\ Y_c$ eq. 12-V

(b) $C_d = Y_i$ + Freeboard as determined from Table 2-V   eq. 13-V

Where: C_d = calculated channel depth, ft.

Y_c = critical depth, ft.

h = height of drop, ft.

Y_i = initial depth of flow before degradation begins, ft.

See Table 2-V below for values for freeboard:

TABLE 2-V: Freeboard Requirements for Various Flow Regimes

FLOW REGIME	FROUDE NUMBER	FREEBOARD	MINIMUM* ALLOWED
Tranquil	$F < 0.86$		1'
Near Critical or Super-Critical	$F \geq 0.86$	$\frac{1}{6}\left[y + \frac{v^2}{2g}\right]$	1'

y = depth of flow, ft.

v = velocity, fps

g = acceleration due to gravity, 32.2 ft/sec.

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*Where the depth of flow is less than three feet the minimum of one foot shall not apply rather the freeboard may be determined using the equation of freeboard 1/6 $(y + \frac{v^2}{2g})$.

Example (5): From example (4) where h = 2 ft., q = 22.5 cfs/ft, L_r = 215 ft, what is the design depth for the channel?

a) $Y_c = \frac{v^2}{2g}$

$Y_c = \sqrt[3]{q^2/g}$

Y_c = 2.5 ft.

C_d = 1.25 (2.5) = 3.13 ft. from eq. 11-V

b) From example (2) n = 0.021, S = 0.01

Q_{100} = 450 cfs, then using Mannings equation

Y_i = 2.0 ft., V = 10.14 fps, F = 1.26

$$\text{Freeboard} = 1/6 \left[2.07 + \quad 2.07 \quad + \frac{(10.14)^2}{2(32.2)}\right]$$

$$= 0.6 \text{ ft.}$$

C_d = 2.0 + 0.6 = 2.6 ft.

The value 3.13 feet is the largest, therefore, the design channel depth shall be no less than 3 feet.

GENERAL SCOUR EQUATIONS

General scour refers to the horizontal and vertical changes which occur in a stream channel during a particular flood event. The following general scour formulas may be used as a guide in predicting the expected depth of scour and bank erosion width caused by a given flow event. Where piers or pipeline crossings exist, which will cause local scouring, other analyses are necessary to determine the total scour which is summation of the local and general scour. Long term degradation of the channel bed must also be considered if the bed slope if unstable.

In cases where the depth of scour is less than 3 feet, then 3 feet must be used as a minimum. if the channel is unstable or if changes in the top of width, expected land use, or discharge could occur which would cause a change in the present slope, then additional toe-down would be required to account for long-term degradation.

For subcritical flow in minor and major washes the bank protection toedown should be one half the depth of flow with 3 feet used as a minimum value.

Depth of toedown for major watercourses shall be determined by sediment transport study with consideration for local scour and regional long-term degration. All design work in a major water course should be substantiated with detailed hydraulic analyses. Accepted standards for design are to provide a minimum toe-down depth of not less than seventy-five percent (75%) of the normal depth of flow or ten (10) feet whichever is greater.

SECTION VI

ROADSIDE DRAINAGE

Roadways and/or street right-of-ways are often subjected to flooding either by runoff transported in the streets or at drainage crossings. The following policies concerning roadside drainage have been developed to reduce the hazards associated with storm runoff carried within or across a roadway and to provide maximum safety to the motoring public. The criteria thus established should serve as general guidelines in conjunction with existing field and topographic conditions. In all cases engineering judgement should be exercised in order to minimize adverse affects to adjoining property while maintaining traffic safety.

ROADWAY FLOW

Flow may be carried in local streets where curbs are provided, however, the depth of flow during the 100-year peak discharge shall not exceed the right-of-way width, be greater than one (1) foot in depth, nor exceed 10 feet per second in velocity. For collector and arterial roadways the spread of flow during the 10-year flood should not cover more than a bike lane, if existing, and one travel lane in each direction of travel. At a minimum one lane of travel in each direction shall be free of flow during the 10-year storm. Where flow is being carried in a street which has a normal crown, the longitudinal gutter shall be four (4) feet wide where a bike lane is provided and two (2) feet wide otherwise. The gutter shall be constructed with a minimum thickness of eight (8) inches of concrete to preclude erosion and high maintenance costs.

All flow which is carried in the streets must drain to a logical point; that is, flow must be discharged into a storm sewer system, a constructed or natural channel or a drainage easement. A drainage easement will be required whenever any natural watercourse has been altered to such a degree as to need a constructed channel cross-section and periodic maintenance; or if through urbanization, storm runoff is concentrated to such a degree as to require a defined channel.

ROADSIDE DRAINAGE

So that the ultimate traffic capacity and safety of a roadway are not infringed upon, channels should not be placed within a street right-of-way. Pima County's policy regarding roadside drainage is that drainage channels carrying offsite flow may not be constructed adjacent to roadways without written permission from the Pima County Department of Transportation and Flood Control District. Drainage swales within the right-of-way to carry flow generated by the roadway are acceptable where velocities as less than 4fps or where appropriate protection is provided.

Where permission is granted by Pima County, channels may be placed adjacent to the outside of a local street right-of-way provided that adequate horizontal seperation is maintained and access is preserved. Any channel paralleling a major uncurbed roadway shall have its top of bank set no closer than eight (8) feet from the edge of pavement with the bank closest to the roadway having a maximum side slope of 6 to 1 (horizontal to vertical) unless a guard rail is installed. Where a guard rail is used, the channel bank adjacent to the roadway may be vertical but the top of bank must then be set no closer than twelve (12) feet to the edge of pavement. Drainageways next to arterial or collector streets may not be used unless such installation is part of staged construction and the flow may be accomodated in a reasonably sized storm sewer at a later phase in construction. There are instances where it may not be possible to follow the aforementioned guidelines; these situations shall be addressed on a case-by-case basis by the Pima County Engineer.

ROADWAY DRAINAGE CROSSINGS

Roadway drainage crossing should be designed in accordance with the standards outlined below. Their design shall not create an adverse effect on adjacent properties and/or improvements, nor shall their design cause runoff to be diverted into another drainage basin. At a minimum, all weather access shall be provided to all lots within a subdivision. All weather access means that the ten (10) year flow must be taken underneath the local road; the twenty-five (25) year flow must be equal to or less than one (1) foot in the dip section, and the 100-year flow will be contained in the dip section. This policy shall apply to both public and private roadways, as these roadways are travelled by the "general" public regardless of whether the responsibility for safety, control and maintenance of the roadway is public or private.

FUNCTIONAL CLASSIFICATION

The extent of the drainage improvements are based on the functional classification of the roadway under which the runoff must flow, and by the amount of flow generated from a 100-year event at the location in question. Table I-VII outlines the minimum requirements for drainage crossing designs. Exceptions to these requirements will be allowed only with written permission from the Pima County Engineer where site conditions prevent compliance.

TABLE I-VI

DRAINAGE CROSSING REQUIREMENTS FOR
VARIOUS ROADWAY CLASSIFICATIONS

FUNCTIONAL CLASSIFICATION	TYPE OF CROSSING
Major Arterials	A
Minor Arterials	A
Major Collectors	A
Minor Collectors	B
Urban Collectors	B
Local Collectors	C
Local Streets	C*

TYPE A: Q_{50} under roadway; Q_{100} contained within dip (or Q_{100} under roadway, if possible)

TYPE B: Q_{25} under roadway; Q_{50} less than 1 foot in depth within dip and Q_{100} contained within dip

TYPE C: Q_{10} under roadway; Q_{25} less than 1 foot in depth within dip, and Q_{100} contained with dip.

*As required to provide all weather access for 100-year flows greater than 100 cfs. If this requirement is not met for any reason, then an obvious disclosure must be made on the final plat stating that all weather access is not provided to all lots.

Whenever a structure is designed which does not convey the 100-year event under the roadway, a hydraulic analysis should be made to investigate the impact of the 100-year discharge. If it is determined that a proposed drainage crossing may adversely affect adjoining properties and/or improvements, or that such a crossing will divert runoff into another drainage basin, the Engineer shall enlarge the structure under the roadway to mitigate the possible flood hazard and/or diversion. Additionally, for either the easements or dedicated right-of-way associated with a culvert's construction and maintenance along arterials and major collectors, enough land should be obtained to allow expansion of the culvert's capacity at a later date to accomodate the 100-year peak discharge should this prove necessary.

CULVERTS

For designing culverts, the Engineer is referred to Hydraulic Engineering Circular No. 5 entitled, "Hydraulic Charts for the Selection of Highway Culverts", by the Federal Highway Administration. Most culverts in Pima County are governed by inlet control; however, where gradients are flat or a culvert is unusually long then the culvert may be under outlet control.

End Treatment

Inlet

A headwall at a culvert inlet should normally be provided for pipes greater than thirty (30) inches in diameter as well as multiple pipes and box culverts. Headwalls will reduce entrance losses thus increasing the culvert's capacity and guaranteeing the stability of the inlet. The capacity can be further increased when the lip is beveled. Additionally, corrugated metal pipes (CMP's) require upstream headwalls to protect the entrance against the force of the flow as well as uplift and bouyancy forces and hydrostatic pressures.

Where channels are poorly defined and flow velocities are subcritical, simple headwalls are adequate. For supercritical flow and channels which are well defined flared endwalls as well as headwalls should be utilized.

Specially designed inlet transitions for constructed channels above what is normally required are necessary where the upstream flow is supercritical and/or the contraction is sudden. Transition rates for inlets recommended by the U.S. Army Corps of Engineers are 1 to 10 (horizontal to longitudinal) for mean channel velocities of 10 to 15 fps and 1 to 15 for mean channel velocities of 15 to 30 fps. The Engineer is also referred to Section IV for a more specific description of sudden contraction designs.

Normally erosion at the culvert inlet is not a major problem unless the upstream channel is skewed with regard to the culvert inlet, or no provisions are made to accomodate flow draining from the top of the road. Where these conditions exist special measures should be taken. If flow will directly impinge upon one of the upstream banks, then bank stabilization and/or transition curves should be provided. If storm runoff may enter upstream of the culvert, stabilized reinforced spillways should also be provided.

Outlet

Culvert outlets require special end treatment to protect against erosion caused by flow which could overtop the road as well as the flow discharging from the culverts.

When the 100-year flow over the road is expected to be significant then the downstream embankment containing the end of the culvert shall be protected by paving, grouted riprap or other means of permanent stabilization. The expected flow over the road will be considered significant if the time during which the road is over topped during the 100-year flow is in excess of ten minutes. If the 25-year flow is carried under the road for minor washes then the downstream embankment need not be protected accept for flow from the top of the road. For major washes protection for the downstream embankment will be paved if the structure is designed for less than the 100-year flow.

Culvert outlet velocities should be compared to the existing natural channel velocities, as well as the existing downstream channel conditions, in order to determine if further stabilization is required in order to prevent excessive scour and undermining of the culvert.

Outlet velocities for culverts flowing under outlet control are equal to the discharge divided by the cross-sectional area of flow. The area will either be equal to the area corresponding to critical depth, the tailwater depth or the area when the culvert is flowing full. For culverts flowing with inlet control the outlet velocities can be computed using the Manning's equation. However, because the depth of flow at the outlet is unknown, tables and charts for depth of flow and the equivalent wetted perimeter for pipes and box culverts should be employed. The velocities determined with the Manning's equation for short culverts will be slightly greater than what is actually occurring, as the flow in the culvert will not have established normal depth. While tailwater does not effect the outlet velocity for inlet control, the shape of the outlet structure, including the apron and wingwalls, will influence the exit velocity.

Scour Protection

Scour protection is necessary to maintain stable embankments, to prevent undermining of the culvert and to protect downstream property. All of the requirements for stabilization listed below apply to culverts in channels with stable slopes. It should be recognized that some channel slopes are unstable and that the general scour must be accounted for in the design as well as the localized scour that results from the discharge of flow from the culvert.

The following Arizona Department of Transportation standards are to be employed for determining outlet protection requirements:

1. All culvert outlet velocities which are less than or equal to 1.5 times the average natural stream velocity shall only require a cutoff wall for protection. Exceptions may be made, but these will be dependent upon downstream conditions, soils and the calculated depth of scour.

2. Where the outlet velocity is greater than 1.5 times the natural velocity, but less than 2.5 times, a dumped skin-grouted riprap apron will be provided; or, where soils are rocky, an adequate cut-off-way need only be provided. <u>Whenever outlet velocities are more than 1.5 times the stream channel velocity and exceed 10 fps then the dumped riprap should be or wire-tied</u>.

3. When outlet velocities exceed 2.5 times the natural downstream flow velocities some type of energy dissipator shall be required or an increase in the culvert opening will be necessary to reduce the outlet velocities. A description of St. Anthony stilling basins, U.S. Bureau of Reclamation Type VI basins, straight drop stilling basins or other common energy dissipators may be found in Chow's <u>Open Channel Hydraulics</u> or in the Federal Highway Administration Hydraulic Engineering Circular No. 14, <u>Hydraulic Design of Energy Dissipators for Culverts and Channel</u>.

Downstream Scour

The scour hole geometry which forms downstream of a culvert may be described in terms of the outlet velocity, Froude number and depth of flow as it exits the culvert. Figure I-IV shows the dimensionless scour hole geometry as it was determined from experiments conducted at the U.S. Army Corps of Engineers Waterway Experimental Station. They found the following equations described the maximum depth, width and length of the scour hole for culverts under inlet control.

$$D_{sm} = aD_c(R)^{0.375}t^{0.1} \qquad \text{eq. 1-VI}$$

$$W_{sm} = bD_c(R)^{0.915}t^{0.15} \qquad \text{eq. 2-VI}$$

$$L_{sm} = cD_c(R)^{0.71}t^{0.125} \qquad \text{eq. 3-VI}$$

Where, D_{sm} = Maximum depth of scour, ft.

W_{sm} = Maximum width of scour, ft.

L_{sm} = Maximum length of scour, ft.

D_c = Culvert diameter or depth, ft.

t = 0.2 times the time of concentration, minutes (With man-made or natural detention storage the time of concentration may be longer)

$R = \dfrac{Q}{D_c^{2.5}}$ for circular or square culverts

$= q/(D_c)^{1.5}$, for other shapes

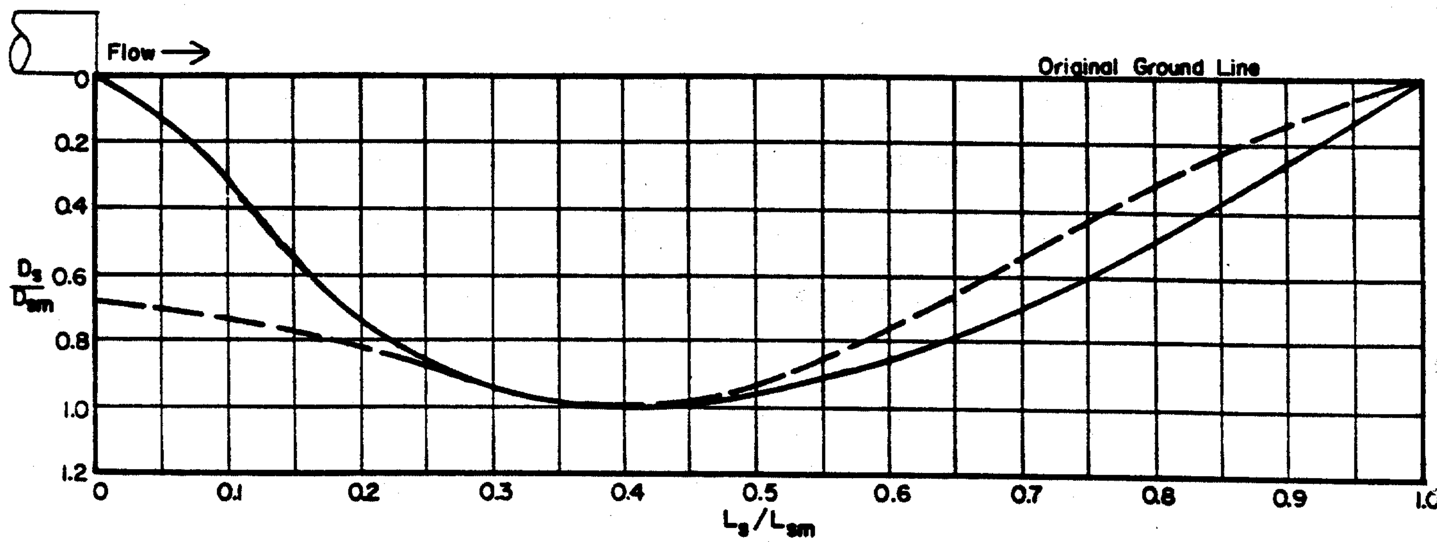
For minimum tailwater i.e. less than $0.5\ D_c$

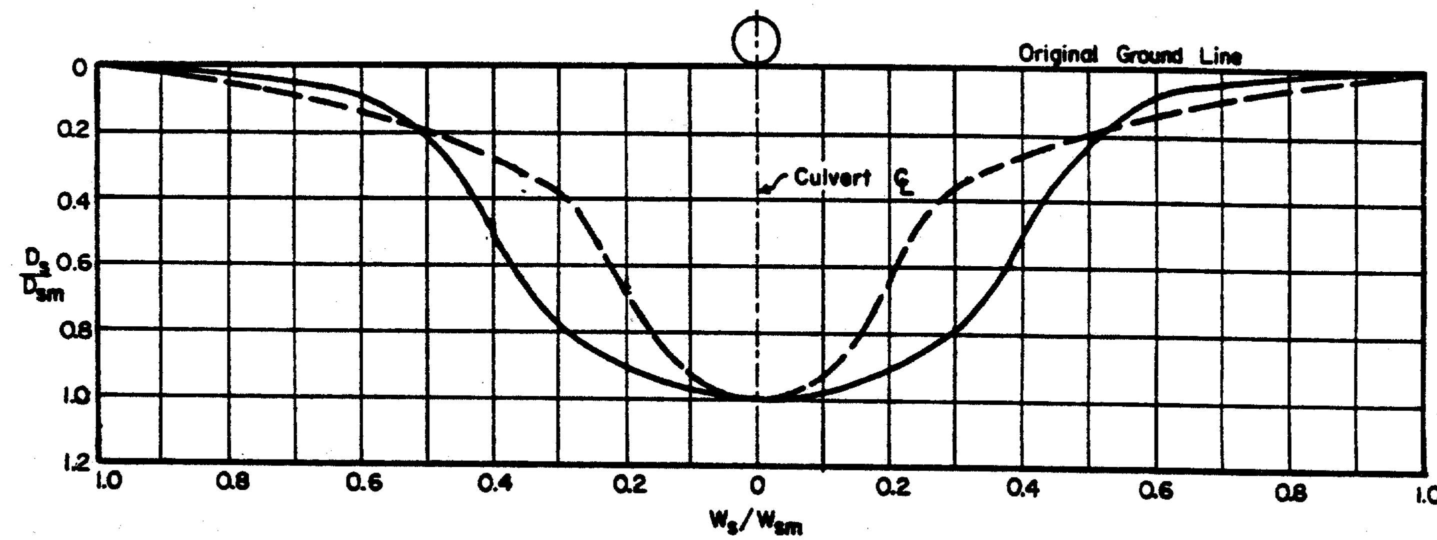
$a = 0.8$, $b = 1.0$, $c = 2.4$

For maximum tailwater, i.e. greater than $0.5\ D_o$ to $1D_o$

$a = 0.74$, $b = 0.72$, $c = 4.1$

Dimensionless Center-Line Profile





Dimensionless Cross-Section At 0.4 L_{sm}

Figure I-VI : Dimensionless Scour Hole Geometry

D_s = Depth of scour

D_{sm} = Maximum depth of scour

W_s = Width of scour

W_{sm} = Maximum width of scour

L_s = Length of scour

L_{sm} = Maximum Length of scour

——— Maximum Tailwater

– – – Minimum Tailwater

The engineer is also referred to Hydraulic Circular No. 14, <u>Hydraulic Design of Energy Dissipators for Culverts and Channels</u> by the Federal Highway Administration.

Cutoff Walls

A cutoff wall placed at the culvert's outlet provides adequate protection downstream when the scour will not be excessive, or where the development of a scour hole will not undermine nearby structures so that it is practical to allow localized scour.

The following design criteria are applicable to cutoff walls.

1. The depth of the cutoff wall shall equal to 70% of the maximum depth of scour.
2. The appropriate width of the cutoff wall will be a minimum of one-third the maximum scour width.
3. The depth of the cutoff wall should not normally exceed six (6) feet. <u>Where a deeper wall is necessary either another form of protection shall be required or further engineering analysis will be required to substantiate the wall's structural stability.</u>

Riprap Apron

Riprap aprons placed downstream of culverts provide protection against scour immediately around the culvert as well as providing for the uniform spreading of the flow and descreasing the flow velocity, thus mitigating downstream damages.

These riprap aprons may be designed as simple horizontal aprons as shown on Figure 2-VII, with tapered sides of 2 to 1 for minimum tailwater and 5 to 1 for maximum tailwater. The total length of the basin should be:

$$L_{sb} = D_c \ (8 + 17 \log F), \text{ for minimum tailwater} \qquad \text{eq. 4-VI}$$

and

$$L_{sb} = D_c \ (8 + 55 \log F), \text{ for maximum tailwater} \qquad \text{eq. 5-VI}$$

Where L_{sb} = length of scour basin, ft

F = Froude number of flow at the culvert outlet

D_c = Culvert diameter or depth, ft.

<u>When the Froude number is less than or equal to one, an apron length of approximately eight (8) times the culvert diameter should be used. If the length of protection becomes excessive another form of protection should be used as the outlet velocity exceeds the design criteria for rock aprons.</u> The correct size of riprap for this type of horizontal apron is:

$$d_{50} = 0.02 \left[\frac{Dc^2}{TW}\right] (R)^{\overset{1.33}{0\cancel{.1}33}} \qquad \text{eq. 6-VI}$$

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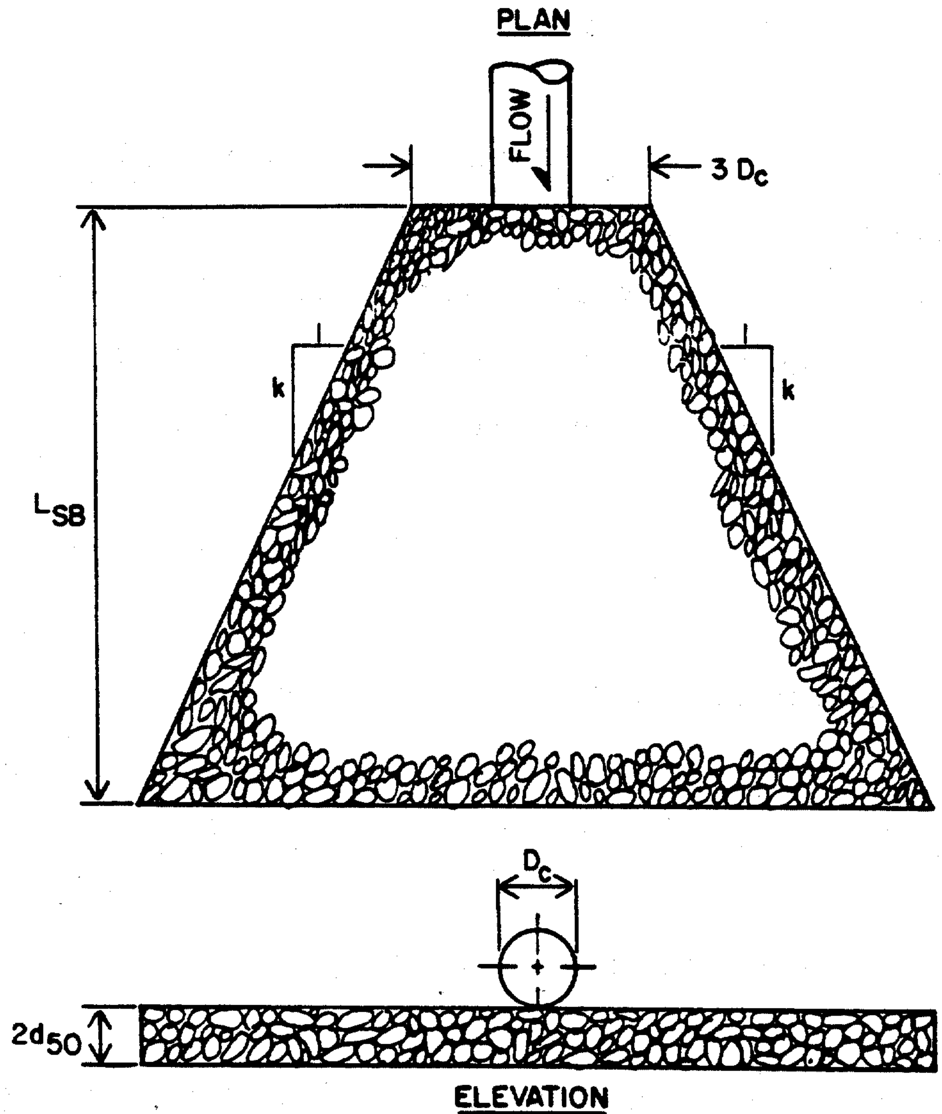

Figure 2-VI:
Recommended Configuration Of Riprap Blanket

k= 2 for minimum tailwater

k= 5 for maximum tailwater

L_{SB}= Length of scour basin, ft.

D_c = Culvert diameter, ft.

d_{50}= Riprap median diameter

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Where d_{50} = median riprap diameter, ft

TW = tailwater depth above invert of culvert outlet, ft

and the other variable are the same as given for equation 1-IV.

Riprap basins may also be built as plunge basins where the dissipation pool is pre-formed. Figure 3-IV illustrates some typical dimensions of the basin with 3 to 1 sides. The advantage of pre-formed basins is that the required riprap size is reduced. With a pre-formed basin the size of riprap required is:

$$d_{50} = 0.0125 \frac{D_c^2}{TW} (R)^{1.33} \quad \text{eq. 7-IV}$$

for depression equal to 0.5 D_c and

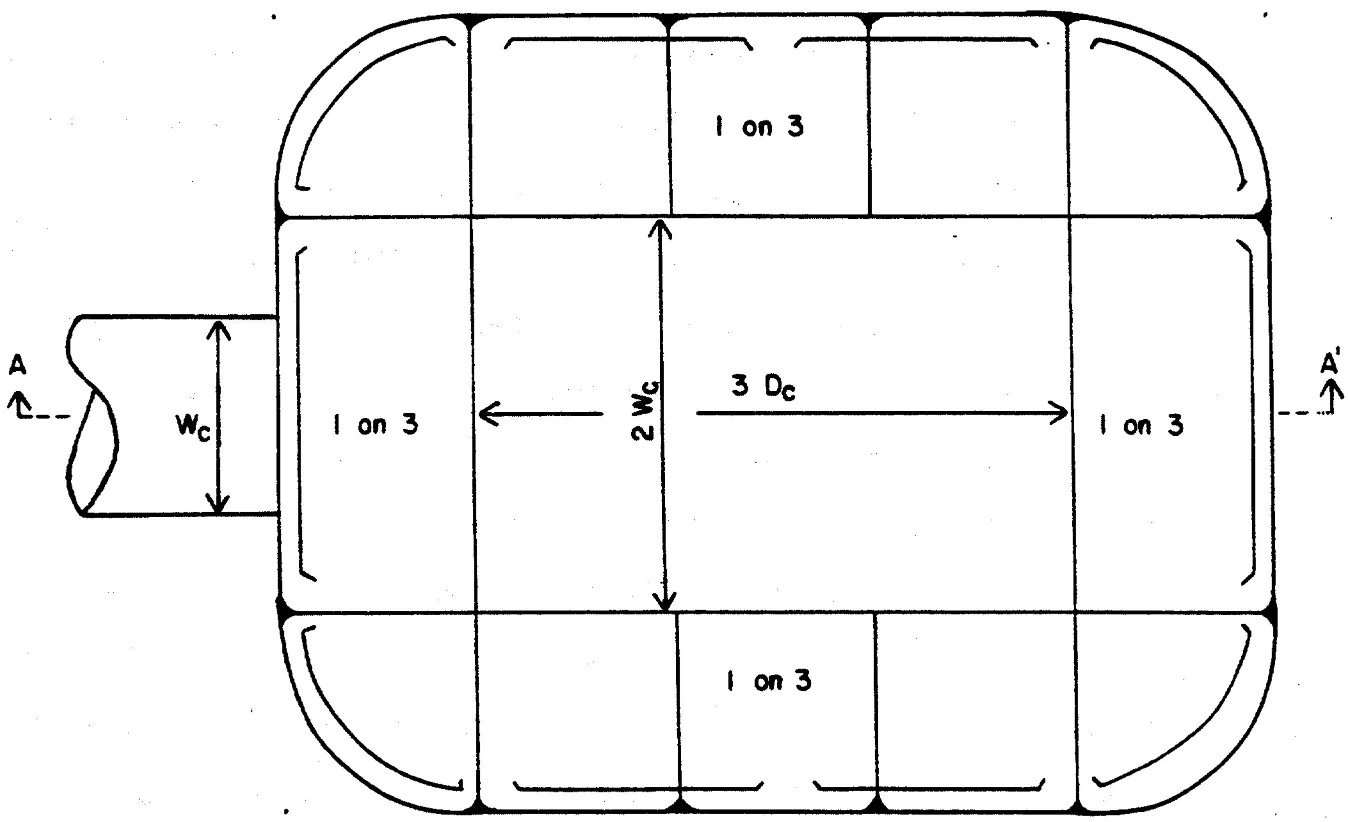
$$d_{50} = 0.082 \frac{D_c^2}{TW} (R)^{1.33} \quad \text{eq. 8-IV}$$

for a depression equal to 1.0 D_c

Rip rap basins should not be utilized if the Froude number exceeds three (3).

There are various other forms of preformed riprap plunge basins that are also acceptable with the approval of the Pima County Department of Transportation and Flood Control District. Figure and charts from HEC-14 have also been included for the design of plunge basins.

PLAN



SECTION A-A'

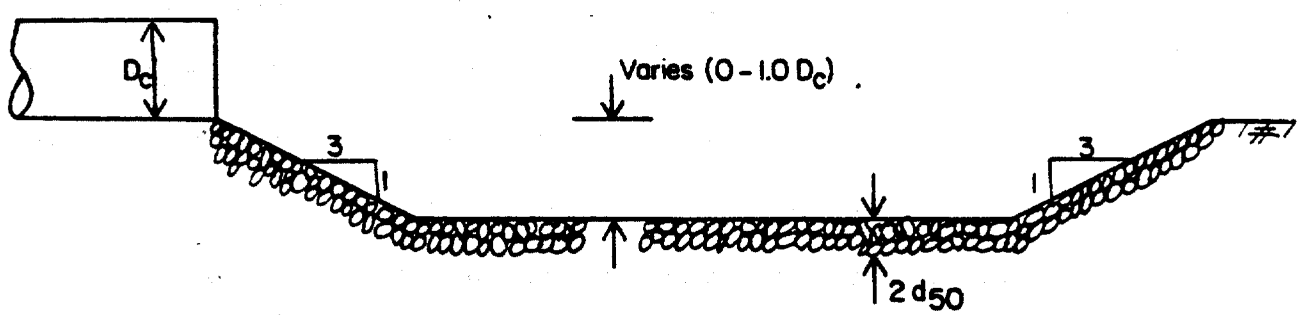


Figure 3-VI: Riprap-Lined, Preformed Scour Hole

W_c = Width of Culvert, ft.

D_c = Depth of culvert, ft.

d_{50} = Riprap median size diameter, ft.

AT-GRADE CROSSINGS

Most stream crossings in Pima County have been built in such a manner as to convey all or part of the flow across the road. This is usually acceptable because of the low frequency with which stream flows occur in Pima County. However, when floods do occur they may cause hazardous conditions both during the flow and immediately afterwards because of possible erosion downstream of the crossing and/or sediment and debris which may be deposited in the crossing. The following design criteria shall be used where applicable for at-grade crossings to reduce potential hazards and lower maintenance requirements.

1. Dip crossings should be built at a minimum 4% cross slope to reduce deposition of sediment in the dip section. This 4% grade shall be produced by supplying the vertical rise on the upstream side of the dip section with the downstream side meeting existing grade (See Figure 4-IV). The 4% cross slope shall be met in all cases unless horizontal and vertical controls for traffic safety dictate otherwise. In designing dip crossings the engineer is referred to the chart on comfortable speed on vertical curves contained in Pima County's Highway Design Standards.
2. Depth of flow for the 25-year storm shall not exceed one-foot within the dip section.
3. A downstream cutoff wall will be provided to prevent erosion and road damage.

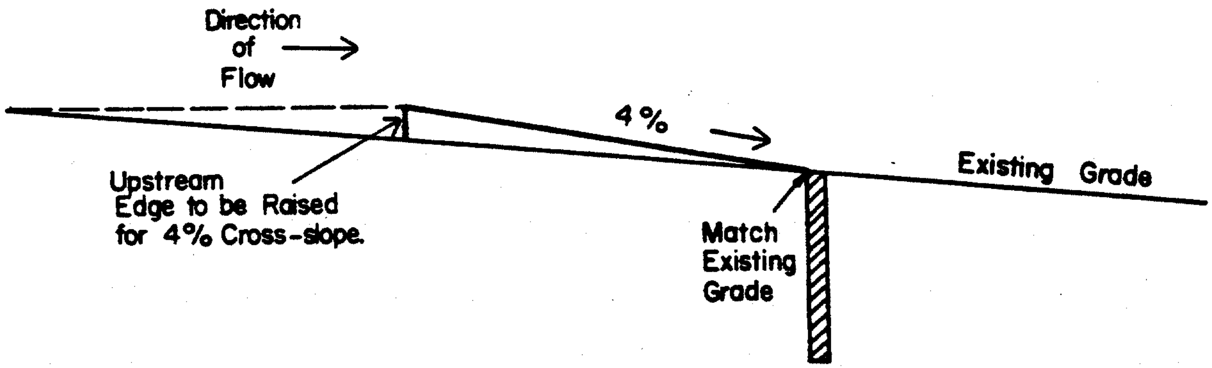


Figure 4-VI :Dip Crossing Design With A 4% Cross-Slope

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4. An upstream cutoff wall shall be provided that is a minimum of two feet in depth.

5. Length of the cutoff wall will be at a minimum equal to the top width of the 10-year flow through the dip section.

Cutoff Wall Design

The following method for determining the depth of the cutoff wall downstream of the dip crossing was developed from information on scour presented in Hydraulic Circular No. 14, <u>Hydraulic Design of Energy Dissipator for Culverts and Channels</u> by the Federal Highway Administration. The downstream depth of scour is:

$$D = 0.82\ (Y_e) \left[1.3v\ (2\ Y_e^2)/Y_e^{\ 2.5}\right]^{0.375} (0.2t_c)^{0.1}$$

eq. 9-VI

or

$$D = (Y_e)^{0.813}\ (V)^{0.375}\ (t_c)^{0.1}$$ eq. 10-VI

Where, D = depth of scour, ft

V = average velocity, fps

Y_e = maximum depth of flow for a parabolic section, ft.

t_c = time of concentration, minutes.

These variables are all for the 100-year design storm. This method uses the upstream flow conditions to predict the amount of scour based on the expected effective time of scour which is equal to the time when the discharge is equal to 90% of the 100-year peak discharge. This time is approximately twenty (20) percent of the time of concentration for the 100-year peak discharge as determined by a snythetic hydrograph developed utilizing the Soil Conservation Service method or the one developed by the Pima County Department of Transportation and Flood Control District for short-duration thunderstorms.

If the depth of the required downstream cutoff wall is in excess of six (6) feet, another form of protection shall be required and/or further engineering analysis will be necessary to establish the cutoff wall's structural stability.