HANDBOOK FOR ARIZONA COMMUNITIES
On Floodplain Management and the National Flood Insurance Program

APPENDIX M

ARIZONA DEPARTMENT OF WATER RESOURCES
FLOOD WARNING AND DAM SAFETY SECTION

REQUIREMENT FOR

FLOODPLAIN AND FLOODWAY DELINEATION

IN RIVERINE ENVIRONMENTS

The Director of the Arizona Department of Water Resources under the authority outlined in ARS 48-3605(A) establishes the following standard for delineation of floodplains and floodways in riverine environments, and for use in floodplain management in Arizona:

Flood discharge rates, water surface elevations, and floodway limits determined for use in fulfilling the requirements of approved local community and county flood damage prevention ordinances will be determined by applying the alternative procedures outlined in State Standard Attachment 2-96 entitled "Delineation of Riverine Floodplains and Floodways in Arizona" (SSA 2-96) or by an alternative procedure reviewed and accepted by the Director.

For the purpose of application of these procedures, floodplains will include all watercourses officially identified by the Federal Emergency Management Agency as part of the National Flood Insurance Program; all watercourses which have been identified by a local floodplain administrator as having significant potential flood hazards; or all watercourses with drainage areas more than 1/4 of a square mile or a 100-year estimated flow rate of more than 500 cubic feet per second. Application of administrative floodway procedures will be only for streams that do not currently have a floodway identified by the Federal Emergency Management Agency as part of the National Flood Insurance Program. Application of the procedures outlined in SSA 2-96 will not be necessary if the local community or county has in effect a drainage, grading or stormwater ordinance which, in the opinion of the Director, results in the same or a more stringent level of flood protection than application of the procedure would ensure.

This requirement is effective July 1, 1996. State Standard 2-96 and State Standard Attachment 2-96 replace State Standard 2-92 and State Standard Attachment 2-92, adopted in September, 1992, and State Standard 2-92 (Supplement 1) and State Standard Attachment 2-92 (Supplement 1), adopted in November, 1994. Please discard all copies of the superseded standards and attachments.

Copies of this State Standard and State Standard Attachment can be obtained by contacting the Department's Flood Warning and Dam Safety Section at (602) 417-2445.

NOTICE

This document is available in alternative formats. Contact the Department of Water Resources, Flood Warning and Dam Safety Section at (602) 417-2445 or (602) 417-2455 (TDD).

ARIZONA DEPARTMENT OF WATER RESOURCES
FLOOD WARNING AND DAM SAFETY SECTION

Delineation of

Riverine Floodplains and Floodways

in Arizona

500 North Third Street
Phoenix, Arizona 85004

(602) 417-2445

STATE STANDARD ATTACHMENT JULY, 1996

SSA 2-96

DISCLAIMER OF LIABILITY

The methods contained in this publication are intended to be a reasonable way of setting minimum floodplain management requirements where better data or methods do not exist. As in all technical methods, engineering judgment and good common sense must be applied and the methods rejected where they do not offer a reasonable solution.

It must be recognized that while enforcement of the criteria established herein will generally reduce flood damages to new and existing development, there will continue to be flood damages in Arizona. Where future-condition hydrology (which considers the cumulative effects of development) is not used, future development will probably increase downstream peak discharge rates which may result in flooding. Unlikely or unpredictable events such as dam failures may also cause extreme flooding.

The Arizona Department of Water Resources is not responsible for the application of the methods outlined in this publication and accepts no liability for their use. Sound engineering judgment is recommended in all cases.

The Arizona Department of Water Resources reserves the right to modify, update, or otherwise revise this document and its methodologies. Questions regarding information or methodologies contained in this document and/or floodplain management should be directed to the local floodplain administrator or the office below:

Flood Warning and Dam Safety Section
Arizona Department of Water Resources
500 North Third Street
Phoenix, Arizona 85004

Phone: 602-417-2445
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INTRODUCTION

Purpose

State Standard 2-96. The intent of this document is to provide methodologies for estimating 100-year peak discharges, delineating 100-year floodplain limits, and determining administrative floodway boundaries for riverine floodplains in Arizona. Methodologies for non-riverine floodplain areas, such as alluvial fans, are not addressed. The purpose of estimating 100-year peak discharges is for use in delineating 100-year floodplain and floodway limits and estimating hydraulic conditions associated with 100-year flooding, and for use in other water resource management purposes. The purpose of delineating 100-year floodplain limits is to prevent or reduce flood risk for activities in flood-prone areas. The purpose of determining an administrative floodway is to provide a zone of acceptable encroachment which will allow for development while reducing or eliminating damage to property and preventing hazards to life and health. An administrative floodway is defined as a zone of conveyance which will safely pass floodwaters.

Alternative Methodologies. The purposes of the floodplain management methodologies recommended in this document are to reduce or eliminate flood damage to property, to prevent the disruption of normal activities by flooding, to prevent hazards to life and health from flooding, and to regulate the use of flood-prone lands. It is necessary to adopt uniform floodplain management criteria that provide the desired degree of protection throughout Arizona. However, these criteria must be flexible enough to allow communities to use approved alternative methodologies which may not be described in this document. To be approved, an alternative methodology must be reviewed and approved in writing by appropriate Arizona Department of Water Resources (ADWR) staff, as well as by the floodplain administrator for the community in which it will be applied. Application of the methodologies outlined in this document is not mandated if the local community or county has in effect a drainage, grading or stormwater ordinance which, in the opinion of the ADWR Director, results in the same or a more stringent level of flood protection than application of the methodologies described in this document.

Floodplain Management. To provide a context for the recommended floodplain management methodologies, a brief discussion of the National Flood Insurance Program (NFIP), the role of the Arizona Department of Water Resources, and floodplain and floodway standards are provided in Appendices A, B, and C, respectively.[1]

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[1] Many floodplain activities require permits from or review by other local, state, or federal agencies (e.g., Army Corps of Engineers Section 404 permit (placement of fill), or Environmental Protection Agency NPDES stormwater permit). Individuals should check with the appropriate agencies to determine specific needs.

General Information

Although the data needed for floodplain management and flood hazard identification are generally straightforward, the procedures for obtaining these data are widely varied. In general, four types of information are required:

- Discharge Rate
- Floodplain Limits/Water Surface Elevation
- Floodway Boundaries
- Documentation

Discharge Rate. The 100-year peak discharge rate is used for all floodplain management methodologies described in this document. The 100-year flood, or one percent flood, is the highest rate of flow expected during the 100-year flood event and is commonly measured in the U.S. in cubic feet per second (cfs). Appendix C presents a discussion of the 100-year flood as the nationwide floodplain management standard.

Floodplain Limits/Water Surface Elevation. The 100-year floodplain limits are the area of inundation resulting from the 100-year peak discharge. The area of flood inundation may be estimated by predicting the width of flooding over a floodplain, or by comparing the estimated 100-year water surface elevation to known ground elevations. It is also possible to estimate the flow velocity, depth, scour potential, and other flood characteristics for a particular location when estimating the floodplain limits and water surface elevation.

Floodway Boundaries. The floodway is the area along a watercourse which must be reserved for the conveyance of flood waters. The floodway width is estimated using the 100-year peak discharge and appropriate local encroachment criteria. Encroachment means placing fill, structures, developments, or other material within the floodplain limits. This document describes specific procedures for estimating *administrative* floodway boundaries. An administrative floodway is a floodway delineated by the procedures described in this document, as opposed to floodway delineation procedures mandated by the NFIP.

Documentation. Documentation of the procedures used to estimate peak discharge, floodplain limits, water surface elevation or other hydraulic characteristics, and floodway boundaries must be maintained by each individual NFIP participating community for all development within the regulatory floodplain. State Standard 1-90 describes documentation requirements for floodplain studies in Arizona.

Three-Level Approach

Procedures for estimating the first three types of floodplain management information listed above are described below. Three levels of analysis are presented for each type of information. *Level 1* procedures are the minimum level of regulation acceptable, and are intended for use where only limited site and flood data are available, and where site improvements are minimal. *Level 2* procedures require a basic understanding of hydrologic principles and mathematics, and are appropriate for single lot developments where some site and flood data are available. *Level 3* involves detailed engineering analysis, and is intended for use on larger developments or where regional floodplain management issues are impacted.

Throughout this document Level 1, Level 2, and Level 3 refer to increasing levels of effort in analysis. **It should be understood that the lowest level of effort generally produces the most conservative results.** Level 1 will generally produce more conservative results than Level 2. Likewise, Level 2 will generally produce more conservative results than Level 3. It is the responsibility of each community and the individuals proposing floodplain improvements to determine the appropriate level of analysis. Communities and/or property owners are encouraged, and in some cases will be required to spend the necessary time and money to perform a Level 2 or Level 3 engineering analysis in order to comply with local, state, or federal regulations, and to ensure that all new construction in Arizona is protected against flood damages.

Wherever appropriate, existing detailed engineering or hydrologic information should be used, instead of the results of Level 1, 2 or 3 methodologies. For example, where a detailed hydrologic model for a watershed has already been developed or USGS gage information is available for a watercourse, the existing flow rate estimates should be used to determine floodplain limits or floodway boundaries. Similarly, where a detailed HEC-2 model has been developed for a stream reach for other purposes, the results of that model should generally be used to delineate the floodplain and floodway, rather than the Level 1 or Level 2 methodologies described in this document. In general, for any given site, the same Level methodology should be used for hydrology, floodplain delineation and floodway boundary determination. For example, if a Level 2 floodway is being delineated, the Level 2 (or lower) hydrology methodology should be used to estimate the 100-year discharge.

FEMA Floodplain Zones

This document presents floodplain hazard identification procedures which are acceptable for use in Arizona. However, a driving force in floodplain management and implementation of the NFIP in the United States is Federal Emergency Management Agency (FEMA), which has prepared the majority of floodplain delineations in Arizona. Because it is vital for all Arizona communities to remain eligible for the NFIP, any existing study which has been adopted by FEMA shall be considered the minimum base for floodplain management for the specific study area or flooding source. That is, the rate of flow, water surface elevations, floodplain limits, and floodway boundaries as accepted by FEMA are the minimum values to be used. Therefore, the first step by a community, before performing any of the analyses described in this document, should be to determine if there is an existing FEMA study which has been performed for the subject area.

ADWR will assist any community that requires help locating existing FEMA studies. If the subject area is located within an existing FEMA flood hazard zone delineated by detailed methods, a Level 3 analysis will be required if any changes are proposed. In cases where a community feels strongly that a FEMA study is incorrect, ADWR will assist the community in appealing to FEMA to correct the study's deficiencies.

Guidance for managing development in FEMA flood zones is provided in:

1. FEMA, National Flood Insurance Program & Related Regulations, 44CFR, Chap. 1.

2. FEMA, 1995, Guidelines: Specifications for Study Contractors - FEMA 37.

3. FEMA, 1995, Managing Floodplain Development in Approximate Zone A Areas - A Guide for Obtaining and Developing Base Flood (100-year) Flood Elevations - FEMA 265.

PROCEDURES

Overview

This section briefly describes the recommended methodologies for estimating the 100-year discharge, floodplain limits or water surface elevation, and floodway boundaries at any riverine site in Arizona. More detailed information and example applications for Level 1 and Level 2 methodologies are provided in Appendices D - I. Detailed descriptions of Level 3 methodologies are beyond the scope of this document, but should be readily available from an Arizona-registered professional engineer.

Level 1 Procedures

The purpose of Level 1 analysis is to provide floodplain management procedures that are simple to use and that require data which are readily available. Level 1 procedures estimate flood depth and floodway width independent of detailed site topography, hydraulic equations, or hydrologic models. Level 1 methodologies will provide conservative values for peak discharge, flood depths, and floodway widths, so that finished floor elevations and floodway setbacks can be estimated for a proposed project with very little data and engineering expertise. However, it is necessary to be able to delineate and measure the watershed area to apply Level 1 procedures. Watershed area can be delineated on readily-available U.S. Geological Survey (USGS) topographic quadrangle maps.

Table 1. Level 1 Methodology Summary

Data Type	Variable Obtained	Data Required	Methodology	Example
Discharge	100-Year Discharge	Watershed Area	USGS Data Envelope Curve	Appendix D
Floodplain	Flow Depth	Watershed Area	FEMA Data Regression Equation	Appendix E
Floodway	Floodway Width	Watershed Area	FEMA Data Regression Equation	Appendix F

Discharge. The Level 1 discharge methodology was derived from a recently published comprehensive analysis of stream gage records in the Southwest (Thomas et. al., 1994). The methodology consists of an envelope curve constructed using the maximum discharges from Arizona and the Southwest gaged by the USGS. Because the methodology is based on an envelope curve, the peak discharge estimates tend to be conservative. An example application of the Level 1 discharge methodology is provided in Appendix D.

Floodplain/Floodway. Equations for the Level 1 floodplain and floodway methodologies were derived using depth, floodway width and drainage area from FEMA flood insurance studies in Arizona. Regression analyses were performed using these data for various regions in Arizona. Estimation of a 100-year discharge rate is not required to estimate floodplain depths or floodway widths using the Level 1 procedures. Detailed information on the development of the regression equations used in this report is available from the Flood Warning and Dam Safety Section of ADWR. Level 1 floodplain and floodway examples are provided in Appendices E and F, respectively.

Within the Level 1 floodway fringe, the regulatory elevation shall be a minimum of 1 foot above the highest adjacent existing ground elevation, or one foot above the estimated floodplain elevation, whichever is higher. If a drainage structure such as a combination culvert/roadway dip-section, bridge, or embankment of any kind is to be placed across a watercourse, a Level 2 or Level 3 analysis is required.

Level 2 Procedures

Level 2 requires the estimation of the 100-year peak discharge (hydrology) and the 100-year floodplain (hydraulics) using simplified engineering procedures. If a drainage structure such as a combination culvert/roadway dip-section, bridge, or embankment of any kind is to be placed across a watercourse that will create a significant backwater effect or hydraulic obstruction, a Level 3 analysis may be required. The Level 2 procedures are:

Table 2. Level 2 Methodology Summary

Data Type	Variables Obtained	Data Required	Methodology	Example
Discharge	100-Year Discharge	Watershed Area Mean Elevation Mean Annual Evaporation Mean Annual Precipitation	USGS Regression Equations	Appendix G
Floodplain	Water Surface Elevation Channel Velocity Channel Depth	Channel Cross Sections Roughness Value 100-Year Discharge Rate Channel Slope	Manning's Rating	Appendix H
Floodway	Floodway Width Floodway Elevation	Channel Cross Sections Roughness Value 100-Year Discharge Rate Channel Slope	Administrative Floodway	Appendix I

Discharge. Equations for estimating peak discharges for ungaged watersheds in Arizona and the Southwest were developed by the U.S. Geological Survey using stream gage records, regression analyses and newly developed statistical procedure for arid regions. Unique equations were developed for each of seven regions within Arizona, including a region for watersheds at high elevation (> 7,500 feet). Required information includes the watershed area, and may include one of the following: (1) mean annual precipitation, (2) mean elevation, or (3) mean annual evaporation. Figures showing the required precipitation and evaporation data for the entire State of Arizona, and sample applications are provided in Appendix G. Mean elevation may be determined from USGS topographic maps, as described in Appendix G.

The Level 2 discharge methodology illustrated in Appendix G is generally not recommended for certain types of watersheds, including the following:

- Urban watersheds
- Alluvial fan and distributary flow area watersheds
- Watersheds with significant areas of flood irrigation agricultural fields
- Watersheds with highly permeable soils (e.g., fractured limestone, volcanic cinders)
- Watersheds with significant flood control reservoirs or diversions

For the types of watersheds listed above Level 1 or Level 3 discharge methodologies generally should be used. Other Level 2 application guidelines are provided in Appendix G.

Floodplain Limits/ Water Surface Elevation. Floodplain limits can be delineated by using Manning's equation to estimate water surface elevation (normal depth) for channel cross sections located at the proposed development. The developer should provide normal depth calculations at several representative cross-section locations adjacent to the proposed improvement/development. Cross-sections should also be located both upstream and downstream of the proposed improvement/development. Cross sections should be spaced at 300 to 500 feet intervals with a minimum of three cross sections required along short reaches. Calculations must include pre- and post-development conditions. Manning's equation, applied by using manual calculations or computer software, is recommended to compute normal depth. Floodplains will be delineated using normal depth or critical depth, whichever is greater. Where critical or supercritical flow exists, the finished floor elevation should established using the energy grade line as the base flood elevation, as illustrated in Appendix H.

It is recommended that structures not be placed in the 100-year floodplain without some type of floodway analysis. At minimum, an assessment of flood depth and velocity should be performed and structures[2] should not be placed within the area where the following criteria are exceeded:

- Houses built on foundations: Depth x Velocity > 10 and Depth > 2.5 ft.
- Mobile homes: Depth x Velocity > 6 and Depth > 1.5 ft.

A Level 2 floodplain delineation example is provided in Appendix H.

Administrative Floodway Boundaries. The administrative floodway is an area reserved for the conveyance of flood flows. Figure 1 illustrates a typical riverine 100-year floodplain, administrative floodway, and floodway fringe. The area between the edge of the 100-year floodplain and the administrative floodway is commonly referred to as the floodway fringe. Encroachment into the floodway fringe area shall be allowed for development as long as the structure(s) is protected from the

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[2] These criteria do not apply to structures constructed on engineered fill material elevated above the regulatory water surface elevation.

100-year flood. No encroachment of any kind shall be allowed within the area determined as an administrative floodway using a Level 2 procedure.

The administrative floodway procedures may be used if all of the following criteria are met:

- A detailed FEMA Flood Insurance Study does not exist for the watercourse.
- The local floodplain administrator has approved the use of the Level 2 floodway procedure.
- The procedure is being used for a single lot residential or single lot commercial development.
- The watercourse consists of an identified flooding area which may or may not have overbank flooding and has a subcritical flow regime.
- The flood conveyance area of the watercourse is generally uniform, without significant changes in cross-sectional geometry or longitudinal slope.

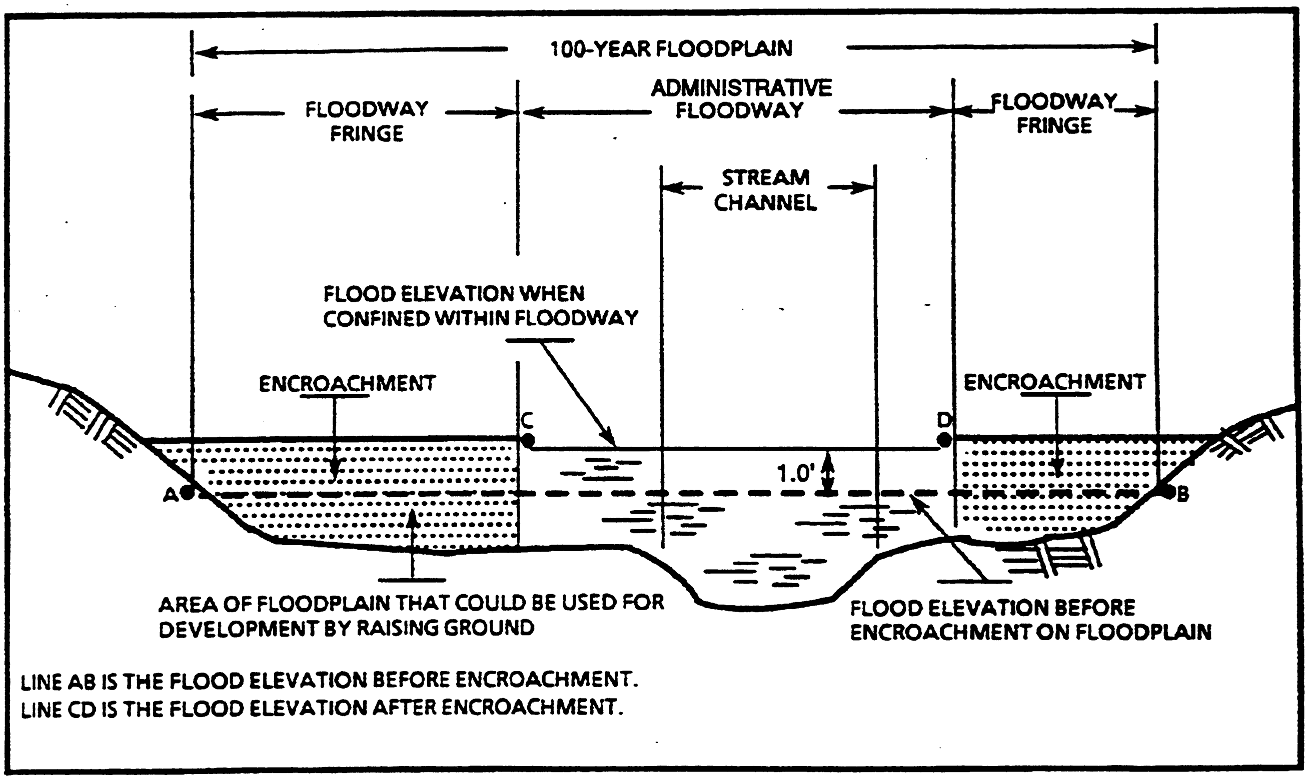
The procedure is not intended for use if a detailed FEMA Flood Insurance Study exists or for the following conditions:

- Multi-lot development.
- Alluvial fan, distributary, or bifurcated flood hazard areas.
- Watercourses having a supercritical flow regime.
- Watercourses which have large changes in cross-sectional geometry or longitudinal slope within close proximity of the study area.
- Watercourses where significant hydraulic structures are present that would create backwater effects, such as large roadway crossings with culverts or bridges, dams, and detention/retention structures.

Level 3 floodway delineation procedures are required for the situation listed above. Administrative floodway procedures are described in Appendix I.

Figure 1

FLOODPLAIN - FLOODWAY - FLOODWAY FRINGE ILLUSTRATION



Administrative Floodway Fringe Encroachment Standards. Encroachment into the floodway fringe area is typically accomplished using various methods including pilings, piers, columns, and/or engineered fill material. Finished pad elevations for structures shall be constructed 1 foot above the encroached 100-year water surface elevation[3], regardless of the method of encroachment used. For pilings, piers, columns, and similar methods, scour and lateral force analysis shall be performed and incorporated into the design.

For engineered fill, the following criteria shall be met:

- Fill material will be placed to raise the ground surface uniformly 1 foot above the encroached 100-year water surface elevation.
- Fill must be compacted. The typical compaction standard is 95% of the maximum density obtainable with the Standard Proctor Test method issued by the American Society for Testing and Materials (ASTM Standard D-698). This requirement applies to fill pads prepared for residential or commercial structure foundations. This requirement does not apply to filled areas intended for other uses.
- The toe of the proposed fill material shall intercept the natural ground surface at the floodway boundary location established by the encroachment analysis.
- Fill slopes for granular materials shall not be steeper than three horizontal on one vertical (3:1) unless substantiating data (e.g., a geotechnical report) justifying steeper slopes is submitted.
- Adequate protection shall be provided for fill slopes exposed to 100-year peak discharges with velocities of five feet per second or less by covering them entirely with grass, vines, or similar vegetative growth.
- Adequate protection shall be provided for fill slopes exposed to 100-year peak discharges with velocities greater than five feet per second by armoring them entirely with stone or rock slope protection or some other acceptable method.
- Fill areas or building pads should extend beyond the outside perimeter of a structure. A minimum of twenty-five (25) feet in all directions is recommended.

Note that a Level 2 approach may not be applicable to all cases. The local floodplain administrator may request that a more detailed Level 3 procedure be used.

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[3] One foot above the 100-year Energy Grade Line if flow is critical or supercritical.

Level 3 Procedures

Level 3 procedures require the estimation of the 100-year discharge (hydrology), and the 100-year floodplain and floodway (hydraulics) using more sophisticated engineering procedures than in Level 1 or Level 2. The Level 3 analyses will generally be more expensive, though an overall cost savings may be realized in drainage structure and/or flood-proofing construction costs. Level 3 documentation will comply with those defined in SSA 1-90 (Instructions for Organizing and Submitting Technical Documentation for Flood Studies) by the Arizona Department of Water Resources.

Methods approved for use in hydrologic analyses include frequency/peak discharge estimation using the computer programs HEC-1 by the Corps of Engineers and TR-55 and TR-20 by the Soil Conservation Service for synthetic peak discharge estimation. Where possible, any synthetic peak discharge estimation techniques should be calibrated to locally observed hydrologic conditions. Where stream gage records are available, flood frequency estimates can be made using statistical analysis. Floodplain and floodway analyses will be conducted using step-backwater methodology. The computer models HEC-2 or HEC-RAS by the Corps of Engineers are preferred. A Level 3 example is not provided in this document.

Table 3. Level 3 Methodology Summary

Information Type	Variable Obtained	Data Required	Acceptable Methodologies
Discharge	100-Year Discharge Flood Hydrograph	Detailed Information Watershed Data Precipitation Data	Computer Models: HEC-1, TR-20, TR-55, others Approved Local Methodologies Flood Frequency fron Gage Data
Floodplain	Water Surface Profile Channel Hydraulics	Surveyed Cross Sections Hydraulic Data	Computer Models: HEC-2, others
Floodway	Floodway Width Channel Hydraulics	Surveyed Cross Sections Hydraulic Data	Computer Models: HEC-2, others

References

Arizona Department of Water Resources, Flood Warning and Dam Safety Section "Requirement for Floodplain Delineation in Riverine Environments - State Standard 2-92 (and Attachment)," September 1992.

Arizona Department of Water Resources, Flood Warning and Dam Safety Section "Requirement for Floodplain and Floodway Delineation in Riverine Floodplains - State Standard Attachment SSA 2-92," January 1996.

Chow, Ven Te, "Open Channel Hydraulics," McGraw-Hill Book Company, New York, 1959.

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Henderson, F.M., "Open Channel Flow," Macmillan Publishing Co., Inc., New York, 1966.

Thomas, B.E., Hjalmarson, H.W., Waltemeyer, S.D., "Methods for Estimating Magnitude and Frequency of Floods in the Southwestern United States," USGS Open File Report 93-419, 1994.

Thomsen, B.W., and Hjalmarson, H.W., 1991, U.S. Geological Survey, Water Resources Division, "Estimated Manning's Roughness Coefficients for Stream Channels and Floodplains in Maricopa County, Arizona," Prepared for the Flood Control District of Maricopa County, April.

APPENDIXES

Appendix A: National Flood Insurance Program

The United States Congress passed the National Flood Insurance Act of 1968 as a first attempt to provide relief for individuals with property in flood-prone areas, and to begin to develop uniform standards for floodplain management. Since 1968, the Act has been amended several times. This Appendix contains passages from the Act wherein the definition of community includes the state.

The National Flood Insurance Act of 1968 was enacted by Title XIII of the Housing and Urban Development Act of 1968 (L. 90-448, August 1, 1968) to provide previously unavailable flood insurance protection to property owners in flood-prone areas. Mudslide protection was added to the Program by the Housing and Urban Development Act of 1969. Flood-related erosion protection was added to the Program by the Flood Disaster Protection Act of 1973 (L. 93-234, December 31, 1973). The Flood Disaster Protection Act of 1973 requires the purchase of flood insurance on and after March 2, 1974, as a condition of receiving any form of Federal or federally-related financial assistance for acquisition or construction purposes with respect to insurable buildings and mobile homes within an identified special flood, mudslide, or flood-related erosion hazard area that is located within any community participating in the Program. The Act also requires that on and after July 1, 1976, or one year after a community has been formally notified by the Administrator of its identification as a community containing one or more special flood, mudslide, or flood-related erosion hazard areas, no such Federal financial assistance, shall be provided within such an area unless the community in which the area is located is then participating in the Program, subject to certain exceptions.

To qualify for the sale of federally-subsidized flood insurance a community must adopt and submit to the Administrator as part of its application, floodplain management regulations, satisfying at a minimum the criteria designed to reduce or avoid future flood, mudslide (i.e., mudflow) or flood-related erosion damages. These regulations must include effective enforcement provisions.

The NFIP has been successful in requiring new buildings to be protected from damage by the 100-year flood. However, the program had few incentives for communities to do more than enforce the minimum regulatory standards. Flood insurance rates had been the same in all participating communities, even though some do much more than regulate construction of new buildings to the national standards.

Until 1990 the program did little to recognize or encourage community activities to reduce flood damages to existing buildings, to manage development in areas not mapped by the NFIP, to protect new buildings beyond the minimum NFIP protection level, to help insurance agents obtain flood data, or to help people obtain flood insurance. Because these activities can have a great impact on the insurance premium base, flood damages, flood insurance claims, and federal disaster assistance payments, the Federal Insurance Administration (FIA) has implemented the Community Rating System (CRS). The deadline for the first applications to participate in the CRS program were due to FEMA Region IX offices by December 5, 1990.

Flood insurance premium credits are available in communities based on their CRS classification. There are ten classes with Class 1 having the greatest premium credit and Class 10 having no premium credit. A community's CRS class is based on the number of credit points calculated for the activities that are undertaken to reduce flood losses, facilitate accurate insurance rating, and promote the awareness of flood insurance. A community is automatically in Class 10 unless it applies for CRS classification and it shows that the activities it is implementing warrant a better class. The amount of premium credit for each class is published annually by the Flood Insurance Administration. The CRS rewards those communities that are doing more than the minimum NFIP requirements which encourage their residents to prevent or reduce flood losses. The system also provides an incentive for communities to initiate new flood protection activities.

Appendix B: Arizona Department of Water Resources

In 1973, the Arizona Legislature required the Arizona Water Commission (now the Arizona Department of Water Resources) to develop and adopt criteria for the 50- and 100-year floods for use by the Arizona communities for the purpose of floodplain management. In response, the Water Commission published Floodplain Delineation Criteria and Procedures, Report Number Four in October 1973.

In 1979, the Governor designated the Arizona Water Commission as the State Coordinating Agency for the National Flood Insurance Program (NFIP). In 1980, the Legislature created the Arizona Department of Water Resources (ADWR). The State NFIP responsibility was then shifted to the ADWR. The State Statutes do not spell out any specific duties for the coordinating agency, although the Water Commission/ADWR has had certain responsibilities for floodplain management since 1973.

The Arizona Legislature added a specific requirement for ADWR to develop and adopt criteria for floodplain delineation throughout the state under ARS Titles 45 and 48, in 1984. This requirement has led the Department to review, revise and supplement the criteria established in 1973. The National Flood Insurance Act as amended in 1986 lists 12 duties and responsibilities for the state:

1. Enact enabling legislation in floodplain management. The Legislature adopted such legislation in 1973 and has amended it as needed.

2. Encourage and assist communities in qualifying for participation in the NFIP. <u>All Arizona communities with flood prone areas are participating in the NFIP</u>.

3. Assist communities in the adoption of ordinances. The ADWR staff works continually with communities to keep their ordinances up-to-date with the NFIP and the State Statutes.

4. Provide communities and the public with information on floodplain management. ADWR staff works with the public and communities on an ongoing basis. A Community Assistance Handbook and a quarterly newsletter are two of the methods used. ADWR staff also meet with community officials and speak at public meetings.

5. Assist communities in disseminating elevation requirements for flood-prone areas. Due to limited staff, ADWR refers most public requests for information to the communities. ADWR staff assists communities in obtaining information and understanding it so that they may respond effectively to public requests.

6. Assist in the delineation of flood-prone areas. ADWR has delineated floodplains and contributed financially to such delineations. Staff reviews delineations performed by others.

7. Recommend priorities for Federal floodplain management activities within the state. ADWR has worked with a number of Federal agencies on priorities.

8. Notify FIA of community failures in floodplain management. ADWR works with communities to correct deficiencies in their programs. In extreme cases, staff will notify FIA of problems.

9. Establish state floodplain management standards. Current State Statutory requirements equal or exceed the minimum FIA requirements.

10. Assure coordination and consistency of floodplain management activities with other agencies. ADWR meets with other agencies as necessary to coordinate activities.

11. Assist in the identification and implementation of flood hazard mitigation recommendations. ADWR has several mitigation functions and works with other agencies as necessary to optimize mitigation opportunities.

12. Participate in floodplain management training activities. ADWR staff support quarterly workshops for community staff and others on floodplain management and assist in training when opportunities arise.

Appendix C: 100-Year Floodplain and Floodway Standards

The 100-Year Floodplain

Throughout the United States the standard for floodplain management is the 100-year flood or peak discharge. The 100-year flood is a flood with a one percent chance of being equaled or exceeded in any given year. Since there is seldom enough data to exactly define the 100-year flood at a particular location, the value is estimated from existing records using statistical and/or empirical hydrologic engineering methods. Inherent in the estimating procedure is the risk that as additional data becomes available previous estimates may require revision. Also, peak discharge estimates often assume that weather characteristics remain constant and that the watershed and channel characteristics remain the same during the entire period of record.

The FIA and FEMA have adopted the 100-year flood as the national standard for floodplain management and floodplain study purposes. The 100-year flood is also referred to as the regulatory flood or base flood. In addition to floodplain studies, the 100-year flood also has been used as the level of protection for the design of many drainage structures. Primary considerations in determining the level of flood protection necessary are health and safety, acceptable risk, and cost. Flood control projects such as dams and emergency spillways which provide protection to critical downstream or adjacent developments, are sometimes designed to a much higher standard (i.e., the 250-year, 1,000-year, or Probable Maximum Flood). Storm drains for street drainage may be designed to a much lower standard for cost saving reasons, and when the capacity of the storm drain is exceeded the excess storm water may cause flooding.

While the mandated standard of the 100-year flood for floodplain management can be debated, the concept is sound and a uniform standard must be used. The Federal Office of Management and Budget re-evaluated the 100-year flood standard for the National Flood Insurance Program in the early 1980's and found no reason to change. It is anticipated that none of the criteria presently used by Federal Emergency Management Agency will change in the near future.

FEMA criteria and the Arizona Revised Statutes require that all residences and occupied structures must be constructed so that their lowest floor is a minimum of one-foot above the 100-year water surface elevation of the 100-year flood. Local floodplain regulation standards must meet the minimum federal and state standards. However, a community may adopt stricter local floodplain regulations if they wish. Several communities in Arizona have adopted more stringent floodplain regulations.

The 100-Year Floodway

The FEMA floodway standard is essential for the success of floodplain management. Any development in a floodplain which obstructs the flow of water generally causes the water surface elevation to be higher across the rest of the floodplain. Limitations on floodplain encroachment are necessary to help reduce adverse impacts from new development in floodways on existing structures. Under the Arizona Revised Statutes and the National Flood Insurance Program, floodplain encroachment is allowed only to the extent that it causes no more than a one foot rise in the 100-year water surface elevation when considered across the entire floodplain. The remaining unencroached area is reserved for conveyance of the 100-year flood and is referred to as the regulatory floodway. Once a regulatory floodway is established, no further development is allowed within this special conveyance area without approval of the local community and FEMA. Technical data which supports the floodway revision must be provided. A community may adopt stricter floodway regulations if they wish. Several communities throughout Arizona and the U.S. have adopted regulations which require that floodway encroachments raise the natural water surface elevation less than the one foot FEMA criteria (e.g., one-tenth foot, one-half foot).

Appendix D: Level 1 Peak Discharge Estimate - Example

100-Year Discharge Estimate

Level 1 discharge estimates may be obtained from the envelope curve shown in Figure D-1. The envelope curve is drawn above the largest discharges in Arizona and the Southwest gaged by the U.S. Geological Survey. To estimate the 100-year discharge at proposed development, measure the watershed area on a USGS topographic quadrangle map in square miles. Plot the measured watershed area on the curve and read the corresponding peak discharge estimate.

Discharge Example #D1: Estimate the 100-year peak discharge for a proposed development in Cochise County on Double Dry Creek.

<u>STEP 1</u>: Measure the watershed area. The watershed area at the site is measured on a USGS topographic quadrangle map at 17 square miles.

<u>STEP 2</u>: Compute the 100-year peak discharge using Figure D-1. A = 17 sq. mi.

$$Q100 = 31{,}000 \text{ cfs}$$

Results: The Level 1 methodology indicates that the 100-year peak discharge is 31,000 cfs for the example site[1]. In all cases, the floodplain administrator should review the discharge estimate to determine if the Level 1 estimate is appropriate, and if a Level 2 or Level 3 analysis is warranted.

[1] Note that application of Level 2 procedures for this example would indicate a 100-year discharge estimate of about 6,200 cfs.

Figure D-1

LEVEL 1 ENVELOPE CURVE

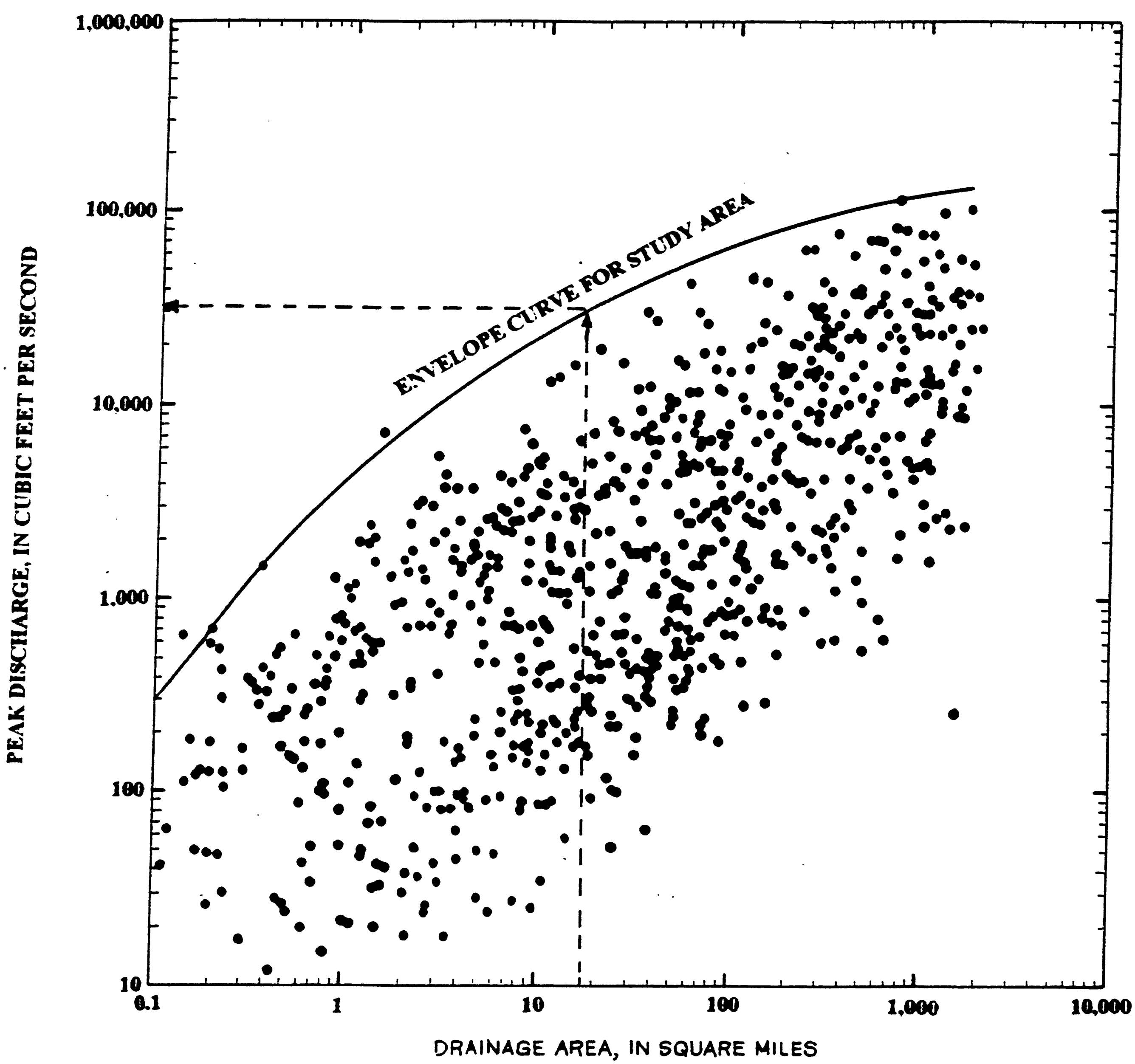


Figure modified from Thomas, B.E. and others, 1994, Methods for Estimating Magnitude & Frequency of Floods in the Southwestern United States. USGS Open File Report 93-419. Figure #17.

Appendix E: Level 1 Floodplain Limits - Equations and Example

Floodplain Depth Estimation

Three depth equation regions are presented for use in Figure E-1. Flood depth estimating equations are presented below for each region shown on Figure E-1. For areas not included in one of the regions described below and shown in Figure E-1, contact the Arizona Department of Water Resources for guidance.

Flood depth = Y (ft.)
Drainage Area = A (sq. mi.)

Region I-D. Encompasses the area north of the Mogollon Rim, including the upper Verde River Basin, excluding the Little Colorado River at and below Woodruff.

$$Y = 5.47 \times A^{0.213}$$

Region II-D. Encompasses the area within Apache, Cochise, Coconino, Gila, Graham, Greenlee, Maricopa, Mohave and Yavapai Counties, except above the Mogollon Rim.

$$Y = 9.89 \times A^{0.132}$$

Region III-D. Encompasses the area within LaPaz, Pima, Pinal, Santa Cruz and Yuma Counties, except the Colorado River.

$$Y = 7.62 \times A^{0.118}$$

Floodway Width Example #E1: Estimate the floodplain elevation for a proposed development in Cochise County on Double Dry Creek. The drainage area at the site is measured from a USGS topographic quadrangle map at 17 square miles.

STEP 1: Determine Flood Depth Region on Figure E-1.Cochise County is in Region II-D.

STEP 2: Compute flood depth (y) from equation II-D. A = 17 sq. mi.

$$Y = 9.89 \times (17)^{0.132} = 14 \text{ feet}$$

STEP 3: Estimate Base Flood Elevation (BFE) at Y + 1 ft. = 15 feet.

Results: The Level 1 methodology indicates that the lowest finished floor of the proposed development should be at least 15 feet above the bottom of the adjacent wash as illustrated in Figure E-2. The floodplain administrator should review the floodplain depth estimate to determine if a Level 2 or Level 3 analysis is warranted.

Figure E-1

MAP OF REGIONS FOR FLOOD DEPTH FORMULAE

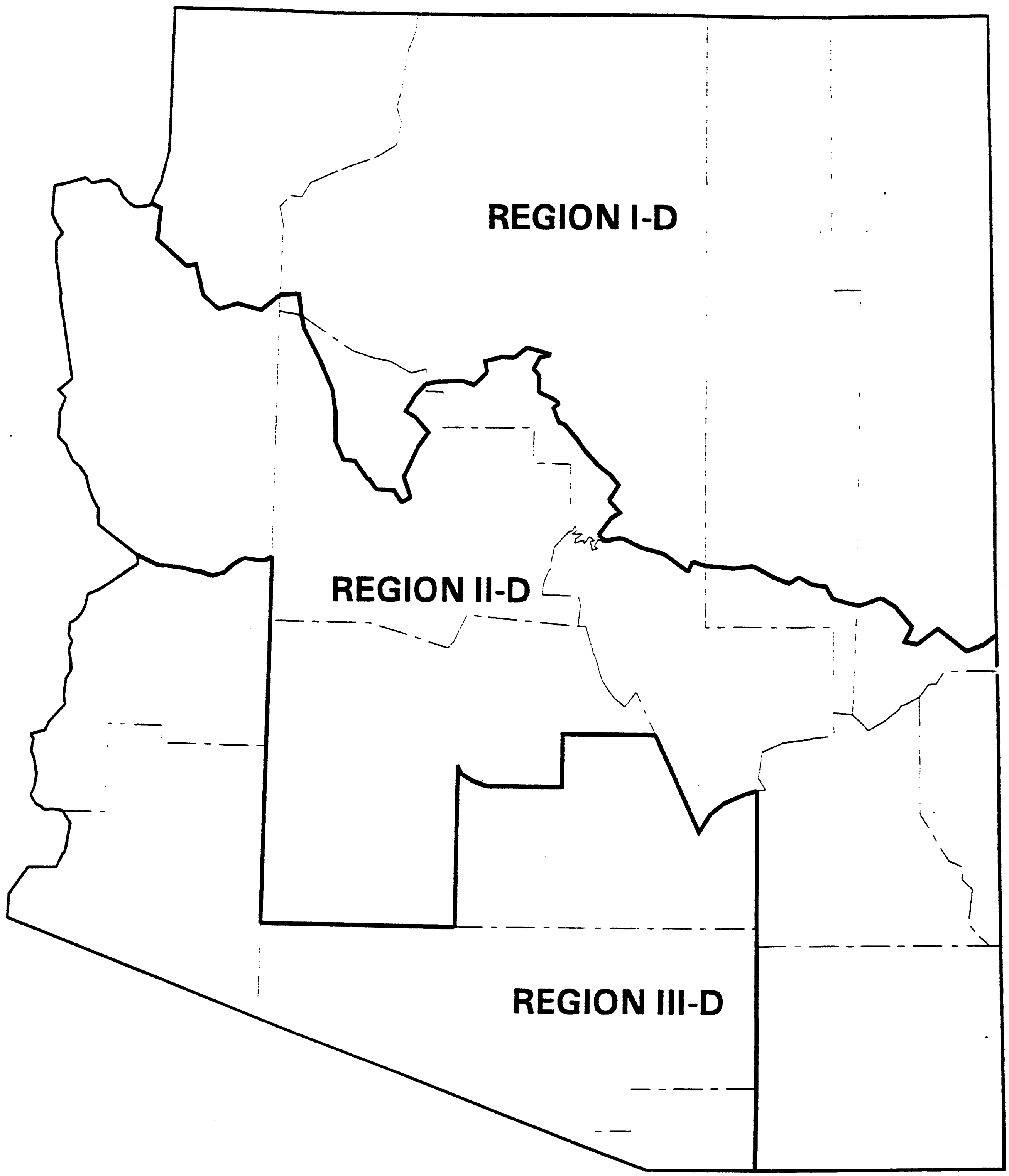
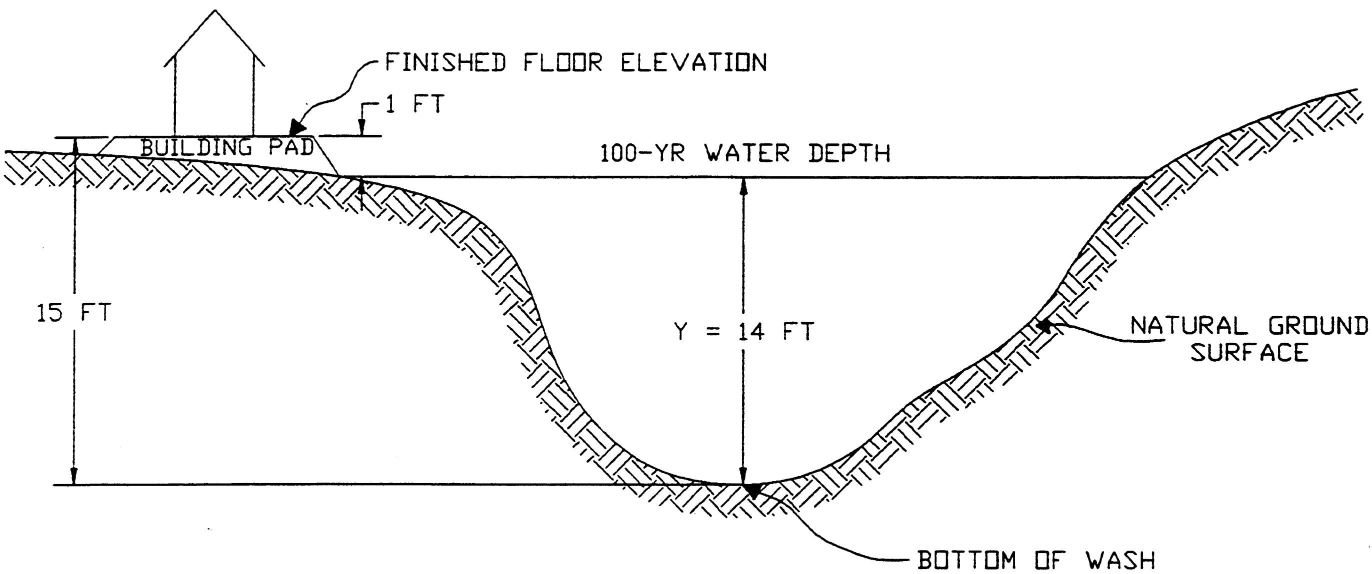


FIGURE E-2 FLOOD DEPTH EXAMPLE #E1 (N.T.S.)



CROSS-SECTIONAL VIEW OF SITE #E1

Appendix F: Level 1 Floodway Width - Equations and Examples

Four floodway-width equation regions are presented for use in Figure F-1. Floodway width estimating equations are presented below for each region shown on Figure F-1. For areas not included in one the regions described below and shown in Figure F-1, contact the Arizona Department of Water Resources for guidance.

Floodway Width = FW (ft.)
Drainage Area = A (sq. mi.)

Region I-W. Encompasses the area north of the Mogollon Rim, including the Arizona Strip north of the Grand Canyon, and the Verde River watershed upstream of Sycamore Creek near Perkinsville:

$$FW = 105 \times A^{0.449}$$

Region II-W. Encompasses the area within Apache, Gila, Graham, Greenlee, LaPaz, Mohave and Yuma Counties below the Mogollon Rim.

$$FW = 157 \times A^{0.407}$$

Region III-W. Encompasses the area within portions of Cochise, Coconino, Santa Cruz Counties and Yavapai County below the Mogollon Rim and in the Verde River Basin below Sycamore Creek near Perkinsville.

$$FW = 218 \times A^{0.261}$$

Region IV-W. Encompasses the area within Maricopa, Pima and Pinal Counties.

$$FW = 377 \times A^{0.289}$$

Floodway Width Example #F1: Estimate the floodway set-back requirement for a proposed single lot development in Cochise County on Double Dry Creek. The drainage area at the site is measured from a USGS topographic quadrangle map at 17 square miles.

<u>STEP 1</u>: Determine Floodway Region on Figure F-1. Cochise County is in Region III-W.

<u>STEP 2</u>: Compute flood depth (y) from equation III-W. A = 17 sq. mi.

$$FW = 218 \text{ X } (17)^{0.261} = 457 \text{ ft.}$$

STEP 3: Estimate floodway setback at ½ the estimated floodway width = 228.5 ft.

Results: The Level 1 methodology indicates that the structure should be located a minimum of 457 feet/2, or 228.5 feet, from the center of the wash, as illustrated in Figure F-2. The setback is equal to half the floodway width since the floodway extends on both sides of the wash. The local floodplain administrator should review the floodway width estimate to determine if the Level 1 estimate is appropriate, or if a Level 2 or Level 3 analysis is warranted.

Figure F-1

MAP OF REGIONS FOR FLOODWAY WIDTH FORMULAE

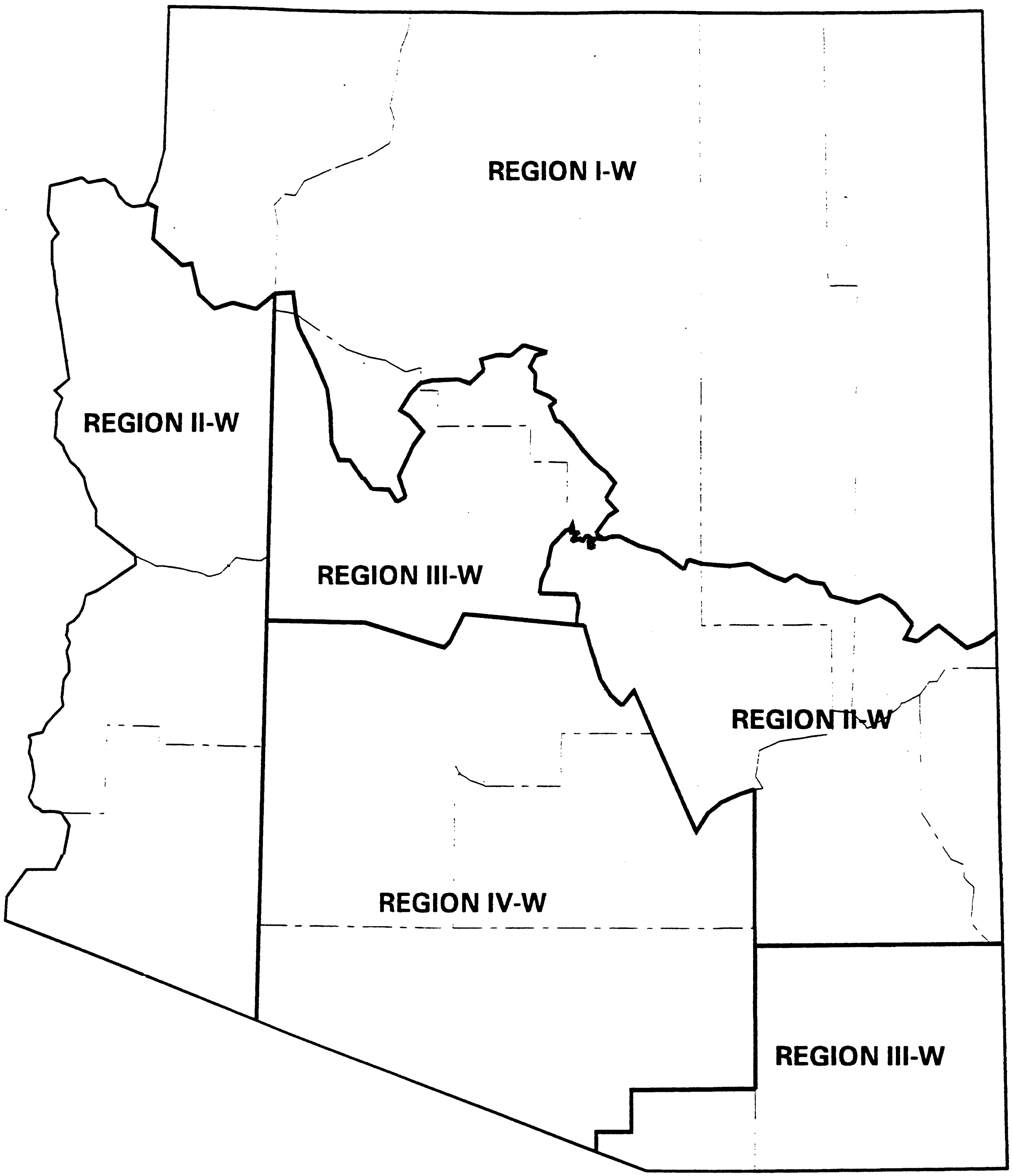
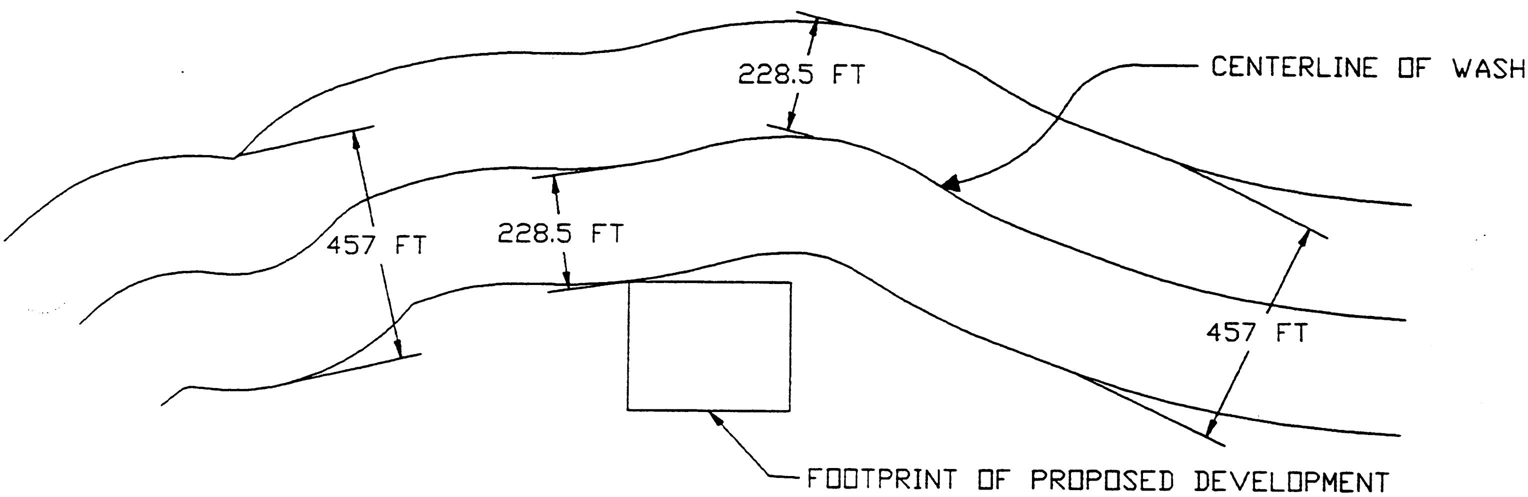


FIGURE F-2 FLOODWAY WIDTH EXAMPLE #F1 (N.T.S.)



FLOODWAY WIDTH = 457 FT
SETBACK FROM CENTERLINE OF WASH = 228.5 FT

Appendix G. Level 2 - Estimating Peak Discharges on Ungaged Rural Watersheds

Overview of Methodology

Equations for estimating peak discharges for ungaged watersheds in Arizona and the Southwest were developed by the US Geological Survey using stream gage records and regression analyses and a newly developed statistical procedure for arid regions, the hybrid method. Unique equations were developed for each of seven regions within Arizona, including a region for watersheds at high elevation (> 7,500 feet). Required information includes the watershed area and up to one of the following: (1) mean annual precipitation, (2) mean elevation, or (3) mean annual evaporation. Figures G-2 to G-3 show some of these required data for the entire State of Arizona. A detailed description of the procedures and numerous examples are provided by the USGS in Thomas et. al., 1994.

Step-by-Step Procedures

Step 1 Locate watershed on region map (Figure G-1)

Step 2 Select appropriate regional equations (Tables G-1 to G-7)

Step 3 Determine required input parameters for region (Tables G-1 to G-7)

a. Determine watershed area (A, square miles)
b. Estimate mean annual precipitation (P, inches) Region 1, Figure G-2
c. Estimate mean annual evaporation (EV, inches) Region 11, Figure G-3
d. Estimate mean elevation of watershed (EL, feet) Regions 8,14,12,

Step 4 Check if watershed is within "Cloud of Common Values[2]" (Figures G-4 to G-8)

Step 5 Apply equations to obtain discharge estimates

a. Watershed in one region (See Example #G1)
b. Watershed elevation above 6,750 feet? (See Example #G2)
c. Watershed located within two adjacent regions? (See Example #G3)

Limitations:

1. Methodology generates discharge *estimates*, NOTE error range given (Tables G1-G7).

2. Watersheds characteristics analyzed should fall within the range of data used to develop the equations. Watersheds with values outside these data ranges may have higher standard error than indicated in Tables G-1 to G-7.

---

[2] There is no cloud of common values for Regions 10 and 13 because only drainage area is required for the recommended procedure.

3. The equations may not be appropriate for the following watershed types:

- Urban Areas[3]
- Alluvial Fan/Distributary Flow/Sheet Flow Areas[4]
- Agricultural Areas with flood irrigation structures[4]
- Areas with highly permeable bedrock or cinders[4]
- Areas with large dams or diversions[4]

For the watershed types above use the Level 1 or 3 discharge methodology.

Table G-1. Region 1 Equations		
Recurrence Interval	**Equation**	**Average Standard Error (%)**
2	$Q = 0.124\ A^{0.845}\ P^{1.44}$	59
5	$Q = 0.629\ A^{0.807}\ P^{1.12}$	52
10	$Q = 1.43\ A^{0.786}\ P^{0.958}$	48
25	$Q = 3.08\ A^{0.768}\ P^{0.811}$	46
50	$Q = 4.75\ A^{0.758}\ P^{0.732}$	46
100	$Q = 6.78\ A^{0.750}\ P^{0.668}$	46
Q = discharge, cfs A = drainage area, sq. miles P = mean annual precipitation, inches		

Table G-2. Region 8 Equations		
Recurrence Interval	**Equation**	**Average Standard Error (%)**
2	$Q = 598\ A^{0.501}\ EL^{-1.02}$	72
5	$Q = 2620\ A^{0.449}\ EL^{-1.28}$	62
10	$Q = 5310\ A^{0.425}\ EL^{-1.40}$	57
25	$Q = 10500\ A^{0.403}\ EL^{-1.49}$	54
50	$Q = 16000\ A^{0.390}\ EL^{-1.54}$	53
100	$Q = 23300\ A^{0.377}\ EL^{-1.59}$	53
NOTE: EL = mean elevation in watershed/1000. See Thomas et. al., 1994 for procedure for estimating elevation.		

[3] The recommended equations will tend to *underestimate* peak discharges.

[4] The recommended equations will tend to *overestimate* peak discharges.

Table G-3. Region 10 Equations		
Recurrence Interval	**Equation**	**Average Standard Error (log units)**
2	$Q = 12\ A^{0.58}$	1.14
5	$Q = 85\ A^{0.59}$	0.602
10	$Q = 200\ A^{0.62}$	0.675
25	$Q = 400\ A^{0.65}$	0.949
50	$Q = 590\ A^{0.67}$	0.928
100	$Q = 850\ A^{0.69}$	1.23`

Q = discharge, cfs
A = drainage area, sq. miles

Table G-4. Region 11 Equations		
Recurrence Interval	**Equation**	**Average Standard Error (log units)**
2	$Q = 26\ A^{0.62}$	0.609
5	$Q = 130\ A^{0.56}$	0.309
10	$Q = 0.10\ A^{0.52}\ EV^{2.0}$	0.296
25	$Q = 0.17\ A^{0.52}\ EV^{2.0}$	0.191
50	$Q = 0.24\ A^{0.54}\ EV^{2.0}$	0.294
100	$Q = 0.27\ A^{0.58}\ EV^{2.0}$	0.863

Q = discharge, cfs
A = drainage area, sq. miles
EV = mean annual evaporation, inches

Table G-5. Region 12 Equations		
Recurrence Interval	**Equation**	**Average Standard Error (%)**
2	$Q = 41.1\ A^{0.629}$	105
5	$Q = 238\ A^{0.687}\ EL^{-0.358}$	68
10	$Q = 479\ A^{0.661}\ EL^{-0.398}$	52
25	$Q = 942\ A^{0.630}\ EL^{-0.383}$	40
50	$Q = 10^{(7.36-4.17\ A\text{^}(-0.08))}\ (EL)^{-0.440}$	37
100	$Q = 10^{(6.55-3.17\ A\text{^}(-0.11))}\ (EL)^{-0.454}$	39

Q = discharge, cfs
A = drainage area, sq. miles
NOTE: EL = mean elevation in watershed/1000. See Thomas et. al., 1994 for procedure for estimating elevation.

Table G-6. Region 13 Equations		
Recurrence Interval	**Equation**	**Average Standard Error (%)**
2	$Q = 10^{(6.38-4.29\ A^{\wedge}(-0.06))}$	57
5	$Q = 10^{(5.78-3.31\ A^{\wedge}(-0.08))}$	40
10	$Q = 10^{(5.68-3.02\ A^{\wedge}(-0.09))}$	37
25	$Q = 10^{(5.64-2.78\ A^{\wedge}(-0.10))}$	39
50	$Q = 10^{(5.57-2.59\ A^{\wedge}(-0.11))}$	43
100	$Q = 10^{(5.52-2.42\ A^{\wedge}(-0.12))}$	48

Q = discharge, cfs
A = drainage area, sq. miles

Table G-7. Region 14 Equations		
Recurrence Interval	**Equation**	**Average Standard Error (%)**
2	$Q = 583\ A^{0.588}\ EL^{-1.3}$	74
5	$Q = 618\ A^{0.524}\ EL^{-0.70}$	63
10	$Q = 361\ A^{0.464}$	65
25	$Q = 581\ A^{0.462}$	63
50	$Q = 779\ A^{0.462}$	64
100	$Q = 1010\ A^{0.463}$	66

Q = discharge, cfs
A = drainage area, sq. miles
NOTE: EL = mean elevation in watershed/1000. See Thomas et. al., 1994 for procedure for estimating elevation.

FLOOD REGIONS IN ARIZONA

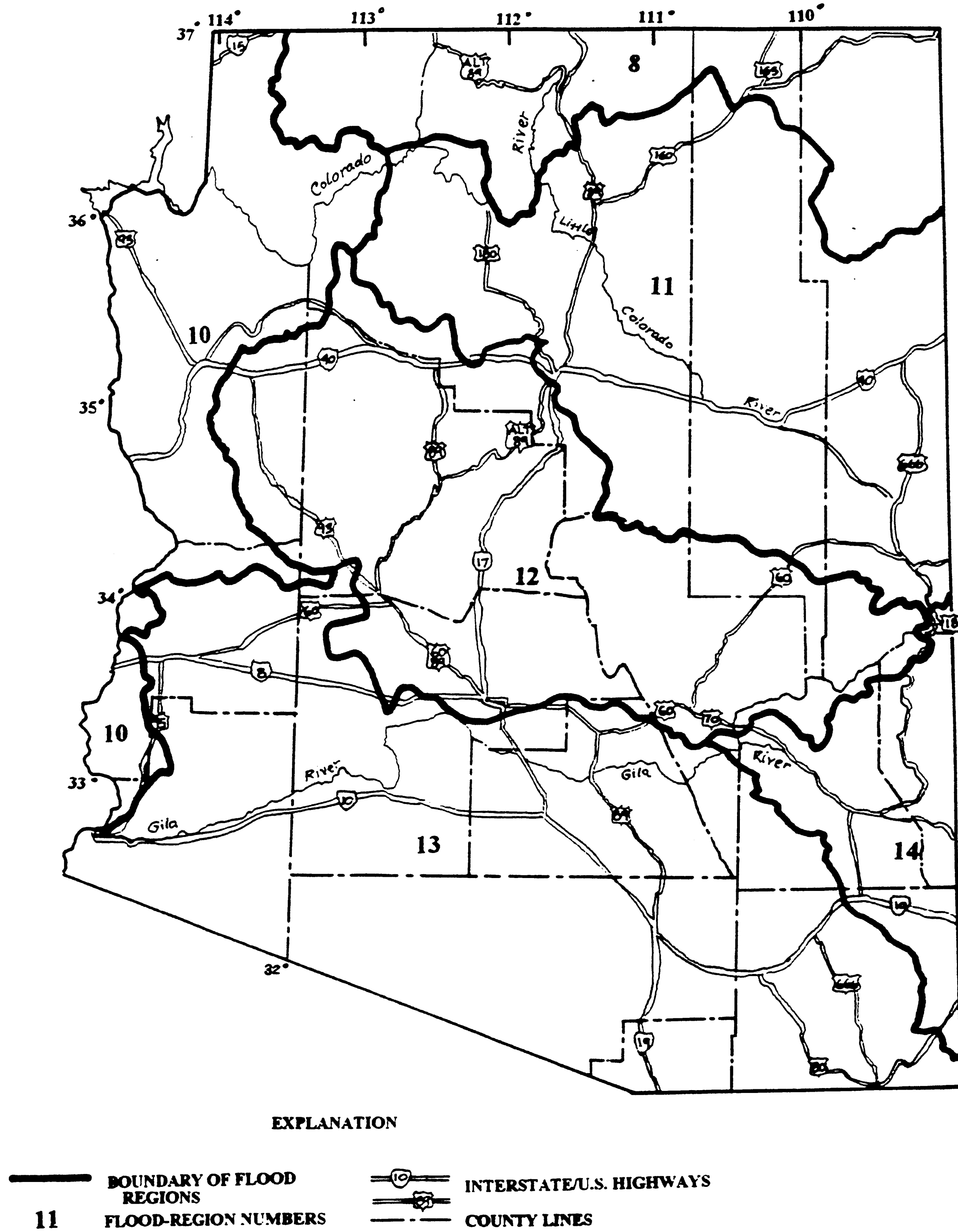
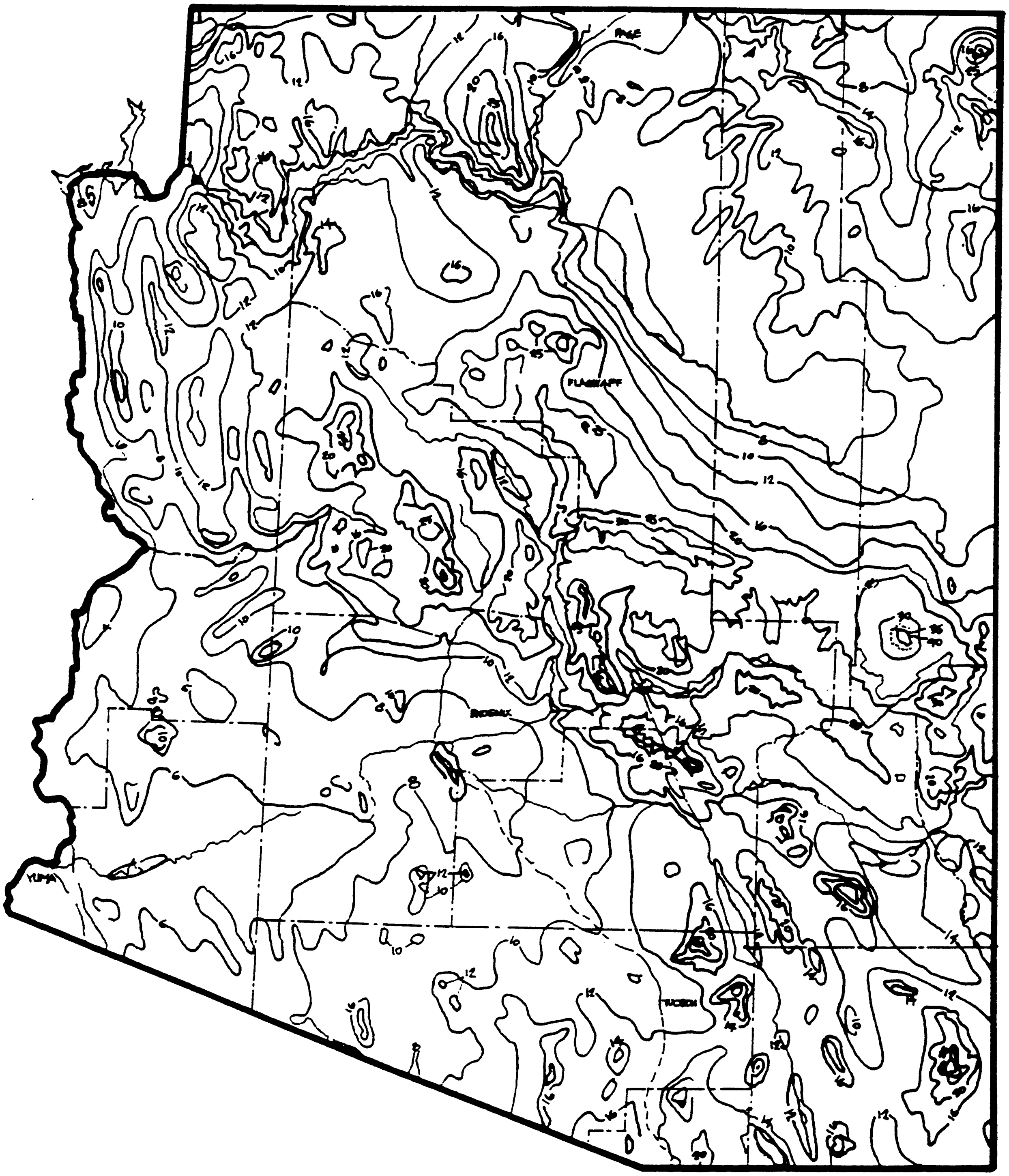


Figure modified from Thomas, B.E. and others, 1994, Methods for Estimating Magnitude & Frequency of Floods in the Southwestern United States, USGS Open File Report 93-419. Figure # 7.

Figure G-2 **MEAN ANNUAL PRECIPITATION (PREC.), 1931-1960**

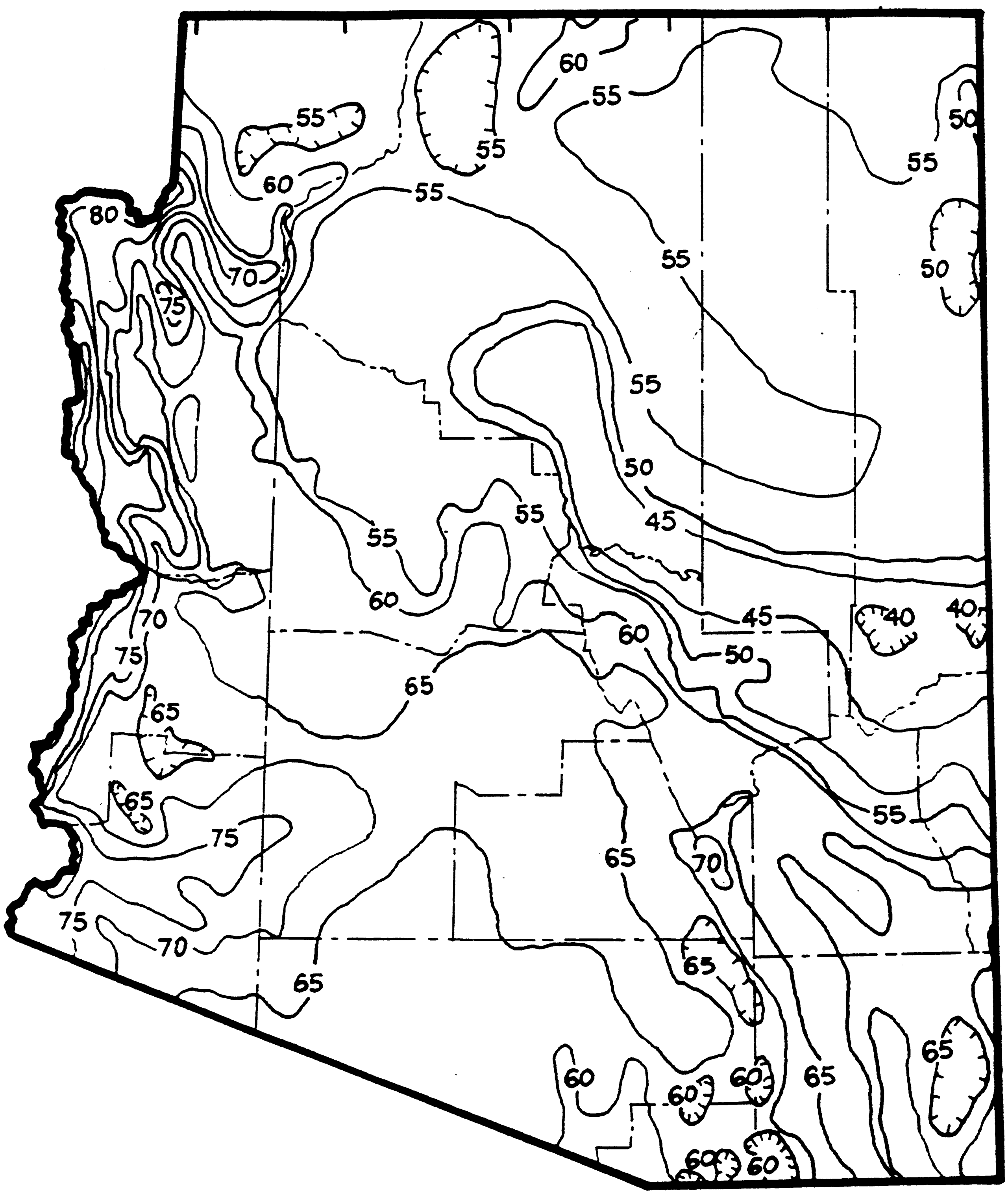


— 65 — **Mean Annual Precipitation, in inches**

Figure modified from ADOT, 1993, Highway Drainage Design Manual--Hydrology. Figure #10-10.

Figure G-3

MEAN ANNUAL EVAPORATION (EVAP)



— 65 — **Mean Annual Evaporation, in inches**

Figure modified from ADOT, 1993, Highway Drainage Design Manual--Hydrology. Figure #10-11.

Figure G-4 **Joint distribution of mean annual precipitation and drainage area for gaged sites in the High-Elevation Region 1.**

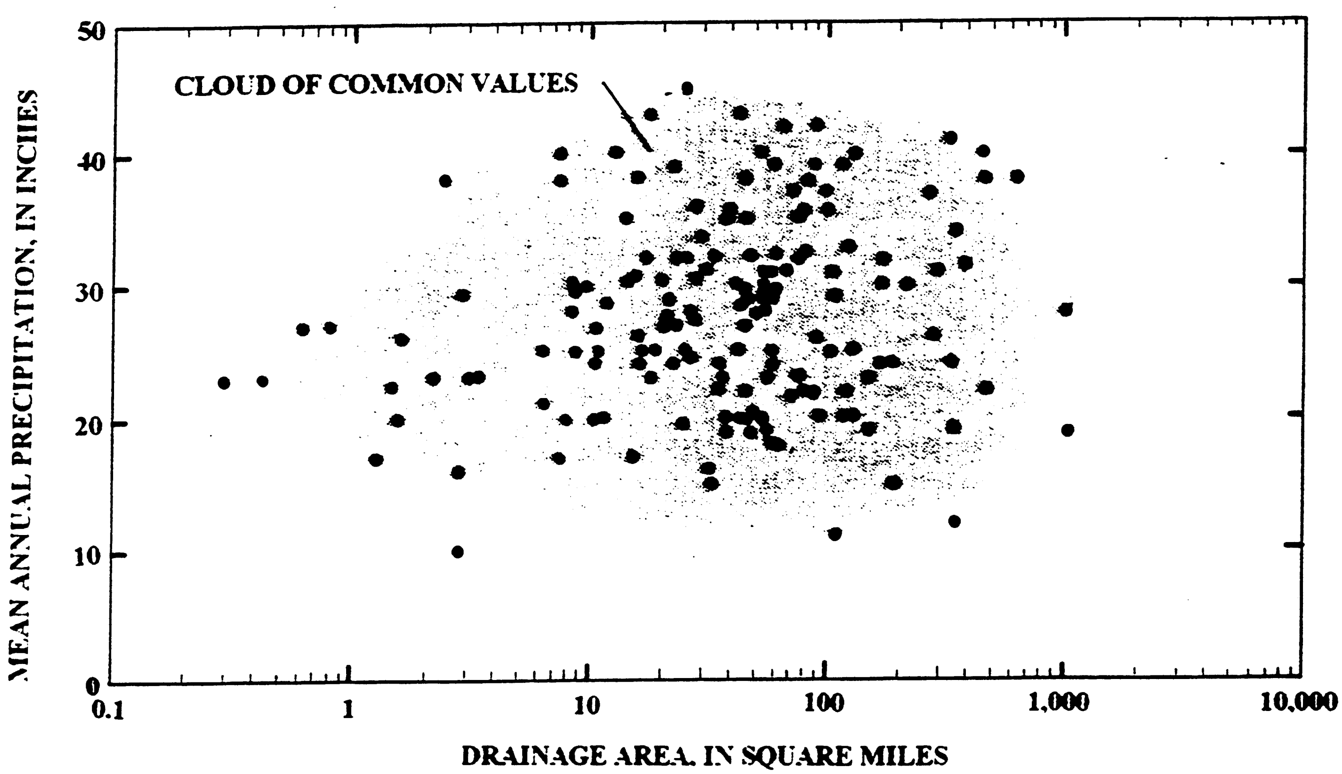


Figure modified from Thomas, B.E. and others, 1994, Methods for Estimating Magnitude & Frequency of Floods in the Southwestern United States. USGS Open File Report 93-419. Figure # 18.

Figure G-5 **Joint distribution of mean basin elevation and drainage area for gaged sites in the Four Corners Region 8.**

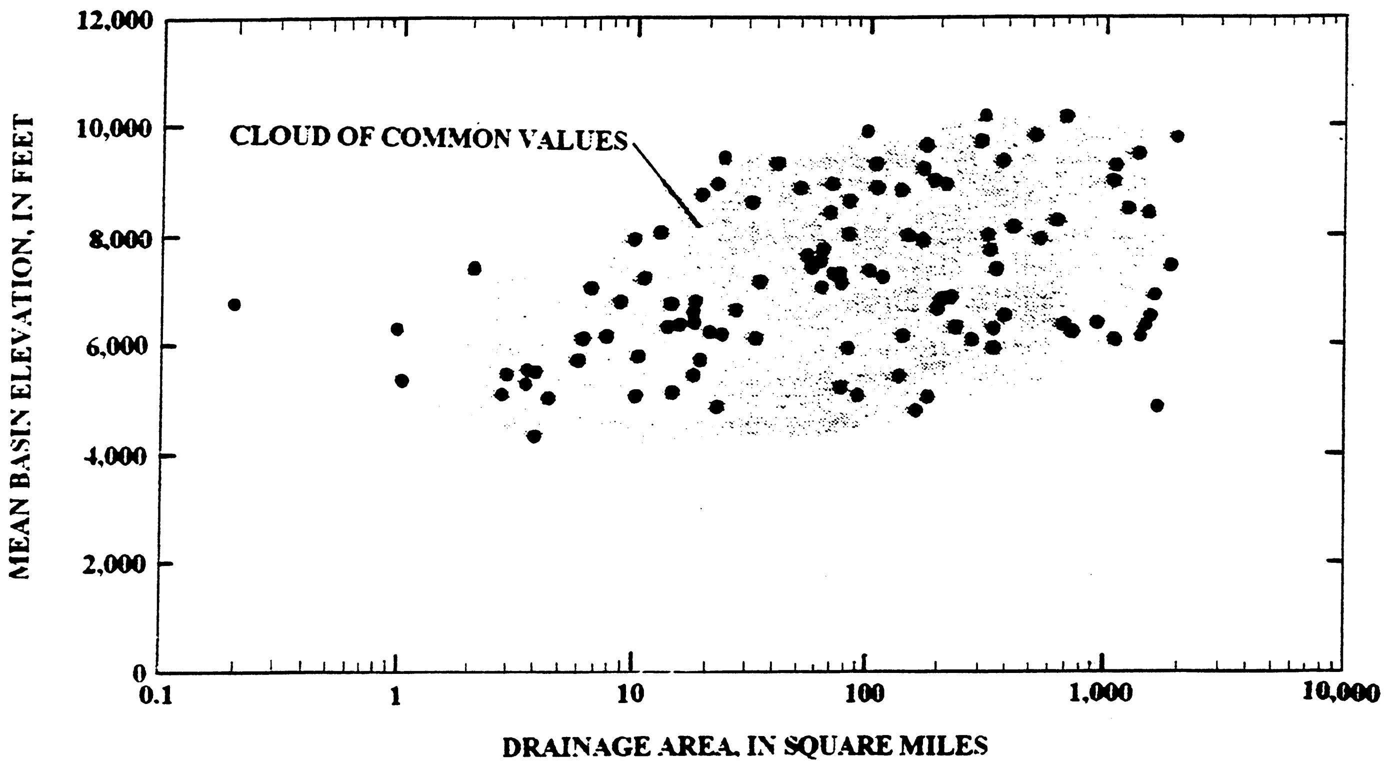


Figure modified from Thomas, B.E. and others, 1994, Methods for Estimating Magnitude & Frequency of Floods in the Southwestern United States. USGS Open File Report 93-419. Figure # 33.

Figure G-6 **Joint distribution of mean annual evaporation and drainage area for gaged sites in the Northeastern Arizona Region 11.**

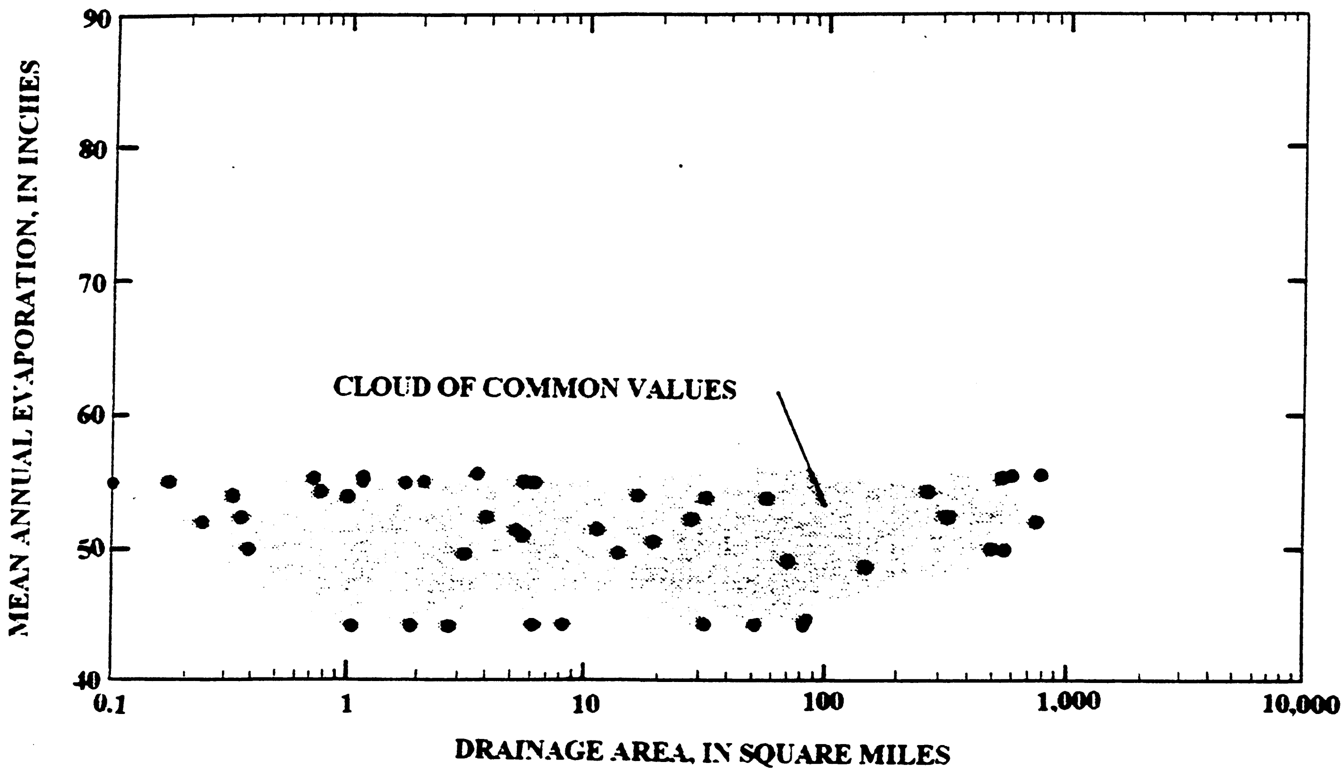


Figure modified from Thomas, B.E. and others, 1994, Methods for Estimating Magnitude & Frequency of Floods in the Southwestern United States. USGS Open File Report 93-419. Figure # 36.

Figure G-7 **Joint distribution of mean basin elevation and drainage area for gaged sites in the Central Arizona Region 12.**

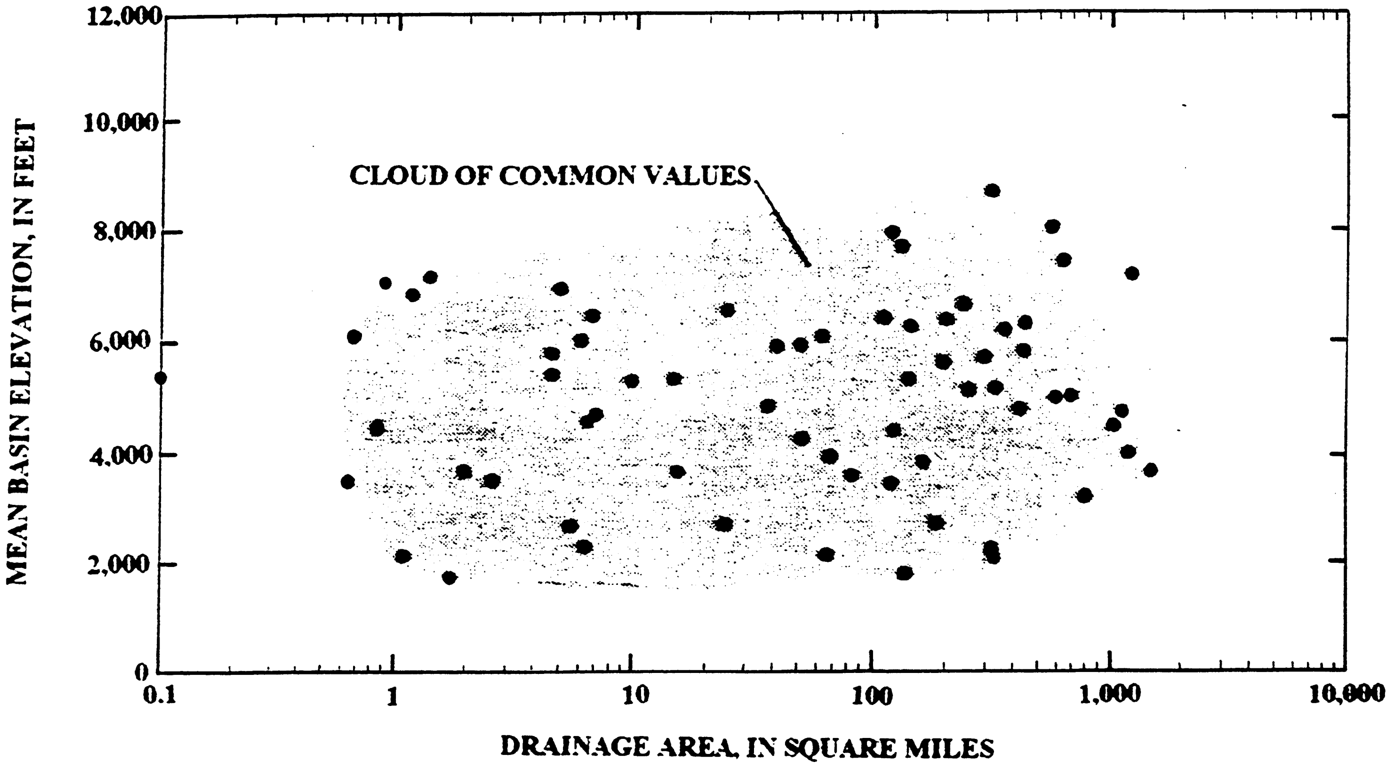


Figure modified from Thomas, B.E. and others, 1994, Methods for Estimating Magnitude & Frequency of Floods in the Southwestern United States. -USGS Open File Report 93-419. Figure # 40.

Figure G-8 **Joint distribution of mean basin elevation and drainage area for gaged sites in the Upper Gila Basin Region 14.**

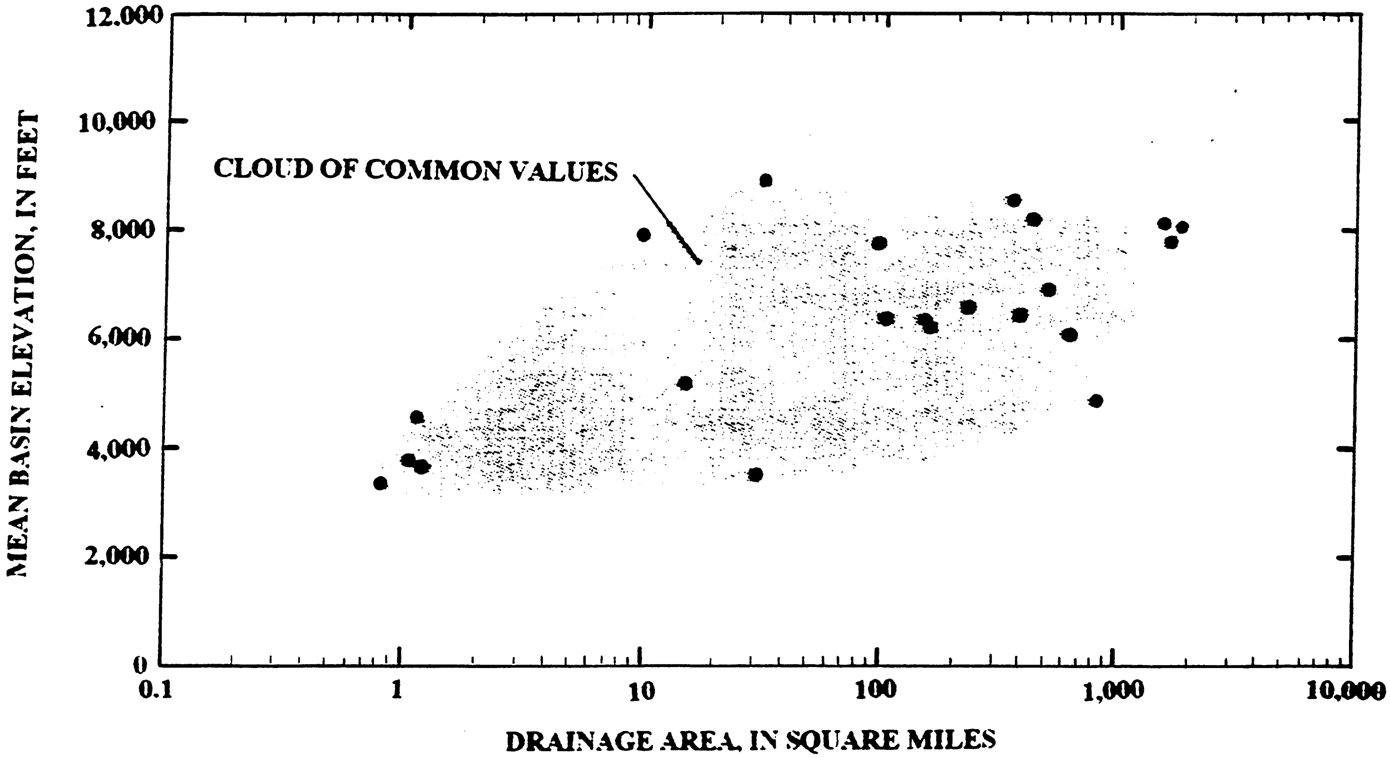


Figure modified from Thomas, B.E. and others, 1994, Methods for Estimating Magnitude & Frequency of Floods in the Southwestern United States. USGS Open File Report 93-419. Figure # 43.

Example #G1. Rural Watershed Within One Flood Region.

Estimate peak discharges for recurrence intervals of 50 and 100 years (Q_{50} and Q_{100}) for an ungaged site in Central Arizona Region 12 (Figures G-1 and G-7, Table G-5). The required basin characteristics are drainage area (A), in square miles, and mean basin elevation (EL), in feet. The drainage area was planimetered from a USGS topographic map, and measures 110 mi^2, and the mean basin elevation is 5,900 ft. The drainage area and mean elevation are within the cloud of common values for the region (Figure G-7).

The characteristics are inserted into the appropriate equations as follows:

$$Q_{50} = 10^{(7.36-4.17\,A^\wedge(-0.08))}\,(EL)^{-0.440}$$

$$Q_{50} = 10^{(7.36-4.17\,(110)^\wedge(-0.08))}\,(5.90)^{-0.440}$$

$$Q_{50} = 14{,}381 \text{ cfs} = 14{,}400 \text{ cfs}$$

and

$$Q_{100} = 10^{(6.55-3.17\,A^\wedge(-0.11))}\,(EL)^{-0.454}$$

$$Q_{100} = 10^{(6.55-3.17\,(110)^\wedge(-0.11))}\,(5.90)^{-0.454}$$

$$Q_{100} = 20{,}410 \text{ cfs} = 20{,}400 \text{ cfs}$$

Example #G2. Rural Watershed Within Two Flood Regions.

For watersheds that lie within two or more regions, an averaging procedure based on the percentage of area in each region may be used to determine a peak discharge estimate. The peak discharges are estimated for each region as if the drainage area is entirely in one region. Then, a weighted peak discharge is estimated using the procedures illustrated below.

A hypothetical study site has a drainage area that lies within Southern Great Basin Region 10 and Southern Arizona Region 13 (Figure G-1). Estimate the peak discharge for recurrence intervals of 10 and 100 years. The only required basin characteristic is drainage area (A), in square miles, measured at 57 mi^2 by planimetering from high altitude aerial photographs of known scale. The drainage area falls within the range of data for Regions 10 and 13[5]. The watershed is divided so that 36 mi^2 falls within Region 10, and 21 mi^2 is located within Region 13.

For Region 10:

$$Q_{10} = 200\ A^{0.62}$$
$$Q_{10} = 200\ (57)^{0.62} = 2{,}453 \text{ cfs}$$
$$Q_{10} = 2{,}450 \text{ cfs}$$

$$Q_{100} = 850\ A^{0.69}$$
$$Q_{100} = 850\ (57)^{0.69} = 13{,}835 \text{ cfs}$$
$$Q_{100} = 13{,}800 \text{ cfs}$$

For Region 13:

$$Q_{10} = 10^{(5.68-3.02\ A^\wedge(-0.09))}$$
$$Q_{10} = 10^{(5.68-3.02\ (57)^\wedge(-0.09))} = 3{,}812 \text{ cfs}$$
$$Q_{10} = 3{,}810 \text{ cfs}$$

$$Q_{100} = 10^{(5.52-2.42\ A^\wedge(-0.12))}$$
$$Q_{100} = 10^{(5.52-2.42\ (57)^\wedge(-0.12))} = 10{,}722 \text{ cfs}$$
$$Q_{100} = 10{,}700 \text{ cfs}$$

Weighted Discharges are:

$$Q_{10(\text{weighted})} = \frac{(2{,}450 \times 36) + (3{,}810 \times 21)}{57} = 2{,}950 \text{ cfs}$$

$$Q_{100(\text{weighted})} = \frac{(13{,}800 \times 36) + (10{,}700 \times 21)}{57} = 12{,}700 \text{ cfs}$$

---

[5] No cloud of common values available for these regions since area is the only independent variable.

Example G3. Rural Watershed Partially Within High-Elevation Region

A hypothetical study site, at the concentration point, is in a low- to middle-elevation flood region (Regions 8-14), but is within 700 feet of the high elevation region (Region 1) boundary. Discharge estimates for watersheds within 700 feet of the high elevation boundary should be weighted using the Region 1 equations, Therefore, an averaging procedure based on the relation between the elevation of the study site and the 700-foot transition zone should be used. The peak discharges are estimated for each region as if the drainage area is entirely located in one region. Then, a weighted discharge is estimated using the procedures illustrated below.

Estimate the peak discharges for recurrence intervals of 2 and 50 years for an ungaged site in Northeastern Arizona Region 11 with site elevation of 7,100 feet. The site is within 700 feet of the boundary of High Elevation Region 1, which is 7,500 feet in Arizona. The required basin and climatic characteristics are drainage area (A) in square miles, mean annual evaporation (E) in inches, and mean annual precipitation (P) in inches. The drainage area was measured on USGS topographic maps at 45 mi2, the mean annual evaporation was determined to be 55 inches using Figure G-3, and the mean annual precipitation was determined to be 12 inches using Figure G-2. The drainage area, mean annual evaporation, and mean annual precipitation fall within the cloud of common values shown in Figures G-4 and G-7.

For Region 11:

$$Q_2 = 26\ A^{0.62}$$
$$Q_2 = 26\ (45)^{0.62}$$
$$Q_2 = 275 \text{ cfs}$$

$$Q_{50} = 0.24\ A^{0.54}\ E^{2.0}$$
$$Q_{50} = 0.24\ (45)^{0.54}\ (55)^{2.0} = 5{,}671 \text{ cfs}$$
$$Q_{50} = 5{,}670 \text{ cfs}$$

For High-Elevation Region 1:

$$Q_2 = 0.124\ A^{0.845}\ P^{1.44}$$
$$Q_2 = 0.124\ (45)^{0.845}\ (12)^{1.44}$$
$$Q_2 = 111 \text{ cfs}$$

$$Q_{50} = 4.75\ \ A^{0.758}\ P^{0.732}$$
$$Q_{50} = 4.75\ \ (45)^{0.758}\ (12)^{0.732}$$
$$Q_{50} = 525 \text{ cfs}$$

Weighted Discharges are:

$$Q_{2(\text{weighted})} = (275 \text{ x } (7{,}500\text{-}7{,}100)/700) + (111 \text{ x } (1 - (7{,}500\text{-}7{,}100)/700))$$
$$Q_{2(\text{weighted})} = 205 \text{ cfs}$$

$$Q_{50(\text{weighted})} = (5{,}670 \text{ x } (7{,}500\text{-}7{,}100)/700) + (525 \text{ x } (1- (7{,}500\text{-}7{,}100)/700))$$
$$Q_{50(\text{weighted})} = 3{,}470 \text{ cfs}$$

For Additional Information and Examples[6]:

Thomas, B.E., Hjalmarson, H.W., Waltemeyer, S.D., 1994, *Methods for Estimating Magnitude and Frequency of Floods in the Southwestern United States*. USGS Open File Report 93-419.

Transition Zones (weighting equations) .... p. 21
Limitations .... p. 22, 66
Examples .... p. 67-71
Measuring Variables, including Mean Elevation .... p. 17-18
Drainage Area Size .... p. 19
Explanation of Methodology .... p. 77-115

See Also:

"A Study to Evaluate Existing Methods for Determining Peak Discharges for Ungaged Watersheds in Arizona, Phase II & III Report," Report prepared for the State Standards Work Group, May 1995. Prepared by Benchmark Consulting Services, Ltd.

Notice: *A spreadsheet software program described in Appendix J is available from ADWR. This program is set up to perform the Level 2 discharge calculations for Arizona. Contact ADWR for more information.*

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6 The procedures and examples described above are based on, or taken directly from the references cited. These references should be consulted in the event of errors, omissions or other discrepancies.

Appendix H: Level 2 Floodplain Limits/Water Surface Elevation Example

Manning's Equation: Manning's equation is an empirical formula that can be used to estimate flow velocity, depth, and/or discharge. Detailed background information and example applications can be found in any standard civil engineering or hydraulics manual. In addition, there are numerous computer software applications that use Manning's equation to estimate water surface elevation, velocity, depth, width, or other hydraulic variables. The U.S. Forest Service distributes a Manning's rating program "XSPRO," which can be obtained through the National Technical Information Service, or from the ADWR Flood Warning and Dam Safety Section.

Procedures for hand calculation of the normal depth for a channel with simple geometry is illustrated below. Manning's Equation for velocity of flow in an open channel is:

$$V = [1.49\ R^{2/3}\ S^{1/2}]\ /n$$

where:
- V = Mean velocity, feet per second (fps)
- n = Manning coefficient of channel roughness, dimensionless
- S = Channel slope, feet per feet
- R = Hydraulic radius, feet; $R = A/WP$

and
- A = Cross-sectional area of the flowing water, square feet
- WP = Wetted perimeter, feet

Velocity, V, can be related to discharge, Q (cfs), by the continuity equation ($Q = AV$). For wide rectangular channels where flow width is more than ten times the depth, hydraulic radius is approximately equal to the average depth ($R = Y$). Using these relationships, the following form of Manning's equation can be written:

$$Q = [1.49\ A\ Y^{2/3}\ S^{1/2}]\ /n$$

Various hydraulic textbooks and handbooks provide tables of "n" values for various types of channels. A conservative estimate of "n" is recommended for this level of study. When channel cross-section consists of different roughness, the cross-section should be subdivided and different roughness should be used for main channel and overbanks.

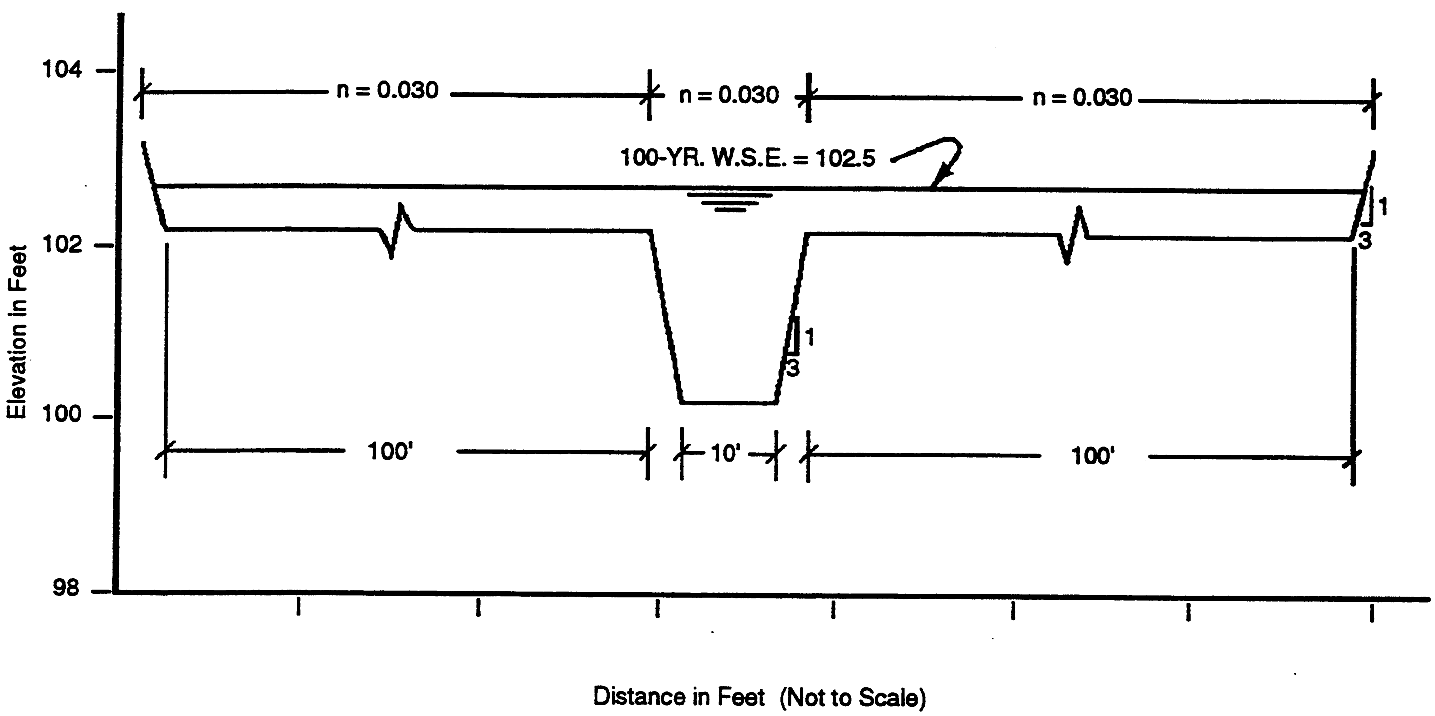
Critical depth may also be computed using the hydraulic data obtained by applying Manning's equation, using the following relationships.

- If $Q^2/g > A^3/T$ then the flow is supercritical
- If $Q^2/g = A^3/T$ then the flow is at critical depth
- If $Q^2/g < A^3/T$ the flow is subcritical

where: Q = Peak Discharge (cfs); A = Conveyance Area (ft^2)
g = 32.2 ft/sec^2; T = Top Width (ft)

Figure H-1

NORMAL DEPTH EXAMPLE CROSS SECTION



Example #H1: Find the 100-year flow depth and velocity for the channel illustrated in Figure H-1. The 100-year discharge has been estimated at 375 cfs (Q100 = 375 cfs), the channel roughness (n) is 0.030, and the channel slope measured from a USGS topographic map is 0.005 ft/ft.

STEP 1: Find the Normal Depth, Y_n and velocity, V, by trial and error.

Try Elev. 102 ft. Where: $Y_n = 2$ ft

Areas = [(22 + 10)/2] x 2 = 32 ft^2
WP = 10 + 2 x 6.3 = 22.6 ft
R = 32/22.6 = 1.4 ft
$V = [1.49\ (1.41)^{2/3}\ (0.005)^{1/2}]/0.030 = 4.4$ fps
Q = 32 x 4.4 = 142 cfs
142 cfs < 375 cfs (not deep enough)

Try elev. = 102.5, Where: $Y_n = 2.5$ ft.

A = [(225 + 222)/2] x 0.5 + 32 = 143.75 ft^2
A = (0.5 X 22) + 32 = 43 ft^2
WP = 225.81 ft
R = 143.75/225.81 = 0.64 ft
$V = [1.49\ (0.64)^{2/3}\ (0.005)^{1/2}]/0.030 = 2.61$ fps
Q = 143.75(2.61) = 375 cfs

Therefore: $Y_n = 2.5$ ft, and
V = 2.61 fps

STEP 2: Add the depth to the channel elevation to obtain the 100-yr. water surface elevation.

100 ft. + 2.5 ft. = 102.5 ft

STEP 3: Check the flow regime using the equations provide above.

$Q^2/g > A^3/T$
$(375\text{ cfs})^2/32.2\text{ ft/sec}^2 < (143.75\text{ ft.}^2)^3/225\text{ ft.}$
4,367 < 13,202 (ft^5). Therefore, flow is subcritical.

Results. The water surface elevation solution of 102.5 feet ($Y_n = 2.5$ ft) should be used. If the flow regime is critical or supercritical then additional analysis should be made and the energy gradeline[7] rather than the normal depth should be used. This process is repeated at several cross-sections and the respective water surface elevations are estimated. Water surface elevations between two cross-sections may be interpolated and an approximate

---

[7] The energy grade line elevation is estimated as the water surface elevation plus the velocity head. The velocity head is computed as $V^2/2g$.

floodplain plotted. The finished floor elevation of a structure must be a minimum of 1 foot above the highest water surface elevation adjacent to the structure.

Alternatively, the channel geometry and hydraulic information can be entered into a computer program, such as XSPRO[8] or another commercial application, and the water surface elevation, velocity, depth, energy grade line, and flow regime computed automatically. Additional descriptive information on application of Manning's equation is provided in Appendix I.

Example #H2. Find the 100-year flow depth and velocity for the channel shown in Figure H-2 and determine the required finished floor elevation. The 100-year discharge was estimated at 11,000 cfs using a Level 2 analysis. Channel slope is 1.2 percent (0.012 ft./ft.). A constant, composite Manning's n value of 0.045 is estimated for the channel and floodplain.

STEP 1: Since the channel has irregular geometry, the XSPRO computer program (See Appendix J) was used to perform a channel rating. XSPRO Output is shown in Table H-2.1.

Table H-2.1. XSPRO Manning's Rating Output

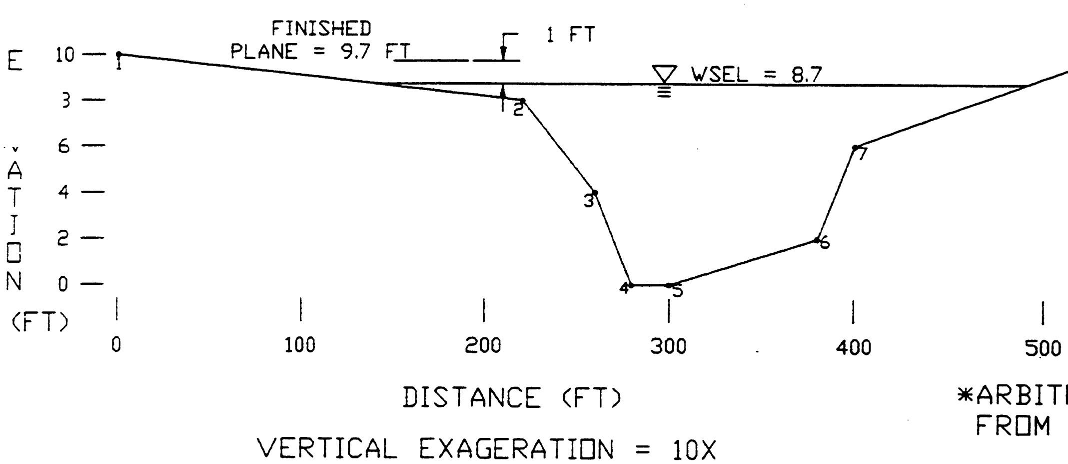
Stage (ft)	Average Depth (ft)	Average Velocity (ft/s)	Discharge (cfs)	Width (ft)	Froude Number
8.6	3.7	8.7	10,796	332	0.80
8.7	37	8.6	10,981	347	0.79
8.8	3.6	8.5	11,189	361	0.79

STEP 2: Match 100-year discharge estimate of 11,000 cfs to XSPRO profiles, and determine 100-year water surface elevation (depth) and finished floor elevation.

Depth of 100-year event: 8.7 feet
Width of 100-year event: 347 feet
Required finished floor elevation: 9.7 feet.

[8] XSPRO is a Manning's equation program distributed by the U.S. Forest Service. Numerous other private, for-profit vendors of Manning's equation software exist.

FIGURE H-2 LEVEL 2 FLOODPLAIN EXAMPLE



SURVEY POINTS*

PT#	Y	X
1	10	0
2	8	220
3	4	260
4	0	280
5	0	300
6	2	380
7	6	400
8	12	600

*ARBITRARY DATUM, STATIONING FROM LEFT END OF SECTION

Appendix I. Level 2 Administrative Floodway Boundary Methodology

Methodology Overview

Three steps that are generally required to delineate an administrative floodway:

STEP 1. Estimate the 100-year peak discharge.
STEP 2. Determine the 100-year floodplain limits or water surface elevation.
STEP 3. Determine the administrative floodway width.

Administrative Floodway Determination

The procedures for estimating the administrative floodway boundaries using manual calculation procedures are:

STEP 1. Estimate the 100-year peak discharge for the watercourse using the Level 2 discharge methodology described in Appendix G. For drainage areas that do not meet the criteria for a Level 2 methodology, use the Level 1 or 3 methodology.

STEP 2. Provide normal depth calculations, using the procedures outlined in Appendix H, at several representative cross section locations adjacent to the proposed improvement/ development for existing conditions. Cross sections should be located both upstream and downstream of the proposed improvement, as well as adjacent to the proposed improvement. Cross sections should be spaced at 300 to 500 feet intervals, with a minimum of three cross sections required along short reaches. A sufficient number of cross section points should be obtained to describe the channel and overbank geometry. Manning's roughness coefficients can be estimated using the references provided earlier or other similar publications.

STEP 3. Cross sections should be plotted on engineering type 10 x 10 grid line paper (i.e., ten lines to the inch in both the vertical and horizontal direction). Cross sections should be plotted at a scale that will make it easy to perform the floodway encroachment computations explained below. A vertical scale of 1 or 2 feet per inch and a horizontal scale of 100 or 200 feet per inch works well with wide watercourses. The horizontal scale may be adjusted to 10 feet per inch on narrow watercourses.

Provide normal depth calculations, using the same cross sections as in Item 2 above, to determine the administrative floodway area. This is accomplished using a trial and error procedure by encroaching from <u>both</u> edges of the 100-year floodplain equally <u>so as not to increase the existing condition water surface elevation more than one foot</u>. The encroachment shall be by equal conveyance of flood flow area and not necessarily by equal overbank encroachment lengths. *Encroachment beyond the channel bank and into the main channel area is not permitted. Encroachments must stop at the channel bank.* Channel banks can typically be identified by a distinct grade break between the bank slope and the overbank floodplain, a change in vegetative density between the channel bed and overbank floodplain or geomorphic characteristics of the stream. Where the bank is not visually identifiable, the Corps of Engineers definition of the overbank area beginning where depth of flow is less than 3 feet and velocities less than 3 feet per second may be used.

Begin the floodway encroachment procedure by drawing a vertical line (wall) set within the floodplain from the left edge of the floodplain limit. The distance from the left floodplain limit to the vertical line should be about 25% (¼) of the total floodplain width (i.e., if the floodplain has a 400 foot top width, then the first left vertical wall should be set 100 feet into the floodplain from the left edge of the floodplain limit). The next step is to count the number of squares on the 10 x 10 grid paper that contain flood flow area between the left edge of the floodplain limit and the left vertical wall. Next, set the right vertical wall by simply starting at the right floodplain limit, moving to the left and counting the number of squares that contain flood flow until there are the same number of squares as contained in the left floodway fringe, and then set the right vertical wall. At this point there will be a cross section showing the 100-year water surface, the 100-year floodplain, and two vertical walls set in from the left and right edges of the floodplain limits. Remember that the vertical walls are set in from the floodplain limits using the same total squares (area) and not the same distance.

Next, re-compute the 100-year water surface elevation using Manning's equation and the left and right vertical walls as the new left and right top of bank. Set the left and right bank elevations 2 feet above the unencroached 100-year water surface elevation. Repeat this procedure by moving the left and right vertical walls an equal number of squares each trial until you have obtained a 1 foot rise in the 100-year water surface elevation from the unencroached condition. Usually 3 to 5 trial computations are necessary. This procedure is repeated at each cross section.

Once the administrative floodway limits have been established at each cross section the location can be plotted on the site plan of the property as shown in Example #I1 and Example #I2.

Notice: *A computer program developed by the U.S. Army Corps of Engineers "Simplified Floodway Determination (SFD) Computer Program User's Manual" is available from ADWR and may used in place of the procedures described is this Appendix. See Appendix J for more information.*

Example #I1. A land owner owns 5 acres of land in Santa Cruz County, adjacent to a large wash (Figure I-1). Determining how much of his property is located in the 100-year floodplain and administrative floodway and how much land may be claimed for future land planning purposes.

STEP 1. Measure the drainage area using a USGS topographic quadrangle map and estimate the 100-year peak discharge rate. For a drainage area of 85.2 square miles, using the Level 2 discharge methodology shown in Appendix G, the 100-year peak discharge is estimated at 12,600 cfs.

STEP 2. Using three field-surveyed cross sections, determine the 100-year normal depth and floodplain limits using the Level 2 methodology described in Appendix H. The locations of the cross-sections are shown on a property map of the parcel (Figure I-1). Figure I-2, I-3, and I-3 show the plotted cross-sections and normal depth calculations. The estimated 100-year water surface at each cross-section is plotted on Figure I-1 to delineate the 100-year floodplain.

STEP 3. Establishes the administrative floodway by equal encroachment into the 100-year floodplain using cross-sections in Figures I-2, I-3, and I-4. This is accomplished by trial and error procedures using vertical walls (representing equal conveyance areas being moved from the floodplain) and recalculating a new normal depth until the water surface elevation is raised not greater than 1 foot. The resultant information from each cross-section is plotted on Figure I-1.

STEP 4. The available development area, amount of reclaimed land, and fill requirements can be estimated from this information. Fill requirements should be based on the highest water surface elevation adjacent to the proposed structure(s) by interpolation from the encroached cross-sections.

STEP 5. The hydraulic calculations indicate velocities exceeding 5 feet per second. The fill material which is exposed to the stream's flow will require erosion protection.

Figure I-1

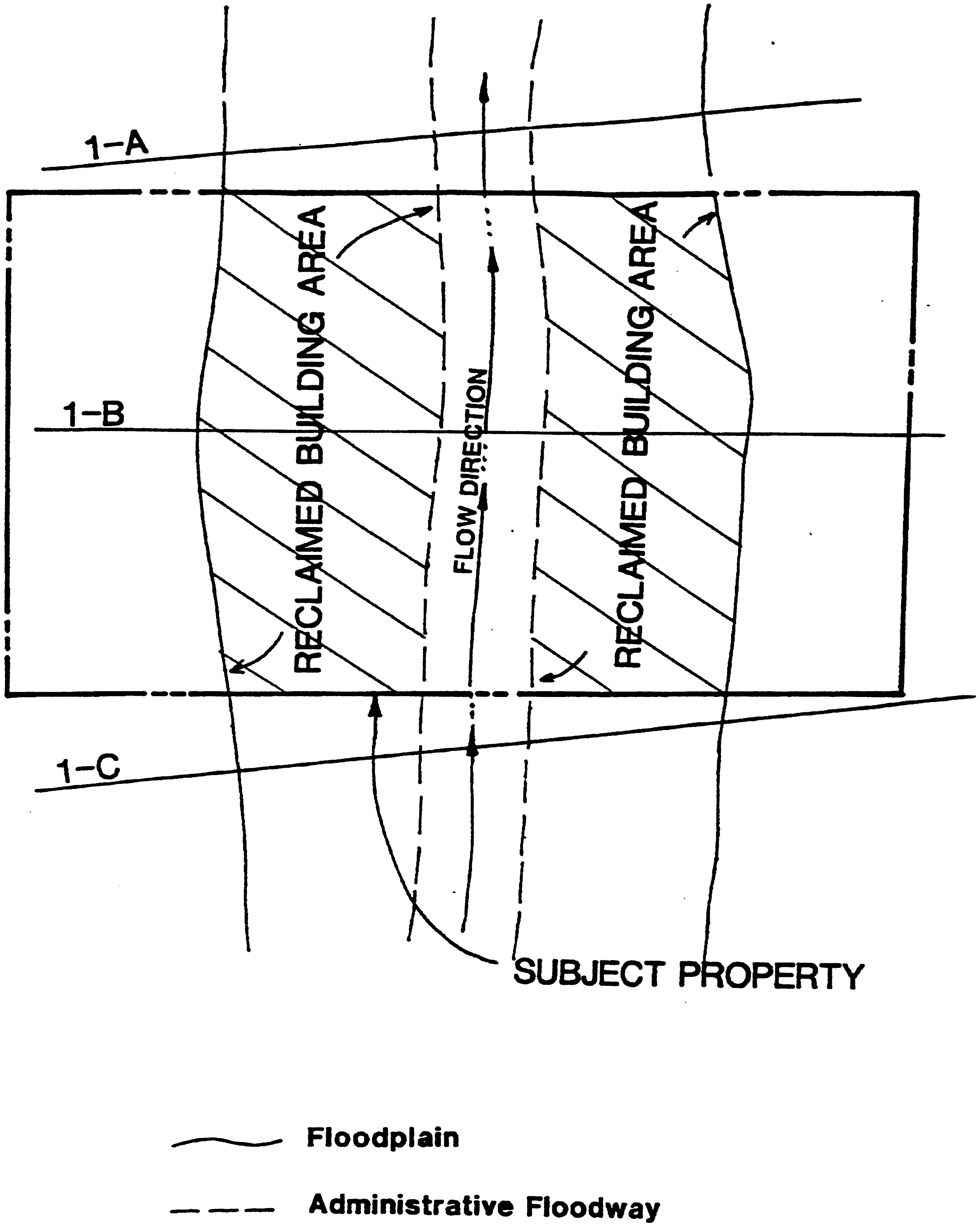
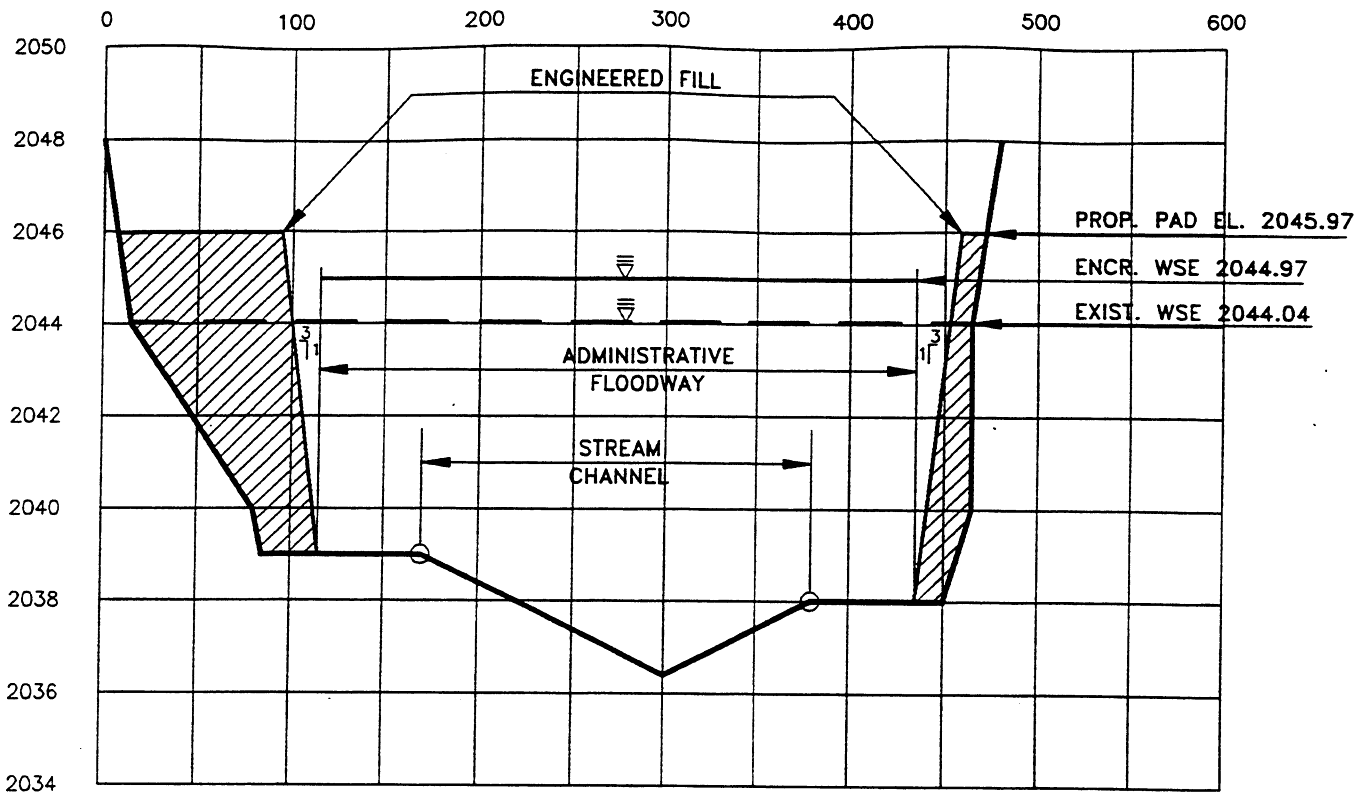


ILLUSTRATION NOT TO SCALE



SECTION 1-A

GIVEN: Q_{100} = 12,600 CFS S = 0.8% N = 0.048

O = CHANNEL BANK

EXISTING CONDITION:

WSE = 2044.04 FT.

DEPTH = 6.04 FT.

VELOCITY = 6.49 FPS

AREA = 1942.42 S.F.

PERIMETER = 541.54 FT.

TOP WIDTH = 541 FT.

FLOW REGIME IS SUBCRITICAL

ENCROACHED CONDITION:

WSE = 2044.97 FT.

DEPTH = 6.97 FT.

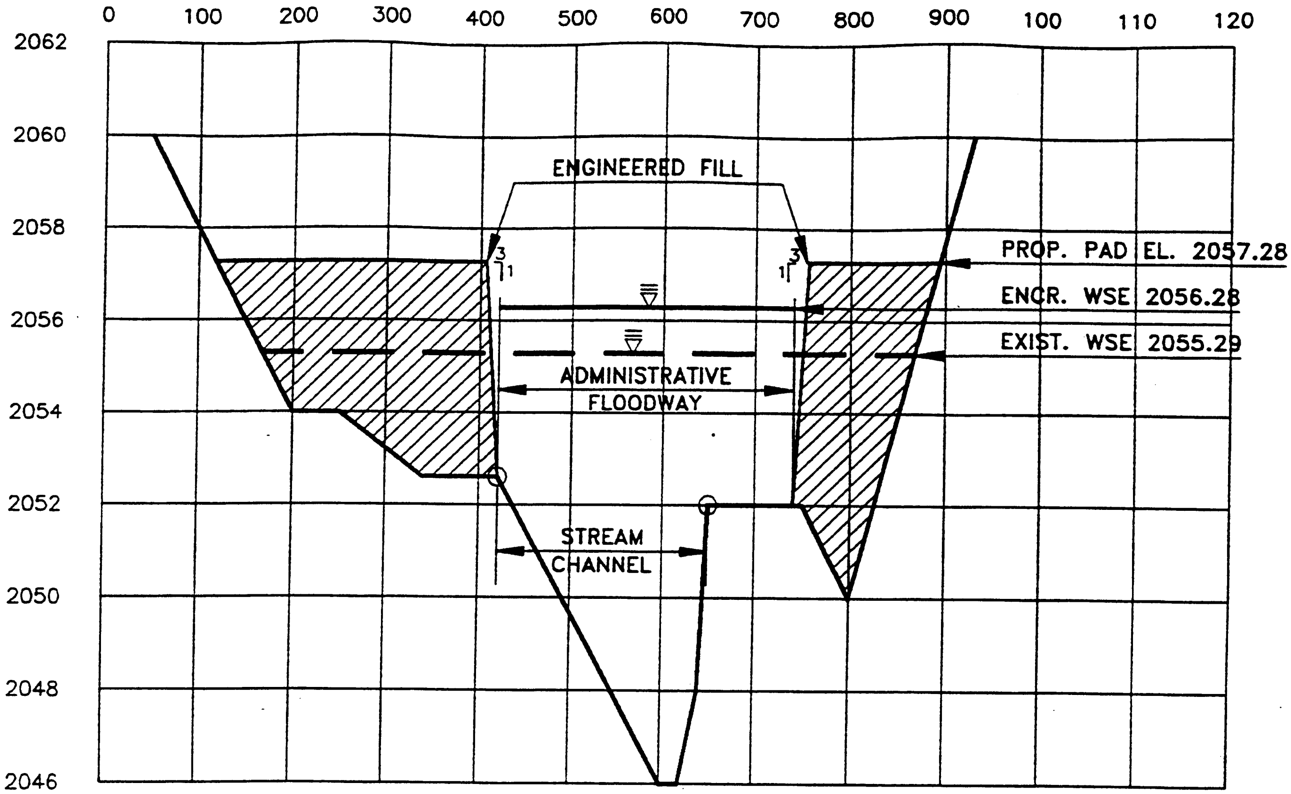
VELOCITY = 7.88 FPS

AREA = 1599.18 S.F.

PERIMETER = 333.13 FT.

TOP WIDTH = 320 FT.

FLOW REGIME IS SUBCRITICAL



SECTION 1–B

GIVEN: Q_{100} = 12,600 CFS S = 0.8% N = 0.048

O = CHANNEL BANK

EXISTING CONDITION:

WSE = 2055.29 FT.

DEPTH = 9.29 FT.

VELOCITY = 5.82 FPS

AREA = 2167.07 S.F.

PERIMETER = 711.83 FT.

TOP WIDTH = 710 FT.

FLOW REGIME IS SUBCRITICAL

ENCROACHED CONDITION:

WSE = 2056.28 FT.

DEPTH = 20.28 FT.

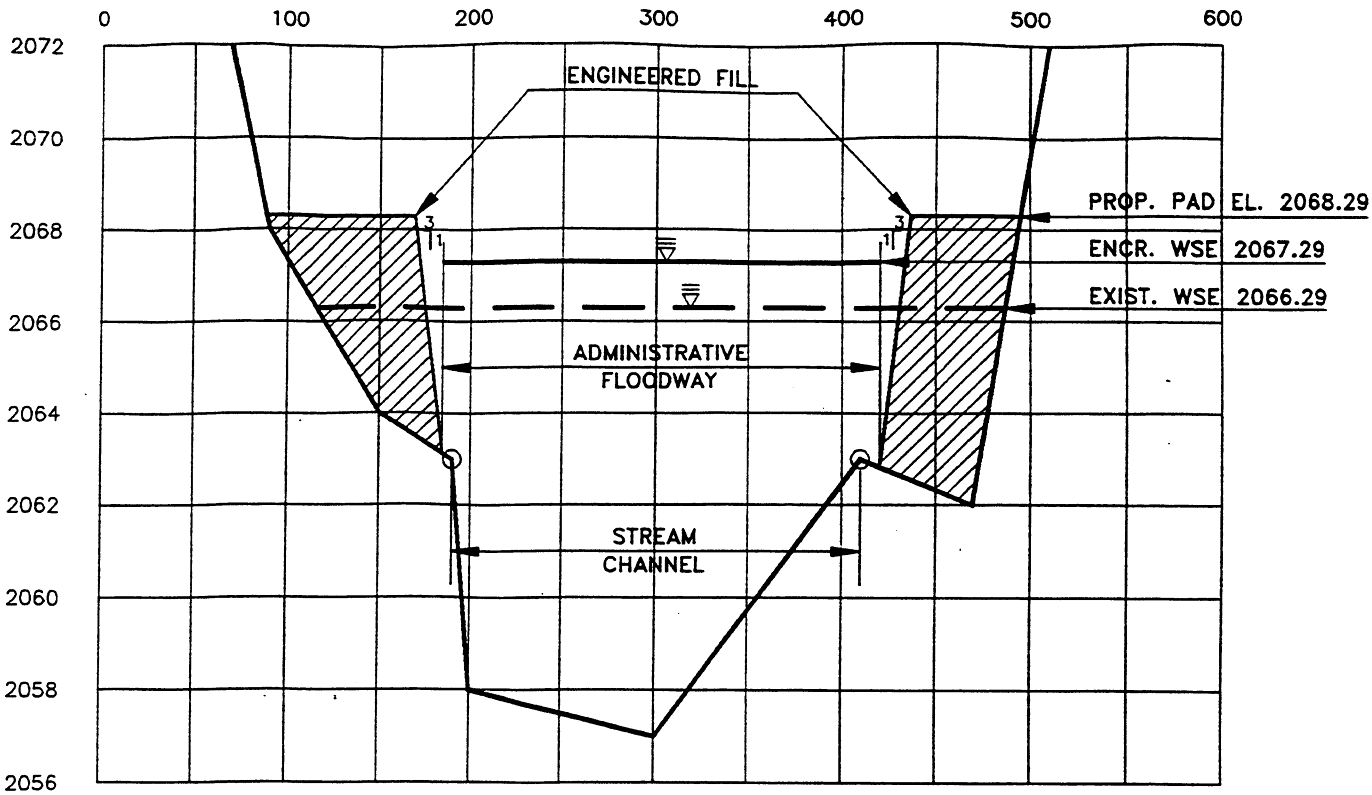
VELOCITY = 8.17 FPS

AREA = 1542.04 S.F.

PERIMETER = 304.09 FT.

TOP WIDTH = 295 FT.

FLOW REGIME IS SUBCRITICAL



SECTION 1-C

GIVEN: Q_{100} = 12,600 CFS S = 0.8% N = 0.048

O = CHANNEL BANK

EXISTING CONDITION:

WSE = 2066.29 FT.

DEPTH = 8.29 FT.

VELOCITY = 7.58 FPS

AREA = 1663.74 S.F.

PERIMETER = 367.68 FT.

TOP WIDTH = 366 FT.

FLOW REGIME IS SUBCRITICAL

ENCROACHED CONDITION:

WSE = 2067.29 FT.

DEPTH = 9.29 FT.

VELOCITY = 8.93 FPS

AREA = 1411.51 S.F.

PERIMETER = 243.77 FT.

TOP WIDTH = 230 FT.

FLOW REGIME IS SUBCRITICAL

Example #I2. A land owner has 10 acres located in a moderately urbanized sheet flow area within unincorporated Maricopa County at an elevation of 1,200 ft. above sea level (Figure I-5). The land owner would like to construct a new home on fill within an area which has historically experienced flooding and has been told by the local floodplain administrator to use a Level 2 methodology to conduct a drainage study of the area.

STEP 1. Estimate the 100-year peak discharge. Using a USGS Topographic Quadrangle Map, it is determined that the drainage area is approximately 0.8 square miles. Because the watershed is located in an urban watershed on an alluvial fan, and because the watershed elevation is outside the "cloud of common values" for Region 12 shown in Appendix G, the Level 2 discharge methodology may not be used. Therefore, the Level 1 discharge methodology will be used. Using the envelope curve provided in Appendix D, the 100-year peak discharge is estimated at 3,090 cfs.

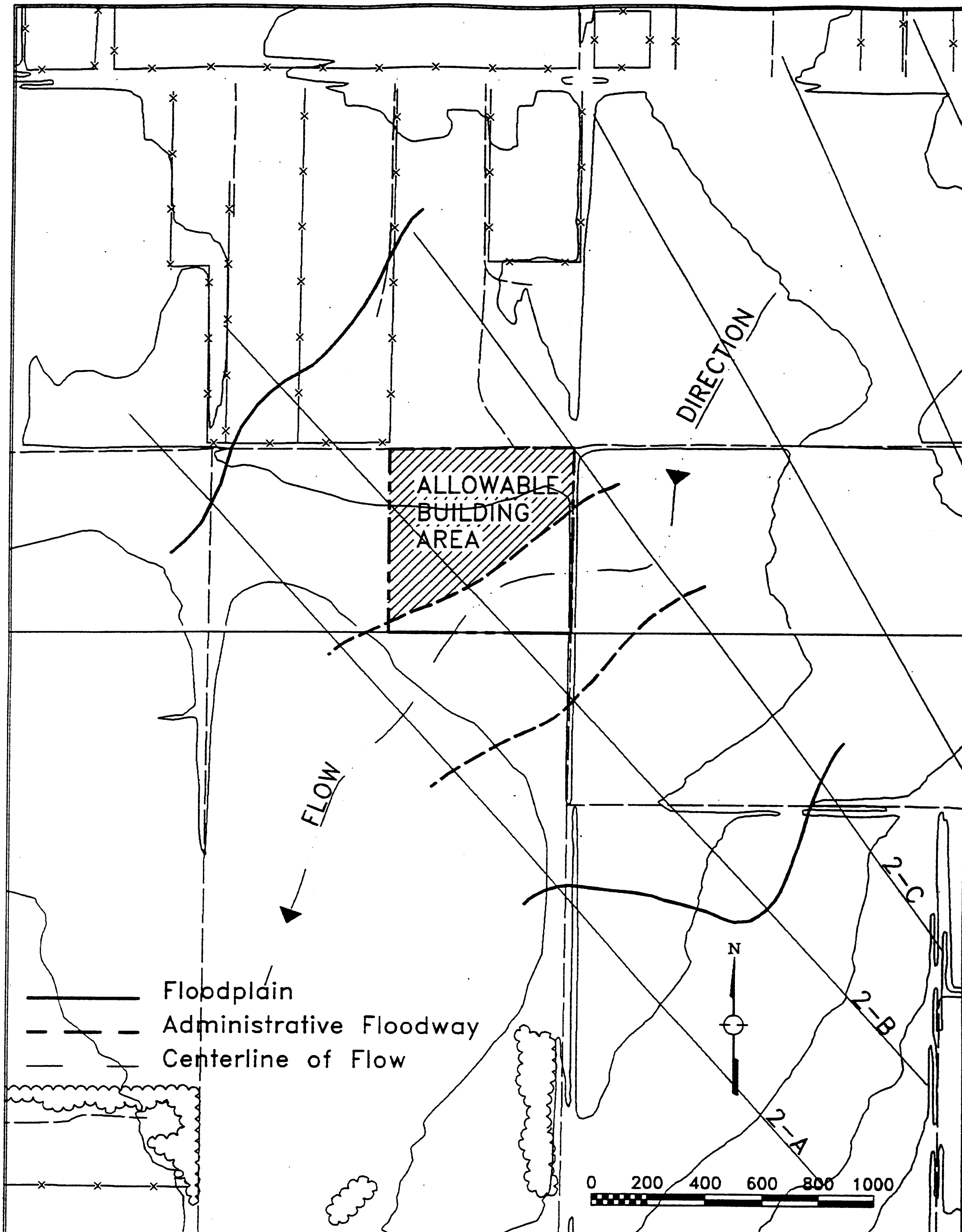
STEP 2. Estimate the floodplain limits using Manning's ratings of cross sections at the site. Cross sections of the site were acquired from recent aerial mapping of the area and were used to obtain three cross-sections. The locations of the cross-sections are shown on Figure I-5. Figures I-6, I-7, and I-8 show the plotted cross-sections and normal depth calculations. The resultant 100-year water surface at each cross-section is plotted on Figure I-5 to delineate the 100-year floodplain.

STEP 3. Determine the administrative floodway limits using equal encroachment into the 100-year floodplain and the cross-sections from Figures I-6, I-7, and I-8. This is accomplished by a trial and error procedure where vertical walls (representing equal conveyance areas being removed from the floodplain) are placed within the cross-section and a new normal depth calculation is performed. This is repeated until a maximum increase of 1 foot above the existing condition water surface elevation is obtained.

STEP 4. The finished building pad will be constructed of engineered fill material placed at an elevation one foot above the encroached 100-year water surface elevation. The highest encroached water surface elevation adjacent to the structure (typically upstream) is used to determine the building pad elevation.

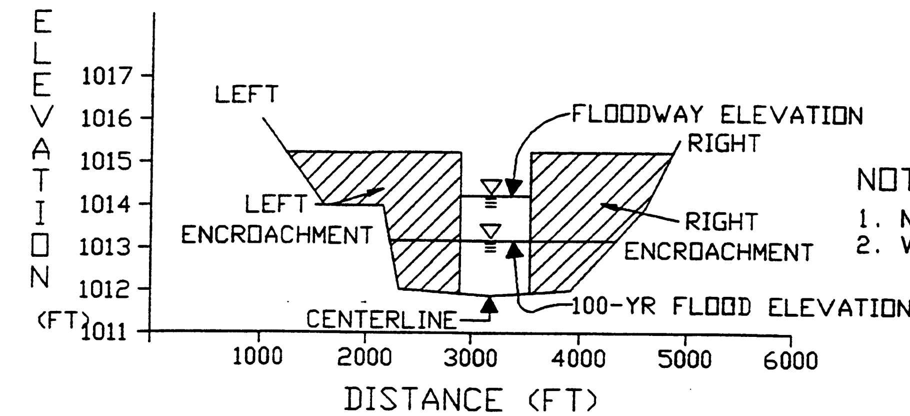
STEP 5. The hydraulic calculations indicate velocities less than 5 feet per second. The fill material exposed to the watercourse's flow require vegetative erosion protection as approved by the local floodplain administrator.

Figure I-5



ALLOWABLE BUILDING AREA
DIRECTION
FLOW
2-C
2-B
2-A
N
0 200 400 600 800 1000
Floodplain
Administrative Floodway
Centerline of Flow

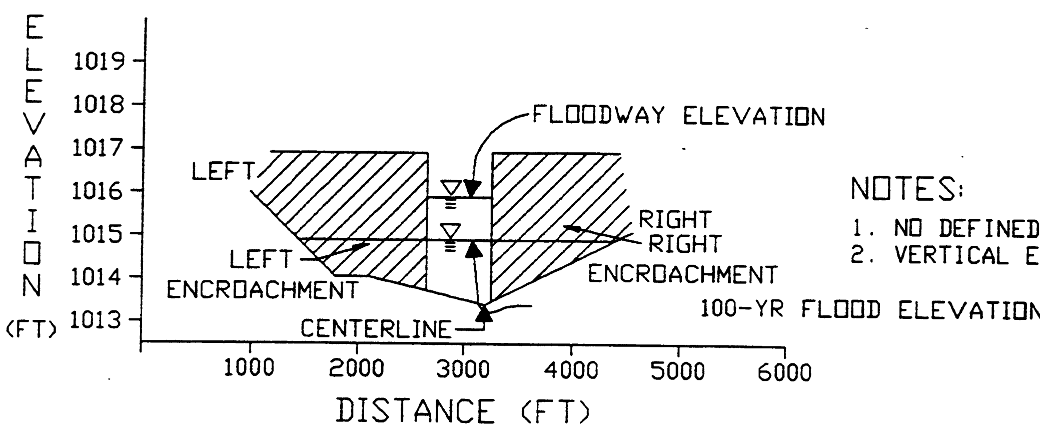
SECTION #I2-A



NOTES:

1. NO DEFINED CHANNEL BANKS
2. VERTICAL EXAGERATION = 400X

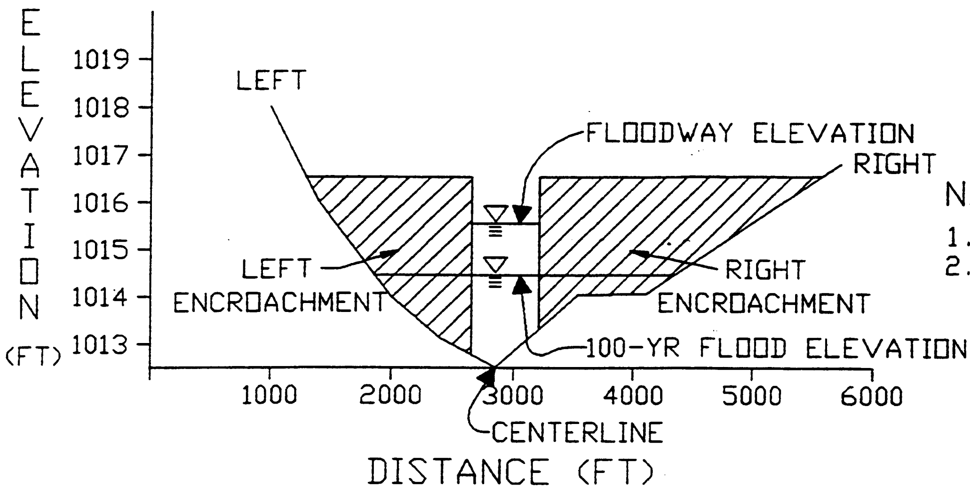
SECTION #I2-B



NOTES:

1. NO DEFINED CHANNEL BANKS
2. VERTICAL EXAGERATION = 400X

SECTION #I2-C



NOTES:
1. NO DEFINED CHANNEL BANKS
2. VERTICAL EXAGERATION = 400X

HYDRAULIC DATA SUMMARY	SECTION #A F/P	SECTION #A F/W	SECTION #B F/P	SECTION #B F/W	SECTION #C F/P	SECTION #C F/W
WSEL	1013.21	1014.21	1014.85	1015.85	1014.39	1015.39
DEPTH (MAXIMUM)	1.31	2.31	1.45	2.45	1.89	2.89
VELOCITY	1.49	2.18	1.64	2.33	1.83	2.44
TOPWIDTH	2114	650	2896	630	2452	528
FLOW REGIME	SUBCRITICAL		SUBCRITICAL		SUBCRITICAL	
SLOPE	0.0016		0.0017		0.0020	
N	0.0480		0.0480		0.0550	
Q100	3200		3200		3200	

Appendix J: Application Software

Computer software is available from ADWR to assist in the application of the methodologies described in this document. Software includes the following:

Discharge Methodology

A spreadsheet program, originally developed in QuattroPro, was developed to assist in computation of the USGS regression equations (Level 2 Hydrology, as described in this report). The spreadsheet was set up to compute regression equations for a single or for multiple regions.

Floodplain Methodology

The U.S. Forest Service computer program, XSPRO, a Manning's rating software application, was used in the development of the examples shown in this document. XSPRO is available from the U.S. Forest Service, from the National Technical Information Service, or from government document repositories as follows:

- XSPRO: A Channel Cross-Section Analyzer. Technical Note 387. August 1992. U.S. Dept. Of The Interior - Bureau of Land Management and U.S. Dept. Of Agriculture - Forest Service.

It is noted that numerous other Manning's equation computer programs, software packages, and publications are available from commercial vendors or public agencies that would meet the objectives of the state standard.

Floodway Determination

The U.S. Army Corps of Engineers has developed a computer program called Simplified Floodway Determination (SFD). This program is described in a publication entitled:

- Simplified Floodway Determination (SFD) Computer Program User's Manual, The Hydrologic Engineering Center, U.S. Army Corps of Engineers. May 1989.

Availability of Software

The software described above is available from the Department of Water Resources Flood Warning and Dam Safety Section.

STATE OF ARIZONA
DEPARTMENT OF WATER RESOURCES
ENGINEERING DIVISION

State Standard

for

Supercritical Flow

Under authority of ARS 45-3605(a), the Director of the Arizona Department of Water Resources establishes the following standard for delineation of floodways in riverine environments with supercritical flow in Arizona:

Floodway limits on streams in Arizona which have supercritical flow, for use in fulfilling the requirements of Flood Insurance Studies, and local community and county flood damage prevention ordinances will be determined using the guidelines outlined in State Standard Attachment 3-94 entitled "Floodway Modeling Standards for Supercritical Flow" or by an alternative procedure reviewed and accepted by the Director.

For the purpose of application of these guidelines, supercritical floodway modeling standards will apply to all watercourses identified by the Federal Emergency Management Agency as part of the National Flood Insurance Program, all watercourses which have been identified by a local floodplain administrator as having significant potential flood hazards and all watercourses with drainage areas more than 1/4 square mile or a 100-year estimated flow of more than 500 cubic feet per second. Application of these guidelines will not be necessary if the local community or county has in effect a drainage, grading or stormwater ordinance which, in the opinion of the Department, results in the same or greater level of flood protection as application of these guidelines would ensure.

This requirement is effective December 1, 1994. Copies of this State Standard and State Standard Attachment 3-94 can be obtained by contacting the Department's Engineering Division at (602) 417-2445.

STATE OF ARIZONA
DEPARTMENT OF WATER RESOURCES
ENGINEERING DIVISION

Floodway Modeling Standards

for

Supercritical Flow

500 North 3rd Street
Phoenix, Arizona 85004

(602) 417-2445

STATE STANDARD ATTACHMENT
SSA 3-94

NOVEMBER 1994

Disclaimer of Liability

The methods contained in this publication are intended to be a reasonable way of setting minimum floodplain management requirements where better data or methods do not exist. As in all technical methods, engineering judgement and good common sense must be applied and the methods rejected where they obviously do not offer a reasonable solution.

It must be recognized that while the criteria established herein will generally reduce flood damages to new and existing development, there will continue to be flood damages in Arizona. Where future-condition hydrology (which considers the cumulative effects of development) is not used, future development will probably increase downstream runoff which may result in flooding. Unlikely or unpredictable events such as earthquakes or dam failures may also cause extreme flooding.

The Arizona Department of Water Resources is not responsible for the application of the methods outlined in this publication and accepts no liability for their use. Sound engineering judgement is recommended in all cases.

The Arizona Department of Water Resources reserves the right to modify, update or otherwise revise this document and its methodologies. Questions regarding information or methodologies contained in this document and/or floodplain management should be directed to the local floodplain administrator or the office below:

Engineering Division
Arizona Department of Water Resources
500 North 3rd Street
Phoenix, Arizona 85004

Phone: (602) 417-2445
FAX: (602) 417-2401

Contents

Introduction

The National Flood Insurance Program (NFIP) regulations define a floodway as the floodplain area that must be reserved to discharge the base (100-year) flood without increasing the water surface elevation by more than one foot. This NFIP criterion assumes that streams flow at subcritical[1] depth, such that a decrease in floodplain width results in an increase in the flood water surface elevation. However, in high-velocity streams flowing at or below critical depth, a decrease in floodplain width may result in a decrease in water surface elevation. Therefore, the hydraulics of floodway determination for streams with high velocity flow is more complex.

In Arizona, many streams flow near or below critical depth. Steep, bedrock streams may be supercritical at flood stages. Many alluvial streams flow at or near critical depth. Application of subcritical floodway modeling standards to supercritical or near-critical flow may result in unacceptable increases in flow velocity or unsafe encroachment, and may expose future and existing development to excessive flood hazard.

The Arizona Department of Water Resources (ADWR) has established guidelines to be used when modeling floodways for supercritical or near critical flow in Arizona. Accurate floodway delineation for supercritical flow requires special procedures. This document describes the guidelines for modeling types of supercritical floodways for Flood Insurance Studies and floodplain management. In addition, special cases of supercritical flow are described and illustrated in example applications of the guidelines.

When to Apply Guidelines

The guidelines described in this document are to be used for all detailed Flood Insurance Studies and floodplain management applications on streams with supercritical flow in the State of Arizona. These guidelines for supercritical floodway modeling should be applied to streams or stream reaches[2] which meet any of the following criteria:

- A subcritical HEC-2 model of the stream (non-floodway run) defaults to critical depth[3] at three consecutive cross sections, or at 40 percent or more of the cross sections in a reach, or

---

[1] For definitions of the terms *"critical," "subcritical,"* and *"supercritical."* see *V.T. Chow, 1959, Open Channel Hydraulics, McGraw Hill Publishing, New York,* or *R.H. French, 1985, Open Channel Hydraulics, 2nd Ed., McGraw Hill Publishing, New York*

[2] A reach may be defined as section of a channel or stream which has similar hydraulic or geomorphic characteristics, such as vegetation, roughness coefficients, area of conveyance, channel geometry, and/or channel slope. Within a reach, cross sections are relatively uniform.

[3] The presence of critical depth should be determined from detailed HEC-2 output, not from the list of error messages at the end of the HEC-2 output printout.

- A subcritical HEC-2 floodway run indicates that the encroached water surface elevation decreases at three consecutive cross sections, or 40 percent or more of the cross sections in a reach, or

- Sound engineering judgement indicates supercritical floodway standards should be applied.

Special Cases of Supercritical Flow

Guidelines for five special cases of supercritical floodway problems are described and illustrated. The five special cases are:

- **Bank Station Designation.** In some cases, the location of the channel bank stations may not be obvious. Because floodways may not encroach within the channel banks of a stream accurate definition of the channel stations is important for floodway modeling.

- **High-Velocity, Near-Critical Flow.** HEC-2 may become computationally unstable at depths near critical depth, and default to critical depth, even where critical or supercritical depth do not occur.

- **Channelized Supercritical Flow.** Where supercritical flow is confined within the designated channel banks, the floodway and floodplain widths are identical.

- **Composite Flow.** Composite flow occurs where both supercritical flow and subcritical flow are present within a single cross section.

- **Braided Flow.** Supercritical flow on braided streams is usually a special case of composite flow, or a case of floodway delineation around islands.

Modeling Guidelines

Appropriate modeling procedures for supercritical floodway modeling may not be intuitively obvious, may require advanced knowledge of hydraulics, and may require minor adjustments for site specific variables. In this document, it is assumed that HEC-2 will be used for floodway modeling. In practice, any hydraulic model which meets local, state, and federal criteria may be used. Modeling guidelines are outlined below.

General Guidelines

These procedures apply to all cases of supercritical floodway modeling outlined in this document. Specific requirements include:

- **Subcritical Profile.** Floodway limits should be determined in the subcritical flow regime when using the HEC-2 program, as required by current FEMA guidelines, regardless of the actual flow regime.

- **Energy Grade Line.** Floodway limits for near-critical or supercritical flow will be determined using the rise in the energy grade line (rather than water surface elevation) caused by encroachment. This corresponds to HEC-2 encroachment method #6.

- **Bank Station Limit.** Floodway limits may not be located inside the channel banks, except in entrenched channels where the entire base flood is contained within the channel banks.

- **Floodway Velocities.** The following comment should be added to the Flood Insurance Study floodway tables when the supercritical flow conditions are present: "Supercritical, or near-critical, flow conditions may exist at the cross sections listed above. The floodway velocities or other velocities shown in this Table should not be used for design purposes, unless an engineering analysis indicates that subcritical flow conditions are present at appropriate cross sections."

- **Floodway Velocity Determination.** Velocities for design and floodplain management purposes should be determined using the supercritical flow option of HEC-2 or an equivalent model. Design velocities should reflect maximum encroachment limits determined using the procedures outlined in this standard.

- **Perched Flow.** These guidelines do not apply to perched flow, except when the perched flow is modeled separately from the main channel floodway. Perched flow originates along well defined channels where overbank flooding becomes separated from the main flow path, and develops hydraulic characteristics unique from the main channel.

- **Roughness Coefficients.** Manning's "N" values should be carefully selected for streams with steep slopes which experience supercritical flow. Manning's "N" values for low gradient streams may not apply. Guidelines for determining "N" values on steep streams are given in Jarrett (1984, 1985).

Channel Bank Designation

In many cases, it is obvious where channel bank stations should be located. Key indicators include the grade break between the bank slope and overbank floodplain, the change in vegetative density between the channel bed and riparian area, or the geomorphic characteristics of the stream. Where channel banks cannot readily be identified from topographic and other data, the Corps of Engineers (1988) definition of channel banks[4] should be used. The Corps defines the channel banks (or the beginning of the overbank area) as the point where depths become less than 3 feet and velocities become less than 3 feet per second. This bank definition may also be used as the starting point for floodway encroachment modeling. It is necessary to perform an initial HEC-2 run to obtain a velocity distribution in order to apply the Corps bank station definition. Subsequent runs will be necessary to refine floodway limits.

For supercritical floodway modeling channel bank stations should be identified using the following:

- **Topographic/Geomorphic Data.** Grade breaks, vegetative and bed sediment characteristics, and channel shape usually help identify bank stations.

- **Hydraulic Data.** Where bank stations cannot be identified from topographic or geomorphic characteristics, the bank station (or the beginning of the overbank) is defined as the point closest to the center of the channel where:

  depth = 3 ft., and
  velocity = 3 ft/s

Example 1: Illustrates Channel Bank Station Designation.

High-Velocity, Near Critical Flow

For streams which flow at or near critical depth, the HEC-2 model may be computationally unstable. Therefore, the modeler should use a optimal number of cross section and data points, as well as verify the accuracy of energy loss coefficients used. HEC-2 critical depth messages may be an indication of unstable modeling, rather than supercritical or critical flow depths. HEC-2 models generally may be regarded as stable

---

[4] Channel bank definition is intended only for floodway delineation purposes.

if the velocity head is less than 1/3 the flow depth[5]. Where possible, near critical flow models should be calibrated to measured highwater marks.

The following are floodway modeling guidelines and stability tests for high velocity, near critical flow, which supplement the general guidelines outlined above:

- **Velocity Head Criteria.** Compare velocity head and channel depth for channel sections within the stream reach. If the velocity head is less than 1/3 the flow depth (subcritical profile) or greater than 2/3 the flow depth (supercritical profile), the model may be regarded as stable.

- **Additional Cross Section Points.** Compare channel geometry described by ground reference (GR) points relative to upstream and downstream cross sections. Remove or add points to achieve an optimum number of points which accurately describe the section and reach geometry.

- **Energy Loss Coefficients.** Test the sensitivity of the model to variation in energy loss coefficients, such as Manning's roughness coefficients ("N" values). Check model to determine if coefficients selected reflect factors such as bed form roughness, sediment transport, channel slope, and flow depth, as well as bed sediment size, channel shape, and vegetative obstructions.

- **Calibrate.** Obtain high water marks from the channel, where possible, and calibrate computed water surface elevations to the high water mark profile. If an independent estimate of the peak discharge is available, the model can be calibrated using the known discharge as well as the highwater marks.

- **Additional Cross Sections.** Insert new cross sections to determine if flow is actually supercritical or if the model is unstable due to insufficient data.

Example 2: Illustrates Procedures and Output From a Near-Critical Water Surface Profile

Channelized Supercritical Flow

For confined supercritical flow (no overbank flow), floodway (encroachment) modeling should be abandoned. The floodplain limits should be regarded as the floodway boundaries. In some cases, the floodplain limits may be within the channel bank stations defined for the HEC-2 model.

Example 3: Illustrates Two Cases of Channelized Supercritical Flow.

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[5] Corps of Engineers, 1988, "Floodway Determination Using Computer Program HEC-2," Training Document No. 5, Prepared by Vern Bonner, Hydrologic Engineering Center, Davis, California, January, 1988, p. 70.

Composite Flow

For composite flow situations, with supercritical flow in the channel and subcritical or near critical flow in the overbanks, floodway definition may be possible. However, the modeler must ensure that overbank flow modeling is computationally stable using procedures described above. A procedure described in Schoellhamer et. al. (1985) is recommended to determine if composite flow exists. Schoellhamer's procedure involves determining "subdivision Froude numbers" for subdivisions of a cross section. Cross section subdivisions may be the right overbank, left overbank, and main channel, or may be further divided by areas with similar "N" values or by cross section geometry. For cross sections with composite flow, portions of the section will have subdivisions Froude numbers greater than one, and other portions will have subdivision Froude numbers less than one. If composite flow exists and the model is computationally stable, then the floodway may be delineated by assuming the floodway limit is located where overbank depths exceed 3 feet and velocities exceed 3 feet per second, or by applying the guidelines for high-velocity, near critical flow.

The following guidelines are to be used for floodway modeling of composite flow, in addition to the general modeling guidelines outlined above:

- Composite Flow. Use the method of Schoellhamer (1985) to test for the presence of composite flow. It may be necessary to request a trace (J2.10=15) in the HEC-2 input file to use Schoellhamer's procedures.

- Depth/Velocity Limit. Determine if overbank depths and velocities exceed 3 ft. and 3 ft/s, respectively. If these limits are exceeded, and if supercritical flow occurs in the main channel, use the floodplain limits as the floodway limits.

- Additional Cross Sections. Test the model to determine if critical depth message result from insufficient cross sections, or from supercritical flow.

Example 4: Outlines Computations Required to Test for Presence of Composite Flow.

Braided Flow

Application of floodway modeling techniques may not be appropriate for braided streams, and should be considered on a case-by-case basis. Consultation with local floodplain officials and federal agencies is recommended prior to initiating a floodway study for a braided stream. Braided flow, if supercritical flow occurs in flow braids, is essentially a case of composite flow. Therefore, the guidelines for composite flow should be applied. Floodway limits should include all of the flow braids (all of the channel area). Where islands are present between braids, floodway standards for streams with islands should be followed, in addition to supercritical floodway modeling standards. The Corps of Engineers floodway manual, referenced earlier, discusses application of the floodway modeling criteria to braided streams.

Example 5: Illustrates Maximum Encroachment Limits for Streams with Braided Flow.

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Test Applications

Example 1: Channel Bank Designation

- **Problem Statement.** Two channel cross sections are presented in Figures 1 and 2. In Figure 1, channel banks are readily defined by topographic, vegetative, and geomorphic characteristics. In Figure 2, 100-year channel bank stations are less obvious, and the depth/velocity criteria are used. Note that Figure 2 illustrates an example of composite flow.

- **Objective.** Define channel bank stations prior to supercritical floodway modeling.

- **Discussion.** See Figures 1 and 2.

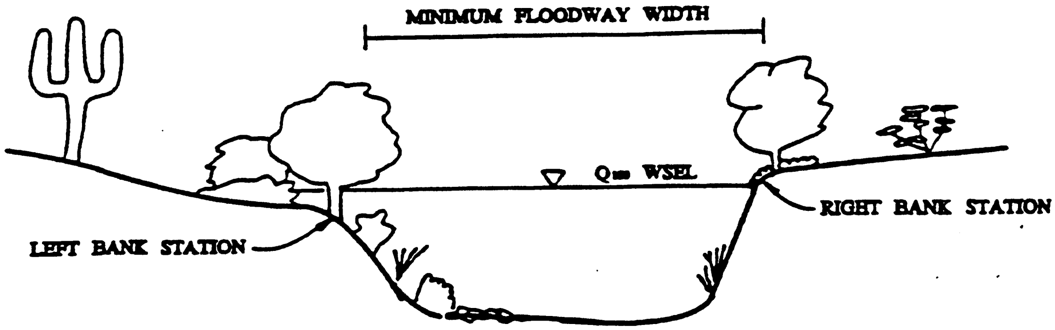


FIGURE 1

CHANNEL BANK STATION DESIGNATION

SIMPLE CHANNEL - DEFINED CHANNEL BANKS USING:

1. SLOPE BREAK
2. VEGETATION

(ILLUSTRATION NOT TO SCALE)

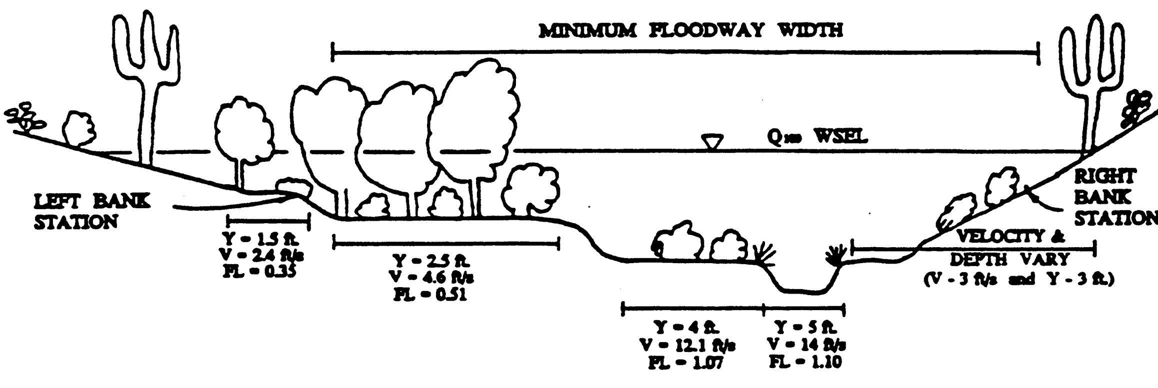


FIGURE 2

CHANNEL BANK STATION DESIGNATION

COMPLEX CHANNEL BANK STATIONS DEFINED AS THE POINTS WHERE FLOW DEPTH BECOMES LESS THAN 3 FT. AND FLOW VELOCITY BECOMES LESS THAN 3 FT/S.

(ILLUSTRATION NOT TO SCALE)

Figure 2 and Figure 2

Example 2: High-velocity, Near Critical Flow

- **Problem Statement.** Cross sections and a plan view profile of a stream is shown in Figures 3 and 4. HEC-2 modeling for a stream indicates critical depth for both subcritical and supercritical profiles, as shown in Figure 5. Tests for stability are outlined. Floodway limits are determined using the energy grade line approach.

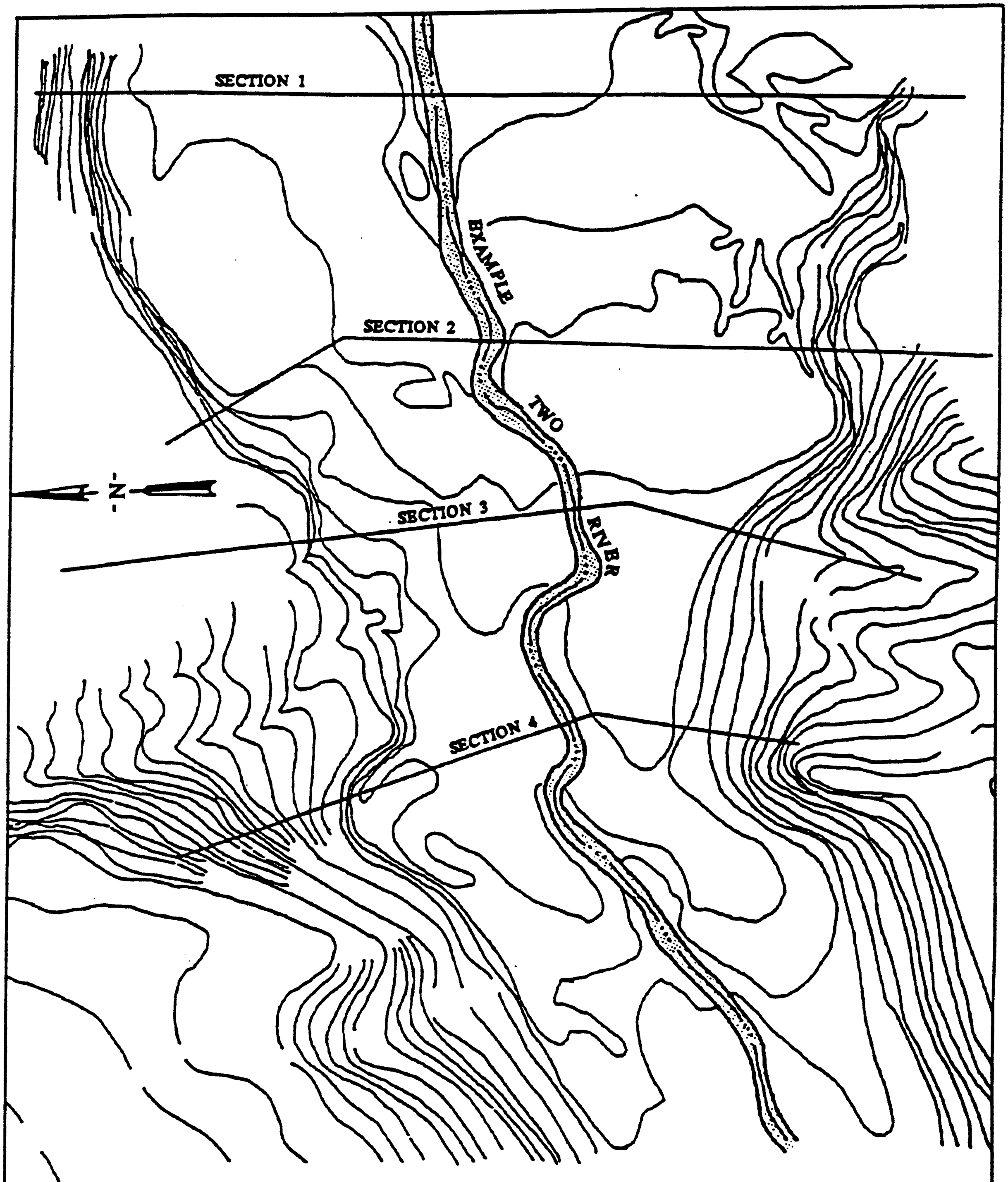
- **Objectives.** (1) Determine if subcritical or supercritical flow occurs, (2) determine if HEC-2 model is computationally stable, and (3) determine floodway limits.

- **Discussion.** The HEC-2 model defaulted to critical depth at three of four cross sections when a subcritical flow regime was assumed (See Table 1). According to the guidelines since more than 40% of the sections were assumed critical, the supercritical floodway modeling guidelines should be used. A supercritical HEC-2 model also assumes critical depth at three of four cross sections (See Table 2). Velocities for both runs average 11.5 feet per second (fps). (However, note the difference in channel velocities computed for the supercritical and critical runs.) Therefore, the profile qualifies as high-velocity, near-critical flow.

  According to the guidelines, additional cross sections should be added, energy coefficients checked, and the model calibrated to insure that the model is computationally stable. A check of the HEC-2 model output indicates that velocity head is less than 1/3 the flow depth for all of the subcritical run. (However, velocity head is not greater than 2/3 the depth for the supercritical run. Therefore, the supercritical run may not be stable.) Additional cross sections were added by interpolation (J1.7=0.1), but did not change computation of critical depth at surveyed cross sections. There is no basis for adjusting energy loss coefficients, or no data for calibration. Therefore, the subcritical HEC-2 model must be assumed to be computationally stable.

  Once the model is checked for stability, the floodway modeling may begin using the subcritical profile HEC-2 model. Encroachment method 6 is used to determine the change in energy grade line, rather than water surface elevation used by method 4, to estimate floodway limits. Encroachment method 6 will not allow encroachment within the channel bank stations. Encroachment stations and floodway data are shown in Table 3. For comparison, floodway data determined using encroachment method 4 are shown in Table 4. Note that use of encroachment method 4 results in a narrower floodway, higher floodway velocities, and decreases in floodway water surface elevation at two of four cross sections. Natural and floodway water surface elevations are shown on the cross section plots in Figures 4a to 4d. HEC-2 input files are shown in Tables 5 through 8.

Note: Floodway velocities for design should be taken from the supercritical run, not the floodway run. Compare Tables 2 and 4.



PLAN VIEW OF STREAM IN EXAMPLE 2
(ILLUSTRATION NOT TO SCALE)

Figure 3

FIGURE 4a
NEAR CRITICAL FLOW
CROSS SECTION 1

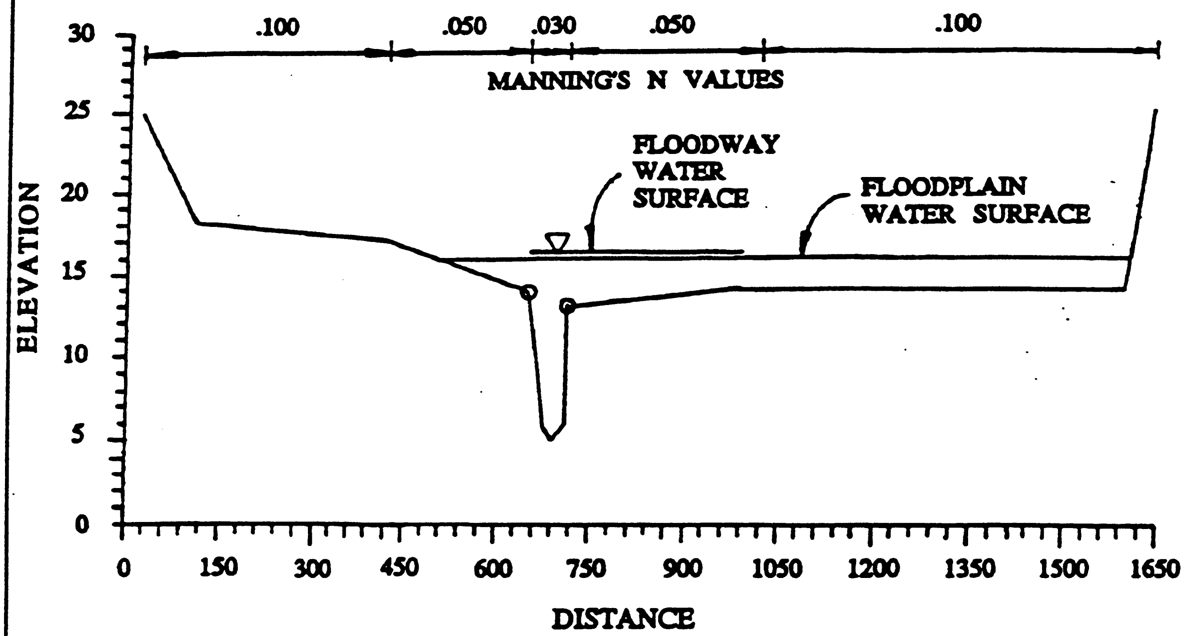


FIGURE 4b
NEAR CRITICAL FLOW
CROSS SECTION 2

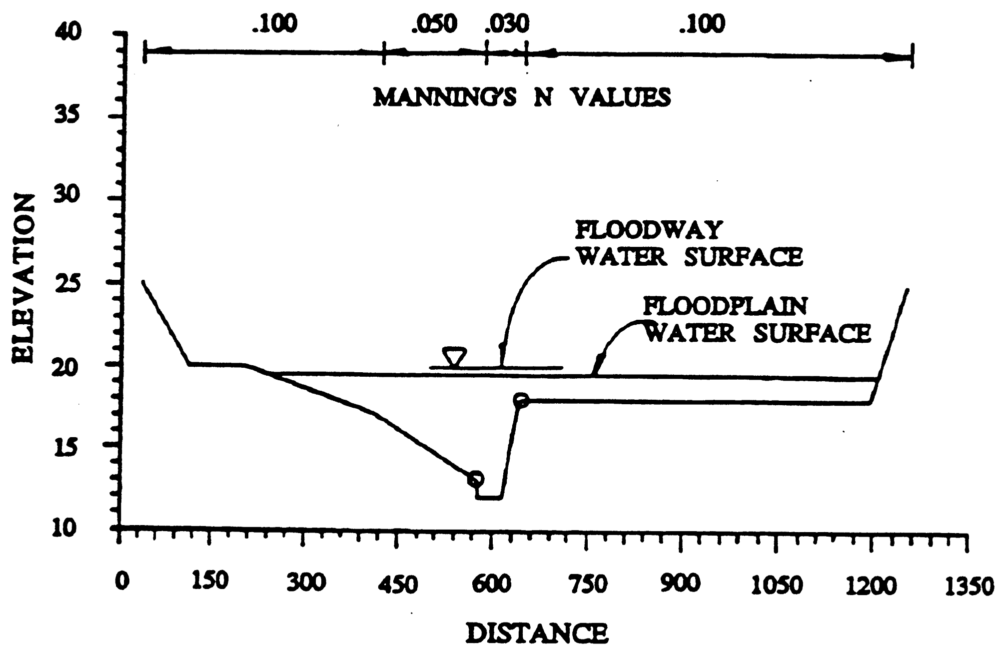


Figure 4

FIGURE 4c
NEAR CRITICAL FLOW
CROSS SECTION 3

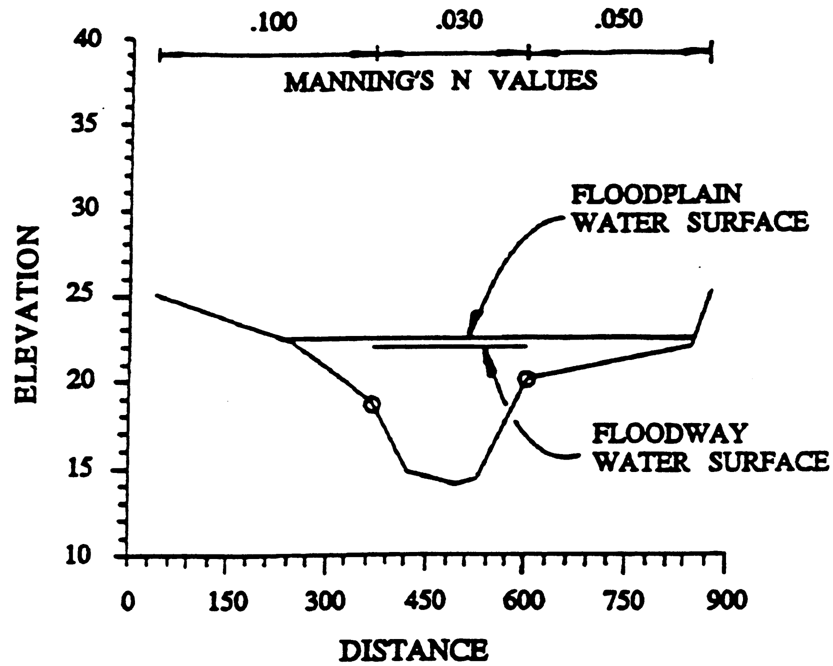


FIGURE 4d
NEAR CRITICAL FLOW
CROSS SECTION 4

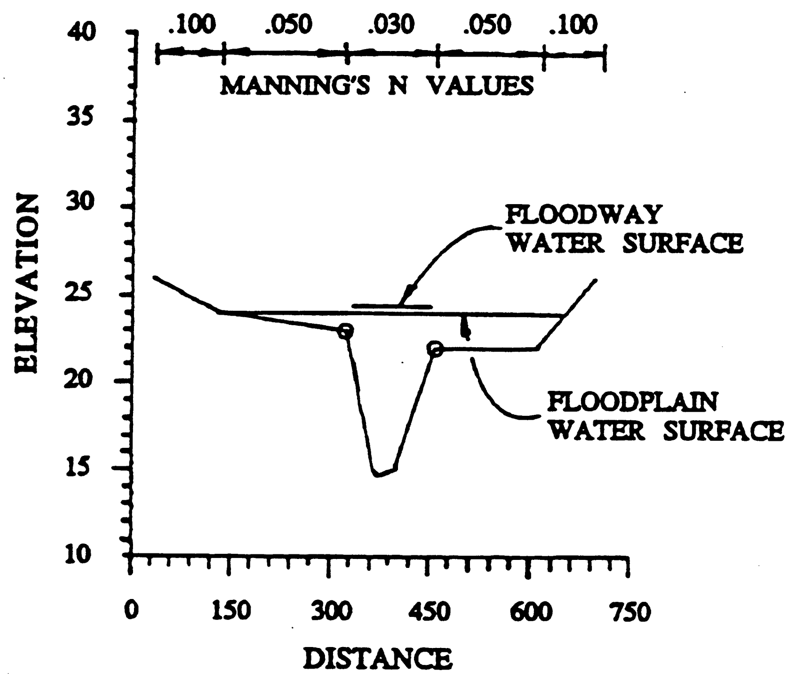
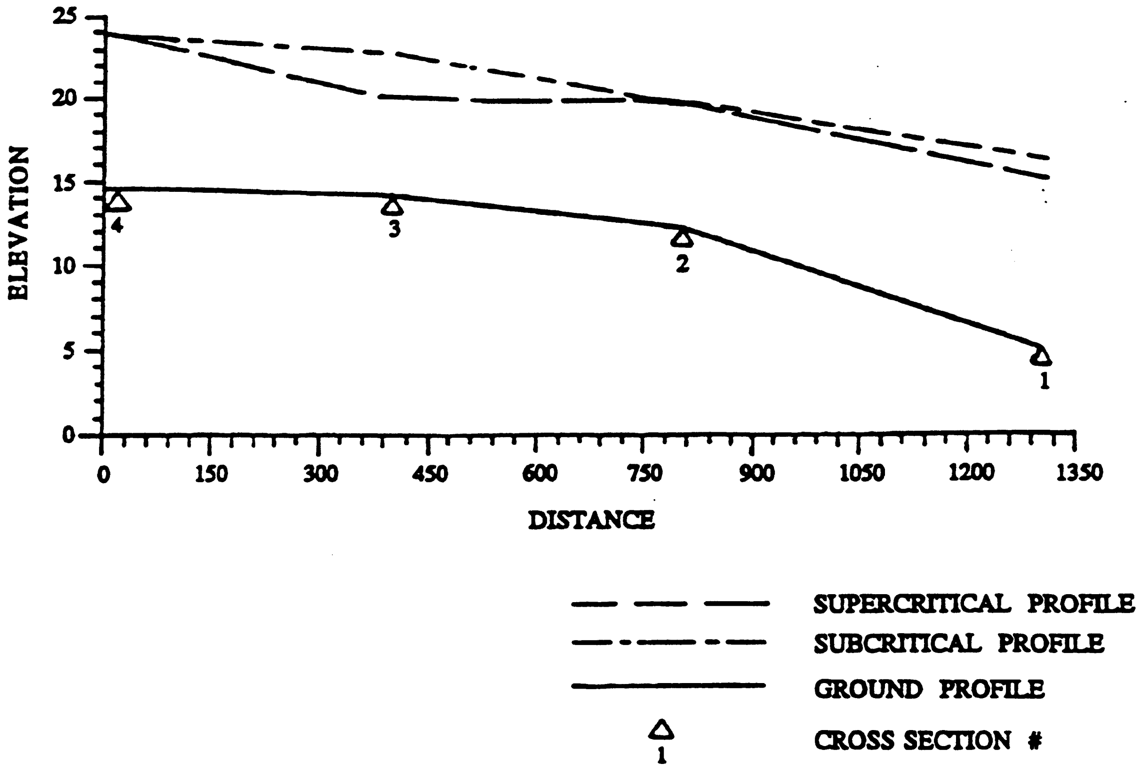


Figure 4



SUBCRITICAL AND SUPERCRITICAL PROFILES
FOR EXAMPLE 2

Figure 5

```
*******************************************
 HEC-2 WATER SURFACE PROFILES              *
*                                          *
* Version   4.6.2;  May 1991               *
*                                          *
* RUN DATE    26APR94    TIME    11:01:39  *
*******************************************
```

```
****************************************
* U.S. ARMY CORPS OF ENGINEERS         *
* HYDROLOGIC ENGINEERING CENTER        *
* 609 SECOND STREET, SUITE D           *
* DAVIS, CALIFORNIA 95616-4687         *
*          (916) 756-1104              *
****************************************
```

NOTE- ASTERISK (*) AT LEFT OF CROSS-SECTION NUMBER INDICATES MESSAGE IN SUMMARY OF ERRORS LIST

CRITICAL FLOW

SUMMARY PRINTOUT

	SECNO	Q	CWSEL	CRIWS	HV	DEPTH	TOPWID	ALPHA	KRATIO	QCH	VCH	FRCH
*	1.000	10000.00	16.02	16.02	1.09	11.02	1106.85	4.82	.00	5682.49	10.95	.66
*	2.000	10000.00	19.38	19.38	1.18	7.38	961.38	3.29	1.00	4759.89	11.84	.84
*	3.000	10000.00	22.46	.00	.55	8.36	627.57	1.59	2.17	9299.16	6.17	.42
*	4.000	10000.00	23.95	23.95	1.61	9.45	514.23	1.59	1.00	8767.24	10.81	.76

Table 1. Example #2, Subcritical Flow HEC-2 Run Summary Printout.

```
*********************************************
* HEC-2 WATER SURFACE PROFILES              *
*                                           *
* Version  4.6.2;  May 1991                 *
*                                           *
* RUN DATE   26APR94    TIME   11:02:05     *
*********************************************
```

```
**************************************
* U.S. ARMY CORPS OF ENGINEERS       *
* HYDROLOGIC ENGINEERING CENTER      *
* 609 SECOND STREET, SUITE D         *
* DAVIS, CALIFORNIA 95616-4687       *
*        (916) 756-1104              *
**************************************
```

NOTE- ASTERISK (*) AT LEFT OF CROSS-SECTION NUMBER INDICATES MESSAGE IN SUMMARY OF ERRORS LIST

CRITICAL FLOW

SUMMARY PRINTOUT

	SECNO	Q	CWSEL	CRIWS	HV	DEPTH	TOPWID	ALPHA	KRATIO	QCH	VCH	FRCH
*	4.000	10000.00	23.95	23.95	1.61	9.45	514.36	1.59	.00	8766.73	10.81	.76
*	3.000	10000.00	19.77	19.77	1.96	5.67	262.07	1.04	1.08	9983.10	11.24	1.00
*	2.000	10000.00	19.30	19.30	1.26	7.30	955.27	3.25	1.06	4819.30	12.13	.87
*	1.000	10000.00	14.78	16.06	3.94	9.78	1004.69	4.39	1.07	7929.97	17.84	1.15

Table 2. Example #2, Supercritical Flow HEC-2 Run Summary Printout.

```
 *****************************************            ****************************************
 - HEC-2 WATER SURFACE PROFILES            *            * U.S. ARMY CORPS OF ENGINEERS           *
 *                                         *            * HYDROLOGIC ENGINEERING CENTER          *
 * Version   4.6.2;  May 1991              *            * 609 SECOND STREET, SUITE D             *
 *                                         *            * DAVIS, CALIFORNIA 95616-4687           *
 * RUN DATE    26APR94     TIME   08:59:23 *            *          (916) 756-1104                *
 *******************************************            ****************************************
```

NOTE- ASTERISK (*) AT LEFT OF CROSS-SECTION NUMBER INDICATES MESSAGE IN SUMMARY OF ERRORS LIST

CRITICAL FLOW

SUMMARY PRINTOUT

	SECNO	Q	CWSEL	CRIWS	EG	TOPWID	STENCL	STENCR	DEPTH	HV	QCH	VCH
*	1.000	10000.00	16.02	16.02	17.11	1106.85	.00	.00	11.02	1.09	5682.49	10.95
*	1.000	10000.00	16.24	16.24	18.11	339.49	650.00	989.49	11.24	1.88	6894.38	12.96
*	2.000	10000.00	19.38	19.38	20.56	961.38	.00	.00	7.38	1.18	4759.89	11.84
	2.000	10000.00	19.53	19.24	21.56	260.50	463.87	724.36	7.53	2.03	5715.21	13.88
*	3.000	10000.00	22.46	.00	23.01	627.57	.00	.00	8.36	.55	9299.16	6.17
*	3.000	10000.00	22.08	.00	22.85	230.00	370.00	600.00	7.98	.77	10000.00	7.05
*	4.000	10000.00	23.95	23.95	25.56	514.23	.00	.00	9.45	1.61	8767.24	10.81
	4.000	10000.00	24.16	.00	26.37	130.00	330.00	460.00	9.66	2.21	10000.00	11.93

Table 3. Example #2, Floodway Encroachment Method 6 HEC-2 Summary Printout.

```
*********************************************
* HEC-2 WATER SURFACE PROFILES              *
*                                           *
* Version   4.6.2;  May 1991                *
*                                           *
* RUN DATE    26APR94     TIME    09:25:42  *
*********************************************
```

```
**************************************
* U.S. ARMY CORPS OF ENGINEERS       *
* HYDROLOGIC ENGINEERING CENTER      *
* 609 SECOND STREET, SUITE D         *
* DAVIS, CALIFORNIA 95616-4687       *
*          (916) 756-1104            *
**************************************
```

NOTE- ASTERISK (*) AT LEFT OF CROSS-SECTION NUMBER INDICATES MESSAGE IN SUMMARY OF ERRORS LIST

CRITICAL FLOW

SUMMARY PRINTOUT

	SECNO	Q	CWSEL	CRIWS	EG	TOPWID	STENCL	STENCR	DEPTH	HV	QCH	VCH
*	1.000	10000.00	16.02	16.02	17.11	1106.85	.00	.00	11.02	1.09	5682.49	10.95
*	1.000	10000.00	16.34	16.34	18.44	276.16	650.00	926.16	11.34	2.11	7227.82	13.44
*	2.000	10000.00	19.38	19.38	20.56	961.38	.00	.00	7.38	1.18	4759.89	11.84
	2.000	10000.00	19.35	19.32	22.06	167.18	487.44	654.63	7.35	2.70	6204.29	15.50
*	3.000	10000.00	22.46	.00	23.01	627.57	.00	.00	8.36	.55	9299.16	6.17
*	3.000	10000.00	22.59	.00	23.24	230.00	370.00	600.00	8.49	.66	10000.00	6.51
*	4.000	10000.00	23.95	23.95	25.56	514.23	.00	.00	9.45	1.61	8767.24	10.81
*	4.000	10000.00	23.34	23.34	26.24	130.00	330.00	460.00	8.84	2.90	10000.00	13.66

Table 4. Example #2, Floodway Encroachment Method 4 HEC-2 Run Summary Printout.

```
T1 SUPERCRITICAL FLOODWAY STATE STANDARD
T2 EXAMPLE #2 - AKA RED FOX RIVER, HEC2 TRAINING WORKSHOP 3A/3B
T3 NEAR CRITICAL FLOW                SUBCRITICAL RUN
J1               2                    .014              .1              3.8
J2      -1              -1                                                     15
J3      38      43       1       2      10       8       4      57      58      14
J3      26      68
J6       1
QT       1   10000
NC                              .1      .3
NH       5      .1     415     .05     650     .03     710     .05    1020      .1
NH    1635
X1       1      11     650     710
GR      25      20      18     110      17     415      14     650       6     675
GR       5     690       6     710      13     710      14    1020      14    1590
GR      25    1635
NH       4      .1     415     .05     575     .03     640      .1    1250
X1       2      10     575     640     500     500     500
GR      25      30      20     110      20     200      17     415      13     575
GR      12     580      12     615      18     640      18    1195      25    1250
NC      .1     .05     .03
X1       3      10     370     600     400     400     400
GR      25      40      22     260    18.7     370      15     420    14.1     500
GR    14.5     530    17.3     560      20     600      22     850      25     875
NH       5      .1     130     .05     330    .036     460     .05     610      .1
NH     700
X1       4       8     330     460     400     400     400
GR      26      30      24     130      23     330    14.5     370      15     400
GR      22     460      22     610      26     700
EJ
ER
```

Table 5. Example #2, Subcritical Flow HEC-2 Run Data Input File.

```
T1 SUPERCRITICAL FLOODWAY STATE STANDARD
T2 EXAMPLE #2 - AKA RED FOX RIVER, HEC2 TRAINING WORKSHOP 3A/3B
T3 NEAR CRITICAL FLOW                SUPERCRITICAL RUN
J1                2               1    .001               .1               3.8
J2      -1                -1                                                       15
J3      38       43        1      2      10       8        4      57      58      14
J3      26       68
J6       1
QT       1    10000
NC                                .1      .3
NH       5       .1      130    .05     330    .036      460     .05     610      .1
NH     700
X1       4        8      330    460     400     400      400
GR      26       30       24    130      23     330     14.5     370      15     400
GR      22      460       22    610      26     700
NC      .1      .05      .03
X1       3       10      370    600     400     400      400
GR      25       40       22    260    18.7     370       15     420    14.1     500
GR    14.5      530     17.3    560      20     600       22     850      25     875
NH       4       .1      415    .05     575     .03      640      .1    1250
X1       2       10      575    640     500     500      500
GR      25       30       20    110      20     200       17     415      13     575
GR      12      580       12    615      18     640       18    1195      25    1250
NH       5       .1      415    .05     650     .03      710     .05    1020      .1
NH    1635
X1       1       11      650    710
GR      25       20       18    110      17     415       14     650       6     675
GR       5      690        6    710      13     710       14    1020      14    1590
GR      25     1635
EJ
ER
```

Table 6. Example #2, Supercritical Flow HEC-2 Run Data Input File.

```
T1 SUPERCRITICAL FLOODWAY STATE STANDARD
T2 EXAMPLE #2 - AKA RED FOX RIVER, HEC2 TRAINING WORKSHOP 3A/3B
T3 NEAR CRITICAL FLOW          FLOODWAY RUN ENCROACHMENT METHOD 6
J1             2                .014             .1             3.8
J2       1             -1                                                15
J3      38    43        1       2       3       4      27      28       8      10
J3      14    26
J6       1
NC                              .1      .3
QT       2 10000    10000
ET                   10.6
NH       5    .1      415     .05     650     .03     710     .05    1020      .1
NH    1635
X1       1    11      650     710
GR      25    20       18     110      17     415      14     650       6     675
GR       5   690        6     710      13     710      14    1020      14    1590
GR      25  1635
NH       4    .1      415     .05     575     .03     640      .1    1250
X1       2    10      575     640     500     500     500
GR      25    30       20     110      20     200      17     415      13     575
GR      12   580       12     615      18     640      18    1195      25    1250
NC      .1   .05      .03
X1       3    10      370     600     400     400     400
GR      25    40       22     260    18.7     370      15     420    14.1     500
GR    14.5   530     17.3     560      20     600      22     850      25     875
NH       5    .1      130     .05     330    .036     460     .05     610      .1
NH     700
X1       4     8      330     460     400     400     400
GR      26    30       24     130      23     330    14.5     370      15     400
GR      22   460       22     610      26     700
EJ
T1 SUPERCRITICAL FLOODWAY STATE STANDARD
T2 EXAMPLE #4 - AKA RED FOX RIVER, HEC2 TRAINING WORKSHOP 3A/3B
T3 COMPOSITE FLOW -      FLOODWAY RUN
J1             3                                 .1             16.02
J2      15             -1                                                15
ER
```

Table 7. Example #2, Floodway Encroachment Method 6 HEC-2 Run Data Input File.

```
T1 SUPERCRITICAL FLOODWAY STATE STANDARD
T2 EXAMPLE #2 - AKA RED FOX RIVER, HEC2 TRAINING WORKSHOP 3A/3B
T3 NEAR CRITICAL FLOW        FLOODWAY RUN ENCROACHMENT METHOD 4
J1                2                    .014              .1               3.8
J2       1                -1                                                       15
J3      38       43        1       2       3       4      27      28       8      10
J3      14       26
J6       1
NC                                .1      .3
QT       2    10000    10000
ET                      10.4
NH       5       .1      415     .05     650     .03     710     .05    1020      .1
NH    1635
X1       1       11      650     710
GR      25       20       18     110      17     415      14     650       6     675
GR       5      690        6     710      13     710      14    1020      14    1590
GR      25     1635
NH       4       .1      415     .05     575     .03     640      .1    1250
X1       2       10      575     640     500     500     500
GR      25       30       20     110      20     200      17     415      13     575
GR      12      580       12     615      18     640      18    1195      25    1250
NC      .1      .05      .03
X1       3       10      370     600     400     400     400
GR      25       40       22     260    18.7     370      15     420    14.1     500
GR    14.5      530     17.3     560      20     600      22     850      25     875
NH       5       .1      130     .05     330    .036     460     .05     610      .1
NH     700
X1       4        8      330     460     400     400     400
GR      26       30       24     130      23     330    14.5     370      15     400
GR      22      460       22     610      26     700
EJ
T1 SUPERCRITICAL FLOODWAY STATE STANDARD
T2 EXAMPLE #4 - AKA RED FOX RIVER, HEC2 TRAINING WORKSHOP 3A/3B
T3 COMPOSITE FLOW -     FLOODWAY RUN
J1                3                                      .1             16.02
J2      15                -1                                                      15
ER
```

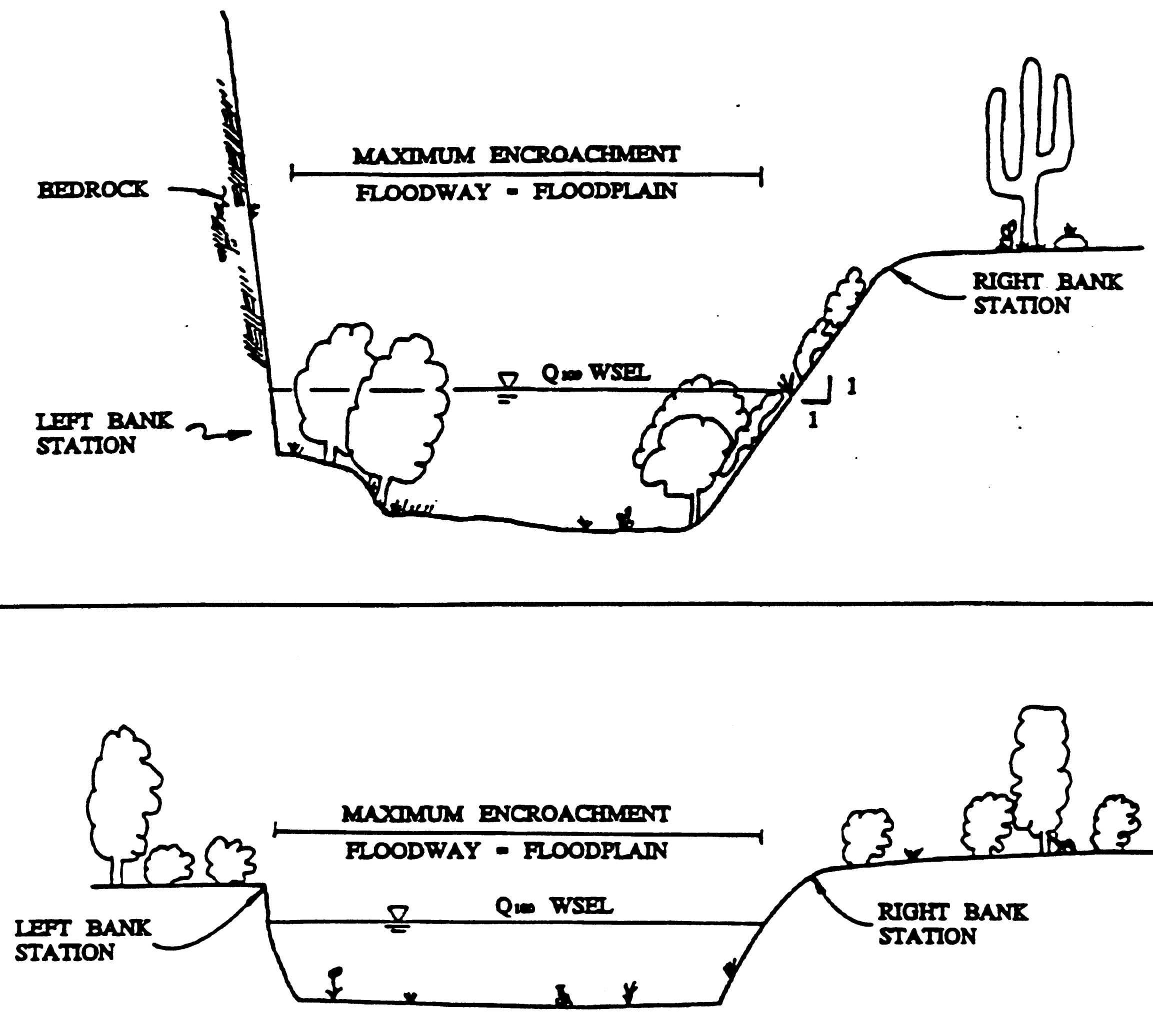
Table 8. Example #2, Floodway Encroachment Method 4 HEC-2 Run Data Input File.

Example 3: Channelized Supercritical Flow

- **Problem Statement.** Supercritical flow within two confined channels are illustrated in Figure 6. No floodway analysis is needed, since floodway limits are the floodplain limits.

- **Objective.** Illustrate examples of channelized supercritical flow.

- **Discussion.** Encroachment within the confined channel would be hazardous due to high velocities, the potential to cause hydraulic jumps, and disruption of channel processes. Current federal regulations prevent definition of floodway limits within channel boundaries. Also, only a very limited area within the banks would have depths and velocities less than 3 feet and 3 fps. Supercritical HEC-2 modeling would demonstrate the presence of supercritical flow at most sections in the reach. Floodplain limits would be determined using the subcritical HEC-2 profile. Design velocities should be obtained from the supercritical HEC-2 profile. No floodway modeling would be required.



CHANNELIZED SUPERCRITICAL FLOW
EXAMPLES OF NO FLOODWAY ENCROACHMENT ALLOWED

(ILLUSTRATIONS NOT TO SCALE)

Figure 6

Example 4: Composite Flow

- **Problem Statement.** The stream shown in Example 2 is tested for composite flow. Refer to Figures 3 and 4. Elements of composite flow are illustrated.

- **Objectives.** Demonstrate composite flow tests.

- **Discussion.** The test for composite flow follows the procedure described by Schoellhamer (1986) and uses equations developed in Blalock (1981). Copies of articles by Schoellhamer and Blalock are attached. The example problem is modified from a HEC-2 training problem supplied with the HEC-2 program, and was discussed in Schoellhamer. The procedure involves computation of the subdivision Froude number. The subdivision Froude number describes the ratio of gravitational to inertial forces within segments of a cross section, rather than as an average of the entire cross section. The subdivision Froude number is calculated for each cross section segment to determine if portions are supercritical and portions are subcritical.

  In order to apply the subdivision Froude number procedure, certain hydraulic variables are required. These variables include the total discharge, the energy slope, the topwidth, the left and right end stations of flow, the water surface elevation, cross section conveyance, and total flow area. For the subdivision sections, many of these variables are listed in the detailed output summaries in the HEC-2 output. A trace was requested in the HEC-2 input file (J3.10 = 15) to obtain hydraulic variables for each subdivision of the cross section. Variables requested for output are shown in Table 1 (See Example 2).

  The basic equation for subdivision Froude number is:

$$F_i = \left(\frac{\alpha V_i}{g A_i} \left[\frac{Q_t}{K_t^2} \left(K_i \frac{dK_t}{dy} - K_t \frac{dK_i}{dy} \right) + V_i T_i \right] - \frac{V_i^2}{2g} \frac{da}{dy} \right)^{0.5} \quad ; \textit{ where:}$$

F_i = subdivision Froude number, dimensionless
α = velocity coefficient alpha (Coriolis coefficient)
V_i = subdivision velocity, ft/sec
g = gravitational acceleration, ft/sec^2
A_i = subdivision area, ft^2
A_t = total cross section area, ft^2
P_t = total cross section wetted perimeter, ft
P_i = subdivision cross section wetted perimeter, ft
T_i = subdivision topwidth, ft
Q_t = discharge within total cross section, ft^3/sec
K_t = conveyance of total cross section, ft^3/sec
= $(1.49/n_t)A_tR_t^{0.67}$; where:
n_t = Manning's roughness for total section
R_t = hydraulic radius, ft for total section
= A_t/P_t

K_i = subdivision conveyance, ft^3/sec
= $(1.49/n_i)A_iR_i^{0.67}$; where:
n_i = subdivision Manning's roughness
R_i = subdivision hydraulic radius, ft
= A_i/P_i

dK_i/dy = derivative of subdivision conveyance
= $0.33(K_i/A_i)[5T_i - 2R_i\, dp_i/dy]$; where:
dp_i/dy = measured directly, see Blalock (1981)

dK_t/dy =derivative of total conveyance
= $0.33(K_t/A_t)[5T_t - 2R_t\, dp_t/dy]$; where:
dp_t/dy = measured directly, see Blalock (1981)

$d\alpha/dy$ = derivative of the Coriolis coefficient
= $A_t^2 s_1/K_t^3 + s_2(2A_tT_t/K_t^3 - A_t^2 s_3/K_t^4)$; where:
s_1 = $[\,(K_i/A_i)^3\,(3T_i - 2R_i\, dp_i/dy)]$
s_2 = (K_i^3/A_i^2)
s_3 = $[\,(K_i/A_i)\,(5T_i - 2R_i\, dp_i/dy)]$

Subdivision Froude numbers were calculated using the equations shown above for the example cross sections, as shown in Tables 9a-d. Unreal[6] values of the subdivision Froude number indicate subcritical flow. Composite flow was found to exist at each of the sections in the example.

Floodway computations performed.

[6] Unreal, or imaginary numbers, occur when the main term of the basic subdivision Froude number is negative. The square root of a negative number is unreal.

Table 9a
Channel Subdivision
Cross Section 1

GR Station	491	650	710	1,020	1,590	1,598
GR Elevation	16.02	14	13	14	14	16.0

Manning's	n:	0.05	0.03	0.05	0.1	0.1
Discharge	q:	270	5680	2470	1570	10
Flow Area	A:	160.5	519	782.5	1153.8	8.4
Mean Velocity	v:	1.7	10.9	3.2	1.4	0.8
Depth,	y:	1	8.6	2.5	2	1
Topwidth	TW:	159	60	310	570	8
Wetted Perimeter	P:	159	68	310	570	8
Hydraulic Radius	R:	1.0	7.6	2.5	2.0	1.1
Conveyance	k:	4813	99992	43243	27516	129
	dp/dy:	159	0	0	0	8
	S_1:	4205922	1287262432	156956450	23193833	24337
	S_2:	4327965	3711606680	132062820	15649734	30636
	S_3:	14213	57799	85657	67968	349
	dk/dy:	4738	19266	28552	22656	116
Subdivision Froude#	F:	0.45	2.64	0.56	0.21	0.21

Table 9b
Channel Subdivision
Cross Section 2

GR Station		244	415	575	640	1,195	1,206
GR Elevation		19.38	17	13	18	18	19.3

Manning's	n:	0.1	0.05	0.03	0.1	0.1
Discharge	q:	240	3980	4760	1010	10
Flow Area	A:	202.9	700.7	402.2	756.7	7.5
Mean Velocity	v:	1.2	5.7	11.8	1.3	0.8
Depth,	y:	1.2	4.4	6.2	1.4	0.7
Topwidth	TW:	171	160	65	555	11
Wetted Perimeter	P:	171	160	66	555	11
Hydraulic Radius	R:	1.2	4.4	6.1	1.4	0.7
Conveyance	k:	3389	55920	66686	13865	87
	dp/dy:	171	0	0	0	11
	S_1:	499161	243980834	888813847	10241632	27489
	S_2:	945120	356161188	1833235535	4654560	11529
	S_3:	7501	63845	53886	50845	460
	dk/dy:	2500	21282	17962	16948	153
Subdivision Froude#	F:	0.26	0.78	1.81	Unreal #	Unreal #

Table 9c
Channel Subdivision
Cross Section 3

GR Station	226	260	370	600	850	854
GR Elevation	22.46	22	18	20	22	22.4

Manning's	n:	0.1	0.1	0.03	0.05	0.05
Discharge	q:	50	200	9300	500	50
Flow Area	A:	7.8	232.1	1507.3	365	0.9
Mean Velocity	v:	0.2	0.9	6.2	1.4	0.4
Depth,	y:	0.2	2.1	6.6	1.5	0.3
Topwidth	TW:	34	110	230	250	4
Wetted Perimeter	P:	34	110	230	250	4
Hydraulic Radius	R:	0.2	2.1	6.6	1.5	0.2
Conveyance	k:	44	5691	262332	14000	10
	dp/dy:	34	0	0	0	4
	S_1:	15020	4863646	3637511122	42323358	13629
	S_2:	1356	3420764	7946116688	20597368	1204
	S_3:	862	13485	200147	47946	200
	dk/dy:	287	4495	66716	15982	67
Subdivision Froude#	F:	Unreal #	Unreal #	0.54	Unreal #	Unreal #

Table 9d
Channel Subdivision
Cross Section 4

GR Station	140	330	460	610	654
GR Elevation	23.95	23	22	22	23.9

Manning's	n:	0.05	0.036	0.05	0.1
Discharge	q:	130	8770	1060	50
Flow Area	A:	90.6	811.2	292.7	42.8
Mean Velocity	v:	1.4	10.8	3.6	1.1
Depth,	y:	0.5	6.2	2	1
Topwidth	TW:	190	130	150	44
Wetted Perimeter	P:	190	130	150	44
Hydraulic Radius	R:	0.5	6.2	2.0	1.0
Conveyance	k:	1647	113868	13624	626
	dp/dy:	190	0	0	44
	S_1:	2337761	1078649320	45374783	144964
	S_2:	544759	2243590585	29513776	133959
	S_3:	13980	91240	34908	1965
	dk/dy:	4660	30413	11636	655
Subdivision Froude#	F:	Unreal #	0.92	Unreal #	0.11

Example 5: Braided Flow

- **Problem Statement.** Figure 7 illustrates a braided flow situation which may or may not have supercritical flow. Maximum floodway limits are defined by the location of flow braids.

- **Objective.** Illustrate maximum floodway encroachment on a braided stream.

- **Discussion.** Since floodway limits cannot be located within designated channel bank stations, the minimum floodway width is the distance between the most distant flow braids. Substantial floodway widths may be defined using these guidelines. For this reason, floodway modeling of braided flow areas should be discussed with local floodplain administrators and review agencies. Where flow braids are separated by significant land areas not inundated by the base flood, modelers should refer to state standards for floodways around islands.

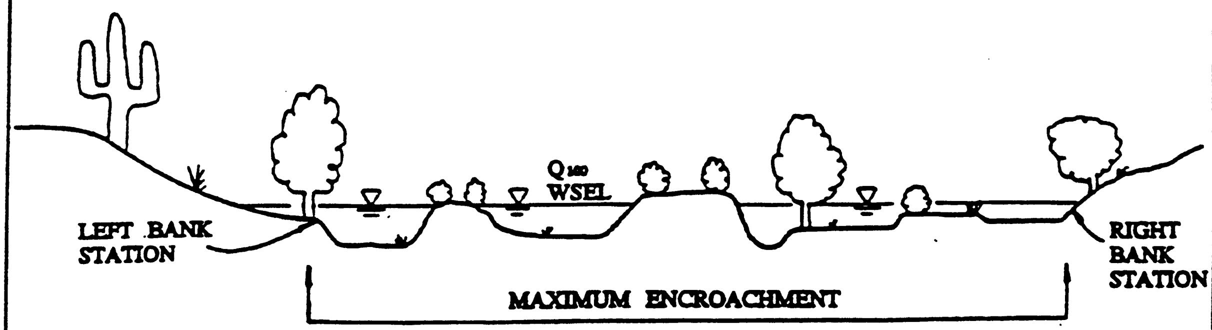


FIGURE 7a
BRAIDED FLOW

PROFILE

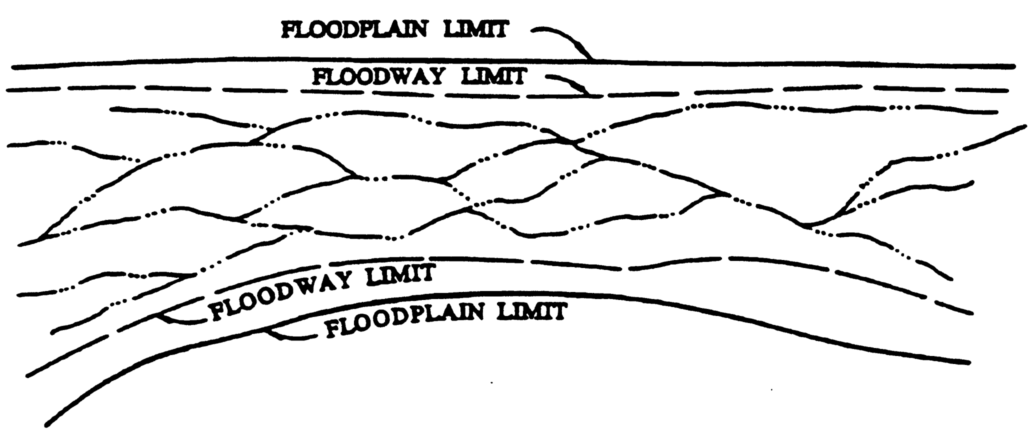


FIGURE 7b
BRAIDED FLOW

PLAN VIEW

MAXIMUM ENCROACHMENT AT MOST EXTREME BRAIDS
(ILLUSTRATIONS NOT TO SCALE)

Figure 7

Appendix

SUBDIVISION FROUDE NUMBER

By David H. Schoellhamer,[1] A. M. ASCE, John C. Peters,[2] and Bruce E. Larock,[3] Members, ASCE

INTRODUCTION

The standard step method calculates one-dimensional steady state water surface profiles by iterating upon the equations for energy conservation and head loss between adjacent cross sections (3). These calculations begin at and proceed away from the controlling boundary cross section. If the flow regime is subcritical the calculations proceed upstream from the downstream boundary, and if the flow regime is supercritical the calculations proceed downstream from the upstream boundary. But this procedure must in some sense be invalid for compound sections in which both flow regimes may occur in different portions of a cross section. Usually when this occurs, the flow in the main channel is in the supercritical regime and the flow in the overbanks is in the subcritical regime (6).

The development and testing of a subdivision Froude number with which the flow regime in each of the three major cross-sectional subdivisions (the two overbanks and the main channel) can be identified is described. This Froude number is compatible with HEC2, a widely used model that employs the standard step method (3,4). The determination of a Froude number for each flow subdivision can enhance the engineer's ability to evaluate the validity of a one-dimensional analysis.

FROUDE NUMBERS

The Froude number indicates the flow regime. A value less than one indicates subcritical flow, and a value of greater than one indicates supercritical flow. The simplest definition of the Froude number assumes a uniform velocity distribution so that

$$\mathsf{F} = \frac{V}{\sqrt{gD}} \qquad (1)$$

in which F = Froude number; V = mean velocity; g = gravitational acceleration; and D = hydraulic depth (area divided by top width) (5). A Froude number that considers a nonuniform velocity distribution is

$$\mathsf{F} = V\left(\frac{\alpha}{gD}\right)^{1/2} \qquad (2)$$

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in which α = Coriolis coefficient. Petryk and Grant (6) developed a Froude number that is the discharge-weighted average of the simple Froude number of Eq. 1 within every subsection. Blalock and Sturm (1) derived a composite Froude number that accounts for the variation of the Coriolis coefficient as a function of the water surface elevation.

Froude number is related to the slope of the specific energy curve. Both Henderson (5) and Blalock and Sturm (2) show for their Froude numbers that

$$\frac{dE}{dy} = 1 - F^2 \qquad (3)$$

in which E = the specific energy

$$E = y + \alpha \frac{V^2}{2g} \qquad (4)$$

and y = depth. Therefore, when the slope of the specific energy curve is positive, the flow is subcritical, and when the slope is negative, the flow is supercritical.

Subdivision Froude Number

A problem in developing a subdivision Froude number is that the discharge in a subdivision is dependent on the water surface elevation. Therefore the two simple Froude numbers that are defined by Eqs. 1 and 2 are not appropriate for subdivisions of a cross section. Considering subdivision discharge to be a function of the water surface elevation also invalidates the Froude number of Petryk and Grant (6), which Blalock and Sturm (1) showed was inaccurate. Blalock and Sturm's (1) composite Froude number is accurate for an entire cross section, but it is not accurate for subdivisions because it also fails to consider the change of subdivision discharge with water surface elevation.

A subdivision Froude number which allows the discharge to vary with the water surface elevation can be derived from the definition of specific energy. The derivative of specific energy in a subdivision with respect to depth is taken, and both the Coriolis coefficient and the subdivision velocity are assumed to vary with depth. The derivative is substituted into Eq. 3 to arrive at the expression for the subdivision Froude number

$$F = \left\{ \frac{\alpha V_{sd}}{g A_{sd}} \left[\frac{Q}{K^2} \left(K_{sd} \frac{dK}{dy} - K \frac{dK_{sd}}{dy} \right) + V_{sd} T_{sd} \right] - \frac{V_{sd}^2}{2g} \frac{d\alpha}{dy} \right\}^{1/2} \qquad (5)$$

in which V_{sd} = subdivision velocity; A_{sd} = subdivision area; Q = cross section discharge; K = cross section conveyance; K_{sd} = subdivision conveyance; and T_{sd} = subdivision top width. The derivatives of subdivision conveyance and Coriolis coefficient are given elsewhere (1,7). The complete derivation of Eq. 5 is given by Schoellhamer (7).

Blalock and Sturm used the same approach to derive their compound Froude number and showed that it was in agreement with experimental results (1). They later stated that use of a celerity that is derived from the method of characteristics produces the identical Froude number (2). Because the compound and subdivision Froude numbers are very similar, the method of characteristics would also be expected to show that the subdivision Froude number is correct. In addition, testing shows that the subdivision Froude number is compatible with both the velocity and the specific energy that one finds in a subdivision.

Testing Subdivision Froude Number

The sample trapezoidal cross section of Fig. 1 was initially used to test the subdivision Froude number (7). Five flow rates were tested—100, 1,000, 5,000, 10,000, and 50,000 cfs (1 cfs = 0.028 m^3/s). These flow rates represent extremely low flow, critical depth in the main channel, multiple critical depths, critical depth above the main channel, and extremely high flow, respectively. Each flow rate was tested over a wide range of depths. Two subdivision Froude numbers were calculated, one for the main channel and one for the two identical overbanks. In addition, both the specific energy (Eq. 4) and the derivative of the specific energy were calculated in both subdivisions.

The results of applying the subdivision Froude number to the main channel are very good. For the three largest flow rates, the subdivision Froude number correctly indicates the depth at which the specific energy in the main channel is a minimum, as shown in Table 1. The subdivision Froude number is also compatible with the calculated specific energy for all depths, thus demonstrating the validity of the energy approach used to derive the subdivision Froude number.

The results of applying the subdivision Froude number to the overbank are quite interesting. As shown in Table 2, when the depth in the overbank is very shallow, less than 1.3 ft (0.40 m) for this cross section, the derivative of specific energy with respect to depth is greater than one. This occurs because the velocity head in the overbank increases with depth up to 1.3 ft (0.40 m) and decreases for greater depths. And because the velocity distribution in the overbank is nearly uniform, the velocity behaves like the velocity head. The increase in velocity head over shallow depths in the overbank is intuitively reasonable.

Because the derivative of specific energy is greater than one, Eq. 3 shows that the Froude number squared is equal to a negative number. For this condition Eq. 5 shows that

$$K_{sd}\left(T_{sd} + \frac{A_{sd}}{K}\frac{dK}{dy} + \frac{A_{sd}^3}{2\alpha}\frac{d\alpha}{dy}\right) < A_{sd}\frac{dK_{sd}}{dy} \quad \text{(6)}$$

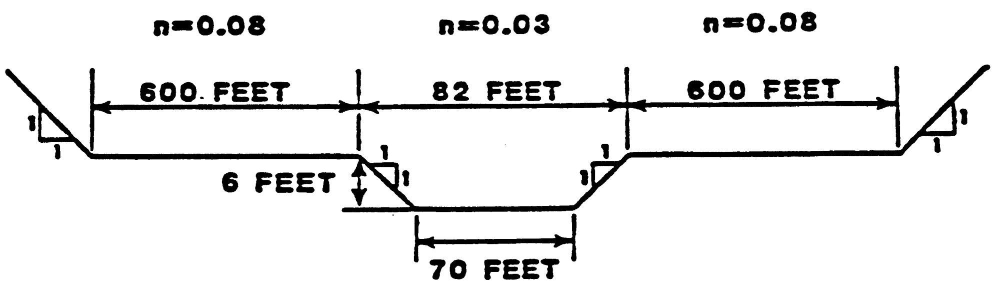

FIG. 1.—Trapezoidal Test Section (1 ft = 0.3 m)

TABLE 1.—Subdivision Froude Number, Main Channel Results[a]

Flow rate (cfs) (1)	Depth (ft) (2)	Subdivision F (3)	E (ft) (4)	dE/dy (5)
5,000	6.5	1.071	7.837	−0.146
	6.6	1.049	7.825	−0.100
	6.7	1.019	7.818	−0.038
	6.8	0.983	7.818	0.034
	6.9	0.944	7.825	0.108
	7.0	0.904	7.839	0.183
10,000	7.9	1.114	9.555	−0.240
	8.0	1.062	9.536	−0.128
	8.1	1.013	9.529	−0.027
	8.2	0.968	9.530	0.064
	8.3	0.924	9.541	0.145
	8.4	0.884	9.559	0.219
50,000	13.1	1.057	16.686	−0.116
	13.2	1.034	16.676	−0.069
	13.3	1.012	16.672	−0.025
	13.4	0.991	16.671	0.018
	13.5	0.971	16.675	0.058
	13.6	0.951	16.683	0.096

[a]1 cfs = 0.028 m^3/s, 1 ft = 0.3 m.

TABLE 2.—Subdivision Froude Number, Overbank Results[a]

Flow (cfs) (1)	Depth (ft) (2)	Velocity (fps) (3)	Subdivision F (4)	E (ft) (5)	dE/dy (6)
5,000	1.0	0.827	[b]	1.011	1.003
	1.1	0.836	[b]	1.111	1.002
	1.2	0.840	[b]	1.211	1.001
	1.3	0.841	0.010	1.311	1.000
	1.4	0.840	0.027	1.411	0.999
	1.5	0.836	0.035	1.511	0.999
10,000	1.0	1.655	[b]	1.043	1.011
	1.1	1.672	[b]	1.143	1.007
	1.2	1.681	[b]	1.244	1.003
	1.3	1.683	0.019	1.344	1.000
	1.4	1.680	0.054	1.444	0.997
	1.5	1.672	0.070	1.543	0.995
50,000	1.0	8.273	[b]	2.063	1.278
	1.1	8.359	[b]	2.185	1.164
	1.2	8.403	[b]	2.296	1.069
	1.3	8.414	0.095	2.399	0.991
	1.4	8.398	0.268	2.495	0.928
	1.5	8.360	0.349	2.585	0.878

[a]1 cfs = 0.028 m^3/s, 1 fps = 0.3 m/s, 1 ft = 0.3 m.
[b]Imaginary number.
Note: The datum for depth and specific energy is the bottom of the overbank.

Eq. 6 shows that the range of depths over which the subdivision Froude number is imaginary is independent of the cross section discharge. This independence has already been implicitly assumed and is confirmed by the results.

When the two sides of Eq. 6 are equal, the subdivision Froude number equals zero and the derivative of specific energy equals one. The depth at which the derivative in the overbank exactly equals one is the depth at which the derivative of the velocity head in Eq. 5 equals zero. This is the depth of maximum overbank velocity head, which for all practical purposes is the depth of maximum overbank velocity, as verified by Table 2.

Thus an imaginary subdivision Froude number indicates that the velocity head is increasing with depth, and therefore the depth in the floodplain is relatively shallow. For this condition it can be concluded that the flow in the overbanks is subcritical because the derivative of specific energy is positive. An imaginary subdivision Froude number may indicate that the overbank flow is too shallow to be modeled properly by the standard step method.

Five test problems containing 193 cross sections were run with a modified version of HEC2 which calculated subdivision Froude numbers. The first test problem was the Red Fox River, which is a problem used by the Hydrologic Engineering Center in training courses on HEC2. Four other test cases were chosen from the test data that is provided to users with each copy of the program (4). These tests (numbers 1, 5, 14, and 15) provided a wide variety of both natural and artificial cross sections. Of the cross sections tested, eleven had a mixed flow regime and 36 had at least one imaginary subdivision Froude number.

Conclusion

A subdivision Froude number has been developed and tested. A knowledge of the magnitude of the subdivision Froude numbers improves the engineer's ability to identify mixed flow regimes and shallow floodplain flow, both of which invalidate the assumptions of the standard step method. A two-dimensional analysis is probably more appropriate in these circumstances.

Acknowledgments

The Hydrologic Engineering Center provided computer facilities used during this investigation. In addition, the writers would like to thank Bill Eichert, Richard Hayes, and Al Montalvo of the Hydrologic Engineering Center and Johannes DeVries, of the University of California, Davis, for their valuable assistance.

Appendix I.—References

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Appendix II.—Notation

The following symbols are used in this paper:

A = cross section area;
A_{sd} = subdivision area;
D = hydraulic depth (area divided by top width);
E = specific energy;
F = Froude number;
g = acceleration of gravity;
K_{sd} = subdivision conveyance;
K = cross section conveyance (sum of K's);
Q_{sd} = subdivision discharge;
Q = cross section discharge;
T_{sd} = subdivision top width;
V = mean cross section velocity;
V_{sd} = mean subdivision velocity;
y = water depth; and
α = Coriolis coefficient.

Minimum Specific Energy in Compound Open Channel

By Merritt E. Blalock,[1] M. ASCE and Terry W. Sturm,[2] A. M. ASCE

Introduction

Analysis of open channel flow by the application of the energy principle is often clarified and aided by the concept of specific energy, which was introduced by Bakhmeteff (1) in 1912. Specific energy is defined for one-dimensional open-channel flow as the height of the energy grade line above the channel bottom. It leads to a classification of open-channel flow into subcritical and supercritical flow regimes, distinguished by flow depths that are respectively greater or less than the depth at which specific energy is minimum (critical depth). A mathematical consideration of minimum specific energy gives rise to the definition of a Froude number having a value of unity at critical depth. The value of the Froude number is greater than unity for supercritical flow and less than unity for subcritical flow.

The occurrence of critical depth and its associated minimum specific energy is of considerable practical importance to hydraulic engineers. It is one type of channel control which may provide the boundary condition for computation of water-surface profiles in steady, gradually varied flow. Water-surface profile computations are an integral part of water resources investigations involving flood-plain delineations, evaluation of flood control measures, and the design of irrigation and drainage channels.

Petryk and Grant (9) show that the determination of critical depth in channels with overbank or flood-plain flow (compound channels) can be troublesome. Customary definitions of the Froude number generally do not indicate critical depth at the point of minimum specific energy. In addition, there are some compound-channel geometries which produce specific-energy diagrams with two points of minimum specific energy. It is the purpose of this paper to present an analytical formulation of a compound-channel Froude number which correctly identifies the occurrence of points of minimum specific energy for flow in compound open channels. The proposed compound-channel Froude number ca[n]

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be used in conjunction with existing computer programs for water-s...ace profile computation (5,13,16), and is necessarily limited by the same simplifying assumptions that are associated with the conventionally used, one-dimensional equation of steady, gradually varied flow (17). The results of an experimental investigation in a laboratory flume are also presented, demonstrating the existence of two points of minimum specific energy and identifying these points by the proposed compound-channel Froude number.

Froude Number-Flow Regime Discrepancies

For a simple channel of nonrectangular section and uniform cross-sectional velocity distribution, the Froude number F is defined by

$$\mathsf{F} = \left(\frac{Q^2 T}{g A^3}\right)^{1/2} \quad \text{(1)}$$

in which Q = discharge; T = the top width of the water surface; g = acceleration of gravity; and A = the cross-sectional area of flow. For a compound channel it is customary to include the kinetic energy flux correction coefficient, α, in the definition of specific energy. As a result, α appears as follows in the definition of the Froude number assuming α is constant with depth:

$$\mathsf{F}_\alpha = \left(\frac{\alpha Q^2 T}{g A^3}\right)^{1/2} \quad \text{(2)}$$

For natural channels with overbank flow, it is often assumed that the major contribution to α is the large difference in mean velocity between main channel and overbank sections. By comparison the nonuniformity of the velocity distribution within each subsection can be neglected.

Two major problems arise in the computation of one-dimensional, steady, gradually varied flow profiles in compound channels as a result of using the Froude numbers F or F_α. First, incorrect solutions are generated when numerical methods are used to solve the gradually varied flow equation written in a form involving the Froude number F_α. Second, incorrect solutions may be accepted when the standard step method is used to compute water-surface profiles near critical depth. These difficulties are the result of neglecting the variation of α with depth in compound-channel flows.

Consider the equation of gradually varied flow in the following form:

$$\frac{dy}{dx} = \frac{S_0 - S_e}{1 - \mathsf{F}_\alpha^2} \quad \text{(3)}$$

in which dy/dx = the rate of change in depth of flow with respect to distance along the channel; S_0 = the bed slope of the channel; and S_e = the slope of the energy grade line. Prasad (10) has proposed a numerical solution procedure for Eq. 3 which can be applied to natural channels. In addition to the assumption that α is constant, the assumptions involved in obtaining Eq. 3 include: no lateral flow, a hydrostatic pressure distribution, a constant bed slope, and a straight, very wide channel, or alternatively, an approximately prismatic channel (17). Because the variation in α with depth and thus with distance along the channel has been neglected, application of Eq. 3 to a gradually varied flow in a compound channel will lead to incorrect water-surface elevations. The denominator of the term on the right-hand side in Eq. 3 arises from a consideration of the variation of specific energy with depth, a portion of which is due to changes in α with depth in compound-channel flow. Furthermore, the use of F_α can cause the right-hand side of Eq. 3 to become indefinite at a depth that does not correspond to the actual critical depth.

As an alternative to Eq. 3, water-surface profiles are computed in natura[l] channels by the standard step method (6) in which the specific energy is compute[d] explicitly. In this case, F_α does not appear in the equation to be solved, bu[t] is used instead to indicate whether the solution is in the supercritical or subcritica[l] flow regime. For compound channels, neither F nor F_α correctly indicates th[e] flow regime. Thus, incorrect solutions of the energy equation can be accepte[d] when the depth is near critical depth.

Compound-Channel Froude Number

Previous Investigations.—Previous investigations of the problems associate[d] with defining the Froude number in compound channels are limited. Numerou[s] laboratory investigations of compound-channel flow have been undertake[n] (8,11,15), but the focus of these experiments has been the quantification [of] changes in the boundary shear stress distribution resulting from momentu[m] exchange between the main channel and floodplain. The Federal agencies whi[ch] maintain and use water-surface profile programs recognize the Froude-numb[er] difficulties in compound channels as described in the previous section of th[is] paper, and they examine these difficulties in their user's manuals. The So[il] Conservation Service (16), e.g., warns of differences of as much as 2 ft betwe[en] the critical depth determined by F (Eq. 1) and the critical depth determin[ed] by minimum specific energy.

The Corp of Engineers (5) presents an algorithm to solve for the dep[th] corresponding to minimum specific energy when their water-surface prof[ile] program attempts to obtain a solution close to critical depth. The depth [of] minimum specific energy is compared with the profile depth to check the fl[ow] regime rather than using the Froude number as a check.

The United States Geological Survey (USGS) (12) proposes the use of [an] index Froude number based on the Froude number of the subsection carryi[ng] the greatest discharge. The index Froude number is thought by the USGS [to] better reflect the flow regime of the entire cross section, but it is also recogniz[ed] as having limitations. The USGS does not consider the index Froude num[ber] to be a true Froude number, but rather a warning flag that identifies possi[ble] flow-regime problems. A later version of the USGS Water Surface Profile Progr[am] incorporates a routine to determine the depth of minimum specific energy.

Petryk and Grant (9) have proposed a discharge-weighted Froude num[ber] without experimental corroboration in order to eliminate the computatio[nal] problems associated with the occurrence of two points of minimum speci[fic] energy in compound-channel flows. Although their proposed Froude num[ber] succeeds in doing this by identifying only one value of critical depth, i[t is] nevertheless somewhat arbitrary and is divorced from the concept of minim[um] specific energy.

Clearly, the Froude number should be formulated to reflect the specific ene[rgy] curve under consideration and should indicate critical depth at the point (or points) of minimum specific energy. Such a Froude number would produce correct numerical solutions of the gradually varied flow equation (Eq. 3) and would eliminate the need for time-consuming routines used to solve for the depth of minimum specific energy in standard step water-surface profile computations.

Derivation and Formulation.—The specific energy, E, for a one-dimensional compound-channel flow is given by

$$E = y + \frac{\alpha Q^2}{2gA^2} \quad (4)$$

in which y = the depth of flow. The kinetic energy flux correction coefficient, α, is defined as

$$\alpha = \frac{\int v^3 dA}{V^3 A} \quad (5)$$

in which v = the velocity through the element of area, dA; and V = the mean cross-sectional velocity (3,6). Alpha is thus a measure of the nonuniformity of the velocity distribution. For computational purposes, flow is conventionally divided into channel and overbank subsections by appropriately located vertical lines which are assumed not to transmit shear stress from one section of flow to another, and which do not contribute to wetted perimeter. Wright and Carstens (15) have suggested that the wetted perimeter of the subsection dividing line be retained for the main channel, and that the shear stress applied by the main-channel flow section on the overbank section be considered. Regardless of the manner in which the main flow-flood-plain interaction is treated, the basic assumption in the computation of α, as previously mentioned, is that the contribution of the nonuniformity of the velocity distribution within each subsection is negligible in comparison to the variation in mean velocity between subsections. If Eq. 5 is applied with this assumption to a compound channel which has been divided into subsections, the kinetic energy flux correction coefficient becomes

$$\alpha = \frac{\sum_i \left(\frac{k_i^3}{a_i^2}\right)}{\frac{K^3}{A^2}} \quad (6)$$

in which k_i = the conveyance of the ith subsection; a_i = the area of the ith subsection; and $K = \Sigma k_i$ = the conveyance of the total cross section (3,6). The subsection conveyance is computed from the Manning equation as follows:

$$k_i = \frac{1.49}{n_i} a_i r_i^{2/3} \quad (7)$$

in which r_i (= a_i/p_i) = the subsection hydraulic radius; p_i = the subsection wetted perimeter; and n_i = the subsection n value. In the SI system of units, the constant 1.49 is replaced by unity.

The point (or points) of minimum specific energy is obtained by differentiating Eq. 4 with respect to y and setting the derivative equal to zero. Because both α and area are functions of depth, the differentiation produces (14)

$$\frac{dE}{dy} = 1 - \frac{\alpha Q^2}{gA^3}\frac{dA}{dy} + \frac{Q^2}{2gA^2}\frac{d\alpha}{dy} = 0 \quad (8)$$

Noting that $dA/dy = T$, and that by rearranging terms, the following expression is obtained:

$$\frac{\alpha Q^2 T}{gA^3} - \frac{Q^2}{2gA^2}\frac{d\alpha}{dy} = 1 \quad (9)$$

The left-hand side of Eq. 9 is unity at the point of minimum specific energy; therefore, a compound-channel Froude number F_c can be defined from Eq. 9 as

$$F_c = \left(\frac{\alpha Q^2 T}{gA^3} - \frac{Q^2}{2gA^2}\frac{d\alpha}{dy}\right)^{1/2} \quad (10)$$

At the point of minimum specific energy F_c will have a value of 1.

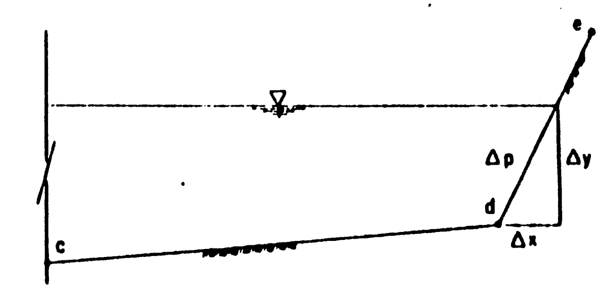

FIG. 1.—Definition Sketch for Evaluation of dp_i/dy

With the exception of $d\alpha/dy$, all of the terms on the right-hand side of Eq. 10 are routinely determined in water-surface profile computations. Evaluation of $d\alpha/dy$ can be achieved by differentiating Eq. 6 with respect to y. As shown in Appendix I, the derivative becomes

$$\frac{d\alpha}{dy} = \frac{A^2\sigma_1}{K^3} + \sigma_2\left(\frac{2AT}{K^3} - \frac{A^2\sigma_3}{K^4}\right) \quad (1$$

in which $\sigma_1 = \sum_i \left[\left(\frac{k_i}{a_i}\right)^3 \left(3t_i - 2r_i\frac{dp_i}{dy}\right)\right] \quad (1$

$$\sigma_2 = \sum_i \left(\frac{k_i^3}{a_i^2}\right) \quad (1$$

$$\sigma_3 = \sum_i \left[\left(\frac{k_i}{a_i}\right)\left(5t_i - 2r_i\frac{dp_i}{dy}\right)\right] \quad (1$$

In Eqs. 12-14, t_i = the top width of the ith subsection; and [illegible] y = the rate of change in wetted perimeter with respect to depth of flow in the ith subsection. Evaluation of dp_i/dy is simplified by the fact that the cross-section geometry of natural channels is defined by ground points connected with straight lines. The definition sketch in Fig. 1 (which is a portion of a right overbank subsection) shows the water surface intersecting the line segment $\overline{de}$. This line segment makes a contribution of Δp to the subsection wetted perimeter. The rate of change in wetted perimeter with respect to depth is a constant along $\overline{de}$, and therefore can be evaluated as

$$\frac{dp_i}{dy} = \frac{\Delta p}{\Delta y} \qquad (15)$$

The terms Δp and Δy are generally determined when computing the geometric properties of a cross section for use in a water-surface profile program. It should be noted that if the water surface is at point e, dp_i/dy should be evaluated for the line segment $\overline{de}$, but if the water surface is at point d, dp_i/dy should be evaluated for the line segment $\overline{cd}$. In situations where the water surface

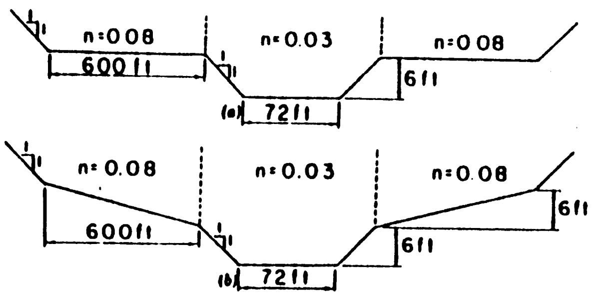

FIG. 2.—Channel Cross Sections for Evaluation of Specific Energy and Froude Numbers: (*a*) Cross Section A; (*b*) Cross Section B (1 ft = 0.3 m)

does not intersect the wetted perimeter of a subsection (e.g., the boundary between the main channel and overbank section above bankfull stage), dp_i/dy is zero. For a subsection where the water surface intersects both a left and right bank (e.g., the main channel below bankfull stage), dp_i/dy is the sum of $\Delta p/\Delta y$ for each of the banks.

The working equation for the compound-channel Froude number can be obtained by substituting Eq. 11 into Eq. 10 and simplifying:

$$F_c = \left[\frac{Q^2}{2gK^3}\left(\frac{\sigma_2\sigma_3}{K} - \sigma_1\right)\right]^{1/2} \qquad (16)$$

If the Manning's n value is considered to vary with depth of flow in any subsection, σ_1 and σ_3 can be written to reflect the variation:

$$\sigma_1 = \sum_i \left[\left(\frac{k_i}{a_i}\right)^3\left(3t_i - 2r_i\frac{dp_i}{dy} - \frac{a_i}{n_i}\frac{dn_i}{dy}\right)\right] \qquad (17)$$

$$\sigma_3 = \sum_i \left[\left(\frac{k_i}{a_i}\right)\left(5t_i - 2r_i\frac{dp_i}{dy} - \frac{a_i}{n_i}\frac{dn_i}{dy}\right)\right] \qquad (18)$$

in which dn_i/dy = the rate of change in n_i with respect to depth of flow.

Evaluation.—The behavior of the compound-channel Froude number, F_c, may be evaluated by examining the specific-energy diagrams of two idealized, symmetric cross sections, each conveying 5,000 cfs (142 m^3/s). Cross section A [Fig. 2(*a*)] is from Petryk and Grant (9). In Fig. 3, the specific-energy curve for this cross section reveals two points of minimum specific energy at depths of flow of approx 6.8 ft (2.07 m) and 5.3 ft (1.62 m). These points are indicated by C1 and C2, respectively, in Fig. 3.

F_c (Eq. 16) for this cross section is plotted in Fig. 4 along with F (Eq. 1) and F_a (Eq. 2). As expected, all three equations produce the same curve below top of bank (simple channel situation), but only Eq. 16 for F_c correctly locates

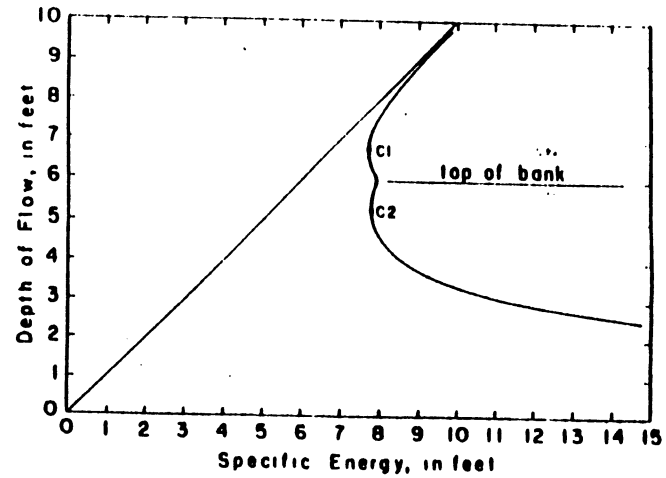

FIG. 3.—Specific Energy for Cross Section A Conveying 5,000 cfs (1 cfs = 0.028 m^3/s; 1 ft = 0.3 m)

C1, the upper depth of minimum specific energy (6.8 ft or 2.07 m), and connects with the lower curve at the top of bank depth.

The shape of the Froude number curve is independent of the discharge, and the fiducial point (F_c = 1) can be shifted left or right by varying the discharge. This means that once F_c is plotted for a particular cross section and discharge, points of minimum specific energy for other discharges may be determined without the necessity of constructing new specific-energy diagrams. In effect, the variable F_c/Q provides a universal horizontal scale for Fig. 4 which depends only on the conveyance and geometric properties of the particular cross section. Thus, for a given depth of flow, the critical discharge, Q_c, can be computed by taking the reciprocal of the corresponding value of F_c/Q, because F_c/Q for the given depth equals $1/Q_c$ for the critical condition.

Cross section B is presented in Fig. 2(*b*) and differs from cross section A only in that the flood plains have a 100:1 slope toward the channel. The specific-energy diagram of cross section B (Fig. 5) reveals a single point of minimum specific energy below top of bank at the same depth of flow as for cross section A (point C2). The three Froude number curves shown in Fig. 6 for cross section B are again identical below top of bank, but F (Eq. 1) and F_a (Eq. 2) each indicate another point of minimum specific energy above top of bank at depths of flow of 6.5 ft (1.98 m) and 6.8 ft (2.07 m), respectively. The occurrence of these false points of minimum specific energy is a more serious deficiency of Eqs. 1 and 2 than the errors in critical depth shown in Fig. 4.

It is evident from these two examples that the Froude numbers generated by Eqs. 1 and 2 are not acceptable for use in the gradually varied flow equation (Eq. 3). Neither definition of Froude number faithfully reflects the specific-energy diagram in overbank flow situations, and either would produce divergence from a correct profile solution. It is equally evident that Eqs. 1 and 2 are not satisfactory

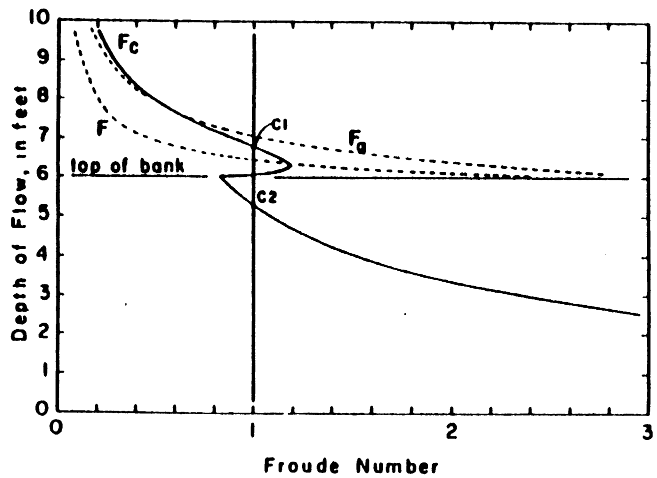

FIG. 4.—Froude Numbers for Cross Section A Conveying 5,000 cfs (1 cfs = 0.028 m^3/s; 1 ft = 0.3 m)

for checking the flow regime in the standard step method. Only F_c (Eq. 17) accurately reflects the specific-energy diagram and indicates the correct flow regime. The experimental investigation into the occurrence of two points of minimum specific energy in the following portion of this paper offers guidance for the interpretation of the flow regime between the two points of minimum specific energy, C1 and C2, in cross section A (Fig. 3).

Experimental Investigation

The experimental investigation consisted of measuring point velocities in a compound-channel cross section which was formed by constructing a single rectangular overbank section in a laboratory flume. Sufficient point velocity measurements were made at eight different depths of flow (at approximately the same discharge for each depth) to compute the discharge, velocity,

kinetic energy flux correction coefficient, and specific energy for each depth. Complete details of the experimental procedure are given by the first writer (2).

The experiments were conducted in a tilting steel flume 80 ft (24.38 m) long, 3.5 ft (1.07 m) wide, and 1.5 ft (0.46 m) deep. This flume was also used by Tracy and Lester (14) and details of its construction are given by them. The overbank section was constructed of 3/4-in. exterior plywood and two-by-six fir framing lumber, resulting in the channel dimensions shown in Fig. 7. All wood was coated with sanding sealer and exterior acrylic-latex paint. The overbank section was attached to the flume with silicon adhesive.

Point velocities were measured with a 0.072-in. (1.83-mm) outside diameter pitot-static tube operated in conjunction with a differential pressure transducer. Data collection, reduction, and analysis were accomplished with an HP9825A desktop computer controlling a digital voltmeter which measured the voltage

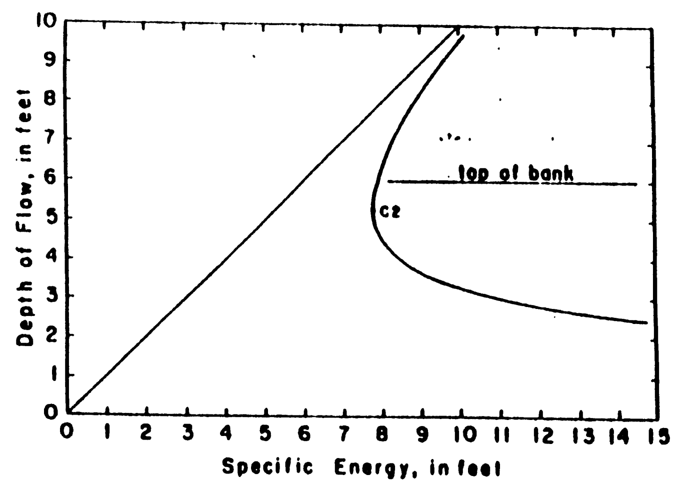

FIG. 5.—Specific Energy for Cross Section B Conveying 5,000 cfs (1 cfs = 0.028 m^3/s; 1 ft = 0.3 m)

output from the pressure transducer and preamplifier. Point velocity measurements were made at a station 65 ft (19.81 m) downstream of the flume entrance. Preliminary measurements were made at a station 60 ft (18.29 m) downstream. Comparison of dimensionless profiles of velocity between the two stations indicated that the flow was fully developed.

The preliminary experiments indicated that a discharge of 1.7 cfs (0.048 m^3/s) would produce a specific-energy curve with two points of minimum specific energy. An estimate of the error in setting the discharge to 1.7 cfs (0.048 m^3/s) included the calibration error of the Venturi meter used to measure the discharge and also included an estimate of the error introduced by observed fluctuations in the Venturi-meter manometer during the course of an experimental run. The estimated error in discharge was of the order of ±3%, which was the same range of error observed between individual discharges [illegible]ined from the

Venturi meter and the discharges determined by integration of the pc locity measurements.

Establishing a truly uniform flow profile for the experimental runs proved impossible. Any discharge flowing near the depth corresponding to minimum specific energy, as these were, could be expected to be inherently unstable. The instability was exacerbated by the variations in the overbank surface, which were of the order of ±0.01 ft (0.3 cm). Standing waves in the overbank section and a cross-hatched water surface in the channel thwarted efforts to achieve a uniform water-surface profile. As a result, the adopted experimental procedure was to establish a profile as close to uniform as possible such that the desired depth of flow was obtained where the point velocities were to be measured. The maximum observed change in depth for overbank-flow runs was approx 0.05 ft (1.5 cm) between the channel entrance and the measuring station where the flow depth was 0.567 ft (17.3 cm). For larger depths of flow, the water-surface

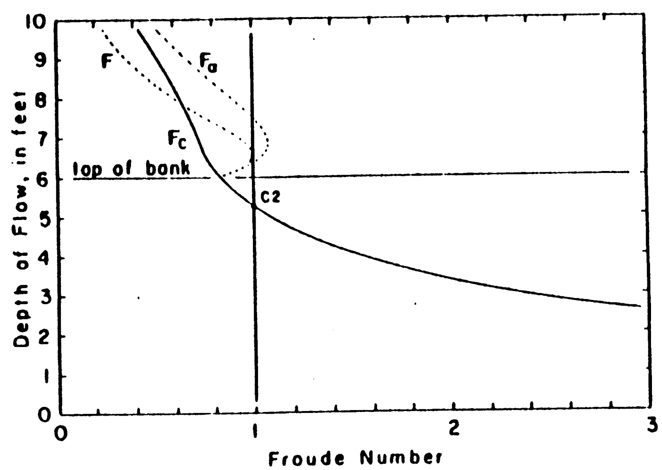

FIG. 6.—Froude Numbers for Cross Section B Conveying 5,000 cfs (1 cfs = 0.028 m^3/s; 1 ft = 0.3 m)

profiles tended to be more stable and more nearly uniform. A profile at a depth of flow of 0.7 ft (21.3 cm) was established to demonstrate that a uniform profile could be obtained in the downstream reach of the flume if the depth of flow was sufficiently greater than the depth corresponding to minimum specific energy.

Results

Table 1 presents the values of area, discharge, kinetic energy flux correction coefficient, and specific energy computed from experimental measurements for each of the eight reported runs. Runs 5 and 6 are not reported in the table because of operational difficulties during each run. It is apparent from the results presented in Table 1 that as the depth increased for those experimental runs with overbank flow, the proportion of the total discharge in the overbank

section increased. It should also be noted that the values of ɔr the main channel alone are measurably larger than 1.0 because of the narrowness of the main channel section.

Observations of the water surface for the four experimental runs with overbank flow indicated greater instability as the depth of flow decreased. The water-surface instability was manifested by standing waves in the overbank section and a choppy, cross-hatched water surface in the channel section. Beginning at the upper depth of minimum specific energy (run 2) and continuing with decreasing depth, the standing wave fronts in the overbank section were perpendicular to the mean flow direction and then were bent downstream into a cross-hatched pattern in the channel section characteristic of supercritical flow. The surface instability continued to increase for the experimental runs as depth decreased below top of bank. The fact that the water surface was unstable for experimental runs 7 and 8, the first two runs below top of bank in Table 1, suggests that the upper point of minimum specific energy could be considered the limit of subcritical flow for situations in which two points of minimum specific energy occur in water-surface profile computations.

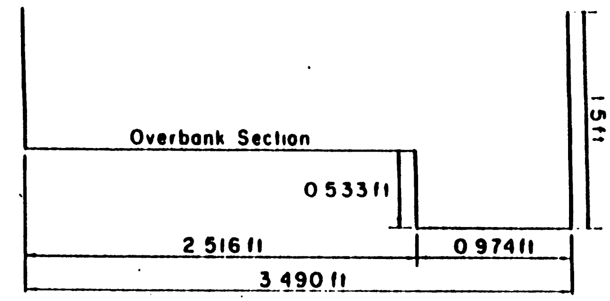

FIG. 7.—Cross Section of Flume and Overbank Section, Looking Downstream (1 ft = 0.3048 m)

The experimental specific-energy data in Table 1 are plotted in Fig. 8(*a*). Although the variation in discharge from run to run causes some scatter in the plot, there is evidence of two points of minimum specific energy. The experimental values of α plotted in Fig. 8(*b*) show little scatter and indicate that α is primarily a function of depth of flow. This observation suggests that a specific-energy diagram for a single value of discharge can be constructed by substituting the average discharge of eight runs (1.692 cfs or 0.048 m^3/s) into Eq. 4 while using the experimental data for all other variables. Fig. 9 presents the resulting average specific-energy diagram. The two points of minimum specific energy are more clearly apparent in this figure.

The concept of computing a Froude number for the flow in a subsection of a compound channel has already been mentioned with regard to the USGS index Froude number (12). The subsection Froude numbers (computed with Eqs. 1 and 2) for the experimental data of this investigation are presented in Table 2. The Froude number of the channel (Col. 3 or 4 of Table 2) is the index Froude number of these experimental runs because the channel is the subsection with the largest discharge. All four depths of flow above top

TABLE 1.—Experimental Data

				Channel			Overbank			Total		
Run (1)	y, in feet (2)	S_0 (3)	E, in feet (4)	A, in square feet (5)	Q, in cubic feet per second (6)	α (7)	A, in square feet (8)	Q, in cubic feet per second (9)	α (10)	A, in square feet (11)	Q, in cubic feet per second (12)	α (13)
1	0.650	0.001018	0.718	0.633	1.363	1.084	0.294	0.411	1.108	0.927	1.774	1.192
4	0.625	0.001128	0.702	0.609	1.388	1.083	0.231	0.326	1.132	0.840	1.714	1.198
2	0.600	0.001485	0.700	0.584	1.496	1.082	0.169	0.230	1.169	0.753	1.726	1.224
3	0.567	0.002096	0.701	0.552	1.592	1.088	0.083	0.087	1.340	0.635	1.680	1.238
10	0.533	0.001903	0.704	0.519	1.648	1.093				0.519	1.648	1.093
7	0.500	0.002118	0.700	0.487	1.676	1.087				0.487	1.676	1.087
8	0.467	0.003300	0.690	0.455	1.645	1.096				0.455	1.645	1.096
9	0.433	0.004455	0.701	0.422	1.671	1.100				0.422	1.671	1.100

Note: 1 ft = 0.3048 m; 1 cfs = 0.028317 m^3/s.

of bank are subcritical based on the index Froude number, but as shown in Fig. 9, the two lower overbank depths are not subcritical. For this experimental investigation, the index Froude number does not correctly indicate the flow regime of compound-channel flow.

Petryk and Grant (9) apply the concept of a subsection Froude number to obtain their weighted Froude number F_r, which is given by

$$F_r = \frac{\sum_i (q_i F_i)}{Q} \quad \text{(19)}$$

in which q_i = the subsection discharge; and F_i = the subsection Froude number computed by Eq. 1. Values of F_r for the experimental data are presented in Col. 7 of Table 2. As in the case of the index Froude number, the weighted Froude number does not correctly indicate the flow regime.

Analysis

The proposed compound-channel Froude number cannot be directly determined from the experimental data. Attempts to use Eq. 10 fail because it is difficult to determine $d\alpha/dy$ from the limited number of experimental data points. Eq. 16 fails because the slope of the energy grade line is not precisely known which means that the subsection resistance coefficient and thus the conveyance k_i, cannot be determined from the experimental data. If it had been possible to establish a uniform flow condition for each run, the energy gradient would parallel the flume slope, and the conveyance for each subsection could be computed from the experimental data alone. The compound-channel Froude number can only be determined indirectly through an independent prediction of the experimental results.

Working in the same flume as used in the present investigation, Tracy and Lester (14) experimentally determined a friction-factor relationship for smooth rectangular channels of the form

$$\frac{1}{\sqrt{f}} = 2.03 \log (R\sqrt{f}) - 1.30 \quad \text{(20)}$$

in which f = the Darcy-Weisbach friction factor; and R = the Reynolds number. If it is assumed that Eq. 20 is valid when applied independently to each channel subsection, the friction factor, f_i, can be determined for the ith subsection. The mean velocity in the ith subsection, v_i, is then given by

$$v_i = \left(\frac{8 g r_i S_e}{f_i}\right)^{1/2} \quad \text{(21)}$$

in which r_i = the hydraulic radius of the ith subsection; and S_e = the slope of the energy grade line. Because the values of f_i and v_i obtained from Eqs. 20 and 21 must be such that the subsection discharges sum to the average measured discharge, Q_m, of 1.692 cfs (0.048 m^3/s), the following equation must be satisfied:

$$S_e = \frac{Q_m^2}{\left[\sum_i \left(\frac{8g}{f_i} r_i a_i^2\right)^{1/2}\right]^2} \quad \text{(22)}$$

It has been implicitly assumed that S_e is the same for all subsections. Eqs. 20, 21, and 22 can be solved iteratively for the friction factor and velocity in each subsection for a given total discharge and depth. The iterative solution procedure is given in detail by the first writer (2).

The velocities, v_i, were calculated by the procedure just described for the mean measured discharge of 1.692 cfs (0.048 m^3/s). It was assumed that the imaginary vertical boundary between the main channel [illegible] overbank section made no contribution to wetted perimeter. Furthermore, the [illegible] factors determined for each subsection were converted to Manning's n va[illegible] because the formulation for the compound Froude number, F_c, is in terms of n. The n values so obtained exhibited a slight variation with depth; however, to facilitate the computations, constant n values of 0.009 and 0.010 were adopted for the channel and overbank sections, respectively. From the velocities and n values for each subsection, the specific energy and compound Froude number were computed for a series of depths within the range of measured depths. In the

TABLE 2.—Froude Numbers for Experimental Data

		Channel		Overbank		Weighted
Run (1)	y, in feet (2)	F (Eq. 1) (3)	F_α (Eq. 2) (4)	F (Eq. 1) (5)	F_α (Eq. 2) (6)	F_c (Eq. 19) (7)
1	0.650	0.471	0.490	0.721	0.759	0.529
4	0.625	0.508	0.529	0.821	0.873	0.586
2	0.600	0.583	0.606	0.925	1.001	0.629
3	0.567	0.675	0.704	1.017	1.177	0.692

Note: 1 ft = 0.3048 m.

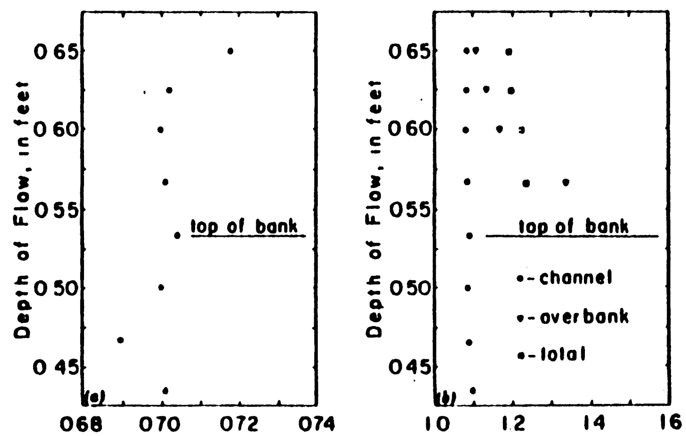

FIG. 8.—Specific Energy and Kinetic Energy Flux Correction Factor from Experimental Data (1 ft = 0.3 m): (a) Specific Energy, in feet; (b) Alpha

computation of the specific energy and F_c, it was assumed that α of each subsection had the value 1.0 rather than the measured value. In this way, the computational procedure remained independent of the measured data and was executed in the same manner as would be expected when determining F_c for a natural river channel in the course of a water-surface profile computation.

The predicted specific-energy diagram is shown in Fig. 10(*a*), and two depths of minimum specific energy are apparent, although each depth is approximately 2/100 ft smaller than the corresponding depths in Fig. 8(*a*) or Fig. 9. The

entire specific-energy curve in Fig. 10(*a*) is skewed sligh[illegible] [illegible]nward and to the left when compared with the measured curve in Fig. 8(*a*) or the average curve in Fig. 9. The predicted compound-channel Froude number curve in Fig. 10(*b*) exhibits the behavior typical for two points of minimum specific energy,

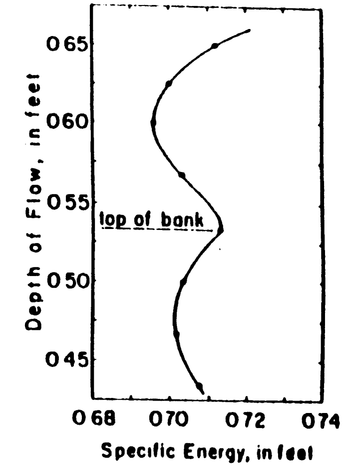

FIG. 9.—Experimental Specific Energy Curve for an Average Discharge of 1.692 cfs (1 cfs = 0.028 m³/s; 1 ft = 0.3 m)

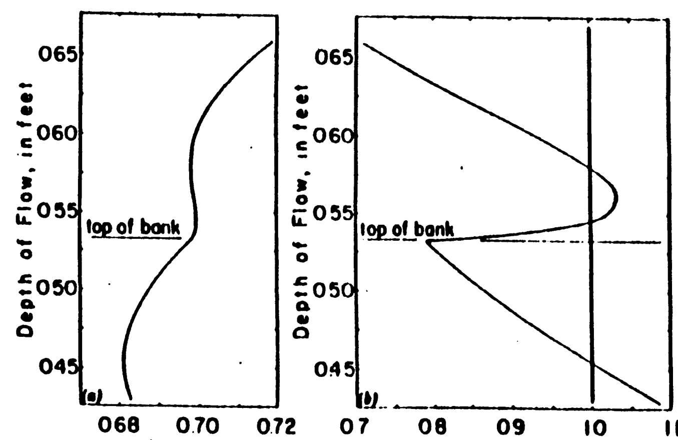

FIG. 10.—(*a*) Predicted Specific Energy in Experimental Flume for 1.692 cfs; (*b*) Compound Channel Froude Number for Fig. 10(*a*) (1 cfs = 0.028 m³/s; 1 ft = 0.3 m)

and is in correspondence with the predicted specific-energy curve as expected.

To investigate the role that neglecting the transfer of linear momentum to the overbank section plays in the skew of the predicted specific-energy curve, the correction suggested by Wright and Carstens (15) was considered. Although the correction improved the agreement between the measured and computed discharges in the overbank section, especially at the larger depths, the effect on the computed specific-energy curve was minimal because of the relatively small changes in α which resulted from the correction.

The skew in the specific-energy curve is most pronounced below top of bank depth where transfer of linear momentum to the overbank does not occur. The skew in this portion of the curve can be attributed to selecting subsection α values of unity in computing specific energy. It should be noted that the depths of flow in the flume were small compared to depths of flow normally found in field situations. For this reason, the velocity head in the flume makes a large relative contribution to specific energy, and any adjustment to velocity head (such as subsection α) has far more effect on specific energy in the flume than it would in the field.

The same analysis can be applied to subcritical and supercritical flow regimes in field situations where kinetic energy correction coefficients can be as much as 1.4 or more in the main channel (7). For subcritical flow where the velocity head is small, an α-adjustment to velocity head would be insignificant. For supercritical flow, the velocity head can be 50% or more of the depth, and an α-adjustment to velocity head would have a significant effect on specific energy. This reasoning explains the increasing leftward shift in Fig. 10(*a*) as the depth of flow decreases, and the implication is that predicted specific energies and Froude numbers in field channels under subcritical flow conditions would be closer to measured values.

Conclusions

Existing formulations of the Froude number (Eqs. 1 and 2) do not accurately reflect the specific-energy curve for flow in a compound open channel and do not correctly locate points of minimum specific energy. A compound-channel Froude number (Eq. 16) has been derived and has been shown to accurately reflect the specific-energy curve of flow in a compound open channel by correctly locating points of minimum specific energy. When applied to a simple channel with uniform velocity distribution, the compound channel Froude number is identical to Eq. 1, the conventional definition of Froude number.

The compound-channel Froude number is appropriate for use with the gradually varied flow equation (Eq. 3) and provides the proper check of the flow regime when used in conjunction with the standard step method of water-surface profile computation. The proposed Froude number is subject to the same assumptions that apply to the equation of gradually varied flow commonly employed in water-surface profile computations.

For some compound-channel geometries characterized by wide, level flood plains, two points of minimum specific energy can be computed for certain discharges. Laboratory investigation of a one-dimensional flow demonstrates that this phenomenon can in fact occur, and indicates that the upper point of minimum specific energy may be considered the proper limit of subcritical flow.

Acknowledgments

Financial support for the first writer was provided by the USGS during the experimental investigation. The School of Civil Engineering, Georgia Institute of Technology, provided material support for the construction of the experimental apparatus and supplied the electronic instrumentation. The writers wish to express their appreciation to C. S. Martin, Georgia Institute of Technology, for his assistance with the instrumentation.

Appendix I.—Derivation of $d\alpha/dy$

Writing Eq. 6 as

$$\alpha = \frac{A^2}{K^3} \sum_i \left(\frac{k_i^3}{a_i^2}\right) \quad \text{(23)}$$

and differentiating with respect to y produces

$$\frac{d\alpha}{dy} = \frac{A^2}{K^3} \sum_i \left[3\left(\frac{k_i}{a_i}\right)^2 \frac{dk_i}{dy} - 2\left(\frac{k_i}{a_i}\right)^3 \frac{da_i}{dy}\right] + \sum_i \left(\frac{k_i^3}{a_i^2}\right)\left[\frac{2A}{K^3}\frac{dA}{dy} - \frac{3A^2}{K^4}\frac{dK}{dy}\right] \quad \text{(24)}$$

Noting that $da_i/dy = t_i$, $dA/dy = T$, and $dK/dy = \Sigma_i (dk_i/dy)$, the following is obtained:

$$\frac{d\alpha}{dy} = \frac{A^2}{K^3} \sum_i \left[3\left(\frac{k_i}{a_i}\right)^2 \frac{dk_i}{dy} - 2t_i\left(\frac{k_i}{a_i}\right)^3\right] + \sum_i \left(\frac{k_i^3}{a_i^2}\right)\left[\frac{2AT}{K^3} - \frac{3A^2}{K^4}\sum_i\left(\frac{dk_i}{dy}\right)\right] \quad \text{(25)}$$

Evaluate dk_i/dy by writing Eq. 7 as

$$k_i = \left(\frac{1.49}{n_i}\right)\frac{a_i^{5/3}}{p_i^{2/3}} \quad \text{(26)}$$

and differentiate with respect to y to obtain

$$\frac{dk_i}{dy} = \left(\frac{1.49}{n_i}\right)\left[\frac{5}{3}\left(\frac{a_i}{p_i}\right)^{2/3}\frac{da_i}{dy} - \frac{2}{3}\left(\frac{a_i}{p_i}\right)^{5/3}\frac{dp_i}{dy}\right] \quad \text{(27)}$$

Again noting that $da_i/dy = t_i$, and multiplying and dividing by a_i, the following is obtained:

$$\frac{dk_i}{dy} = \frac{1}{3}\left(\frac{k_i}{a_i}\right)\left[5t_i - 2r_i\frac{dp_i}{dy}\right] \quad \text{(28)}$$

Substituting Eq. 28 into Eq. 25 and simplifying, results in Eq. 11.

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Appendix III.—Notation

The following symbols are used in this paper:

A = total cross-section area;
a_i = subsection area;
E = specific energy;
F = Froude number;
F_c = compound-channel Froude number;
F_i = subsection Froude number;
F_r = weighted Froude number;
F_α = Froude number with kinetic energy flux correction;
f = Darcy-Weisbach friction factor;
f_i = subsection friction factors;

g = acceleration of gravity;
K = total cross-section conveyance;
k_i = subsection conveyance;
n = Manning's n value;
n_i = subsection n value;
p_i = subsection wetted perimeter;
Q = total cross-section discharge;
Q_m = average measured discharge;
q_i = subsection discharge;
R = Reynolds number;
r_i = subsection hydraulic radius;
S_f = slope of energy grade line;
S_0 = bed slope of channel or flume;
T = total cross-section top width;
t_i = subsection top width;
V = total cross-section mean velocity;
v = mean velocity associated with incremental area, dA;
v_i = subsection mean velocity;
x = distance along channel;
y = depth of flow;
α = kinetic energy flux correction coefficient;
Δp = increment of wetted perimeter;
Δy = increment of depth; and
$\sigma_1, \sigma_2, \sigma_3$ = subsection parameters of compound-channel Froude number.

STATE OF ARIZONA
DEPARTMENT OF WATER RESOURCES
ENGINEERING DIVISION

State Standard

for

Identification of and Development Within

Sheet Flow Areas

Under authority of ARS 45-3605(a), the Director of the Arizona Department of Water Resources establishes the following standard for identification of and development within sheet flow areas in Arizona:

Identification of, or regulation of development within, sheet flow areas in Arizona for use in fulfilling the requirements of Flood Insurance Studies, and local community and county flood damage prevention ordinances will use the guidelines outlined in State Standard Attachment 4-95 entitled "Identification of and Development Within Sheet Flow Areas" or an alternative procedure reviewed and accepted by the Director.

For the purpose of application of these guidelines, sheet flow areas will include all sheet flow areas identified by the Federal Emergency Management Agency as part of the National Flood Insurance Program, all sheet flow areas which have been identified by a local floodplain administrator as having significant potential flood hazards, sheet flow floodplains meeting the site identification criteria outlined in Attachment 4-95 with drainage areas more than 1/4 square mile or a 100-year estimated flow of more than 500 cubic feet per second. Application of these guidelines will not be necessary if the local community or county has in effect a drainage, grading, or stormwater ordinance which, in the opinion of the Department, results in the same or greater level of flood protection as application of these guidelines would ensure.

This requirement is effective January 1, 1995. Copies of this State Standard and State Standard Attachment 4-95 can be obtained by contacting the Department's Engineering Division at (602) 417-2445.

STATE OF ARIZONA
DEPARTMENT OF WATER RESOURCES
ENGINEERING DIVISION

Identification of and Development within Sheet Flow Areas

500 North 3rd Street
Phoenix, Arizona 85004

(602) 417-2445

STATE STANDARD ATTACHMENT
SSA 4-95

JANUARY 1995

Disclaimer of Liability

The methods contained in this publication are intended to be a reasonable way of setting minimum floodplain management requirements where better data or methods do not exist. As in all technical methods, engineering judgement and good common sense must be applied and the methods rejected where they obviously do not offer a reasonable solution.

It must be recognized that while the criteria established herein will generally reduce flood damages to new and existing development, there will continue to be flood damages in Arizona. Where future-condition hydrology (which considers the cumulative effects of development) is not used, future development will probably increase downstream runoff which may result in flooding. Unlikely or unpredictable events such as earthquakes or dam failures may also cause extreme flooding.

The Arizona Department of Water Resources is not responsible for the application of the methods outlined in this publication and accepts no liability for their use. Sound engineering judgement is recommended in all cases.

The Arizona Department of Water Resources reserves the right to modify, update or otherwise revise this document and its methodologies. Questions regarding information or methodologies contained in this document and/or floodplain management should be directed to the local floodplain administrator or the office below:

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Contents

Introduction

Sheet flooding is a type of surface water runoff which occurs on broad, unconfined floodplains with low relief. Sheet flooding can occur in urban, rural, and natural areas. Because sheet flooding often occurs in areas which lack defined stream channels, identification of sheet flood areas can be difficult. Although types of sheet flooding have been identified in every geographic region of Arizona, floodplain management standards for sheet flood areas are generally lacking. This state standard for development in sheet flow areas is intended to promote sound floodplain management of these unique hazard areas.

This document details minimum floodplain management standards for identification of and development within sheet flooding areas in Arizona. Types of sheet flooding are defined, and identifying characteristics are given for each type. Flood hazards associated with sheet flooding are described. General floodplain management requirements and recommended development guidelines are presented. Three methods of sheet flow hydraulic analysis are presented which reflect increased levels of complexity and accuracy. Finally, sample applications of the standards are provided to demonstrate application of the development standards.

Definitions and Identifying Characteristics

Sheet flow is a loosely defined term, as it is used in Arizona. In general, the term "sheet flow" may refer to any form of unconfined runoff which occurs over a broad, expansive area. This broad definition of sheet flow incorporates several more narrowly defined flow types, including natural (classic) sheet flow, urban sheet flow, agricultural sheet flow, overland flow, perched flow, anastomosing flow, and distributary flow. The variety of terms used for sheet flow probably reflects the variety of flow types which occur within specific geographic regions of the state. For this study, definitions of types of sheet flooding are provided for use by regulatory agencies. The term "sheet flow" will be used generically, to include all types defined within this document.

In general, sheet flooding in Arizona may have the following characteristics:

- The primary identifying characteristic of sheet flow is that a significant part of floodwater is not conveyed in a single, well-defined channel. Flood flow is conveyed over the unchannelized land surface.

- Water moving over a smooth stable surface does not move as a uniform film. If the surface is broad, the sheet differentiates into parallel streams of greater depth and relatively rapid flow, separated by shallower bands of relatively sluggish flow; and at the same time, both streams and intervening bands differentiate into series of transverse waves which move forward more rapidly than the body of the undifferentiated sheet.

- Sheet flow over poorly vegetated surfaces often has the ability to transport large sediment particles relatively large distances over low slopes without significant reduction in sediment diameter, angularity, or degree of sorting, such as may be considered typical of most well defined streams.

- Sheet flooding has markedly different hydraulic characteristics for sediment laden and sediment deprived flows. Sheet flooding may not have gradually varied or steady flow, and may have a strong two-dimensional character.

- Significant loss of flow volume may occur during sheet flooding due to infiltration and other abstractions.

- Sheet flow often enters a larger channel or drainage system that intersects its flow, but occasionally dissipates due to infiltration or other loss mechanisms before ever reaching a channel.

In addition to these general characteristics of sheet flow, the specific types of sheet flow found in Arizona have unique identifying characteristics, described below.

Natural Sheet Flow

Natural sheet flow is flowing water characterized by a tendency to spread widely in relatively shallow sheets over gently sloping areas with low topographic relief which lack defined drainage systems. Figure 1 shows a natural sheet flow area.

Identifying characteristics of natural sheet flow areas include:

- Low topographic relief perpendicular to the primary flow direction.

- Very poorly defined channels (or none) downstream of a relatively large drainage area. When viewed on aerial photographs, no channel banks may be readily identified.

- Very uniform vegetative characteristics which extend laterally over an expansive area affected by sheet flow. Many natural sheet flow areas are covered by grass.

- Soil characteristics may not be visible on aerial photographs due to vegetation density. Soils characteristics are usually very uniform within the sheet flow area. In lower desert regions, very little surficial soil reddening may be present.

- Soil units mapped by the Soil Conservation Service as floodplain soils.

Figure 1

Natural Sheet Flow Area

Urban Sheet Flow

Urban sheet flow occurs where development has obscured natural drainage patterns or where urban drainage facilities are severely undersized. Urban sheet flow areas differ from natural sheet flow areas in that identifying soil and vegetative characteristics may be obscured by development. Urban sheet flow areas are usually identified from historic records of unconfined flooding. Urban sheet flow areas occasionally may be identified by detailed topographic maps that show low relief in known flooding areas. Figure 2 shows a urban sheet flow area.

Identifying characteristics of urban sheet flow areas include:

- Low topographic relief perpendicular to the primary flow direction.

- Lack of defined channels downstream of a relatively large drainage area.

- Significant flow in streets during ordinary rainstorms.

Distributary Flow

Distributary flow areas[1] have channels which split and rejoin in a complex pattern. The number of channel forks commonly exceeds the number of channel confluences, creating a *dis*tributary, rather than tributary drainage pattern. The separate channels downstream of a channel fork may have terraces independent of other channels within the distributary flow system. A distributary channel is a stream branch flowing away from the main stream and not rejoining it. Distributary flow may be characterized as sheet flow with a strong channelized flow component. Figure 3 shows a distributary flow area.

Identifying characteristics of distributary flow areas include:

- Low, but distinguishable topographic relief perpendicular to the primary flow direction. Topographic relief is sufficient to create isolated islands during flood conditions within the overall floodplain.

- Channels which divide in the downstream direction so that the number of flow paths conveying floodwaters increases in the downstream direction. Distributary flow may occur on alluvial fans.

- An increase in vegetative density along flow lines, with more uniform upland vegetation types found between flow lines, extending laterally over an expansive area.

- Soils units mapped by the Soil Conservation Service as alluvial fan terraces, inactive alluvial fans, or alluvial fans.

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[1] See Hjalmarson and Kemna, 1991 for additional information.

Figure 2

Urban Sheet Flow Area With Moderate Development Density

Note Flow patterns in adjacent natural areas

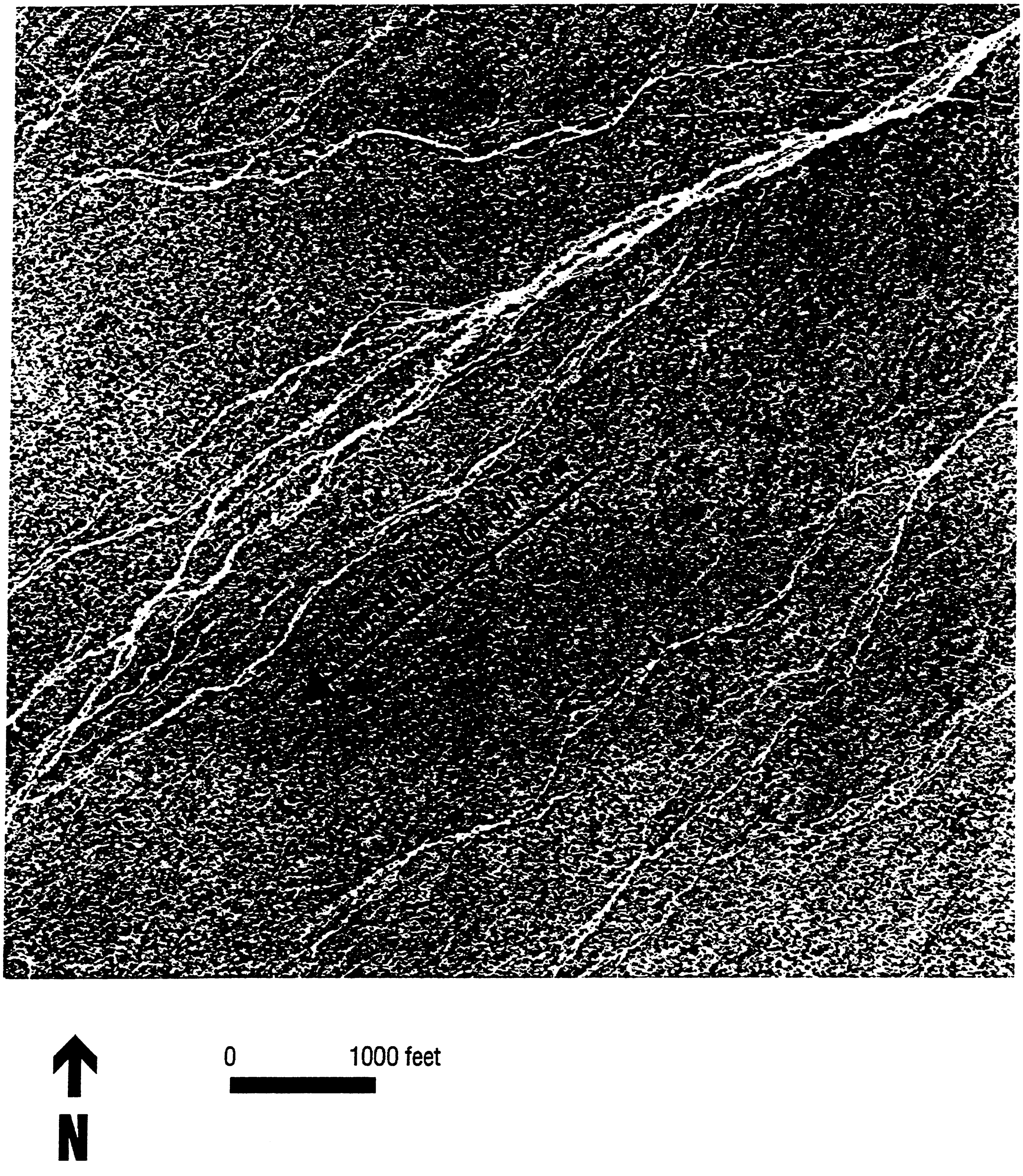

Figure 3

Distributary Flow Area

- During large floods, the distribution of flow between various existing distributary flow paths may not be predictable. However, flow lines are relatively stable, especially during smaller floods.

- Large floods may cause isolated or widespread bank erosion, or sediment deposition within the channel which changes channel capacity or may change overbank conveyance.

Anastomosing Flow

Anastomosing[2] flow is quasi-sheet flooding with slightly incised flow lines which creates a system of interwoven channels. This type of anastomosing is found in intermittent to perennial stream systems with net long-term erosion, in contrast to braided streams which are characterized by net long-term deposition, and which occur within well defined floodplains. Anastomosing flow differs from sheet flow (greater) and distributary flow by the (lesser) degree of flow line incision. Anastomosing streams are geologically temporary features. Figure 4 shows an anastomosing flow area.

Identifying characteristics of anastomosing flow areas include:

- An anastomosing stream has branching, interlacing, interconnecting flow paths, which produce a net-like or braided appearance.

- Anastomosing flow areas have slight topographic relief perpendicular to the primary flow direction.

- Anastomosing flow areas have poorly defined channels downstream of a relatively large drainage area. When viewed on aerial photographs, channel banks may not be visible for large portions of the anastomosing alluvial surface. Anastomosing may occur on the lowest portion of alluvial fans.

- An increase in vegetative density may occur along flow lines in anastomosing flow areas, with uniform vegetative characteristics between flow lines, extending laterally over an expansive area.

- Soils mapped by the Soil Conservation Service as floodplain soils.

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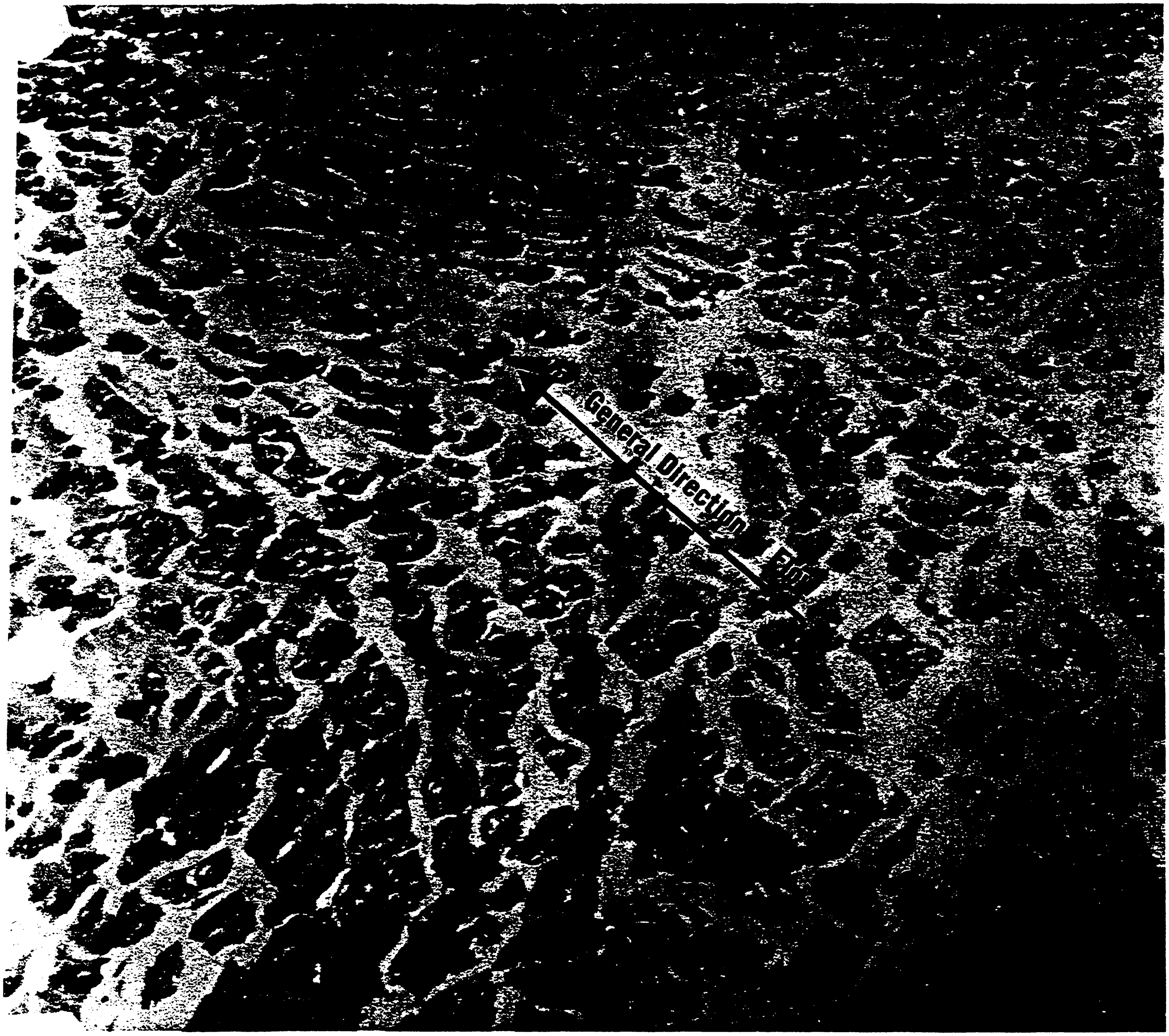
[2] The term anastomosing means netted; intervened; and is also used to describe leaves marked by cross veins forming a network; sometimes the vein branches meet only at the margin.



Note:
Exact photo scale and aspect unknown

Figure 4

Anastomosing Flow Area During October 1954 Flood

Agricultural Sheet Flow

Agricultural sheet flow occurs on land surfaces that have been graded or flattened for agricultural use. Lack of topographic variation within the field create sheet flow conditions. Agricultural sheet flow areas differ from natural sheet flow areas in that soil and vegetative identifying characteristics may be obscured by regrading or leveling for irrigation and crop development. Agricultural sheet flow areas may be identified from pre-development photographic or topographic data, or from historic records of flooding. Figure 5 shows a agricultural sheet flow area.

Identifying characteristics of agricultural sheet flow areas include:

- Distributary, anastomosing, or sheet flow channel patterns which are intercepted in the downstream direction by agricultural areas which have no identified drainage facilities.

- Low topographic relief perpendicular to the primary flow direction.

Overland Flow

Overland flow is the movement of water resulting from rainfall on hill slopes in upper watershed areas prior to entering defined channels. The development standards detailed in this document should not be applied to overland flow areas. Overland flow is illustrated in Figure 6.

Identifying characteristics of overland flow areas include:

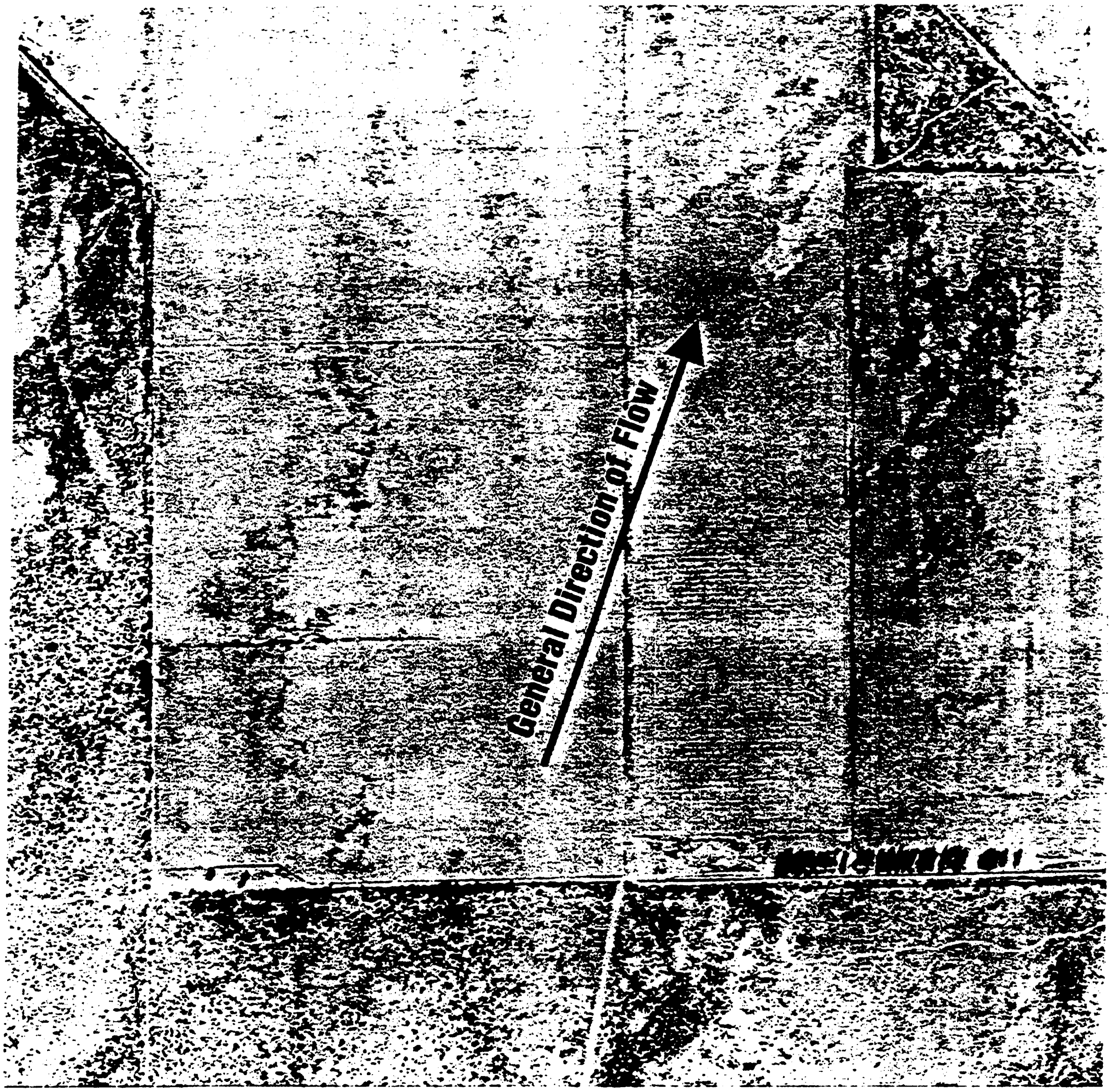
- Overland flow occurs over relatively short distances between the point where surface runoff begins and a nearby, well-defined channel.

- Overland flow occurs near the watershed divides, rather than at the outlet of a watershed, at depths usually less than 6 inches.

- Overland flow usually is a site drainage concern, rather than a regional floodplain management problem.

- Overland flow areas may have a micro-drainage pattern which may be distributary, anastomosing, or completely lacking, but which generally flow into a tributary drainage network.

Overland flow is generally not an important consideration for floodplain management. The development standards outlined in this document generally should not be applied to overland flow areas.



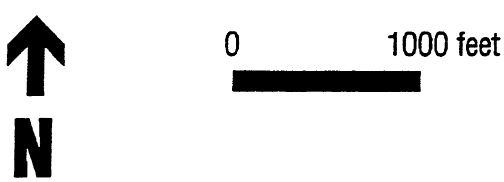


Figure 5

Agricultural Sheet Flow Area

Note Natural Sheet Flow Areas
Upstream of Fields

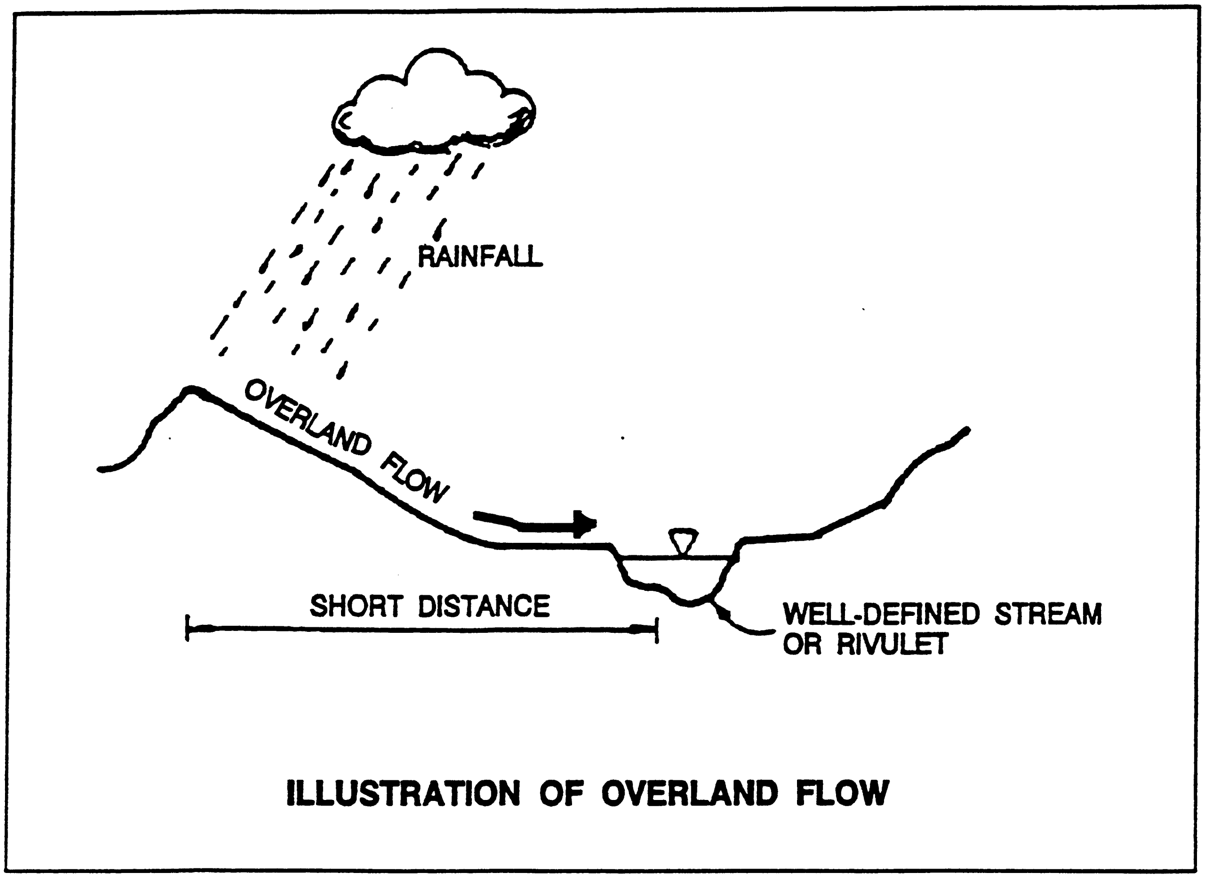


Figure 6

Perched Flow

Perched flow originates along well defined channels where overbank flooding becomes separated from the main flow path, and develops hydraulic characteristics unique from the main channel. For this study, and for the proposed state standard, perched flow is not considered to be sheet flow, unless it meets other characteristics described above. Perched flow is illustrated in Figure 7.

Braided Flow

Braided flow occurs where flow within a well defined channel or floodplain is divided into separate flow paths created by shifting patterns of sediment deposition. Braided flow is not a form of sheet flow. Braided flow is illustrated in Figure 8.

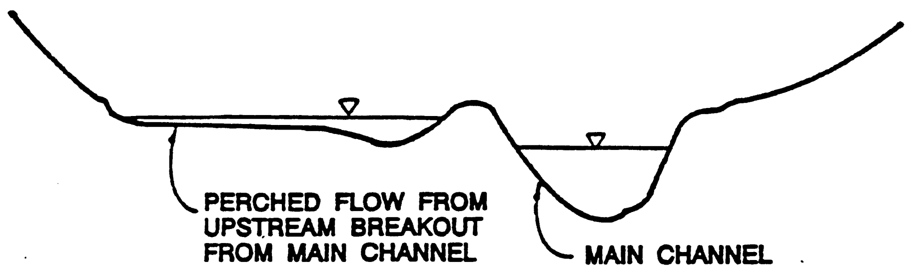


FIGURE 7

ILLUSTRATION OF PERCHED FLOW

CROSS - SECTION

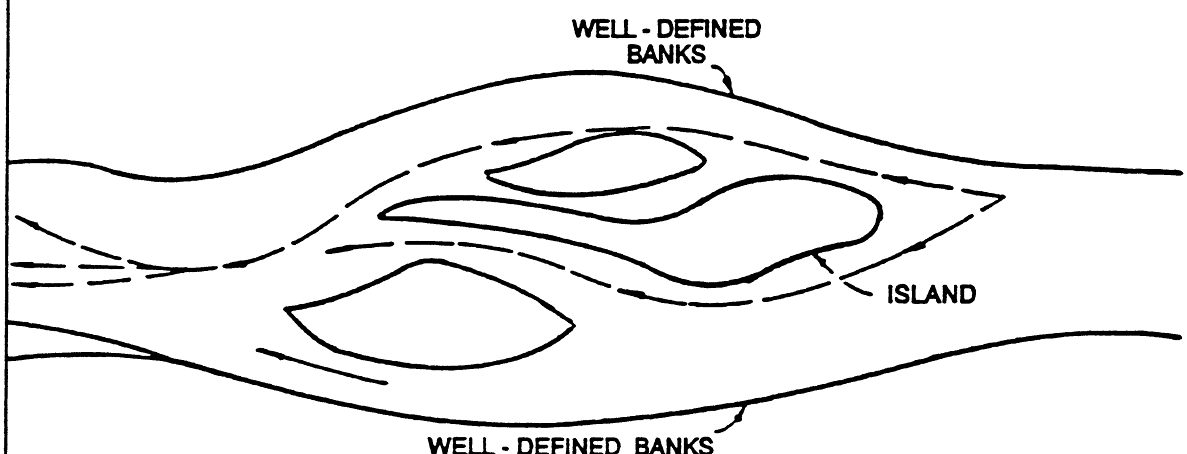


FIGURE 8

ILLUSTRATION OF BRAIDED FLOW

PLAN VIEW

Figure 7 and Figure 8

Sheet Flow Flood Hazards

Sheet flow areas are hydraulically and geomorphically different than riverine, alluvial fan, or other Arizona floodplains. They also have unique flood hazards, including:

- Sheet flooding often occur in areas that have no defined channel or in areas between minor channels in anastomosing or distributary flow networks. Therefore, flood inundation may be unexpected by residents or land owners unfamiliar with sheet flow. Untrained observers may find no indication of the potential for flooding prior to developing a property.

- In sheet flow areas with minor channels, floods frequently exceed bank heights. Development above channel banks does not guarantee adequate flood protection.

- Distribution of runoff between channels may vary between storm events due to minor channel changes upstream. Hydrologic and hydraulic analyses should be done using conservative assumptions for drainage area to ensure that all areas that could contribute runoff are included. Minor watershed changes may significantly increase flood hazards at any given property.

- Sheet flooding may occur over such a broad expanse that a single given property may not have a significant portion which is less flood prone than any other portion.

- Some types of development in sheet flood areas may concentrate flow and alter flow conditions on downstream properties. Accessory development features such as fences, perimeter walls, or roads can have significant impacts on downstream flood hazards.

- Concentration of flow may result in channel (arroyo) formation and initiate headcuts which could propagate upstream and damage structures.

- If natural ground cover is disturbed, flow induced shear stresses on steep land surfaces may cause erosion.

- Sheet flooding over roadways with no drainage structures may prevent access of emergency vehicles for significant periods of time. Sediment deposition on road crossings in sheet flow areas may also delay property access.

- Significant backwater conditions may occur in sheet flow areas upstream of roadways with drainage structures that are not sized for the 100-year flood. Flood depths resulting from these backwater conditions may exceed depths indicated by local geomorphology or field conditions. Required finish floor elevations should consider the potential for backwater.

- Alteration of flow characteristics in sheet flow areas may also alter important wildlife habitat, groundwater recharge, or receiving water characteristics.

Development Standards for Sheet Flow Areas

Minimum development standards for management of all natural and urban sheet flow areas, distributary and anastomosing flow areas in Arizona are shown below. In addition, general recommendations for regulation of development in all sheet flow areas are also outlined. The minimum and recommended standards reflect the types of flood hazards identified for Arizona sheet flow areas.

Required Development Standards

Based on the criteria and information outlined above, the following are minimum standards for development in sheet flow areas:

Natural and Urban Sheet Flow Areas

Habitable structures built in areas subject to natural sheet flooding shall at minimum:

- Elevate the lowest finished floors of all habitable structures. Elevation requirements are described in the *Method of Flow Analysis* section of this document.

- Use appropriate site grading practices to direct nuisance runoff away from the building pad.

Distributary and Anastomosing Flow Areas

Habitable structures built in areas subject to distributary and anastomosing flooding shall at minimum:

- Elevate the lowest finished floor of all habitable structures. Elevation requirements are described in the *Method of Flow Analysis* section of this document.

- Protect the building foundation and related facilities from scour damage and from undercutting from erodible channel banks.

- Use appropriate site grading to direct nuisance runoff away from the building pad.

Recommended Development Standards

The following minimum standards are recommended, but not required, for development in all types of sheet flow areas:

Single Lot Development

- Chain link fences should be elevated 0.5 foot above adjacent grade (a single-strand wire may be allowed below the chain link), or be designed to collapse under hydrostatic pressure, or set back from property line.

- Fences over existing natural channels/flow paths should be elevated or configured to pass bankfull flows unobstructed. Fences that obstruct flow can trap flood debris, and cause erosion or diversion of flow.

- Solid perimeter walls should be set back from property lines to provide flow conveyance between lots, or should have the ability to pass drainage through the walls. Walls designed to pass drainage through should be designed to account for blockage of openings by vegetation and floating debris, and should be able to withstand hydrostatic pressure and scour caused by flow impingement.

- Site grading and building pad locations should allow for continuity of drainage for all recognizable flow paths.

- Homes in single lot developments should be aligned parallel to the primary flow direction.

- Manufactured housing should be anchored to prevent flotation and overturning.

- Building pads should be protected against scour damage.

- Zoning densities higher than 1 residence per acre (RAC) are not recommended in designated sheet flow areas unless drainage studies which analyze potential concentration of flow and downstream impacts are completed or regional flood control facilities are constructed.

- Significant backwater conditions may occur in sheet flow areas upstream of roadways with drainage structures that are not sized for the 100-year flood. Flood depths resulting from these backwater conditions may exceed depths indicated by local geomorphology or field conditions. Required finish floor elevations should consider the potential for backwater. Finished floors should be elevated at least to 0.5 feet above the elevation of the roadway which creates the backwater conditions.

Major Development

Major developments are defined as legal subdivisions with proposed densities greater than 1 residence per acre (RAC), or industrial/commercial developments. For major developments in sheet flow areas, the following standards are recommended:

- Development should not divert or concentrate flow on adjacent properties, unless concentrated flow is conveyed in a drainage facility or natural channel with demonstrated capacity for the base flood discharge.

- Drainage studies prepared for major developments should evaluate the hydrologic impacts to the point where the sheet flow enters a drainage facility or natural channel with demonstrated capacity for the base flood discharge.

- Major facilities should be protected from scour caused by flow concentration, and from erosion of adjacent channel banks.

Methods of Flow Analysis

For development in sheet flow areas, a three-level method of analysis is proposed. Higher levels of analysis are intended to provide more accurate hydraulic data, but may require greater knowledge of hydraulics and increased expense to the floodplain manager or developer. These methodologies must be applied only in sheet flow areas, as defined above, with drainage areas greater than 0.25 square miles, or with a 100-year peak flow rates greater than 500 cfs.

Level I is the minimum level of regulation acceptable, and should be used where only limited site and flood data are available, and where site improvements are minimal. Level II requires a minimal understanding of hydraulics, and is appropriate for single lot development where some flood and site data are available. Level III analysis should be used if regional floodplain management will be impacted by the proposed development.

Level I

Minimum level of site analysis. No hydraulic analysis required. Finished floors should be elevated above the highest natural grade adjacent to the building pad as shown in Table 1, or 1.0 foot above any AO Zone on a Flood Insurance Rate Map (FIRM) for the area, unless greater flooding depths can be predicted from other readily available data, such as historical information. Development standards outlined above in the *General Recommendations for Development in Sheet Flow Areas* section apply.

Table 1—Level I Minimum Finished Floor Elevations (FFE)	
Drainage Area (mi^2)	**Minimum FFE (inches)**
0.25 - 1.0	18
1.0 - 5.0	24
> 5.0	30

Level II

Estimate base flood elevation using Manning's rating[3] or equivalent procedure. Note that in no case shall the minimum finished floor elevation of habitable structures be elevated less than 1.0 foot above highest adjacent natural grade adjacent to the building pad. To perform a Manning's rating the following data are needed: (1) Discharge—the 100-year flow rate; (2) Topography or cross sections—of site and sheet flow area; (3) Roughness Coefficient—Manning's "N" value; and (4) Slope—valley slope parallel to the primary flow direction. Potential data sources are described below.

Discharge

The 100-year discharge may be estimated using simplified methodologies such as ADWR State Standard #2 (SS 2-92), USGS regression equations[4], or other appropriate local or more detailed methods. Drainage areas should be estimated conservatively to account for all possible sources of runoff. USGS topographic quadrangle maps usually provide sufficient detail for delineating watershed areas.

Topography

Topography should be obtained from the best available information. Topography should describe ground contours for both the site and the total sheet flow area. For natural sheet flow areas, topography may be obtained from USGS topographic quadrangle maps, unless better data are available. For distributary and anastomosing flow areas, topography should be obtained from detailed mapping, tape and level survey data obtained during a site visit, or estimated from aerial photography. Topographic data for distributary and anastomosing flow areas should include descriptions of channel widths, bank heights, and vegetation density. For urban sheet flow areas, descriptions of topography should include areas where flow would be blocked by buildings, fences, or other obstructions.

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3 Use of Manning's equation assumes that uniform flow conditions exist. Floodplain managers should verify likelihood of uniform flow, prior to applying Level II method of analysis.

4 The current USGS equations are in Blakemore, 1994.

Roughness Coefficient ("N" value)

Table 2 lists roughness coefficients acceptable for use in sheet flow areas. The Manning's "N" value selected should adequately account for vegetation, sediment size, blocking of flow by flood debris, and variations in channel geometry. Several publications describe techniques for estimating "N" values (Arcement and Scheider, 1984; Thomsen and Hjalmarson, 1991).

Table 2
Recommended Manning "N" Values for Sheet Flow Areas[5]

Surface	N Value	Range
Concrete	.011	.010 to .013
Bare Sand	.01	.010 to .016
Gravel	.02	.012 to .03
Desert Brush	.05	.03 to .07
Natural Rangeland	.13	.01 to .32
Dense Grass	.24	.17 to .30
Bermuda Grass	.41	.30 to .48

Slope

Slope used in the rating should be the valley slope or channel slope, whichever is less. Slope may be measured from USGS topographic quadrangle maps or measured during a site visit. Slope should be measured parallel to the general direction of flow.

Minimum Elevation

In no case shall the minimum finished floor elevation of new habitable structures in sheet flow areas which meet the criteria of this standard be less than 12 inches above the highest natural existing grade adjacent to the building pad.

Level III

Full hydrologic and hydraulic analysis using computer models. Hydraulic modeling should consider the potential for a strong two- or three-dimensional character to flooding; one-dimensional computer modeling of water surfaces and depths may not be appropriate in many sheet flow areas. Two- and three-dimensional models may not be cost-effective

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[5] Sources: Woolhiser, D.A., 1975; Engman, E.T., 1986; Weltz, M.A., Arslan, A.B., and Lane, L.J., 1992

for smaller developments. Selection and application of appropriate modeling techniques should be made by a qualified and experienced registered engineer. The FEMA alluvial fan methodology should not be used for floodplain management purposes on sheet flow areas in Arizona.

Works Cited

Arcement, G.J., Jr, and V.R. Scheider. 1984. Guide for Selecting Manning's Roughness Coefficients for Natural Channels and Flood Plains: USGS Water Supply Paper 2339.

Blakemore, E.T., H.W. Hjalmarson, and S.D. Waltemeyer. "Methods for Estimating Magnitude and Frequency of Floods in the Southwestern United States." *USGS Open-File Report.* 1994. 93-419.

Engman, E.T. "Roughness Coefficients for Routing Surface Runoff." *ASCE Journal of Irrigation and Drainage Engineering.* V. 112, No. 1. 1986. Pp. 39-53.

Hjalmarson, H.W., and S.P. Kemna. "Flood Hazards of Distributary-Flow Areas in Southwestern Arizona." *USGS Water-Resources Investigation.* 1991. 91-4171.

Thomsen, B.W., and H.W. Hjalmarson. Estimated Manning's Roughness Coefficients for Stream Channels and Flood Plains in Maricopa County, Arizona. Report to the Flood Control District of Maricopa County. April, 1991.

Weltz, M.A., A.B. Arslan, and L.J. Lane. "Hydraulic Roughness Coefficients for Native Rangelands." *ASCE Journal of Irrigation and Drainage Engineering.* V. 118, No. 5. 1992. Pp. 776-790.

Woolhiser, D.A. "Simulation of Unsteady Overland Flow." In K. Mahmood and V. Yevjevich, eds. *Unsteady Flow in Open Channels.* V. II. P. 502. 1975.

Test Applications

Example 1: Natural Sheet Flow

- **Problem Statement.** Single lot development proposed on 1-acre parcel with no visible channels. Area is covered by dense grass and brush. A watershed of two square miles drains toward the site, which is located in a broad, flat valley approximately 1/2-mile wide. See Figure E1.

Example 2: Distributary/Anastomosing Flow

- **Problem Statement.** Single lot development proposed on 1-acre parcel on lower portion of alluvial fan with distributary channels, and covered by desert brush with some riparian vegetation along more defined flow paths. Defined channels have sand and gravel bed material. A watershed of one square mile drains toward the site. See Figure E2.

Example 3: Urban Sheet Flow

- **Problem Statement.** Single lot development proposed on 1/6-acre parcel in residential urban area with no flood control channels or storm drains. Low flow is conveyed in the streets. Higher flows overflow into yards. Backyard areas are generally surrounded by block wall or solid fences. A watershed of one square mile drains toward the site. See Figure E3.

Example 1: Single Lot Development in Natural Sheet Flow Area

- **Description.** Dense tall grass, no defined channels, flat valley bottom
- **Discharge.** 1,000 cfs, obtained from local hydrology methodology
- **Drainage Area.** 2 square miles (mi^2, 1280 acres)
- **Topography.** Determined by hand-level survey during site visit
- **N Value.** 0.24 (Table 2)
- **Valley Slope.** 0.009 ft/ft, measured on USGS quadrangle map for area

Results of Level I Analysis (Figure E1-b)

- **Drainage Area = 2 mi^2.** Elevate finished floor 24 inches (2.0 ft) above highest adjacent natural grade.

Results of Level II Analysis (Figure E1-c)

- Used Manning's rating of valley section A-A' to estimate flow depth = 0.6 ft (7 inches).
- **Finished Floor Elevation = 1.6 ft.** Elevate finished floor 19 inches (1.6 ft) above highest grade adjacent to the building pad.

Results of Level III Analysis

- The advanced computer modeling of design discharge and flow hydraulics required is not illustrated here. Regardless of results of Level III analysis, the minimum finished floor elevation should be 1.0 foot above computed water surface elevation, and no less than 1.0 foot above highest adjacent grade adjacent to the building pad. Level III analysis is probably not cost-effective for this application.

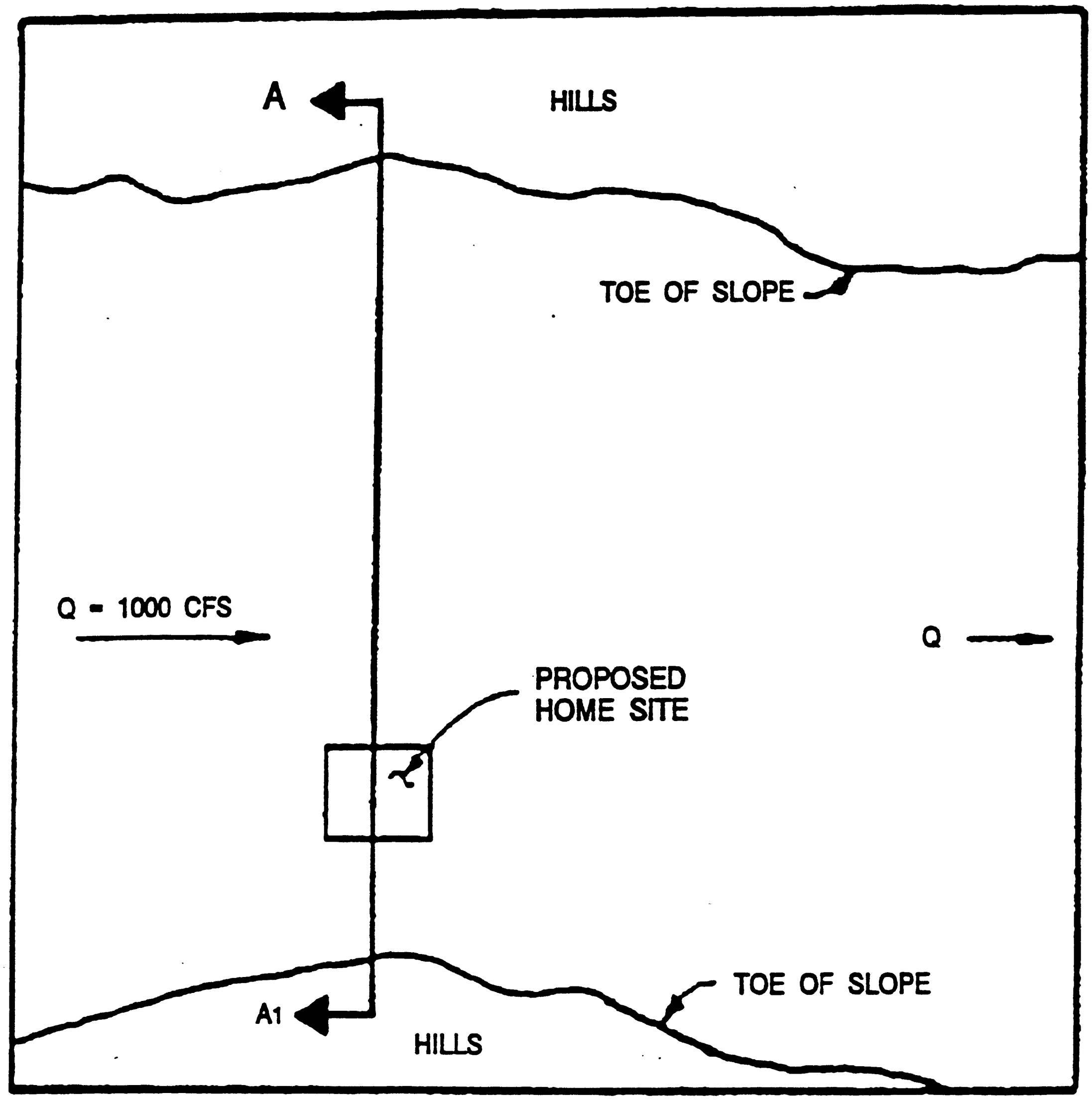

FIGURE E1-a

GENERAL PLAN VIEW OF AREA FOR EXAMPLE 1

ILLUSTRATION NOT TO SCALE

Figure E1-a

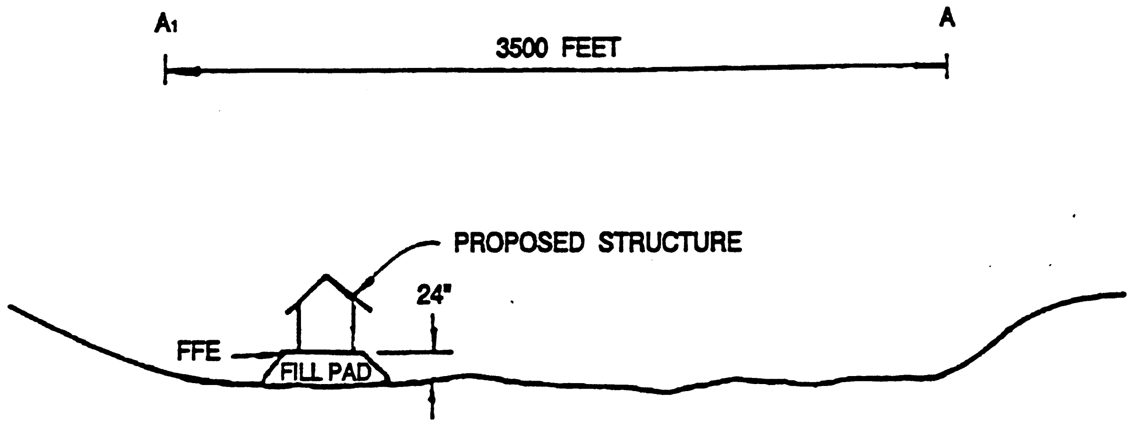

FIGURE E1-b

LEVEL I ANANYSIS OF EXAMPLE 1
(FFE = NATURAL GRADE + 2 FEET)

ILLUSTRATION NOT TO SCALE

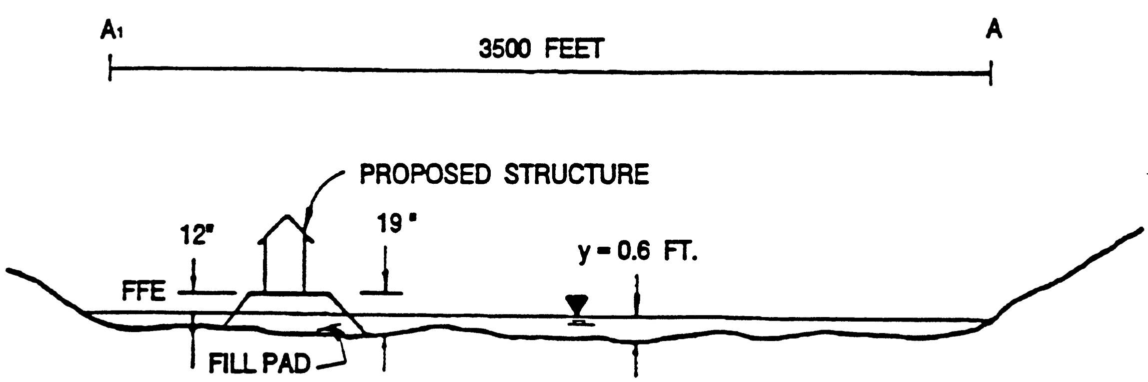

FIGURE E1-c

LEVEL II ANALYSIS OF EXAMPLE 1

ILLUSTRATION NOT TO SCALE

Figure E1-b and Figure E1-c

Example 2: Single Lot Development in Distributary/Anastomosing Flow Area

- **Description.** Desert brush, sand and gravel, with interconnected channels
- **Discharge.** 2,000 cfs, obtained from USGS regression equations
- **Drainage Area.** 5.2 square miles (5.2 mi^2, 3330 acres)
- **Topography.** Determined by tape and level survey during site visit, verified on vertical aerial (stereo) photographs
- **N Value.** 0.045 (Table 2)
- **Valley Slope.** 0.02 ft/ft, measured on USGS quadrangle

Results of Level I Analysis (Figure E2-a)

- **Drainage Area = 5.2 mi^2.** Elevate finished floor 30 inches (2.5 ft.) above highest adjacent natural grade (Elevation 5.9 in Figure E2-a).

Results of Level II Analysis (Figure E2-b)

- Manning's rating using HEC-2 program with single cross section, and tape and level survey points. Computed water surface elevation = 4.3 ft.
- **Finished floor elevation = 5.4 ft.** Elevate lowest floor 1.0 foot above computed water surface elevation of 4.3 ft. (5.3 ft.), and highest adjacent natural grade of 4.4 ft. (5.4 ft.). Use the higher value of 5.4 ft.
- Floodplain manager should also make judgement regarding erosion hazards.

Results Level III Analysis

- Advanced computer modeling of design discharge and flow hydraulics required. Minimum finished floor elevation 1.0 foot above computed water surface elevation, and no less than 1.0 foot above highest adjacent grade adjacent to the building pad.

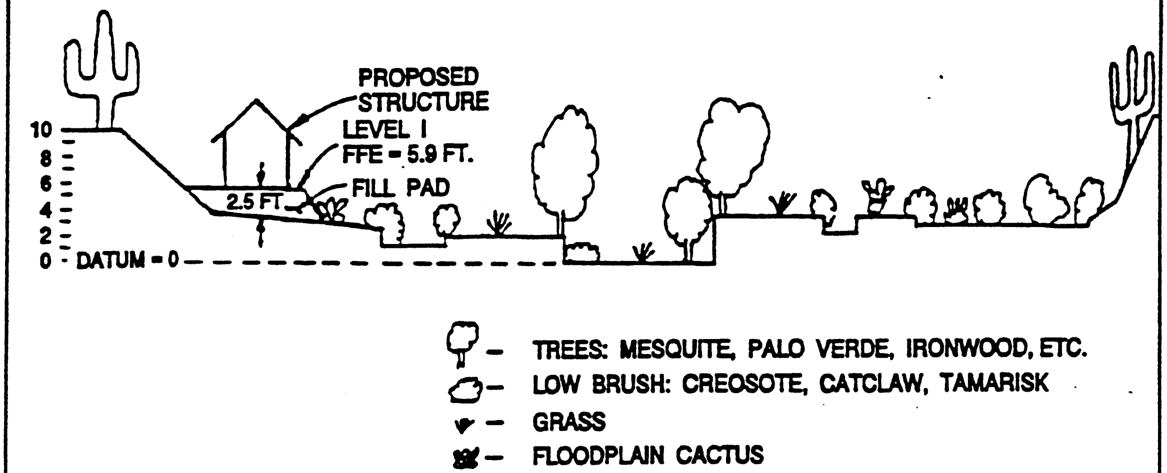

FIGURE E2-a

LEVEL 1 ANALYSIS OF EXAMPLE 2
(FFE = NATURAL GRADE + 2.5 FEET)

ILLUSTRATION NOT TO SCALE

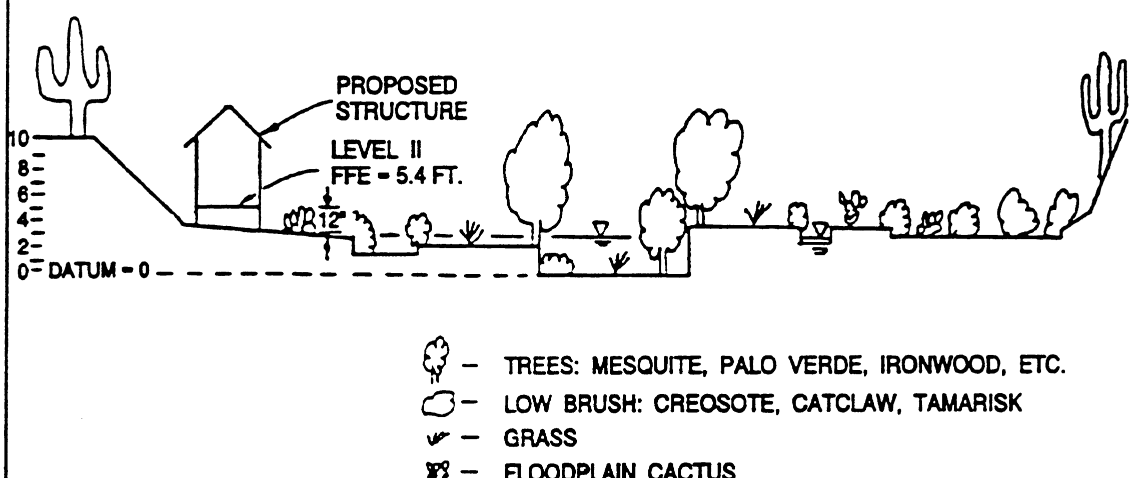

FIGURE E2-b

LEVEL II ANALYSIS OF EXAMPLE 2
(FFE = 100-YEAR WATER SURFACE + 1 FOOT)

ILLUSTRATION NOT TO SCALE

Figure E2-a and Figure E2-b

Example 3: Urban Sheet Flow

- **Description.** Residential landscaping, with perimeter walls between lots

- **Discharge.** 2,000 cfs, obtained from State Standard 92-02.

- **Drainage Area.** 1.5 square miles (1.5 mi^2, 960 acres)

- **Topography.** Determined from 1:1200, 2 ft contour interval mapping by local community, checked during site visit

- **N Value.** 0.3 for landscaping, 0.011 for streets and sidewalks, block out fence and home areas (Table 2)

- **Valley Slope.** 0.005 ft/ft, measured on detailed city topography

Results of Level I Analysis (Figure E3-a)

- **Drainage Area 1.5 mi^2.** Elevate finished floor 24 inches (2.0 ft) above natural grade. (Elevation 4.5 in Figure E3-a.)

Results of Level II Analysis (Figure E3-b)

- Manning's rating using HEC-2 program with single cross section, and ground elevation points from topographic map. Computed water surface elevation = 3.0 ft.

- Finished floor elevation = 4.0 ft, but no less than 1.0 foot above highest grade adjacent to the building pad (grade at 2.5 ft in Figure E2-b).

Results of Level III Analysis

- Advanced computer modeling of design discharge and flow hydraulics required. Minimum finished floor elevation 1.0 foot above computed water surface elevation, and no less than 1.0 foot above highest adjacent grade adjacent to the building pad.

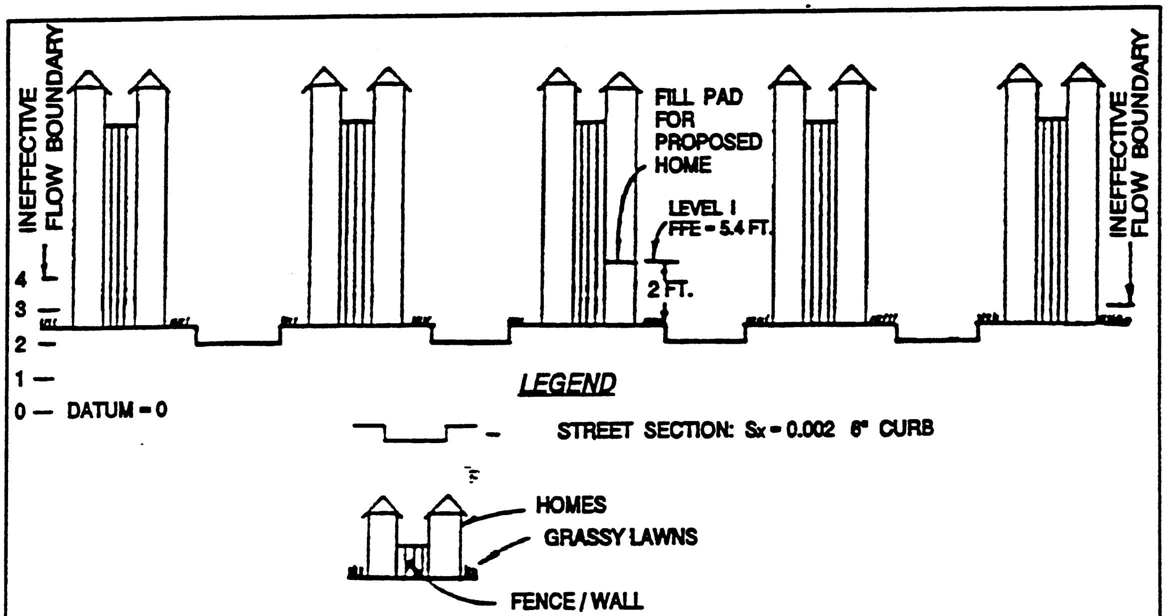

FIGURE E3-a

LEVEL I ANALYSIS OF EXAMPLE 3
(FFE = NATURAL GRADE + 2.0 FEET)

ILLUSTRATION NOT TO SCALE : VERTICAL EXAGGERATION INTENTIONAL

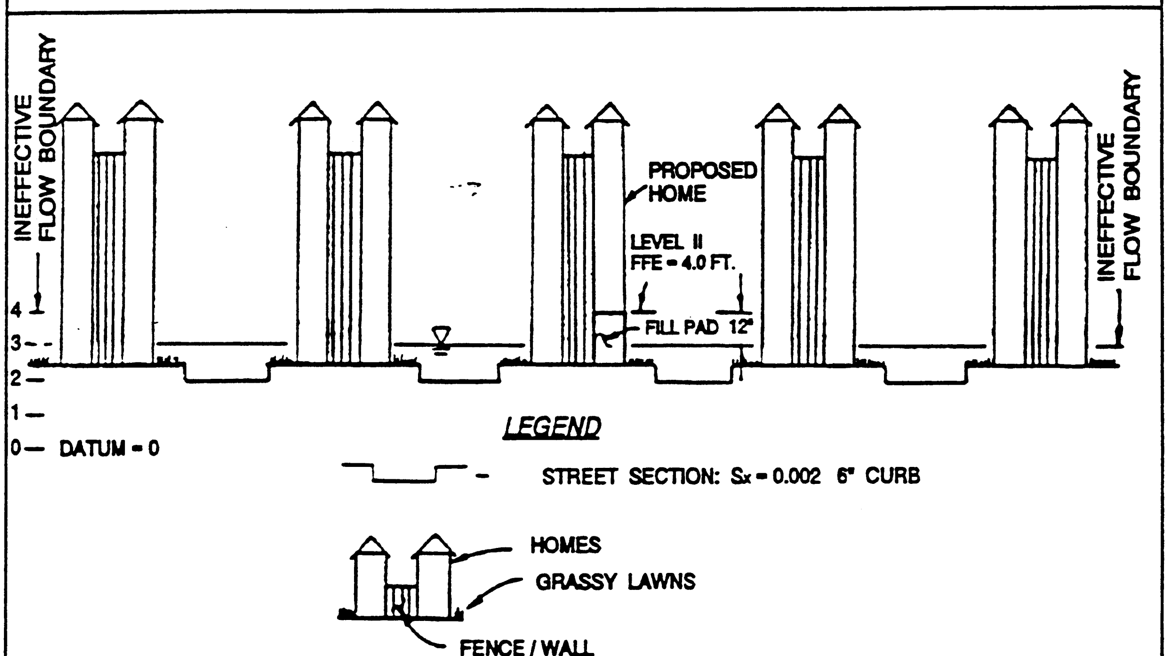

FIGURE E3-b

LEVEL II ANALYSIS OF EXAMPLE 3
(FFE = 100-YEAR WATER SURFACE + 1 FOOT)

ILLUSTRATION NOT TO SCALE : VERTICAL EXAGGERATION INTENTIONAL

Figure E3-a and Figure E3-b

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ARIZONA DEPARTMENT OF WATER RESOURCES
FLOOD WARNING AND DAM SAFETY SECTION

State Standard

for

Watercourse System Sediment Balance

Under authority of ARS 48-3605(a), the Director of the Arizona Department of Water Resources establishes the following standard for identification of and development within erosion hazard areas and areas affected by a net system sediment deficit or surplus in Arizona:

The guidelines outlined in State Standard Attachment 5-96 entitled "Watercourse System Sediment Balance" or by an alternative procedure reviewed and accepted by the Director will be used in the identification of, or regulation of development within erosion hazard areas, and watercourses affected by a net system sediment deficit or surplus in Arizona for fulfilling the requirements of Flood Insurance Studies, and local community and county flood damage prevention ordinances.

For the purpose of application of these guidelines, erosion hazard area and watercourse system sediment balance standards will apply to all watercourses identified by the Federal Emergency Management Agency as part of the National Flood Insurance Program, all watercourses which have been identified by the local floodplain administrator as having significant potential flood hazards and all watercourses with drainage areas more than 1/4 square mile or a 100-year discharge estimate of more than 500 cubic feet per second. Application of these guidelines will not be necessary if the local community or county has in effect a drainage, grading or stormwater ordinance which, in the opinion of the Department, results in the same or greater level of flood protection as application of these guidelines would ensure.

This requirement is effective October 1, 1996. Copies of this State Standard and State Standard Attachment 5-96 can be obtained by contacting the Department's Flood Warning and Dam Safety Section at (602) 417-2445.

NOTICE

This document is available in alternative formats. Contact the Department of Water Resources, Flood Warning and Dam Safety Section at (602) 417-2445 or (602) 417-2455 (TDD).

ARIZONA DEPARTMENT OF WATER RESOURCES
FLOOD WARNING AND DAM SAFETY SECTION

Watercourse System Sediment Balance

500 North Third Street
Phoenix, Arizona 85004

(602) 417-2445

STATE STANDARD ATTACHMENT
SSA 5-96

SEPTEMBER 1996

Contents

Introduction

The National Flood Insurance Program regulations 44 CFR 60.5 require local communities to review permits for development with regard to erosion hazards in "flood-related erosion-prone areas". Specifically 44 CFR 60.5.a.2 states "...Require review of each permit application to determine whether the proposed site alterations and improvements will be reasonably safe from flood-related erosion and will not cause flood-related erosion hazards or otherwise aggravate the existing flood-related erosion hazard....".

This document contains three guidelines for identification of, and development within erosion hazard areas, watercourses with a net sediment deficit, and watercourses with a net sediment surplus. The three guidelines in this document each contain their own table of contents relevant to its particular subject. These guidelines are:

Guideline 1: Lateral Migration Setback Allowance for Riverine Floodplains in Arizona

Guideline 2: Channel Degradation Estimation for Alluvial Channels in Arizona

Guideline 3: Evaluation of River Stability Impacts associated with Sand and Gravel Mining

Guideline 1 presents procedures for estimating the size of buffer (setback distance) that shall be provided along watercourses to allow for the lateral migration that may occur during future floods. Three methods of setback evaluation are discussed -- a first level procedure to be applied in normal conditions, a second level procedure for use in demonstrating the erosion resistance of existing channel materials, and a third level procedure which may be applied in unusual circumstances, or where more definite dimensioning of lateral migration potential is desired.

Guideline 2 presents procedures that may be used for estimation of channel degradation in unlined watercourses within Arizona. Three levels of procedures are provided, with data requirements, procedural complexity, and accuracy of results all increasing as the analysis level is incremented. The Level I approach provides an initial estimate of local channel degradation potential for generally stable, natural channel conditions. The resulting initial estimate may be reduced through use of the more rigorous Level II methodologies. Level III procedures are outlined for situations that warrant more detailed channel degradation determination.

Guideline 3 presents general guidelines that have been developed for determination of the adequacy of buffer areas between proposed mining operations and active river channels, and procedures that are available for analysis of the effects of instream activities.

A large part of Arizona has a "Basin and Range" topography which consists of mountain "blocks" of hard rock areas and adjoining basins that are filled with sediments which have been deposited by water (alluvium). The mountain areas do not have a problem with channel migration due to the stability of bed rock and large fragment rock found there. Basin areas, or the valley and low land areas containing alluvium are characterized by sediments that are erodible. The many variables associated with channel lateral migration, sediment balance, river mechanics, and hydraulic engineering preclude the development of a comprehensive design manual in this short document: therefore, these guidelines are intended to be utilized with good engineering judgement and common sense.

Within this document the following acronyms will be used:

ADWR	Arizona Department of Water Resources
FEMA	Federal Emergency Management Agency
NFIP	National Flood Insurance Program

GUIDELINE 1

Lateral Migration Setback Allowance for Riverine Floodplains in Arizona

TABLE OF CONTENTS

Introduction

The floodplain boundaries associated with a given watercourse are not fixed features if the channel shifts and migrates over the course of time. Lateral migration of river channels is commonly observed in the arid southwest, where the flows are predominantly ephemeral and the bed and banks tend to be erodible. The migration relocates the channel banks and redefines the location of the river for the current and subsequent flow events.

This document presents procedures for estimating the size of buffer (setback distance) that shall be provided along watercourses to allow for the lateral migration that may occur during future floods. Three methods of setback evaluation are discussed -- a first level procedure to be applied in normal conditions, a second level procedure for use in demonstrating the erosion resistance of existing channel materials, and a third level procedure which may be applied in unusual circumstances, or where more definite dimensioning of lateral migration potential is desired.

Procedure

General

Three levels of analysis procedures are presented for determination of recommended setback distances for development in areas adjacent to watercourses. The Level I procedure provides a reasonable estimate of safe setback distance under normal conditions, with minimal channel geometry and hydrologic information required in its application. The higher level procedures, Level II and Level III, are more rigorous means of determining lateral migration potential, requiring knowledge of site specific hydraulic and channel material characteristics. The Level II procedure is provided as a straightforward means of demonstrating the stability of channel banks, in cases where a developer or floodplain manager seeks to apply a lesser setback than may be computed through application of the Level I equations. A flowchart outlining the procedure is provided on the following page. The Level III approaches referenced may be used for this purpose as well, or may be required by the local regulating agency for analysis of areas of particular concern, such as the following situations where the Level I allowances or Level II evaluations may not fully demonstrate the lateral migration potential:

(i) areas where massive shifting of the river channel has been observed in the past;
(ii) areas undergoing channel filling (aggradation) to a significant degree;
or, (iii) areas where local river mining, channelization, or other modifications could result in flow redirection unanticipated in the development of the Level I or Level II approaches.

Level I

This level of analysis requires the following information:

Drainage area. The area of the watershed contributing to the site of interest. Drainage areas should be estimated conservatively to account for all possible sources of runoff. USGS topographic quadrangle maps usually provide sufficient detail for delineating watershed areas.

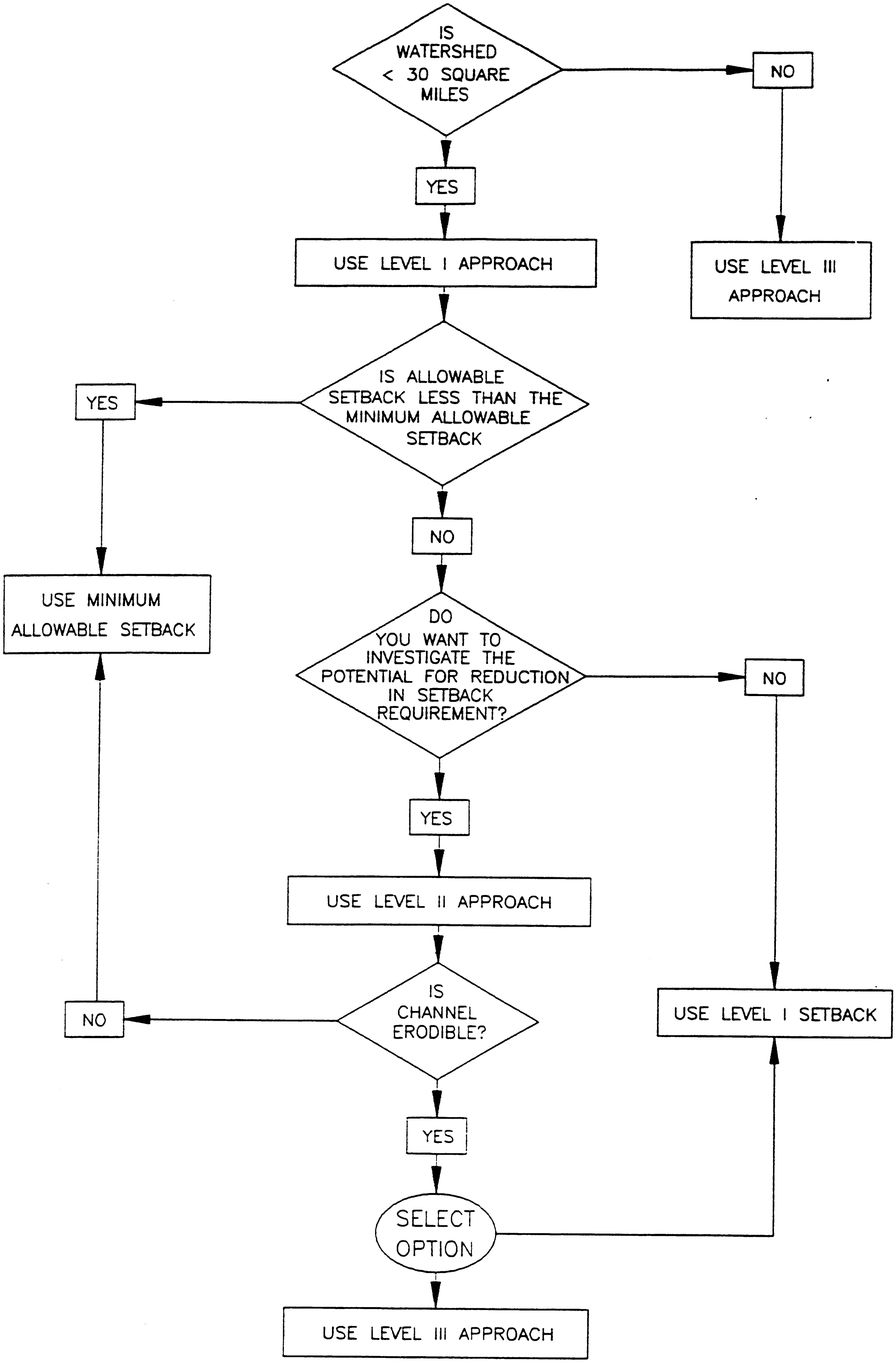
Peak discharge associated with the 100-year flood (Q_{100}). May be estimated using simplified methodologies such as ADWR State Standard #2 (SS 2-96), USGS regression equations, or other similar approximate method.

A Level I or Level II analysis should not be used on watercourses which have drainage areas greater than 30 square miles. If the watercourse has a drainage area greater than 30 square miles, a Level III analysis shall be performed.

For watercourses which have drainage areas of less than 30 square miles, the recommended setback allowances are as follows:

for straight channel reaches or reaches with minor curvature:	setback $= 1.0(Q_{100})^{0.5}$
for channels with obvious curvature or channel bend:	setback $= 2.5(Q_{100})^{0.5}$

where setback is in feet and Q_{100} is in cubic feet per second.

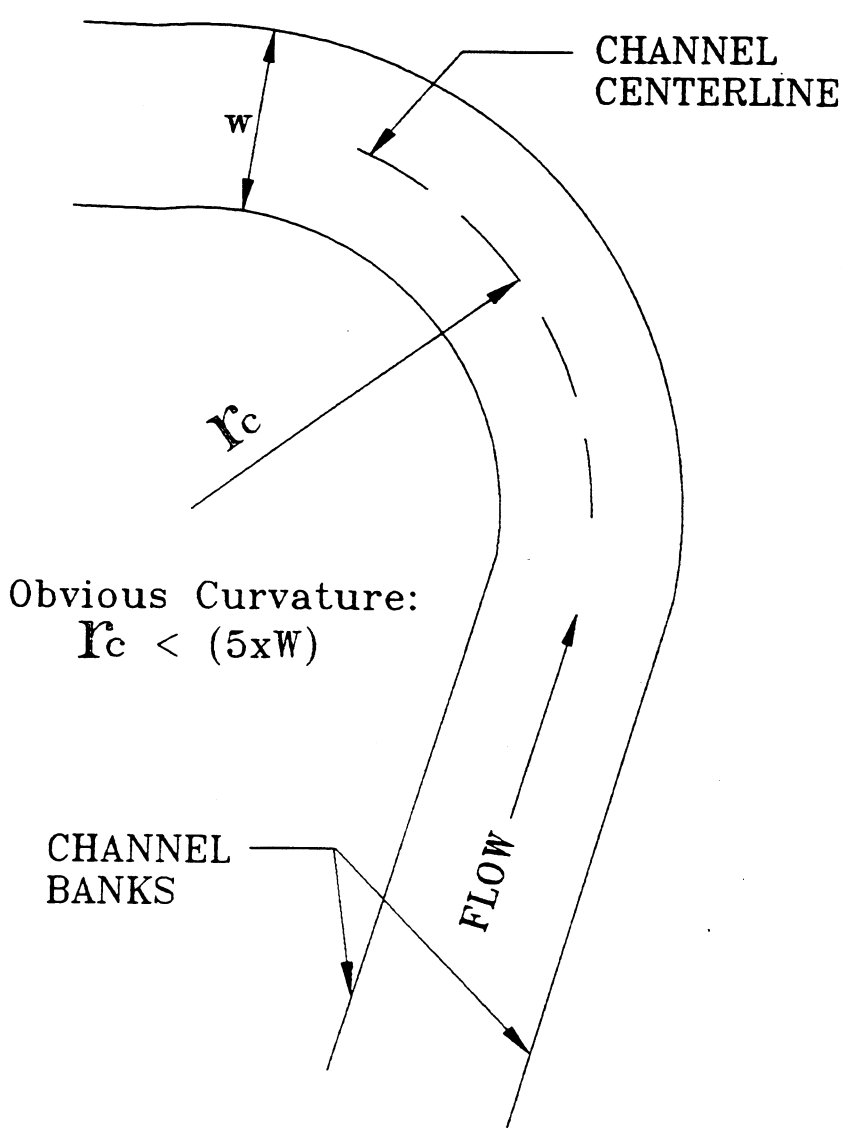

IS WATERSHED < 30 SQUARE MILES
NO
YES
USE LEVEL I APPROACH
USE LEVEL III APPROACH
IS ALLOWABLE SETBACK LESS THAN THE MINIMUM ALLOWABLE SETBACK
YES
NO
USE MINIMUM ALLOWABLE SETBACK
DO YOU WANT TO INVESTIGATE THE POTENTIAL FOR REDUCTION IN SETBACK REQUIREMENT?
NO
YES
USE LEVEL II APPROACH
IS CHANNEL ERODIBLE?
NO
USE LEVEL I SETBACK
YES
SELECT OPTION
USE LEVEL III APPROACH

FLOWCHART

In all cases for the Level I analysis, the minimum setback shall be 20 feet for straight channel reaches and 50 feet for channels with obvious curvature. Obvious curvature is defined as a channel centerline with a radius of curvature less than 5 times the channel top width.

The setback allowance is to be measured outward from the 100-year floodway or the top of the channel bank, whichever is greater. The above equations provide a larger setback allowance in areas with relatively tight channel bends. This larger setback allowance is to applied in areas adjacent to the outside bend of the channel.

A sketch is provided below to help differentiate between minor curvature and obvious curvature.



CHANNEL CURVATURE

Level II

This approach may be applied to demonstrate the stability of the channel material under 100-year flood conditions, and to justify a lesser setback requirement than that computed using the Level I equations. Setback allowances for conditions which pass one or more of the following channel stability approaches, and which are not located in areas of specific concern (i.e. areas adjacent to river mining sites, highly aggradational areas, or areas with artificial flow redirection) should be based on normal building safety criteria rather than the Level I equations presented above, since the bank limits would not be expected to change during the course of a 100-year design event.

Allowable velocity analysis

Under this approach, the velocity of the 100-year peak flow within the watercourse adjacent to the site under consideration is compared to an "allowable" velocity -- the velocity at and below which erosion is not expected to occur.

The basic maximum allowable velocity for unprotected earthen channels is determined from a relationship developed by the USDA Soil Conservation Service, shown in the attached **Figure 1**. In order to use this figure, flow must be classified as either sediment free or sediment laden. Sediment free flow is defined as flow in which fine material in suspension is at concentrations so low that it has negligible effect upon channel stability. Sediment free flows generally have sediment concentrations of less than 1,000 parts per million (ppm) by weight. Sediment-laden flows are classified as flows carrying sediments in concentrations equal to or exceeding 20,000 ppm, by weight.

Typical natural channel flows within Arizona can be characterized as sediment-laden when flow occurs. The sediment-free curve in Figure 1 should be used only under unusual circumstances, such as for runoff which emanates from a totally impervious watershed.

Use of Figure 1 requires that the D_{75} particle size (the size for which 75% of the sediment, by weight, is finer) be known for the soil forming the channel banks. This information can be obtained from a sieve analysis or alternate means should there be large fragmented rock present.

The basic allowable maximum velocity obtained from Figure 1 must normally be modified to account for variations in channel design. This is done by the use of correction factors for channel alignment, bank slope, and depth of flow. The equation for allowable velocity, V_a, in an unprotected earthen channel then becomes:

$$V_a = V_b \times C_a \times C_b \times C_d$$

where

V_a = Maximum allowable flow velocity, in feet per second;
V_b = Basic maximum allowable flow velocity obtained from Figure 1, in feet per second; and,
C_a, C_b, C_d = Correction factors for channel alignment, bank slope, and flow depth, respectively (see Figure 2 through 4).

Tractive stress analysis

Flowing water exerts a tangential boundary pull on the wetted perimeter of the channel boundary. The total force exerted on the boundary by the flow of water is called the tractive force. The tractive stress is the tractive force per unit area of the boundary. Tractive force and tractive stress are equal to the friction forces resisting the flow of water. Tractive stress can therefore be used as a method of determining the erodibility of an earthen channel. To accomplish this, the tractive stress is compared to an allowable tractive stress for the bed material.

Case 1: 0.25 inches $< D_{75} <$ 5.0 inches

The tractive stress acting on the soil grains in an infinitely wide channel can be computed from:

$$\tau_\infty = \gamma_w Y \, [\, D_{75}^{1/6} / 39n \,]^2 \, S_e$$

where

τ_∞	=	Tractive stress for an infinitely wide channel, in lbs/ft^2;
γ_w	=	Unit weight of water = 62.4 lbs/ft^3;
D_{75}	=	Diameter of soil particle for which 75 percent of the total soil consists of smaller particles, in inches;
n	=	Manning's roughness coefficient for the channel;
Se	=	Energy slope of flowing water, in feet per foot; and,
Y	=	Depth of flow, in feet.

Once the tractive force for an infinitely wide channel is determined, it must be modified for a narrower trapezoidal channel. **Figures 5 through 7** give correction factors for tractive stresses in trapezoidal and curved channels. The correction factors taken from these figures are multiplied by the tractive stress computed from the above equation to obtain the actual tractive stress.

The definitions of the symbols shown in Figures 5 through 7 are as follows:

τ_s	=	Actual maximum tractive stress on sides of straight trapezoidal channels, in pounds per square foot;
τ_{sc}	=	Actual maximum tractive stress on sides of trapezoidal channels within a curved reach, in pounds per square foot;
τ_{st}	=	Actual maximum tractive stress on sides of trapezoidal channels in straight reaches immediately downstream from curved reaches, in pounds per square foot;
Z	=	Channel side slope (horizontal/vertical), in feet per foot;
b	=	Channel bottom width, in feet;
y	=	Flow depth, in feet;
r_c	=	Radius of curvature of channel centerline, in feet; and,
L_c	=	Length of curve, in feet.

The actual tractive stress is compared to an allowable tractive stress to determine the propensity of the soil to erode under the expected hydraulic conditions. The allowable tractive stress is calculated by:

$$\tau_{ls} = 0.4 \left[(Z^2 - \mathrm{Cot}^2\, \phi R) / (1+Z^2) \right]^{1/2} D_{75}$$

where

τ_{ls}	=	Allowable tractive stress, in lb/ft^2; and,
ϕR	=	Angle of repose of soil, in degrees (see **Figure 8**).

Case 2: $D_{75} \leq 0.25$ inches

Under these conditions, a reference tractive stress as determined from **Figures 9 and 10** is used, following the steps listed below:

1. Determine the velocity (V), kinematic viscosity (ν), and the energy slope (S_e) for the channel.

2. Enter Figure 9 or 10, from the top, with a value computed from the expression:

$$V^3 / (g \nu S_e)$$

Find the point of intersection of the above value and the value of:

$$V / (gk_sS_e)^{1/2}$$

where

k_s = Equivalent roughness height = D_{65}, in feet (the size for which 65% of the sediment, by weight, is finer).

3. Move horizontally along the figure to read the numerical value for:

$$V / (\tau/\rho)^{1/2}$$

where

τ = Reference tractive stress, in pounds per square foot;
V = Flow velocity, in feet per second; and,
ρ = Density of water = 1.94 slugs per cubic foot.

The value for τ can be found by equating the numeric value read from Figure 9 or 10 to this expression.

The maximum tractive stress on the sides of the channel, τ_s, can be computed from the reference tractive stress and a correction factor obtained from **Figure 11**. Figures 6 and 7 may be used to further modify the reference tractive stress for curved channel reaches. The adjusted reference tractive stress is then compared to the allowable tractive stress determined from **Figure 12**.

Curve number 1 in Figure 12 is to be used when the flow is expected to have a high sediment content. A high sediment content is considered to be 20,000 ppm, by weight, or more of sediment. Curve number 2 is to be used for watercourses with low sediment contents of no more than 2,000 ppm, by weight. This curve should only be used in association with areas of high impervious cover (> 50%) and/or downstream of urban area detention basins. Interpolate between curves 1 and 2 for water courses with known sediment content between 2,000 ppm and 20,000 ppm. Curve number 3 is to be used for watercourses conveying clear water, and should not be used unless unusual circumstances exist (e.g., runoff which emanates from a totally impervious watershed).

Tractive power analysis

Tractive power is defined as the product of the mean velocity of flow and the tractive stress. The tractive power analysis takes into consideration the effects of cementation, partial lithification, and other long-term processes that can affect the ability of the channel to withstand erosion. Neither the velocity analysis nor the tractive stress analysis account for the effects of these long-term processes. With the tractive power approach, the stability of saturated soils comprising the channel banks is first assessed by the use of an unconfined compression test. The unconfined compressive strength (UCS) of these saturated embankment soils is then reduced by at least a factor of two, for design purposes, and compared to the tractive power of the flow by use of **Figure 13**. Conditions falling above the S-line in this figure are considered to be erosive, and those falling below the S-line are considered to be non-erosive. The method has some limitations due to variability and stratification of material along natural channels, and the limited data available to develop Figure 13.

Bank Lining Adequacy Analysis

Bank lining of some form may be proposed or already in place which may act to limit the lateral migration potential of the watercourse of concern. In some areas within Arizona, procedures are in place for assessment of the adequacy of the bank protection measures. For areas without standardized procedures, two references are recommended which detail evaluation procedures:

Design Manual for Engineering Analysis of Fluvial Systems, Arizona Department of Water Resources, 1985.

Standards Manual for Drainage Design and Floodplain Management in Tucson, Arizona, City of Tucson Department of Transportation, Engineering Division, 1989.

Level III

This level of analysis involves modeling the hydraulic and sediment transport characteristics of the local watercourse in order to simulate the erosion/sedimentation and channel deformation processes which are expected to occur in the area of concern. For this level of analysis, Level III hydrology shall be performed to generate required hydrographs. Level III analyses should be performed by persons with knowledge and experience in the fields of sediment transport and river geomorphology. It is recommended that any movable boundary river modeling used for establishment of setback be the culmination of a thorough analysis consisting of:

(1) evaluation of historical trends;
(2) qualitative analysis based on field evaluation and application of geomorphic principles;
and, (3) steady state hydraulic and sediment transport analysis.

Works Cited

Arizona Department of Water Resources, Flood Warning and Dam Safety Section, "Requirement for Floodplain Delineation in Riverine Environments - State Standard 2-96", July 1996.

Arizona Department of Water Resources, "Design Manual for Engineering Analysis of Fluvial Systems", March 1985.

City of Tucson Department of Transportation, Engineering Division. "Standards Manual for Drainage Design and Floodplain Management in Tucson, Arizona. December 1989.

Thomas, B.E., H.W. Hjalmarson, and S.D. Waltemeyer. "Methods for Estimating Magnitude and Frequency of Floods in the Southwestern United States." USGS Open-File Report. 1994. 93-419

Example Application

Example 1: Development Adjacent to a Watercourse

- **Problem Statement.** Single lot development proposed on 1-acre parcel bordered on one side by a small, earthen channel. The contributing watershed upstream of the site is 700 acres in area.

- **Objective.** Determine setback allowance from top of channel bank.

Level I Analysis

A 100-year peak discharge value of 530 cfs was determined from local hydrology methodology. The width of the 100-year floodplain in the site vicinity is 35 feet. The site is adjacent to the outside of a mild bend (i.e., radius of curvature greater than 5 times topwidth) in the channel.

Calculations:

A = 700 acres x (1 sq. mile/640 acres) = 1.09 sq. miles < 30 square miles

setback = $1.0\ (530)^{0.5}$ = 23 feet

Since the calculated setback is greater than the minimum recommended setback of 20 feet, use a 23 foot setback. The setback is measured from the top of the channel bank or the 100-year floodway limit, whichever is greater.

Level II Analysis

The developer would like to minimize the setback as much as possible without having to provide bank lining. Accordingly, the site specific hydraulic and grain size information is collected to check if erosion of the channel would be naturally limited. Local geometry for the channel is obtained using site measurements:

Bottom Width = 15 feet
Side Slope = 2 horizontal to 1 vertical
Channel Slope = Energy Slope = 0.01 feet/foot
Radius of curvature = 500 feet

The Manning n value for the channel is estimated at 0.030.

Using normal depth procedures, the hydraulic characteristics of the local channel under 100-year flood conditions are determined:

Flow Depth = 3.0 feet
Flow Velocity = 8.4 feet/second

Results of a sieve analysis of a local channel material sample yields the following information:

D_{75} = 4 mm = 0.013 ft = 0.16 inches
D_{65} = 1.2 mm = 0.0039 ft = 0.05 inches
D_{50} = 0.6 mm = 0.002 ft = 0.024 inches

Calculations:

(1) Allowable velocity approach, assuming sediment laden flow

Entering Figure 1 with D_{75} = 4 mm yields a basic velocity of 4.0 ft/sec.
Entering Figure 2 with r/w = 18.5 yields C_a = 1.0
Entering Figure 3 with Z = 2 yields C_b = 0.72
Entering Figure 4 with Depth = 3.0 feet yields C_c = 1.0

Maximum allowable velocity = (4.0)(1.0)(0.72)(1.0) = 2.9 ft/sec

Since the computed velocity of 8.4 ft/sec exceeds the maximum allowable velocity, erosion may be expected to occur.

(2) Tractive stress approach

Since D_{75} is less than 0.25 inches, the reference tractive stress method is used:

Assuming a water temperature of 60° F, the kinematic viscosity (ν) = 0.0000121 ft²/sec, and the density (ρ) = 1.94 slugs/ft³

Compute $V^3/(g\nu S_e) = 1.52 \times 10^8$

Compute $V/[(gD_{65}S_e)^{1/2}] = 237$

From Figure 9, $V/(\tau/\rho)^{1/2} = 19.0$

Solving the above equation yields $\tau = 0.38$ lb/ft².

From Figure 11, with bottom width over flow depth (b/Y) = 15/3 = 5, $\tau_s = (1.03)\tau = 0.39$ lb/ft^2.

From Figure 6, with radius of curvature over bottom width (r/b) = 500/15 = 33, $\tau_{sc} = 1.0\ \tau_s = 0.39$ lb/ft^2.
[Note that radius of curvature over bottom width is used in this procedure while radius of curvature over top width of flow is used in the allowable velocity approach.]

From Figure 12, Curve 1 (for high sediment content), the allowable tractive force is 0.083 lb/ft^2. Since 0.083 is less than 0.39, the channel is erosive.

(3) Tractive power approach

An unconfined compressive strength (UCS) test of the saturated embankment soils is performed, yielding a strength of 1000 lb/ft3.

Assuming half of this strength for design purposes, UCS_{design} = 500 lb/ft^3.

Compute tractive power = $V\tau_{sc} = 3.3$

From Figure 13, the condition falls above the S-Line, indicating that the channel is erosive.

All three approaches indicate that the channel is erosive. Therefore, the 23 foot setback allowance determined by Level I procedures can not be reduced unless the channel banks are armored or the channel is obviously in bedrock.

Level III Analysis

The conclusions derived from the Level II analysis and the small size of the development indicate that the Level III analysis would probably not be applied in this case. However, should the developer wish to proceed with the setback allowance investigation, a registered engineer with experience in sediment transport modeling could be employed for this purpose. The engineer would be expected to collect available historic information, document the historic planform changes to the watercourse under events of varying frequency, apply steady state hydraulic and sediment transport calculation procedures to determine the erosion/sedimentation characteristics of the local reach of channel, and, potentially apply a moveable boundary river simulation model to quantify the changes likely along the study reach under design event conditions.

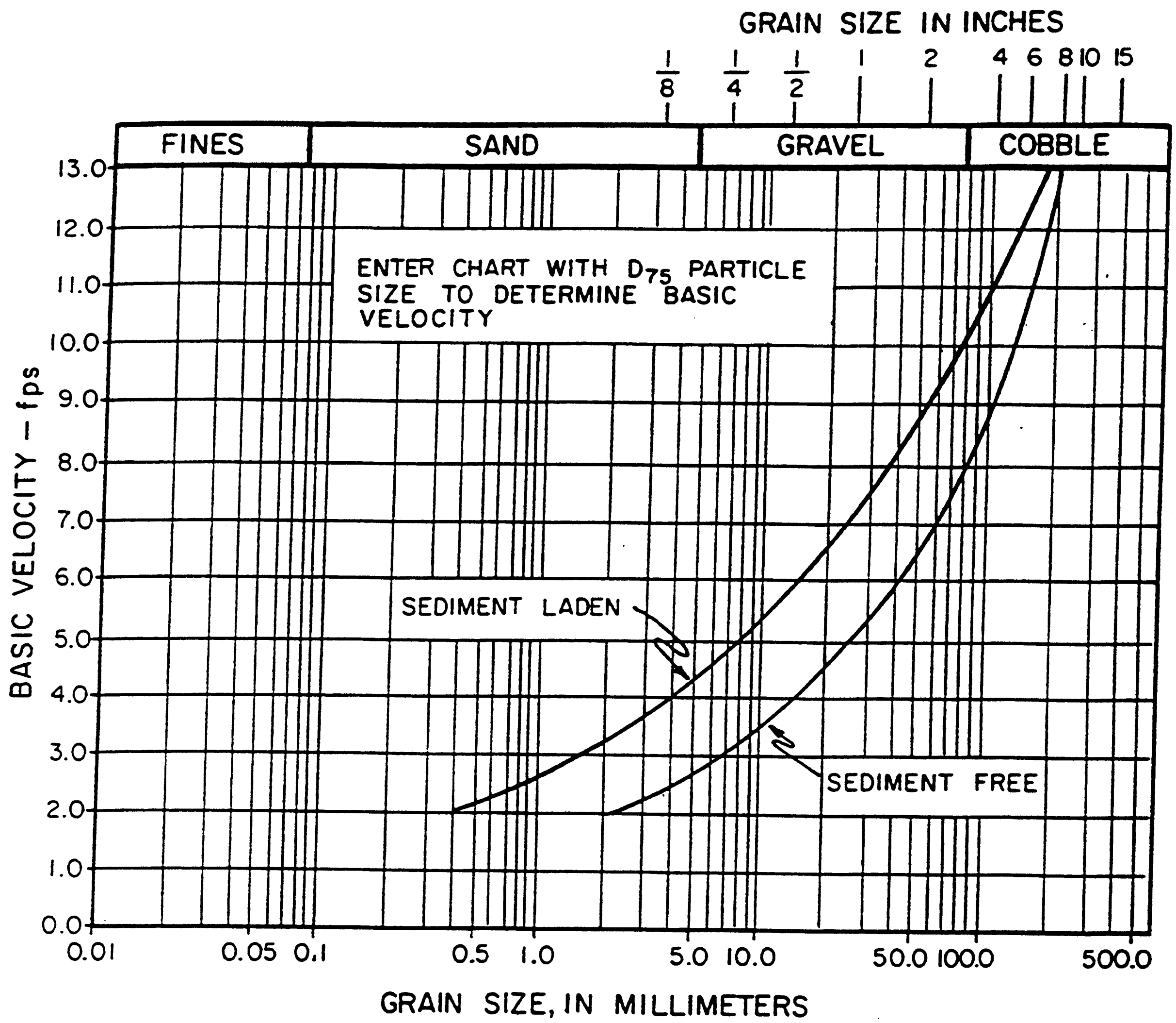


FIGURE 1

BASIC ALLOWABLE VELOCITY FOR EARTHEN CHANNELS

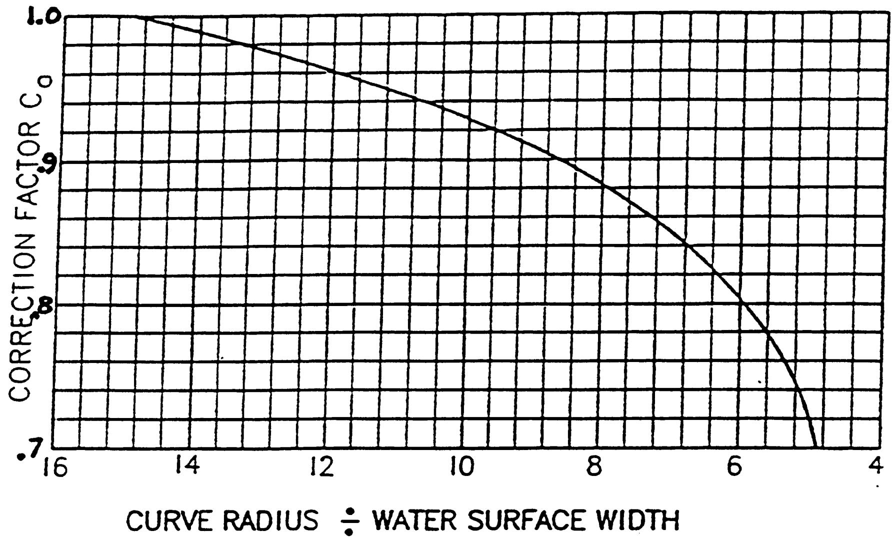


FIGURE 2

CORRECTION FACTOR C_a FOR CHANNEL ALIGNMENT

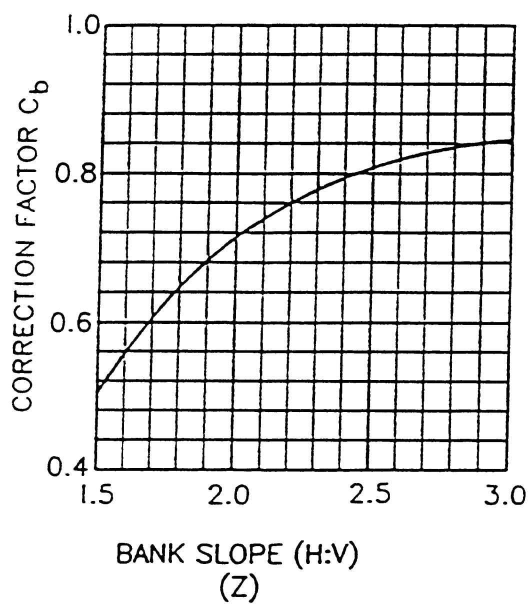


FIGURE 3

CORRECTION FACTOR C_b FOR BANK SLOPE

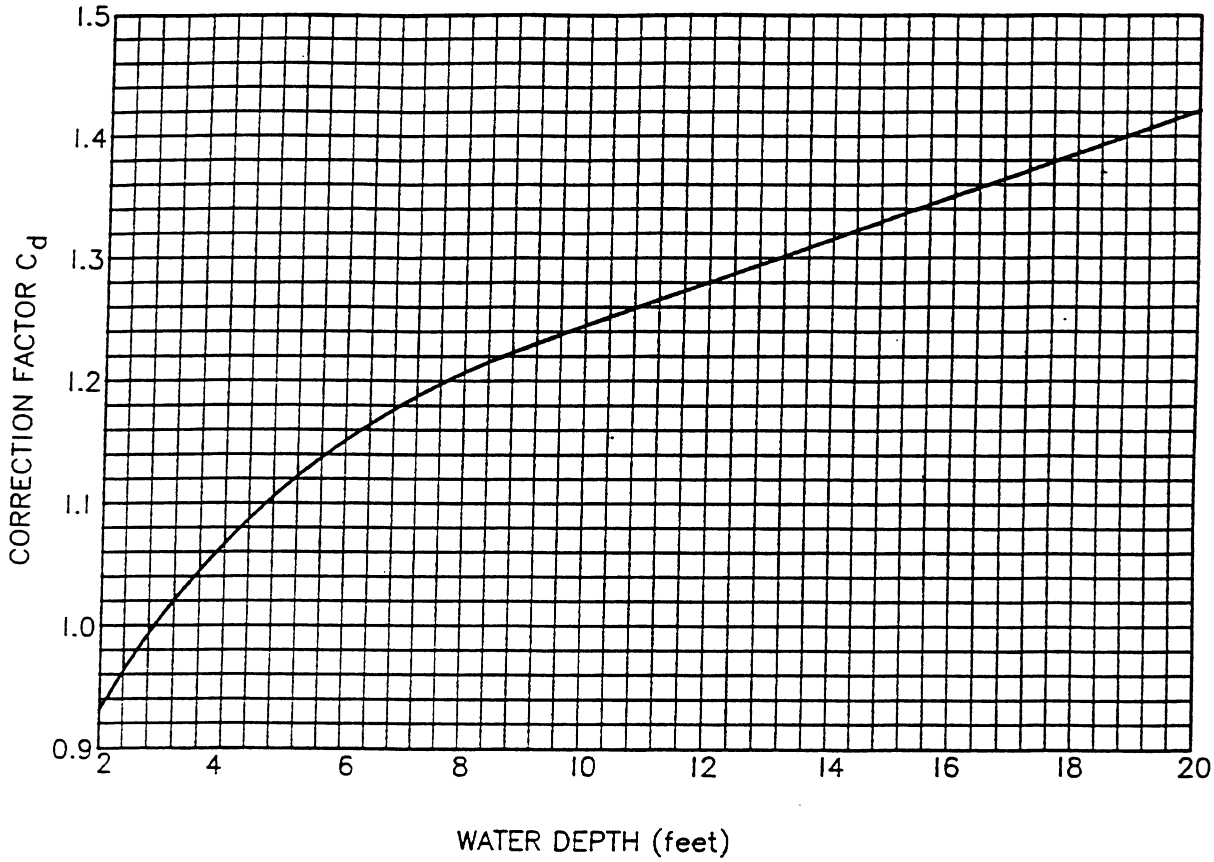


FIGURE 4

CORRECTION FACTOR C_d FOR DEPTH OF FLOW

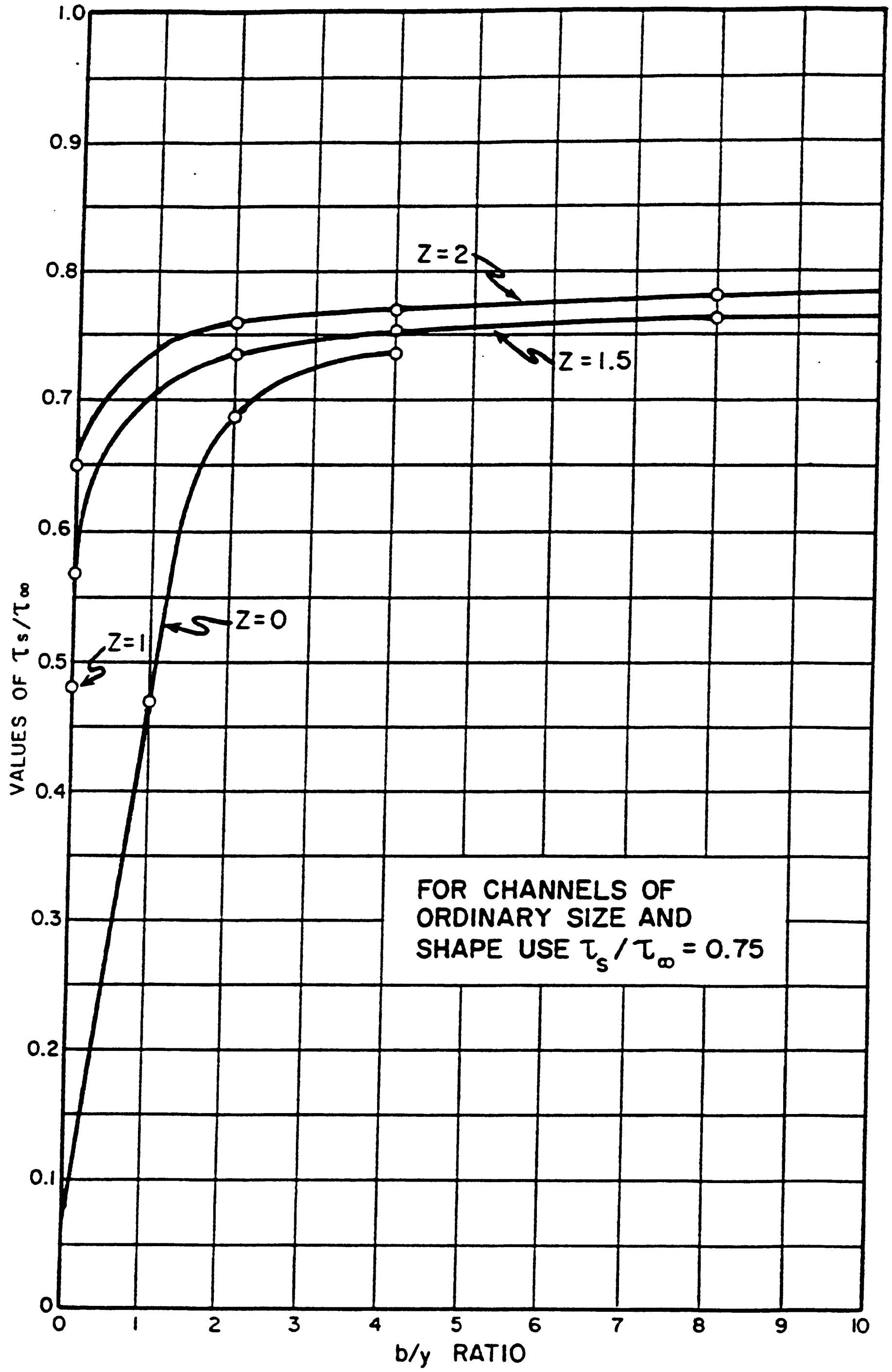


FIGURE 5

ACTUAL MAXIMUM TRACTIVE STRESS, τ_s, ON SIDES OF STRAIGHT TRAPEZOIDAL CHANNELS

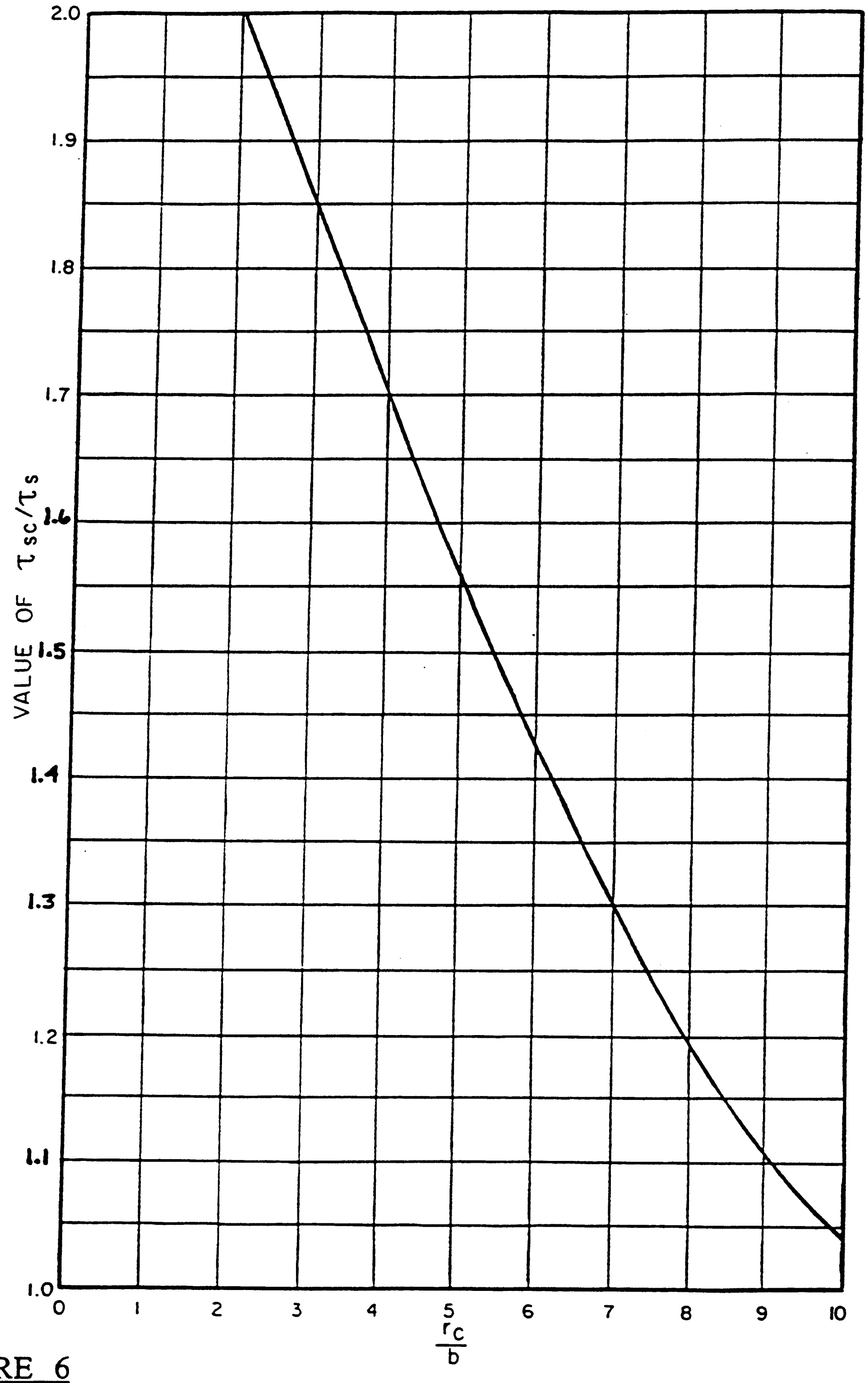


FIGURE 6

ACTUAL MAXIMUM TRACTIVE STRESS, τ_{sc}, ON SIDES OF TRAPEZOIDAL CHANNELS WITHIN A CURVED REACH

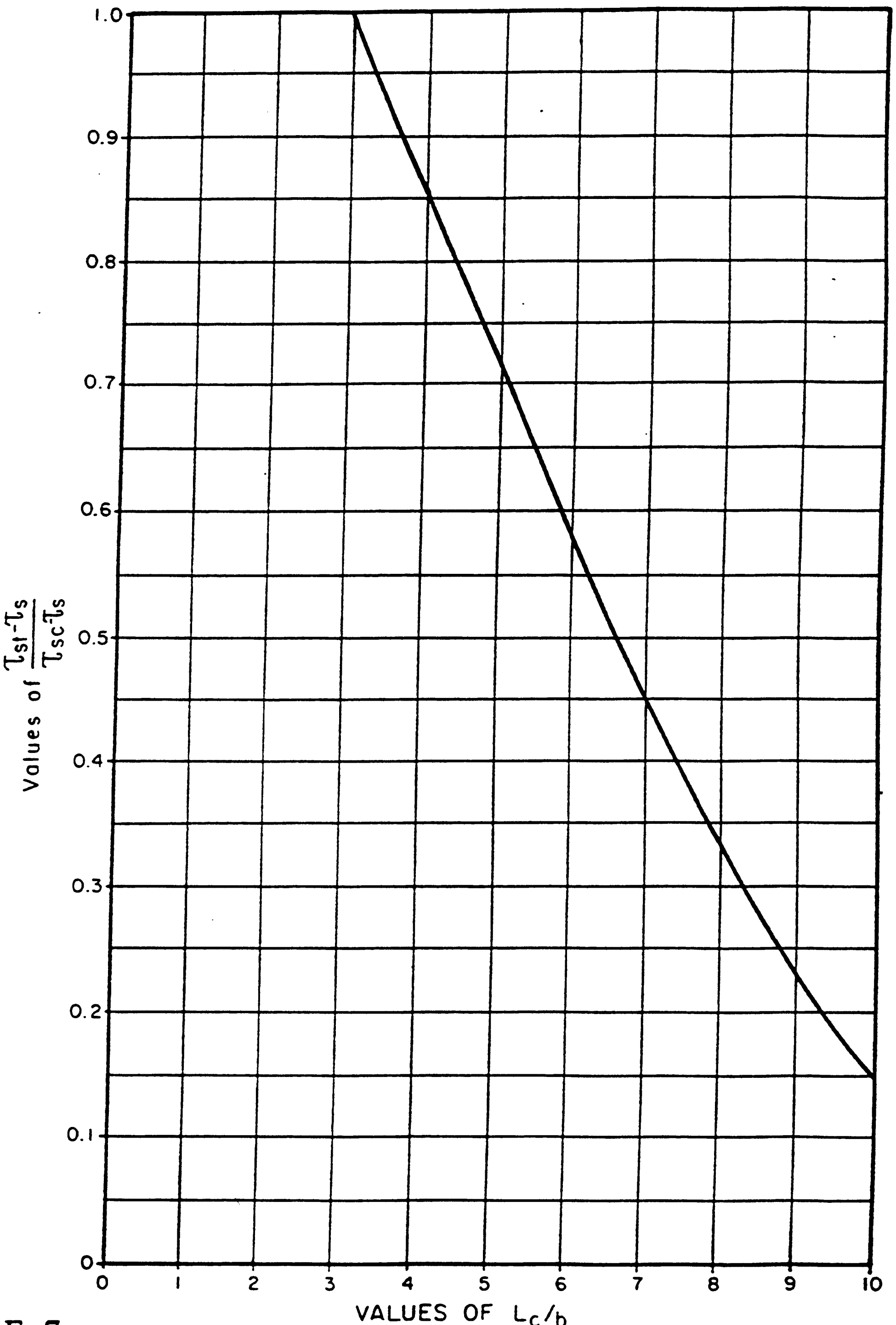


FIGURE 7

ACTUAL MAXIMUM TRACTIVE STRESS, τ_{st}, ON SIDES OF TRAPEZOIDAL CHANNELS IN STRAIGHT REACHES IMMEDIATELY DOWNSTREAM FROM CURVED REACHES

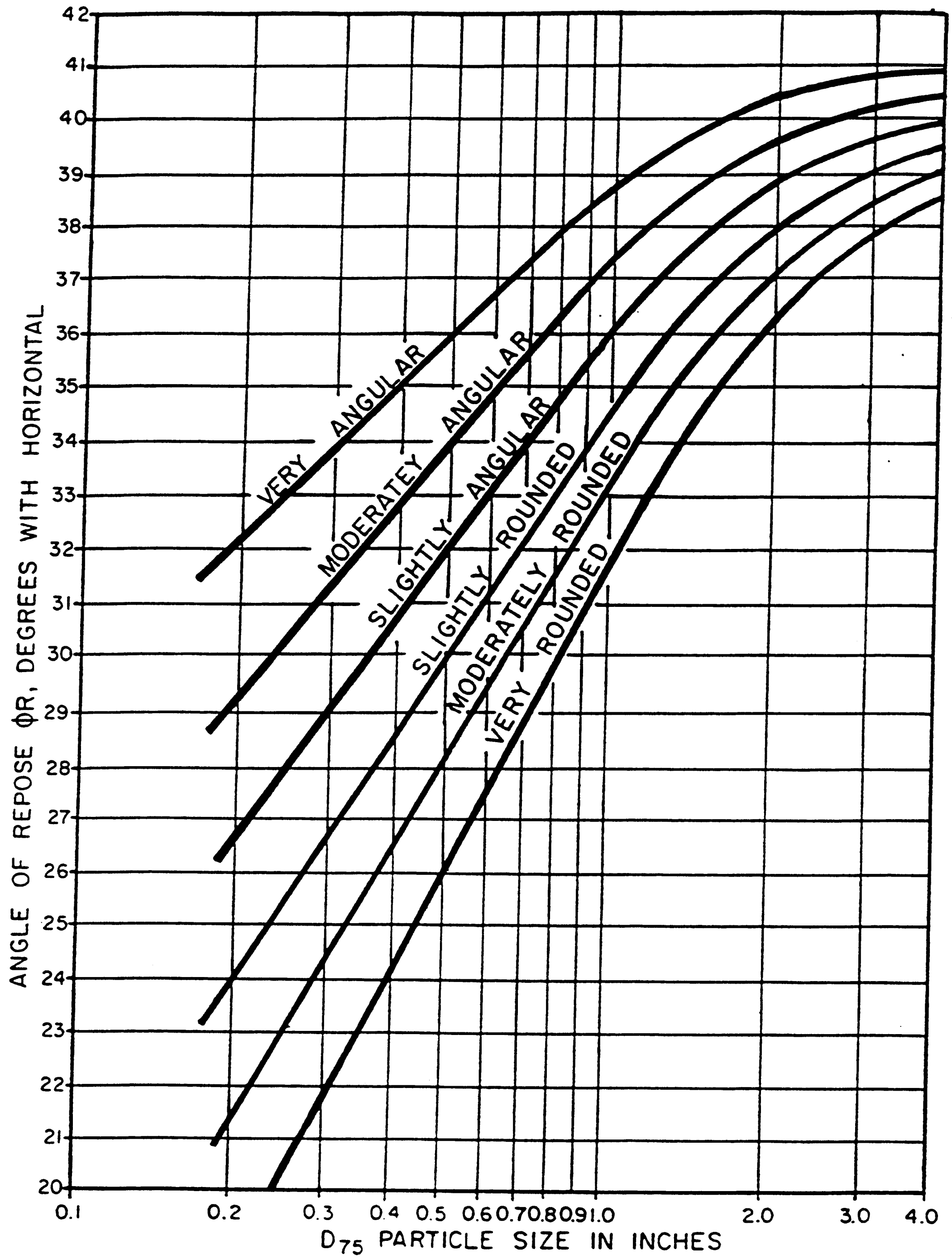


FIGURE 8

ANGLE OF REPOSE, ΦR, FOR NON-COHESIVE MATERIALS

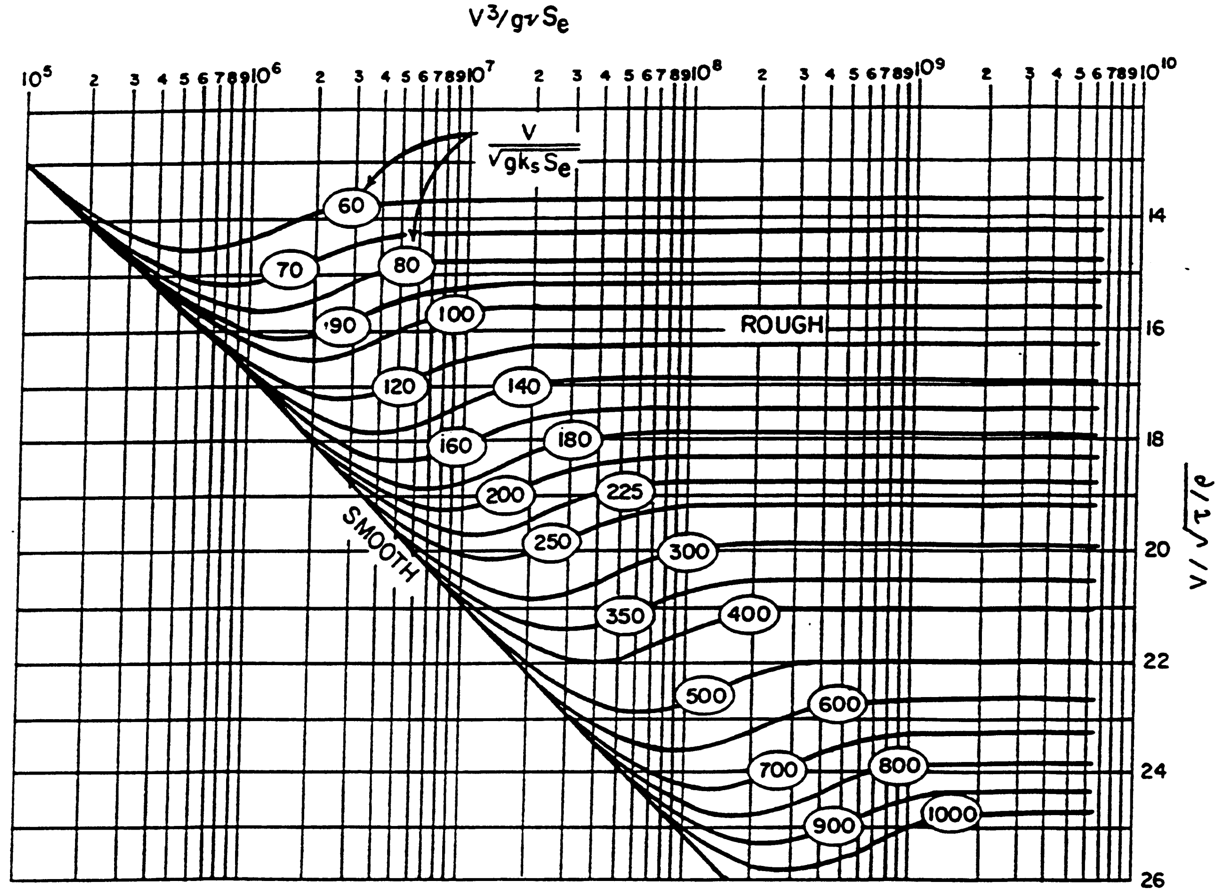


FIGURE 9

GRAPHIC SOLUTION OF REFERENCE TRACTIVE STRESS

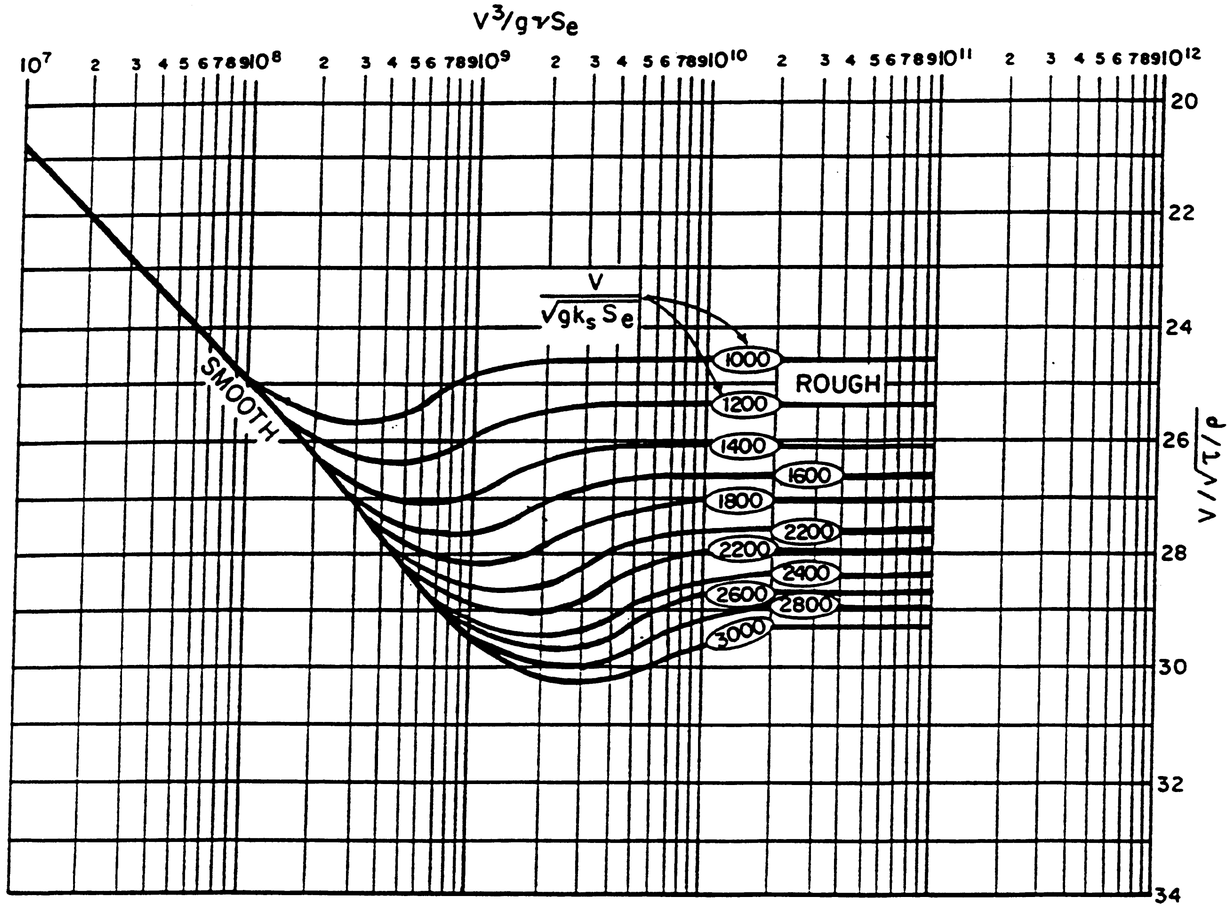


FIGURE 10

GRAPHIC SOLUTION OF REFERENCE TRACTIVE STRESS (CONTINUED)

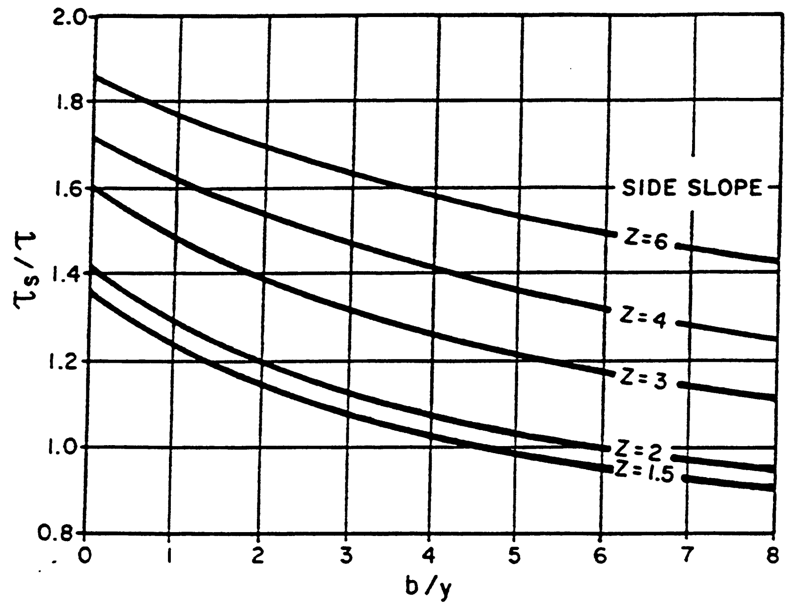


FIGURE 11

APPLIED MAXIMUM TRACTIVE STRESSES, τ_s, ON SIDES OF STRAIGHT TRAPEZOIDAL CHANNELS

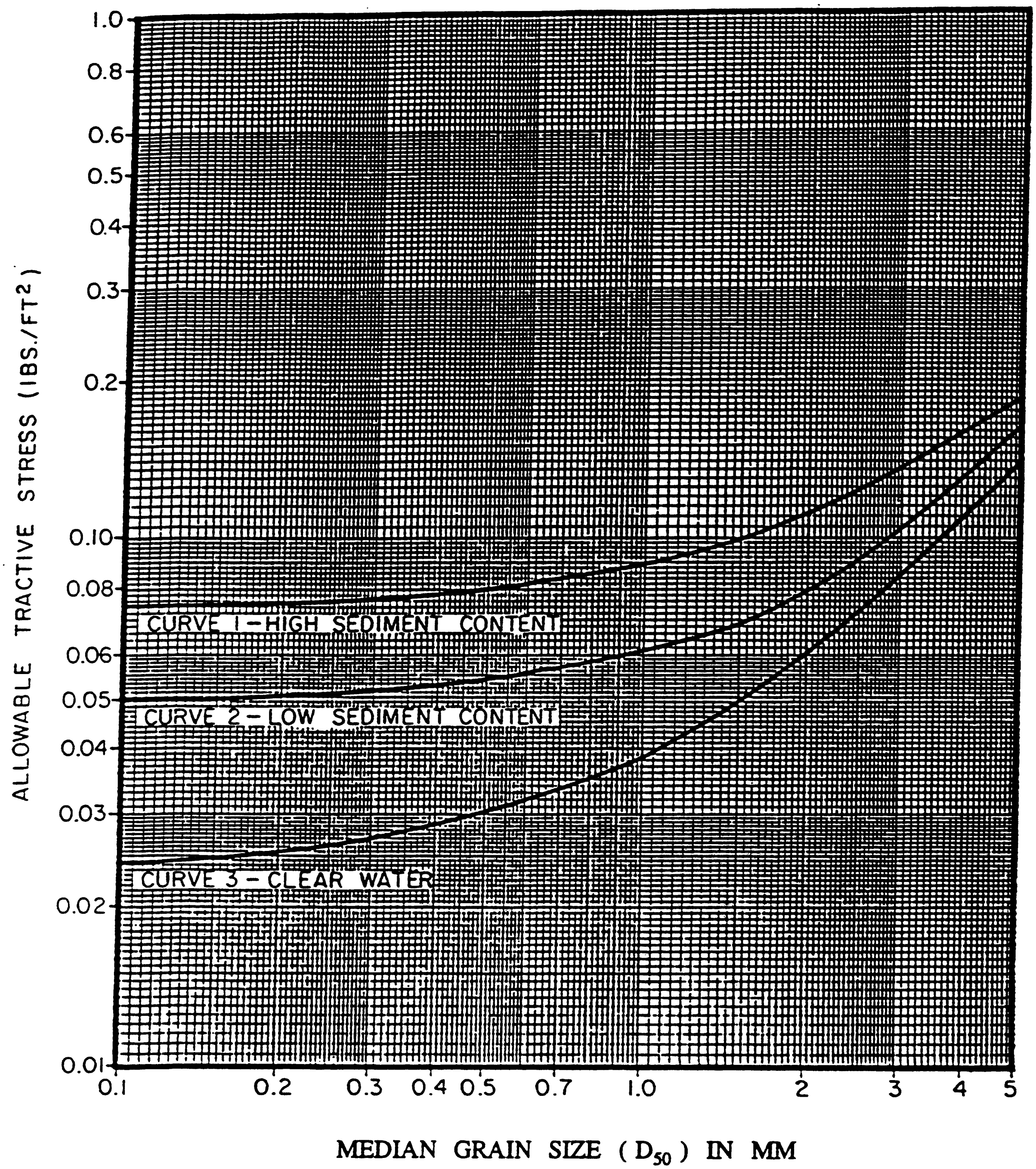


FIGURE 12

MAXIMUM ALLOWABLE TRACTIVE STRESS FOR NON-COHESIVE SOILS, $D_{75} < 0.25"$

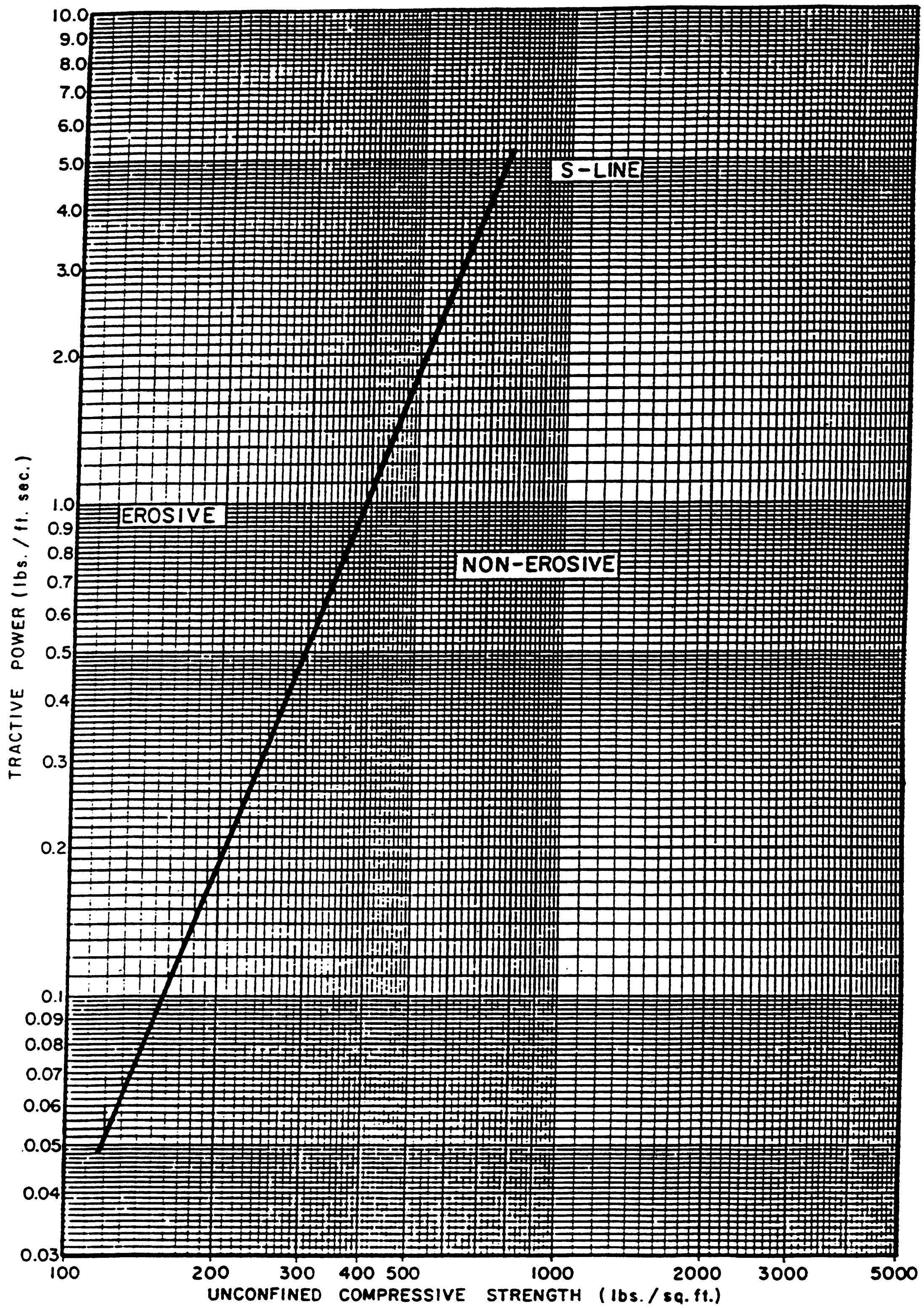


FIGURE 13

UNCONFINED COMPRESSIVE STRENGTH AND TRACTIVE POWER AS RELATED TO CHANNEL STABILITY

GUIDELINE 2

Channel Degradation Estimation for Alluvial Channels in Arizona

TABLE OF CONTENTS

Introduction

Channel degradation occurs within watercourses composed of erodible material, where local or general differentials in sediment transport capacity exist. Numerous factors control the short and long term degradation potential of channel reaches, including the size and cohesiveness of the material of which the channel is composed, the vegetation type and density in the channel, the hydraulic characteristics generated within the channel under flood events, and the existence of flow redirection or concentration structures within the channel. A key factor, however, is the amount of variation in channel properties from reach to reach. A channel reach attempts to adjust to conditions imposed on it by factors occurring up- and downstream; thus, the more uniform the channel is along the system under study, the less the potential exists for channel degradation to be a significant factor. Natural and man-made discontinuities along the system can create local increases in sediment transport potential, which often result in local degradation of the channel. System-wide disturbances, such as those associated with urbanization of the watershed or dam construction, have more far reaching impact, as the entire channel is forced to adjust to a change in sediment supply.

This document presents procedures that may be used for estimation of channel degradation in unlined watercourses within Arizona. Three levels of procedures are provided, with data requirements, procedural complexity, and accuracy of results all increasing as the analysis level is incremented. The Level I approach provides an initial estimate of local channel degradation potential for generally stable, natural channel conditions. The resulting initial estimate may be reduced through use of the more rigorous Level II methodologies. Level III procedures are outlined for situations that warrant more detailed channel degradation determination.

Procedure

General

Three levels of procedures for estimation of channel degradation depth are described in the following paragraphs. The first level of analysis provides an initial estimate of the potential scour depth to consider for design of structures to be placed near a streambed or along the banks of a channel. This first level of analysis is recommended only for channel reaches that are expected to be in general balance with the surrounding system -- i.e. no major disturbances (dams, bridges, encroachments, etc..) are evident in the site vicinity -- and where the desire is to establish a "safe" scour depth to allow for the concentration of flows that can naturally occur within channels composed of erodible material. The Level II procedures provided are methods for demonstrating the site specific limits to erosion potential, involving computations which require local hydraulic information and sediment size distributions, or historical evidence of channel performance. The third level of procedures outlined will provide more definitive determination of channel stability in the reaches under study. This level of analysis is recommended in areas where local flow characteristics are complex, where the channel has been redirected or otherwise modified by acts of man, or where the safety of local paralleling or crossing structures is of high concern.

Level I

This level of analysis requires the following information :

> Peak discharge associated with the 100-year flood (Q_{100}). May be estimated using simplified methodologies such as ADWR State Standard #2 (SS 2-96), USGS regression equations, or other appropriate local or more detailed methods.

The total scour depth, d_s, is the combination of general degradation and long term degradation and can be computed as follows:

$$d_s = d_{gs} + d_{lts}$$

where:

d_s = Total scour depth, in feet
d_{gs} = General degradation, in feet
d_{lts} = Long term degradation, in feet

General degradation can be computed as follows:

$$d_{gs} = 0.157(Q_{100})^{0.4} \quad \text{for straight channel reaches.}$$

and

$$d_{gs} = 0.219(Q_{100})^{0.4} \quad \text{for channel reaches with curvature.}$$

The second equation will give the worst-case scour for channel curvature. and is not recommended unless significant curvature is evident along the channel reach.

Long term degradation can be computed as follows:

$$d_{lts} = 0.02(Q_{100})^{0.6}$$

This equation for long term degradation should only be used when no downstream controls exist within the channel system.

The total scour depth, d_s, should be applied to the lowest point in the local cross section for determination of the elevation to which scour will occur.

For Level I, the minimum total scour depth, d_s, shall be 3 feet.

Level II

The Level II approaches presented below may be used to demonstrate the ability of the existing channel system to resist degradation, and to justify a lesser burial requirement than that computed using the Level I equations.

Erodibility evaluation

Three procedures for determination of the erodibility of local channel material under computed hydraulic conditions are presented in the ADWR's State Standard for Lateral Migration Setback Allowance for Riverine Floodplains in Arizona. These procedures are: (1) the allowable velocity approach; (2) the tractive stress approach; and, (3) the tractive power approach. One or more of these procedures can be used to demonstrate the adequacy of the material of which the channel is composed to resist the erosive action of the flow under 100 year flow conditions.

Armoring potential evaluation.

An evaluation of relative channel stability can be made by evaluating incipient motion parameters and determining armoring potential. The definition of incipient motion is based on the critical or threshold condition where hydrodynamic forces acting on a grain of sediment have reached a value that, if increased even slightly, will move the grain. Under critical conditions, or at the point of incipient motion, the hydrodynamic forces acting on the grain are just balanced by the resisting forces of the particle. For given hydrodynamic forces, or equivalently for a given discharge, incipient motion conditions will exist for a single particle size. Particles smaller than this will be transported downstream and particles equal to or larger than this will remain in place.

The Shields diagram (Figure 1) may be used to evaluate the particle size at incipient motion for a given discharge. The Shields diagram was developed through measurements of bed-load transport for various values of the Shields parameter (y axis of Figure 1) at least twice as large as the critical value, and extrapolated to the point of vanishing bed load. In the turbulent range, where most flows of practical engineering interest occur, this diagram suggests that the Shields parameter is independent of flow conditions and the following relationship is established:

$$D_c = \tau_p / [0.047 (\gamma_s - \gamma)]$$

where D_c is the diameter of the sediment particle for conditions of incipient motion, τ_p is the boundary shear stress acting on the particle, γ_s and γ are the specific weights of sediment and water, respectively, and 0.047 is a dimensionless coefficient. Any consistent set of units may be used with this equation. Typical values for γ_s and γ in English units are 165 lb/ft^3 and 62.4 lb/ft^3, respectively.

For computation of shear stress on the boundary particles, the following relations are recommended:

$$\tau_p = \tfrac{1}{8} f \rho V^2$$

$$f = 116.5\, n^2 / R^{1/3}$$

$$n = D_{90}^{1/6} / 26$$

where f = friction factor (dimensionless)
ρ = density of the water
V = flow velocity
n = Manning resistance value
R = hydraulic radius of the channel
D_{90} = particle size which is larger than 90 percent of all sizes

The units of the above are as follows: τ is in lb/ft^2; ρ is in $slugs/ft^3$ (typically 1.94 $slugs/ft^3$); V is in feet per second; and R is in feet. The relation presented above relating the Manning n value to the D_{90} of the local bed material yields the resistance factor associated with the particle roughness only, and assumes D_{90} is in meters.

The shear stress computed from the above equation should be increased in areas of channel curvature using Figure 2.

The armoring process begins as the non-moving coarser particles segregate from the finer material in transport. The coarser particles are gradually worked down into the bed, where they accumulate in a sublayer. Fine bed material is leached up through this coarse sublayer to augment the material in transport. As movement continues and degradation progresses, and increasing number of non-moving particles accumulate in the sublayer. This accumulation interferes with the leaching of fine material so that the rate of transport over the sublayer is not maintained at its former intensity. Eventually, enough coarse particles accumulate to shield, or "armor," the entire bed surface. When fines can no longer be leached from the underlying bed, degradation is arrested.

Potential for development of an armor layer can be assessed using Shields' criteria for incipient motion and a representative bed-material composition. In this case a representative bed material composition is that which is typical of the depth of anticipated degradation. Using the equation presented above, the incipient-motion particle size can be computed for a given set of hydraulic conditions. If no sediment of the computed size or larger is present in significant quantities in the bed, armoring will not occur. Armoring is probable when the particle size computed from the above equation is equal to or smaller than the D_{90} size.

After determination of the percentage of the bed material equal to or larger than the armor particle size (D_c), the depth of scour necessary to establish an armor layer (ΔZ_a) can be calculated from the following equation:

$$\Delta Z_a = y_a \, [(1/P_c) - 1]$$

where y_a is the thickness of the armoring layer and P_c is the decimal fraction of material coarser than the armoring size. The thickness of the armoring layer (y_a) ranges from one to three times the armor particle size (D_c), depending on the value of D_c. Field observations suggest that a relatively stable armoring conditions requires a minimum of two layers of armoring particles.

Channel profile history comparison

This procedure, applicable where sufficient data is available, relies on the historical record for indication of the degradation potential of the local channel reach. This procedure should be used to demonstrate the stable or aggrading tendency of the reach in question, rather than to estimate potential degradation depths. Given a reach of channel with successive record of channel profile changes, associated with hydrologic information for the events occurring between surveys, the reviewer can determine the trend of the channel changes and assess the likelihood of trend continuation for the future. Where the stable or aggradational trend is obvious, and no changes are anticipated in the channel system to alter the on-going trend, a lesser degradation allowance than that provided under the Level I guidelines would be reasonable.

Grade stabilization measures adequacy analysis

Grade stabilization measures of some form may be proposed or already in place which may act to limit the degradation potential of the watercourse of concern. In some areas within Arizona, procedures are in place for assessment of the adequacy of channel stabilization measures. For areas without standardized procedures, two references are recommended which detail evaluation procedures:

> <u>Design Manual for Engineering Analysis of Fluvial Systems</u>, Arizona Department of Water Resources, 1985.
>
> <u>Standards Manual for Drainage Design and Floodplain Management in Tucson, Arizona</u>, City of Tucson Department of Transportation, Engineering Division, 1989.

Level III

This level of analysis involves modeling the hydraulic and sediment transport characteristics of the local watercourse in order to simulate the erosion/sedimentation and channel deformation processes which are expected to occur in the area of concern. For this level of analysis, Level III hydrology shall be performed to generate required hydrographs. Level III analyses should be performed by persons with knowledge and experience in the fields of sediment transport and river geomorphology. It is recommended that any movable boundary river modeling used for establishment of degradation potential be the culmination of a thorough analysis consisting of:

(1) evaluation of historical trends;
(2) qualitative analysis based on field evaluation and application of geomorphic principles;
and, (3) steady state hydraulic and sediment transport analysis.

Works Cited

Arizona Department of Water Resources, Flood Warning and Dam Safety Section, "Requirement for Floodplain Delineation in Riverine Environments - State Standard 2-96", July 1996.

Arizona Department of Water Resources, Flood Warning and Dam Safety Section, "Lateral Migration Setback Allowance for Riverine Floodplains in Arizona - State Standard 5-96", September 1996.

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Blench, T., "Mobile-Bed Fluviology", Edmonton: University of Alberta Press, 1969.

City of Tucson Department of Transportation, Engineering Division. "Standards Manual for Drainage Design and Floodplain Management in Tucson, Arizona". December 1989.

Lacey, G., "Stable Channels in Alluvium," in Proceedings of the Institution of Civil Engineers, Vol. 229, 1930.

Thomas, B.E., H.W. Hjalmarson, and S.D. Waltemeyer. "Methods for Estimating Magnitude and Frequency of Floods in the Southwestern United States." USGS Open-File Report. 1994. 93-419

Example Application

Example 1: Proposed Siphon Crossing of an Earthen Channel

- **Problem Statement.** A natural earthen channel traverses a site where an irrigation channel is being constructed. The watershed contributing to the earthen channel upstream of the site is 700 acres in area. A siphon is proposed to convey irrigation water across the channel.

- **Objective.** Determine the burial depth for the proposed siphon.

Level I Analysis

A 100-year peak discharge value of 530 cfs was determined from local hydrology methodology. The channel in the site vicinity has 2:1 side slopes and a bottom width of 15 feet. The proposed crossing site is at a mild bend in the channel. A sieve analysis of the local bed material yields a median grain size D_{50} = 1.0 mm = 0.0033 feet.

Calculations:

General degradation, $d_{gs} = 0.157(530)^{0.4} = 1.93$ feet

Long term degradation, $d_{lts} = 0.02(530)^{0.6} = 0.86$ feet

Total scour, d_s = 1.93 feet + 0.86 feet = 2.79 feet

Since the total scour calculated is less than the recommended minimum of 3 feet, use a total scour depth of 3.0 feet.

Level II Analysis

Further evaluation is desired to investigate the potential for reducing the burial depth indicated through application of the Level I procedure. Although no historical data is available for determination of the local aggradation/degradation trends of the earthen channel, the erodibility and armoring potential of the existing channel material can be checked using the recommended Level II procedures. The site specific hydraulic and grain size information is collected to check if erosion of the channel would be naturally limited. The channel slope in the site vicinity is estimated from USGS quadrangle maps at 0.010 feet/foot, and the Manning n value for total channel resistance is estimated at 0.030.

Using normal depth procedures, the hydraulic characteristics of the local channel under 100-year flood conditions are determined:

Flow Depth = 3.0 feet
Flow Velocity = 8.4 feet/second

The sieve analysis of the local channel material sample yields the following information:

D_{90} = 55 mm = 0.180 ft = 0.217 inches
D_{75} = 4 mm = 0.013 ft = 0.16 inches
D_{65} = 1.9 mm = 0.0062 ft = 0.07 inches

Calculations:

Erodibility Evaluation (using procedures and figures provided in Attachment 1 to this State Standard)

(1) Allowable velocity approach, assuming sediment laden flow

Entering Figure 1 with D_{75} = 4 mm yields a basic velocity of 4.0 ft/sec.

In this case, we are concerned with erosion of the channel invert in a reach containing only a mild bend, so the correction factors for channel curvature reduces to 1.0. The correction factor for side slope, which must be considered for evaluating the erodibility of the channel banks, is not applied in this case.

Entering Figure 4 with Depth = 3.0 feet yields C_c = 1.01

Maximum allowable velocity = (4.0)(1.0)(1.01) = 4.0 ft/sec

Since the computed velocity of 8.4 ft/sec exceeds the maximum allowable velocity, erosion may be expected to occur.

(2) Tractive stress approach

Since D_{75} is less than 0.25 inches, the reference tractive stress method is used;

Assuming a water temperature of 60° F, the kinematic viscosity (ν) = 0.0000121 ft^2/sec, and the density (ρ) = 1.94 slugs/ft^3

Compute $V^3/(g\nu S_e) = 1.52 \times 10^8$

Compute $V/(gD_{65}S_e)^{1/2} = 188$

From Figure 9, $V/(\tau/\rho)^{1/2} = 18.2$

Solving the above equation yields $\tau = 0.41$ lb/ft^2.

No correction factor for side slope is applied, and the correction factor for channel curvature reduces to 1.0 for a mild bend.

From Figure 12, Curve 1 (for high sediment content), the allowable tractive force is 0.09 lb/ft^2. Since 0.09 is less than 0.41, the channel is erosive.

(3) Tractive power approach

An unconfined compressive strength (UCS) test of the saturated channel soils is performed, yielding a strength of 800 lb/ft^3.

Assuming half of this strength for design purposes, UCS_{design} = 400 lb/ft^3.

Compute tractive power = $V\tau_s = 3.44$

From Figure 13, the condition falls above the S-Line, indicating that the channel is erosive.

Armoring potential evaluation

Manning's n related to particle roughness = $[55/1000]^{1/6} / 26 = 0.024$

Channel flow area = $[15+2(3.0)](3.0) = 63.0$ square feet

Channel wetted perimeter = $15 + 2(3.0)(5)^{1/2} = 28.4$ feet

Hydraulic Radius = $63.0/28.4 = 2.22$ feet

Friction factor = $f = 116.5\,(0.024)^2 / (2.22)^{1/3} = 0.051$

Particle shear stress = $\tau_p = \frac{1}{8}(0.051)(1.94)(8.4)^2 = 0.87$ lb/ft^2

Critical particle size = $D_c = .87/[0.047(165-62.4)] = 0.18$ feet = 54.9 mm

Since the critical particle size is essentially equal to D_{50}, armoring is a possibility.

Therefore, the percent of material greater than $D_c = 54.9$ mm is 10%

Armor thickness $= y_a = 2D_c = 0.36$ feet

Depth of degradation required for armoring to form:

$$\Delta Z_a = y_a\,[(1/P_c) - 1] = 0.36[(1/0.10) -1] = 3.24 \text{ feet}$$

Since the depth required for armoring to occur exceeds the Level I burial depth, armoring will not control, and the recommended burial depth is the minimum allowable value of 3.0 feet.

Level III Analysis

The conclusions derived from the Level II analysis and the nature of the problem indicate that the Level III analysis would probably not be applied in this case. However, should the designer wish to proceed with the degradation investigation, a registered engineer with experience in sediment transport modeling could be employed for this purpose. The engineer would be expected to collect available historic information, document the historic planform changes to the watercourse under events of varying frequency, apply steady state hydraulic and sediment transport calculation procedures to determine the erosion/sedimentation characteristics of the local reach of channel, and, potentially apply a moveable boundary river simulation model to quantify the changes likely along the study reach under design event conditions.

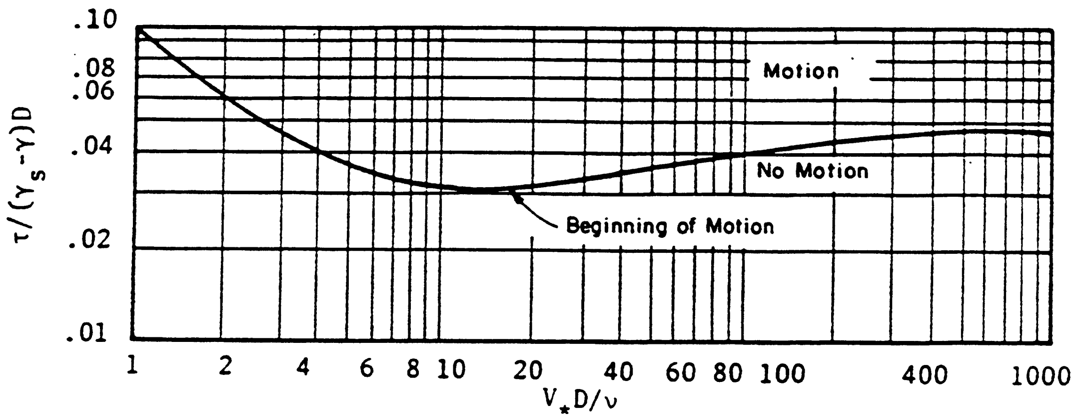


FIGURE 1

SHIELD'S RELATION FOR BEGINNING OF MOTION

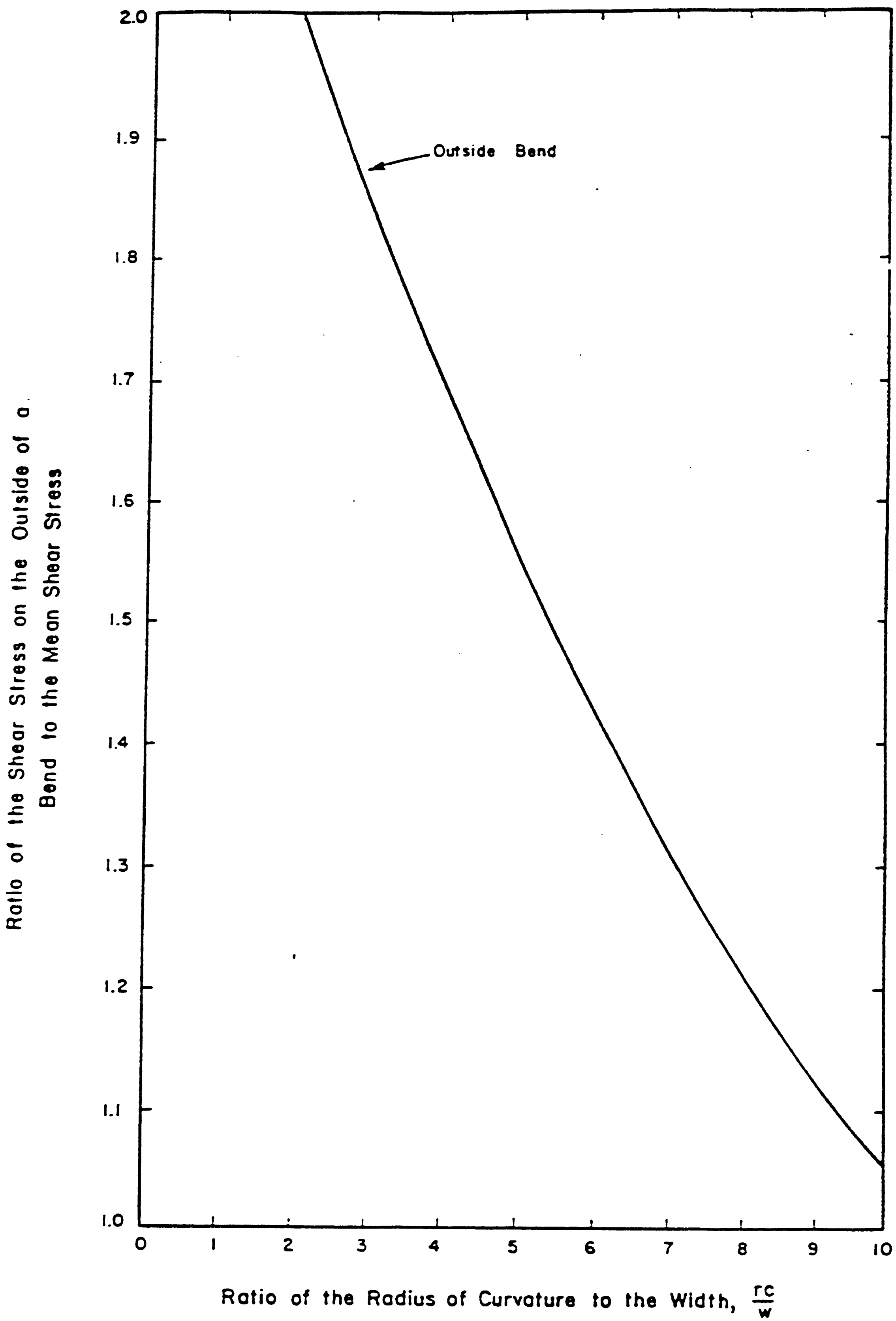


FIGURE 2
EFFECT OF BEND ON BOUNDARY SHEAR STRESS

GUIDELINE 3

Evaluation of River Stability Impacts
associated with
Sand and Gravel Mining

TABLE OF CONTENTS

Introduction

The river stability impacts associated with instream or near-stream sand and gravel operations depend on the local watershed and river characteristics, and on the mining and management practices followed. Excessive sand and gravel removal from a river channel can endanger the stability of the river system by inducing general scour and headcutting. These processes can undermine the burial and/or support materials for facilities that cross or parallel the watercourse, increasing the likelihood of structure failure. These processes can also increase the rate of erosion of a dike or buffer zone designed to separate a near-river pit from an active river channel. A headcut and erosion through such a buffer zone could alter local river channel characteristics and transport rates, and impact both upstream and downstream reaches. If the channel reach adjacent to a floodplain mining pit is geomorphically active (e.g., migrating laterally), the same result might occur if protective measures or an adequate buffer zone are not provided during site development.

The scour and deposition problems associated with sand and gravel mining are very complicated. The dominant physical processes include water runoff, sediment transport, sediment routing, degradation, aggradation, and breaking and forming of the armor layer. These processes are unsteady and complicated in nature. Furthermore, each situation is unique and requires independent analysis. No standard equation or formula can be adopted which is universally applicable to all gravel mining evaluations. However, general guidelines have been developed for determination of the adequacy of buffer areas between proposed mining operations and active river channels, and procedures are available for analysis of the effects of instream activities.

Procedure

General

This document presents three levels of procedures that may be applied for evaluation of sand and gravel operations in areas adjacent to or within watercourses. The first level procedure may be applied to estimate the size of an adequate erosion buffer area between an active river channel and a near-stream operation. The second level procedure may be used to investigate the erosion resistance of buffer materials, in cases where the applicant desires to reduce the buffer area developed using the Level 1 procedures. A third level procedure is presented to enable more definitive determination of the erosional/depositional tendencies of a channel adjacent to a near-stream mining site, or to determine the potential impacts of instream mining operations.

The aggradation/degradation trends of river reach that contains or is adjacent to a sand and gravel mining operation are governed by the same processes that act on an unmined reach -- differentials in sediment transport capacities and sediment supply result in degradation in areas of deficit and aggradation in areas of surplus. The potential hazard associated with sand and gravel mining operations in the vicinity of watercourses may be evaluated using the same procedures as those described in the Channel Degradation and Lateral Migration portions of this State Standard. The mining area is analyzed either as a particular portion of the river (for the case of an instream site), or as an off-channel development (for an operation established adjacent to a river's banks).

For mining operations that are to be established outside of the floodplain, the Level I, II, or III techniques detailed in the Lateral Migration guideline would apply. Instream operations, however, require the application of more rigorous procedures. The mining area is separated into subreaches of similar geometry and hydraulics (i.e., (1) the reach upstream of the mining area, (2) the upstream slope down into the pit, (3) the pit itself, and (4) the reach immediately downstream of the pit), and analyzed using river modeling procedures.

The recommended approaches for evaluation of sand and gravel mining operations in the vicinity of watercourses are summarized below:

Level I

Estimate of the required buffer distance between a near-stream site and the active channel.

Setback the top of the proposed mining pit a distance from the floodplain given by the Level I setback criteria (as detailed in the Lateral Migration Guidelines).

Level II

Evaluation of the erodibility of the buffer materials for minimization of near-stream site setback requirements.

Require a smaller setback from the floodplain boundary if justified by application of the Level II setback criteria (as detailed in the Lateral Migration Guidelines).

Level III

Mathematical modeling of the river channel to better determine the adequacy of the buffer provided for a near-stream operation or to quantify the river stability impacts associated with an instream operation.

Use steady state or movable boundary sediment transport analysis (backed up by qualitative analysis and historical evidence) to determine the short and long term impact of proposed mining operation, including headcut impacts and downstream impacts due to sediment deficit. For this level of analysis, Level III hydrology shall be performed to generate required hydrographs. Level III analyses should be performed by persons with knowledge and experience in the fields of sediment transport and river geomorphology. It is recommended that any movable boundary river modeling used for determination of lateral channel stability or for evaluation of instream mining impacts be the culmination of a thorough analysis consisting of:

(1) evaluation of historical trends;
(2) qualitative analysis based on field evaluation and application of geomorphic principles:
and, (3) steady state hydraulic and sediment transport analysis.

ARIZONA DEPARTMENT OF WATER RESOURCES
FLOOD WARNING AND DAM SAFETY SECTION

State Standard for Development of Individual Residential Lots Within Floodprone Areas

Under authority of ARS 48-3605 (A), the Director of the Arizona Department of Water Resources establishes the following standard for *Development of Individual Residential Lots Within Floodprone Areas* in Arizona:

In addition to providing floodwater surface elevations, floodplain limits and floodway limits for use in fulfilling the requirements of Flood Insurance Studies, local community officials may require the information specified in State Standard Attachment 6-96 (SSA 6-96) or by an alternative procedure reviewed and accepted by the Director.

These guidelines shall apply to individual residential lots located in all flood hazard areas identified either by the Federal Emergency Management Agency as part of the National Flood Insurance Program or by the local Floodplain Administrator. Application of these guidelines will not be necessary if the local community or county has in effect a drainage, grading, or stormwater ordinance which, in the opinion of the Department, results in the same or greater level of flood protection as application of these guidelines would ensure.

This requirement is effective November 1, 1996. Copies of this State Standard can be obtained by contacting the Arizona Department of Water Resources at (602) 417-2445. Should you need this publication in alternate format, please contact the Arizona Department of Water Resources with your needs at (602) 417-2445 or (602) 417-2455 (TDD).

SITE PLAN CHECKLIST

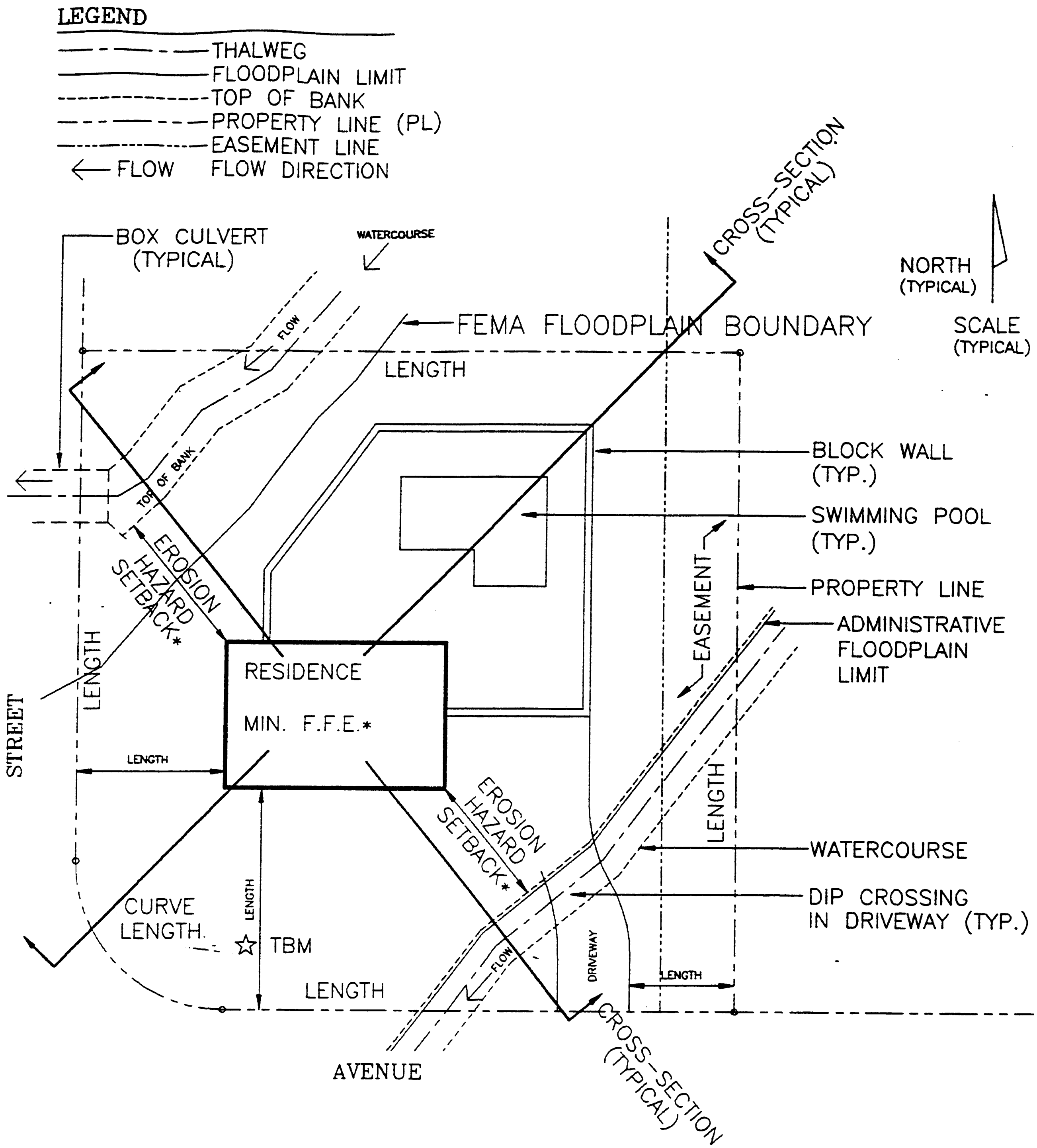
A site plan (plot plan) is required and should be drawn to a scale (not smaller than 1" = 60') in black ink suitable for reproduction and include the following information at a minimum:

1. All watercourses on the subject lot or within 300 feet of existing or proposed buildings. For purposes of this State Standard a watercourse is defined as having a drainage area greater than one quarter square mile or yielding a peak flow rate greater than 500 cfs (cubic feet per second) during a 100-year flood event.
2. Subject lot boundary dimensions with drawing scale and north orientation arrow.
3. Proposed structure location, including its external dimensions.
4. Any existing structure location, including its external dimensions.
5. Adjacent alleys, roads, streets or means of access.
6. Location of driveway(s) and distance to nearest property line.
7. Building setback distances (measured from nearest top of bank) - erosion hazard (if applicable).
8. Distance(s) from existing and proposed buildings to property line.
9. Distance(s) between buildings (if applicable).
10. Location of entire septic system (if applicable).
11. Location of all on-site utility poles, meters (and elevations), lines, etc.
12. Terrain slope - local drainage flow directions.
13. Slope information (may be given in units of feet per foot or percentage of slope).

    A. Indicate high point and low point of subject lot if terrain slopes.
    B. Indicate by arrow or contour the direction of terrain slope.
    C. Indicate difference in elevation between high point and low point of lot.
    D. Field photographs with scale of watercourse.

14. All road cuts or fills within 50 feet of the subject parcel, roadside ditches and culverts (including size).
15. Location and type of walls and fences (and adjacent property), existing and proposed.
16. Minimum FFE (finished floor elevation).
17. Two cross sections of the parcel drawn to an appropriate scale. Both cross sections should include the house site, and at least one of the cross sections should include the watercourse(s).
18. Grading limits.
19. FEMA (Federal Emergency Management Agency) Floodplain Boundaries.
20. Administrative floodplain limits.
21. All easements.
22. Temporary Benchmark

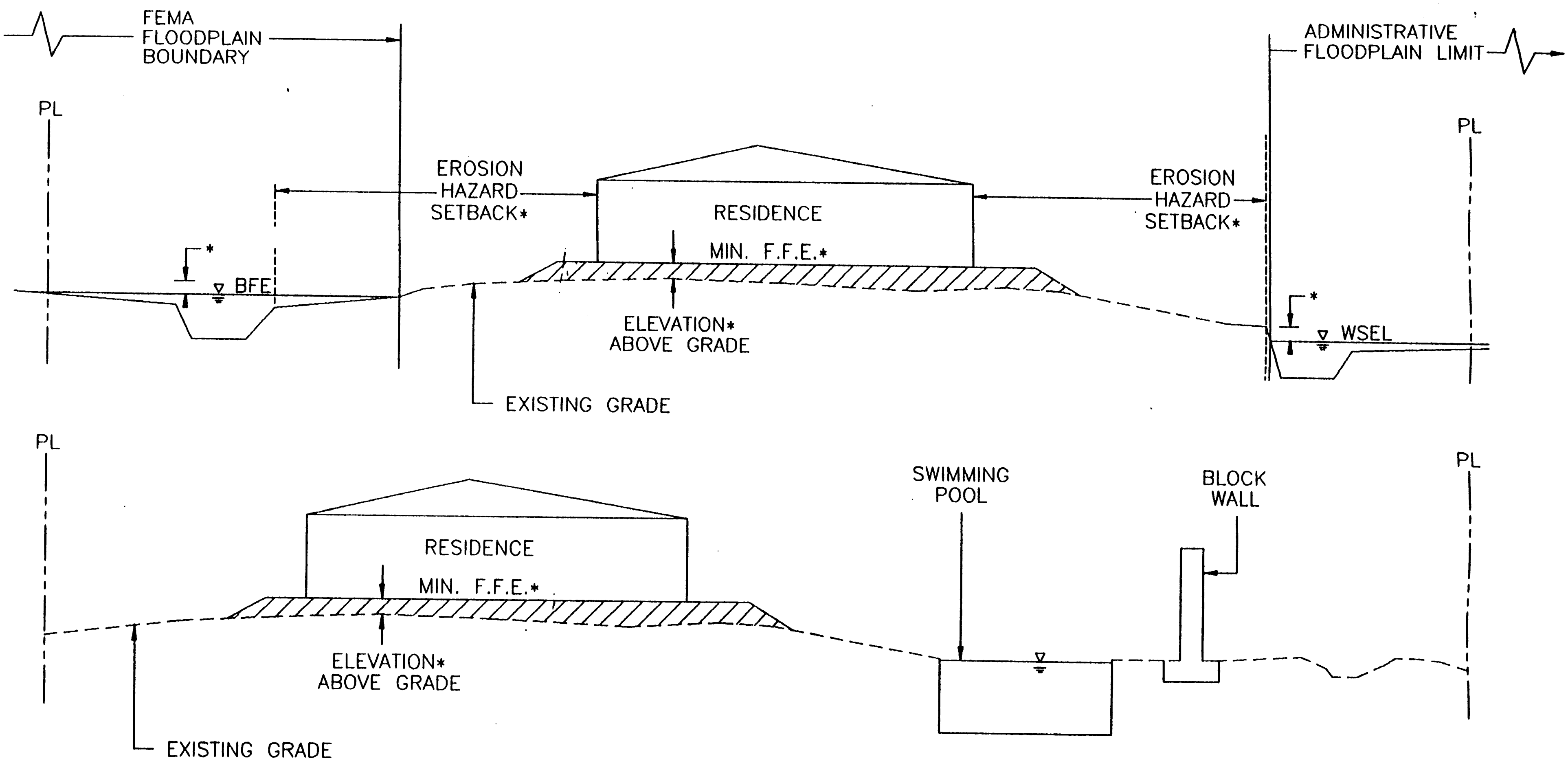
Note: All measurements must be in English Units (i.e., feet).



MINIMUM REQUIRED SITE PLAN FOR DRAINAGE AND FLOODPLAIN INFORMATION

* Established by State Standard or Local Jurisdiction
☆ Temporary Benchmark (If Required by Local Jurisdiction)

State Standard Attachment 6-96 November, 1996



TYPICAL CROSS-SECTIONS

SCALE
(TYPICAL)

* Established by State Standard or Local Jurisdiction

NOTE: Maximum fill and structure height as established by local jurisdiction.

BFE = Base flood elevation.
WSEL = Water surface elevation.

ARIZONA DEPARTMENT OF WATER RESOURCES
FLOOD MITIGATION SECTION

REQUIREMENT FOR

FLOOD STUDY TECHNICAL DOCUMENTATION

The Director of the Arizona Department of Water Resources under the authority outlined in ARS 48-3605(A establishes the following technical documentation requirement for all flood studies submitted to the Arizon Department of Water Resources or the Federal Emergency Management Agency by communities, counties or individuals in Arizona:

Flood Studies submitted to the Arizona Department of Water Resources or the Federal Emergency Management Agency for the purpose of delineating floodplains or revising existing floodplains shall meet th technical documentation standards as set forth in the Department's publication entitled "Instructions for Organizing and Submitting Technical Documentation for Flood Studies".

This requirement is effective November 1, 1997. State Standard 1-97 and State Standard Attachment 1-97 replace State Standard 1-90 and State Standard Attachment 1-90, adopted in September, 1990. Please discard all copies of the superseded standard and attachment.

Copies of this State Standard and State Standard Attachment can be obtained by contacting the Department's Flood Mitigation Section at (602) 417-2445.

NOTICE

This document is available in alternative formats. Contact the Department of Water Resources, Flood Mitigation Section at (602) 417-2445 or (602) 417-2455 (TDD).

ARIZONA DEPARTMENT OF WATER RESOURCES
FLOOD MITIGATION SECTION

Instructions for

Organizing and Submitting

Technical Documentation

for Flood Studies

500 North Third Street
Phoenix, Arizona 85004

(602) 417-2445

STATE STANDARD ATTACHMENT
SSA1-97

November 1997

DISCLAIMER OF LIABILITY

The Arizona Department of Water Resources is not responsible for the application of the methods outlined in this publication and accepts no liability for their use. Sound engineering judgment is recommended in all cases.

The Arizona Department of Water Resources reserves the right to modify, update, or otherwise revise this document. Questions regarding information contained in this document and/or floodplain management should be directed to the local floodplain administrator or the office below:

Flood Mitigation Section
Arizona Department of Water Resources
500 North Third Street
Phoenix, Arizona 85004

Phone: 602-417-2445
FAX: 602-417-2423

TABLE OF CONTENTS

I. INTRODUCTION

Overview

The Arizona Department of Water Resources (ADWR) has established documentation standards that affect flood studies submitted to the ADWR or to the Federal Emergency Management Agency (FEMA). Flood studies for the purpose of delineating floodplains or revising floodplains in Arizona must meet the technical documentation standards outlined in this publication. This technical documentation standard is to be applied for all Level 3 Methodology studies as defined in ADWR State Standard Attachment SSA2-96.

The purpose of this requirement is to ensure that adequate technical documentation for all flood studies will be available in the future. Past experiences with the documentation available from studies completed for FEMA indicate that many of the technical details of the studies have been lost. This results in additional costs to public agencies and private individuals whenever the studies need to be updated or changed. In addition, adequate review by communities, counties, ADWR and FEMA of any proposed revisions or additions to the floodplain areas of the state will insure that the quality of all studies remains adequate.

This publication requires the study preparer to incorporate all essential technical data into one comprehensive data package to be known as the Technical Data Notebook (TDN). This publication outlines the documentation indexing system to be used in preparation of the TDN.

Submission of a completed TDN is required whenever a study is to be reviewed by ADWR and forwarded to FEMA. ADWR suggests that all Arizona communities require TDN submittals whenever floodplain studies are submitted that modify existing flood hazard areas or delineate new flood hazard areas. A TDN should be forwarded to ADWR for inclusion in the Archives even if approval by ADWR and FEMA is not a requirement.

Reports for Submittals to ADWR and FEMA

The outline for a TDN report submitted to ADWR and FEMA is different than for submittals only made to local government entities and ADWR. Section 1 of the TDN is the same for both types of submittals. Section 2 of the TDN is the main body of the report and contains the Study Documentation Abstract for FEMA submittals, and the application and certification forms for Conditional Letters of Map Revision (CLOMR), Letters of Map Revision (LOMR), and Physical Map Revisions (PMR). The remaining sections of the TDN are to be used for supplemental information that cannot fit within the space allowed on the appropriate FEMA form. FEMA MT-2 Form 3 (Hydrologic Information Form), for instance, would be expanded using Section 4 of the TDN. A reference to the applicable section and sub-section in the TDN is to be placed on the appropriate line of the FEMA form. Maps are to be organized and located as described in the TDN outline. Refer to Table 1 for a direct comparison of the outlines used for the two types of submittals. Refer to Section III of this document for additional information.

Reports for Submittals to Local Government and ADWR

The outline for a TDN report submitted to local government entities and ADWR is different than for submittals to ADWR and FEMA. Section 1 of the TDN is the same for both types of submittals. Section 2 of the TDN is used for abstract data as listed and described in the Appendix. The remaining sections of the TDN are used as the main body of the report and should contain detailed descriptions o. the substantiating data, assumptions, results and conclusions for the floodplain delineation. Refer to Table 1 for a direct comparison of the outlines used for the two types of submittals. Refer to Section III of this document for additional information.

II. REPORT STRUCTURE

Description

The general TDN structure is shown in Table 1.

Table 1

General structure of a TDN report

Section	Report outline main headings: ADWR/FEMA Submittals	Report outline main headings: Local Government/ADWR Submittals
TOC	Table of Contents	Table of Contents
1	Introduction	Introduction
2	ADWR/FEMA Forms	Local Government/ADWR Abstracts
3	Mapping and Survey Information	Mapping and Survey Information
4	Hydrology	Hydrology
5	Hydraulics	Hydraulics
6	Erosion and Sediment Transport	Erosion and Sediment Transport
7	Draft FIS Data	N/A
Appendix A	References	References
Appendix B	General Documentation and Correspondence	General Documentation and Correspondence
Appendix C	Survey Field Notes	Survey Field Notes
Appendix D	Hydrologic Analysis Supporting Documentation	Hydrologic Analysis Supporting Documentation
Appendix E	Hydraulic Analysis Supporting Documentation	Hydraulic Analysis Supporting Documentation
Appendix F	Erosion and Sediment Transport Analyses Supporting Documentation	Erosion and Sediment Transport Analyses Supporting Documentation
Diskettes	Digital data files	Digital data files
A. Maps	Hydrology Exhibit Maps	Hydrology Exhibit Maps
B. Maps	Hydraulics Exhibit Maps	Hydraulics Exhibit Maps
C. Maps	Floodplain Work Study Maps	Floodplain Work Study Maps

Each report is to follow the structure set forth in Table 1. The structure is presented in more detail in section III of this document. The following are guidelines for use of the structure in preparing TDN reports. Refer to section IV for general standards for the appearance of the materials contained in the TDN.

Guidelines for use of the TDN Report Structure

1. TDN reports submitted to ADWR and FEMA: The latest edition of the MT-2 FEMA form series shall be used for Section 2 of the TDN in addition to the Study Documentation Abstract for FEMA Submittals. Refer to Section 2 for more information, and the Appendix for an abstract form. The remaining sections and appendices shall be used to organize and document overflow and supplemental information that can not fit within the room allotted on the FEMA forms. Report sections that are not pertinent are to be identified accordingly in the text. The MT-2 FEMA Form 81-89, May 96 can be obtained from the following sources:

Arizona Department of Water Resources
Flood Mitigation Section
500 North Third Street
Phoenix, AZ 85004-3903
(602) 417-2445 voice
(602) 417-2401 fax

FEMA Distribution Warehouse
8231 Stayton Drive
Suite E
Jessup, Md 20794
(800) 480-2520

These forms will be soon be available in Adobe Acrobat Reader Version 3.0 digital format on the Internet at HTTP://www.fema.gov/library.

2. TDN reports submitted to Local Government Entities and ADWR: The abstract forms in the Appendix shall be used for Section 2 of the TDN report instead of the FEMA forms. Each field of the abstract forms is explained in Section III of this document.

3. Voluminous reports: Reports that are too large to fit within one binder may be split into multiple volumes. A possible procedure is to separate the hydrology and hydraulics sections into two volumes. Other methods are possible and may be used with the approval of the reviewing agency. For this example, the appropriate data pertinent to each subject should be included in each volume. Project specific information applicable to both categories should be included in only one volume, with appropriate references made in the other volume. Examples, assuming a two volume set with volume 1 covering hydrology and volume 2 covering hydraulics, are:

Section 1, Introduction: The introduction should be similar in both volumes, with reference made to the other volume.

Section 2, ADWR/FEMA forms and Local Government/ADWR Abstracts: The FEMA forms or ADWR abstracts may be placed in either volume, with appropriate references made, or split between volumes by topic, at the discretion of the preparer.

Section 3, Mapping and Survey Information: Mapping and survey information specific to hydrology should be placed in the hydrology volume. Mapping and survey information specific to floodplain and floodway delineation should be placed in the hydraulics volume.

Section 4, Hydrology: A summary of the hydrology results used for floodplain delineation along with a reference to the hydrology volume would be included in the hydraulics volume. The hydrology volume would contain the complete discussion of hydrology methods, parameters and results.

Section 5, Hydraulics: The hydraulics volume should contain a complete discussion of the floodplain and floodway delineation methods, parameters and results. The hydrology volume should contain a reference to the hydraulics volume.

Section 6, Erosion and Sediment Transport: This section is be placed in the hydraulics volume, and reference made in the hydrology volume to the hydraulics volume.

Section 7, Draft FIS Data: This section only applies to reports submitted to ADWR and FEMA. The draft FIS Summary of Discharges, Floodway Data tables, annotated Flood Insurance Rate Maps and Flood Profiles are to be included in either or both volumes, at the discretion of the study preparer.

Appendix A, Reference Materials: The appropriate reference materials specific to hydrology or hydraulics would be included in the appropriate volume.

Appendix B, General Documentation and Correspondence: All general documentation and correspondence should be in either the hydrology or hydraulics volume. Reference should be made in the other volume. The TDN should only contain appropriate technical correspondence between the study preparer and the reviewing agencies. This appendix is not intended to burden the study preparer by requiring a complete project correspondence file.

Appendix C, Survey Field Notes: Field survey notes specific to each topic are be included in the appropriate volume.

Appendix D, Hydrologic Analysis Supporting Documentation: This appendix should only be included in the hydrology volume.

Appendix E, Hydraulic Analysis Supporting Documentation: This appendix should only be included in the hydraulics volume.

Appendix F, Erosion/Sediment Transport Analysis Supporting Documentation: This appendix is to be included in the hydraulics volume.

Diskettes: Diskettes containing hydrology and hydraulics specific digital files are to be included in the appropriate volume.

Maps: Hydrology and hydraulics specific maps or exhibits are be included in the appropriate volume.

4. Exhibits and Maps: Exhibits and maps are to be included in pockets at the back of the report whenever possible. Larger maps, such as the Work Study Maps, can be provided bound and rolled under separate cover.

5. Duplication: Duplication of data within the report is to be avoided where possible. Use references to one location to avoid duplication.

6. Expansion of Report: The TDN outline can be expanded beyond Section 7 and Appendix F. This may be desirable when using the TDN format for a drainage design report or drainage master plan.

The report outline can be broken down into subsections as necessary for a particular report. For example, Section 3.2.2 Physical Parameters could be broken down into sub-sections as follows:

Section 3.2.2.1 Watershed subbasin parameters
Section 3.2.2.2 Reach Route Parameters
Section 3.2.2.3 Storage Route Parameters

7. Sections that are not used, and restructuring: Sections that are not appropriate for a particular study must still be included in the TDN. Make a statement or reference accordingly under the unused section. Renumbering of TDN report sections in the main body for other purposes will not be permitted.

III. TDN OUTLINE

The material contained within the TDN will be organized as listed and described below. Sections that are not applicable do not have to be contained in the TDN; however, the numbering system should not be changed. Specific minimum standards are listed when appropriate.

The following is the detailed outline of the TDN:

Title Page: The title page is to contain the name and location of the study, and the name, address, phone number and project number (or contract number) of the study requestor and the study preparer, and the date of preparation. The title page must bear the professional registration seal of the study preparer.

Table of Contents: The table of contents is to include a list of figures, list of tables, a list of appendices and the professional registration seal of the study preparer.

Section 1: Introduction

The introduction is to be structured to provide an overview of the material contained in the TDN. The introduction should include the following, organized at the discretion of the study preparer:

- Purpose of study (LOMR, CLOMR, new delineation, hydrology only or hydraulics only). Describe why a CLOMR or LOMR is necessary and the reasons for any revisions to hydrology or hydraulics.
- Authority for study (client name, contract number and date, project manager, etc).
- Location of study reach by section, township, range, community and county. Provide a location map and vicinity map.
- Brief statement of methodology used for hydrology and hydraulics.
- Acknowledgments.
- Brief description of study results.

Section 2: ADWR/FEMA Forms and Local Government/ADWR Abstracts

TDN Section 2 is to contain the Study Documentation Abstract for FEMA Submittals and the FEMA MT-2 forms for reports submitted to ADWR and FEMA, or the complete Study Documentation Abstract for reports submitted to Local Governments and/or ADWR. The FEMA forms provide the main report data, with TDN Sections 3 through 7 and the Appendices used for overflow and additional information that cannot be placed within the form structure. The purpose of the Local Government/ADWR abstracts is to provide a number of key facts about the study being documented. Sample abstract forms are contained in Appendix A. The following is an explanatory list of the required information for both the Study Documentation Abstract for FEMA Submittals and the complete Study Documentation Abstract form:

SECTION 2 FOR REPORTS SUBMITTED TO ADWR AND FEMA

The Study Documentation Abstract for FEMA Submittals (see abstract form in Appendix A) is to be provided as Section 2.1 and must contain the following information. Items 2.1.1, 2.1.3, 2.1.4 and 2.1.5 will be completed by ADWR after the study is accepted by FEMA.

Section 2: ADWR/FEMA Forms

Section 2.1: Study Documentation Abstract for FEMA Submittals

2.1.1 **Date Study Accepted:** Date that study was accepted by FEMA

2.1.2 **Study Contractor:** Study Contractor-Firm or agency name, address and telephone number of firm or agency that completed the study. Name of contact person at firm or agency who would be able to discuss the technical aspects of the study. Study Contractor contract number and list of subcontractors.

2.1.3 **FEMA Technical Review Contractor:** Name, address and phone number of FEMA technical review contractor that reviewed the study.

2.1.4 **FEMA Regional Reviewer:** FEMA Regional reviewer and telephone number.

2.1.5 **State Technical Reviewer:** State reviewer (if any) along with telephone number.

2.1.6 **Local Technical Reviewer:** County or community reviewer and telephone number.

2.1.7 **Reach Description:** Description of the reaches of each river, stream or watercourse studied in documented report. This should include FIRM panel numbers and EPA reach number, if available.

2.1.8 **USGS Quad Sheets:** A list of map names and dates for the USGS 7.5' or 15' quadrangle maps of the study area. If desired, other maps that better describe the study area can be referenced instead of the USGS quads if these maps are easily obtainable. Dates of maps and photography referenced should be included.

2.1.9 **Unique Conditions and Problems:** Description of any unique conditions or problems found during the study.

2.1.10 **Coordination of Peak Discharges:** Description of process to coordinate peak flows with applicable agencies. Should include date, agency name, person contacted and indication of agency concurrence or comments.

Section 2.2: FEMA Forms

The FEMA forms are to be placed under this section, with overflow and additional data and information placed in Sections 3 through 7 and the Appendices, as appropriate.

SECTION 2 FOR REPORTS SUBMITTED TO LOCAL GOVERNMENTS AND/OR ADWR

The complete Study Documentation Abstract (see abstract form in Appendix A) is to be provided as Section 2 and must contain the following information:

Section 2: Local Government/ADWR Abstracts

Section 2.1: General Information

2.1.1 **Community:** Community name

2.1.2 **Community Number:** National Flood Insurance Program (NFIP) Community Number

2.1.3 **County:** County or Counties where community is located

2.1.4 **State:** State where community is located

2.1.5 **Date Study Accepted:** Date study was accepted by ADWR and/or the Local Government entity.

2.1.6 **Study Contractor:** Study Contractor-Firm or agency name, address and telephone number of firm or agency that completed the study. Name of contact person at firm or agency that would be able to discuss the technical aspects of the study. Study Contractor contract number and list of subcontractors.

2.1.7 **State Technical Reviewer:** State reviewer (if any) along with telephone number.

2.1.8 **Local Technical Reviewer:** County or community reviewer along with telephone number.

2.1.9 **River or Stream Name:** Names of rivers, streams or watercourses analyzed in the documented study.

2.1.10 **Reach Description:** Description of the reaches of each river, stream or watercourse studied in documented report.

2.1.11 **Study Type:** Type of study completed on each river, stream or watercourse. This item is to clearly identify whether the study was riverine, alluvial fans, or other special hazard type study.

Section 2.2: Mapping Information

2.2.1 **USGS Quad Sheets:** A list of map names and dates for the USGS 7.5' or 15' quadrangle maps of the study area. If desired, other maps that better describe the study area can be referenced instead of the USGS quads if these maps are easily obtainable. Dates of maps and photography referenced should be included.

2.2.2 **Mapping for Hydrologic Study:** Description of maps used in the hydrologic portion of the study (if any) including type/source, scale, the dates of the maps, and the dates aerials were flown.

2.2.3 **Mapping for Hydraulic Study:** Description of maps used in the hydraulic portion of the study including type/source, scale, the dates of the maps, and the dates of aerial topography.

Section 2.3: Hydrology

2.3.1 **Model or Method Used:** Description of the hydrologic methodology or computer model used to estimate the peak flow rates used in the study. Description should include computer model vendor and version of model used.

2.3.2 **Storm Duration:** Indication of the storm duration used to estimate peak flow rate.

2.3.3 **Hyetograph Type:** Description of hyetograph type used in modeling.

2.3.4 **Frequencies Determined:** List of peak flow frequencies estimated in the hydrologic study (i.e., 10, 50, and 100-year, etc.).

2.3.5 **List of Gages used in Frequency Analysis or Calibration:** List of gages used to calibrate the computer model or used in a statistical frequency computation. Information should include gage name, gage location, USGS number (if any), ownership and years of record.

2.3.6 **Rainfall Amounts and Reference:** List rainfall amount(s), duration(s), aerial and temporal distribution(s) used for hydrologic modeling. Provide additional data and description in Section 4.2.5.

2.3.7 **Unique Conditions and Problems:** Description of any unique conditions or problems found during the study.

2.3.8 **Coordination of Q's:** Description of process to coordinate peak flows with applicable agencies. Should include date, agency name, person contacted and indication of agency concurrence or comments.

Section 2.4: Hydraulics

2.4.1 **Model or Method Used:** Description of hydraulic methodology or computer model used to determine flood elevations. Description should include computer model vendor and version of model used and any program modifications made by the contractor with supporting documentation.

2.4.2 Regime: Description of flow regime (i.e., subcritical, supercritical, mixed, etc.)

2.4.3 Frequencies for which Profiles were Computed: List of frequencies for which water surface elevations were calculated.

2.4.4 Method of Floodway Calculation: Description of method used to determine floodway (if any).

2.4.5 Unique Conditions and Problems: Description of any unique conditions or problems that impacted the study. This should include any hydraulic conditions such as jumps as well as any portion of the study where elevations were set, rather than computed by the computer model.

Section 2.5: Additional Study Information: Provide additional detail for any of the above sections.

Section 3: Survey and Mapping Information

3.1 Field Survey Information

Provide a description of all survey information used in the study, including the dates when the survey work was performed. Document the professional responsible for field work, and the company name and project number if the work is done by a sub-consultant. Provide a description of how the field notes in Appendix C are organized, and any other pertinent information necessary to understand the information in Appendix C. The information in Appendix C are to be sealed by a Land Surveyor registered in the State of Arizona.

3.2 Mapping

Provide a description of mapping and map control used in the study. Provide a narrative overview identifying the mapping datum (both horizontal and vertical), date of the aerial photography, mapping scale, and contour interval. Document the date of the last overall vertical control survey upon which the referenced benchmarks are based. Provide additional documentation verifying the accuracy of benchmarks located in areas of known subsidence. Describe the flight path followed, the time-of-day photographs were taken, the number of stereo models used, and the photo scale. Distinguish between mapping used for hydrology and mapping used for hydraulics. Document the professional responsible for developing the mapping, and the company name and project number if the work is done by a sub-consultant.

Section 4: Hydrology

4.1 Method description.

Provide a narrative description of the hydrologic methods or models used in the study. Include the model name, date, and source.

4.2 Parameter estimation.

This section and its subsections should include a complete description of the methodology and calculations used to develop the hydrology.

4.2.1 Drainage area boundaries.

Describe the limits of the study watershed and the general watershed characteristics. Provide a general watershed map of the study area no larger than 11" x 17" to scale, showing the study area boundary, major sub-basin boundaries, and concentration points.

4.2.2 Watershed work maps

Describe the watershed work maps prepared as a part of the study and included as exhibit drawings. Discuss the nomenclature used to name subbasins, concentration points, routing reaches, reservoir routes and flow diversions. Exhibits should be prepared covering the watershed, to scale, that depict the following, as a minimum:

1. Subbasin boundaries and concentration points;
2. Time-of-concentration or lag flow paths;
3. Hydrograph routing paths;
4. Soils boundaries; and
5. Land-use boundaries.

The exhibits are to be placed in pockets at the end of the report, or bound under separate cover if too voluminous. Reduced copies of the exhibits are to be placed in this section if practical.

4.2.3 Gage Data.

Identify and discuss locations of any National Weather Service (NWS), USGS or other agency gage stations in or adjacent to the region and watershed in relation to historic precipitation, watershed runoff and statistical parameters.

4.2.4 Statistical parameters

Provide a narrative discussion of the data record and information available on precipitation, runoff and discharge for the region and the study watershed. Assess the adequacy and applicability of the record for use with Water Resources Council Bulletin 17B, (March 1982). Discuss factors that may effect the reasonableness of frequency analysis for the study watershed and describe why or why not the methods in Bulletin 17B are used for estimating peak discharges for the study. Refer to <u>Basin Characteristics and Streamflow Statistics in Arizona as of 1989, USGS Water-Resources Investigations Report 91-4041</u> for state-wide data and results of log-Pearson Type III analyses.

4.2.5 Precipitation.

Provide further detail than described in Section 4.2.4. The additional detail should include a narrative discussion with supporting data of the historic precipitation records in or adjacent to the study watershed. Discuss the watershed size, the nature of historic flooding, the types of storms that result in flooding and the typical aerial extent of historic storms. State the rainfall duration and distribution pattern and the point rainfall values used for hydrologic modeling. Relate the hypothetical model design precipitation and distribution from stated reference sources to the historic record and statistical parameters.

4.2.6 Physical parameters.

Describe the methods used for estimation of the physical hydrologic parameters, such as rainfall losses, the unit hydrograph used and time-of-concentration or lag. The discussion of rainfall losses should include the soils information used including the data source, surface retention losses, percent impervious estimates for natural and developed watersheds, and the effects of vegetation cover. Provide summary tables listing the physical parameters for every subbasin in the hydrologic models.

4.3 Problems encountered during the study.

4.3.1 Special problems and solutions

Special problems are unique situations that are not addressed by the standard TDN outline. Provide a narrative discussion of any special problems that were encountered during the study. Describe the alternatives examined and the final solution used for each problem.

4.3.2 Modeling warning and error messages

Discuss any warning and error messages present in the computer model output and the effects of such messages on the accuracy of the results.

4.4 Calibration.

Provide a narrative discussion of hydrologic model calibration that was accomplished or attempted. This would include adjustment of model parameters to provide a closer correlation with physical runoff volumes and/or peak discharges of record for the study wash.

4.5 Final results.

4.5.1 Hydrologic analysis results.

Describe the results of the statistical or modeling efforts. Provide summary tables of results for each sub-basin modeled, at the locations necessary for proper floodplain delineation, and at other points of interest. The tables should include the following:

1. Peak discharge and time-to-peak for each recurrence interval storm analyzed;
2. Runoff volume for each recurrence interval storm analyzed;
3. Peak stage and inflow and outflow peak discharges for reservoir route operations; and
4. Peak flow rates for each branch of a flow split or diversion.

4.5.2 Verification of results.

Discuss the reasonableness of the results. Describe comparisons of the results with indirect methods such as:

1. Other FIS studies in the area;
2. Gaged watershed data for similar watersheds; and
3. Indirect methods set forth in the Highway Drainage Design Manual, Hydrology, April 1994 by the Arizona Department of Transportation, including regression equations, envelope curves and other confidence checks.

Section 5: Hydraulics

5.1 Method description.

Describe the location and physical characteristics of the streams or washes for which floodplain limits are defined. Provide a narrative description of the water surface profile model used in the study. Include the model name, date and source. Explain how the starting water surface elevations (WSEL) for the various streams are determined.

5.2 Work study maps

Describe the work study floodplain maps prepared as a part of the report. Discuss the nomenclature used in preparation of the maps. Explain how the streams and washes are divided into reaches based on changes in peak discharge and roughness coefficients. Provide a report figure which is a general overview map of the study area. The floodplain delineation reaches and key features of the study area are to be identified on the map. The figure is to be no larger than 11" x 17" and drawn to scale. Provide reduced scale work-study maps no larger than 11" x 17", in the report volume, in addition to full scale work study maps. All maps must have a graphic scale bar. Refer to Section IV of this document for required information to be placed on all maps and exhibits.

5.3 Parameter estimation.

5.3.1 Roughness coefficients.

Document the source or method of estimating the channel roughness coefficients, such as Manning's n-values. Include photographs of appropriate stream reaches. Provide a summary table of the selected coefficients organized by reach.

5.3.2 Expansion and contraction coefficients.

Document the source or method used to estimate expansion and contraction coefficients. Describe the physical characteristics of the stream and obstructions to flow that require changes in coefficients from the norm.

5.4 Cross section description.

Provide a narrative discussion of the placement of cross sections and the cross section orientation. Describe how the cross sections are obtained.

5.5 Modeling considerations.

5.5.1 Hydraulic Jump and drop analysis.

Describe locations where a hydraulically significant hydraulic jump or drop may be expected to occur. State how the floodplain limits are adjusted, if at all, to account for these phenomena.

5.5.2 Bridges and culverts.

Provide a narrative discussion of the methods used to model bridges and culverts. Describe any assumptions made in the analyses. Provide a summary table listing the location of each structure, a description of the type of structure, and the method used to model it. List any as-built drawings available (with date of preparation and year of construction, if known) for each structure or state that as-built dimensions are obtained by field survey.

5.5.3 Levees and dikes.

Describe the location, extent and physical characteristics of hydraulically significant levees or dikes present along the study streams or washes. Provide a narrative discussion of the methods used to model the effects of these structures. List any as-built drawings available (with date of preparation and year of construction, if known) for each structure or state that as-built dimensions are obtained by field survey.

5.5.4 Islands and flow splits.

Describe the location, extent and physical characteristics of hydraulically significant islands or flow splits present along the study streams or reaches. Provide a narrative description of how the effects of these areas are modeled. List any assumptions made.

5.5.5 Ineffective flow areas.

Describe the location, extent and physical characteristics of hydraulically ineffective flow areas present along the study streams or reaches. Provide a narrative description of how the effects of these areas are modeled. List any assumptions made.

5.5.6 Supercritical flow.

List and describe reaches of supercritical flow in each stream or wash as set forth in ADWR State Standard 3-94.

5.6 Floodway modeling.

Provide a narrative discussion of the encroachment methods and procedures used to define floodway limits.

5.7 Problems encountered during the study.

5.7.1 Special problems and solutions.

Special problems are unique situations that are not addressed by the standard TDN outline. Provide a narrative discussion of any special problems that were encountered during the study. Describe alternatives examined and the final solution used for each problem.

5.7.2 Modeling warning and error messages.

Discuss any warning and error messages present in the computer model output and the effects of such messages on the accuracy of the results.

5.8 Calibration.

Provide a narrative description of any model calibration procedure attempted or accomplished.

5.9 Final results.

5.9.1 Hydraulic analysis results.

Describe the results of the hydraulic modeling efforts. Provide summary tables of results for each stream or wash. The tables should include the following:

Normal stream results:

1. Cross section identifier;
2. Peak discharge;
3 Water surface elevation;
4. Critical water surface elevation;
5. Average velocity;
6. Top width of flow;
7. Depth of flow;
8. Froude number; and
9. Left and right stations where water surface meets existing ground.

Bridge or Culvert Results for all cross sections defining the structure:

1. Cross section identifier;
2. Water surface elevation;
3. Energy grade line elevation;
4. Peak discharge;
5. Discharge through structure;
6. Discharge over weir;
7. Velocity head;
8. Friction loss; and
9. Contraction and expansion coefficients.

5.9.2 Verification of results.

Discuss the reasonableness of the results. Describe comparisons of the results with any previous studies.

Section 6: Erosion and Sediment Transport

6.1 Method description.

Describe the location and physical characteristics of the streams or washes for which erosion limits are estimated and/or sediment transport analyses are performed. Provide a narrative description of the methodologies and models used in the study. Include the model name, date and source. Describe efforts to use historical data such as aerial photographs to establish the geomorphology of the river. List the dates and sources of aerial photographs and any other data sources used. Describe apparent changes to the channel alignment or geometry over time. Document whether those changes are due to natural processes, man-made obstructions or disturbances, or a combination of both.

6.2 Parameter estimation.

This section and any subsections should include a complete description of the methodology, sources and calculations used to develop the parameters for erosion and sediment transport modeling.

6.4 Modeling considerations.

Create subsections as necessary to describe the modeling considerations addressed during the study.

6.5 Problems encountered during the study.

6.5.1 Special problems and solutions.

Special problems are unique situations that are not addressed by the standard TDN outline. Provide a narrative discussion of any special problems that were encountered during the study. Describe alternatives examined and the final solution used for each problem.

6.5.2 Modeling warning and error messages.

Discuss any warning and error messages present in the computer model output and the effects of such messages on the accuracy of the results.

6.6 Calibration.

Provide a narrative description of any model calibration procedure attempted or accomplished.

6.7 Final results.

6.7.1 Erosion and sediment transport analysis results.

Describe the results of the erosion and sediment transport efforts. Provide summary tables of results for each stream or wash.

6.7.2 Verification of results.

Discuss the reasonableness of the results. Describe comparisons of the results with any previous studies.

Section 7: Draft FIS Report Data

This section only applies to reports submitted to ADWR and FEMA.

7.1 Summary of Discharges.

Provide a draft Summary of Discharges results table in FEMA format.

7.2 Floodway Data.

Provide a draft Floodway Data results table in FEMA format.

7.3 Annotated Flood Insurance Rate Maps

Provide draft Flood Insurance Rate Maps in FEMA format.

7.4 Flood Profiles

Provide draft Flood Profiles in FEMA format.

Appendix A: References

A.1 Data collection summary.

Include a list of previous studies, other applicable studies, published and unpublished historical flood information, and research contacts.

A.2 Referenced documents.

Provide a list of all technical papers and documents pertaining to the methodology used in the study. Provide a copy of any paper or document critical to the analysis if there is any question of the reviewing agency having the referenced papers or documents.

Appendix B: General Documentation & Correspondence

B.1 Special Problem Reports.

B.2 Contact (telephone) reports.

Provide copies of correspondence documenting notification of the client and the methods of addressing any special problems described in Sections 4.4.1, 5.5 and 6.5.

B.3 Meeting minutes or reports.

B.4 General Correspondence.

B.5 Contract Documents.
Provide a copy of the contract Scope of Work, not financial documents.

Appendix C: Survey Field Notes

The field survey notes are to be clear and concise with appropriate sketches and notations. All field survey procedures and notes should meet requirements of State Board of Technical Registration and be sealed by a registered land surveyor. Provide copies of the field survey notes organized per sections C.1 through C.3. Document the date of the last overall vertical control survey upon which the referenced benchmarks are based. Provide additional documentation verifying the accuracy of benchmarks located in areas of known subsidence.

C.1 Survey field notes for aerial mapping control.

C.2 Survey field notes for hydrologic modeling.
Routing cross sections
Field reconnaissance notes for subbasin boundary verification and estimation of physical parameters.
Structures

C.3 Survey field notes for hydraulic modeling.
Cross sections
Structures
As-built drawings
ERM's

Appendix D: Hydrologic Analysis Supporting Documentation

D.1 Precipitation data.
Provide a copy of PreFre output or other calculations done to estimate precipitation frequency values described in Section 4.2.4.

D.2 Physical parameter calculations.
Include detailed summaries of parameter calculations in spreadsheet or table format.

D.3 Hydrograph routing data.
Include routing data, confidence checks on results and cross section plots.

D.4 Reservoir routing data.
Include hydraulic calculations and rating curve plots for control structures, and volume calculations.

D.5 Flow splits and diversions data.
Include hydraulic calculations and rating curve plots used to define each flow split and diversion table.

D.6 Hydrologic calculations.

Include computer model output, logic diagrams and any hand calculations.

Appendix E: Hydraulic Analysis Supporting Documentation

E.1 Roughness coefficient estimation.

Include copies of photographs and calculations.

E.2 Cross section plots.

E.3 Expansion and contraction coefficients.

Include any special data or calibration efforts made for estimation of expansion and contraction coefficients.

E.4 Analysis of structures.

Include any separate hydraulic modeling of structures used to estimate control data for floodplain delineation calculations.

E.5 Hydraulic calculations.

Include computer model output for floodplain and floodway hydraulic calculations..

Appendix F: Erosion and Sediment Transport Analysis Supporting Documentation

Include supporting documentation, parameter calculations, computer model output and any other data and results prepared as a part of the analyses.

Exhibit Maps

Hydrology watershed maps
Hydrology soils maps
Hydrology land-use maps
ERM location map
Hydraulics work study map index
Hydraulics work study maps
Hydraulic profiles
Erosion setback limit maps

IV. GENERAL DOCUMENTATION STANDARDS

This section outlines general format standards for the material to be contained in the TDN.

Appearance and Legibility

All materials contained in the TDN must be legible and of an appearance that makes tracking and review possible. It is not required that the material be typed, but if printed, it must be legible. Pencil and colored pens should be avoided unless the writing is dark enough to be reproduced on copier or microfiche. This is especially important in the case of technical analysis notes that describe the assumptions made in any analysis and the parameters selected by the engineer.

Size

Material in the TDN should be 8½" by 11". Material which is larger than standard size may be folded and included in the notebook or it may be rolled separately, clearly marked and referenced in the TDN. Reduced maps and drawings may be included provided a bar scale is legible after reduction.

Data Identification

Material included in the TDN or attached separately will be marked with the following minimum information.

- ☐ Community name, county and state.
- ☐ Date material prepared.
- ☐ Study contractor name and internal project number.
- ☐ Name of flooding sources.
- ☐ Appropriate documentation index number as outlined in Section IV of these instructions.
- ☐ Whether the product is one of several.
- ☐ Any other relevant information that can assist users in identifying the data.

Exhibit Maps

All exhibit maps, included in the TDN or attached separately, will be marked with the following information in addition to the information listed under DATA IDENTIFICATION:

- ☐ Index of maps *(8 ½" x 11" suggested size).*
- ☐ Map bar scale.
- ☐ Source of base map and date including aerial mapping subcontractor, address, telephone number and internal project number, if applicable.
- ☐ Land surveyor's seal and engineer's seal with an appropriate certification and description of what each seal covers.
- ☐ North arrow.
- ☐ Names of streams, and major streets.
- ☐ Date flown (if aerial).
- ☐ Reference marks or known benchmarks. Maps should include section, township and range

lines, and the location and datum of all points used for horizontal control. The maps should also include all bench marks used for vertical control, and the basis for the datum such as NGVD 1988 or local.

Computer Products

Computer printouts will be marked with the following information in addition to the list under DATA IDENTIFICATION:

- ☐ Multiple-profile or single profile.
- ☐ Enough information for the reviewer to understand whether this run is the final run or a supplementary run, and to describe the intent in preparing the computer run.
- ☐ Hydraulic model printouts will be further annotated to show the applicable cross/section lettering used on the draft report text. Include comment cards in the model to clearly identify road crossings, bridges and key concentration points.

Computer runs that are superseded but contained in the TDN for clarity of review will be marked "SUPERSEDED" or "VOID" in large letters.

Input data files of final runs of computerized hydraulic and hydrologic computations from standard programs such as HEC-1, TR-20 or HEC-2 will be submitted on 3½ inch diskettes, CD-ROM, or other media acceptable to the reviewing agency that meet the following specifications:

- ☐ Disks will be formatted for MS DOS 3.1 or greater and have a capacity of 1.44 megabytes.
- ☐ Data files may be partitioned to multiple diskettes provided the files are self-extracting or the extraction software is proved with the TDN.
- ☐ An ASCII text file named "README" will be created for each diskette and will contain a description of each computer file on the diskette along with the information required under DATA IDENTIFICATION. A list of files along with the information required under DATA IDENTIFICATION will be placed on the diskette label.
- ☐ All computer files should be "write protected" by the use of write protect tabs or MS DOS ATTRIB command to make files "read only".

Input and output from other types of computer compilations should be included under the appropriate index number and should clearly be identified by program name and source.

APPENDIX STUDY DOCUMENTATION ABSTRACTS

The forms on the following pages are to be used in Section 2 of the TDN as described in Section III.

Study Documentation Abstract for FEMA Submittals	Initial Study		Restudy		CLOMR		LOMR		Other	

Section 2.1: Study Documentation Abstract for FEMA Submittals		
2.1.1	Date Study Accepted	
2.1.2	Study Contractor Contact(s) Address Phone Internal Reference Number	
2.1.3	FEMA Technical Review Contractor Contact(s) Address Phone Internal Reference Number	
2.1.4	FEMA Regional Reviewer Phone	
2.1.5	State Technical Reviewer Phone	
2.1.6	Local Technical Reviewer Phone	
2.1.7	Reach Description	
2.1.8	USGS Quad Sheet(s) with original photo date & latest photo revision date	
2.1.9	Unique Conditions and Problems	
2.1.10	Coordination of Q's Discharges (Agency, Date, Comments)	

Study Documentation Abstract for Local Government and ADWR Submittals		
Section 2.1: General Information		
2.1.1	Community	
2.1.2	Community Number	
2.1.3	County	
2.1.4	State	
2.1.5	Date Study Accepted	
2.1.6	Study Contractor Contact(s) Address Phone Internal Reference Number	
2.1.7	State Technical Reviewer Phone	
2.1.8	Local Technical Reviewer Phone	
2.1.9	River or Stream Name	
2.1.10	Reach Description	
2.1.11	Study type (Riverine, Alluvial Fan, etc.)	
Section 2.2: Mapping Information		
2.2.1	USGS Quad Sheet(s) with original photo date & latest photo revision date.	
2.2.2	Mapping for Hydrologic Study Type/Source Scale Date	

Study Documentation Abstract for Local Government and ADWR Submittals		
2.2.3	Mapping for Hydraulic Study Type/Source Scale Date Subcontractor (Aerial) Date of Aerial Mapping	
Section 2.3: Hydrology		
2.3.1	Model or Method Used (including vendor and version description)	
2.3.2	Storm Duration	
2.3.3	Hyetograph Type	
2.3.4	Frequencies Determined	
2.3.5	List of Gages Used in Frequency Analysis or Calibration (Location, Years of Record, Gage Ownership)	
2.3.6	Rainfall Amounts and Reference	
2.3.7	Unique Conditions and Problems	
2.3.8	Coordination of Q's (Agency, Date, Comments)	
2.4: Hydraulics		
2.4.1	Model or Method Used (including vendor and version description)	
2.4.2	Regime	
2.4.3	Frequencies for which Profiles Were Computed	
2.4.4	Method of Floodway Calculation	
2.4.5	Unique Conditions and Problems	

Study Documentation Abstract for Local Government and ADWR Submittals

Section 2.5: Additional Study Information

Item	Description / Discussion

ARIZONA DEPARTMENT OF WATER RESOURCES
FLOOD MITIGATION SECTION

State Standard

for

Watercourse Bank Stabilization

Under authority of ARS 48-3605(a), the Director of the Arizona Department of Water Resources establishes the following standard for watercourse bank stabilization in Arizona:

The guidelines outlined in State Standard Attachment 7-98 entitled "Watercourse Bank Stabilization" or by an alternative procedure reviewed and accepted by the Director will be used in the development of designs for watercourse bank stabilization for fulfilling the requirements of Flood Insurance Studies, and local community and county flood damage prevention ordinances.

For the purpose of application of these guidelines, bank stabilization standards will apply to all watercourses identified by the Federal Emergency Management Agency as part of the National Flood Insurance Program, all watercourses which have been identified by a local floodplain administrator as having significant potential flood hazards, and all watercourses with drainage areas more than 1/4 square mile or a 100-year discharge estimate of more than 500 cubic feet per second. Application of these guidelines will not be necessary if the local community or county has in effect a drainage, grading or stormwater ordinance which, in the opinion of the Department, results in the same or greater level of flood protection as application of these guidelines would ensure.

This requirement is effective June 1, 1998. Copies of this State Standard and State Standard Attachment 7-98 can be obtained by contacting the Department's Flood Mitigation Section at (602) 417-2445.

NOTICE

This document is available in alternative formats. Contact the Department of Water Resources, Flood Mitigation Section at (602) 417-2445 or (602) 417-2455 (TDD).

ARIZONA DEPARTMENT OF WATER RESOURCES
FLOOD MITIGATION SECTION

Watercourse Bank Stabilization

500 North Third Street
Phoenix, Arizona 85004

(602) 417-2445

STATE STANDARD ATTACHMENT
SSA 7-98

MAY 1998

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INTRODUCTION

Project Background

The guidelines which follow were developed based on research of available resources and references on the current state of the practice of streambank stabilization within Arizona and the surrounding region as a part of Phase I of this project. Based on the results of the Phase I effort, an evaluation of the following selected methods was performed as a part of Phase II of this project:

- Rock Riprap
- Gabions/Wire-Tied Rock
- Concrete/Shotcrete
- Grouted Rock
- Vegetation/Bio-Mechanical

The guidelines contained herein are a result of the above described evaluation process. Details of the Phase I research and Phase II evaluation are documented in the final reports for each of the respective phases which are available through the Arizona Department of Water Resources.

General

Streambank stabilization is a complex subject. There are no simple approaches which are guaranteed to work under all the possible combinations of stream conditions which exist within Arizona. However, past experience has shown that there is a need to identify procedures which can be utilized for the design of streambank stabilization projects which range from the very simple to the very complex. Simple procedures are needed to provide economical designs for relatively inexpensive streambank stabilization by individual property owners, while identification of acceptable detailed design procedures is needed to provide direction to community government agencies regarding acceptable procedures for larger scale, more complex private-sector and public-sector projects. Utilizing the three-level approach common to most state standards, a series of procedures has been developed herein to provide guidance in the design of streambank stabilization which spans the spectrum from simple to complex designs. Every attempt has been made to develop simple and conservative design procedures for the most basic stabilization methods while, at the same time, providing direction on design procedures for larger and more complex projects.

Prior to developing a design for bank stabilization, the party interested in pursuing the streambank stabilization option should thoroughly review the document titled "Streambank Protection Guidelines for Landowners and Local Governments," Malcolm P. Keown, U.S. Army Corps of Engineers, 1983. That document is an excellent overview of the nature of natural stream systems, the causes of streambank erosion and failure, and possible approaches to stabilization of streambanks. It should be noted that the spectrum of possible streambank stabilization methods is extremely wide, ranging from the most common rock-riprap protection to intricate networks of training devices designed to slow stream flow and induce sediment deposition to reclaim lost streamside lands. After reviewing the referenced document, the party interested in pursuing a streambank stabilization project may well conclude that none of the methods or procedures outlined herein are appropriate for their situation, or that streambank stabilization is not a reasonable or viable option at all for their situation. Upon investigating the time and expense involved in pursuing bank stabilization, many parties may find that avoiding, or simply setting improvements back from, the streambank or erosion-prone area is a more economical solution to their problem.

Limitations of Procedures

In general, the lower the procedure level (e.g., Level 1 is lower than Level 3), the simpler the level of evaluation—but the more conservative the resulting design parameters—will be for a given protection method. This approach reduces the level of evaluation (normally reducing design costs), but usually overestimates the values of the design parameters (typically resulting in increased construction costs). Thus, generally speaking, the lower-level procedures result in lower design costs, while the higher-level procedures result in lower construction costs. It should also be recognized by the owner/builder of the bank-stabilization project that the design of projects utilizing Level 3 procedures will be based upon data more specific to the project site, and which are therefore more likely to yield designs with the highest probability of success in providing long-term protection.

It should also be noted by the user that these procedures are intended primarily for use in areas not mapped by the Federal Emergency Management Agency (FEMA) as Special Flood Hazard Areas (SFHA) on the community's Flood Insurance Rate Maps (FIRMs). *None* of the procedures described herein will necessarily result in designs which satisfy FEMA requirements, such as freeboard and levee-certification criteria. For guidance on designs intended to satisfy FEMA requirements, the reader is referred to the appropriate FEMA regulations regarding revision and/or amendment of FIRMs.

Use-Based Application of Bank Stabilization Procedures

Because of the uncertainties in developing simplified standards for bank-stabilization procedures, a decision was made to limit the applicability of the procedures based on the type of use or protection to be provided by the bank-stabilization project. By limiting the applicability in this manner, the level of confidence in the procedures can be matched to the risk associated with the particular application. The following matrix provides an index to the applicability of the various procedure levels to the types of uses for which they can be confidently applied.

<table>
<tr><th rowspan="2">Intended Purpose of Bank Stabilization Project</th><th colspan="3">Level of Analysis/Design</th></tr>
<tr><th>Level 1</th><th>Level 2</th><th>Level 3</th></tr>
<tr><td>Prevent additional loss of streambank or reclaim land lost to erosion.</td><td colspan="2">Acceptable</td><td rowspan="2">Preferred</td></tr>
<tr><td>Protect existing improvements threatened by erosion.</td><td colspan="2">Not recommended where stabilization is required by local authorities as a condition of approval for repair, expansion or other modification of the existing bank improvements. Otherwise acceptable.</td></tr>
<tr><td>Protect new improvements threatened by erosion.</td><td colspan="2">Not Recommended</td><td>Recommended</td></tr>
<tr><td>Other</td><td colspan="3">Site-specific evaluation by engineer needed to determine appropriate procedure level.</td></tr>
</table>

BANK-STABILIZATION PROCEDURES

Level 1 and Level 2: Rock-Riprap and Wire-Tied Rock Mattress Procedures

These procedures should be pursued only after the interested party has thoroughly reviewed the options and considerations for streambank stabilization as outlined in "Streambank Protection Guidelines for Landowners and Local Governments," Malcolm P. Keown, U.S. Army Corp of Engineers, 1983, and has concluded that these methods of bank stabilization are appropriate for their situation. These procedures should only be utilized where the proposed bank-stabilization project will be confined to a single property or group of properties under one ownership, or the project will be represented by all owners in the plan and application for bank stabilization. The proposed bank-stabilization should be evaluated to insure that it will not adversely impact upstream or downstream areas. The design of the project should also be evaluated to insure that the provisions of all applicable local regulations are respected. Level 1 procedures should only be used where the design discharge is less than or equal to 3,000 cubic feet per second (CFS).

Having affirmatively made the above determinations, determine whether rock riprap or wire-tied rock mattress design is to be used. Then determine which level of analysis (Level 1 vs. Level 2) is to be performed. Then utilize the combination of design procedures and typical sections provided within the following matrix, based upon the level of analysis and type of stabilization to be used:

	Bank-Stabilization Type	
Level of Analysis	**Rock Riprap**	**Wire-Tied Rock Mattress**
Level 1 (for design discharge ≤ 3,000 CFS only)	Determine the design parameters using Table 1, and then complete the "Typical Level 1/Level 2 Design Section for Rock Riprap Bank Stabilization"	Determine the design parameters using Table 2, and then complete the "Typical Level 1/Level 2 Design Section for Wire Mattress Stabilization"
Level 2	Determine the design parameters using Table 3, and then complete the "Typical Level 1/Level 2 Design Section for Rock Riprap Bank Stabilization"	Determine the design parameters using Table 4, and then complete the "Typical Level 1/Level 2 Design Section for Wire Mattress Stabilization"

The typical sections referred to in the table above are located in Appendix A of this document. Tables 1 through 4 are on the pages which follow this section.

The difference between the Level 1 and Level 2 procedures is a function of the level of analysis done to determine key design parameters. The Level 1 procedures rely upon analyses performed to Level 1 standards, utilizing other State Standards, while the Level 2 procedures rely upon the corresponding Level 2 analyses from these other State Standards. The other standards utilized by reference include SSA2-96: "Requirements for Floodplain and Floodway Delineation in

Riverine Environments," and SSA5-96, "State Standard for Watercourse System Sediment Balance." Performance of the more detailed hydrologic and floodplain analyses associated with the Level 2 procedure should, in most cases, result in a more refined determination of the various design parameters (i.e., mean stone size, scour depth, bank-height requirement, etc.) which, in turn, should result in reduced construction costs from that which would be determined utilizing the Level 1 procedures. Mixed use of different level procedures (e.g., using level 2 hydrology with level 1 median riprap stone size determination) may be employed but should be evaluated on a case by case basis at the discretion of the user and with approval of the local jurisdiction.

The resulting typical section can be applied over the reach to be protected. Great care should be taken in insuring that the filter layer and toe are constructed per the specifications on the typical section, as the success or failure of these stabilization methods is highly dependent upon the performance of these two parts of the design. As a part of the design process, the location and alignment of the project must be field staked so as to allow field inspection as a part of the engineering review called out above.

TABLE 1: LEVEL 1 ROCK RIPRAP DESIGN PARAMETERS		
Step No.	**Level 1 Riprap Bank-Stabilization Design Parameter Description (See Appendix A for Typical Section)**	**Variable Determined**
1	Compute the 100-year discharge, Q_{100}, per Level 1 procedures in SSA2-96 ("Requirements for Floodplain and Floodway Delineation in Riverine Environments")	Q_{100}
2	Compute the flood depth, Y, per Level 1 procedures in SSA2-96	Y
3	Compute the Median Riprap Stone Size, D_{50}, using Figure 1.	D_{50}
4	Compute the total scour depth, d_s, per Level 1 procedures in SSA5-96 (State Standard for Watercourse System Sediment Balance)	d_s
5	Compute the required Height of the Bank Protection, H, as follows: H (feet) = Y, if Y $\leq$ the existing bank height;[1] H (feet) = The existing bank height, if Y > existing bank height	H
6	Compute the Riprap Layer Thickness, T, as follows: T (feet) = 2 x D_{50} for hand-placed material; T (feet) = 3 x D_{50} for dumped material	T
7	Compute the Length of Top-of-Bank Key-In, L_k, as the greater of the two values determined as follows: L_k (feet) = 5 x (Y-H) L_k (feet) = 2 x T	L_k
8	Compute the Width of the Bank Stabilization Cut-off, W, as follows: W (feet) = 5 x H This is the distance which the bank stabilization should be keyed back into the existing bank at the upstream and downstream ends of the bank stabilization in order to prevent outflanking by the streamflow.	W

[1] NOTE: Due to the very conservative nature of the level 1 flood depth estimate a freeboard component is not included in this level 1 height of bank protection estimate.

TABLE 2: LEVEL 1 WIRE-TIED ROCK MATTRESS DESIGN PARAMETERS		
Step No.	**Level 1 Wire-Tied Rock Mattress Bank-Stabilization Design Parameter Description (See Appendix A for Typical Section)**	**Variable Determined**
1	Compute the 100-year discharge, Q_{100}, per Level 1 procedures in SSA2-96 ("Requirements for Floodplain and Floodway Delineation in Riverine Environments")	Q_{100}
2	Compute the flood depth, Y, per Level 1 procedures in SSA2-96	Y
3	Compute the total scour depth, d_s, per Level 1 procedures in SSA5-96 (State Standard for Watercourse System Sediment Balance)	d_s
4	Compute the required Height of Bank Protection, H, as follows: H (feet) = Y, if $Y \leq$ the existing bank height;[2] H (feet) = The existing bank height, if Y > existing bank height	H
5	Determine the Wire-Tied Rock Mattress Thickness, T, using Table 5.	T
6	Compute the Length of the Toe Apron, L_{ta}, as follows: L_{ta} (feet) = 2.24 x d_s	L_{ta}
7	Compute the Length of the Top-of-Bank Key-In, L_k, as the greater of the two values determined as follows: L_k (feet) = 5 x (Y-H) L_k (feet) = 2 x T	L_k
8	Compute the Width of Bank Stabilization Cut-off, W, as follows: W (feet) = 5 x H This is the distance which the bank stabilization should be keyed back into the existing bank at the upstream and downstream ends of the bank stabilization in order to prevent outflanking by the streamflow.	W

[2] NOTE: Due to the very conservative nature of the level 1 flood depth estimate a freeboard component is not included in this level 1 height of bank protection estimate.

TABLE 3: LEVEL 2 ROCK RIPRAP DESIGN PARAMETERS		
Step No.	**Level 2 Riprap Bank-Stabilization Design Parameter Description (See Appendix A for Typical Section)**	**Variable Determined**
1	Compute the 100-year discharge, Q_{100}, per Level 2 procedures in SSA2-96 ("Requirements for Floodplain and Floodway Delineation in Riverine Environments")	Q_{100}
2	Compute the flood depth, Y, per Level 2 procedures in SSA2-96	Y
3	Compute freeboard, FB, per "Design Manual for Engineering Analysis of Fluvial Systems," ADWR, 1985, Section 4.6.5. Eqn. 4.28a	FB
4	Compute the Median Riprap Stone Size, D_{50}, using Figure 2.	D_{50}
5	Compute the total scour depth, d_s, per Level 2 procedures in SSA5-96 (State Standard for Watercourse System Sediment Balance)	d_s
6	Compute the required Height of Bank Protection, H, as follows: H (feet) = the existing bank height, if Y+FB > existing bank height; H (feet) = Y+FB, if Y+FB ≤ existing bank height	H
7	Compute the Riprap Layer Thickness, T, as follows: T (feet) = 2 x D_{50} , for hand placed or keyed in place material; T (feet) = 3 x D_{50} , for dumped material	T
8	Compute the Length of the Top-of-Bank Key-In, L_k, as the greater of the two values determined as follows: L_k (feet) = 5 x (Y-H) L_k (feet) = 2 x T	L_k
9	Compute the Width of Bank Stabilization Cut-off, W, as follows: W (feet) = 5 x H This is the distance which the bank stabilization should be keyed back into the existing bank at the upstream and downstream ends of the bank stabilization in order to prevent outflanking by the streamflow.	W

TABLE 4: LEVEL 2 WIRE-TIED ROCK MATTRESS DESIGN PARAMETERS		
Step No.	**Level 2 Wire-Tied Rock Mattress Bank Stabilization Design Parameter Description (See Appendix A for Typical Section)**	**Variable Determined**
1	Compute the 100-year discharge, Q_{100}, per Level 2 procedures in SSA2-96 ("Requirements for Floodplain and Floodway Delineation in Riverine Environments")	Q_{100}
2	Compute the flood depth, Y, per Level 2 procedures in SSA2-96	Y
3	Compute freeboard, FB, per "Design Manual for Engineering Analysis of Fluvial Systems," ADWR, 1985, Section 4.6.5. Eqn. 4.28a	FB
4	Compute the total scour depth, d_s, per Level 2 procedures in SSA5-96 (State Standard for Watercourse System Sediment Balance)	d_s
5	Compute the required Height of Bank Protection, H, as follows: H (feet) = the existing bank height, if Y+FB > existing bank height; H (feet) = Y+FB, if Y+FB $\leq$ existing bank height	H
6	Determine the Wire-Tied Rock Mattress Thickness, T, using Table 6.	T
7	Compute the Length of the Toe Apron, L_{ta}, as follows: L_{ta} (feet) = 2.24 x d_s	L_{ta}
8	Compute the Length of the Top-of-Bank Key-In, L_k, as the greater of the two values determined as follows: L_k (feet) = 5 x (Y-H) L_k (feet) = 2 x T	L_k
9	Compute the Width of Bank Stabilization Cut-off, W, as follows: W (feet) = 5 x H This is the distance which the bank stabilization should be keyed back into the existing bank at the upstream and downstream ends of the bank stabilization in order to prevent outflanking by the streamflow	W

TABLE 5: LEVEL 1 WIRE-TIED ROCK MATTRESS THICKNESS		
MINIMUM RECOMMENDED STANDARD WIRE-TIED ROCK MATTRESS THICKNESS (FT)	APPLICABLE DISCHARGE RANGE (CFS)	
	FOR STRAIGHT REACHES	FOR CURVED REACHES
0.75	0 TO 1,250	0 TO 300
1.00	1,250 TO 2,500	300 TO 600
1.50	2,500 TO 7,000	600 TO 1,800
3.00	7,000 TO 40,000	1,800 TO 10,000

NOTE: The thickness of mattresses used as bank toe aprons should be a minimum of 12 inches.

REFERENCES: Standard Wire-Tied Rock Mattress Thicknesses from FHWA, HEC-11, 1989; Discharge Ranges based on thickness criteria from "Standards Manual for Drainage Design and Floodplain Management in Tucson, Arizona," 1989, combined with Level 1 Median Riprap Stone Size procedure (see Figure 1).

TABLE 6: LEVEL 2 WIRE-TIED ROCK MATTRESS THICKNESS					
MINIMUM RECOMMENDED STANDARD WIRE-TIED ROCK MATTRESS THICKNESS (FT)	APPLICABLE VELOCITY RANGE (FT/S)				
	FOR BEND ANGLE ≤ 18°	FOR BEND ANGLE = 25°	FOR BEND ANGLE = 35°	FOR BEND ANGLE = 45°	FOR BEND ANGLE ≥ 60°
0.75	UP TO 9	UP TO 7	UP TO 6	UP TO 5	UP TO 4
1.00	9 TO 10	7 TO 9	6 TO 7	5 TO 6	4 TO 5
1.50	10 TO 13	9 TO 11	7 TO 9	6 TO 7	5 TO 6
3.00	13 TO 18	11 TO 15	9 TO 13	7 TO 11	6 TO 9

NOTE: The thickness of mattresses used as toe aprons should be a minimum of 12 inches.

REFERENCES: Standard Wire-Tied Rock Mattress Thicknesses from FHWA, HEC-11, 1989; Velocity Ranges based on thickness criteria from "Standards Manual for Drainage Design and Floodplain Management in Tucson, Arizona," 1989, combined with Level 2 Median Riprap Stone Size procedure (see Figure 2).

FIGURE 1

Level 1 Median Riprap Stone Size (D50)

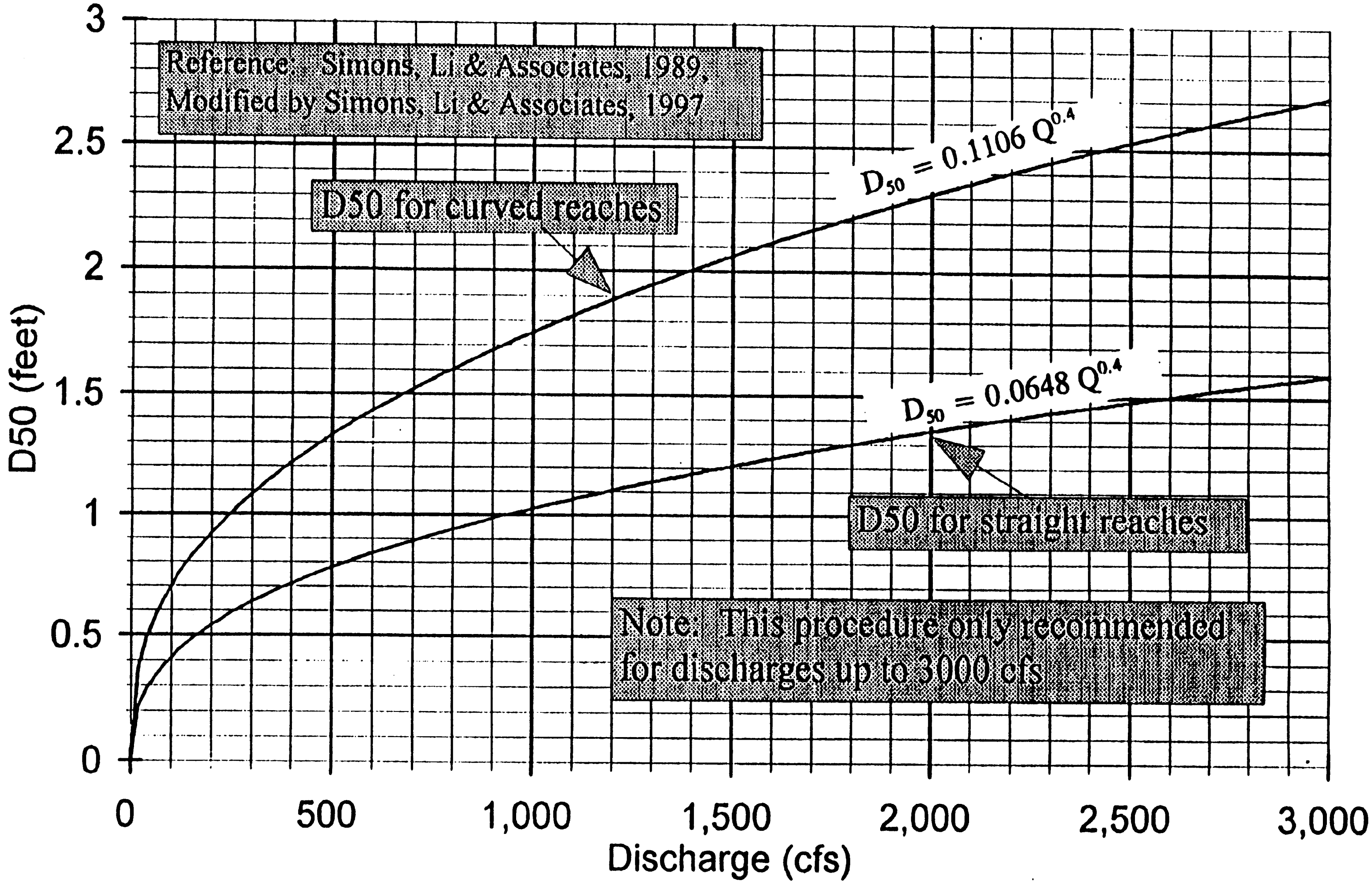
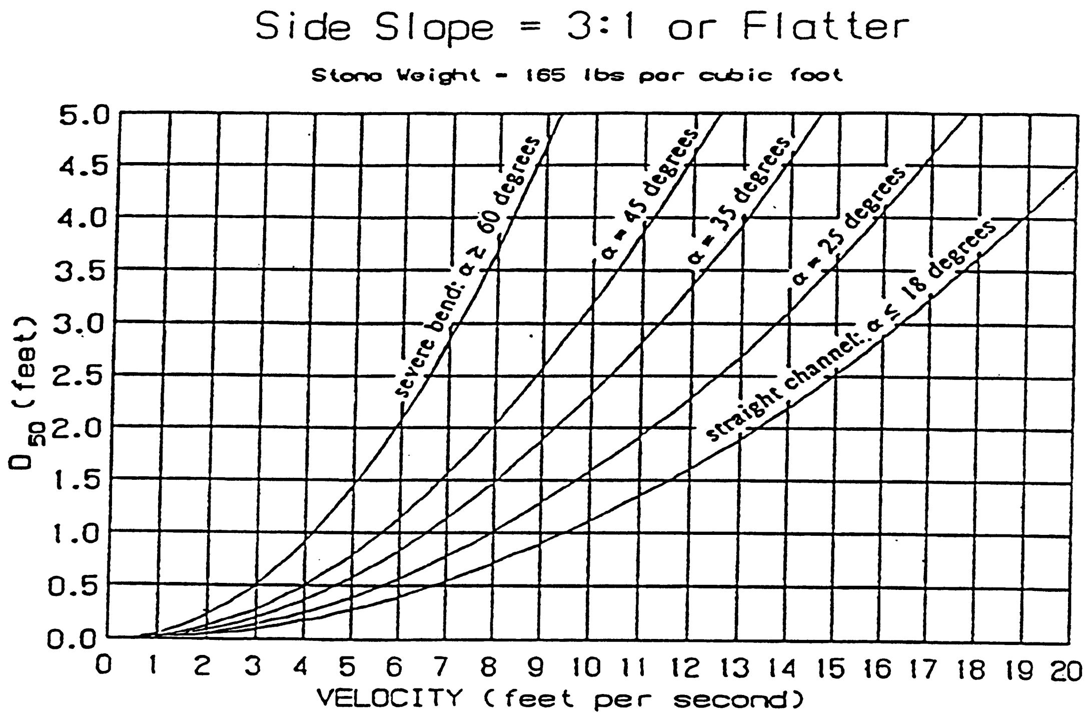
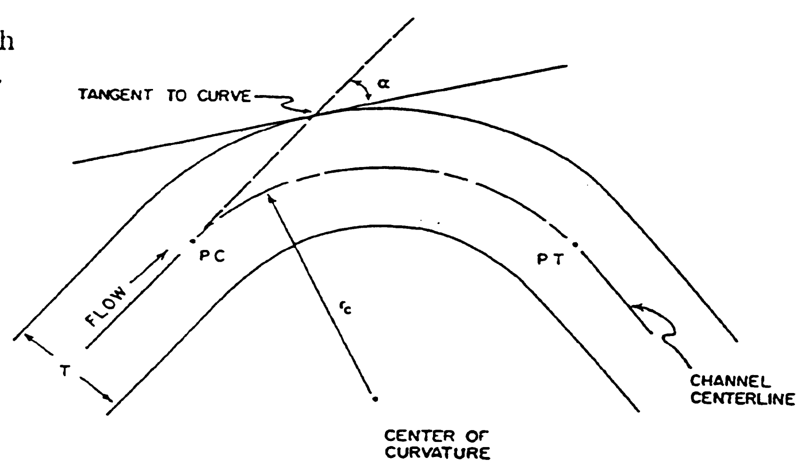


FIGURE 2
Level 2 Median Riprap Stone Size (D_{50})



NOTE: For side slope = 2:1 multiply D_{50} determined from chart by 1.14. Side slopes steeper than 2:1 should not be used.

Definition Sketch for bend angle α



PT = Downstream point of tangency to the centerline radius of curvature.
PC = Upstream point of curvature at the centerline radius of curvature.

SOURCE : SIMONS, LI & ASSOCIATES, INC. (1988)

Level 3: Applicable to All Five Selected Stabilization Methods

This level of evaluation should be utilized for all but the simplest bank-stabilization projects (i.e., all but those which can be addressed within the constraints outlined for Level 1 and Level 2 conditions). This level of evaluation involves modeling of both the hydraulic and sediment-transport characteristics of the local watercourse in order to simulate the erosion/sedimentation and channel deformation processes which are expected to occur in the area proposed for bank stabilization. For this level of analysis, Level 3 hydrologic and floodplain analysis should be performed (per SSA2-96), and Level 3 sediment-transport modeling should be performed (per SSA5-96). Analysis and design should be performed by or under the direction of a Registered Engineer with experience in the fields of surface-water hydrology, hydraulics, sediment-transport, fluvial geomorphology, and the practical applications thereto. The following references are recommended for consultation in the design of the selected bank-stabilization methods:

For general information and guidance:

- "Streambank Protection Guidelines for Landowners and Local Governments," Malcolm P. Keown, U.S. Army Corp of Engineers, 1983.
- For purposes of hydrologic and floodplain analysis, the procedures referenced in SSA2-96 and SSA5-96 should be utilized.
- For design purposes, the references listed in Table 7 (following page) should be utilized.
- Table 8 provides a list of computer programs which are based on well-established procedures referenced for use in other parts of this standard.

Example Applications

Example applications of the Level 1 and Level 2 procedures for Rock-Riprap and Wire-Tied Rock Mattress designs are contained in Appendix B of this report. Example applications of Level 3 procedures can be found in the references listed in Table 7.

TABLE 7: REFERENCES RECOMMENDED FOR LEVEL 3 BANK-STABILIZATION DESIGN					
	Bank-Stabilization Method				
Reference	Rock Riprap	Gabions/ Wire-Tied Rock	Concrete/ Shotcrete	Grouted Rock	Vegetation/ Bio-Mechanical
"Drainage Design Manual for Maricopa County, Vol. II, Hydraulics," Flood Control District of Maricopa County, 1996	●	●	●		
"Standards Manual for Drainage Design and Floodplain Management in Tucson, Arizona," City of Tucson, 1989	●	●	●		
"Hydraulic Design of Flood Control Channels," U.S. Army Corp of Engineers (USACOE), Engineering Manual EM 1110-2-1601, 1995	●				
"Design of Riprap Revetment," Federal Highway Administration (FHWA), HEC-11, 1989	●	●	●	●	
"Urban Highways, Channel Lining Design Guidelines," Arizona Department of Transportation, 1989			●		
"Streambank and Shoreline Protection," Chapter 16, Engineering Field Handbook, Natural Resources Conservation Service, 1996					●

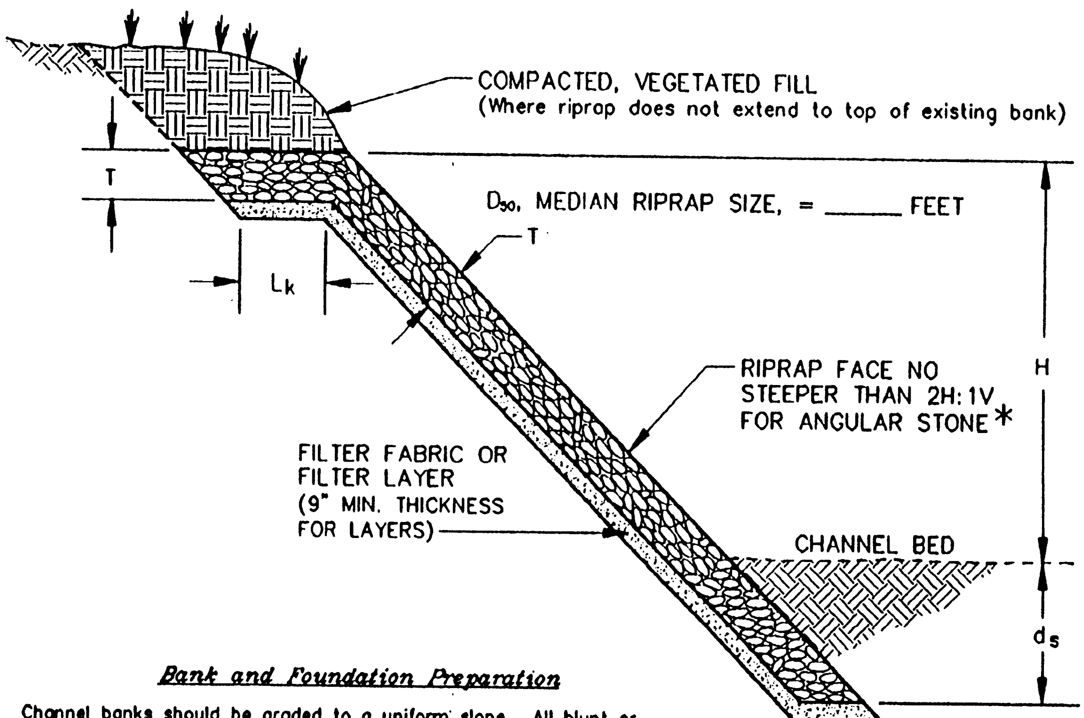
TABLE 8: RECOMMENDED COMPUTER PROGRAMS FOR LEVEL 3 BANK-STABILIZATION DESIGN					
	Bank-Stabilization Method				
Computer Program Reference	Rock Riprap	Gabions/ Wire-Tied Rock	Concrete/ Shotcrete	Grouted Rock	Vegetation/ Bio-Mechanical
HYCHL (SUBROUTINE OF HYDRAIN), FHWA, 1996[3]	●				
RIPRAP DESIGN 2.0, WEST Consultants, 1996	●				
RIPWIN, River & Stream Management Software Company, 1996	●				

[3] It is noted that HYCHL includes methods for the evaluation of other bank-stabilization methods; however, they are based on procedures from HEC-15, which are intended for application where $Q \leq 50$ cfs.

APPENDIX A

TYPICAL SECTIONS FOR LEVEL 1 / LEVEL 2 BANK STABILIZATION

TYPICAL DESIGN SECTION FOR ROCK RIPRAP BANK STABILIZATION



Design Dimensions as determined from Level 1 or Level 2 procedures.

Dimension	Value
H (feet)	
T (feet)	
d_s(feet)	
L_k(feet)	
W (feet)	

W is the distance which the bank stabilization should be keyed back into the existing bank at the upstream and downstream ends of the stabilization.

Bank and Foundation Preparation

Channel banks should be graded to a uniform slope. All blunt or sharp objects (such as rocks or tree roots) protruding from the graded surface should be removed. Large boulders near the outer edge of the toe and apron should be removed.

Riprap Gradation and Stone Shape

The gradation of rock riprap should follow a smooth curve. The ratio of the largest size rock to D_{50} should be about two, and the ratio of D_{20} to D_{50} should be about one-half. The stone should be hard, dense and durable and should be resisont to weathering and fracturing.

The shape of the riprap stone should be "blocky," rather than elongated. More nearly cubicle stones "nest" together, and are more resistant to movement. Also, stones with sharp, clean edges and relatively flat faces will form a riprap mass having an angle of internal friction greater than rounded stones, and therefore will be less susceptible to slope failures. The following shape specifications are suggested for riprap obtained from quarry operations:

* 1. The stone shall be predominantly angular in shape. Where angular stone is not available, side slopes should be no steeper than 3H:1V.
2. Not more than 25 percent of the stones reasonably distributed throughout the gradation shall have a length more than 2.5 times the breadth or thickness.
3. No stone shall have a length exceeding 3.0 times its breadth or thickness.

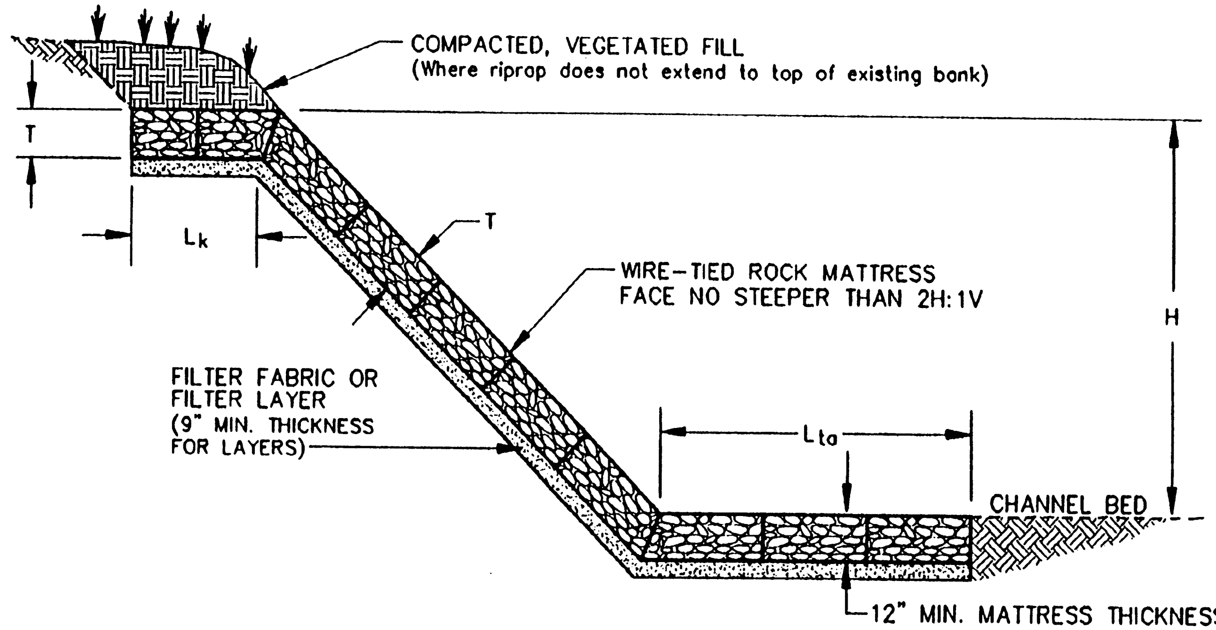
Riprap Filters

Filters are generallly required underneath rock riprap to prevent fine material from being leached out through the riprap. Two types of filter materials are commonly used: gravel filters and fabric filters. Gravel filters consist of a layer of well-graded sands and gravels. Generally, the thickness of a gravel filter should not be less than nine inches, and may vary depending upon the riprap thickness. A suggested specification for a gravel-filter gradation is as follows:

$$\frac{D_{50}\,(\text{filter})}{D_{50}\,(\text{base})} < 40 \quad \textit{and} \quad \frac{D_{15}\,(\text{filter})}{D_{85}\,(\text{base})} < 5 < \frac{D_{15}\,(\text{filter})}{D_{15}\,(\text{base})} < 40$$

Fabric filter cloths have been used beneath riprap and other revetments with good success. Although some care must be exercised in placing large rocks on the fabrics, it is generally much easier and more economical to install a fabric filter than a gravel filter. Unfortunately, a fabric filter will also preclude the growth of vegetation through the riprap. Consult fabric manufacturer for design guidance if filter fabric is used.

TYPICAL DESIGN SECTION FOR WIRE-TIED ROCK MATTRESS BANK STABILIZATION



Design Dimensions as determined from Level 1 or Level 2 procedures.

Dimension	Value
H (feet)	
T (feet)	
L_{ta} (feet)	
L_k (feet)	
W (feet)	

W is the distance which the bank stabilization should be keyed back into the existing bank at the upstream and downstream ends of the stabilization.

Bank and Foundation Preparation

Channel banks should be graded to a uniform slope. All blunt or sharp objects (such as rocks or tree roots) protruding from the graded surface should be removed. Large boulders near the outer edge of the toe and apron area should be removed.

Mattress Unit Size and Configuration

Indivual mattress units should be a size that is easily handled on site. Commercially available gabion units come in standard sizes. Manufacturers literature indicates that alternative sizes can be manufactured when required, provided that the quantities involved are of a reasonable magnitude. The mattress should be divided into compartments so that failure of one section of the mattress will not cause loss of the entire mattress. Compartmentalization also adds to the structural integrity of individual gabion units. For this reason, it is recommended that diaphragms be installed at a nominal 2 foot spacing within each of the gabion units to provide the recommended compartmentalization.

Stone Size and Quality

The maximum size of stone should not exceed the thickness of individual mattress units. The stone should be well graded within the sizes available with no stone smaller than the wire-mesh opening. Common median stone size used in mattress design range from three to six inches for mattresses less than one foot thick. For mattresses of larger thickness, rock having median size up to one foot is used. The stone should be hard, dense, and durable and should be resistant to weathering and fracturing. No stone should have a length exceeding three times its breadth or length.

Basket Fabrication

Refer to FHWA HEC-11, "Design of Riprap Revetment", and to manufacturers literature and specifications.

Riprap Filters

Filters are generally required underneath rock riprap to prevent fine material from being leached out through the riprap. Two types of filter materials are commonly used: gravel filters and fabric filters. Gravel filters consist of a layer of well-graded sands and gravels. Generally, the thickness of a gravel filter should not be less than nine inches, and may vary depending upon the riprap thickness. A suggested specification for a gravel-filter gradation is as follows:

$$\frac{D_{50}(\text{filter})}{D_{50}(\text{base})} < 40 \quad \textit{and} \quad \frac{D_{15}(\text{filter})}{D_{85}(\text{base})} < 5 < \frac{D_{15}(\text{filter})}{D_{15}(\text{base})} < 40$$

Fabric filter cloths have been used beneath riprap and other revetments with good success. Although some care must be exercised in placing large rocks on the fabrics, it is generally much easier and more economical to install a fabric filter than a gravel filter. Fabrics must be keyed in and overlapped and should preferably be of a non-woven type. Unfortunately, a fabric filter will also preclude the growth of vegetation through the riprap or, alternatively, the fabric can be damaged by vegetation. Consult fabric manufacturer for design guidance if filter fabric is used.

APPENDIX B

EXAMPLE APPLICATIONS

EXAMPLE APPLICATIONS

Problem Description: A 100 foot straight reach of a small wash near Holbrook, Arizona in Navajo County, Arizona has a contributing drainage area of 300 acres (0.47 square miles) and has been experiencing erosion along a bank which crosses a privately owned parcel. The owner of the parcel would like to protect the bank of the wash to prevent additional loss of land, loss of riparian vegetation and prevent eventual possible damage to a storage building near the bank. The height of the bank along the 100 foot reach is approximately 5 feet from the sand bed channel to the obvious point of inflection with the adjacent overbank area.

Objective: Develop a simple design for relatively inexpensive bank protection which either the owner can build or which can be built by a small contractor.

Level 1 Rock Riprap Design

The following steps follow the steps in Table 1 of this standard:

Step 1: A Level 1 100-year peak discharge (Q_{100}) of 1,800 cfs is determined using Figure D-1 from SSA 2-96 (Page D-2)

Step 2: A Level 1 flood depth (Y) of 4.7 feet is determined using the Region I-D equation on page E-1 of SSA 2-96.

Step 3: A Level 1 median riprap stone size (D_{50}) of 1.3 feet is determined from Figure 1 of this standard using the curve for straight reaches.

Step 4: A Level 1 total scour depth (d_s) of 4.9 feet is determined as the sum of 3.1 feet of general degradation plus 1.8 feet of long-term degradation using the Level 1 equations for scour from pages CDE-2 and CDE-3, respectively, of SSA 5-96.

Step 5: The required height of bank protection (H) is set equal to the computed flood depth of 4.7 feet.

Step 6: The riprap layer thickness (T) is determined as 2 x D_{50} = 2.6 feet using the first equation (for hand placed material) for this step shown in this standard.

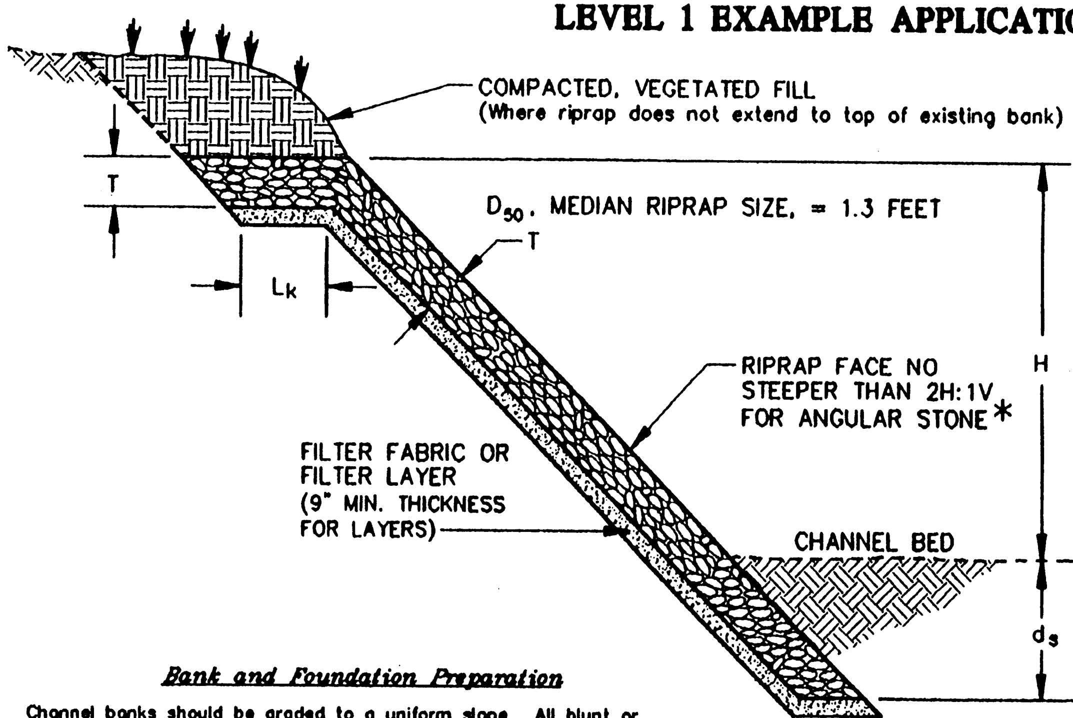
Step 7: The length of the top-of-bank key-in (L_k) is determined to be 2 x T = 5.2 feet.

Step 8: The width of the bank stabilization cut-off is determined to be 5 x H = 23.5 feet.

The typical section for rock riprap stabilization contained in Appendix A of this standard is completed by filling in the table of design parameters using the values determined in Steps 1 through 8. The resulting typical design section is attached. The typical section and supporting calculations are then submitted to the agency having jurisdiction for such activity for review and approval as required by this standard and the owner or his contractor can construct the stabilization along the threatened bank segment.

TYPICAL DESIGN SECTION FOR ROCK RIPRAP BANK STABILIZATION

LEVEL 1 EXAMPLE APPLICATION



Design Dimensions as determined from Level 1 or Level 2 procedures.

Dimension	Value
H (feet)	4.7
T (feet)	2.6
d_s(feet)	4.9
L_k(feet)	5.2
W (feet)	23.5

W is the distance which the bank stabilization should be keyed back into the existing bank at the upstream and downstream ends of the stabilization.

Bank and Foundation Preparation

Channel banks should be graded to a uniform slope. All blunt or sharp objects (such as rocks or tree roots) protruding from the graded surface should be removed. Large boulders near the outer edge of the toe and apron should be removed.

Riprap Gradation and Stone Shape

The gradation of rock riprap should follow a smooth curve. The ratio of the largest size rock to D_{50} should be about two, and the ratio of D_{20} to D_{50} should be about one-half. The stone should be hard, dense and durable and should be resisant to weathering and fracturing.

The shape of the riprap stone should be "blocky," rather than elongated. More nearly cubicle stones "nest" together, and are more resistant to movement. Also, stones with sharp, clean edges and relatively flat faces will form a riprap mass having an angle of internal friction greater than rounded stones, and therefore will be less susceptible to slope failures. The following shape specifications are suggested for riprap obtained from quarry operations:

* 1. The stone shall be predominantly angular in shape. Where angular stone is not available, side slopes should be no steeper than 3H:1V.
2. Not more than 25 percent of the stones reasonably distributed throughout the gradation shall have a length more than 2.5 times the breadth or thickness.
3. No stone shall have a length exceeding 3.0 times its breadth or thickness.

Riprap Filters

Filters are generally required underneath rock riprap to prevent fine material from being leached out through the riprap. Two types of filter materials are commonly used: gravel filters and fabric filters. Gravel filters consist of a layer of well-graded sands and gravels. Generally, the thickness of a gravel filter should not be less than nine inches, and may vary depending upon the riprap thickness. A suggested specification for a gravel-filter gradation is as follows:

$$\frac{D_{50}(\text{filter})}{D_{50}(\text{base})} < 40 \quad \textit{and} \quad \frac{D_{15}(\text{filter})}{D_{85}(\text{base})} < 5 < \frac{D_{15}(\text{filter})}{D_{15}(\text{base})} < 40$$

Fabric filter cloths have been used beneath riprap and other revetments with good success. Although some care must be exercised in placing large rocks on the fabrics, it is generally much easier and more economical to install a fabric filter than a gravel filter. Unfortunately, a fabric filter will also preclude the growth of vegetation through the riprap. Consult fabric manufacturer for design guidance if filter fabric is used.

Level 1 Wire-Tied Rock Mattress Design

The following steps follow the steps in Table 2 of this standard:

Step 1: A Level 1 100-year peak discharge (Q_{100}) of 1,800 cfs is determined using Figure D-1 from SSA 2-96 (Page D-2)

Step 2: A Level 1 flood depth (Y) of 4.7 feet is determined using the Region I-D equation on page E-1 of SSA 2-96.

Step 3: A Level 1 total scour depth (d_s) of 4.9 feet is determined as the sum of 3.1 feet of general degradation plus 1.8 feet of long-term degradation using the Level 1 equations for scour from pages CDE-2 and CDE-3, respectively, of SSA 5-96.

Step 4: The required height of bank protection (H) is set equal to the computed flood depth of 4.7 feet.

Step 5: The wire-tied rock mattress thickness (T) is determined to be 1.0 feet from Table 5 of this standard.

Step 6: The length of toe apron (L_{ta}) is determined to be 2.24 x d_s = 11.0 feet.

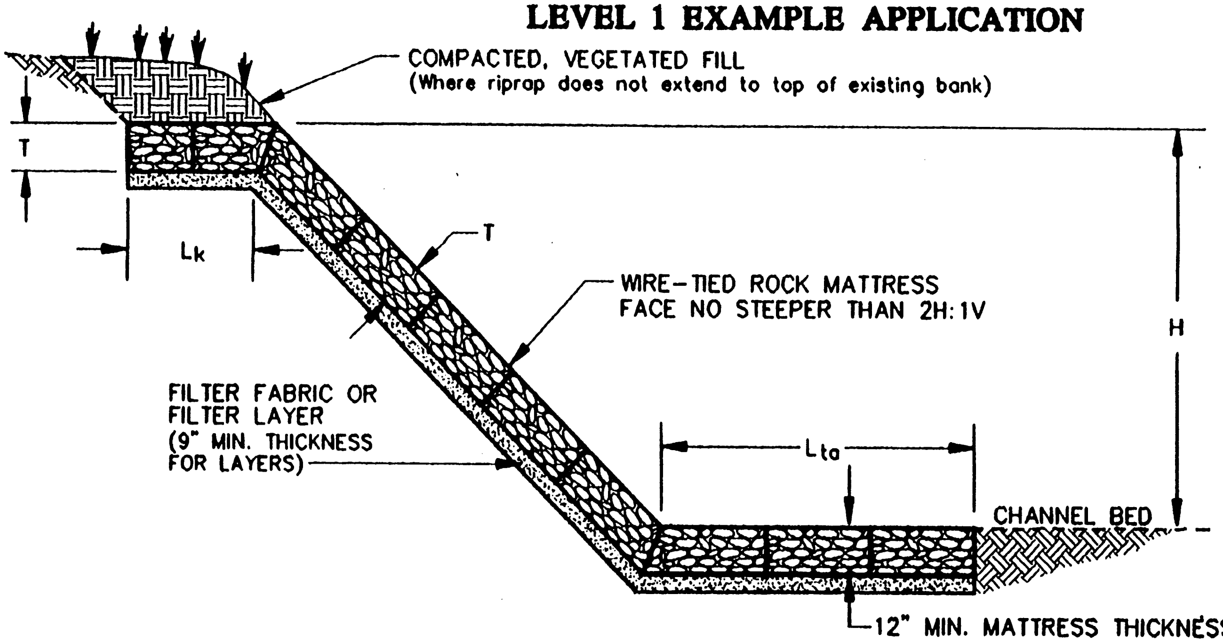
Step 7: The length of the top-of-bank key-in (L_k) is determined to be 2 x T = 2.0 feet.

Step 8: The width of the bank stabilization cut-off is determined to be 5 x H = 23.5 feet.

The typical section for rock riprap stabilization contained in Appendix A of this standard is completed by filling in the table of design parameters using the values determined in Steps 1 through 8. The resulting typical design section is attached. The typical section and supporting calculations are then submitted to the agency having jurisdiction for such activity for review and approval as required by this standard and the owner or his contractor can construct the stabilization along the threatened bank segment.

TYPICAL DESIGN SECTION FOR WIRE-TIED ROCK MATTRESS BANK STABILIZATION

LEVEL 1 EXAMPLE APPLICATION



Design Dimensions as determined from Level 1 or Level 2 procedures.

Dimension	Value
H (feet)	4.7
T (feet)	1.0
L_{ta} (feet)	11.0
L_k (feet)	2.0
W (feet)	23.5

W is the distance which the bank stabilization should be keyed back into the existing bank at the upstream and downstream ends of the stabilization.

Bank and Foundation Preparation

Channel banks should be graded to a uniform slope. All blunt or sharp objects (such as rocks or tree roots) protruding from the graded surface should be removed. Large boulders near the outer edge of the toe and apron area should be removed.

Mattress Unit Size and Configuration

Indivual mattress units should be a size that is easily handled on site. Commercially available gabion units come in standard sizes. Manufacturers literature indicates that alternative sizes can be manufactured when required, provided that the quantities involved are of a reasonable magnitude. The mattress should be divided into compartments so that failure of one section of the mattress will not cause loss of the entire mattress. Compartmentalization also adds to the structural integrity of individual gabion units. For this reason, it is recommended that diaphragms be installed at a nominal 2 foot spacing within each of the gabion units to provide the recommended compartmentalization.

Stone Size and Quality

The maximum size of stone should not exceed the thickness of individual mattress units. The stone should be well graded within the sizes available with no stone smaller than the wire-mesh opening. Common median stone size used in mattress design range from three to six inches for mattresses less than one foot thick. For mattresses of larger thickness, rock having median size up to one foot is used. The stone should be hard, dense, and durable and should be resistant to weathering and fracturing. No stone should have a length exceeding three times its breadth or length.

Basket Fabrication

Refer to FHWA HEC-11, "Design of Riprap Revetment", and to manufacturers literature and specifications.

Riprap Filters

Filters are generally required underneath rock riprap to prevent fine material from being leached out through the riprap. Two types of filter materials are commonly used: gravel filters and fabric filters. Gravel filters consist of a layer of well-graded sands and gravels. Generally, the thickness of a gravel filter should not be less than nine inches, and may vary depending upon the riprap thickness. A suggested specification for a gravel-filter gradation is as follows:

$$\frac{D_{50}(\text{filter})}{D_{50}(\text{base})} < 40 \quad \textit{and} \quad \frac{D_{15}(\text{filter})}{D_{85}(\text{base})} < 5 < \frac{D_{15}(\text{filter})}{D_{15}(\text{base})} < 40$$

Fabric filter cloths have been used beneath riprap and other revetments with good success. Although some care must be exercised in placing large rocks on the fabrics, it is generally much easier and more economical to install a fabric filter than a gravel filter. Fabrics must be keyed in and overlapped and should preferably be of a non-woven type. Unfortunately, a fabric filter will also preclude the growth of vegetation through the riprap or, alternatively, the fabric can be damaged by vegetation. Consult fabric manufacturer for design guidance if filter fabric is used.

Level 2 Rock Riprap Design

The following steps follow the steps in Table 3 of this standard:

Step 1: A Level 2 100-year peak discharge (Q_{100}) of 526 cfs is determined using Table G-4 from SSA 2-96 (Page G-3) (an annual evaporation (EV) of 55 inches from Figure G-3).

Step 2: A Level 2 flood depth (Y) of 3.0 feet by applying the normal depth procedures as outlined in SSA 2-96 (Page 7) to field surveyed cross-sections. A flow velocity (V) of 7 feet per second is also determined from the normal depth procedure.

Step 3: A Level 2 freeboard (FB) of 0.7 feet is determined from Eqn. 4.28a of the manual referenced for this step ("Engineering Analysis of Fluvial Systems", ADWR, 1985). The 0.7 foot value is based on the value of $\frac{1}{2}h_a = \frac{1}{2}(.027xV^2) = 0.7$ and values of 0 for Δy_{se} and Δy_s due to the straight nature of the subject channel reach.

Step 4: A Level 2 median riprap stone size (D_{50}) of 0.6 feet is determined from Figure 2 of this standard using the curve for straight reaches.

Step 5: A Level 1 total scour depth (d_s) of 2.8 feet is determined as the sum of 1.9 feet of general degradation plus 0.9 feet of long-term degradation by applying the Level 2 100-year peak discharge of 526 cfs to the Level 1 equations for scour from pages CDE-2 and CDE-3, respectively, of SSA 5-96. Per SSA 5-96, a minimum total scour depth of 3.0 feet should be used. Application of the Level 2 procedures for scour from SSA 5-96 pages CDE 3 - CDE 6 indicates that the channel is erosive and that armoring will not control degradation so that the Level 1 total scour depth (d_s) of 3.0 feet, determined above, should be used.

Step 6: The required height of bank protection is determined as the sum of the computed flood depth (Y) of 3.0 feet plus the 0.7 foot freeboard (FB) for a total height (H) of 3.7 feet.

Step 7: The riprap layer thickness (T) is determined as 2 x D_{50} = 1.2 feet using the first equation (for hand placed material) for this step shown in this standard.

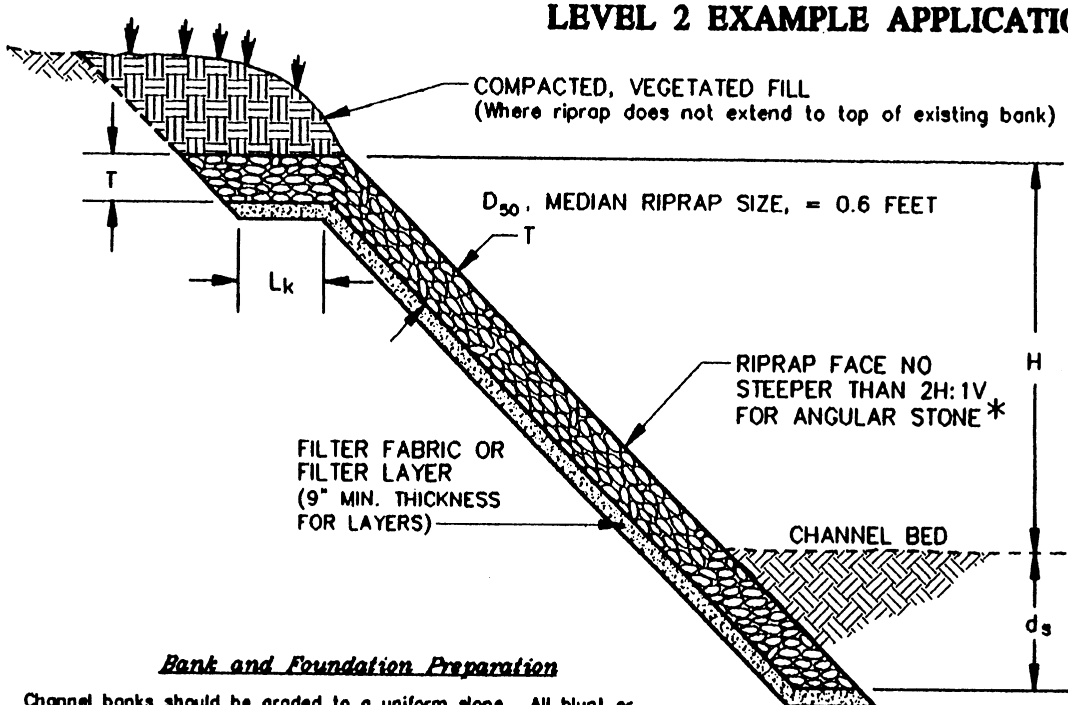
Step 8: The length of the top-of-bank key-in (L_k) is determined to be 2 x T = 2.4 feet.

Step 9: The width of the bank stabilization cut-off is determined to be 5 x H = 18.5 feet.

The typical section for rock riprap stabilization contained in Appendix A of this standard is completed by filling in the table of design parameters using the values determined in Steps 1 through 8. The resulting typical design section is attached. The typical section and supporting calculations are then submitted to the agency having jurisdiction for such activity for review and approval as required by this standard and the owner or his contractor can construct the stabilization along the threatened bank segment.

TYPICAL DESIGN SECTION FOR ROCK RIPRAP BANK STABILIZATION

LEVEL 2 EXAMPLE APPLICATION



Design Dimensions as determined from Level 1 or Level 2 procedures.

Dimension	Value
H (feet)	3.7
T (feet)	1.2
d_s(feet)	3.0
L_k(feet)	2.4
W (feet)	18.5

W is the distance which the bank stabilization should be keyed back into the existing bank at the upstream and downstream ends of the stabilization.

Bank and Foundation Preparation

Channel banks should be graded to a uniform slope. All blunt or sharp objects (such as rocks or tree roots) protruding from the graded surface should be removed. Large boulders near the outer edge of the toe and apron should be removed.

Riprap Gradation and Stone Shape

The gradation of rock riprap should follow a smooth curve. The ratio of the largest size rock to D_{50} should be about two, and the ratio of D_{20} to D_{50} should be about one-half. The stone should be hard, dense and durable and should be resisant to weathering and fracturing.

The shape of the riprap stone should be "blocky," rather than elongated. More nearly cubicle stones "nest" together, and are more resistant to movement. Also, stones with sharp, clean edges and relatively flat faces will form a riprap mass having an angle of internal friction greater than rounded stones, and therefore will be less susceptible to slope failures. The following shape specifications are suggested for riprap obtained from quarry operations:

* 1. The stone shall be predominantly angular in shape. Where angular stone is not available, side slopes should be no steeper than 3H:1V.
2. Not more than 25 percent of the stones reasonably distributed throughout the gradation shall have a length more than 2.5 times the breadth or thickness.
3. No stone shall have a length exceeding 3.0 times its breadth or thickness.

Riprap Filters

Filters are generallly required underneath rock riprap to prevent fine material from being leached out through the riprap. Two types of filter materials are commonly used: gravel filters and fabric filters. Gravel filters consist of a layer of well-graded sands and gravels. Generally, the thickness of a gravel filter should not be less than nine inches, and may vary depending upon the riprap thickness. A suggested specification for a gravel-filter gradation is as follows:

$$\frac{D_{50}\,(\text{filter})}{D_{50}\,(\text{base})} < 40 \quad \textit{and} \quad \frac{D_{15}\,(\text{filter})}{D_{85}\,(\text{base})} < 5 < \frac{D_{15}\,(\text{filter})}{D_{15}\,(\text{base})} < 40$$

Fabric filter cloths have been used beneath riprap and other revetments with good success. Although some care must be exercised in placing large rocks on the fabrics, it is generally much easier and more economical to install a fabric filter than a gravel filter. Unfortunately, a fabric filter will also preclude the growth of vegetation through the riprap. Consult fabric manufacturer for design guidance if filter fabric is used.

Level 2 Wire-Tied Rock Mattress Design

The following steps follow the steps in Table 4 of this standard:

Step 1: A Level 2 100-year peak discharge (Q_{100}) of 526 cfs is determined using Table G-4 from SSA 2-96 (Page G-3) (an annual evaporation (EV) of 55 inches from Figure G-3).

Step 2: A Level 2 flood depth (Y) of 3.0 feet by applying the normal depth procedures as outlined in SSA 2-96 (Page 7) to field surveyed cross-sections. A flow velocity (V) of 7 feet per second is also determined from the normal depth procedure.

Step 3: A Level 2 freeboard (FB) of 0.7 feet is determined from Eqn. 4.28a of the manual referenced for this step ("Engineering Analysis of Fluvial Systems", ADWR, 1985). The 0.7 foot value is based on the value of $\frac{1}{2}h_a = \frac{1}{2}(.027xV^2) = 0.7$ and values of 0 for Δy_{se} and Δy_s due to the straight nature of the subject channel reach.

Step 4: A Level 1 total scour depth (d_s) of 2.8 feet is determined as the sum of 1.9 feet of general degradation plus 0.9 feet of long-term degradation by applying the Level 2 100-year peak discharge of 526 cfs to the Level 1 equations for scour from pages CDE-2 and CDE-3, respectively, of SSA 5-96. Per SSA 5-96, a minimum total scour depth of 3.0 feet should be used. Application of the Level 2 procedures for scour from SSA 5-96 pages CDE 3 - CDE 6 indicates that the channel is erosive and that armoring will not control degradation so that the Level 1 total scour depth (d_s) of 3.0 feet, determined above, should be used.

Step 5: The required height of bank protection is determined as the sum of the computed flood depth (Y) of 3.0 feet plus the 0.7 foot freeboard (FB) for a total height (H) of 3.7 feet.

Step 6: The wire-tied rock mattress thickness (T) is determined to be 0.75 feet from Table 6 of this standard.

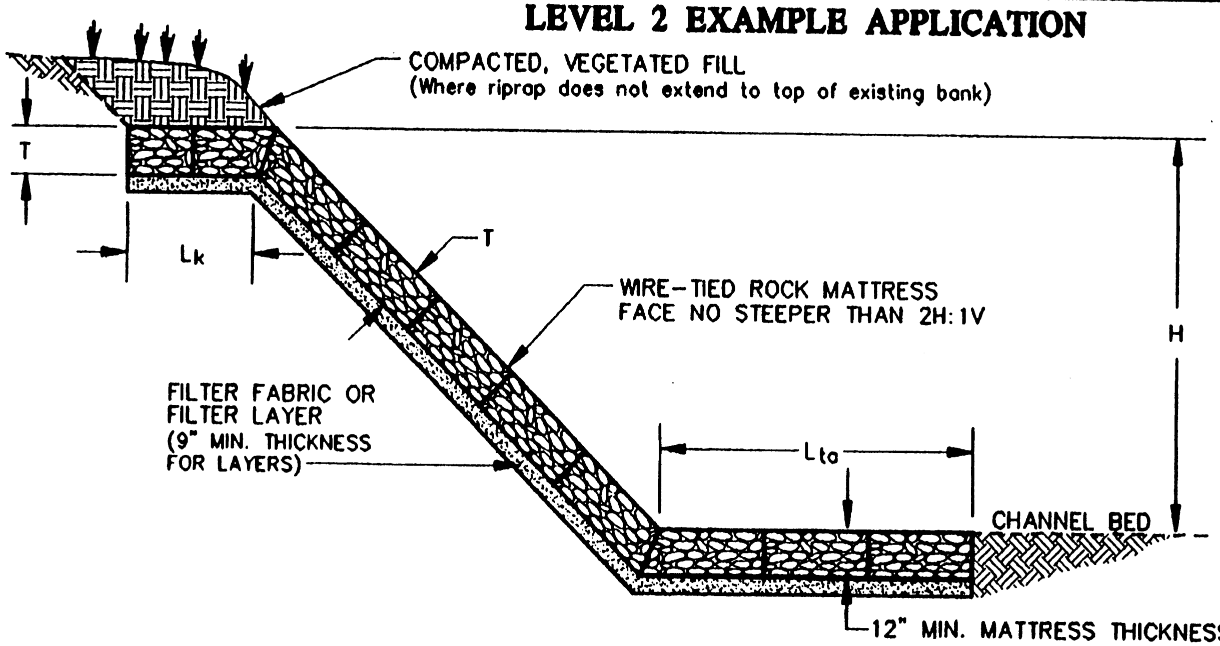
Step 7: The length of toe apron (L_{ta}) is determined to be 2.24 x d_s = 6.7 feet.

Step 8: The length of the top-of-bank key-in (L_k) is determined to be 2 x T = 1.5 feet.

Step 9: The width of the bank stabilization cut-off is determined to be 5 x H = 18.5 feet.

The typical section for rock riprap stabilization contained in Appendix A of this standard is completed by filling in the table of design parameters using the values determined in Steps 1 through 8. The resulting typical design section is attached. The typical section and supporting calculations are then submitted to the agency having jurisdiction for such activity for review and approval as required by this standard and the owner or his contractor can construct the stabilization along the threatened bank segment.

TYPICAL DESIGN SECTION FOR WIRE-TIED ROCK MATTRESS BANK STABILIZATION
LEVEL 2 EXAMPLE APPLICATION



Design Dimensions as determined from Level 1 or Level 2 procedures.

Dimension	Value
H (feet)	3.7
T (feet)	0.75
L_{ta} (feet)	6.7
L_k (feet)	1.5
W (feet)	18.5

W is the distance which the bank stabilization should be keyed back into the existing bank at the upstream and downstream ends of the stabilization.

Bank and Foundation Preparation

Channel banks should be graded to a uniform slope. All blunt or sharp objects (such as rocks or tree roots) protruding from the graded surface should be removed. Large boulders near the outer edge of the toe and apron area should be removed.

Mattress Unit Size and Configuration

Indivual mattress units should be a size that is easily handled on site. Commercially available gabion units come in standard sizes. Manufacturers literature indicates that alternative sizes can be manufactured when required, provided that the quantities involved are of a reasonable magnitude. The mattress should be divided into compartments so that failure of one section of the mattress will not cause loss of the entire mattress. Compartmentalization also adds to the structural integrity of individual gabion units. For this reason, it is recommended that diaphragms be installed at a nominal 2 foot spacing within each of the gabion units to provide the recommended compartmentalization.

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$$\frac{D_{50}(\text{filter})}{D_{50}(\text{base})} < 40 \quad \textit{and} \quad \frac{D_{15}(\text{filter})}{D_{85}(\text{base})} < 5 < \frac{D_{15}(\text{filter})}{D_{15}(\text{base})} < 40$$

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ARIZONA DEPARTMENT OF WATER RESOURCES
FLOOD MITIGATION SECTION

State Standard

for

Stormwater Detention/Retention

Under the authority outlined in ARS 48-3605(a) the Director of the Arizona Department of Water Resources establishes the following standard for Stormwater Detention/Retention in Arizona.

Local community and county flood control districts developing designs for stormwater detention/retention facilities must use either the procedures outlined in State Standard Attachment 8-99 entitled, "Stormwater Detention/Retention," or an alternative procedure accepted by the Director of the Department.

Application of these procedures will not be necessary if the local community or county has in effect a drainage, grading or stormwater ordinance which, in the opinion of the Department, provides for sufficient and effective stormwater management.

This requirement is effective September 15, 1999.

Copies of this State Standard and State Standard Attachment can be obtained by contacting the Department's Flood Mitigation Section at (602) 417-2445.

NOTICE

This document is available in alternative formats. Contact the Department of Water Resources, Flood Mitigation Section at (602) 417-2445 or (602) 417-2455 (TDD).

ARIZONA DEPARTMENT OF WATER RESOURCES
FLOOD MITIGATION SECTION

Stormwater Detention/Retention

500 North Third Street
Phoenix, Arizona 85004

(602) 417-2445

State Standard Attachment
SSA8-99

August 1999

DISCLAIMER OF LIABILITY

The Arizona Department of Water Resources is not responsible for the application of the methods outlined in this publication and accepts no liability for their use. Sound engineering judgment is recommended in all cases.

The Arizona Department of Water Resources reserves the right to modify, update, or otherwise revise this document. Questions regarding information contained in this document and/or floodplain management should be directed to the local floodplain administrator or the office below:

Flood Mitigation Section
Arizona Department of Water Resources
500 North Third Street
Phoenix, Arizona 85004

Phone: 602-417-2445
FAX: 602-417-2423

TABLE OF CONTENTS

I. INTRODUCTION

1.1 Project Background

This report has been prepared to document the results and recommended standards and procedures developed in the Assessment and Development of State Standard for Stormwater Detention/Retention in Arizona project. The purpose of the project was to conduct a literature search and assessment of the practice of stormwater detention/retention in Arizona and the southwest; identify stormwater detention/retention methods and practices; and develop guidelines based on the information gathered. The project was performed under contract to the Arizona Department of Water Resources (ADWR) and under the direction of ADWR's State Standards Work Group (SSWG). The SSWG is a volunteer group of floodplain management officials from around the state working in conjunction with ADWR to make floodplain management throughout Arizona more uniform and efficient. Everyone in Arizona benefits from these standards.

The purpose of this report is to provide general information on stormwater detention/retention and to document the recommended standards and procedures for use in Arizona.

1.2 General

Stormwater Management through Detention/Retention is a widely used tool for mitigating the effects of urbanization on flood peak discharges and runoff volume. Increased runoff associated with urbanization is a widely documented phenomenon. Generally speaking, the principle of stormwater detention/retention is to store runoff from urbanized areas and release it at rates reflecting the natural or unurbanized condition which existed before development.

Proper implementation of stormwater detention/retention depends on a number of factors including the goals of the community, physical conditions within the area where stormwater detention/retention is proposed for use and design assumptions such as rainfall duration, intensity and frequency, and hydrograph shape. Different standards and procedures may be appropriate in different localities depending on the extent of urbanization occurring and the goals of the community. With this in mind, procedures were developed which can be applied on a broad but conservative basis or, alternatively, on a more site specific and detailed basis.

Background documentation on the research and findings leading up to the recommended standards and procedures contained herein are documented in the Phase I and Phase II reports for the project which are available through ADWR.

1.3 Limitations of Procedures

Standards and procedures were developed for stormwater detention/retention for use in Arizona using the Level 1, 2 and 3 format common to other state standards. Generally speaking, the lower the procedure level the simpler the evaluation; and the more conservative the resulting design parameters. The Level 1 procedure requires the least information and associated analysis but, because of the limited investigation, yields the most conservative result relative to the general goal of stormwater detention/retention. The Level 2 procedure results in a less conservative design but requires more information and analysis than the Level 1 procedure. The Level 3 procedure is the most in-depth approach and, in the case of stormwater detention/retention, reflects a more regional approach to the problem.

Application of these guidelines will not be necessary if the local community or county has in effect a drainage, grading or stormwater ordinance which, in the opinion of the Department, provides for sufficient and effective stormwater management.

1.4 Use-Based Application of Procedures

Utilizing simplified standards and procedures involves uncertainty. Consequently, these standards recommend against applying low procedure levels to larger, more complex developments. The following matrix provides an index to the applicability of the various procedure levels to the types of uses for which they can be confidently applied.

Application[1]	**Procedure Level**		
	Level 1	**Level 2**	**Level 3**
Single Commercial Lot	Acceptable		
Small Subdivisions (<160 acres)	Acceptable	Recommended	
Large Subdivisions (>160 acres) and Planned Communities	Not recommended		Recommended

1.5 Criteria for Optional Waiver of Requirements

All requests for waivers must first refer to the local jurisdiction or agency for waiver criteria. If none exist, then the following are offered as waiver requirements. All waivers will require written approval from the local jurisdiction or governing agency prior to issuance.

[1] See waiver provisions of Section 1.5 also.

Waiver of stormwater detention/retention may be observed for:

- Single residential lots (i.e., not associated with a subdivision).
- Residential subdivisions with average lot areas $\geq$ 1 acre in area.
- Projects smaller than 160 acres which drain *directly* into a watercourse intercepting a drainage area of $\geq$ 100 square miles.

II. STORMWATER DETENTION/RETENTION PROCEDURES

2.1 Level 1

The Level 1 procedure is based on storage of the entire 1-hour, 100-year rainfall falling on the project site. The procedure for determining the required storage volume is as follows. A worksheet and design charts are contained in Appendix A.

1. Determine the area of the project site, A (acres).

2. Determine 100-year, 1-hour rainfall depth, $P_{100,1}$ (inches), by finding the 100-year, 6-hour, $P_{100,6}$, and 100-year, 24-hour, $P_{100,24}$, rainfall depths using Precipitation Maps 7 & 8, respectively, from the ADOT Hydrology Manual (1993) and using the 100-year, 1-hour Rainfall Depth Chart, all in Appendix A of this report.

3. Determine the developed condition runoff coefficient, C, for the project site using Figure 2-3 from the ADOT Hydrology Manual (1993) in Appendix A of this report. For purposes of using Figure 2-3 the following residential densities shall be assumed to apply:

   - Heavy Urban - > 4 units/acre
   - Moderate Urban - 2 – 4 units/acre

4. Determine the developed condition 100-year, 1-hour runoff volume, V_r (acre-feet) to be retained, as follows:

   $V_r = (C \times P_{100,1} \times A)/12$

Using the storage volume requirement determined above, a storage basin should be designed using the following general guidelines:

1. Design the basin to intercept site runoff, not offsite runoff. If necessary the storage can be accommodated by constructing more than one basin (e.g., to accommodate off-site drainage through the site, drainage divides through the site or grading constraints).

2. Keep basin ponding depths to three feet or less where possible.

3. Keep basin side slopes to 4:1 or flatter where possible. Basins with steeper side slopes should be properly stabilized if used.

4. Regardless of basin side slope, seeding of the basin to promote vegetation should be considered in the design to prevent rill and gully erosion.

5. Unauthorized access should be physically restricted (i.e., by fencing or other appropriate means) where basin depth is greater than three feet *and* any side slopes steeper than 4:1.

6. Provide a 6" to 8" diameter pipe outlet at the low point of the basin[2]. The pipe should be no longer than 30 feet in length to facilitate cleaning. In order to maintain a reasonable drain time, one 6" diameter pipe should be provided for every acre-foot of required storage volume (or fraction thereof) *or* one 8" diameter pipe should be provided for every two acre-feet of required storage volume (or fraction thereof). The inlet to the pipe should include a grate or riser for debris interception with a total open area as large or larger than the pipe area. The inlet should also be elevated slightly above the basin bottom ($\leq$ 6") where sedimentation is likely to occur at the inlet. The pipe should outlet to a natural/ historic point of drainage outflow. The pipe outlet should include erosion protection to prevent scour at the outlet.[3]

7. Grade the basin bottom to provide a minimum of 0.2% grade toward the pipe outlet.

8. To the extent possible, avoid sharp angular shapes (e.g., squares or rectangles) in favor of gently curving lines for the basin geometry.

9. Vehicular access should be provided to the basin either around the perimeter or into the interior of the basin to allow adequate maintenance.

10. An inspection and maintenance plan should be developed which clearly specifies the party responsible for maintenance and the frequency and method of maintenance. The plan should insure that the original storage volume of the basin is maintained, including sediment removal as needed.

11. The basin should be designed with an emergency overflow level such that ponding in excess of the design level (i.e., due to outlet clogging or extreme/successive flow events) will not cause inundation of unintended areas or improvements. The emergency overflow should act as a weir with a minimum length (in feet) equal to three times the area of the project site (in acres), A, as defined above (e.g., for A = 10 acres, the emergency overflow control weir would be 30 feet long). The emergency overflow should drain to a natural/ historic point of drainage outflow.

12. Adjacent structures should be constructed at an elevation at least two feet above the emergency overflow level described above.

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[2] Alternatively, a larger pipe (e.g., 18" or larger) can be installed and a plate with a 6" to 8" orifice, or a grated or riser-type structure with equivalent flow capacity, can be placed over the inlet. Such an approach may be advisable in areas where significant debris accumulation is possible or where frequent cleaning of the pipe may otherwise be a concern.

[3] In cases where topographic or other physical constraints preclude application of the guidelines under this item, the user should consult the local floodplain management authority for design assistance to comply with drainage requirements.

2.2 Level 2

The Level 2 procedure is based on storage of a portion of the 1-hour, 100-year rainfall falling on the project site to maintain the 100-year pre-development runoff rate from the site. The procedure also includes an adjustment to the design basin outflow rate to account for the cumulative downstream effect of urbanization. The resulting procedure should provide a conservative measure of stormwater detention in the vast majority of applications. If the user or jurisdiction is concerned that application of this procedure in a particular situation will not accomplish the intended goal of downstream peak flow reduction, the user should refer to Level 3 procedures or procedures as directed by the jurisdiction.

The procedure for determination of the required storage volume and design outflow rate is as follows and can be performed using the worksheet and charts in Appendix B[4]:

1. Determine the area of the project site, A (acres).

2. Determine 100-year, 1-hour rainfall depth, $P_{100,1}$ (inches), by finding the 100-year, 6-hour, $P_{100,6}$, and 100-year, 24-hour, $P_{100,24}$, rainfall depths using Precipitation Maps 7 & 8, respectively, from the ADOT Hydrology Manual (1993) and the 100-year, 1-hour Rainfall Depth Chart, all in Appendix A of this report.

3. Determine the developed condition runoff coefficient, C, for the project site using Figure 2-3 from the ADOT Hydrology Manual (1993) in Appendix A of this report. For purposes of using Figure 2-3 the following residential densities shall be assumed to apply:

   - Heavy Urban - > 4 units/acre
   - Moderate Urban - 2 – 4 units/acre

4. Determine the developed condition 100-year, 1-hour runoff volume, V_r (acre-feet) to be retained, as follows:

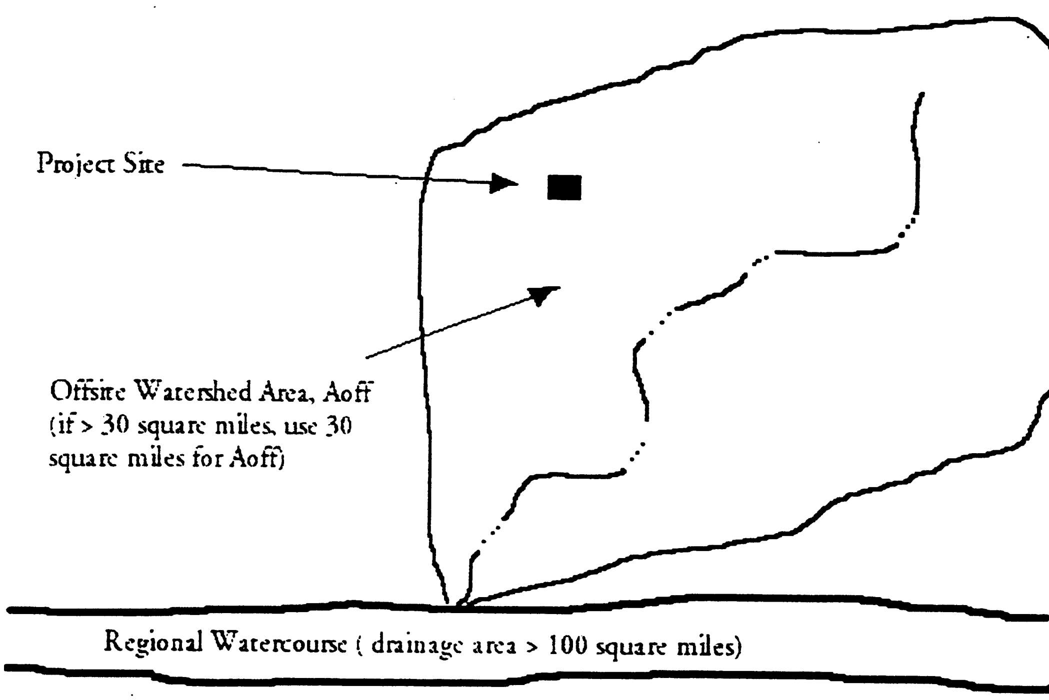
   $$V_r = (C \times P_{100,1} \times A)/12$$

5. Determine the developed condition peak discharge, Q (cfs), for the site using the 1993 ADOT Hydrology Manual rational method procedure as outlined in the Level 2 worksheet in Appendix B of this report.

6. Determine the existing condition peak discharge contribution of the project site to the "offsite" watershed peak discharge, Q_{off} (cfs), as follows:

[4] The first four steps of the Level 2 procedure are identical to the first four steps of the Level 1 procedure. For this reason, design charts for the first four steps are contained in Appendix A.

- Determine the area (in square miles) of the offsite watershed, A_{off}, which the project is located in, at the point where it empties into a regional watercourse (see definition sketch below). Where the size of the offsite watershed exceeds 30 square miles, use 30 square miles for A_{off} (i.e., A_{off} cannot be more than 30 square miles).



- Determine the unit discharge for the offsite watershed, q_{off} (cfs/sq mi), using the 100-year Unit Discharge Chart in Appendix B for the appropriate region.
- Determine Q_{off} (cfs) as follows: $Q_{off} = A \times q_{off}/640$

Note: The calculation of Q_{off} as described above is intended to result in a design outflow which limits the 100-year post-development peak outflow from the project site to a rate which reflects runoff rates associated with natural conditions on a larger watershed scale. As such this adjustment is intended for areas where urbanization is expected to occur widely throughout the regional watershed. However, if it can be documented that existing and future urbanization can not potentially affect more than a total of 10% of the offsite watershed, then Q_{off} can be calculated as the existing condition discharge from the site using the rational method procedure as outlined in the 1993 ADOT Hydrology Manual[5].

7. Divide the value of Q_{off} (i.e., the design basin outflow) by Q (the design basin inflow) to determine the value of Q_{off}/Q and find the value of V_s/V_r using Q_{off}/Q vs. V_s/V_r Chart in Appendix B of this report. Be sure to use the plot that represents the type of outlet structure intended for the detention basin design (i.e., pipe vs. weir).

8. Determine the required detention storage volume for the project site, V_s (acre-feet) as follows: $V_s = V_r \times (V_s/V_r)$ (where Vr is as determined in step 4)

9. Determine an appropriate outflow structure based on the design outflow (Q_{off}) and the maximum depth of the basin (i.e., the maximum headwater, HW)[6].

   - For pipe outflow structures, most structures can be sized using the performance charts from HEC No. 10 "Capacity Charts for the Hydraulic Design of Highway Culverts". For the convenience of the reader, Charts 11, 13, 19 and 22 of HEC-10 have been reproduced in Appendix B of this report along with select passages of text from HEC-10 explaining the use of the charts. For design types or conditions not covered by the charts included in Appendix B, the reader is referred to FHWA HEC-10, (Nov. 1972) or HEC-5 (reprinted June 1980).

   - For weir outflow structures, a simple rectangular weir should be sized/designed by solving for L (weir length) in the weir equation below knowing the other variables: $Q = CLH^{3/2}$

     where: Q = design outflow, Q_{off} (cfs)
     C = weir coefficient (use 3.1 for sharp-crest, 2.7 for broad-crest)
     L = length of the weir (ft)
     H = the head on the weir, HW as defined above, (ft)

Using the storage volume and outflow structure requirements determined above, a storage basin should be designed using the following general guidelines:

1. Design the basin to intercept site runoff, not offsite runoff. If necessary the storage can be accommodated by constructing more than one basin (e.g., to accommodate off-site drainage through the site, drainage divides through the site or grading constraints).

2. Keep basin ponding depths to three feet or less where possible.

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[5] An example of such a situation would be development of a small in-holding in a national forest.

[6] In most instances, a pipe outflow structure will most likely provide the most cost-effective design. If a weir outflow structure is used, the weir crest should be set at the basin low-point to provide a design consistent with the assumptions in the detention volume sizing procedure and to facilitate complete drainage of the pond.

3. Keep basin side slopes to 4:1 or flatter where possible. Basins with steeper side slopes should be properly stabilized if used.

4. Regardless of basin side slope, seeding of the basin to promote vegetation should be considered in the design to prevent rill and gully erosion.

5. Unauthorized access should be physically restricted (i.e., by fencing or other appropriate means) where basin depth is greater than three feet *or* any side slopes steeper than 4:1.

6. The basin outlet should outlet to a natural/historic point of drainage outflow. The pipe outlet should include erosion protection to prevent scour at the outlet. The outlet should be designed/located so as to preclude submergence of the outlet by tailwater.

7. Grade the basin bottom to provide a minimum of 0.2% grade toward the outlet.

8. To the extent possible, avoid sharp angular shapes (e.g., square or rectangular) in favor of gently curving lines for the basin geometry.

9. Vehicular access should be provided to the basin either around the perimeter or into the interior of the basin to allow adequate maintenance.

10. An inspection and maintenance plan should be developed which clearly specifies the party responsible for maintenance and the frequency and method of maintenance. The plan should insure that the original storage volume of the basin is maintained, including sediment removal as needed.

11. The basin should be designed with an emergency overflow level such that ponding in excess of the design level (i.e., due to outlet clogging or extreme/successive flow events) will not cause inundation of unintended areas or improvements. The emergency overflow should act as a weir with a minimum length (in feet) equal to the 100-year discharge from the site, Q, divided by 2.7 (e.g., for Q = 27 cfs, the emergency overflow control weir would be 10 feet long). The emergency overflow should drain to a natural/ historic point of drainage outflow.

12. All habitable floor elevations should be constructed at an elevation at least two feet above the emergency overflow level described above.

2.3 Level 3

Generally speaking, Level 3 procedures are used to provide the most detailed and cost effective design based on evaluation of the most detailed information available.

For purposes of applying Level 3 procedures the following design criteria should be observed:

1. Where possible, stormwater detention/retention should be implemented on a regional basis by the governing authority/district. The stormwater detention/retention program should utilize regional detention/retention based on watershed-wide assessment of the effects of urbanization and planning and development of facilities at the most effective locations to minimize those effects. Such a watershed wide assessment should include an evaluation of the cumulative effects of urbanization such that the implementation of the stormwater detention/retention program addresses both localized increases in runoff and regional effects to the extent possible. Where such a plan can be implemented, on-site stormwater detention/retention should be avoided.

2. It is recognized that the criteria and goals outlined in (1) above are not always practical or attainable for institutional, legal, financial or other reasons. Where implementation of a regional program is not possible or practical, stormwater detention should be provided to the extent necessary to insure that post-development peak discharges from a project site are no greater than pre-development peak discharge rates for the 2-, 10- and 100-year events. Use of the multiple-event criteria described above will aid in minimizing the cumulative regional effects of urbanization on downstream areas. However, even under these circumstances the local jurisdiction should be consulted as to any input they may have on design criteria to address regional effects.

   In support of the application of this criterion, the procedures contained in the following publications are recommended for use in Arizona *where jurisdictions do not already have adopted manuals or criteria*:

   - *Stormwater Detention/Retention Manual*, Pima County Department of Transportation and Flood Control District, 1987
   - *Drainage Design Manual for Maricopa County, Arizona*, Vol. II Hydraulics, Revised 1996
   - *Yavapai County Drainage Criteria Manual*, Yavapai County, 1998

2.4 Example Applications

A blank worksheet and design charts for use in application of the Level 1 procedure can be found in Appendix A of this report. Blank worksheets and design charts for use in application of the Level 2 procedure can be found in Appendix B of this report. Example applications of Level 3 procedures can be found in the references listed in Section 3.3, or within the references listed therein.

III. REFERENCES

1. *Highway Drainage Design Manual*, Arizona Department of Transportation, March 1993

2. *Preliminary Sizing of Detention Reservoirs to Reduce Peak Discharges*, McEnroe, Bruce, M., Journal of Hydraulic Engineering, ASCE, Vol. 118, No. 11, November 1992

3. *Stormwater Detention -- Downstream Effects on Peak Flow Rates*, Lakatos, David F. and Kropp, Richard H., article within "Stormwater Detention Facilities; Planning Design Operation and Maintenance", William DeGroot, Editor, ASCE, 1983

4. *Stormwater Detention/Retention Manual*, Pima County Department of Transportation and Flood Control District, 1987

5. *Drainage Design Manual for Maricopa County, Arizona*, Vol. II Hydraulics, Flood Control District of Maricopa County, Revised 1996

6. *Yavapai County Drainage Criteria Manual*, Yavapai County, 1998

7. *Capacity Charts for the Hydraulic Design of Highway Culverts*, Hydraulic Engineering Circular No. 10, Federal Highway Administration, November 1972

8. *Assessment and Development of State Standard for Stormwater Management Through Detention/Retention in Arizona, Final Report on Phase I: Literature Search and Assessment of Current Practices*, ADWR, August 1998

9. *Assessment and Development of State Standard for Stormwater Management Through Detention/Retention in Arizona, Final Report on Phase II: Literature Review and Evaluation of Methods*. ADWR, October 1998

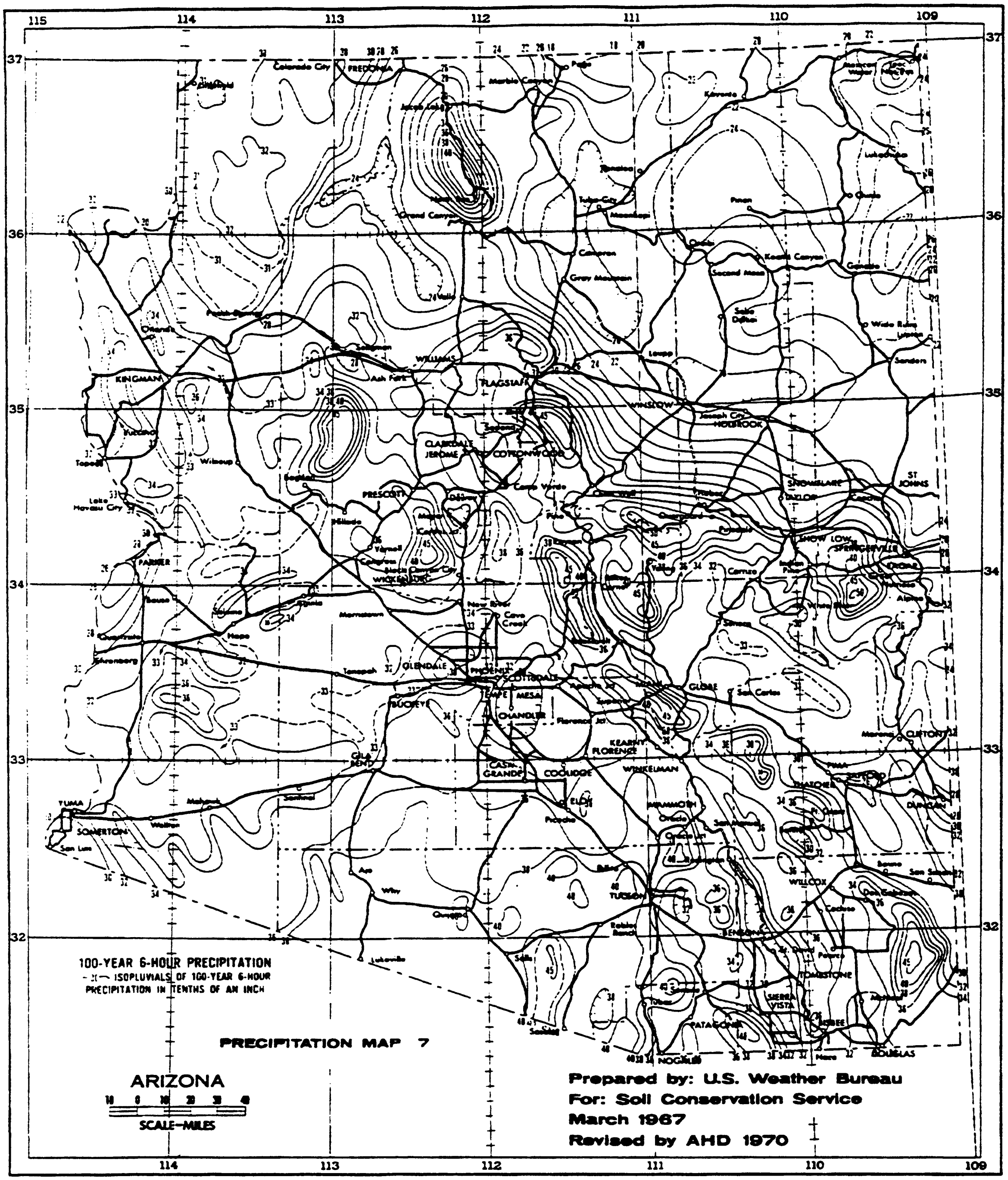
APPENDIX A

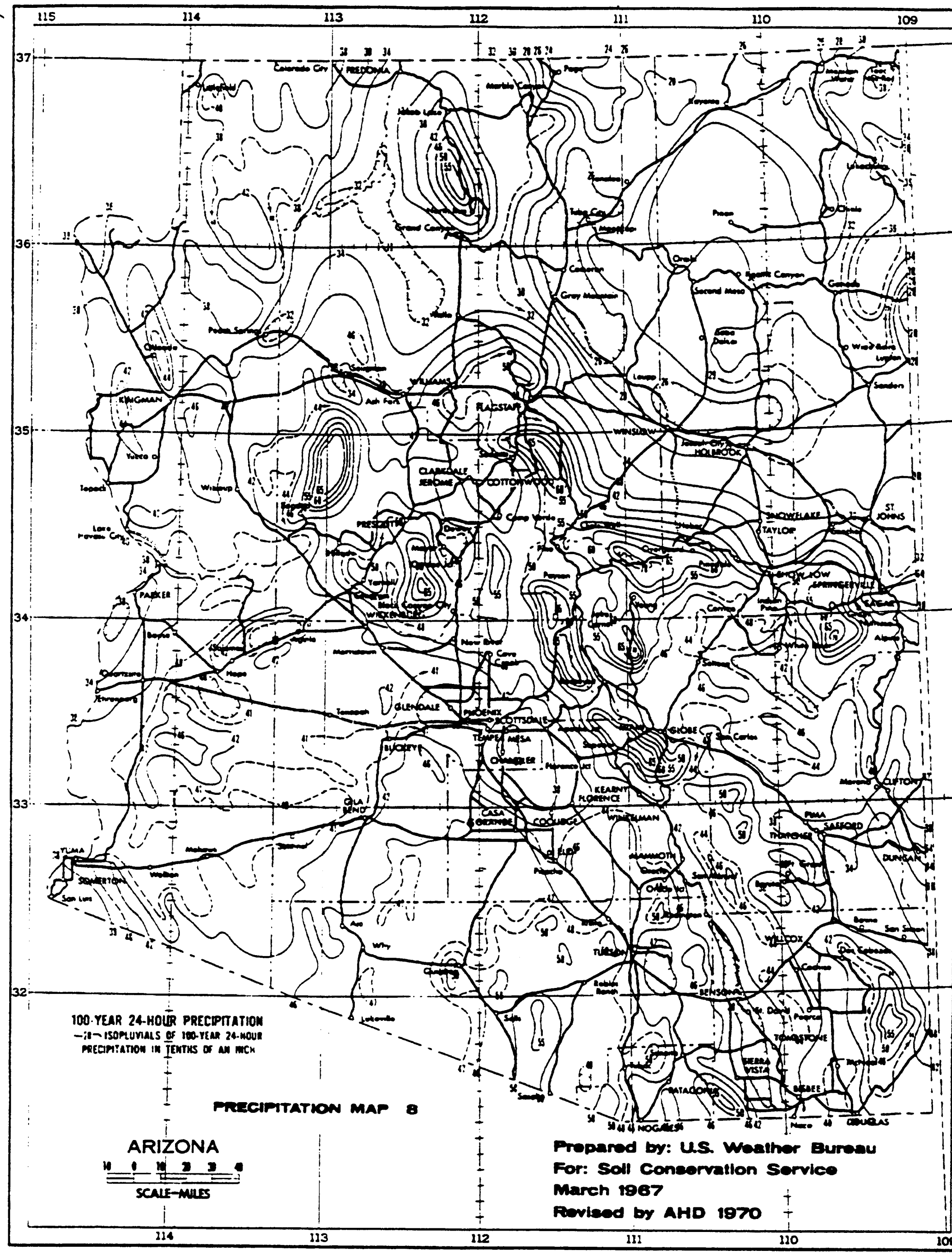
Level 1 Worksheet and Design Charts

LEVEL 1 STORMWATER DETENTION/RETENTION PROCEDURE WORKSHEET

Step	Parameter Description		Equation/ Method Determined	Value	Units
PROJECT DESCRIPTION:					
1	Project Site Area	A	From site data		Acres
2	100-year, 6-hour rainfall depth	$P_{100,6}$	From ADOT Hydrology Manual (1993), Precipitation Map No. 7		Inches
	100-year 24-hour rainfall depth	$P_{100,24}$	From ADOT Hydrology Manual (1993), Precipitation Map No. 8		Inches
	100-year, 1-hour rainfall depth	$P_{100,1}$	From $P_{100,1}$ Chart		Inches
3	Developed condition runoff coefficient for project site	C	From ADOT Hydrology Manual (1993), Figure 2-3		None
4	Developed condition 100-year, 1-hour runoff volume	V_r	$Vr = (CAP_{100,1})/12$		Acre-ft

NOTES:


100-YEAR 6-HOUR PRECIPITATION
ISOPLUVIALS OF 100-YEAR 6-HOUR PRECIPITATION IN TENTHS OF AN INCH
PRECIPITATION MAP 7
ARIZONA
SCALE—MILES
Prepared by: U.S. Weather Bureau
For: Soil Conservation Service
March 1967
Revised by AHD 1970


100-YEAR 24-HOUR PRECIPITATION
ISOPLUVIALS OF 100-YEAR 24-HOUR PRECIPITATION IN TENTHS OF AN INCH
PRECIPITATION MAP 8
ARIZONA
SCALE—MILES
Prepared by: U.S. Weather Bureau
For: Soil Conservation Service
March 1967
Revised by AHD 1970
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FLAGSTAFF
PHOENIX
TUCSON
KINGMAN
PRESCOTT
WINSLOW
HOLBROOK
GLOBE
YUMA
NOGALES
DOUGLAS
BISBEE
SAFFORD
BENSON
WILLCOX
SHOW LOW
SPRINGERVILLE
ST. JOHNS
SNOWFLAKE
TAYLOR
COTTONWOOD
JEROME
CLARKDALE
GLENDALE
SCOTTSDALE
TEMPE
MESA
CHANDLER
CASA GRANDE
COOLIDGE
FLORENCE
KEARNY
WINKELMAN
MAMMOTH
TOMBSTONE
SIERRA VISTA
PATAGONIA
CLIFTON
PIMA
THATCHER
DUNCAN
PARKER
BUCKEYE
GILA BEND
SOMERTON
WILLIAMS
FREDONIA

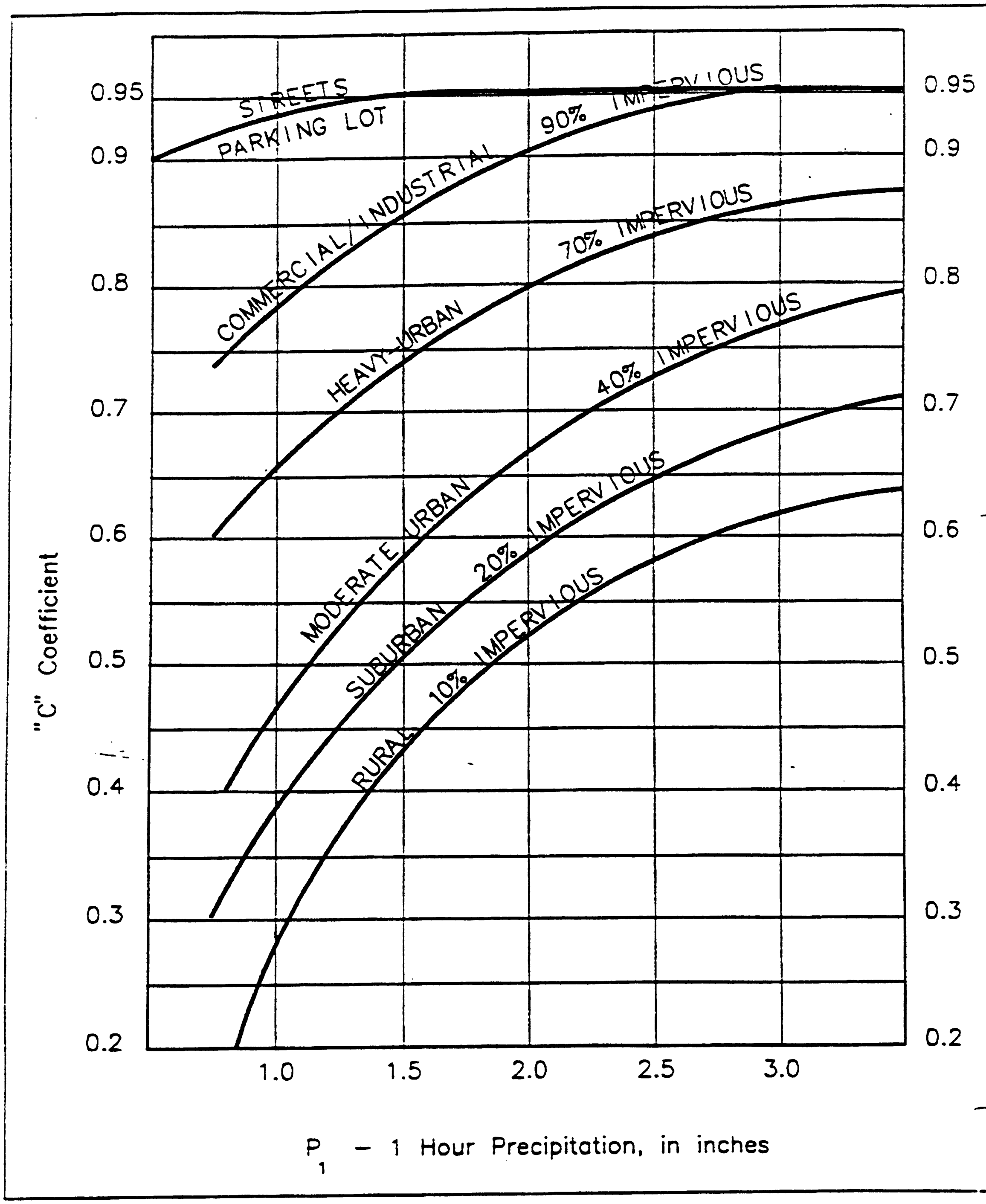
100-YEAR, 1-HOUR RAINFALL TABLE BASED ON ADOT PROCEDURE

Enter left hand column with P100,6 (to the nearest tenth of an inch) and read over to column with appropriate value of P100,24 (to the nearest half inch) and read value of P100, 1 at intersection of proper row and column

P100,6	Columns correspond to P100,24 value shown in shaded row below:										
Value	2	2.5	3	3.5	4	4.5	5	5.5	6	6.5	7
1.5	1.34	1.17	1.06	0.98	0.92	0.87	0.83	0.80	0.78	0.76	0.74
1.6	1.46	1.27	1.14	1.05	0.98	0.92	0.88	0.85	0.82	0.79	0.77
1.7	1.58	1.37	1.22	1.12	1.04	0.98	0.93	0.89	0.86	0.83	0.81
1.8	1.72	1.47	1.31	1.19	1.11	1.04	0.98	0.94	0.90	0.87	0.84
1.9	1.86	1.58	1.40	1.27	1.18	1.10	1.04	0.99	0.95	0.91	0.88
2	2.00	1.70	1.50	1.36	1.25	1.17	1.10	1.04	1.00	0.96	0.93
2.1	2.16	1.83	1.60	1.45	1.33	1.23	1.16	1.10	1.05	1.01	0.97
2.2	2.32	1.96	1.71	1.54	1.41	1.31	1.22	1.16	1.10	1.06	1.02
2.3	2.49	2.09	1.83	1.64	1.49	1.38	1.29	1.22	1.16	1.11	1.06
2.4	2.67	2.23	1.94	1.74	1.58	1.46	1.36	1.28	1.22	1.16	1.12
2.5	2.85	2.38	2.07	1.84	1.67	1.54	1.44	1.35	1.28	1.22	1.17
2.6	3.05	2.54	2.20	1.95	1.77	1.63	1.51	1.42	1.34	1.28	1.22
2.7	3.25	2.70	2.33	2.07	1.87	1.72	1.59	1.49	1.41	1.34	1.28
2.8	3.45	2.86	2.47	2.19	1.97	1.81	1.68	1.57	1.48	1.40	1.34
2.9	3.67	3.03	2.61	2.31	2.08	1.91	1.76	1.65	1.55	1.47	1.40
3	3.89	3.21	2.76	2.44	2.19	2.00	1.85	1.73	1.63	1.54	1.46
3.1	4.12	3.40	2.91	2.57	2.31	2.11	1.95	1.81	1.70	1.61	1.53
3.2	4.36	3.59	3.07	2.70	2.43	2.21	2.04	1.90	1.78	1.68	1.60
3.3	4.60	3.78	3.23	2.84	2.55	2.32	2.14	1.99	1.86	1.76	1.67
3.4	4.86	3.99	3.40	2.99	2.68	2.43	2.24	2.08	1.95	1.84	1.74
3.5	5.12	4.19	3.58	3.14	2.81	2.55	2.34	2.18	2.04	1.92	1.82
3.6	5.39	4.41	3.76	3.29	2.94	2.67	2.45	2.27	2.12	2.00	1.89
3.7	5.66	4.63	3.94	3.45	3.08	2.79	2.56	2.37	2.22	2.08	1.97
3.8	5.95	4.85	4.13	3.61	3.22	2.92	2.67	2.48	2.31	2.17	2.05
3.9	6.24	5.09	4.32	3.78	3.36	3.05	2.79	2.58	2.41	2.26	2.13
4	6.53	5.33	4.52	3.95	3.51	3.18	2.91	2.69	2.51	2.35	2.22
4.1	6.84	5.57	4.72	4.12	3.67	3.31	3.03	2.80	2.61	2.45	2.31
4.2	7.15	5.82	4.93	4.30	3.82	3.45	3.16	2.92	2.71	2.54	2.40
4.3	7.47	6.08	5.15	4.48	3.98	3.60	3.29	3.03	2.82	2.64	2.49
4.4	7.80	6.34	5.37	4.67	4.15	3.74	3.42	3.15	2.93	2.74	2.58
4.5	8.14	6.61	5.59	4.86	4.32	3.89	3.55	3.27	3.04	2.85	2.68
4.6	8.48	6.88	5.82	5.06	4.49	4.04	3.69	3.40	3.16	2.95	2.78
4.7	8.83	7.17	6.05	5.26	4.66	4.20	3.83	3.53	3.27	3.06	2.88
4.8	9.19	7.45	6.29	5.46	4.84	4.36	3.97	3.66	3.39	3.17	2.98
4.9	9.56	7.75	6.54	5.67	5.03	4.52	4.12	3.79	3.52	3.28	3.08
5	9.93	8.04	6.79	5.89	5.21	4.69	4.27	3.93	3.64	3.40	3.19

FIGURE 2-3
RATIONAL "C" COEFFICIENT
DEVELOPED WATERSHEDS

AS A FUNCTION OF RAINFALL DEPTH AND TYPE OF DEVELOPMENT



APPENDIX B

Level 2 Worksheets and Design Charts

LEVEL 2 STORMWATER DETENTION/RETENTION PROCEDURE WORKSHEET PAGE 1 OF 2

Step	Parameter Description		Equation/ Method Determined	Value	Units
PROJECT DESCRIPTION:					
1	Project Site Area	A	From site data		Acres
2	100-year, 6-hour rainfall depth	$P_{100,6}$	From ADOT Hydrology Manual (1993), Precipitation Map No. 7		Inches
	100-year 24-hour rainfall depth	$P_{100,24}$	From ADOT Hydrology Manual (1993), Precipitation Map No. 8		Inches
	100-year, 1-hour rainfall depth	$P_{100,1}$	From $P_{100,1}$ Chart		Inches
3	Developed condition runoff coefficient for project site	C	From ADOT Hydrology Manual (1993), Figure 2-3		None
4	Developed condition 100-year, 1-hour runoff volume	V_r	$Vr = (CAP_{100,1})/12$		Acre-ft
5	Length of longest flow path of site	L	ADOT Hydrology Manual (1993), page 2-4		Miles
	Watershed resistance coefficient for site	K_b	From Kb Chart contained herein		None
	Slope of longest flow path of site	S	ADOT Hydrology Manual (1993), page 2-4		Ft/mile
	Assumed Time of concentration[7]	Tc	Assumed		Hours
	Rainfall intensity	i	ADOT Hydrology Manual (1993), Figure 2-1 or 2-2		Inches/ Hour
	Calculated Time of Concentration	Tc	$Tc = 11.4\, L^{0.5} K_b^{0.52} S^{-0.31} i^{-0.38}$ (Eqn. 2-2 from ADOT Manual)		Hours
	Developed condition 100-year peak discharge for project site	Q	$Q = CiA$		Cfs
6	Area of regional watershed	A_r	Per definition in Level 2 procedure		Mi^2
	Regional runoff rate	q_r	From 100-year Unit Discharge chart for appropriate region		Cfs/ Sq mi
	Regionally adjusted existing condition discharge from project site	Q_{off}	$Q_{off} = A\, q_{off}/640$		Cfs

[7] Adjust this value until assumed value agrees with calculated value. T_c cannot fall below 10 minutes

LEVEL 2 STORMWATER DETENTION/RETENTION PROCEDURE WORKSHEET PAGE 2 OF 2

Step	Parameter Description		Equation/ Method Determined	Value	Units
7	Ratio of design outflow to design inflow	Q_{off}/Q	Q_{off}/Q		Ratio
	Ratio of required storage volume to runoff volume	V_s/V_r	From Q_{off}/Q vs. V_s/V_r Chart		Ratio
8	Required storage volume	V_s	$V_s = V_r\ (V_s/V_r)$		Acre-ft
9	Outflow structure		Use HEC-10 pipe outflow structure design charts (Appendix A of state standard) or other reference		

NOTES:

Area vs. Roughness Coefficient, Kb for Urban Areas For Use with ADOT Rational Method for Level 2 State Standard for Stormwater Detention/Retention

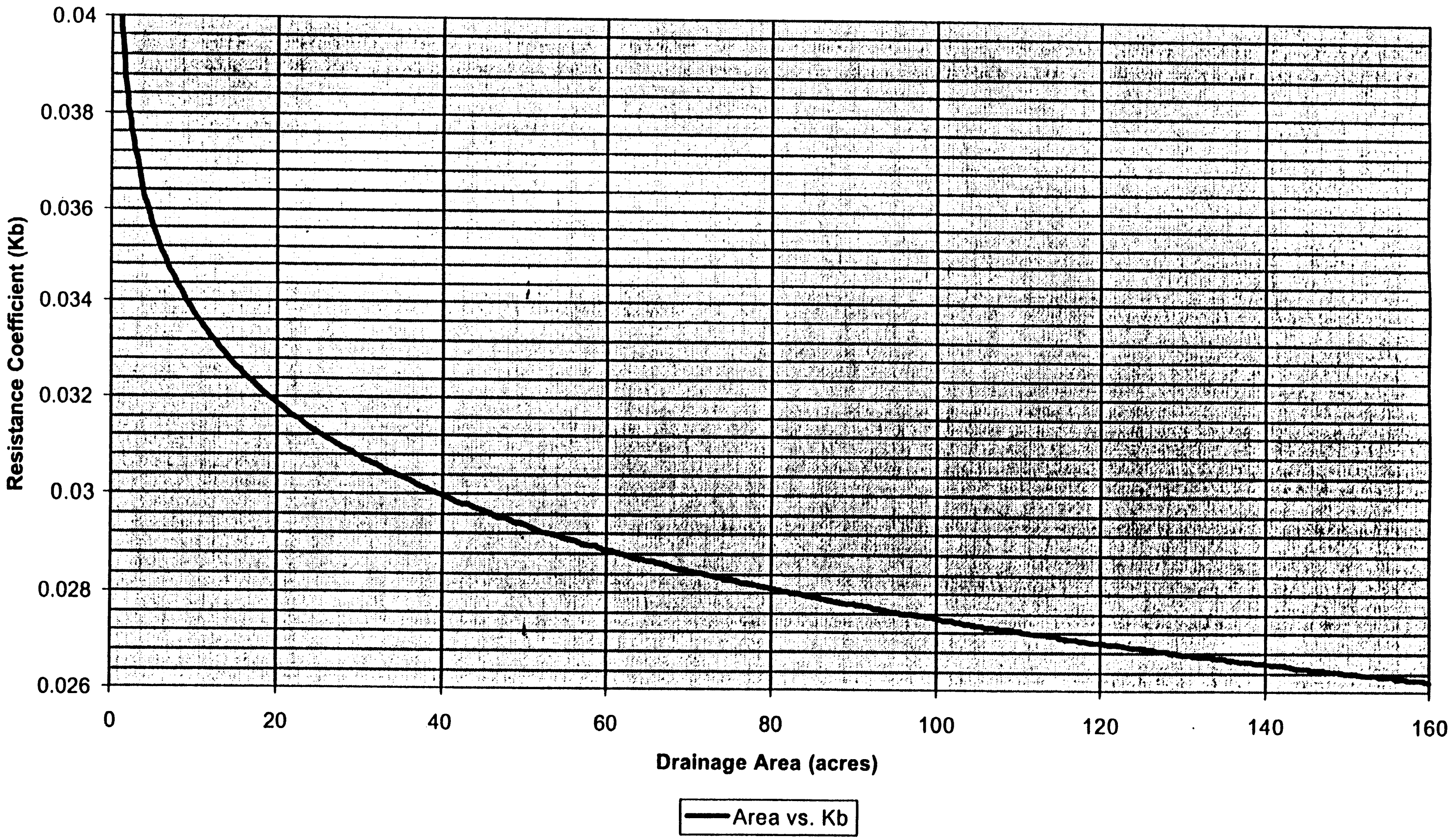


FIGURE 1-1

SHORT-DURATION RAINFALL RATIO ZONES FOR ARIZONA

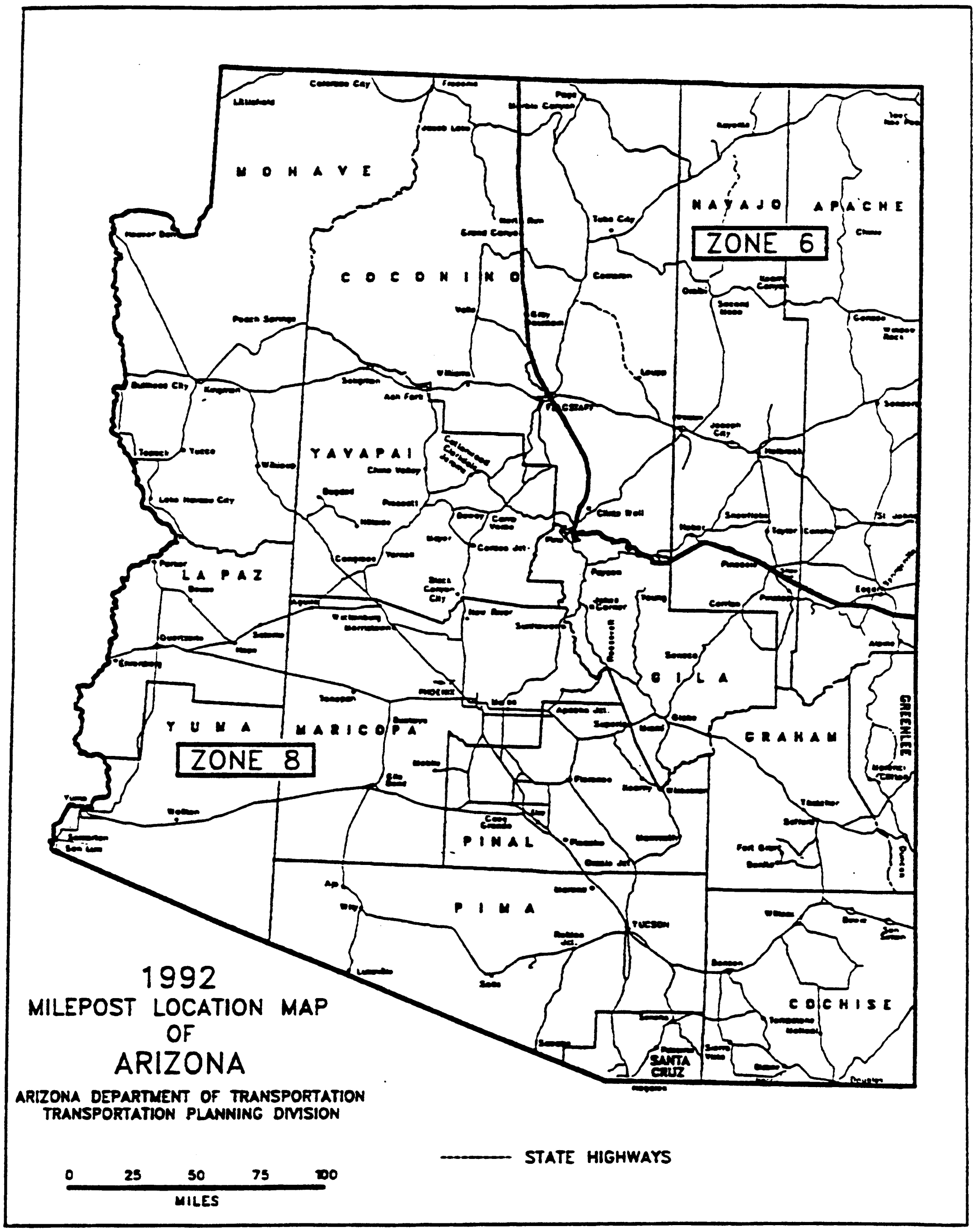


FIGURE 2-1
GENERALIZED I-D-F GRAPH FOR ZONE 6 OF ARIZONA

Example: For a selected 10-year return period, P_1 = 2.0 inches. T_C is calculated as 20 minutes. Therefore, (i) = 4.25 in/hr.

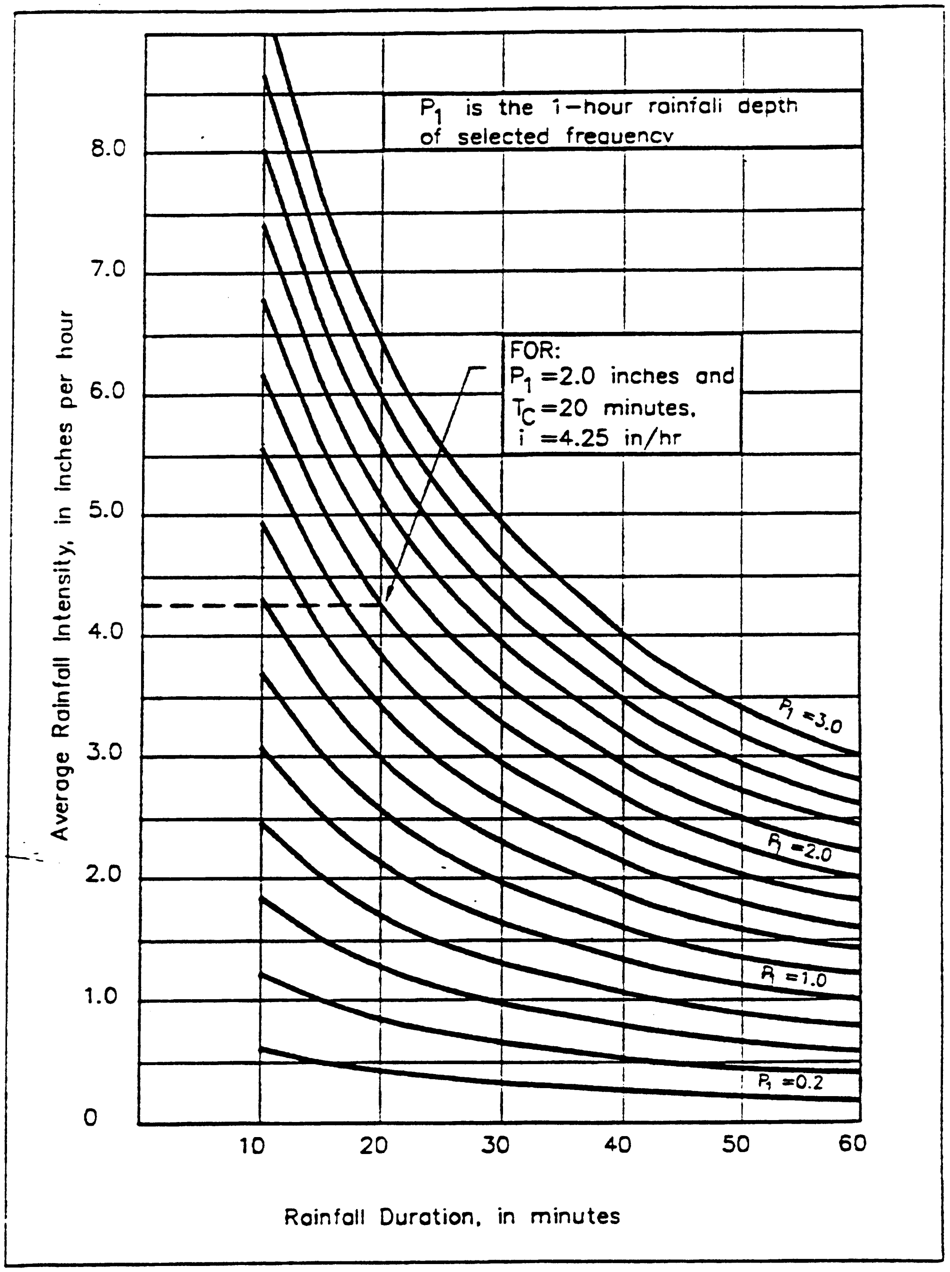
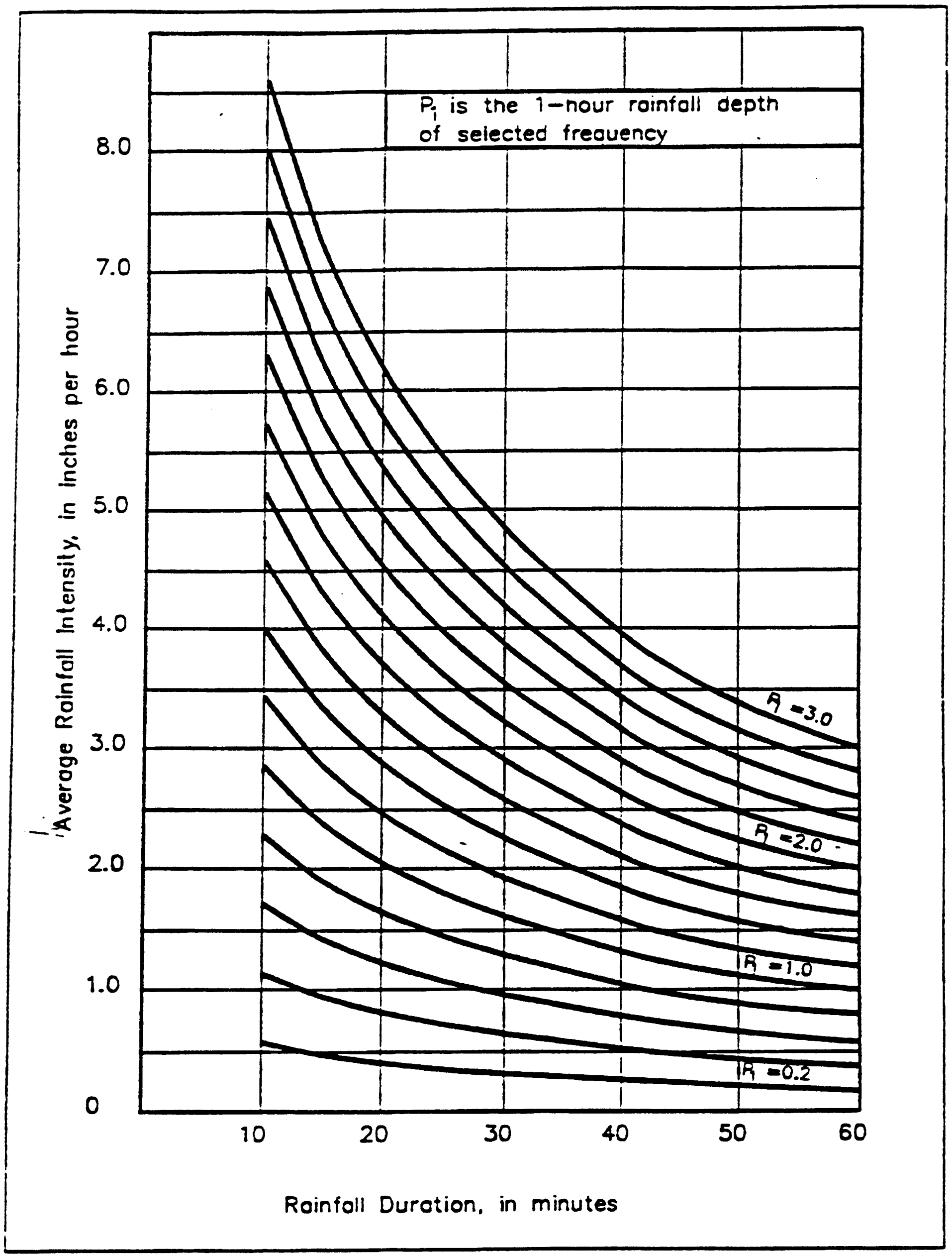


FIGURE 2-2
GENERALIZED I-D-F GRAPH FOR ZONE 8 OF ARIZONA



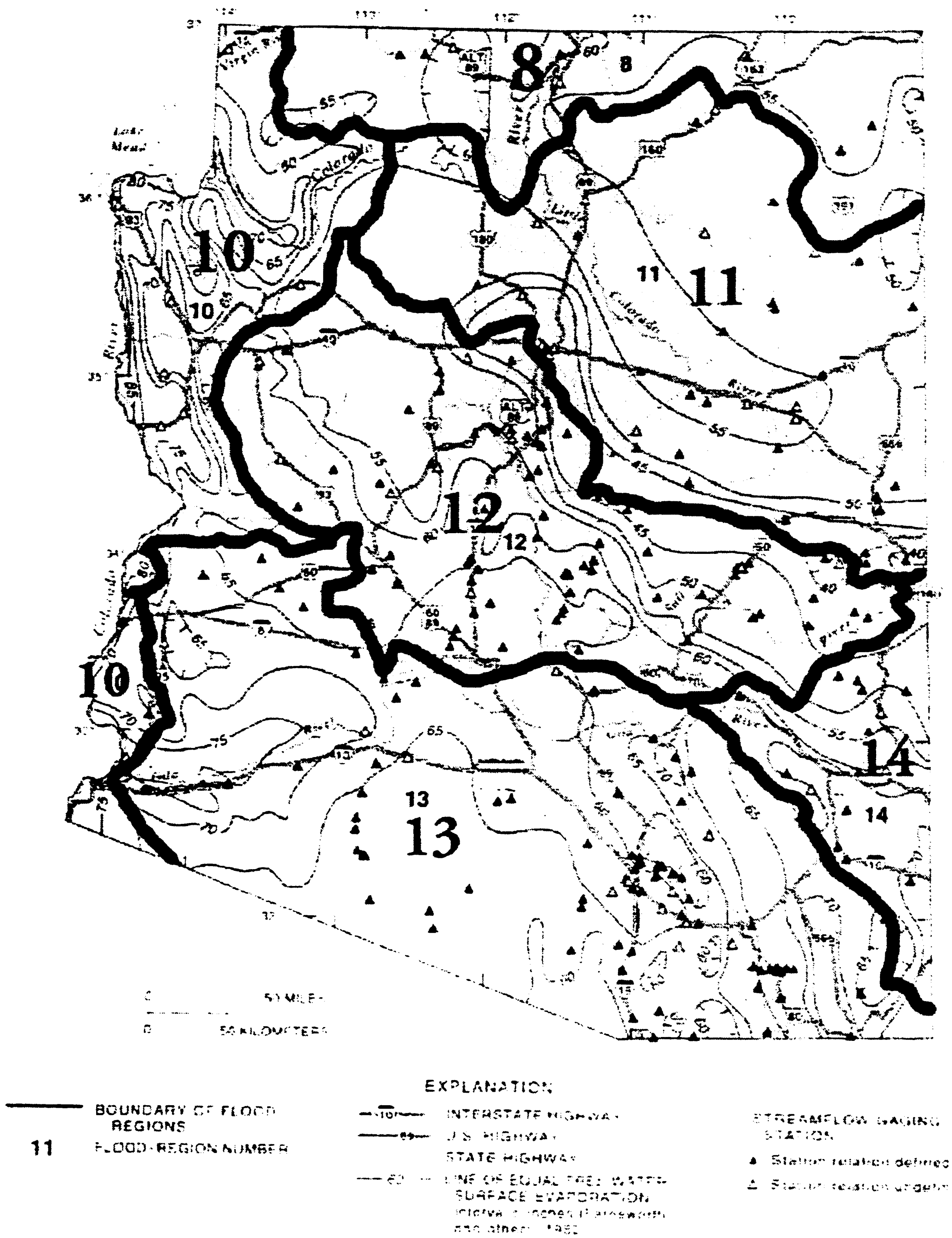
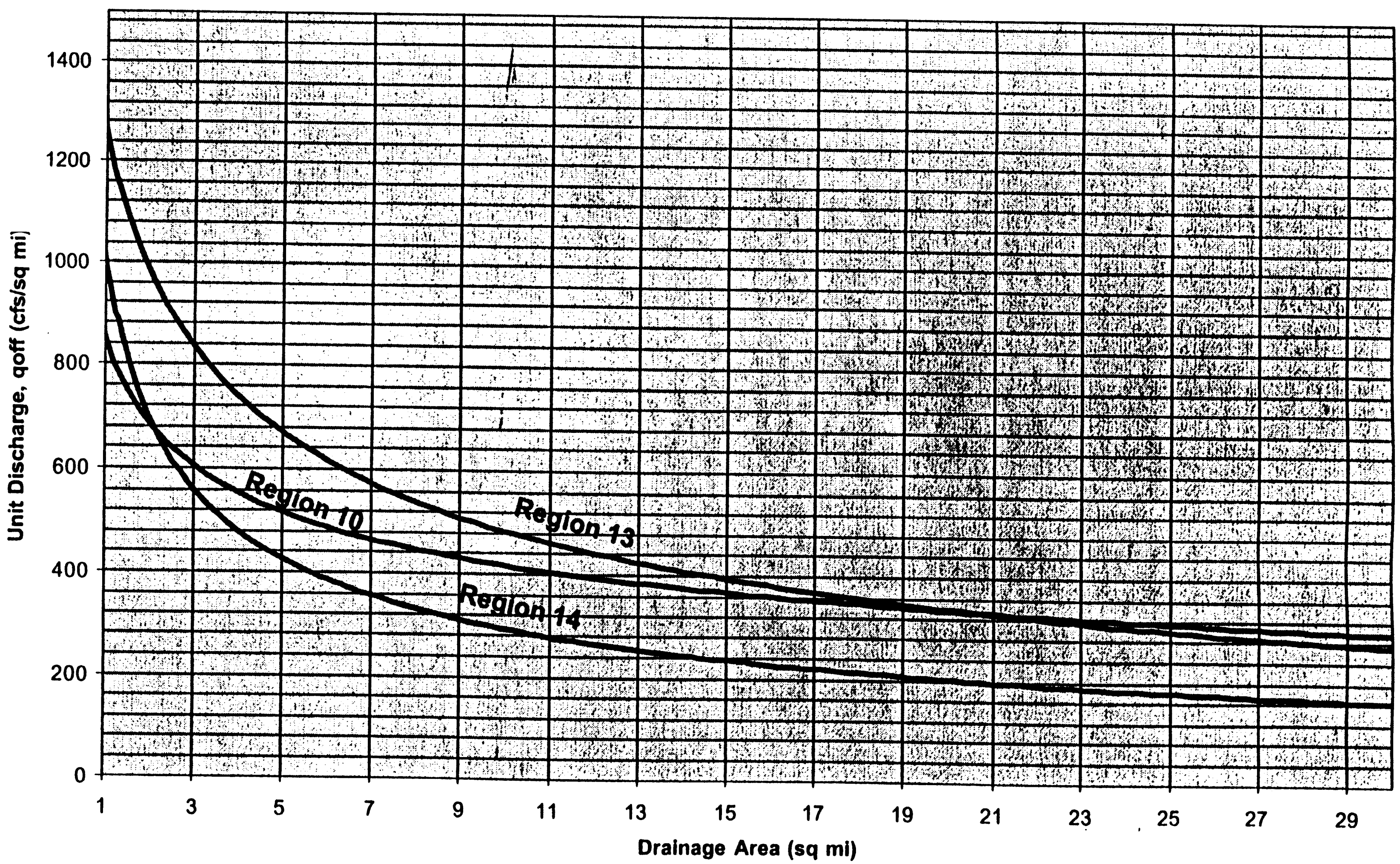
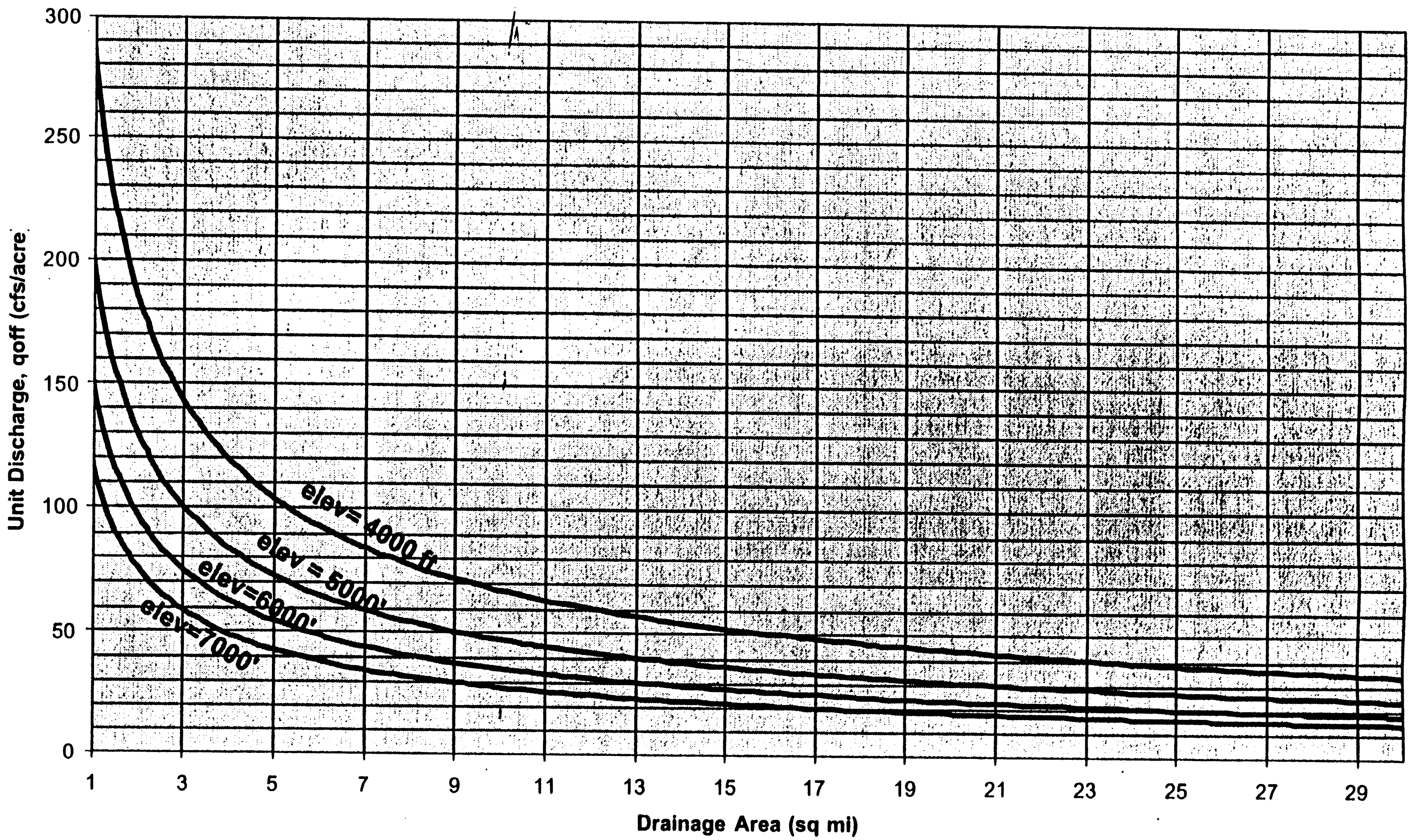


Figure 7. Flood regions in Arizona

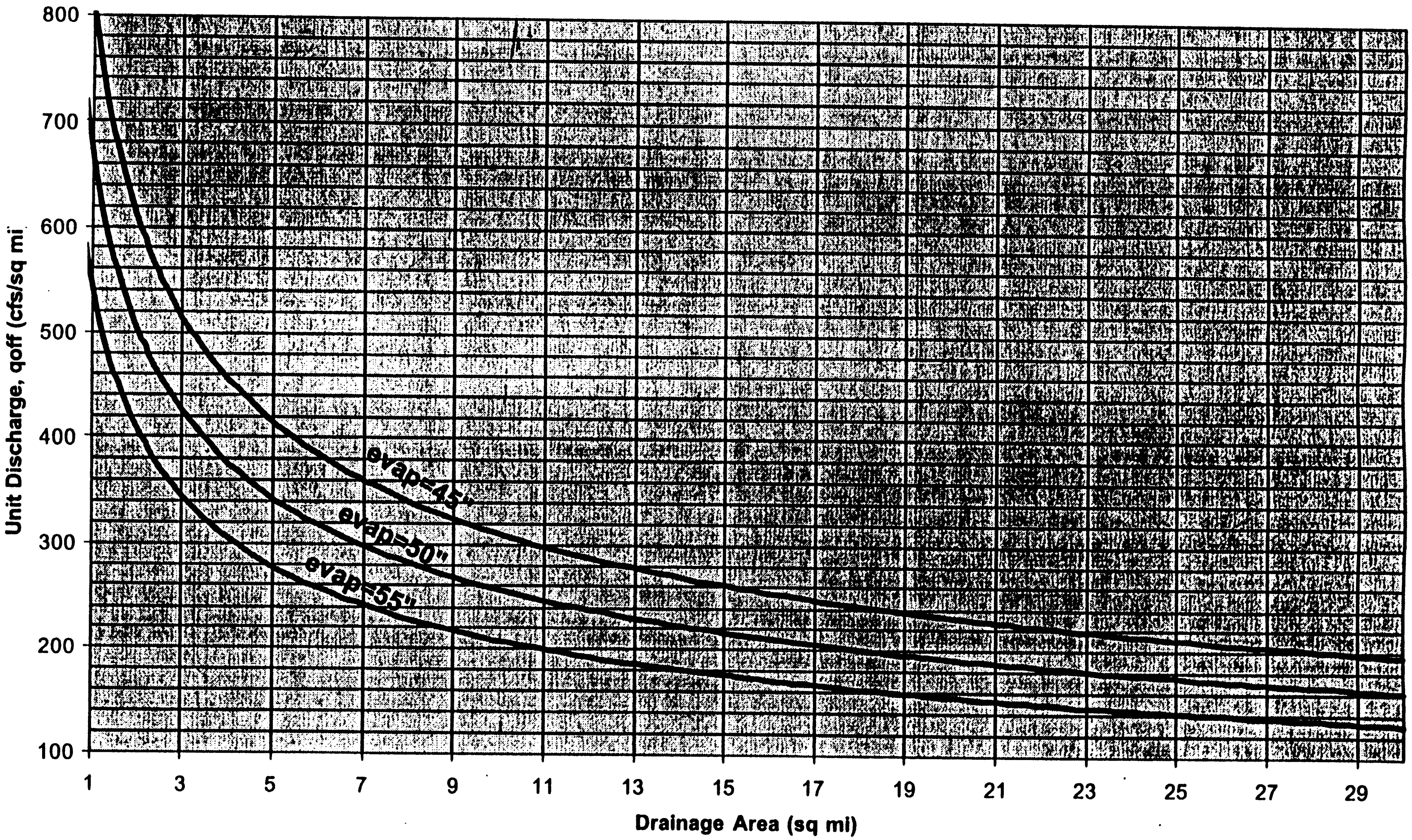
100-yr Unit Dischages for Arizona Regions 10, 13 & 14 from USGS Open File Report 93-419



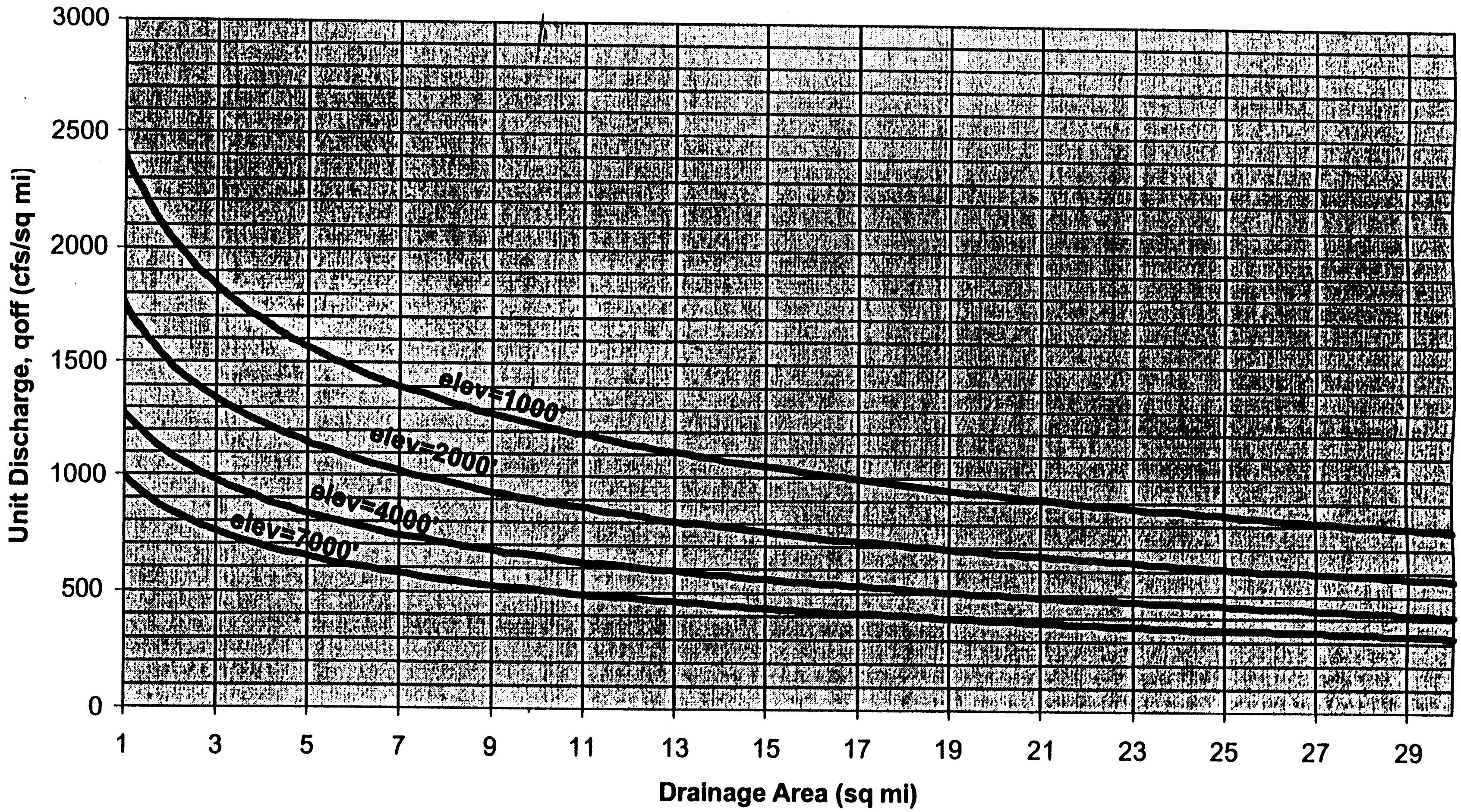
100-yr Unit Discharges for Arizona Region 8 from USGS Open File Report 93-419 for Various Values of Mean Basin Elevations (elev)



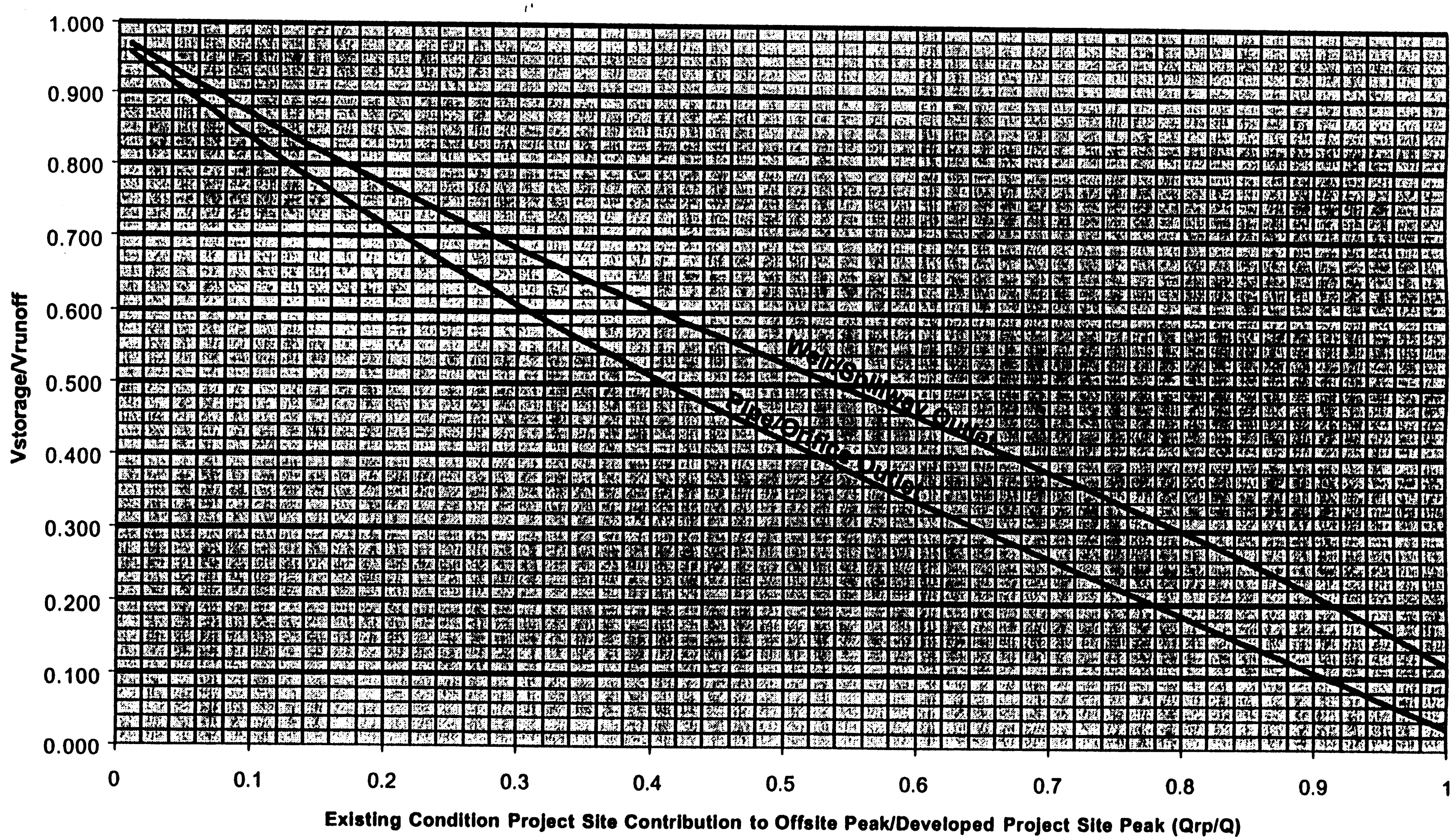
100-yr Unit Discharges for Arizona Region 11 from USGS Open File Report 93-419 for Various Values of Mean Annual Evaporation (evap)

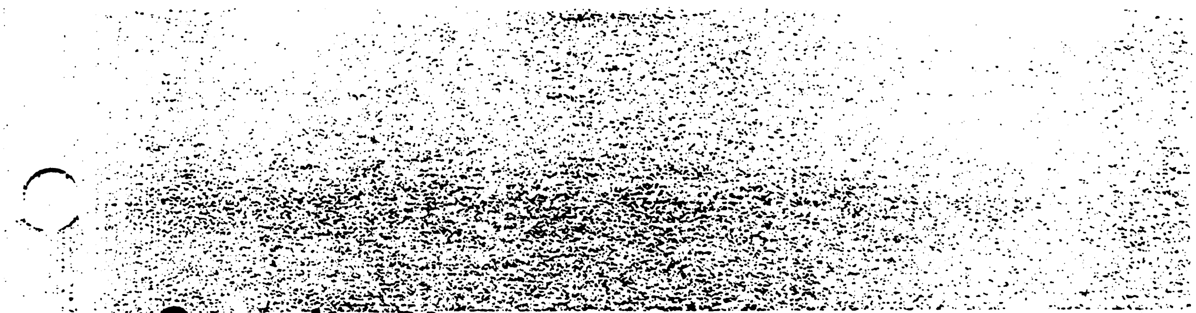


100-yr Unit Discharges for Arizona Region 12 from USGS Open File Report 93-419 for Various Values of Mean Basin Elevation (elev)



Qoff/Q vs. Vs/Vr (after McEnroe, 1992)





Capacity Charts for the Hydraulic Design of Highway Culverts

Hydraulic Engineering
Circular No. 10

November 1972

U.S. Department of Transportation
Federal Highway Administration

State Standard Attachment
SSA8-99

August 1999

III. REQUIREMENTS AND LIMITATIONS FOR USE OF CHARTS

Because culvert flow problems vary in complexity it is difficult to express headwater-discharge relationships in simple curves or charts without some limitations. The culvert capacity charts are designed to provide an easy method for the direct selection of culvert size for the majority of highway culvert installations, but the following requirements and limitations for the direct use of the charts must be observed for correct solutions.

A. Requirements and Limitations

1. The culvert type under consideration must be represented by the chart as noted in the title. (Other inlet types can be used -- see B-1 next page.)

2. The culvert size must be included on the chart.

3. The culvert invert must be on a continuous straight-line slope from inlet to outlet, and slope downward in the direction of flow (not level).

4. The $L/100S_0$ ratio must not exceed the largest value shown on the chart for the size involved.

5. The headwater depth must be less than 2D for the size considered.

6. The elevation of the tailwater in the outlet channel must not submerge critical depth at the outlet. (Critical depth for various culvert sections may be found from charts in HEC No. 5.)

VII. CULVERT CAPACITY CHARTS

The culvert capacity charts in this section provide a means for selecting a culvert of adequate size to convey the design discharge rate per barrel without exceeding an allowable depth of headwater determined by the site conditions. The allowable headwater depth AHW, and the actual headwater depth HW that results from the culvert size selected, are measured in feet above the culvert invert at the inlet. The 36 culvert capacity charts are divided into 8 groups according to eight basic types of culverts as determined by barrel shape and material. The charts appear in the order of the list shown on the last page of this circular.

Each group of charts is preceded by an explanation of the factors determining the two main inlet types represented by the charts. Information regarding other inlets types classified as equivalent to one of the two types shown in the titles and other design data necessary to the use of each group of charts are also included. Tables of dimensions and cross-sectional areas of the available sizes of each type of culvert are given in some instances.

The procedures for accumulating design data and for selecting a culvert size as previously discussed are summarized in the following steps: (This information should be tabulated on a prepared design data sheet to be used as a work sheet and a record. See tabulation sheet in HEC No. 5).

1. Select the average frequency of the design flood.

2. Determine the estimated peak discharge of the design flood.

3. Obtain all site data. Plot a roadway cross section at the culvert site and a stream channel profile. Make a contoured site plan where necessary.

4. Establish the culvert invert elevations at inlet and outlet and the culvert length. Then determine the invert slope S_o and compute $L/100S_o$.

5. Determine the allowable headwater depth (or depths) AHW, considering the factors discussed in sec. V.

6. Compute the depth of flow in the stream channel (including flood plain) for the design flood, and determine TW depth.

7. Select one or more appropriate culvert types. Compute an approximate barrel area $A_b = Q/10$ to guide selection of type and possible numbers and sizes of multiple barrels. Compute the discharge rate Q per barrel if multiple barrels are used.

8. Determine if the culvert types selected together with the governing headwater, length and slope, meet requirements for the direct use of charts, sec. III.

   a. Select the culvert capacity chart for the culvert and entrance type to be considered.

   b. On the chart, locate the point of intersection of Q and AHW.

   c. Use the culvert $L/100S_o$ and the $L/100S_o$ of the chart curves to determine the smallest culvert size which will result in an actual headwater depth HW equal to or less than AHW (sec. II C).

   d. Check tailwater as instructed in sec. III.

9. Culvert size may also be selected from the charts for some conditions where the requirements for direct selection of size from charts are not met and therefore step 8 above cannot be followed. These conditions include the following cases as described in sec. IV.

   Case 1 - Paved Invert C.M. Pipe or Pipe-Arch.

   Case 2 - Fully Paved C.M. Pipe.

   Case 3 - Rectangular Concrete Box sizes not in charts.

   Case 4 - Concrete or C.M. Circular Pipe sizes between those of chart curves.

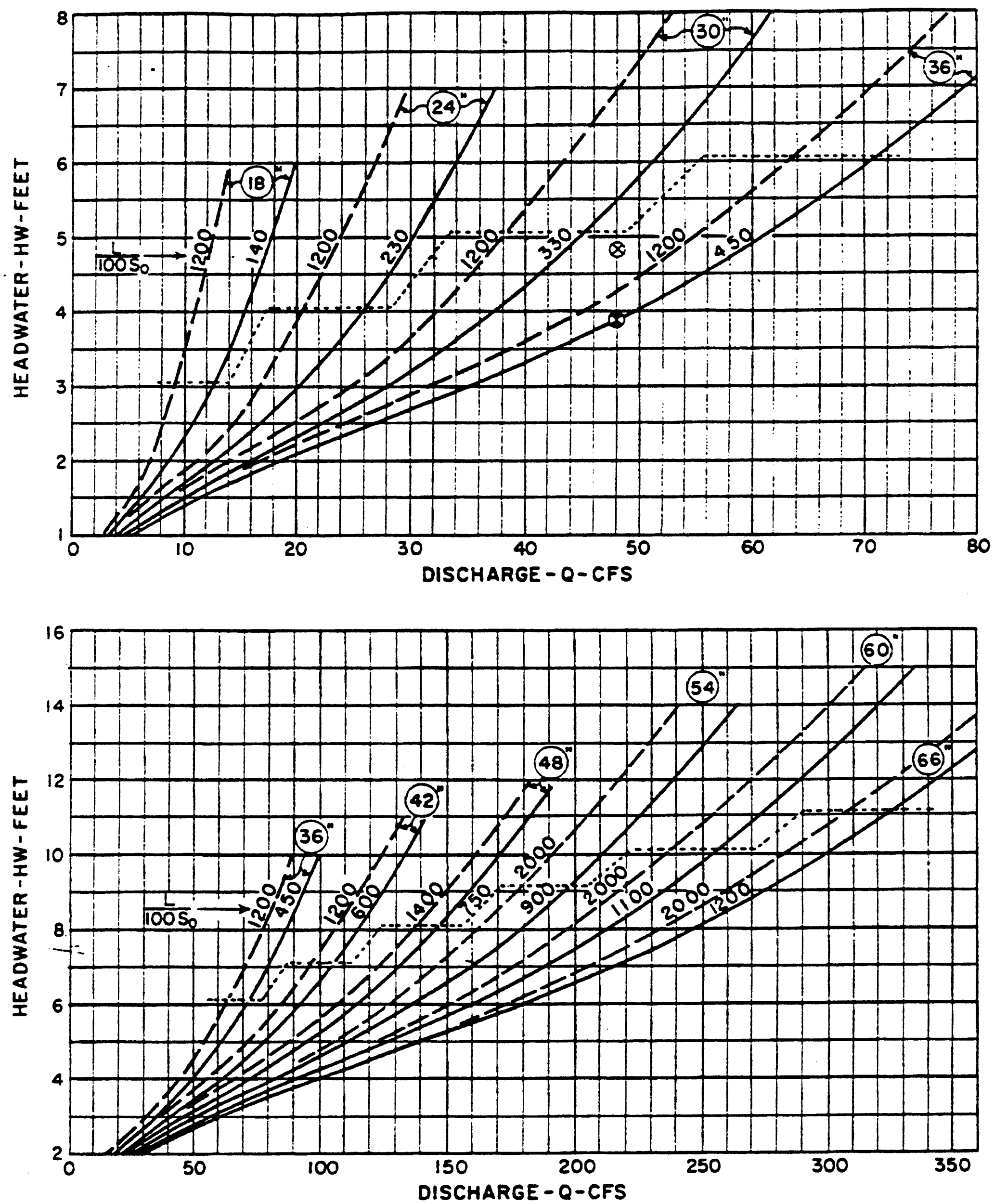
   Case 5 - Oval Concrete Pipe sizes not in chart.

   Case 6 - Corrugated Structural Plate Pipe-Arch sizes not in chart.

   Case 7 - Culvert slope zero (level invert).

   Case 8 - Broken slope culverts.

   Case 9 - $L/100S_o$ exceeds chart value.



EXAMPLE

⊗ GIVEN:
48 CFS; AHW = 4.8 FT.
L = 60 FT.; S_0 = 0.003

⊗ SELECT 36"
HW = 3.9 FT.

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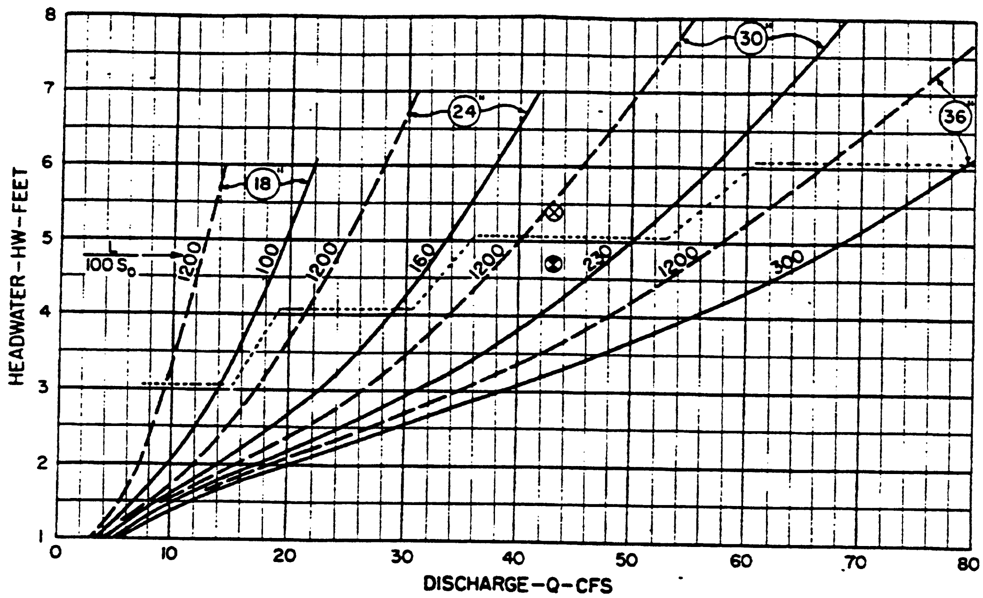
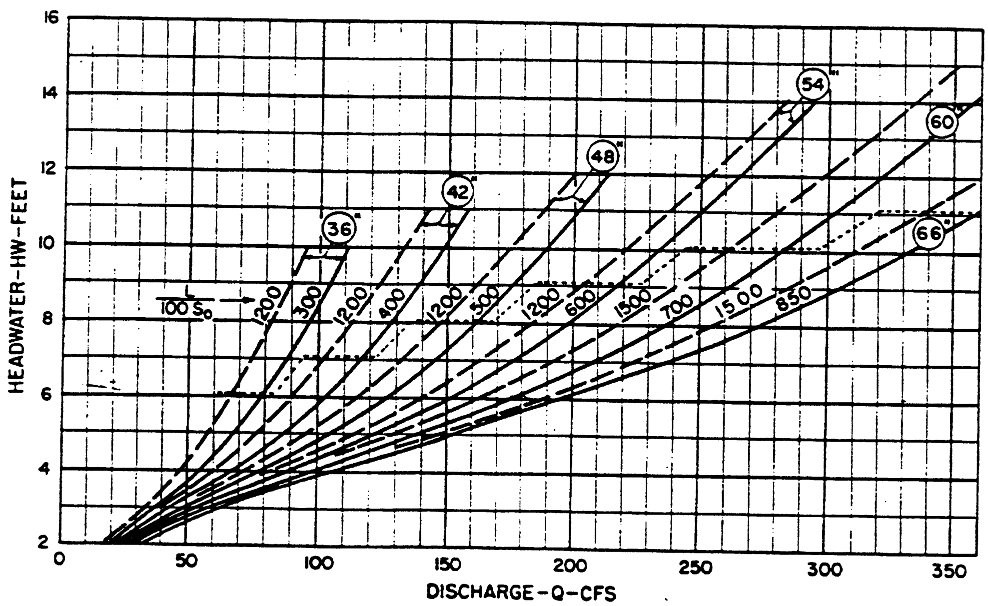
CULVERT CAPACITY
CIRCULAR CONCRETE PIPE
SQUARE-EDGED ENTRANCE
18" TO 66"

CHART 13





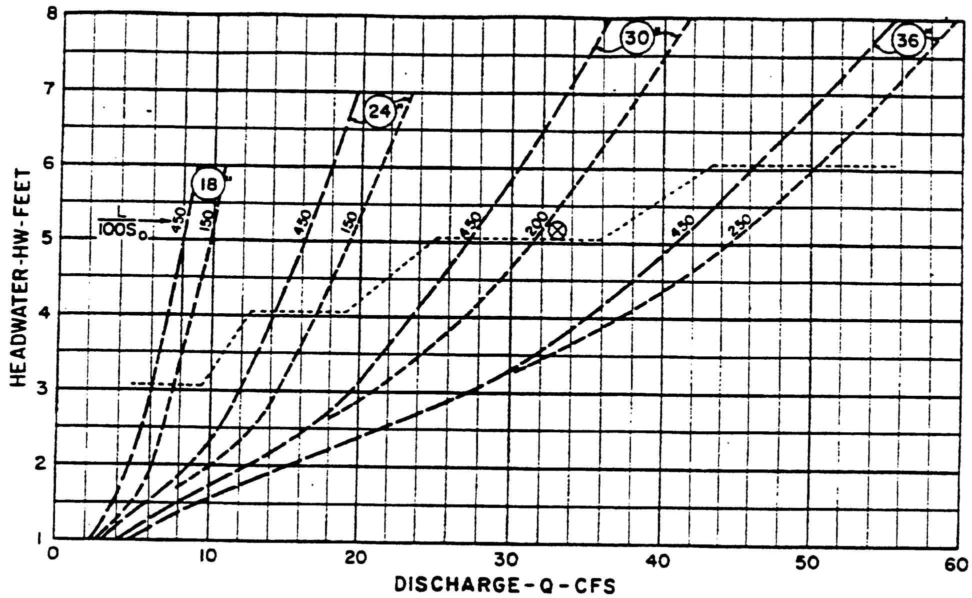
EXAMPLE

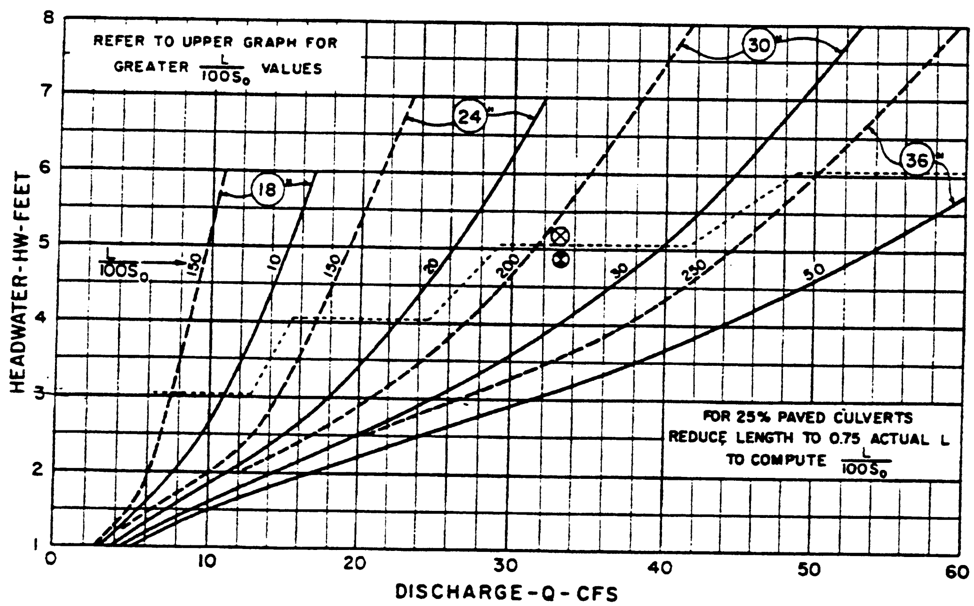
⊗ GIVEN:
43 CFS ; AHW = 5.4 FT.
L = 120 FT.; S_0 = 0.002

⊕ SELECT 30"
HW = 4.7 FT.

BUREAU OF PUBLIC ROADS JAN. 1963

CULVERT CAPACITY
CIRCULAR CONCRETE PIPE
GROOVE - EDGED ENTRANCE
18" TO 66"





EXAMPLE

⊗ GIVEN:
33 CFS: AHW = 5.2 FT.
L = 70 FT.; S_o = 0.005

⊕ SELECT 30" UNPAVED
HW = 4.9 FT.

CULVERT CAPACITY
STANDARD
CIRCULAR CORR. METAL PIPE
PROJECTING ENTRANCE
18" TO 36" ○

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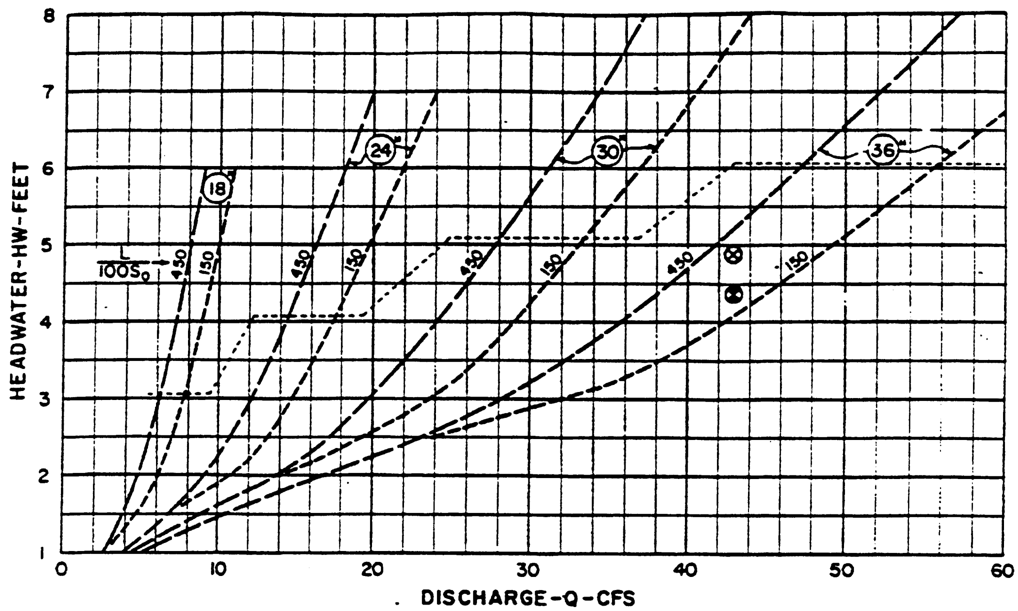
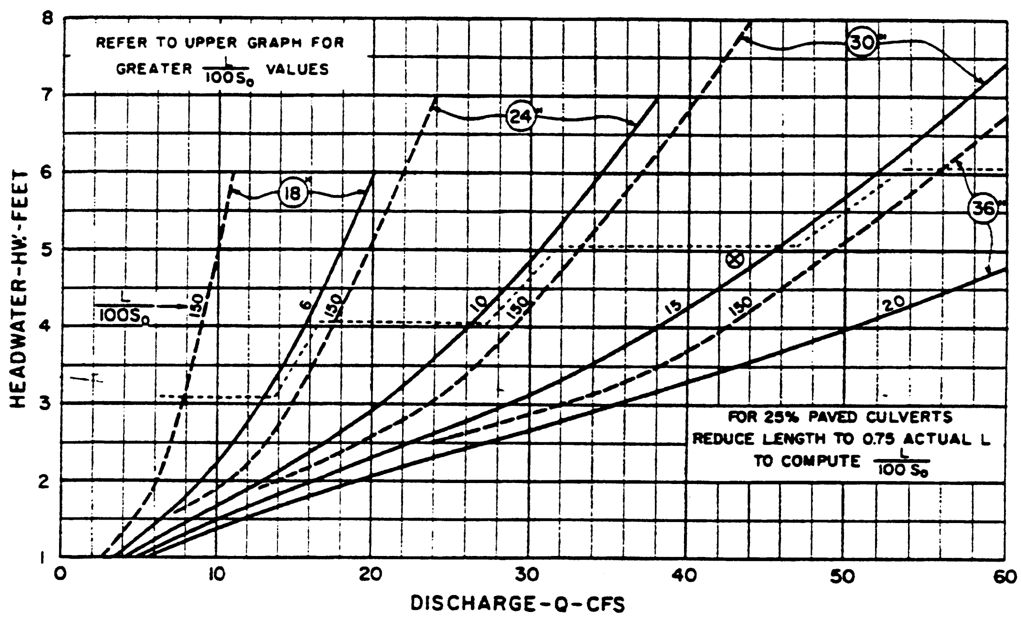
CHART 22





EXAMPLE

⊗ GIVEN:
43 CFS; AHW = 4.9 FT.
L = 72 FT.; S_o = 0.003

⊕ SELECT 36" UNPAVED
HW = 4.4 FT.

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CULVERT CAPACITY
STANDARD
CIRCULAR CORR. METAL PIPE
HEADWALL ENTRANCE
18" TO 36" ○

APPENDIX C

Example Applications

LEVEL 1 STORMWATER DETENTION/RETENTION PROCEDURE WORKSHEET				
Step	**Parameter Description**	**Equation/ Method Determined**	**Value**	**Units**

PROJECT DESCRIPTION:		40 acre residential subdivision in Prescott, 4 houses/acre density			
1	Project Site Area	A	From site data	40	Acres
2	100-year, 6-hour rainfall depth	$P_{100,6}$	From ADOT Hydrology Manual (1993), Precipitation Map No. 7	3.5	Inches
	100-year 24-hour rainfall depth	$P_{100,24}$	From ADOT Hydrology Manual (1993), Precipitation Map No. 8	4.9	Inches
	100-year, 1-hour rainfall depth	$P_{100,1}$	From Figure 1 of State Standard	2.4	Inches
3	Developed condition runoff coefficient for project site	C	From ADOT Hydrology Manual (1993), Figure 2-3	.72	None
4	Developed condition 100-year, 1-hour runoff volume	V_r	$Vr = (CAP_{100,1})/12$	5.76	Acre-ft

NOTES:

A basin approximately 3 acres in area with a 3 foot maximum depth was designed with 4:1 side slopes. Six, 6" diameter pipes were provided at the low-point draining the pond into the wash which provides natural drainage for the site. Rock riprap is provided at the outlet to prevent erosion. Maintenance access is provided by a 6:1 ramp into the basin bottom.

LEVEL 2 STORMWATER DETENTION/RETENTION PROCEDURE WORKSHEET PAGE 1 OF 2					
Step	**Parameter Description**		**Equation/ Method Determined**	**Value**	**Units**

PROJECT DESCRIPTION:		40 acre residential subdivision in Prescott, 4 houses/acre density			
1	Project Site Area	A	From site data	40	Acres
2	100-year, 6-hour rainfall depth	$P_{100,6}$	From ADOT Hydrology Manual (1993), Precipitation Map No. 7	3.5	Inches
	100-year 24-hour rainfall depth	$P_{100,24}$	From ADOT Hydrology Manual (1993), Precipitation Map No. 8.	4.9	Inches
	100-year, 1-hour rainfall depth	$P_{100,1}$	From Figure 1 of State Standard	2.4	Inches
3	Developed condition runoff coefficient for project site	C	From ADOT Hydrology Manual (1993), Figure 2-3	.72	None
4	Developed condition 100-year, 1-hour runoff volume	V_r	$Vr = (CAP_{100,1})/12$	5.76	Acre-ft
5	Length of longest flow path of site	L	ADOT Hydrology Manual (1993), page 2-4	0.27	Miles
	Watershed resistance coefficient for site	K_b	From Kb Chart contained herein	0.03	None
	Slope of longest flow path of site	S	ADOT Hydrology Manual (1993), page 2-4	26	Ft/mile
	Assumed Time of concentration[8]	Tc	Assumed	0.17	Hours
	Rainfall intensity	I	ADOT Hydrology Manual (1993), Figure 2-1 or 2-2	6.75	Inches/ hour
	Calculated Time of Concentration	Tc	$Tc = 11.4\ L^{0.5} K_b^{0.52} S^{-0.31} i^{-0.38}$ (Eqn. 2-2 from ADOT Manual)	0.17	Hours
	Developed condition 100-year peak discharge for project site	Q	Q = CiA	194.5	Cfs

[8] Adjust this value until assumed value agrees with calculated value. T_c cannot fall below 10 minutes

LEVEL 2 STORMWATER DETENTION/RETENTION PROCEDURE WORKSHEET PAGE 2 OF 2					
Step	**Parameter Description**		**Equation/ Method Determined**	**Value**	**Units**
6	Area of regional watershed	A_r	Per definition in Level 2 procedure	5	Mi^2
	Regional runoff rate	q_r	From 100-year Unit Discharge chart for appropriate region	755	Cfs/ Sq mi
	Regionally adjusted existing condition discharge from project site	Q_{off}	$Q_{off} = A\ q_{off}/640$	47.2	Cfs
7	Ratio of design outflow to design inflow	Q_{off}/Q	Q_{off}/Q	0.243	Ratio
	Ratio of required storage volume to runoff volume	V_s/V_r	From Q_{off}/Q vs. V_s/V_r Chart	0.67 (pipe outlet)	Ratio
8	Required storage volume	V_s	$V_s = V_r\ (V_s/V_r)$	3.82	Acre-ft
9	Outflow structure		Use HEC-10 pipe outflow structure design charts (Appendix A of state standard) or other reference	3 – 50' x 24" CMP outlet at 0.5% slope	

NOTES:

A basin approximately 2 acres in area with a 3 foot maximum depth was designed with 4:1 side slopes. Rock riprap is provided at the outlet to prevent erosion. Maintenance access is provided by a 6:1 ramp into the basin bottom.