Colorado Statewide
Drainage and Floodplain Management
Criteria Manual

August 2002

Colorado Water Conservation Board
1313 Sherman Street, Room 721
Denver, Colorado 80203
303.866.3441
Fax 303.866.4474

Project Consultant:
WRC ENGINEERING, INC.
950 S. Cherry
Denver, CO 80246
303.757.8513
Fax 303.758.3208

EXECUTIVE SUMMARY

This section will be completed in Phase 2 of the Criteria Manual project.

ACKNOWLEDGEMENTS

The Colorado Water Conservation Board (CWCB) would like to recognize, credit, and thank those individuals for their involvement and contributions to the development of the *Colorado Statewide Drainage and Floodplain Management Criteria Manual.* The following list of individuals and respective affiliations encompasses those that have participated in the preparation, authoring, and subsequent review of the *Manual.*

COLORADO WATER CONSERVATION BOARD (CWCB)

Tom Browning, P.E	Project Manager
Larry Lang, P.E	Chief of Flood Protection Section

WRC ENGINEERING, INC.

Yong K. Song, P.E.	Project Manager
Alan J. Leak, P.E.	Principal Reviewer
Andy S. Andrews, P.E.	Reviewer
Kara N. Koellner	Reviewer and Editor
Mark A. Stacks	AutoCAD Support
Miriam A. Gaines	Formatting
Evelyn L. Loar	Document Production

TABLE OF CONTENTS

Chapter 1

CHAPTER 1

GENERAL PROVISIONS

This Chapter will be completed in Phase 2 of the Criteria Manual project.

Chapter 2

CHAPTER 2

DATA REQUIREMENTS AND RESOURCES

This Chapter will be completed in Phase 2 of the Criteria Manual project.

Chapter 3

CHAPTER 3

DRAINAGE PLANNING AND REPORT SUBMITTAL

This Chapter will be completed in Phase 2 of the Criteria Manual project.

Chapter 4

CHAPTER 4

FLOODPLAIN ADMINISTRATION AND DELINEATION

CHAPTER 4

FLOODPLAIN ADMINISTRATION AND DELINEATION

SECTION 1.0

FLOODPLAIN MANAGEMENT

CHAPTER 4
FLOODPLAIN ADMINISTRATION AND DELINEATION

SECTION 1.0
FLOODPLAIN MANAGEMENT

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF STANDARD FORMS

CHAPTER 4
FLOODPLAIN ADMINISTRATION AND DELINEATION

SECTION 1.0
FLOODPLAIN MANAGEMENT

1.1 INTRODUCTION

Floodplain management is a program that utilizes corrective and preventative measures to reduce flood damages to public and private properties and to promote public safety and general welfare of the community. Floodplain management program elements include, but are not limited to, floodplain management regulations, structural and non-structural flood mitigation measures, flood warning systems, emergency response procedures, operations and maintenance, flood insurance, and public education.

This section is intended to describe practical floodplain management guidelines to reduce future flood losses for communities in the State of Colorado.

Areas that are subject to flooding should be regulated by each community through appropriate floodplain management practices. This section is intended to describe practical floodplain management guidelines to reduce future flood losses for communities in the State of Colorado. The guidelines outlined in this chapter have been developed to meet or exceed the minimum standards imposed by the Federal Emergency Management Agency (FEMA) (44 CFR Part 60).

1.2 NATIONAL FLOOD INSURANCE PROGRAM

The National Flood Insurance Program (NFIP) was formed by Congress in 1968 to make federally sponsored flood insurance available in communities which agreed to adopt and actively enforce floodplain management regulations. Floodplain management regulations should be consistent with the minimum requirements outlined in the Code of Federal Regulations (44 CFR Part 60). Communities are encouraged to adopt and enforce floodplain management regulations that exceed the minimum NFIP standards.

Working closely together with the participating communities, the NFIP program helps reduce future flood losses by regulating developments in the 100-year floodplains and by providing flood insurance coverage. National flood insurance coverage is available to property owners and occupants of insurable properties in the communities participating in the NFIP. Flood insurance is required for federal or federally insured loans for building structures located within the FEMA Special Flood Hazard Areas (SFHA). Flood Insurance Rate Maps (FIRM) depicting the SFHA are usually available to communities participating in the NFIP regular program. It is strongly recommended that all property owners of structures located within the 100-year floodplains obtain flood insurance coverage to protect their properties against future flood losses.

1.2.1 COMMUNITY RATING SYSTEM

Community Rating System (CRS) was implemented to encourage and recognize the communities actively carrying out their floodplain management programs that exceed the minimum NFIP standards. Under this program, NFIP participating communities can be rated as Class 1 through 10 based on their active floodplain management programs. The community's flood insurance premium can be reduced based on their current CRS rating.

Participation in the CRS program is voluntary and the detailed application procedures can be obtained from FEMA's website (www.fema.gov).

1.3 FLOODPLAIN MANAGEMENT GOALS

When a significant flood event occurs, considerable public and private property damages can occur and there is a higher risk for loss of life. A health risk is also experienced as floodwaters can surcharge sanitary sewer systems and expose stream flows to potentially hazardous materials in the floodplain. These flood hazards are mostly caused by inadequate understanding, planning, and protection of existing and new developments within the floodplains. The collective effects of new developments outside of the floodplains can also increase the flood losses by increasing surface runoff and raising the base floodwater elevations.

The floodplain management guidelines outlined in this section are intended to guide future improvements in the 100-year floodplain in a manner that reduces the potential risks to both existing and future developments in the floodplain.

Local communities are encouraged to develop future (built-out) conditions floodplain information, especially when the area plan indicates a substantial amount of future development. Communities should regulate and guide their proposed floodplain developments based on the floodplain management guidelines provided in this section.

1.3.1 WATERSHED CONDITIONS

The FEMA 100-year floodplain and floodway boundaries shown on the Flood Insurance Rate Maps (FIRM) are delineated to reflect the existing watershed and floodplain conditions at the time of their Flood Insurance Studies (FIS). Likewise, the minimum floodplain management guidelines imposed by FEMA (44 CFR Part 60) and the flood insurance rates are determined based on the existing conditions 100-year floodplain information.

However, as more development occurs within a watershed, the previously estimated existing conditions 100-year peak flow rate and the associated floodplain limits may change considerably. Since, most FEMA FIRM maps are usually not updated frequently, the 100-year floodplain information shown on the FIRM maps may not accurately reflect the current floodplain conditions depending on the nature and amount of new developments.

Local communities are encouraged to develop future (built-out) conditions floodplain information, especially when the area plan indicates a substantial amount of future development. Once developed, communities may request FEMA to show future conditions floodplains on the FIRM maps in addition to the existing conditions floodplains.

Communities should regulate and guide their proposed floodplain developments based on the floodplain management guidelines provided in this section and the available existing conditions 100-year floodplain information. If future (built-out) conditions floodplain information is available to local communities, then, the proposed floodplain developments should be regulated based on the future conditions floodplain information.

1.4 LEVEL OF PROTECTION

The standard of practice, as required by FEMA, is to implement the floodplain management regulations based on a 100-year flood event. The 100-year peak discharge at a given point is the estimated peak discharge that has a 1% probability of occurrence in any given year. Flow rates in excess of the 100-year estimate can and will occur, but with lower probability. In those instances, typically the depth of flow and floodplain widths will be greater than indicated on the 100-year floodplain maps provided by FEMA.

Therefore, the guidelines described in this chapter will not necessarily protect a property owner against flood events that exceed the 100-year peak flow estimate. A property owner may choose to provide a greater level of protection than what is required by the floodplain management guidelines, especially in the case of critical facilities, buildings that store hazardous materials, and where building content damage could be significant.

1.5 SOURCES OF FLOOD HAZARD AREA INFORMATION

Many watercourses in the State of Colorado have been analyzed by various engineering studies sponsored by local, state, or federal agencies. The 100-year floodplain information generated and/or published by FEMA can be found on the Flood Insurance Rate Maps (FIRM) and Flood Insurance Studies (FIS). All floodplain data generated by FEMA and other engineering studies should be available at the local Floodplain Administrators office, the CWCB, or FEMA. The existing floodplain studies and delineations should be evaluated to determine if the information is still valid. When determined appropriate, the existing studies should be used to minimize duplication of work and to maintain continuity of the analysis.

Please note, floodplain data is periodically updated to reflect changes due to floodplain modifications or the use of better technical data. Users of the floodplain information should check with the local Floodplain Administrator to ensure that the information is current. Readers are referred to Sections 2 and 3 of this chapter for detailed discussions on the floodplain delineation and revision methods.

1.6 REGULATORY FLOODWAYS

The floodway is defined as the channel plus any adjacent floodplain areas, that must be kept free of encroachment so that the 100-year discharge can be conveyed with no more than one foot rise in the water surface above the base flood elevations (BFE). However, local communities may choose to adopt stricter floodway delineation criteria by allowing only 0.5 foot or no rise above BFE. The floodway represents the community's regulatory limit of encroachment into the 100-year floodplain for those watercourses with the established floodway boundaries.

Local communities may choose to adopt stricter floodway delineation criteria by allowing only 0.5 foot or no rise above BFE.

Figure CH4-F101 is a conceptual representation of a channel section showing the floodplain, flood fringe, and floodway limits. Encroachment into the designated floodway is prohibited unless it has been demonstrated using appropriate detailed engineering analyses that the proposed encroachment will not cause any rise in the 100-year water surface elevation. Since the floodway is an extremely hazardous area, construction of new building structures is not permitted.

Flood fringe is the area between the delineated floodplain and floodway boundaries. New developments are allowed in the flood fringe in accordance with the development guidelines established in the floodplain management criteria. If the regulatory floodway has not been established, new development in the floodplain is allowed only if it can be demonstrated through appropriate detailed engineering analyses that the proposed development will not adversely impact surrounding properties.

1.7 UNDETERMINED OR APPROXIMATE FLOODPLAIN

Numerous engineering studies have been performed to develop the needed floodplain information throughout the state. However, many areas that are subject to severe flooding have not yet been studied and/or designated as a 100-year floodplain. Areas outside of the designated floodplains can experience substantial flooding during a storm event. These known and/or undetermined flood hazard areas should be studied in detail to determine flow rates and the associated floodplain limits prior to the issuance of a building or grading permit.

New developments within the identified 100-year floodplain will be allowed in accordance with the floodplain development guidelines outlined in this section. If new developments are proposed to encroach into or located adjacent to a previously delineated approximate 100-year floodplain, the

Known and/or undetermined flood hazard areas should be studied in detail to determine flow rates and the associated floodplain limits prior to the issuance of a building or grading permit. If new developments are proposed to encroach into or located adjacent to a previously delineated approximate 100-year floodplain, the previously delineated approximate floodplain limits should be restudied by using the detailed approach before the developments can occur.

previously delineated approximate floodplain limits should be restudied by using the detailed approach before the developments can occur.

Readers are referred to Section 2, Chapter 4 for detailed discussions on the floodplain delineation methods.

1.8 FLOODPLAIN DEVELOPMENT PERMIT REQUIREMENTS

A Floodplain Development Permit is required for all construction and development activities to be undertaken within the 100-year flood hazard areas. These activities include, but are not limited to, building or enlarging a structure, remodeling or improving a structure, placing a manufactured home, mining, dredging, filling, grading, paving, excavating, and drilling within the 100-year flood hazard areas. In other words, any structural or non-structural activity that may affect flooding or flood damage must have a permit.

A Floodplain Development Permit from the local Floodplain Administrator is required before beginning any construction and development activities within a 100-year flood hazard area.

1.8.1 INITIAL FLOODPLAIN DETERMINATION

Prior to the submittal of a building/grading permit or a development application, an initial floodplain determination for the proposed project site must be issued by the local Floodplain Administrator (Standard Form CH4-SF101). The Floodplain Administrator will determine whether the proposed project site is located within the 100-year flood hazard area shown on the community's floodplain map and/or FEMA FIRMs.

A Floodplain Development Permit will be required before a building or grading permit can be issued for any construction and development activities within the 100-year flood hazard area. The initial floodplain determination may also identify other special provisions that may be imposed as a condition of approval for a Floodplain Development Permit.

The initial determination will identify only minimum requirements. This determination is intended only to guide the applicant and the community in its application of the Floodplain Management Regulations. Additional require-ments may be imposed during the project or permitting review process as additional technical information is presented or project modifications are made. Property owners, who wish to obtain a more detailed floodplain determination than can be provided from the existing data, must obtain their own determination from licensed and qualified

Prior to the submittal of a building/grading permit or a development application, an initial floodplain determination for the proposed project site must be issued by the local Floodplain Administrator. A Floodplain Development Permit will be required before a building or grading permit can be issued for any construction and development activities within the 100-year flood hazard area.

professionals. The applicant is encouraged to review those findings with the staff before proceeding with project planning or design.

1.8.2 FLOODPLAIN DEVELOPMENT PERMIT APPLICATION

Before any construction and development activities can begin within a 100-year flood hazard area, the applicant must obtain a Floodplain Development Permit Application (Standard Form CH4-SF102) and the special conditions, as determined in the initial floodplain determination by the Floodplain Administrator. The permit application and the supporting documents should be submitted for evaluation and approval. The local Floodplain Administrator will evaluate the application to determine if the proposed project is consistent and complies with the community's goals and floodplain development guidelines. The permit will be approved or denied based on the compliance. A building or grading permit will not be issued without a Floodplain Development Permit.

1.9 ELEVATION CERTIFICATION

Unless the proposed development site has been removed from the 100-year flood hazard area through floodplain modifications and the appropriate floodplain map revision process, Elevation Certificates shall be issued for all new and substantially improved structures. Readers are referred to Section 3, Chapter 4 for detailed discussions on the FEMA map revision process and to Section 1.20, Chapter 4 for definition of substantially damaged and substantially improved structures.

Unless the proposed development site has been removed from the 100-year flood hazard area through floodplain modifications and the appropriate floodplain map revision process, Elevation Certificates shall be issued for all new and substantially improved structures.

An Elevation Certificate should be certified by a licensed Professional Civil Engineer or Land Surveyor, confirming that the "as-built" lowest floor elevation (including basement and/or crawl space) is at or above the required elevations outlined below. The current FEMA Elevation Certificate form should be used to certify building elevations. This certificate may also be required by an insurance agent for adjustment of flood insurance rates. A copy of the FEMA Elevation Certificate form can be obtained from the Floodplain Administrator or directly from the FEMA website (www.fema.gov).

All new, substantially damaged, and substantially improved buildings shall be constructed to meet or exceed the following lowest floor (including basement and/or crawl space) elevation requirements. Non-residential buildings may be flood-proofed or elevated to the same elevation requirements.

- Zone AE — The lowest floor shall be elevated at least one (1) foot above the 100-year base flood elevation (BFE).
- Zone AO — The lowest floor shall be elevated above the highest adjacent natural grade by at least one (1) foot plus the 100-year flood depth specified.

- Zone A — The lowest floor shall be elevated at least one (1) foot above the 100-year BFE determined by the community through a detailed engineering analysis.

- Zone X — The lowest floor shall be elevated at least two (2) feet above the highest adjacent natural grade or the top of curb elevation.

- Other Zones — The lowest floor shall be elevated at least one (1) foot above the 100-year base flood elevation (BFE).

Lowest floor elevation requirements are provided to account for the uncertainties in the estimated 100-year water surface elevations and the potential impacts created by future developments. Communities that are currently regulating their floodplain developments based on the future (built-out) conditions floodplain information may choose adopt a less conservative requirement that exceed the minimum NFIP standards.

1.10 IMPROVEMENTS TO EXISTING BUILDINGS IN THE FLOODPLAIN

Applicants must obtain Floodplain Development Permits for improvements to existing buildings located within the 100-year floodplain. The lowest floor (including the basement and/or bottom of the crawlspace) of substantially improved or substantially damaged structures must be elevated to meet or exceed the lowest floor elevation requirements. The building structures must be placed at or above the base flood elevation. The Elevation Certificate should be approved by the Floodplain Administrator prior to issuance of a Certificate of Occupancy.

1.11 FLOODPROOFING CERTIFICATION

New, substantially damaged, and substantially improved non-residential structures located within the 100-year floodplain can be floodproofed instead of elevating the structures above the 100-year flood elevations. Flood-proofing consists of designing a structure in such a way that all parts are watertight and resistant to flood damage. Anytime a non-residential structure is flood-proofed, a registered professional engineer or architect must certify that the flood-proofing measures meet the National Flood Insurance Program (NFIP) Design Standards. Please refer to the following FEMA publications.

- FEMA, 1993 - Federal Emergency Management Agency, Flood-Resistant Materials Requirements for Buildings Located in Special Flood Hazard Areas in Accordance with the National Flood Insurance Program, Technical Bulletin 2-93, 1993.

- FEMA, 1993 - Federal Emergency Management Agency, Non-Residential Floodproofing – Requirements and Certification for Buildings Located in Special Flood Hazard Areas in Accordance with the National Flood Insurance Program, Technical Bulletin 3-93, 1993.

A Floodproofing Certificate, certified by a registered professional engineer or architect, must be submitted with the Floodplain Development Permit application. The certificate must demonstrate that the building is floodproofed up to the same elevation as the lowest floor elevation which would have been required if the building was to be elevated to meet the lowest floor elevation requirements outlined in Chapter 4, Section 1.9. The current FEMA Floodproofing Certificate form should be used, and a copy of this form can be obtained from the Floodplain Administrator or directly from the FEMA website (www.fema.gov). The Floodproofing Certificate may also be required by an insurance agent for adjustment of flood insurance rates.

1.12 MANUFACTURED HOMES

Additions or placement of manufactured homes in the floodplain must be elevated and constructed on an approved foundation. The lowest floor of these manufactured homes shall be elevated to meet the standards previously outlined in Section 1.9.

Manufactured homes in the flood hazard area shall be anchored to resist floatation, collapse or lateral movement resulting from the hydrostatic and hydrodynamic loads associated with the 100-year flood flows. Tie-downs and anchors embedded in the ground can be used to securely fasten the homes. The site-specific soil types shall be considered in choosing the appropriate anchoring methods.

1.13 DEVELOPING IN FLOODPLAINS

All new developments proposed within a 100-year flood hazard area, as determined in the Initial Floodplain Determination or through additional analyses, will be required to obtain a Floodplain Development Permit from the local governing authority before construction or development begins within any area of the floodplain. Those who wish to develop in a floodplain must modify the floodplain so that all new structures will be outside of the 100-year floodplain. Requests to modify the floodplain must be reviewed and approved by the effected local government agencies and the Colorado Water Conservation Board (CWCB). If the proposed floodplain modification pertains to a FEMA-designated SFHA, the revision request must also be reviewed and approved by FEMA. Please refer to Chapter 4, Section 3.0 for detailed discussions on the floodplain revision (LOMA, LOMR, etc.) process. A brief description of each of these agencies and the roles they play follows:

Federal Emergency Management Agency (FEMA) - This agency administers the National Flood Insurance Program (NFIP). FEMA conducts Flood Insurance Studies and publishes Flood Hazard Boundary Maps and Flood Insurance Rate Maps, which show regulatory floodplain boundaries for many major drainage-ways. FEMA reviews all requests to modify the designated FEMA floodplains.

Colorado Water Conservation Board (CWCB) - The CWCB is the state agency with the authority to review all floodplain information developed for zoning purposes for streams in the State of Colorado. CWCB reviews and approves floodplain information and designations at the request of a community or by acting on its own initiative.

Local Government Agencies –Local government agencies have the authority to adopt and enforce floodplain management regulations. All communities impacted by the proposed floodplain modification/designation should review the floodplain modification study submittals. Local government agencies include cities, counties, towns, Indian tribes, and flood control districts.

1.14 FLOODPLAIN MODIFICATION STUDY

All non-exempt floodplain developments shall prepare and submit a Floodplain Modification Study for review and approval by the local government agencies and the Colorado Water Conservation Board.

A Floodplain Modification study is required when there is a proposed modification to a 100-year floodplain or floodway. All non-exempt floodplain developments shall prepare and submit a Floodplain Modification Study for review and approval by the local government agencies and the Colorado Water Conservation Board. The general outlines for a Floodplain Modification Study are provided in Section 1.14.2. For modification of a FEMA floodplain, a LOMR request report may be submitted in place of a Floodplain Modification Study.

A Floodplain Modification Study or a LOMR request report shall demonstrate that the proposed project meets the floodplain development guidelines and that the upstream, adjacent, and downstream properties are not being adversely impacted. The study shall be professionally prepared, legible and reproducible.

1.14.1 FLOODPLAIN MODIFICATION STUDY REQUIREMENTS

A Floodplain Modification Study shall be submitted for the following activities:

1. As an initial feasibility study to determine the potential utilization of a site with floodplain impacts.

2. Attempting to develop in the 100-year floodplain designated by FEMA and/or the community.

3. Restudy and revision of a previously delineated floodplain.

4. Public works improvements effecting the designated 100-year floodplain

The effort necessary for a Floodplain Modification Study is dependent upon the amount of information previously generated, the potential for impact on adjacent properties, the magnitude of flow in the channel, the size of the area affected, the need for channel stabilization, and the sediment transport and fluvial morphological aspects of the stream. If there is no existing floodplain delineation for the drainageway in question, the applicant will be required to produce one (refer to Section 2, Chapter 4). The applicant is not required to submit a separate Floodplain Delineation Study report (Section 2.11, Chapter 4) if the new floodplain delineation information is included in the Floodplain Modification Study.
A Floodplain Modification Study must be certified by a registered Civil Engineer licensed to practice in the State of Colorado. The following is a list of general requirements for the Floodplain Modification Study:

1. Hydrologic analyses should be performed to determine the existing and proposed conditions flow rates at selected concentration points and to determine the potential impacts cause by the proposed project. Please refer to Chapter 5 for

discussions on the hydrologic analysis procedures and methods.

2. Hydraulic analyses should be performed to determine the 100-year floodplain information for both the existing and proposed project drainageway conditions. Detailed discussions on the floodplain delineation procedures are provided in Chapter 4, Section 2.0.

3. The study must demonstrate that no new structures are added to the floodplain within and adjacent to the project as a result of the proposed floodplain modification.

4. The study must demonstrate how other properties that share frontage along the floodplain will not be adversely impacted. If other properties are impacted, the submittal must include proof that the impact can be mitigated or appropriate floodplain agreements can be obtained in the form of a letter from the impacted property owners. At the discretion of the Floodplain Administrator, a floodway delineation may be required.

5. The analysis must demonstrate that the existing and/or proposed channel alignment (horizontal and vertical) will be stable and will not be subject to erosion, which may threaten property or public improvements.

6. The analysis must show that sufficient flow conveyance capacity will be maintained in the drainageway.

7. For subdivisions, the modified floodplain should not encroach onto the residential lots.

1.14.2 FLOODPLAIN MODIFICATION STUDY OUTLINE

A floodplain Modification Study must address the following points through actual analyses or through reference to adopted drainage master plans:

1. A description of the project site.

2. A description of the drainage basins and waterways impacting the site

3. Identification of existing drainage master plans and/or Flood Insurance Studies (FIS) with the analysis of the applicability of the existing data to the proposed project site.

4. Hydrologic analysis. This section should include a narrative of the analysis methods, the source of flow rates used for the existing and proposed project conditions floodplain delineations, and the proposed flow conveyance system. Watershed maps should be provided for both conditions.

5. Characteristics of the proposed channel including slope, roughness, depth, velocity, Froude Number, centerline alignment and stationing, and cross sections. Existing topographic maps may be utilized if they are field verified to determine if changes have occurred. The plan and profile shall be provided for the existing channel and the proposed channel including the cross section locations.

6. A description of the method of hydraulic analysis and its application in the study.

7. Identification and discussion of all input parameters and the basis for the input parameters.

8. Discussions of the results and conclusions for the hydrologic and hydraulic analyses. This shall include a narrative summary of the results as well as comprehensive output file printouts, free of modeling errors.

9. Delineation of the existing and proposed project conditions 100-year floodplains, water surface profiles, and cross-section locations.

10. A description of potential impacts, if any, on other property owners along the floodplain.

11. A conceptual design of the proposed drainage system including embankment protections, drop structures, culverts, bridges, and hardened trickle or low flow channel.

12. If appropriate, an analysis of sediment transport and fluvial morphology.

1.14.3 SMALL ENCROACHMENT AREA EXEMPTION

If the floodplain encroachment resulting from the proposed activity is small, the applicant may claim exemption from the requirement of a Floodplain Modification Study provided that the applicant can demonstrate the following:

The proposed floodplain modifications do not remove more than 5% of the 100-year flow conveyance area for a given cross section, and the area removed by the proposed activity will be compensated by additional grading so that there is no net loss of effective flow conveyance area for the 100-year flow.

To claim an exemption, the applicant must submit the following information to the Floodplain Administrator:

- A topographic map showing the location of the proposed project site with the currently accepted floodplain delineation clearly shown.

- A plot of a surveyed cross section of the drainage channel where the maximum channel encroachment resulting from the new development is expected to occur. This plot should clearly show the existing and proposed grading and the currently accepted 100-year water surface elevation.

1.15 FEMA DESIGNATED FLOODPLAINS

If the proposed floodplain modification pertains to a FEMA-designated floodplain, a floodplain revision request should be submitted to FEMA for their review and approval. The applicant has an option to submit a request for a Conditional Letter of Map Revision (CLOMR) before the project is built and then follow the CLOMR with a Letter of Map Revision (LOMR) request, or wait until the project is completed and submit a request for LOMR without a CLOMR. It is recommended that the applicant choose the request for CLOMR option, since that process will allow the requester to modify the project design if required to do so by FEMA prior to construction. Please refer to Section 3.0 of this chapter for detailed discussions on the FEMA map revision procedures.

1.16 STORAGE OF MATERIALS IN THE FLOODPLAIN OR FLOODWAY

Storage of hazardous or floatable materials in the floodplain and floodway is prohibited. These materials represent a significant potential public health, environmental, or safety risks. Floatable materials can become lodged in culverts, bridges, and channels reducing the flow conveyance capacity of these structures resulting in increased flood damages.

Storage of other materials in the floodplain and floodway is prohibited unless permitted by the Floodplain Administrator. Some storage of vehicles or other materials may be permitted depending upon location and type of materials stored as long as the material can be relocated in accordance with an emergency action plan that has been approved by the Floodplain Administrator. Recreational vehicles cannot be stored in the floodplain for longer than 180 days.

1.17 FENCING

Fencing in the floodplain is also subject to the approval of the Floodplain Administrator. The construction of fence in the floodplain can result in significant impacts to flood depths and flow distributions. Even open fencing such as chain link will collect floating debris, resulting in clogging and diversion of flood flows. Therefore, fencing within the floodplain must be approved by the Floodplain Administrator prior to construction. Fencing within the regulatory floodway is not permitted.

1.18 CRITICAL FACILITIES

New critical facilities should be constructed outside of the 100-year flood hazard areas and the lowest floor should be elevated above the 500-year flood elevation. If the 500-year flood elevation is not available, the lowest floor should be elevated at least 3 feet above the 100-year flood elevation.

Critical facilities include, but are not limited to:

- Structures or facilities that use or store highly volatile, flammable, explosive, toxic and/or water reactive materials.
- Hospitals, nursing homes and housing likely to contain occupants who may not be sufficiently mobile to avoid death or injury during a flood.
- Police stations, fire stations, vehicle and equipment storage facilities, and emergency operations centers that are needed for flood response activities before, during and after a flood event.
- Public and private utility facilities that are vital to maintaining or restoring normal services to flooded areas before, during and after a flood event.

1.19 UTILITIES

All utility and service facilities for new and substantially improved buildings shall be designed and constructed to prevent flood damage and penetration of floodwaters during a 100-year flood event. These facilities include, but are not limited to, electrical, gas, plumbing, air-conditioning and heating equipments, phone, cable, and water supply systems. New and replacement sanitary sewer systems within the 100-year flood hazard areas shall be designed and constructed to minimize penetration of floodwaters into the system.

Automatic backflow preventing devices shall be provided for all new, substantially damaged, and substantially improved buildings with sanitary sewer and storm drain system openings below the 100-year water surface elevation.

1.20 DEFINITIONS

Base Flood Elevation (BFE) - The elevation of the 100-year floodwater surface at the location of interest.

Conditional Letter of Map Revision (CLOMR) - FEMA's conditional approval of the proposed modifications to their regulatory Special Flood Hazard Area.

Development - Any man-made changes to improved or unimproved real estate including, but not limited to, building or enlarging a structure, remodeling or improving a structure, placing a manufactured home, mining, dredging, filling, grading, paving, excavating, and drilling.

Elevation Certificate - A certificate prepared by a registered professional engineer or land surveyor that shows various elevations of a building in comparison to the 100-year BFE. This certificate is used to determine if the building complies with local and federal elevation requirements for buildings located in the 100-year floodplain and is also used for adjusting flood insurance rates for buildings that meet the applicable elevation requirements.

Encroachment - A constriction, placement of fill, or other alteration of topography in the floodplain that reduces the area available to convey floodwaters.

Federal Emergency Management Agency (FEMA) - A federal agency that oversees the administration of the National Flood Insurance Program.

Flood Boundary and Floodway Map (FBFM) - The floodplain management map issued by FEMA that depicts, based on detailed analyses, the boundaries of the 100- and 500-year floodplains and the regulatory 100-year floodway.

Flood Hazard Boundary Map (FHBM) - The initial insurance map issued by FEMA that identifies, based on approximate analyses, the areas of 100-year flood hazard in a community.

Flood Insurance Rate Map (FIRM) - The insurance and floodplain management map issued by FEMA that identifies, based on detailed or approximate analyses, the areas of 100-year flood hazard in a community.

Flood Insurance Study (FIS) - An engineering study that is performed under contract to FEMA to identify flood prone areas and to determine BFEs, flood insurance risk zones, and other flood risk data for a community.

Floodplain - The area inundated during a flood event (the 100-year event unless stated otherwise) including ponding and ineffective flow conveyance areas.

Floodplain Administrator - The local official designated to administer and enforce the floodplain management regulations for the community.

Floodway - The regulatory area defined as the channel, plus any adjacent floodplain areas, that must be kept free of encroachment so that the 100-year flood discharge can be conveyed without increases of more than one (1) foot in the BFE.

Flood fringe - The area between the 100-year floodplain and floodway limits in which development and other forms of encroachment may be permitted.

Letter of Map Revision (LOMR) - An official revision, by letter, to an effective NFIP map. A LOMR may change flood insurance risk zones, floodplain boundary delineations, planimetric features, and/or BFEs.

National Flood Insurance Program (NFIP) - The federal program under which flood prone areas are identified and flood insurance is made available to owners of property in participating communities.

Physical Map Revision (PMR) - An official republication of an NFIP map to show changes to floodplain and/or floodway boundary delineations, BFEs, and planimetric features.

Structure - Walled or roofed building or manufactured home that is principally above ground.

Substantial Damage - Damage of any origin sustained by a structure whereby the cost of restoring the structure to its before damaged condition would equal or exceed fifty percent (50%) of the market value of the structure before the damage occurred.

Substantial Improvement - Any reconstruction, rehabilitation, addition, or other improvement of a structure, the cost of which equals or exceeds fifty percent (50%) of the market value of the structure before the "start of construction" of the improvement. This term includes structures which have incurred "substantial damage", regardless of the actual repair work performed. The term does not, however, include either:

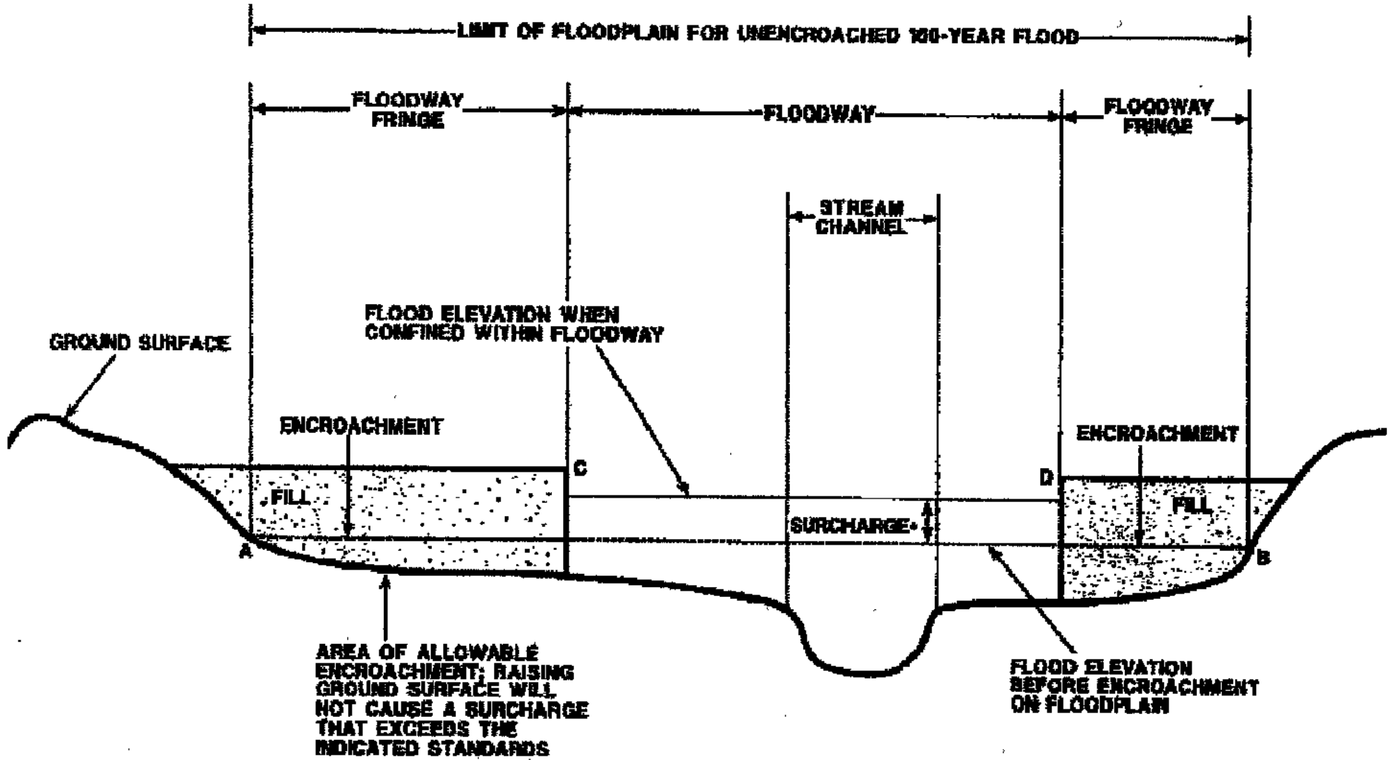
a. Project for improvement of a structure to correct existing violations of state or local health, sanitary, or safety code specifications which have been identified by the local code enforcement official and which are the minimum necessary to assure safe living conditions, or

b. Any alteration of a "historic structure" provided that the alteration would not preclude the structure's continued designation as a "historic structure".

O:/2080/FIGURES, CH4-F101.DWG, 4/5/02 MAS

COLORADO STATEWIDE

DRAINAGE AND FLOODPLAIN MANAGEMENT CRITERIA MANUAL



LINE A - B IS THE FLOOD ELEVATION BEFORE ENCROACHMENT
LINE C - D IS THE FLOOD ELEVATION AFTER ENCROACHMENT

* SURCHARGE NOT TO EXCEED 1.0 FOOT (FEDERAL EMERGENCY MANAGEMENT AGENCY REQUIREMENT) OR LESSER HEIGHT IF SPECIFIED BY STATE.

WRC ENGINEERING, INC.

VERSION: AUGUST 2002

REFERENCE:
FEMA FLOOD INSURANCE STUDY

FIGURE CH4-F101

TYPICAL FLOOD PLAIN CROSS SECTION

Initial Floodplain Determination Form

Applicant Information

Date ________________

Owner/Applicant ____

Address ______________________________ Telephone ________________

Site Information

Project Title ____

Street Address ____

Cross Street/APN ____

Type of Development(s)
- ___ New Building
- ___ Building Addition/Modification
- ___ Grading/Paving
- ___ Fence
- ___ Material Storage
- ___ Equipment Storage

Description of Development(s) ____

Floodplain Information

Property located in a designated floodplain? ___Yes ___ No

Flood Zone ______________________________

Map Source ______________________________

Property located in a designated floodway? ___Yes ___ No

Floodplain Development Permit Application Required? ___ Yes ___ No

Comments ____

Signature of Authorizing Official ________________ Date

Flood Plain Development Permit
Application Information

Date ____________________

Parcel Number ______________________________ Permit Number ____________________

Owner ______________________________ Telephone ____________________

Address ____

Contractor ______________________________ Telephone ____________________

Address ____

Project Location/Directions ____

____

Project Description

______ Single Family Residential
______ Multi-Family Residential
______ Manufactured (Mobile) Home
______ Non-Residential

______ New Construction
______ Substantial Improvement (>50%)
______ Improvement (<50%)
______ Rehabilitation

______ Channelization
______ Fill
______ Bridge/Culvert
______ Levee

Other/Explanations ____

____

Flood Hazard Data

Watercourse Name ____

The project is proposed in the ____________________ Floodway ____________________ Floodway Fringe

Base (100-year) flood elevation(s) at the project site ______________________________

Elevation required for Lowest Floor ____________________ / Floodproofing ____________________

Source Document / Report / Maps ______________________________

____

Proposal Review Checklist

______ Site development plans depict the floodway and base flood elevations
______ Engineering data is provided for map and floodway revisions
______ Floodway certification and data document no increases in flood heights
______ Subdivision proposals minimize flood damage and protect utilities
______ Lowest floor elevations are above the base (100-year) flood level
______ Manufactured (mobile) homes are elevated and adequately anchored
______ Non-residential floodproofing designs meet NFIP watertight standards
______ Other ____

Continued on next form.

Flood Plain Development Permit, continued

Permit Action

______ **Permit Approved**: The information submitted for the proposed project was reviewed and is in compliance with approved flood plain management standards (site development plans are on file)

______ **Permit Denied**: The proposed project does not meet approved flood plain management standards (explanation is on file)

______ **Variance Granted**: A variance was granted form the base (100-year) flood elevations established by FEMA consistent with variance requirements of NFIP regulations Part 60.6 (variance action documentation is on file)

____

Flood Plain Administrator's Signature Date

Comments ____

____

Development Documentation

______ **Map Revision Data**: Certified documentation by a registered professional engineer of as-built conditions for flood plain alterations were received and submitted to FEMA for a flood insurance map revision.

______ **Fill Certification:** A community official certified the elevation, compaction, slope and slope protection for all fill placed in the flood plain consistent with NFIP regulations Part 65.5 for flood insurance map revisions.

______ **Elevation Certificate**: Certified as-built elevation of the building's lowest floor ____________________; floodproofing level _______________. An Elevation Certificate (Part II) completed by a registered professional engineer or land surveyor certifying this elevation is on file

______ **Certificate of Occupancy or Compliance Issued** ____________________
Date

CHAPTER 4

FLOODPLAIN ADMINISTRATION AND DELINEATION

SECTION 2.0

FLOODPLAIN DELINEATION

CHAPTER 4
FLOODPLAIN ADMINISTRATION AND DELINEATION

SECTION 2.0
FLOODPLAIN DELINEATION

TABLE OF CONTENTS

LIST OF FIGURES

CHAPTER 4
FLOODPLAIN ADMINISTRATION AND DELINEATION

SECTION 2.0
FLOODPLAIN DELINEATION

2.1 INTRODUCTION

In order to regulate existing and new floodplain improvements and to reduce the amount of future losses due to flooding, flood hazard areas should be clearly identified, studied, and delineated.

Many drainage-ways have been analyzed by various local, state, and federal agencies, and their floodplain delineations can be found on either the Flood Insurance Rate Maps (FIRMs) published by the Federal Emergency Management Agency (FEMA) or the community's floodplain maps. However, throughout the State, numerous floodplain areas that are subject to severe flooding have not yet been studied and delineated. As new developments occur in these undetermined flood hazard areas, local agencies and developers face the challenge of developing the flood hazard area information.

This section is intended to provide practical guidelines for delineation of flood hazard areas within the State of Colorado. Readers of this manual are encouraged to review the following publication for more detailed discussions on this subject:

- Federal Emergency Management Agency, NFIP Regulations, Title 44, Chapter 1, Part 65, Identification and Mapping of Special Hazard Areas, revised October 1999.
- Federal Emergency Management Agency, Flood Insurance Study Guidelines and Specifications for Study Contractors, March 1993.

2.2 LEVEL OF STUDY

Flood hazard areas can be delineated based on two different analysis approaches: detailed and limited methods. The detailed study approach should be used when accurate floodplain information including floodplain limits, water surface elevations and profiles, flood depths and velocities and, floodway limits, are needed for the drainage-way being studied. The limited study method may be used when detailed floodplain information is not necessary. The limited study usually results in the delineation of approximate flood hazard areas without base flood elevations.

Flood hazard areas can be delineated based on two different analysis approaches: detailed and limited methods.

The following factors should be considered when deciding which study approach to use for a drainageway being studied:

- Size of the contributing watershed
- Size and stability of the drainageway

CHAPTER 4
FLOODPLAIN ADMINISTRATION AND DELINEATION

SECTION 2.0
FLOODPLAIN DELINEATION

- Floodplain development pressure
- Existing and future floodplain encroachments
- Floodway delineation
- Watershed area plan
- FEMA submittals
- Flooding history

The detailed study approach should be used when a new development is proposed within or adjacent to the 100-year floodplain limits. The limited study method may be used to define the 100-year floodplain limits if no encroachment into the natural floodplain is proposed in a foreseeable future.

2.3 WATERSHED CONDITIONS

The FEMA 100-year floodplain and floodway boundaries shown on the Flood Insurance Rate Maps (FIRM) are delineated based on the existing watershed and floodplain conditions at the time of the Flood Insurance Studies (FIS).

To be consistent, hydrologic and hydraulic analyses for all new floodplain delineation studies should be performed, at a minimum, to reflect the existing watershed and floodplain conditions. Public works projects in progress that are planned to be completed within 12 months following the study completion should be included in the analysis. Where construction of a publicly owned, operated and maintained flood control facility will not be completed within 12 months following completion of the study, but adequate progress has been made, the impact/benefit of the project may be included in the hydrologic analysis. The project engineer should coordinate with the public agency in charge of the facility design and construction, effected local agencies and Colorado Water Conservation Board (CWCB) to determine whether to include the subject facility in the existing conditions analysis or not.

As new developments occur, the estimated existing conditions 100-year peak flow and associated floodplain limits may change depending on the nature and amount of new developments within a watershed. Therefore, local communities are encouraged to develop future (built-out) conditions floodplain information in addition to the existing conditions floodplains, especially when the area plan indicates substantial amount of future developments. Once developed, communities may request FEMA to show future conditions floodplains on the community's FIRM maps in addition to the existing conditions floodplains.

Hydrologic and hydraulic analyses for all new floodplain delineation studies should be performed, at a minimum, to reflect the existing watershed and floodplain conditions. Local communities are encouraged to develop future (built-out) conditions floodplain information in addition to the existing conditions floodplains.

2.4 TOPOGRAPHIC MAPPING

For detailed discussions and specifications of the topographic mapping standards, please refer to Chapter 2, Section 1.0.

2.5 FLOOD HAZARD ZONE DESIGNATIONS

Flood hazard zone designations are used to identify the level of study, severity of flooding conditions, type of flooding, and other floodplain information. The flood zone designations can be used by agencies to regulate their floodplain developments and by insurance agents in determining flood insurance rates for properties in the NFIP participating communities.

All new floodplains that are delineated to be shown on the Flood Insurance Rate Maps (FIRM) should be designated using the latest FEMA flood insurance rate zone designations. New floodplain delineations that are not prepared to be shown on the FIRM maps should be designated using the following flood zone designations.

100-year Floodplain (FEMA Zone AE)

100-year Floodplain boundaries are determined using the "detailed method" as described in Section 2.6, Chapter 6. The 100-year water surface elevations should be shown at a selected interval for this zone.

Approximate 100-year Floodplain (FEMA Zone A)

Approximate 100-year Floodplain boundaries are determined based on the "limited method" as described in Section 2.7, Chapter 6. Since no detailed hydraulic analyses are required for the limited method, the 100-year water surface elevations are not shown within this zone.

100-year Shallow Floodplain (FEMA Zone AO or Zone X)

100-year Shallow Floodplain boundaries are determined using the "detailed method" as described in Section 2.6, Chapter 6. Since it is often difficult to define the 100-year water surface elevations for areas of shallow flooding, average flood depths and limits (between 1 and 3 feet) should be shown instead. If the average flood depth is less than 1 foot, the floodplain should be designated as 100-year Shallow Floodplain (1 foot depth).

Special 100-year Floodplain

Special 100-year Floodplain boundaries are determined using the "detailed method" as described in Section 2.6, Chapter 6. The 100-year water surface elevations should be shown at a selected interval for this zone. The floodplain designation should clearly identify the type of flooding (mudflow, alluvial fan, fire area, ice flow, etc.)

500-year Floodplain

500-year Floodplain boundaries are determined using the "detailed method" as described in Section 2.6, Chapter 6. The 500-year water surface elevations need not be shown.

2.6 DETAILED FLOODPLAIN DELINEATION METHOD

Before proceeding with a detailed floodplain delineation study, the project engineer should evaluate the applicability of all available hydrologic and hydraulic studies for the subject watershed/drainage-way. The previously approved studies should be used whenever possible, unless the watershed/drainage-way conditions have changed substantially and/or the original analysis methodology was determined inappropriate.

2.6.1 HYDROLOGIC ANALYSIS

The hydrologic analysis shall include, at a minimum, calculations for the 10-, 50-, 100-, and 500-year frequency discharges. It is recommended that the peak discharge for 2- and 5-year flood events be calculated in addition to the other discharges. The 500-year flow rate may be estimated by multiplying the 100-year flow rate by a factor of 1.7 (FHWA, HEC-18).

2.6.1.1 METHODOLOGY

Peak discharge estimates for the floodplain delineation and administration purposes shall be determined by one or more of the following methods, depending on the length of systematic records available.

- Statistical analysis of stream gage data.
- USGS regression analysis
- Flood estimates using hydrologic models and precipitation records (synthetic analysis)

All available stream-flow data from the drainage basin and adjacent basins shall be inventoried and documented. Detailed hydrologic analysis procedures and standards can be found in Chapter 5, Hydrology. The following guidelines should be used to determine the appropriate hydrologic analysis method.

- When at least 50 years of stream-flow gage records are available, a flow frequency statistical analysis should be performed to determine the flood peaks of selected recurrence intervals.

- When 25 to 50 years of stream-flow records are available, the hydrologic analysis shall include a statistical analysis, and a comparison with similar watersheds. Similar watersheds are defined as watersheds that have similar hydrologic characteristic (precipitation depth and distribution, slope, size, elevation, vegetation cover, etc.) as the watershed being studied.

- When 10 to 24 years of stream-flow records are available, the hydrologic analysis shall include a statistical analysis, comparisons with similar watersheds, and flood estimates using synthetic hydrologic models and precipitation records.

All drainage basin characteristics that affect the rainfall-runoff relationship shall be documented, including, but not limited to, delineation of basin and subbasin boundaries, size, shape, length, slope, general aspect, elevation extremes, time of concentration, land use, and soil types and compositions.

When actual precipitation records of major recorded storm events are available from area weather stations, such data shall be used in conjunction with rainfall data.

- When less than 10 years of stream-flow records are available, the selected flood frequency flows shall be calculated by comparison to similar watersheds, USGS regression analysis, and by estimates using synthetic hydrologic models and precipitation records.

Depending on the floodplain analysis requirements, it may be necessary to develop a synthetic rainfall-runoff model even when sufficient amount of gage peak flow records are available. The following is a list of some of these cases:

- Various flood frequency hydrographs are required, but the statistical analysis alone cannot generate the necessary hydrographs.
- The subject watershed is undergoing or projected to undergo a substantial amount of new development.
- Comparison of before and after development hydrographs.

Whenever possible, synthetic rainfall-runoff models should be calibrated to match the statistical analysis results. The calibrated synthetic model can then be used to generate the hydrographs.

2.6.1.2 DETENTION

The hydrologic analysis should include detention facilities designed and constructed with the purpose of impounding water for flood detention that are owned, operated, and maintained by a government body. Detention structures that are randomly located, privately owned, or privately maintained shall not be included in the hydrologic analyses unless it can be shown that they exacerbate downstream peak discharges.

The hydrologic analysis should include detention facilities designed and constructed with the purpose of impounding water for flood detention that are owned, operated, and maintained by a government body.

If existing detention basins are not included in the hydrologic analysis, discussions should be provided in the report describing the detention basins and reasons why they were not considered in the analysis.

2.6.1.3 PREVIOUS STUDIES

Where appropriate, available flow frequency information should be used so that previous work by federal, state, or local agencies is not duplicated. Where such data is not available, where conditions have changed significantly, or where the methodologies or data used in previous studies are not appropriate, a new hydrologic analysis for each stream shall be prepared.

If a new hydrologic analysis is prepared, a comparison of new discharges with all available published or not published discharge data that exist for the study area shall be provided. If the new hydrologic analysis results are significantly different than the previously adopted flows, the following criteria should be used in deciding which flow estimate should be used. However, the site-specific limitations/conditions may warrant a deviation from the evaluation criteria below. The project engineer should coordinate with the appropriate agencies in deciding which flow estimate should be used.

- For drainage-ways with at least 50 years of stream-flow gage records, the following general FEMA evaluation criteria should be used.

  The latest discharges shall be adopted if the previously established discharges do not fall within the 95 and 5 percent confidence limits (90 percent confidence interval) of the most recent estimates; the previously established discharges shall be adopted if they fall within the 75 and 25 percent confidence limits (50 percent confidence interval) of the most recent estimates.

- For all other cases, the new hydrologic analysis results should be used if the new analysis is proven to be technically superior and if the resulting peak flow rate change is greater than 10%.

2.6.2 HYDRAULIC ANALYSIS

Hydraulic analysis should be performed to define, at a minimum, the water surface profiles for the 10-, 50-, 100-, and 500-year flood frequencies and the 100- and 500-year floodplain boundaries. The 10-year floodplain boundaries should also be delineated if the 10-year flows are not confined within the channel. The floodplain boundaries shown on the floodplain delineation maps shall be consistent with the calculated water surface profiles.

2.6.2.1 METHODOLOGY

The hydraulic analysis for detailed floodplain information should be based on the following methods, or any other method approved by CWCB and/or a federal agency, as appropriate.

a. Step-Backwater Method

Flood water surface profiles may be calculated by the standard step method employing the Bernoulli energy equation with energy losses due to friction evaluated with the Manning equation. Detailed riverine flood water surface elevations are usually determined utilizing hydraulic computer programs including HEC-2 and HEC-RAS.

b. Alternative Methods

Unusual site-specific conditions may require the use of specialized modeling techniques in order to correctly model and delineate the flood hazard areas. For these cases, an alternative hydraulic methodology may be used, provided it has been recommended for general use by a federal governmental agency or notable scientific body, is well documented and is available to the general user. In the case of a computer program, documentation shall include a published user's manual and a programmer's manual.

A list of computer hydraulic modeling programs approved by FEMA and CWCB can be found on the FEMA's and CWCB websites (www.fema.gov, http://cwcb.state.co.us/). The hydraulic modeling results should be calibrated to match the reliable flood data from previous flood events, if available, within 0.5 foot.

Natural riverine flood water surface profile for the purpose of floodplain delineation should be determined using subcritical flow regime calculations. Critical depth should be used for the natural stream reach where supercritical flow occurs. Supercritical flow modeling may be used for man-made channels designed to handle supercritical flows.

For riverine reaches not effected by backwater, the starting water surface elevation should be estimated using normal depth calculations unless a known water surface elevation for the starting cross section can be obtained from an existing model or previous flood events.

Recommended Manning's "n" values for various channel and floodplain conditions can be found in Table CH6-T102. Manning's roughness coefficients should be estimated considering the following factors:

- Channel bed materials
- Type, density, and height of existing vegetations
- Existing structures in the overbanks
- Roughness variations with different flow depths
- Channel maintenance operations
- Past flood data

Past flood data, if available, should be used to calibrate roughness coefficients, taking into consideration any alteration in the channel subsequent to the floods. The calibrated roughness coefficients should closely match the observed channel and floodplain conditions. Photographs shall be taken of the study reaches of the stream channel and floodplain to support roughness coefficients used for hydraulic computations.

2.6.2.2 CROSS SECTIONS

The riverine cross-section data should be obtained by photogrammetric methods at the time of map compilation, from the map contours and spot elevations, or through field surveys. All cross section points derived photogrammetrically or from the contours shall meet the accuracy standards defined in Chapter 2. All field-surveyed cross section points shall be within ±0.5 foot of true elevations.

Cross sections shall be located perpendicular to the direction of flow at appreciable changes in flow area, roughness, or stream gradient. Additional cross sections shall be located at bridges and culverts, the head and tail of levees, confluences with tributaries, and all flow control structures. Based on computed results, additional cross sections may be required if the slope of the energy grade line between successive cross sections decreases by more than 50 percent or increases by more than 100 percent. The location of all cross-sections shall be shown on the floodplain delineation maps.

2.6.2.3 BLOCKAGE

All culverts and bridges shall be considered for the potential to become blocked by floating debris and sediment loads. In determining the potential for blockage, and subsequent reduction in the flow conveyance capacity, the following factors should be considered:

- Old photographs
- History of maintenance during high flows
- Ongoing maintenance operations
- Watershed characteristics such as erodibility of channel banks
- Amount and type of vegetation along stream
- Size and characteristics of the waterway

Blockage may be accounted for in computer runs by increasing width of piers, raising streambed elevation or reducing waterway opening by a percentage. Where the potential for blockage can be shown, human intervention (e.g., snagging) shall be considered only if such flood fighting activities are specifically included in the community's adopted emergency response plan.

2.6.2.4 PREVIOUS STUDIES

A comparison of any proposed 100-year flood profile with all previously designated and approved information shall be provided. Except where a clearly identified change in flooding characteristics or error in the existing data can be shown, the proposed 100-year flood elevations shall agree with those of other contiguous studies on the same stream. Elevations of cross sections shall be computed to match within +/- 0.5 foot of an existing valid elevation; however, the final published 100-year floodwater surface profile shall be drawn to match the contiguous study exactly. Where elevations cannot be reconciled to within +/- 0.5 foot because of changed flooding conditions or an error in the previous analysis, a full explanation and justification for the difference shall be provided.

2.6.3 FLOODPLAIN DELINEATION

The floodplain boundaries shall be delineated based on one of the following methods:

a. Flood Contour Method

A reference line shall be shown down the center of the low flow channel on the floodplain delineation map for all streams studied using the detailed method. Flood contours (BFE lines) derived from the computed water surface profile shall be used to define the boundaries of the 100-year floodplain.

The flood contours shall have a vertical interval equal to the contour interval of the floodplain delineation map if the following criteria are met.

- The average slope of the computed water surface profile between cross sections is flatter than one percent,
- or the width of the floodplain is greater than 200 feet,
- or there are, in judgment of the engineer, unusual topographic features.

Alternate flood contour interval may be used in lieu of the above flood contour interval if the slope of the water surface profile, in combination with the contour interval and map scale, would result in an average horizontal spacing between flood contours of less than 1 inch.

b. Map Contour Method

A reference line shall be shown down the center of the low flow channel on the floodplain delineation map for all streams studied using the detailed method. For the channel and floodplain cross sections used in the hydraulic analysis, the 100-year floodwater surface elevations and floodplain horizontal limits should be computed. The 100-year flood boundaries shall be delineated between these cross sectional locations by transposing the water surface elevations to the topographic map. These intermediate locations must be correlated to the reference line and the 100-year flood profile as to their specific location and elevation.

2.6.4 FLOODWAY DELINEATION

The floodway is defined as the channel, plus any adjacent floodplain areas, that must be kept free of encroachment so that the 100-year discharge can be conveyed with no more than one (1) foot rise in the water surface above the base flood elevations (BFE). However, communities may choose to adopt stricter floodway delineation criteria by allowing only 0.5 foot rise or no rise above BFE. The floodway represents the community's regulatory limit of encroachment into the 100-year floodplain for those watercourses with the established floodway boundaries. The project engineer should coordinate with the effected local agencies and CWCB to determine whether a floodway should be delineated for the drainage-way being studied or not.

Communities may choose to adopt stricter floodway delineation criteria by allowing only 0.5 foot rise or no rise above BFE as opposed to the one (1) foot rise. Floodway limits should be determined based on the "equal conveyance reduction" method.

Floodway limits should be determined based on the "equal conveyance reduction" method. This method reduces an equal amount of flow conveyance from both overbanks allowing potential development areas on both sides of the waterway. The floodway delineation should be clearly shown on the floodplain delineation map in addition to the floodplain boundaries.

2.7 LIMITED FLOODPLAIN DELINEATION METHOD

The limited method results in the delineation of approximate 100-year floodplain boundaries without base flood elevations (BFEs). This approach is usually used to delineate approximate 100-year floodplain boundaries for drainageways in the vicinity of future developments to inform the communities about the potential flood hazards.

If new developments are proposed to encroach into or located adjacent to a previously delineated approximate 100-year floodplain, the previously delineated approximate floodplain limits should be restudied by using the detailed approach before the developments can occur. The hydrologic and hydraulic analysis should include, at a minimum, determination of 100-year event discharge and floodplain limits.

2.7.1 HYDROLOGIC ANALYSIS

The hydrologic analysis should be based on the analysis methods and guidelines previously provided in Section 2.6.1, Chapter 4.

2.7.2 TOPOGRAPHIC INFORMATION

The best available topographic base map shall be used, to develop approximate floodplain information. Such work map shall, at a minimum, be the most recent edition of a 7.5 minute quadrangle as published by the U.S.

Geological Survey (USGS). Unless the available topographic map has contour interval of 4 feet or less, field surveyed channel and floodplain cross sections should be used in the hydraulic analysis.

2.7.3 HYDRAULIC ANALYSIS

The following hydraulic analysis method or more detailed methods should be used for delineation of approximate 100-year floodplain boundaries:

- Calculate 100-year water surface elevations for the cross sections that are representative of the stream reach being studied using normal-depth calculations (Manning's Equation). A sufficient amount of cross-sections should be used to adequately represent and analyze the physical features (bridges, levees, etc.) of the drainageway. The published culvert and bridge rating charts may be used.

2.7.4 FLOODPLAIN DELINEATION

Approximate 100-year floodplain boundaries should be delineated based on the calculated 100-year water surface elevations for the representative cross sections in conjunction with the best available topographic map. If surveyed high water marks from a previous flood event, close to a 100-year event, are available for the drainageway, the high water mark elevations may be used to supplement the computed 100-year water surface elevations.

2.8 AREAS PROTECTED BY LEVEES

In order for a levee system to be recognized as providing flood protections, the levee must be structurally sound and adequately maintained. Certification from a federal or state agency that the levee meets the minimum freeboard criteria and that it appears, on visual inspection, to be structurally sound and adequately maintained will be required. Levees that have obvious structural defects, or that are obviously lacking in proper maintenance, shall not be considered in the hydraulic analysis.

Detailed discussions on the levee freeboard, ownership, design, operations and maintenance, and certification requirements are provided in Chapter 6, Section 4.0.

2.8.1 FLOODPLAIN ANALYSIS

The natural floodplain areas protected from a 100-year event by a levee system that meets the requirements outlined in Chapter 6, Section 4.0 can be designated as 100-year Shallow Floodplain with 1 foot depth (FEMA Zone X). However, areas inundated by the interior drainage behind the levees should be defined, and if necessary, the 100-year water surface elevations, flooding limits and depths, special hazard zones should be clearly identified.

In order for a levee system to be recognized as providing flood protections, the levee must be structurally sound and adequately maintained. Levees that have obvious structural defects, or that are obviously lacking in proper maintenance, shall not be considered in the hydraulic analysis.

If levees protecting the subject area do not meet the requirements outlined in Chapter 6, Section 4.0, the 100-year flood elevations of the protected area should be computed as if the levees did not exist. For the unprotected areas between the levee and the source of flooding, the 100-year flood elevations should be obtained from either the flood profile computed with the levees in place or the profile computed as if the levees did not exist, whichever is higher. This procedure recognizes the increase in flood elevation in the unprotected area caused by the levees. This procedure may result in the 100-year flood elevations being shown as several feet higher on one side of the levee than on the other. Both profiles should be shown in the final delineation with a line drawn along the levee centerline separating the areas with different BFEs.

If the calculated water surface elevation (WSEL) of the unprotected area for other frequency events (10-, 50-, and 500-year events) is lower than the top of levee elevation, the computed WSEL should be used. If the computed WSEL is higher than the top of levee, than the top of levee elevation should be used as the WSEL for the unprotected area. Floodplain delineation analysis (except the internal drainage analysis) of the protected area for flood events less than a 100-year event is not required.

If levees exist on both sides of a drainage-way, several levee failure scenarios should be considered including simultaneous levee failure, left levee only failure, and right levee only failure scenarios. For more detailed discussion on the levee floodplain and floodway delineation analysis, please refer to the FEMA publications.

Where credit will be given to levees providing 100-year protection, the adequacy of interior drainage systems shall be evaluated. Areas subject to flooding from inadequate interior drainage behind levees will be mapped using standard procedures.

2.9 AREAS PROTECTED BY DAMS

2.9.1 FLOOD CONTROL DAMS

If a publicly owned, operated and maintained dam is specifically designed and operated, either in whole or in part, for flood control purposes, then its effects shall be taken into consideration when delineating the floodplain below such a dam. Full credit should be given to the diminution of peak flood discharges, which would result from normal dam operating procedures.

2.9.2 NON-FLOOD CONTROL DAMS

If a dam is not specifically designed and operated, either in whole or in part, for flood control purposes, then its effects, even if it provides inadvertent flood routing capabilities which reduce the 100-year flood downstream, shall not be taken into account and the delineation of the floodplain below such a dam shall be based upon the 100-year flood that would occur absent of the dam. However, if adequate assurances have been obtained to preserve the flood routing capabilities of such a dam, then the delineation of the floodplain below the dam may, but need not, be based on the assumption that the reservoir formed by the dam will be filled to the elevation of the dam's emergency spillway. The project engineer should coordinate with appropriate government agencies and CWCB in determining whether a non-flood control dam should be included in the analysis or not.

If existing dams are not included in the hydrologic analysis, discussions should be provided in the report describing the dams and reasons why they were not considered in the analysis.

2.10 ALLUVIAL FAN FLOODING

Alluvial fan flooding is quite different than a riverine flooding, and consequently, the alluvial fan floodplains should be studied and delineated based on a different set of criteria. Alluvial fan flooding can be characterized by unpredictable flow paths, mud-flows, high flow velocity, and erosion and sediment deposition. Alluvial fans typically do not have a well-defined channel capable of conveying a 100-year flows, although, it is not unusual to have smaller defined channel(s). Typically, flood flows do not spread over the entire alluvial fan surface, but are conveyed down from the apex to the toe of the fan by a network of old and new flow paths/channels.

For detailed discussions on the floodplain analysis of active or semi-active alluvial fans, readers are referred to the following publications:

- National Research Council, Alluvial Fan Flooding, 1996
- Federal Emergency Management Agency, Guidelines for Determining Flood Hazards on Alluvial Fans, February 23, 2000
- Federal Emergency Management Agency, FAN, An Alluvial Fan Flooding Computer Program & User's Manual, September 1990.

2.11 FLOODPLAIN DELINEATION STUDY REPORT

A floodplain delineation study is required in order to delineate and designate new floodplains or modify existing floodplains. All floodplain information shall be developed by a qualified hydrologist or hydraulic engineer under the direct supervision of a professional Civil Engineer registered in the State of Colorado, or by an employee of a state or federal government agency that has executed a memorandum of understanding or other written agreement with CWCB. The memorandum of understanding shall be based on a written statement, which demonstrates to the satisfaction of CWCB their equivalent qualifications to perform such work in Colorado.

The floodplain delineation study shall be professionally prepared, legible and able to be reproduced.

2.11.1 OUTLINE FOR A FLOODPLAIN DELINEATION STUDY

In general, the following points should be addressed through actual analysis or through reference to adopted studies:

1. A description of the floodplain and channel areas (i.e. vegetation, slopes, constrictions & etc.).

2. A description of the contributing drainage basin(s).

3. Identification of previous significant flood events, applicable floodplain studies, and FEMA Flood Insurance Studies with analysis of the applicability of data to the subject area.

4. Hydrologic analysis.

5. A description of the method of hydraulic analysis and its application in the study. The hydraulic calculations for each frequency flood shall be summarized in a frequency-elevation table or, in lieu of the table, may be shown on the graph of water surface profiles. The table shall include, at a minimum, the stream station for each cross section, cross section identification, peak discharges, and water surface elevations.

6. Identification and discussion of all input parameters and basis for input parameters.

7. Discussion of the analysis results, comparison with existing studies, and conclusions of the hydrologic and hydraulic analysis. This shall include a narrative summary of the results as well as comprehensive output data.

8. The delineation of the floodplain and floodway boundaries and water surface profiles. Include cross section locations.

9. If appropriate, an analysis of sediment transport and fluvial morphology.

2.11.2 FLOODPLAIN DELINEATION MAP

Floodplain delineation maps should be included in the report. The floodplain delineation maps should show the floodplain information including the floodplain/floodway boundaries, the location of all cross sections used in the hydraulic analysis, a reference line drawn down the center of the low flow channel, and a sufficient number of flood contours in order to reconstruct the flood water surface profiles to an accuracy of ±0.5 foot. Flood contours shall be shown as wavy lines drawn normal to the direction of flow of floodwater and shall extend completely across the area of the 100-year floodplain. Each flood contour shall indicate its elevation to the nearest whole foot. An example floodplain delineation map is shown on Figure CH4-F201.

The floodplain delineation map scale for detailed studies should be 1-inch equals 400 feet or such map scale as showing sufficient detail.

:/2080/FIGURES, CH4-F201.DWG, 4/10/02 NAS

VERSION: AUGUST 2002	REFERENCE:	FIGURE CH4-F201
WRC ENGINEERING, INC.		EXAMPLE: FLOOD PLAIN DELINEATION MAP

CHAPTER 4

FLOODPLAIN ADMINISTRATION AND DELINEATION

SECTION 3.0

FEMA MAP REVISIONS

CHAPTER 4
FLOODPLAIN ADMINISTRATION AND DELINEATION

SECTION 3.0
FEMA MAP REVISIONS

TABLE OF CONTENTS

CHAPTER 4
FLOODPLAIN ADMINISTRATION AND DELINEATION

SECTION 3.0
FEMA MAP REVISIONS

3.1 INTRODUCTION

The Federal Emergency Management Agency (FEMA) has been conducting Flood Insurance Studies (FISs) and restudies to identify and delineate flood hazard areas for the National Flood Insurance Program (NFIP) participating communities. Many drainageways within the State of Colorado have been studied by FEMA, and the FEMA designated floodplains are shown on the Flood Insurance Rate Maps (FIRMs).

The FIRM maps are used by local governmental agencies in regulating the floodplain developments and by insurance agents in determining flood insurance rates for properties located within the FEMA flood hazard zones. The floodplain development regulation criteria and the insurance rate are determined based on the flood hazard zone designation on the subject property. Therefore, it is important to create and maintain accurate floodplain information on the FIRM maps. The designated FEMA floodplains and floodways can be modified by submitting appropriate map revision requests for the following general cases:

- Errors in the original floodplain delineation study
- Substantial change in the drainageway hydrology and hydraulics
- Physical modifications that change the flooding conditions
- Floodplain restudy using detailed modeling methods

This section is intended to provide practical guidelines to help local agencies and engineers in selecting an appropriate FEMA Map revision process for a given situation.

This section is intended to provide practical guidelines to help local agencies and engineers in selecting an appropriate FEMA Map revision process for a given situation. For detailed discussions on the map revision submittal requirements, readers are referred to the following FEMA publications:

- Federal Emergency Management Agency, Appeals, Revisions and Amendments to Flood Insurance Maps, A guidebook for Local Officials (FIA-12)
- Federal Emergency Management Agency, NFIP Regulations, Title 44, Chapter 1, Parts 60, 65, 70, and 72, revised October 1999.

3.2 FEMA MAP REVISION REQUEST SUMMITTALS

NFIP participating communities are required to make map revision request submittals to FEMA for projects and developments that modify the FEMA designated floodplains and floodways. Depending on the type and extent of proposed improvements, the applicant can submit a request for one or more of the following FEMA map revisions:

- Conditional Letter of Map Amendment (CLOMA) – "A letter from FEMA stating that a proposed structure that is not to be elevated by fill would not be inundated by the base (100-year) flood if built as proposed" (FEMA Form MT-1).

- Letter of Map Amendment (LOMA) – "A letter from FEMA stating that an existing structure or parcel of land that has not been elevated by fill would not be inundated by the base (100-year) flood" (FEMA Form MT-1).

- Conditional Letter of Map Revision based on Fill (CLOMR-F) – "A letter from FEMA stating that a parcel of land or proposed structure that is to be elevated by fill would not be inundated by the base (100-year) flood if fill is placed on the parcel as proposed or the structure is built as proposed" (FEMA Form MT-1).

- Letter of Map Revision based on Fill (LOMR-F) – "A letter from FEMA stating that an existing structure or parcel of land that has been elevated by fill would not be inundated by the base (100-year) flood" (FEMA Form MT-1).

- Conditional Letter of Map Revision (CLOMR) – " A letter from FEMA commenting on whether a proposed project, if built as proposed, would justify a map revision or proposed hydrology changes" (FEMA Form MT-2).

- Letter of Map Revision (LOMR) – " A letter from FEMA officially revising the current NFIP map to show changes to floodplains, floodways, or flood elevations" (FEMA form MT-2).

The applicant submitting a map revision request is required to fill out and include appropriate FEMA forms. FEMA utilizes these standard forms to guide the applicant in preparing the necessary information and technical data for the revision request submittal. The applicant and affected communities are required to sign the appropriate FEMA forms. FEMA form "MT-1" should be used for CLOMA, LOMA, CLOMR based on Fill, and LOMR based on Fill request submittals and form "MT-2" should be used for CLOMR and LOMR submittals. The applicant is also required to submit a review/processing fee associated with the revision request. The current FEMA forms and fee schedule can be obtained from the local floodplain administrator or directly from FEMA's website (www.fema.gov).

Issuance of a conditional approval by FEMA does not remove structures and/or properties from the FEMA flood hazard zone. After the completion of construction, the applicant must submit an additional request for LOMA or LOMR following the issuance of CLOMA or CLOMR to officially remove the subject lot and/or structure from the FEMA SFHA.

Issuance of a conditional approval (CLOMA, CLOMR based on fill, or CLOMR) by FEMA does not remove structures and/or properties from the FEMA flood hazard zone. After the completion of construction, the applicant must submit an additional request for LOMA or LOMR following the issuance of CLOMA or CLOMR to officially remove the subject lot and/or structure from the FEMA SFHA. However, by submitting for a CLOMA or CLOMR, the applicant can get a written assessment from FEMA on the proposed project plan. This process allows the applicant to modify the project design if required to do so by local agencies, CWCB, or FEMA, prior to construction. Local agencies and CWCB may request or the applicant may choose to obtain a CLOMA or CLOMR, if the site-specific issues and conditions warrant the additional submittal. The applicant should coordinate with the local floodplain administrator in determining whether or not a conditional revision request should be submitted for the subject property. The following factors should be considered:

- Size and complexity of the proposed improvements
- Change in the drainageway hydrology and/or hydraulics
- Modification of the regulatory floodway boundary
- Potential adverse impacts to adjacent properties
- New development in FEMA Zone A (BFEs not defined)
- Alluvial fan flooding

3.2.1 SUBMITTAL PROCESS

A request to modify the FEMA floodplain and/or floodway must be reviewed by the affected local government agencies and the Colorado Water Conservation Board (CWCB) before submitting to FEMA. The applicant should prepare a complete revision request report, including FEMA forms, based on the submittal requirements outlined in the FEMA publications previously referenced in Section 3.1. The following list summarizes the FEMA map revision submittal process.

a. If required as a condition of approval or desired by the applicant, a request for CLOMA, CLOMR-F, or CLOMR report should be prepared and submitted to the affected local agencies for their review and comments. The applicant should submit a copy of the report to all local agencies affected by the proposed floodplain modifications. The review comments should be addressed prior to submitting the report to CWCB. Community Acknowledgement Forms signed by the local agencies should be included in the CWCB and FEMA submittals.

b. Once the application has been reviewed by the local agencies, the applicant should submit the updated revision request report to CWCB. The review comments provided by CWCB should be addressed prior to submitting the report to FEMA.

c. The applicant should submit the conditional map revision request report to FEMA along with the required FEMA review/processing fee and forms. FEMA review procedures are discussed in detail in Section 3.5, Chapter 4.

d. Once the application has been reviewed and approved by FEMA, a CLOMA, CLOMR-F, or CLOMR will be issued by FEMA for the proposed project.

e. The applicant should design and construct the necessary drainage improvement facilities and prepare "as-built" drawings.

f. Repeat steps "a" though "d" to obtain a LOMA, LOMR-F or LOMR from FEMA to officially remove the property from the FEMA SFHA.

3.3 LETTER OF MAP AMENDMENT (LOMA)

Since a LOMA essentially is a correction of previous mapping errors, there is no FEMA review fee for a LOMA request. For a lot to be removed from the SFHA, the lowest lot ground elevation must be at or above the 100-year flood elevation. For a structure to be removed, the lowest floor elevation (including basement and/or crawl space) and the lowest adjacent ground touching the structure must be at or above the 100-year flood elevation.

The regulatory Special Flood Hazard Areas (SFHAs) designated by FEMA are determined as a result of engineering studies utilizing FEMA approved floodplain analysis methods and topographic maps. Depending on the accuracy of physical features and elevations shown on the topographic maps used in their studies, some properties that are naturally (or by fill placed prior to the FEMA original study) at or above the 100-year flood elevation might be incorrectly shown within the SFHA. In such a case, the property owner can request FEMA to issue a LOMA to remove his or her property from the SFHA.

The applicant should prepare a request for LOMA submittal including the required property and elevation information and FEMA forms MT-1 or MT-EZ. Detailed LOMA submittal requirements are outlined in the FEMA publications referenced in Section 3.1, Chapter 4 and the FEMA forms. A request for LOMA can be submitted for a single lot/structure using Form MT-EZ or multiple lots/structures using Form MT-1. Since a LOMA essentially is a correction of previous mapping errors, there is no FEMA review fee for a LOMA request. The following is a list of general LOMA submittal requirements:

- FEMA forms – MT-1 or MT-EZ
- Property information – a copy of recorded Plat Map or recorded deed and other map that shows the property boundary and physical features (streets, buildings, drainageways, etc.) surrounding the property
- Lot and/or structure elevations certified by a licensed Professional Engineer or Land Surveyor

An Elevation Information or Elevation Certificate Form for the property must be filled out and certified by a Professional Civil Engineer or a Land Surveyor licensed to practice in the State of Colorado. FEMA compares the certified elevations provided in the form with the effective 100-year flood elevation to determine if the subject property can be removed from the SFHA. For a lot to be removed from the SFHA, the lowest lot ground elevation must be at or above the 100-year flood elevation. For a structure to be removed, the lowest floor elevation (including basement and/or crawl space) and the lowest adjacent ground touching the structure must be at or above the 100-year flood elevation.

The issuance of a LOMA officially removes the subject lot and/or structure from the FEMA SFHA, eliminating the FEMA's flood insurance requirement. However, it should be noted that some lending institutions might still require purchase of flood insurance regardless of the FEMA flood hazard zone designation. Unless the entire lot is elevated at least one (1) foot above the 100-year base flood elevation (BFE) or the lowest floor (including basement and/or crawl space) is elevated at least 1 foot above the BFE, it is strongly recommended that the property be insured with the flood insurance.

If the revision request is based on fill placed after FEMA's original floodplain study, the applicant should submit a request for LOMR based on fill. If the amendment request involves regulatory floodway boundaries, alluvial fan floodplains, flood conveyance improvements, and changes in the base flood elevations (BFE), a request for LOMR using FEMA forms MT-2 should be submitted.

3.4 LETTER OF MAP REVISION BASED ON FILL (LOMR-F)

Placement of fill to elevate areas above the 100-year flood elevation within the flood fringe may be allowed by the local agency. Flood fringe is the area between the adopted 100-year floodway and floodplain boundaries. Placement of fill within a 100-year floodplain without an established floodway should only be allowed based on the floodplain management criteria outlined in Chapter 4, Section 1.0. Construction of new building structures in the floodway is not permitted.

A request for LOMR-F should be submitted to remove the property, elevated sufficiently above the 100-year flood elevation by fill, from the SFHA. Detailed LOMR-F submittal requirements are outlined in the FEMA publications referenced in Section 3.1 and the FEMA forms. A request for LOMR-F can be submitted for a single lot/structure using Form MT-EZ or multiple lots/structures using Form MT-1. The following is a list of general LOMR-F submittal requirements:

- FEMA forms – MT-1 or MT-EZ
- Property information – a copy of recorded Plat Map or recorded deed and other map that shows the property boundary and physical features (streets, buildings, drainageways, & etc.) surrounding the property
- Lot and/or structure elevations certified by a licensed Professional Civil Engineer or a Professional Land Surveyor
- A signed Community Acknowledgment or Requests Involving Fill Form.

An Elevation Information or Elevation Certificate Form for the property must be filled out and certified by a Professional Engineer or a Land Surveyor licensed to practice in the State of Colorado. FEMA compares the certified elevations provided in the form with the effective 100-year flood elevation to determine if the subject property can be removed from the SFHA.

In the State of Colorado, to remove the entire lot from the SFHA, fill should be placed to elevate the entire lot, so that the lowest point of the lot is at least one (1) foot above the 100-year flood elevation. For just a structure to be removed from the SFHA, fill should be placed to elevate the lowest floor (including basement and/or crawl space) of the structure to meet the requirements specified in Chapter 4, Section 1.9. Also, the lowest adjacent ground touching the structure must be at least one (1) foot above the 100-year flood elevation. It is strongly recommended that basements should not be constructed on fill. If the entire lot is elevated at least

To remove the entire lot from the SFHA, fill should be placed to elevate the entire lot, so that the lowest point of the lot is at least one (1) foot above the 100-year flood elevation. It is strongly recommended that basements should not be constructed on fill.

one (1) foot above the 100-year flood elevation, but the structure is within 50 feet from the SFHA, the lowest floor of the structure should meet the requirements specified in Chapter 4, Section 1.9.

If the revision request involves regulatory floodway boundaries, alluvial fan floodplains, flood conveyance improvements, and changes in the base flood elevations (BFE), a request for LOMR using FEMA forms MT-2 should be submitted.

If desired by the applicant or required by the local agency, a request for Conditional Letter of Map Revision based on Fill (CLOMR-F) should be submitted based on the conceptual project site design. The submittal requirement for a request for CLOMR-F is same as LOMR-F. A request for LOMR-F should be submitted following the CLOMR-F using the "as-built" project information.

3.5 LETTER OF MAP REVISION (LOMR)

FEMA map revision requests that involve new hydrologic and hydraulic analyses, regulatory floodway boundaries, alluvial fan floodplains, flood conveyance improvements, and changes in the base flood elevations (BFE), a request for LOMR using FEMA forms MT-2 should be submitted. The applicant has the option to submit a request for CLOMR before the project is built and then follow the CLOMR with a LOMR request, or wait until the project is completed and submit a request for LOMR without a CLOMR. The main difference between CLOMR and LOMR request submittals is that a CLOMR request can be submitted based on preliminary project design plans but a LOMR request must be based on "as-built" drawings. For projects that involve a considerable amount of drainageway modifications or complex drainageway hydraulics, it is recommended that the applicant choose the request for CLOMR option, since that process will allow the requester to modify the project design, if required to do so by FEMA prior to construction.

For projects that involve a considerable amount of drainageway modifications or complex drainageway hydraulics, it is recommended that the applicant choose the request for CLOMR option, since that process will allow the requester to modify the project design, if required to do so by FEMA prior to construction.

3.5.1 REQUEST FOR A CONDITIONAL LETTER OF MAP REVISION (CLOMR)

The CLOMR process is intended to allow local, state, and FEMA review of a project before construction to assure that the proposed project will be consistent with the local, state, and FEMA floodplain management standards. A CLOMR may be required by the local community prior to formal project approval if the project has unusual or complex floodplain mapping conditions that could result in alternate mapping procedures or requirements to be imposed by FEMA. In this instance, it will be necessary to obtain concurrence from FEMA before the final design and construction in order to assure that FEMA will accept the method of analysis and/or proposed improvements for a map revision and change in flood insurance requirements.

A CLOMR request must be prepared in accordance with the adopted FEMA guidelines and must be accompanied by the standard FEMA certification forms MT-2 and a Community Acknowledgment Form. Copies of these forms can be obtained from the Floodplain Administrator or directly from FEMA's website.

3.5.1.1 CLOMR REVIEW PROCEDURES

The submittal requirements for a request for CLOMR are outlined in the FEMA publications referenced in Section 3.1 and the FEMA forms MT-2. The analyses, maps, plans, and drawings should reflect the conditions after the proposed activity takes place. Consequently, the required information cannot be certified "as-built". A request for a CLOMR must clearly demonstrate how the proposed model will tie into existing FEMA models both upstream and downstream of the proposed activity.

The affected local government agencies should review the CLOMR submittal first. The applicant should address the review comments to the satisfaction of the local agencies and submit a revised CLOMR submittal to CWCB for their review and approval as well. Local and CWCB comments must be resolved prior to submittal to FEMA. If the required supporting forms, data, or fee/fee waiver have not been provided, FEMA will send a letter to the party that submitted the request. This letter will identify any forms, supporting data, or fee that the requester has not submitted. Until the requested forms, data, and fee are submitted, FEMA will not undertake a detailed review of the request and will not take any further action concerning the request.

If FEMA determines from its preliminary review that the basic supporting data has been provided and that either the required initial fee has been provided or the fee requirement has been waived, FEMA will then inform the requester of the amount of time that FEMA will need to complete a detailed review of the request and supporting data. After completing its detailed review, FEMA will inform the requester by letter of any additional supporting data that must be submitted as appropriate.

Once all required data has been provided, FEMA will complete its review and make a determination concerning the effects of the proposed modifications. FEMA will issue either a CLOMR, which describes the changes that could be made to the FIRM and/or FBFM after the proposed modifications are completed, or a letter that explains why FEMA could not recognize the effects of the proposed modifications if those modifications were completed as planned.

Before issuing the determination, FEMA will determine if all fees needed to cover review costs have been received. If additional fees are needed, FEMA will send an invoice letter to the requester. In such cases, the determination will not be issued until the required fees have been received by FEMA. All FEMA review fees must be paid by the applicant.

A CLOMR approval by FEMA is only a conceptual approval of the project concept and does not modify the floodplain delineation shown on the FEMA Flood Insurance Rate Map or the flood insurance requirements.

3.5.2 REQUEST FOR LETTER OF MAP REVISION (LOMR)

After the drainage system for the project has been constructed, the "as-built" plans of the facilities should be submitted to FEMA as a Letter of Map Revision (LOMR) request with the supporting hydrologic, hydraulic, geomorphologic, and other data necessary to reflect effects of the floodplain modifications. The request for LOMR is submitted based on the effects of physical changes that have occurred in the floodplain or on the use of alternative methodologies that are technically superior.

The request for LOMR is submitted based on the effects of physical changes that have occurred in the floodplain or on the use of alternative methodologies that are technically superior.

The procedures for a LOMR review are similar to those for a CLOMR review. The specific requirements are outlined in the FEMA certification forms MT-2 and the FEMA publications referenced in Section 3.1. A copy of these procedures can be obtained from the local Floodplain Administrator or FEMA website.

The LOMR request can be submitted to FEMA without obtaining a CLOMR in advance for the project. However, for complex projects, it is recommended that the applicant submit and obtain a CLOMR prior to construction since that process will allow FEMA to review the proposed floodplain modifications and make comments to the requester, if necessary.

Chapter 5

CHAPTER 5

HYDROLOGY

CHAPTER 5
HYDROLOGY

SECTION 1.0
RAINFALL

CHAPTER 5
HYDROLOGY

SECTION 1.0
RAINFALL

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

CHAPTER 5
HYDROLOGY

SECTION 1.0
RAINFALL

1.1 INTRODUCTION

Due to the complexity of the natural terrain, orographic effects of the Rocky Mountains, and semi-arid climate of the region, the type and duration of rainstorms can vary substantially within the State of Colorado. However, rainstorm events can be generally defined as short duration convective storms or long duration general rainstorms.

The short duration convective storms (cloudbursts/thunderstorms) can produce high rainfall intensities for a short period and generally cover watershed areas of less than 200 square miles. Convective storms are commonly known to be responsible for high peak flows and flooding problems for many small drainage basins. The long duration general rainstorms can produce rain coverage over a large watershed area for a period in excess of six hours up to several days. General rainstorms can produce large amount of total rainfall runoffs and sometimes generate higher peak flows than the convective storms. Depending on the purpose of the hydrologic analysis, it may be necessary to analyze both types of rainstorms in order to estimate the high peak flow rate and the high runoff volume for a given drainage basin.

The information presented in this section is the state-of-art information available at the time of preparation of this manual and should be updated as better techniques and new rainfall data become available in the future.

1.2 RAINFALL DATA

The rainfall data published by National Oceanic and Atmospheric Administration (NOAA) in their "Precipitation-Frequency Atlas of the Western United States, Volume III – Colorado, 1973" should be used to perform necessary hydrologic calculations within the State of Colorado, unless site-specific rainfall studies have been performed and adopted by the local government agency having jurisdiction over the study area.

The NOAA Atlas 6-hour and 24-hour precipitation frequency maps for various storm events for the State of Colorado are included as Figures CH5-F101 through CH5-F112. The 6-hr and 24-hr point precipitation values can be estimated directly from Figures CH5-F101 through CH5-F112, and these point rainfall values can then be used to develop 5-minute, 10-minute, 15-minute, 30-minute, 1-hour, 2-hour, 3-hour, and 12-hour rainfall depths using the procedures outlined in this section.

The Rational Method can be used for drainage basins with a total contributing area of less than 160 acres.

For drainage basins with an area greater than 90 acres, CHUP and/or HEC-1 computer models should be used to estimate the runoff data.

The methodology used to generate the rainfall-runoff data will depend on the size of the drainage basin to be studied. The Rational Method for determining runoff is widely accepted as providing a sufficient level of detail for generating runoff from relatively small basins and can be used for drainage basins with a total contributing area of less than 160 acres. The Rational Method utilizes rainfall data in the form of time-intensity-frequency curves.

Since the assumptions used in the Rational Method become less valid for larger areas, larger basins require a more rigorous analysis to generate runoff data. For drainage basins with an area greater than 90 acres, CUHP and/or HEC-1 computer models should be used to estimate the runoff data. For drainage basins with an area between 90 acres and 160 acres, either the Rational Method or the computer models may be used.

For drainage basins with an area between 90 acres and 160 acres, either the Rational Method or a computer model (HEC-1 or CUHP) mav be used.

The CUHP (Colorado Urban Hydrograph Procedure) method has been used widely within the Urban Drainage and Flood Control District (UDFCD) jurisdictional area. The CUHP method was developed and calibrated to effectively model short duration convective storms within the Denver Metro area. Therefore, the CUHP method should only be used for urban areas that have similar hydrologic characteristic as the Denver Metro area. The CUHP model can be used to generate sub-basin hydrographs and the UDSWM (Urban Drainage Storm Water Management) computer program can be used to route and combine hydrographs. For detailed discussions on the CUHP and UDSWM methods and programs, please refer to the latest UDFCD Drainage Criteria Manual.

The HEC-1 computer model developed by the US Army Corps of Engineers is a commonly used rainfall-runoff simulation model. HEC-1 model can be used to generate runoff hydrographs for both short and long duration storms based on various hydrologic analysis methods. HEC-1 model can route and combine flows of multiple drainage basins. The SCS Unit Hydrograph method within HEC-1 model is recommended for use in the State of Colorado. The SCS Unit Hydrograph method has been widely used and accepted in the western United States. The US Army Corps of Engineers' Window-based hydrologic program HEC-HMS may be used in place of HEC-1 when appropriate.

1.3 RAINFALL DEPTHS

The 6-hr and 24-hr point precipitation values for 2-, 5-, 10-, 25-, 50-, and 100-year storm events can be estimated directly from Figures CH5-F101 through CH5-F112. The point precipitation values for each storm duration (6- and 24-hr) obtained from the isopluvial maps should be plotted on the return-period diagram (Figure CH5-F113), and a straight line of best fit should be drawn. If any rainfall value deviates substantially from the best-fit line, the value read from the line should replace the original point precipitation value from the map.

Once the 6- and 24-hr rainfall values have been obtained and adjusted (if necessary), the rainfall depths for other durations can be estimated using the following procedures from the NOAA Atlas II, Volume III, 1973. The State of Colorado has been divided into four (4) geographic regions by NOAA and they are shown on Figure CH5-F114. Before applying the empirical methods outlined below, it is necessary to determine the region and apply appropriate equations for the drainage basin. If the drainage basin is located within few miles of a regional boundary, computations shall be made using equations for both regions and the average rainfall values shall be used for the rainfall-runoff analysis.

The 1-hour frequency values for 2- and 100-year storm events can be estimated utilizing the appropriate regional equations from Table CH5-T102. Once computed, the 2-year and 100-year, 1-hour values can be plotted on Figure CH5-F113, and a straight line between the two values can be drawn. Then, the 1-hour values for return periods between 2- and 100-year events can be obtained from the line.

Rainfall depths for the 2-hour and 3-hour events can be estimated using the following formulas (NOAA Atlas 2, 1973).

Region 1	$D_{X,2} = 0.342^{*}D_{X,6} + 0.658^{*}D_{X,1}$	(Eq. CH5-100)
Region 2	$D_{X,2} = 0.341^{*}D_{X,6} + 0.659^{*}D_{X,1}$	(Eq. CH5-101)
Region 3 &4	$D_{X,2} = 0.250^{*}D_{X,6} + 0.750^{*}D_{X,1}$	(Eq. CH5-102)

Where $D_{X,2}$= "X"-year, 2-hour rainfall depth (Inches)
$D_{X,1}$= "X"-year, 1-hour rainfall depth (Inches)
$D_{X,6}$= "X"-year, 6-hour rainfall depth (Inches)

Region 1	$D_{X,3} = 0.597^{*}D_{X,6} + 0.403^{*}D_{X,1}$	(Eq. CH5-103)
Region 2	$D_{X,3} = 0.569^{*}D_{X,6} + 0.431^{*}D_{X,1}$	(Eq. CH5-104)
Region 3&4	$D_{X,3} = 0.467^{*}D_{X,6} + 0.533^{*}D_{X,1}$	(Eq. CH5-105)

Where $D_{X,3}$= "X"-year, 3-hour rainfall depth (Inches)
$D_{X,1}$= "X"-year, 1-hour rainfall depth (Inches)
$D_{X,6}$= "X"-year, 6-hour rainfall depth (Inches)

Based on Figure 17 in the NOAA Atlas 2, the 12-hour duration rainfall depth for the desired recurrence frequency is essentially the average of the 6-hour and 24-hour storm events (NOAA, 1973).

$$D_{X,12} = (D_{X,6} + D_{X,24})/2 \qquad \text{(Eq. CH5-106)}$$

Where $D_{X,12}$= "X"-year, 12-hour rainfall depth (Inches)
$D_{X,6}$= "X"-year, 6-hour rainfall depth (Inches)
$D_{X,24}$= "X"-year, 24-hour rainfall depth (Inches)

Rainfall depths for durations less than 1-hour can be estimated using the adjustment ratios supplied in Table CH5-T101 and the estimated "X"-year, 1-hour rainfall depth (NOAA, 1973). These adjustment ratios were originally published in the US Weather Bureau Technical Paper No. 40 in 1961, and later evaluated and adopted by NOAA.

$$D_{X,Y} = D_{X,1} {}^{*} RATIO_{X,Y} \qquad \text{(Eq. CH5-107)}$$

Where $D_{X,Y}$= "X"-year, Y-minute rainfall depth (Inches)
$D_{X,1}$ = "X"-year, 1-hour rainfall depth (Inches)
$RATIO_{X,Y}$ = Ratio to convert "X"-year, 1-hour rainfall depth to the "X"-year, Y-minute depth

1.4 RAINFALL DISTRIBUTION FOR RATIONAL METHOD

Time-intensity-frequency curves and the Rational Method can be used to produce rainfall-runoff data for drainage basins of less than 160 acres. The time-intensity-frequency curves can be developed for a desired location utilizing the rainfall depths of durations less than 1-hour for storm events between 2- and 100-year.

Utilizing the estimated rainfall depths of the 5-, 10-, 15-, 30-, and 60-minute durations for a given recurrence frequency, rainfall intensities can be estimated by dividing the rainfall depth by the duration of the storm.

$$I_{X,Y} = D_{X,Y}/Duration_Y \qquad \text{(Eq. CH5-108)}$$

Where $I_{X,Y}$ = "X"-year, Y-minute rainfall intensity (Inches/Hour)
$D_{X,Y}$ = "X"-year, Y-minute rainfall depth (Inches)
$Duration_Y$ = Duration Y minute divided 60 (Hour)

A time-intensity curve for a given recurrence frequency can be developed by plotting the intensity values versus their corresponding storm duration values. An example showing the development of a time-intensity-frequency curve is given in Section 1.7.2 of this chapter.

1.5 RAINFALL DISTRIBUTION FOR CUHP METHOD

The CUHP (Colorado Urban Hydrograph Procedure) computer model has been used widely within the Urban Drainage and Flood Control District (UDFCD) jurisdictional area to estimate urban sub-basin hydrographs. The CUHP method was developed and calibrated to simulate short duration convective storms in the Denver Metro area and other similar urban drainage environments. Convective storms are commonly known to be responsible for high peak flows and flooding problems for many small drainage basins.

The CHUP method should only be used for urban areas that have similar hydrologic characteristics as the Denver Metro Area.

1.5.1 CUHP STORM DISTRIBUTION

The rainfall intensity and distribution analysis performed by UDFCD using 73-years of rainfall record data at the Denver rain gage revealed that the majority of the past intense rainstorms produced their largest rainfall within the first hour of the storm. The analysis further discovered that out of the 73 storm events analyzed, 68 events produced the most intense rainfall beginning and ending within the first hour of the storm and 52 events produced the most intense rainfall beginning and ending within the first half hour of the storm. The UDFCD analysis concluded that these "leading intensity" convective storms were the main cause of most of the flooding problems in the Denver Metro Region (UDFCD, 2001).

The rainfall distributions recommended to be used with CUHP were developed to reflect the "leading intensity" characteristics of the previously recorded convection storms in the Denver Region, and they vary from 2- to 6-hours depending on the size of the drainage basin. The rainfall distributions for 2-, 3- and 6-hour storm durations can be developed using the following procedures from the UDFCD Drainage Criteria Manual, 2001.

For drainage basins less than 10 square miles but greater than 90 acres, two-hour storm distribution rainfall values without area adjustments of the values shall be used with CUHP. For drainage basins between ten and twenty square miles, three-hour storm distribution rainfall values with the area-adjustment shall be used. For basins equal to and larger than 20 square miles, six-hour storm distribution values with the area-adjustment shall be used. Area adjustments of the rainfall values for drainage basins equal to or greater than 10 square miles are necessary to determine the average depth of precipitation over the entire drainage basin being analyzed.

The 1-, 3-, and 6-hour point rainfall depths estimated using the NOAA Atlas 2 procedure described previously can be used to develop storm distributions for a given recurrence frequency. The estimated NOAA point precipitation values can be distributed to develop 2-, 3- or 6-hour temporal distribution values using a 5-minute time increment following the distribution procedures from the UDFCD Drainage Criteria Manual, 2001.

The 2-hour temporal distribution for a given recurrence frequency can be developed by multiplying the NOAA 1-hour rainfall depth by the incremental distribution percentages (0 to 120 minutes) given in Table CH5-T103. The 2-hour design storm distribution can be used without further modifications with CUHP for drainage basins less than 10 square miles.

The 3-hour storm distribution can be developed by adding incremental precipitation values for the period between 125 minutes and 180 minutes to the 2-hour distribution discussed above. The incremental precipitation values for the period between 125 minutes and 180 minutes can be determined by evenly distributing the difference between the NOAA 3-hour rainfall depth and the 2-hour total precipitation developed using Table CH5-T103. In a similar approach, the 6-hour distribution can be developed by evenly distributing the difference between the NOAA 3-hour and 6-hour rainfall depths over the period of 185 minutes to 360 minutes. The first three hours of the 6-hour distribution is same as the three-hour distribution discussed above.

1.5.2 DEPTH-AREA ADJUSTMENT

The NOAA precipitation depths are related to rainfall frequency at an isolated point. Storms, however, cause rainfall to occur over extensive areas simultaneously, with more intense rainfall typically occurring near the center of the storm. Rainfall depth-area adjustment is necessary to determine the average depth of precipitation over the entire drainage basin being analyzed. This is normally performed using depth-area reduction curves relating point precipitation reduction factor to drainage basin area and storm duration. The depth-area adjustment curves from NOAA Atlas 2, as modified by UDFCD, are shown on Figure CH5-F115.

The 3- and 6-hour design storm distributions should be adjusted in accordance with the suggested depth-area adjustment procedures outlined in NOAA Atlas 2 before using them with CUHP. In order to assist engineers with the depth-area adjustment application procedures, UDFCD developed an adjustment factor table for drainage basins between 10 and 75 square miles. The UDFCD table is included in this section as Table CH5-T104. The 3- and 6- hour storm distribution values can be adjusted by multiplying each incremental rainfall depth by the appropriate adjustment factor from Table CH5-T104 for a given time increment and the size of the drainage basin. An example showing the development of a CUHP design storm distribution is given in Section 1.7.3 of this chapter.

1.6 RAINFALL DISTRIBTION FOR SCS UNIT HYDROGRAPH METHOD (HEC-1 or HEC-HMS)

The HEC-1 computer program was developed by the US Army Corps of Engineers Hydrologic Engineering Center to assist engineers in performing rainfall-runoff analyses for both short and long duration storms based on various hydrologic analysis methods. The SCS Unit Hydrograph method within the HEC-1 program is recommended for use in the State of Colorado. The SCS Unit Hydrograph method has been widely used and accepted in the western United States.

The rainfall depth data used in the HEC-1 model should be a centrally distributed storm event with depths at time intervals of 5-minutes, 15-minutes, 60-minutes, 2-hours, 3-hours, 6-hours, 12-hours, and 24-hours for the desired recurrence frequency. The NOAA procedures to determine these rainfall depth values were discussed previously in Section 1.3. The 24 –hour rainfall distribution is centered around the midpoint of the design storm (time = 12 hours), and is commonly known as "Balanced Storm" distribution. These rainfall values are input into the HEC-1 model using the PH record. When using the PH record, a value of 0.001 should be input into Field 2 to prevent the program from using an internal point rainfall reduction adjustment.

1.6.1 DEPTH-AREA REDUCTION FACTORS

Figure CH5-F115 provides the depth-area reduction curve for the 24-hour storm event (NOAA, 1973). Depth-area values are input into the HEC-1 model using the JR record. The peak flow value at a given point should be determined using the depth-area value for the total watershed area tributary to the subject point of interest.

1.7 EXAMPLES

The following examples are provided to lead the reader through the rainfall distribution development procedures outlined in this section by analyzing a hypothetical drainage basin.

1.7.1 EXAMPLE 1: RAINFALL DISTRIBUTION CALCULATION FOR HEC-1 INPUT

Problem:

Develop the 100-year, 24-hour design storm distribution for Basin A.

Solution:

Step 1: Determine the average 100-year, 6-hour rainfall depth and the average 100-year, 24-hour rainfall depth in Basin A from Figures CH5-F106 and CH5-F112, respectively (Basins that have highly variable rainfall depths for a given frequency and duration may need to be subdivided into areas of common rainfall depth. A weighted average of the rainfall depth can then be calculated using the areas and rainfall depths of the sub-basins).

$D_{100,6}$ = 3.6 inches (Assumed for example purposes only)
$D_{100,24}$ = 5.0 inches (Assumed for example purposes only)

The average 6-hr and 24-hr rainfall depths in Basin A for 2-, 5-, 10-, 25-, and 50-year storm events should also be estimated from Figures CH5-F101 through CH5-F112. The average rainfall values of the six recurrence frequencies for each storm duration (6- and 24-hr) should be plotted on the return-period diagram (Figure CH5-F113), and a straight line of best fit should be drawn. If any recurrence frequency rainfall value deviates substantially from the best-fit line, the value read from the line should replace the original rainfall value from the map. For the purpose of this example, it is assumed no adjustments of the rainfall values are necessary.

Step 2: Calculate the average 100-year, 1-hour rainfall depth, Y_{100}.

From Figure CH5-F114, determine the geographic Region of Basin A. For the purpose of this example, Basin A is located in Region 1 and the average basin elevation is 6,000 ft.

From Table CH5-T102,

Region 1, $Y_{100} = 1.897 + 0.439[(3.6)(3.6/5.0)] - 0.008(60) = 2.56$ inches

Step 3: Determine the 100-year, 5 minute and the 100-year, 15-minute rainfall values. The conversion ratios provided in Table CH5-T101 are multiplied by the 1-hour rainfall depth.

RATIO5 = 0.29
RATIO15= 0.57

$D_{100,5}$ = RATIO5*$D_{100,1}$ = 0.29*2.56 = 0.74 inches
$D_{100,15}$ = RATIO15*$D_{100,1}$ = 0.57*2.56 = 1.46 inches

Step 4: Compute the 100-year, 2-hour, 100-year, 3-hour, and the 100-year, 12-hour rainfall values using Equations CH5-100 and CH5-103, respectively.

$$D_{100,2} = 0.342 * D_{100,6} + 0.658 * D_{100,1}$$
$$= 0.342*3.6+0.658*2.56 = 2.92 \text{ inches}$$
$$D_{100,3} = 0.597 * D_{100,6} + 0.403 * D_{100,1}$$
$$= 0.597*3.6+0.403*2.56 = 3.18 \text{ inches}$$
$$D_{100,12} = 0.5 * D_{100,6} + 0.5 * D_{100,24}$$
$$= 0.5*3.6+0.5*5.0 = 4.3 \text{ inches}$$

Step 5: Estimate the depth-area reduction factor from Figure CH5-F115.

Assuming the drainage area for Basin A is 2,140 acres, or 3.34 square miles, the area-reduction factor is approximately 0.995. (As runoff flows through the drainage basin, the drainage area increases, and the depth-area reduction factor will vary. To account for this, a range of depth-area reduction factors may need to be estimated for large basins that have several sub-basin design points. For instance, if the drainage basin was 15 square miles, three depth-area reduction values may be used to estimate runoff for a design point at 5 square miles, one at 10 square miles, and one at 15 square miles. The respective depth-area reduction values would be 0.992, 0.985, and 0.978).

The rainfall depths for durations of 5 minutes, 15 minutes, 1 hour, 2 hours, 3 hours, 6 hours, 12 hours, and 24 hours are entered on the PH record of HEC-1 input data to define the 24-hour storm distribution. A value of 0.001 is entered in Field 2 of the PH record.

The depth-area reduction factor(s) is entered in the JR record of the HEC-1 input data.

1.7.2 EXAMPLE 2: TIME-INTENSITY-FREQUENCY CURVE GENERATION

Problem:

Develop the 100-year time-intensity-frequency curve for Rocky Subdivision located in Basin A.

Solution:

Step 1: Calculate the 100-year, 1-hour rainfall depth for Rocky Subdivision as explained in the Example 1.
$D_{100,1}$ = 2.56 inches (Assumed for example purposes only)

Step 2: Generate the 100-year rainfall depths for storm durations of 5 minutes, 10 minutes, 15 minutes and 30 minutes.

$D_{100,5} = RATIO5*D_{100,1} = 0.29*2.56 = 0.74$ inches
$D_{100,10} = RATIO10*D_{100,1} = 0.45*2.56 = 1.15$ inches
$D_{100,15} = RATIO15*D_{100,1} = 0.57*2.56 = 1.46$ inches
$D_{100,30} = RATIO30*D_{100,1} = 0.79*2.56 = 2.02$ inches

Step 3: Calculate 100-year rainfall intensity values for storm durations of 5 minutes, 10 minutes, 15 minutes, 30 minutes, and 60 minutes.

$I_{100,5} = D_{100,5}/Duration = 0.74/(5/60) = 8.88$ inches/hour
$I_{100,10} = D_{100,10}/Duration = 1.15/(10/60) = 6.90$ inches/hour
$I_{100,15} = D_{100,15}/Duration = 1.46/(15/60) = 5.84$ inches/hour
$I_{100,30} = D_{100,30}/Duration = 2.02/(30/60) = 4.04$ inches/hour
$I_{100,60} = D_{100,60}/Duration = 2.56/(60/60) = 2.56$ inches/hour

Step 4: Plot the time-intensity-frequency curve for the 100-year storm for Rocky Subdivision. (See Figure CH5-F116).

Application: The time-intensity-frequency curve is used to determine the rainfall intensities used in the Rational Method of determining runoff described in Chapter 5, Section 2.0.

1.7.3 EXAMPLE 3: CUHP STORM DISTRIBUTION

Problem:

Develop a 100-year rainfall distribution to be used with CUHP model for a 17 square mile drainage basin.

100-year, 1-hr rainfall depth = 2.20 inches
100-year, 3-hr rainfall depth = 2.75 inches

Solution:

Step 1: Since the drainage basin is less than 20 square miles but greater than 10 square miles, a three-hour storm distribution should be used with CUHP. First, a two-hour temporal distribution should be developed by multiplying the 100-year, 1-hour rainfall value of 2.2 inches by the incremental distribution percentages from Table CH5-T103.

Step 2: Calculate incremental rainfall depths for the period between 125 and 180 minutes by evenly distributing the rainfall depth difference between the 100-year, 3-hour rainfall depth of 2.75 inches and the 2-hour total precipitation.

Step 3: Apply the depth-area reduction factors from Table CH5-T104 to the calculated incremental rainfall depths for the entire storm duration.

Results of the above three steps are shown in Table CH5-T105.

Duration (min)	5	10	15	30
Ratio to 1-hr	0.29	0.45	0.57	0.79

(Adopted from *U.S. Weather Bureau Technical Paper* No. 40, 1961.)

TABLE CH5-T101
ADJUSTMENT FACTORS FOR DURATIONS LESS THAN ONE HOUR

Region of applicability*	Equation	Corr. coeff.	No. of stations	Mean of computed stn. values (inches)	Standard error of estimate (inches)
South Platte, Republican, Arkansas, and Cimarron River Basins (1)	$Y_2 = 0.218 + 0.709[(X_1)(X_1/X_2)]$	0.94	75	1.01	0.074
	$Y_{100} = 1.897 + 0.439[(X_3)(X_3/X_4)] - 0.008Z$	.84	75	2.68	.317
San Juan, Upper Rio Grande, Upper Colorado, and Gunnison River Basins and Green River Basin below confluence with the Yampa River (2)	$Y_2 = -0.011 + 0.942[(X_1)(X_1/X_2)]$	.95	86	0.72	.085
	$Y_{100} = 0.494 + 0.755[(X_3)(X_3/X_4)]$	.90	85	1.96	.290
Yampa and Green River Basins above confluence of Green and Yampa Rivers (3)	$Y_2 = 0.019 + 0.711[(X_1)(X_1/X_2)] + 0.001Z$	.82	98	0.40	.031
	$Y_{100} = 0.338 + 0.670[(X_3)(X_3/X_4)] + 0.001Z$	.80	79	1.04	.141
North Platte (4)	$Y_2 = 0.028 + 0.890[(X_1)(X_1/X_2)]$	.93	90	0.60	.062
	$Y_{100} = 0.671 + 0.757[(X_3)(X_3/X_4)] - 0.003Z$	.91	88	1.71	.236

* Numbers in parentheses refer to geographic regions shown in figure 19. See text for more complete description.

List of variables

Y_2 = 2-yr 1-hr estimated value
Y_{100} = 100-yr 1-hr estimated value
X_1 = 2-yr 6-hr value from precipitation-frequency maps
X_2 = 2-yr 24-hr value from precipitation-frequency maps
X_3 = 100-yr 6-hr value from precipitation-frequency maps
X_4 = 100-yr 24-hr value from precipitation-frequency maps
Z = point elevation in hundreds of feet

Time Minutes	Percent of 1-Hour NOAA Rainfall Atlas Depth				
	2-Year	5-Year	10-Year	25- and 50-Year	100- and 500-Year
5	2.0	2.0	2.0	1.3	1.0
10	4.0	3.7	3.7	3.5	3.0
15	8.4	8.7	8.2	5.0	4.6
20	16.0	15.3	15.0	8.0	8.0
25	25.0	25.0	25.0	15.0	14.0
30	14.0	13.0	12.0	25.0	25.0
35	6.3	5.8	5.6	12.0	14.0
40	5.0	4.4	4.3	8.0	8.0
45	3.0	3.6	3.8	5.0	6.2
50	3.0	3.6	3.2	5.0	5.0
55	3.0	3.0	3.2	3.2	4.0
60	3.0	3.0	3.2	3.2	4.0
65	3.0	3.0	3.2	3.2	4.0
70	2.0	3.0	3.2	2.4	2.0
75	2.0	2.5	3.2	2.4	2.0
80	2.0	2.2	2.5	1.8	1.2
85	2.0	2.2	1.9	1.8	1.2
90	2.0	2.2	1.9	1.4	1.2
95	2.0	2.2	1.9	1.4	1.2
100	2.0	1.5	1.9	1.4	1.2
105	2.0	1.5	1.9	1.4	1.2
110	2.0	1.5	1.9	1.4	1.2
115	1.0	1.5	1.7	1.4	1.2
120	1.0	1.3	1.3	1.4	1.2
Totals	115.7	115.7	115.7	115.6	115.6

	2-, 5-, and 10-Year Design Rainfall Area—Square Miles				25-, 50-, 100-, and 500-Year Design Rainfall Area—Square Miles			
Time Minutes	10-20	20-30	30-50	50-75	10-20	20-30	30-50	50-75
5	1.00	1.00	1.10	1.10	1.00	1.00	1.05	1.10
10	1.00	1.00	1.05	1.10	1.00	1.00	1.05	1.10
15	1.00	1.00	1.05	1.00	1.00	1.00	1.05	1.10
20	0.90	0.81	0.74	0.62	1.00	1.00	1.05	1.00
25	0.90	0.81	0.74	0.62	0.90	0.81	0.74	0.60
30	0.90	0.81	0.74	0.62	0.90	0.81	0.74	0.60
35	1.00	1.00	1.05	1.00	0.90	0.81	0.74	0.70
40	1.00	1.00	1.05	1.10	1.00	1.00	1.05	1.00
45	1.00	1.00	1.05	1.10	1.00	1.00	1.05	1.10
50	1.00	1.00	1.05	1.10	1.00	1.00	1.05	1.10
55	1.00	1.00	1.05	1.10	1.00	1.00	1.05	1.10
60	1.00	1.00	1.05	1.10	1.00	1.00	1.05	1.10
65 - 120	1.00	1.00	1.05	1.10	1.00	1.00	1.05	1.10
125 - 180	1.00	1.15	1.20	1.40	1.00	1.15	1.20	1.40
185 - 360	N/A	1.15	1.20	1.20	N/A	1.15	1.20	1.20

Time (min.)	2-hr Dist. %	2-hr Design Storm Dist. (inches)	3-hr Design Storm Dist. (inches)	Area Adjustment Factors	Adjusted 3-hr Design Storm Dist. (inches)
0		0.00	0.00		0.00
5.0	1.0	0.02	0.02	1.00	0.02
10.0	3.0	0.07	0.07	1.00	0.07
15.0	4.6	0.10	0.10	1.00	0.10
20.0	8.0	0.18	0.18	1.00	0.18
25.0	14.0	0.31	0.31	0.90	0.28
30.0	25.0	0.55	0.55	0.90	0.50
35.0	14.0	0.31	0.31	0.90	0.28
40.0	8.0	0.18	0.18	1.00	0.18
45.0	6.2	0.14	0.14	1.00	0.14
50.0	5.0	0.11	0.11	1.00	0.11
55.0	4.0	0.09	0.09	1.00	0.09
60.0	4.0	0.09	0.09	1.00	0.09
65.0	4.0	0.09	0.09	1.00	0.09
70.0	2.0	0.04	0.04	1.00	0.04
75.0	2.0	0.04	0.04	1.00	0.04
80.0	1.2	0.03	0.03	1.00	0.03
85.0	1.2	0.03	0.03	1.00	0.03
90.0	1.2	0.03	0.03	1.00	0.03
95.0	1.2	0.03	0.03	1.00	0.03
100.0	1.2	0.03	0.03	1.00	0.03
105.0	1.2	0.03	0.03	1.00	0.03
110.0	1.2	0.03	0.03	1.00	0.03
115.0	1.2	0.03	0.03	1.00	0.03
120.0	1.2	0.03	0.03	1.00	0.03
125.0			0.02	1.00	0.02
130.0			0.02	1.00	0.02
135.0			0.02	1.00	0.02
140.0			0.02	1.00	0.02
145.0			0.02	1.00	0.02
150.0			0.02	1.00	0.02
155.0			0.02	1.00	0.02
160.0			0.02	1.00	0.02
165.0			0.02	1.00	0.02
170.0			0.02	1.00	0.02
175.0			0.02	1.00	0.02
180.0			0.02	1.00	0.02
Total	**115.6**	**2.54**	**2.75**		**2.64**

TABLE CH5-T105
EXAMPLE 3: 100-YEAR RAINFALL DISTRIBUTION RESULTS

COLORADO STATEWIDE

DRAINAGE AND FLOODPLAIN MANAGEMENT CRITERIA MANUAL

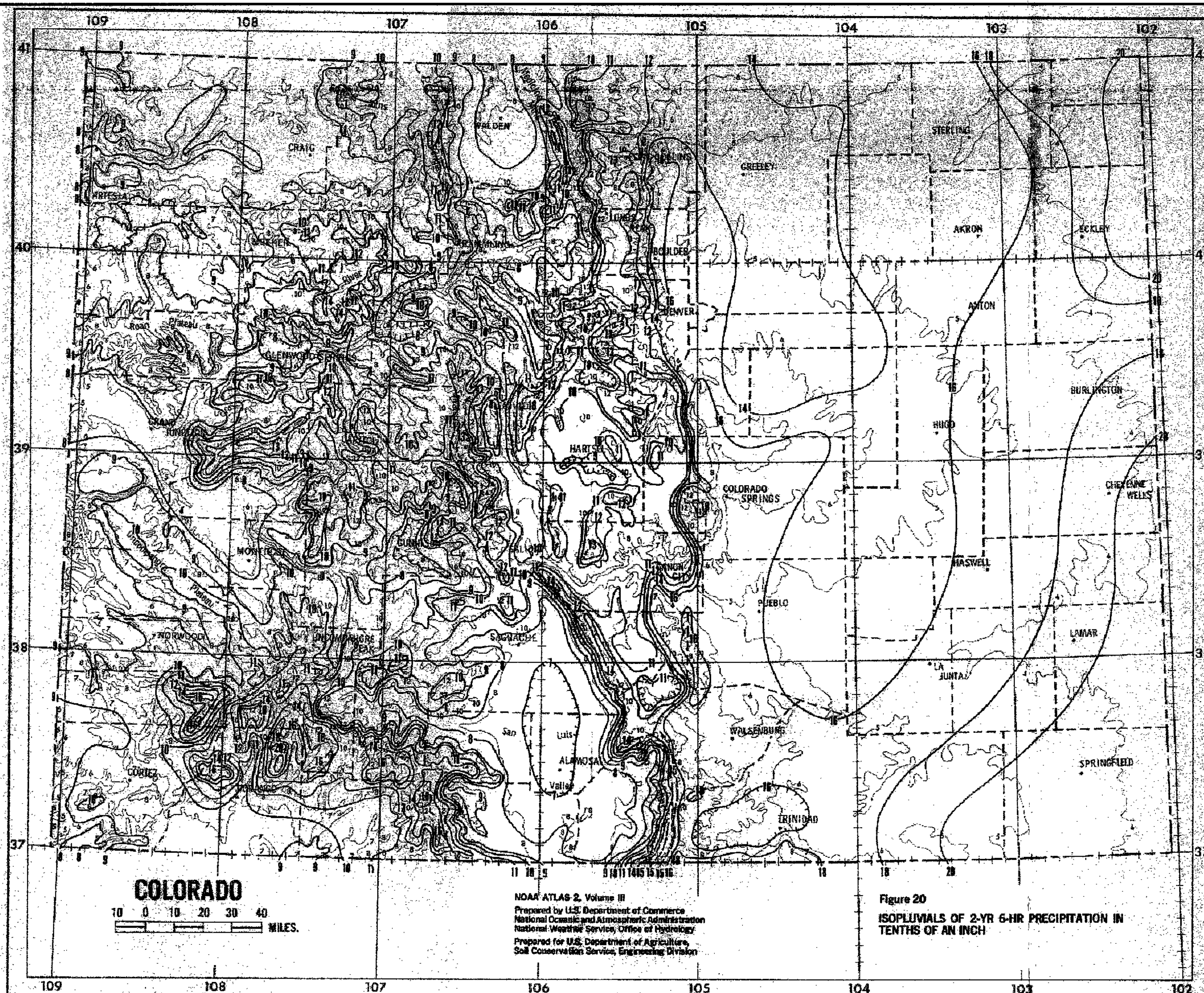


Figure 20

ISOPLUVIALS OF 2-YR 6-HR PRECIPITATION IN TENTHS OF AN INCH

FIGURE CH5-F101

2-YEAR, 6-HOUR PRECIPITATION MAP

REFERENCE: NOAA ATLAS 2, VOL III, 1973

VERSION: AUGUST 2002

WRC ENGINEERING, INC

COLORADO STATEWIDE
DRAINAGE AND FLOODPLAIN MANAGEMENT CRITERIA MANUAL

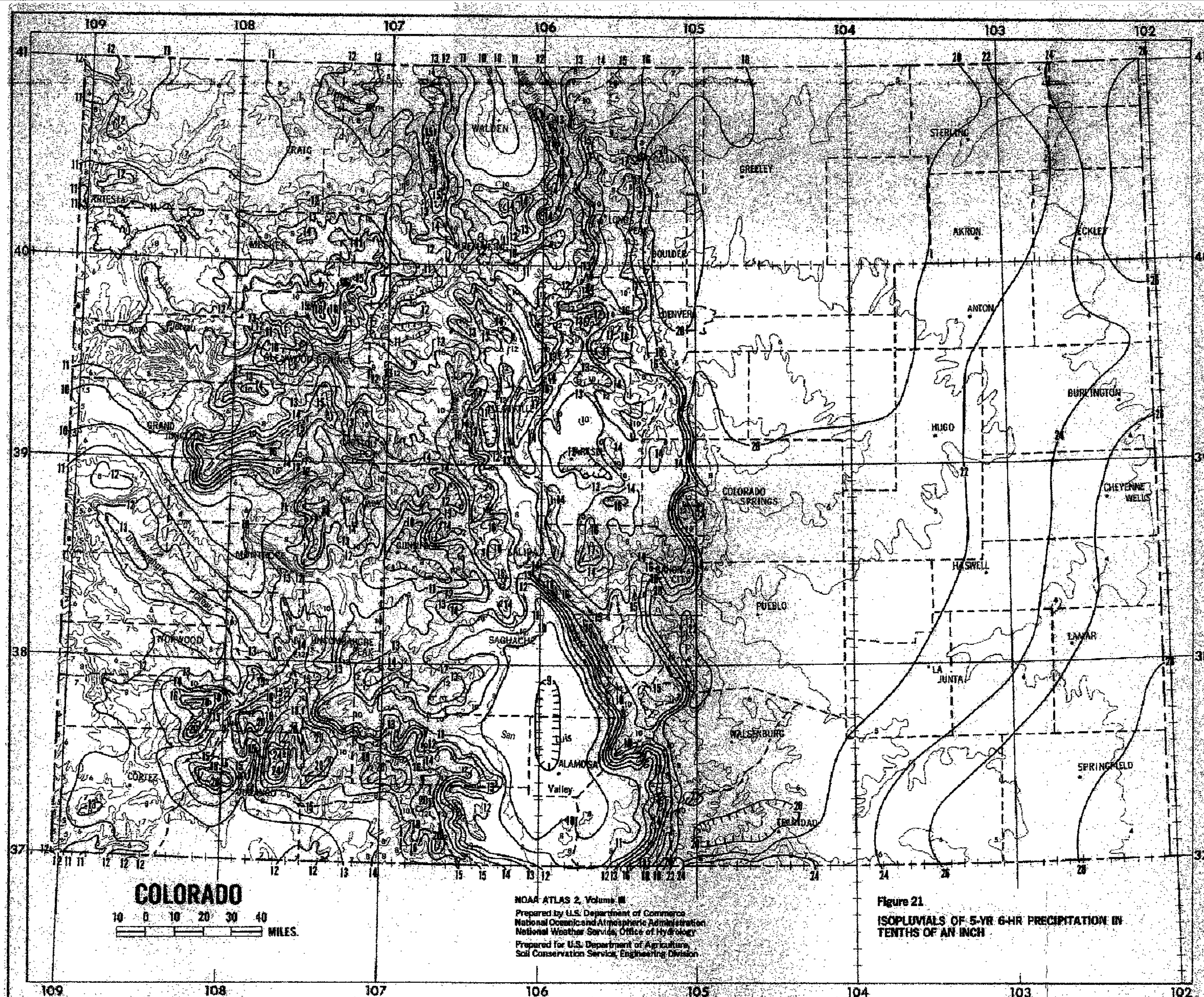


Figure 21
ISOPLUVIALS OF 5-YR 6-HR PRECIPITATION IN TENTHS OF AN INCH

FIGURE CH5-F102
5-YEAR, 6-HOUR PRECIPITATION MAP

REFERENCE: NOAA ATLAS 2, VOL III, 1973

VERSION: AUGUST 2002

WRC ENGINEERING, INC

COLORADO STATEWIDE

DRAINAGE AND FLOODPLAIN MANAGEMENT CRITERIA MANUAL

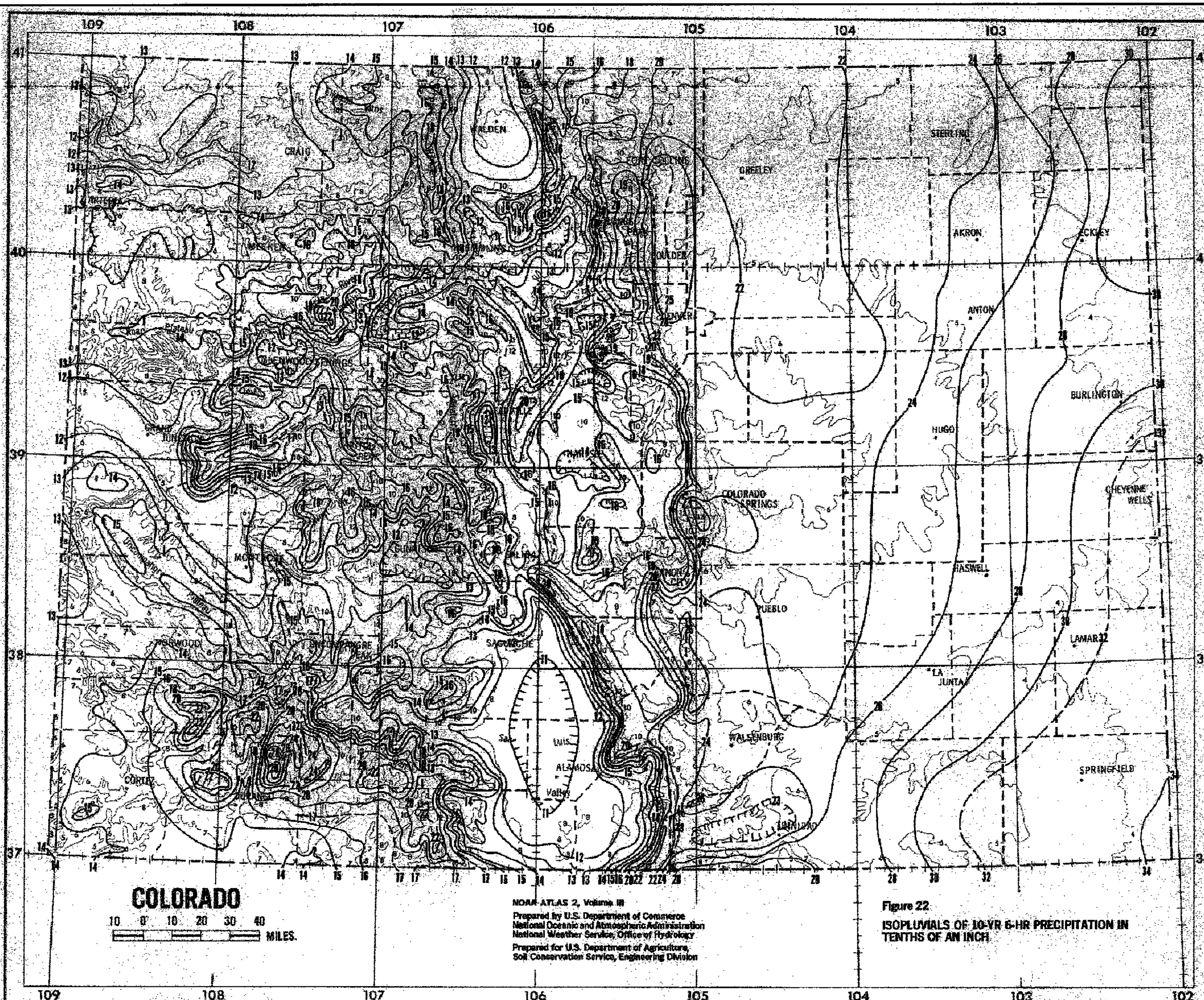


FIGURE CH5-F103

10-YEAR, 6-HOUR PRECIPITATION MAP

REFERENCE: NOAA ATLAS 2, VOL III, 1973

VERSION: AUGUST 2002

WRC ENGINEERING, INC

O:/2080/FIGURES, CH5-F103.DWG, 4/10/02 MAS

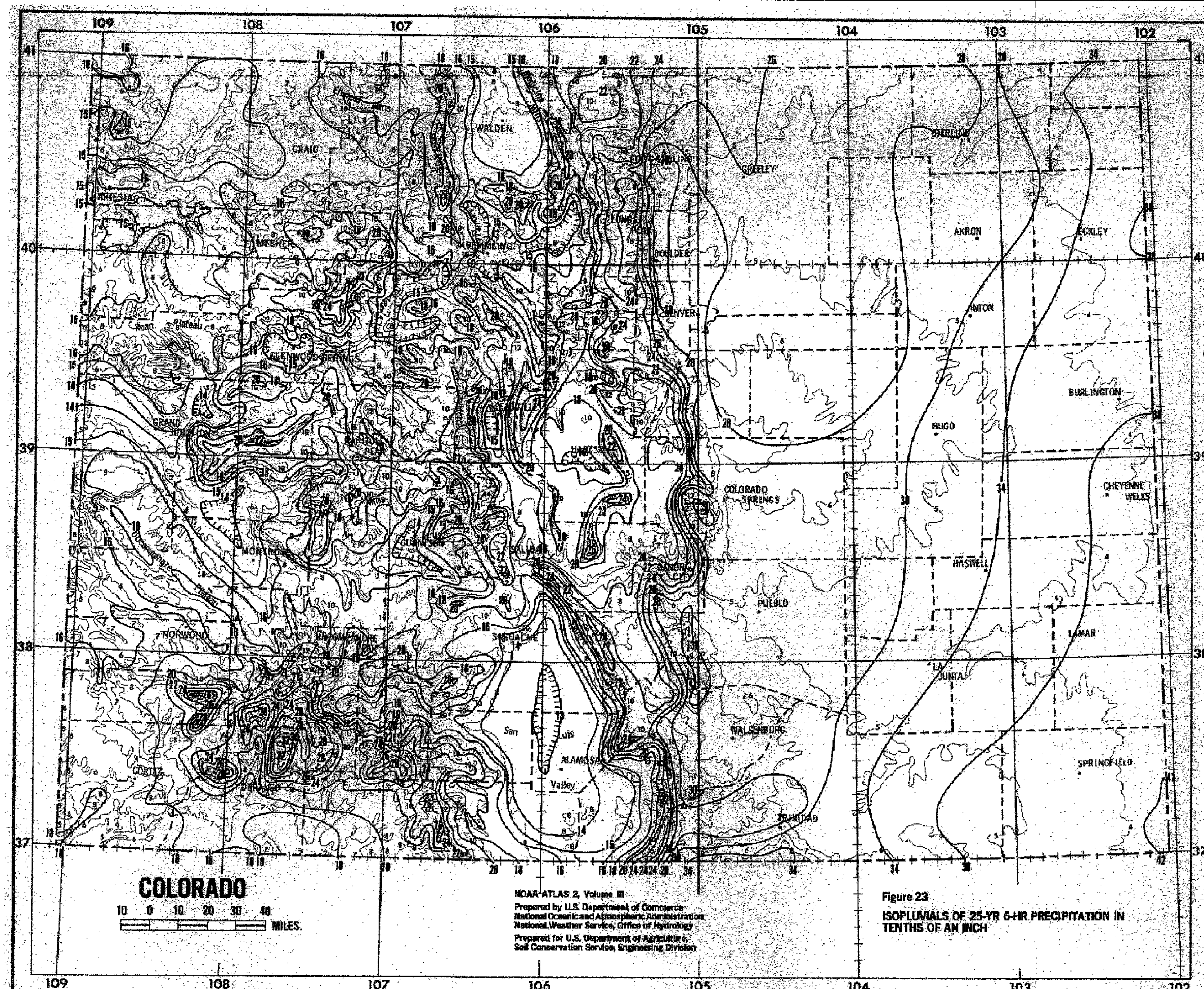


Figure 23

ISOPLUVIALS OF 25-YR 6-HR PRECIPITATION IN TENTHS OF AN INCH

FIGURE CH5-F104

25-YEAR, 6-HOUR PRECIPITATION MAP

REFERENCE: NOAA ATLAS 2, VOL III, 1973

VERSION: AUGUST 2002

WRC ENGINEERING, INC

O:/2080/FIGURES, CH5-F104.DWG, 4/10/02 MAS

COLORADO STATEWIDE

DRAINAGE AND FLOODPLAIN MANAGEMENT CRITERIA MANUAL

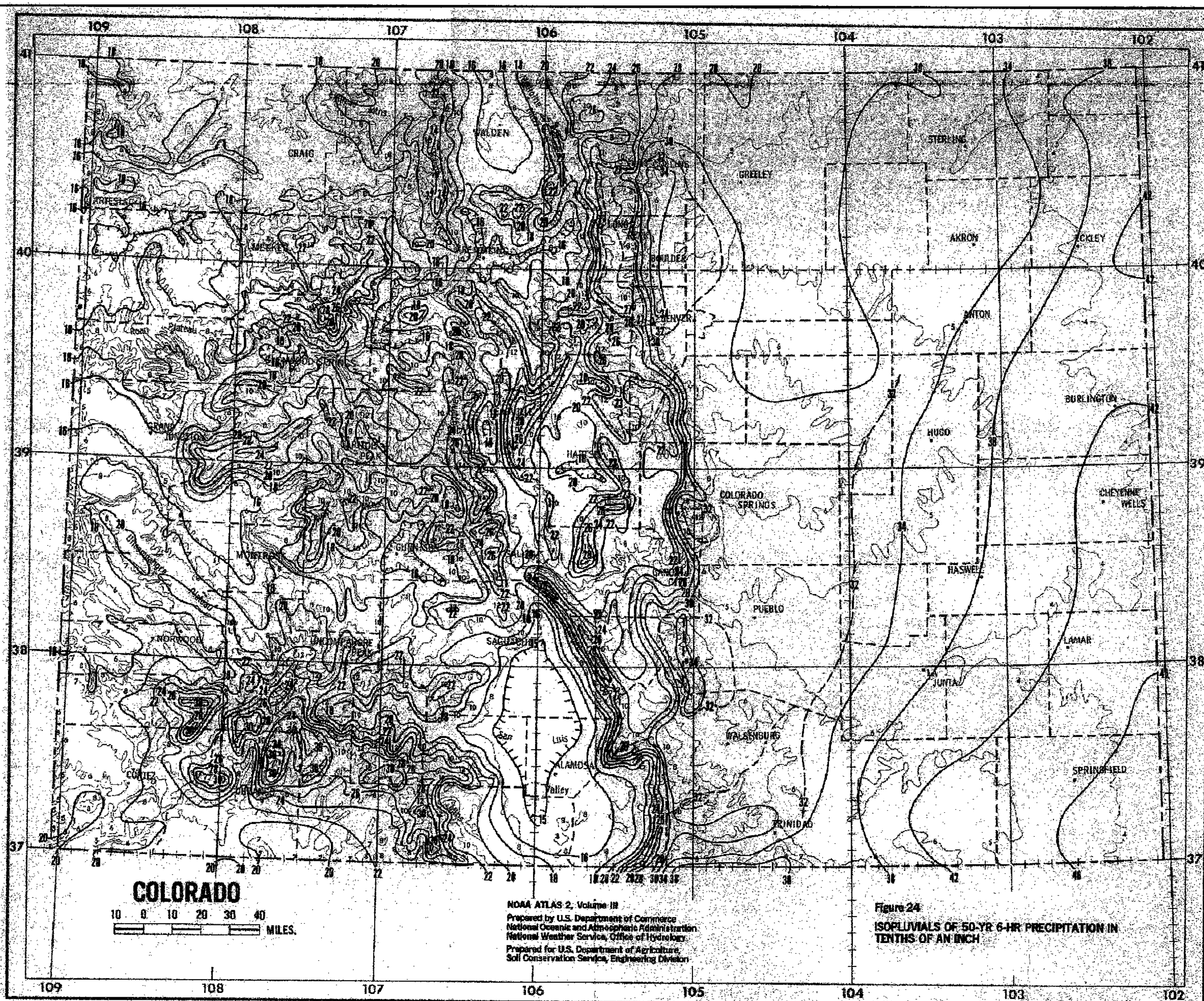


FIGURE CH5-F105

50-YEAR, 6-HOUR PRECIPITATION MAP

REFERENCE: NOAA ATLAS 2, VOL III, 1973

VERSION: AUGUST 2002

WRC ENGINEERING, INC

FIGURE CH5-F106
100-YEAR, 6-HOUR PRECIPITATION MAP

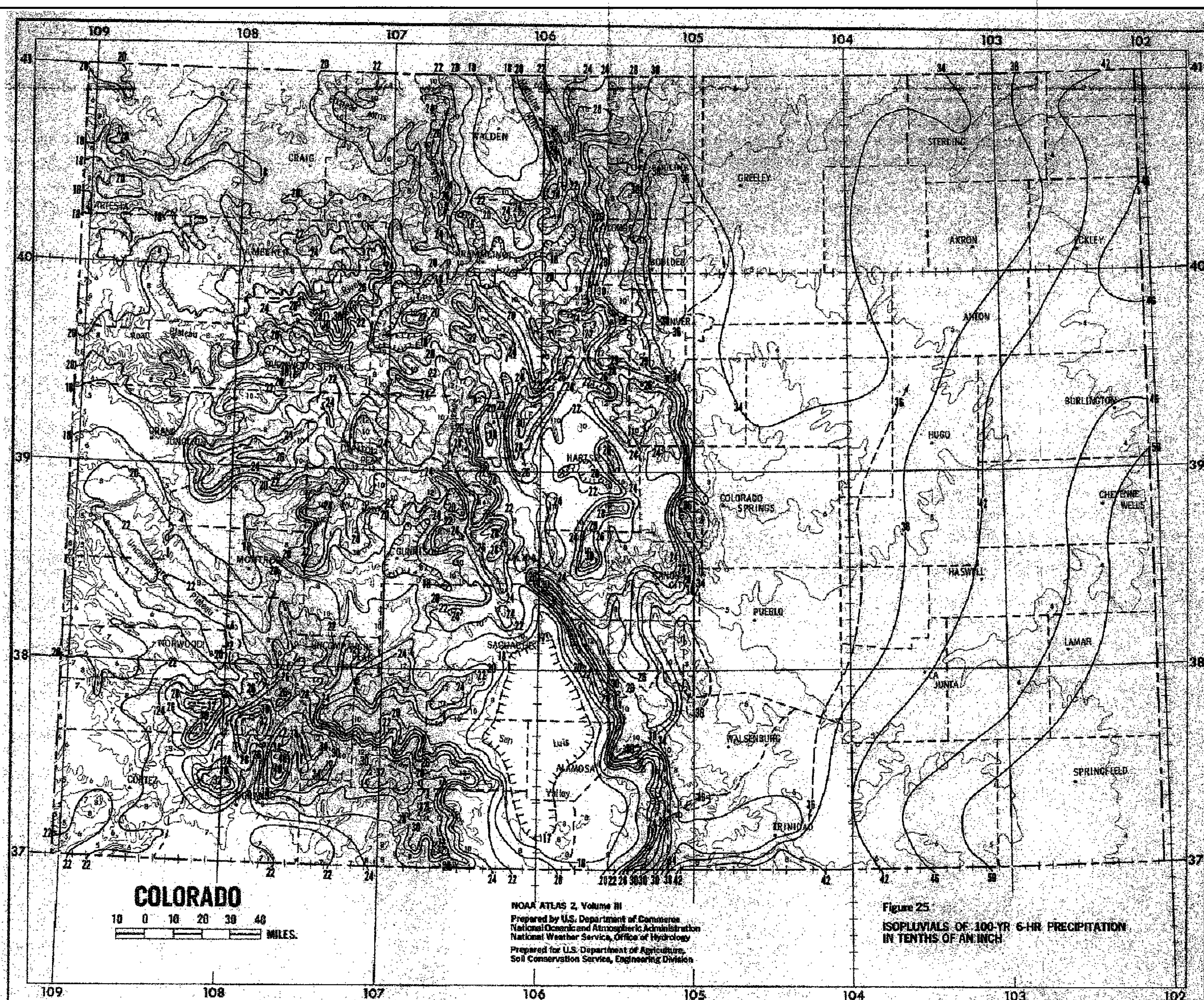


Figure 25
ISOPLUVIALS OF 100-YR 6-HR PRECIPITATION IN TENTHS OF AN INCH

REFERENCE: NOAA ATLAS 2, VOL III, 1973

FIGURE CH5-F107
2-YEAR, 24-HOUR PRECIPITATION MAP

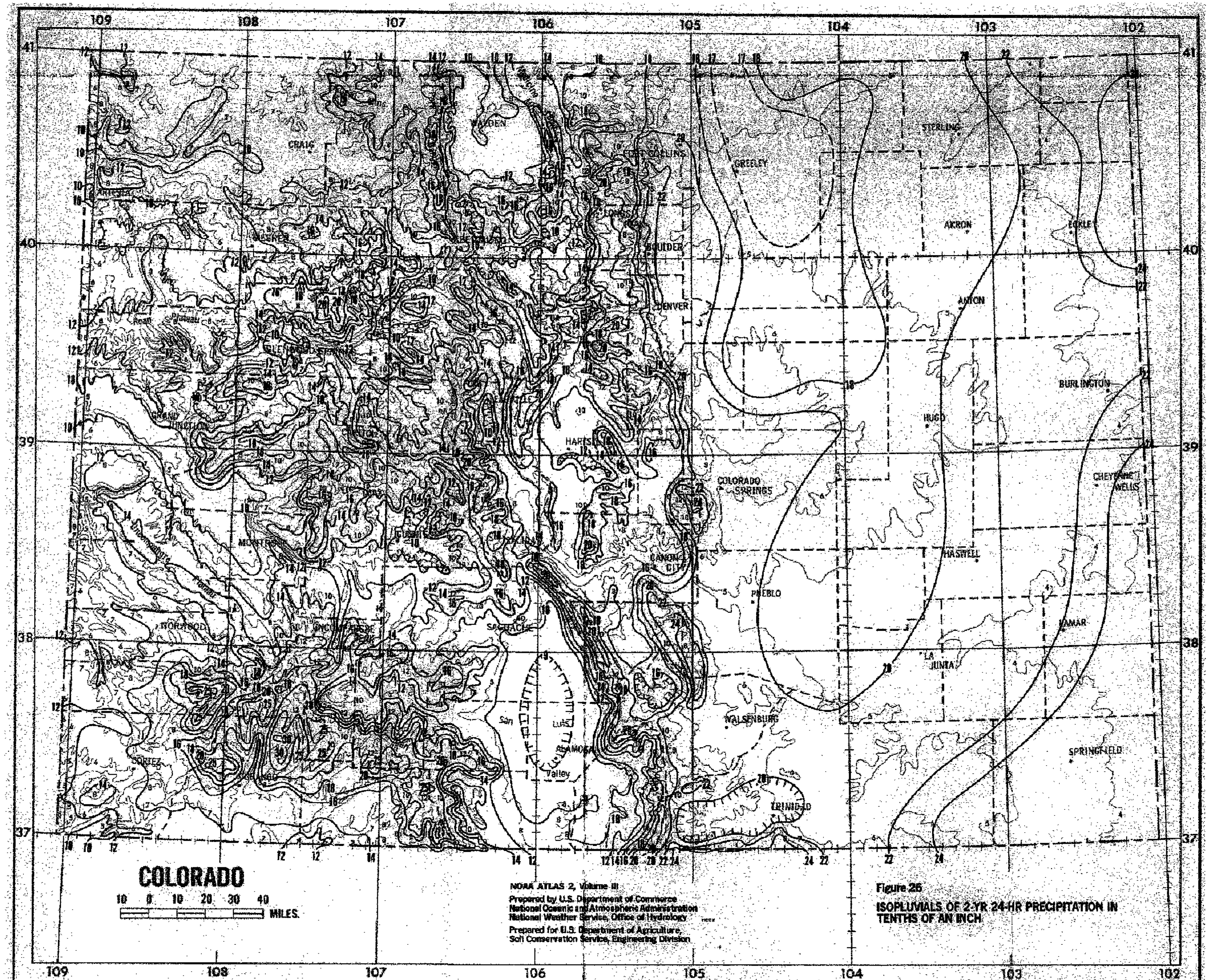


Figure 25
ISOPLUVIALS OF 2-YR 24-HR PRECIPITATION IN TENTHS OF AN INCH

REFERENCE: NOAA ATLAS 2, VOL III, 1973

COLORADO STATEWIDE

DRAINAGE AND FLOODPLAIN MANAGEMENT CRITERIA MANUAL

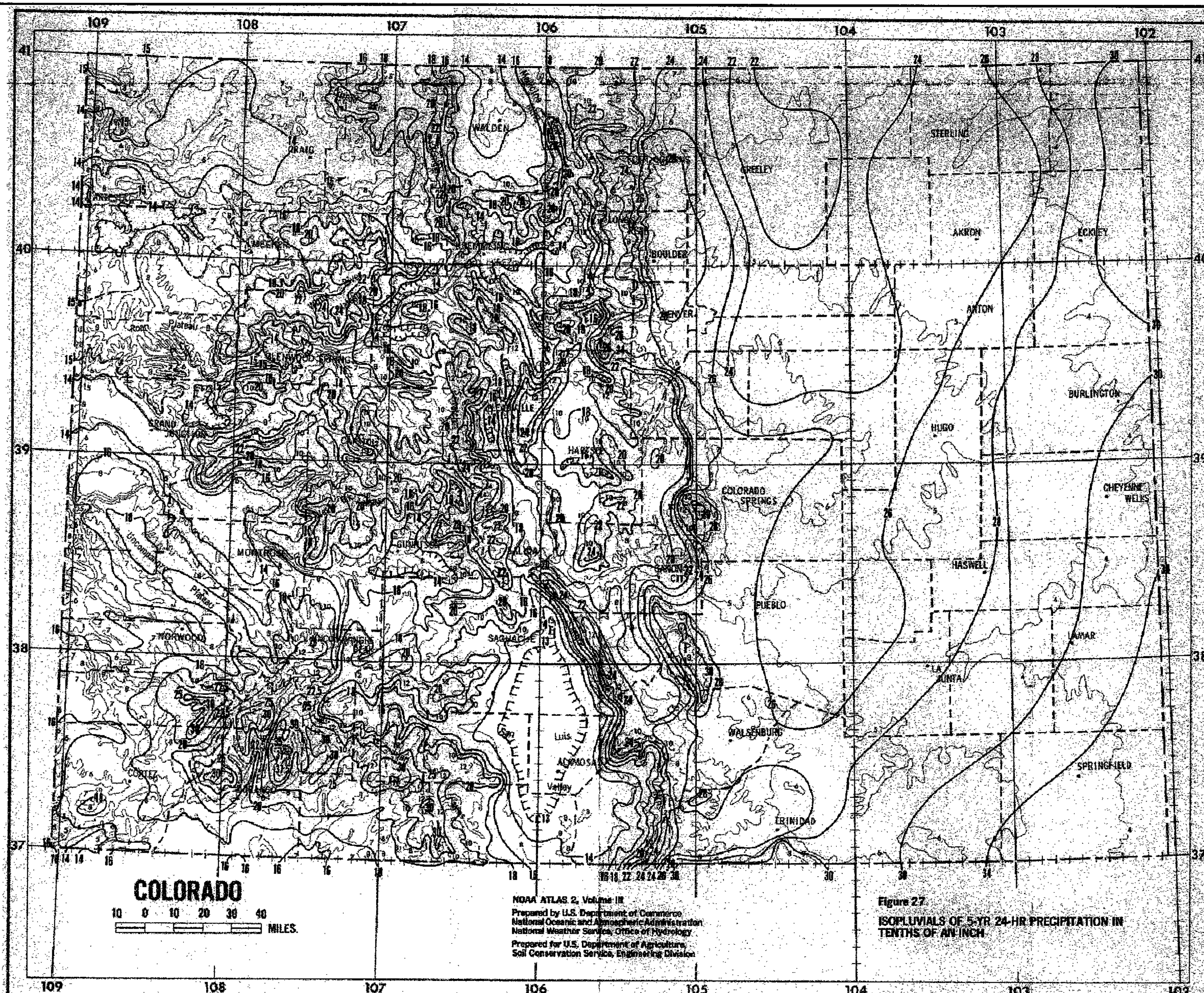


Figure 27
ISOPLUVIALS OF 5-YR 24-HR PRECIPITATION IN TENTHS OF AN INCH

FIGURE CH5-F108

5-YEAR, 24-HOUR PRECIPITATION MAP

REFERENCE: NOAA ATLAS 2, VOL III, 1973

VERSION: AUGUST 2002

WRC ENGINEERING, INC

COLORADO STATEWIDE
DRAINAGE AND FLOODPLAIN MANAGEMENT CRITERIA MANUAL

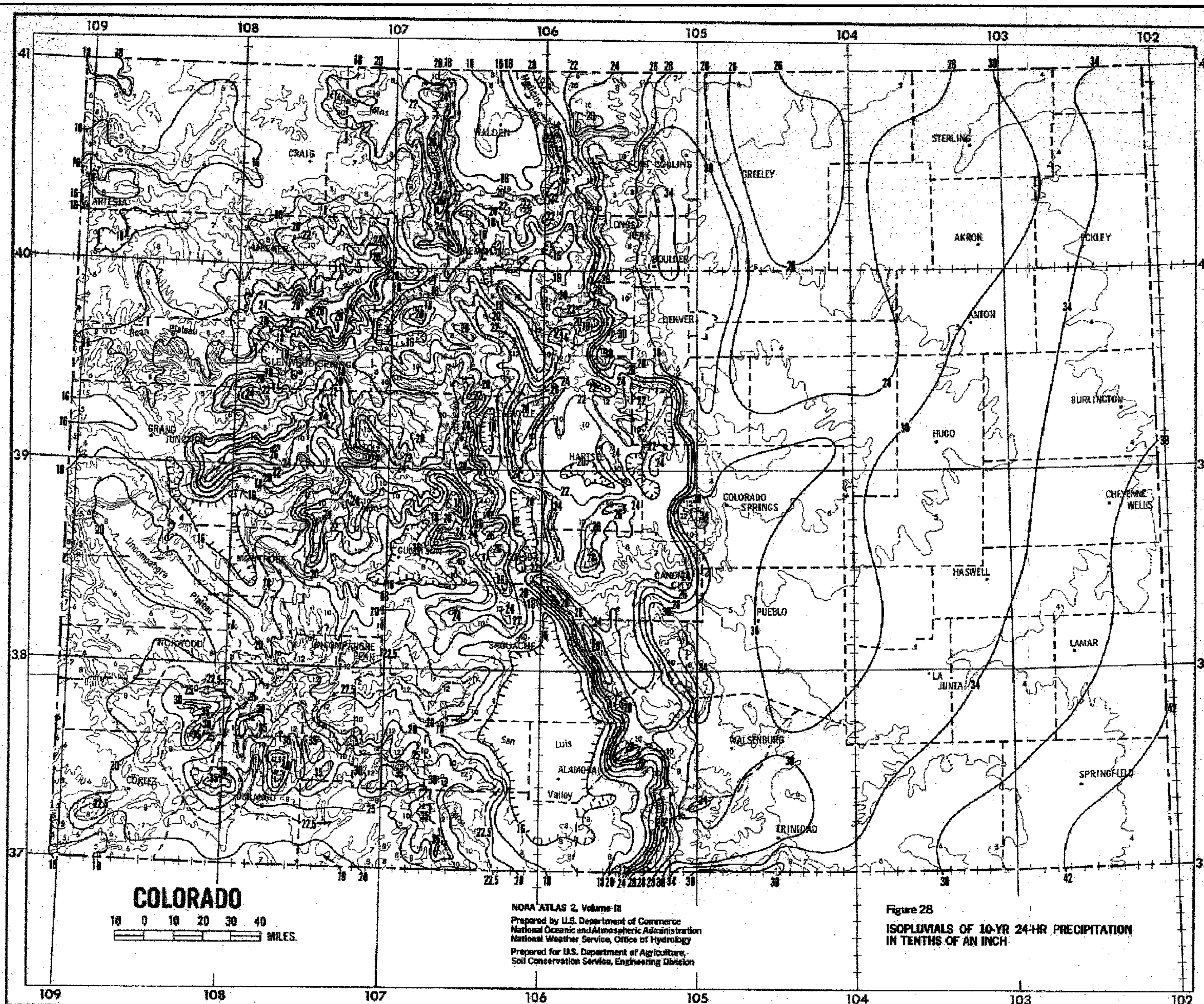


FIGURE CH5-F109
10-YEAR, 24-HOUR PRECIPITATION MAP

REFERENCE: NOAA ATLAS 2, VOL III, 1973

VERSION: AUGUST 2002

WRC ENGINEERING, INC

COLORADO STATEWIDE
DRAINAGE AND FLOODPLAIN MANAGEMENT CRITERIA MANUAL

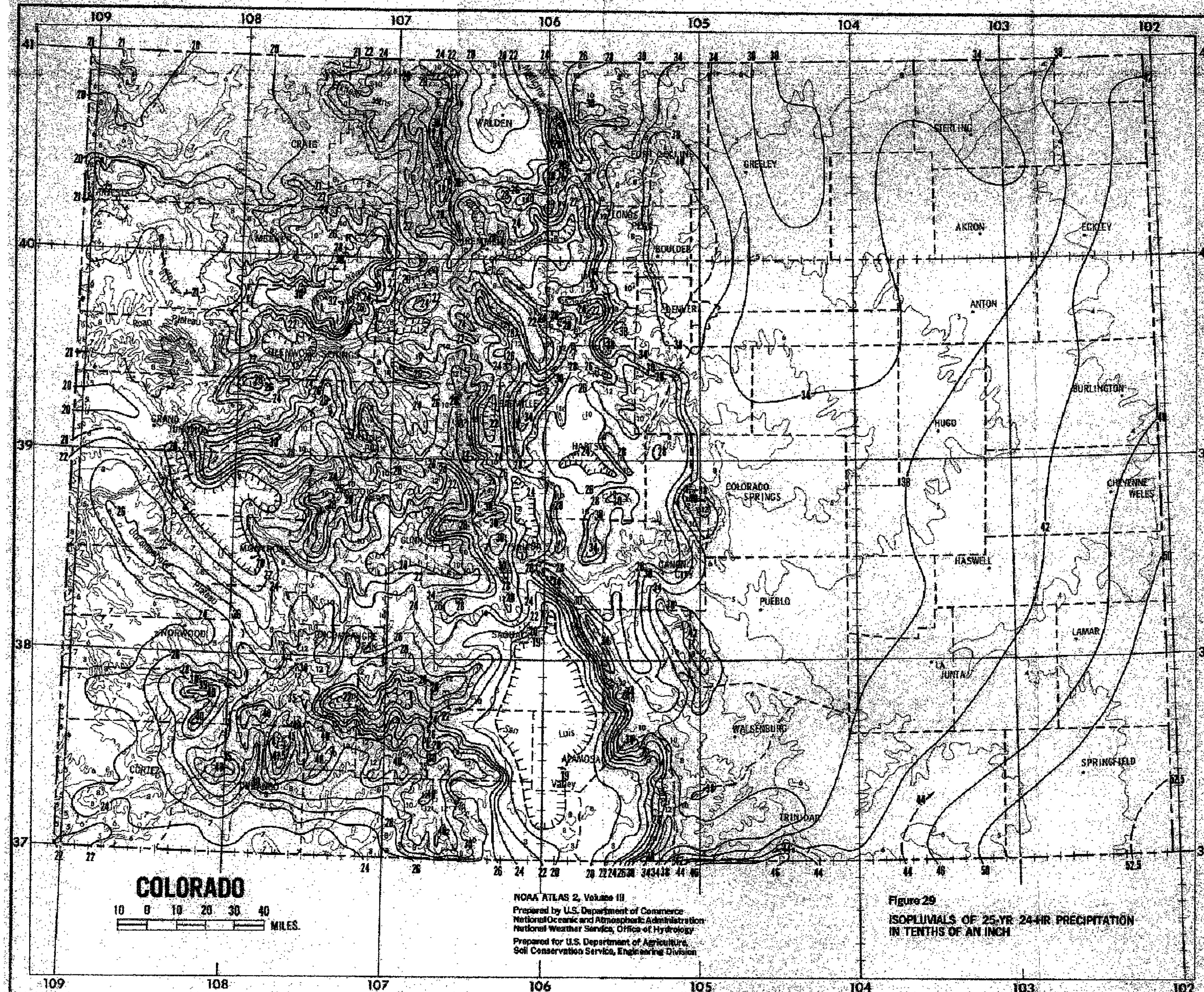


FIGURE CH5-F110
25-YEAR, 24-HOUR PRECIPITATION MAP

REFERENCE: NOAA ATLAS 2, VOL III, 1973

VERSION: AUGUST 2002

WRC ENGINEERING, INC.

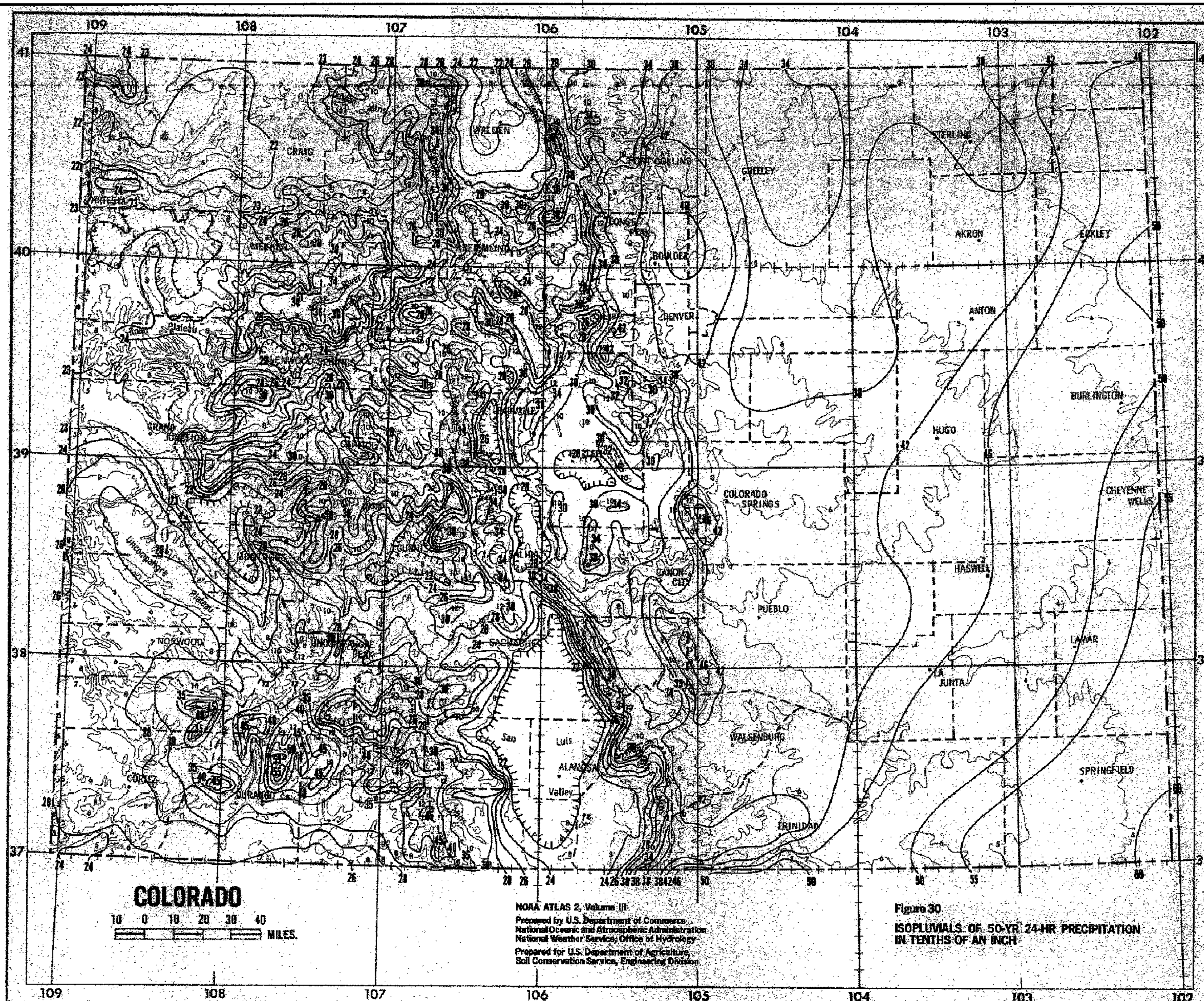


Figure 30
ISOPLUVIALS OF 50-YR 24-HR PRECIPITATION IN TENTHS OF AN INCH

FIGURE CH5-F111
50-YEAR, 24-HOUR PRECIPITATION MAP

REFERENCE: NOAA ATLAS 2, VOL III, 1973

2/2080/FIGURES, CH5-F111.DWG, 4/10/02 MAS

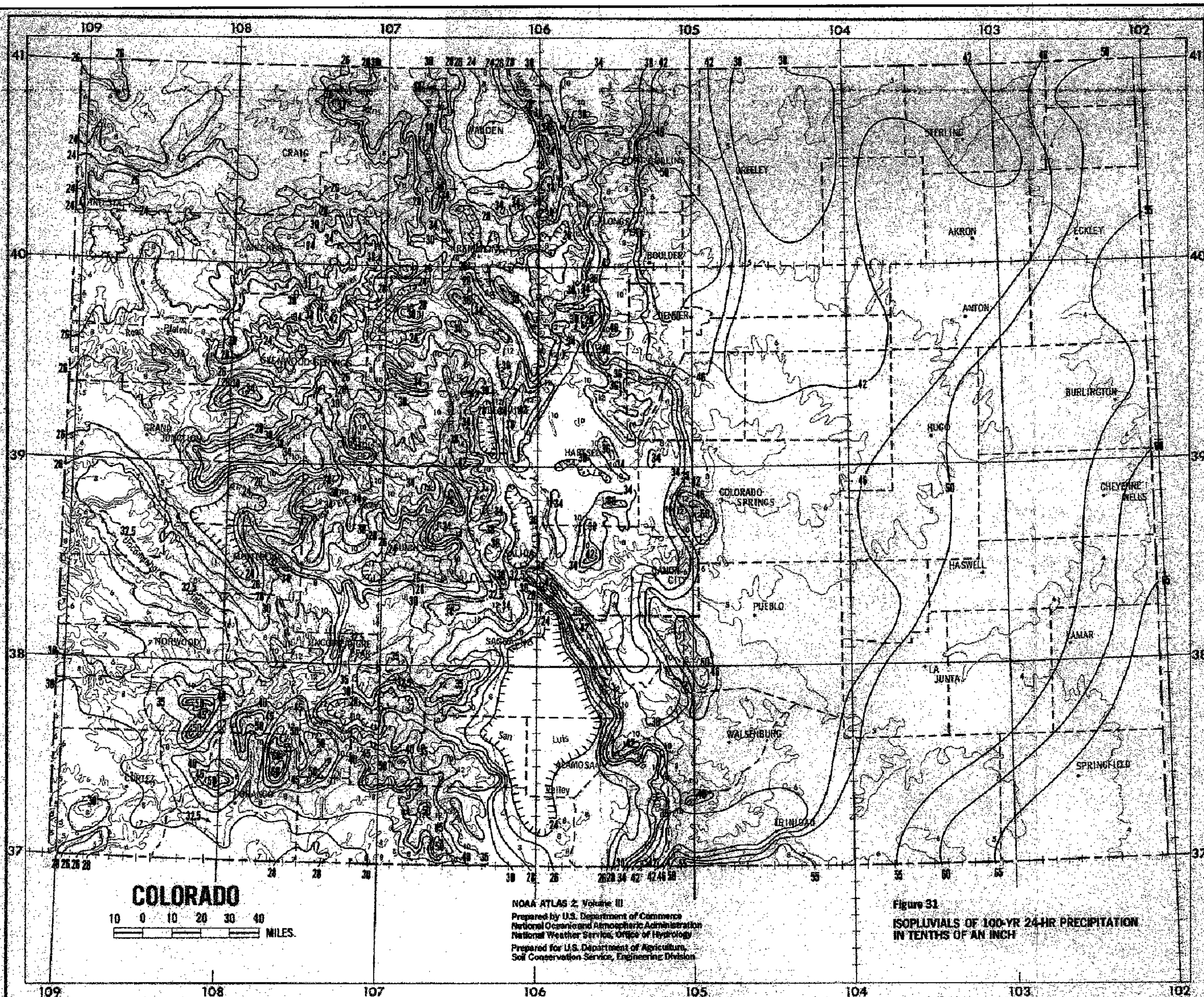


Figure 31
ISOPLUVIALS OF 100-YR 24-HR PRECIPITATION IN TENTHS OF AN INCH

FIGURE CH5-F112
100-YEAR, 24-HOUR PRECIPITATION MAP

REFERENCE: NOAA ATLAS 2, VOL III, 1973

VERSION: AUGUST 2002

WRC ENGINEERING, INC.

Q:/2080/FIGURES_CH5-F112.DWG, 4/10/02 MAS

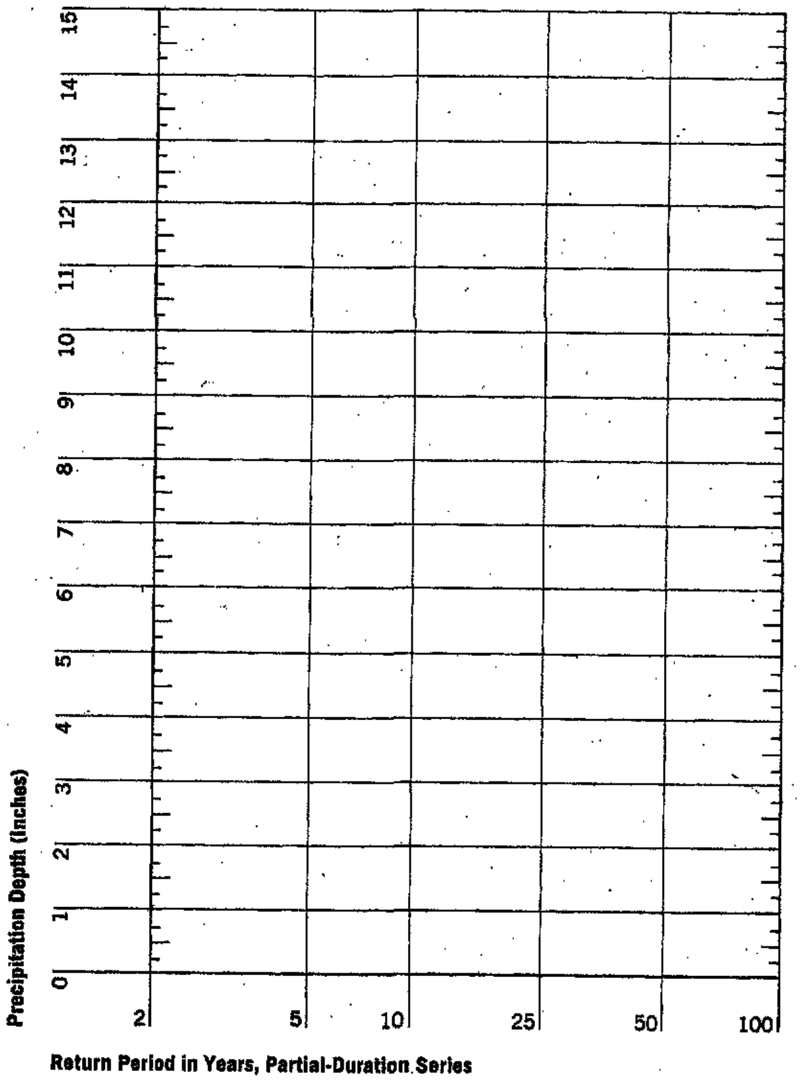


Figure 6. *Precipitation depth versus return period for partial-duration series.*

COLORADO STATEWIDE
DRAINAGE AND FLOODPLAIN MANAGEMENT CRITERIA MANUAL

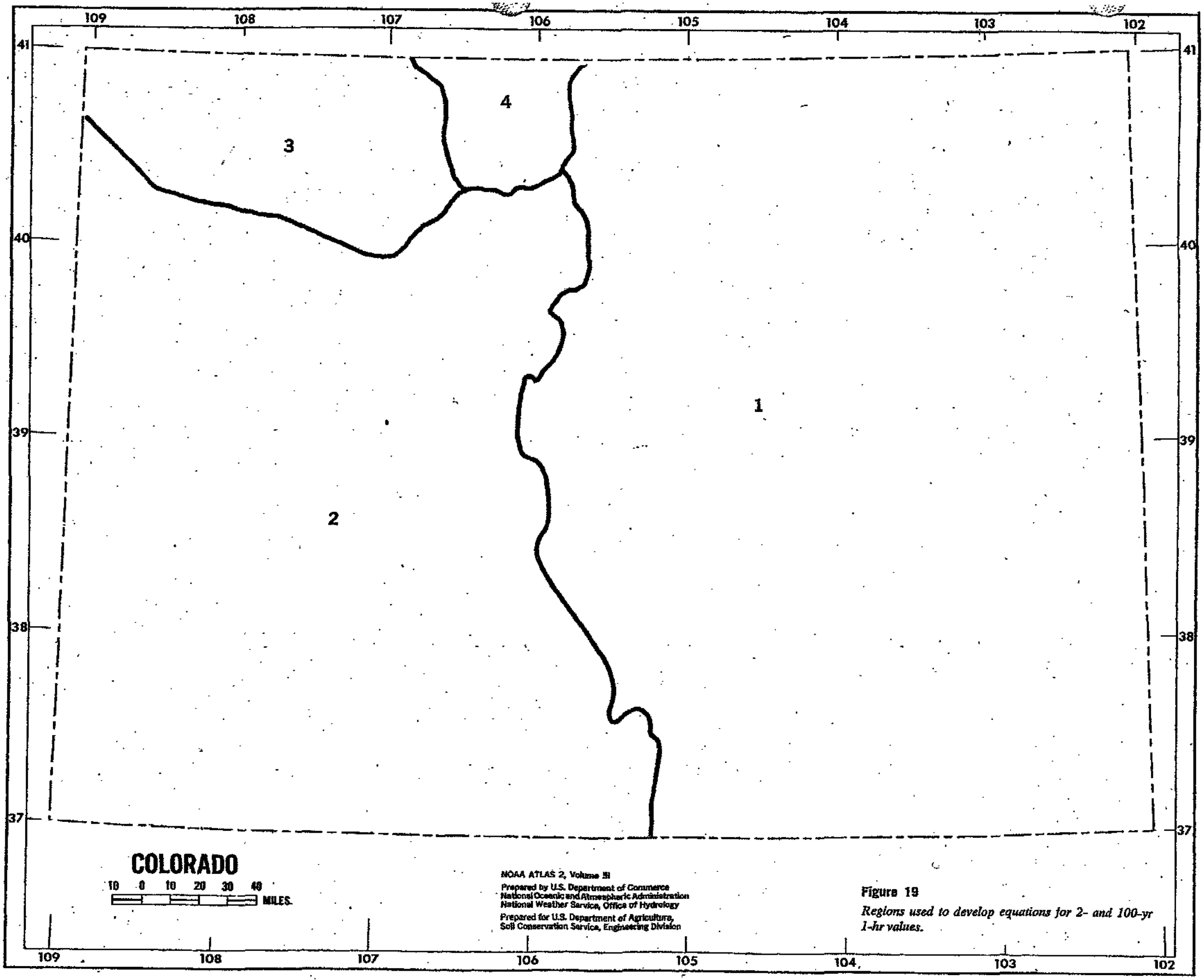


Figure 19

Regions used to develop equations for 2- and 100-yr 1-hr values.

WRC ENGINEERING, INC

VERSION: AUGUST 2002

REFERENCE:
NOAA ATLAS 2, VOL II, 1973

FIGURE CH5-F114
NOAA COLORADO
GEOGRAPHICAL REGIONS

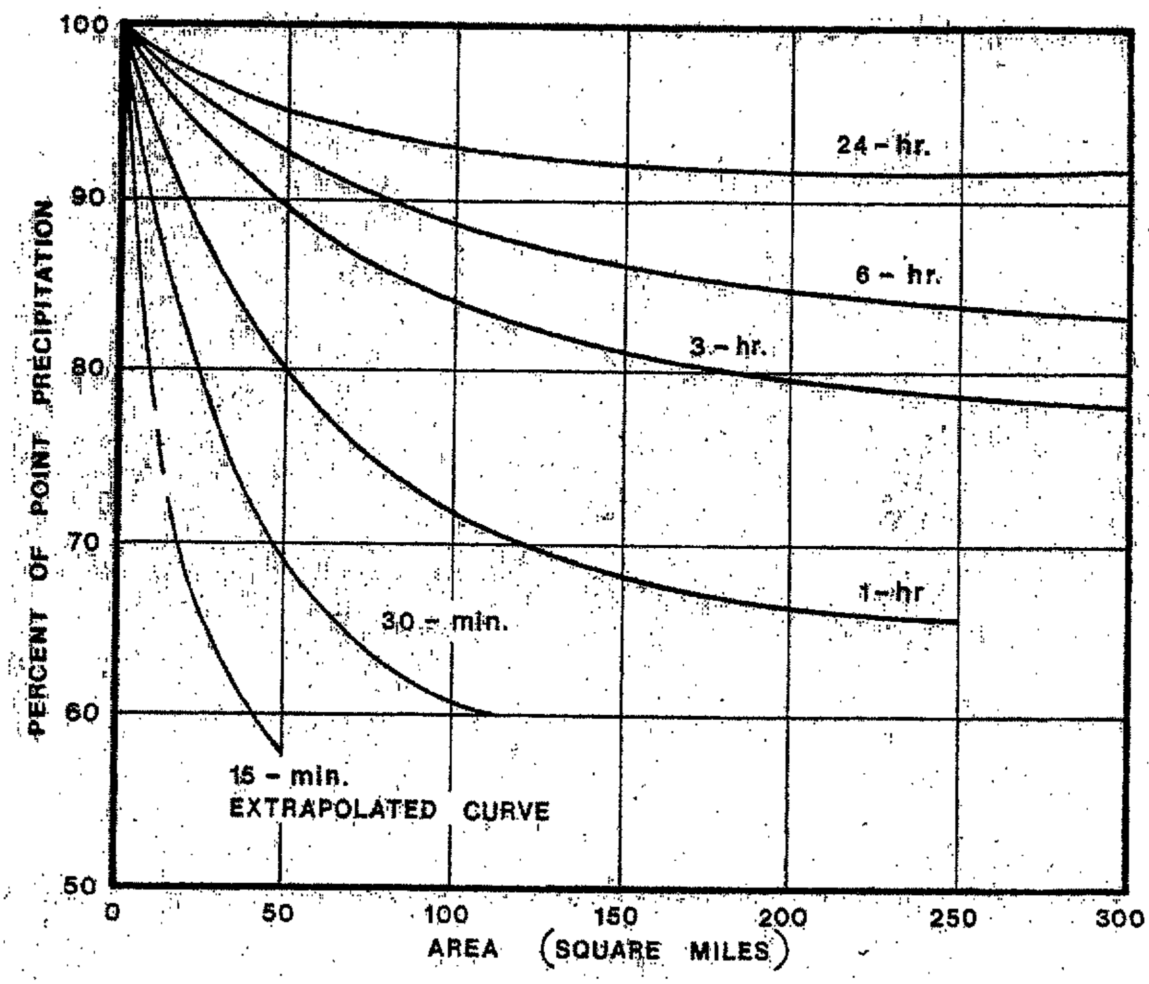


FIGURE CH5-F115

DEPTH-AREA ADJUSTMENT CURVES

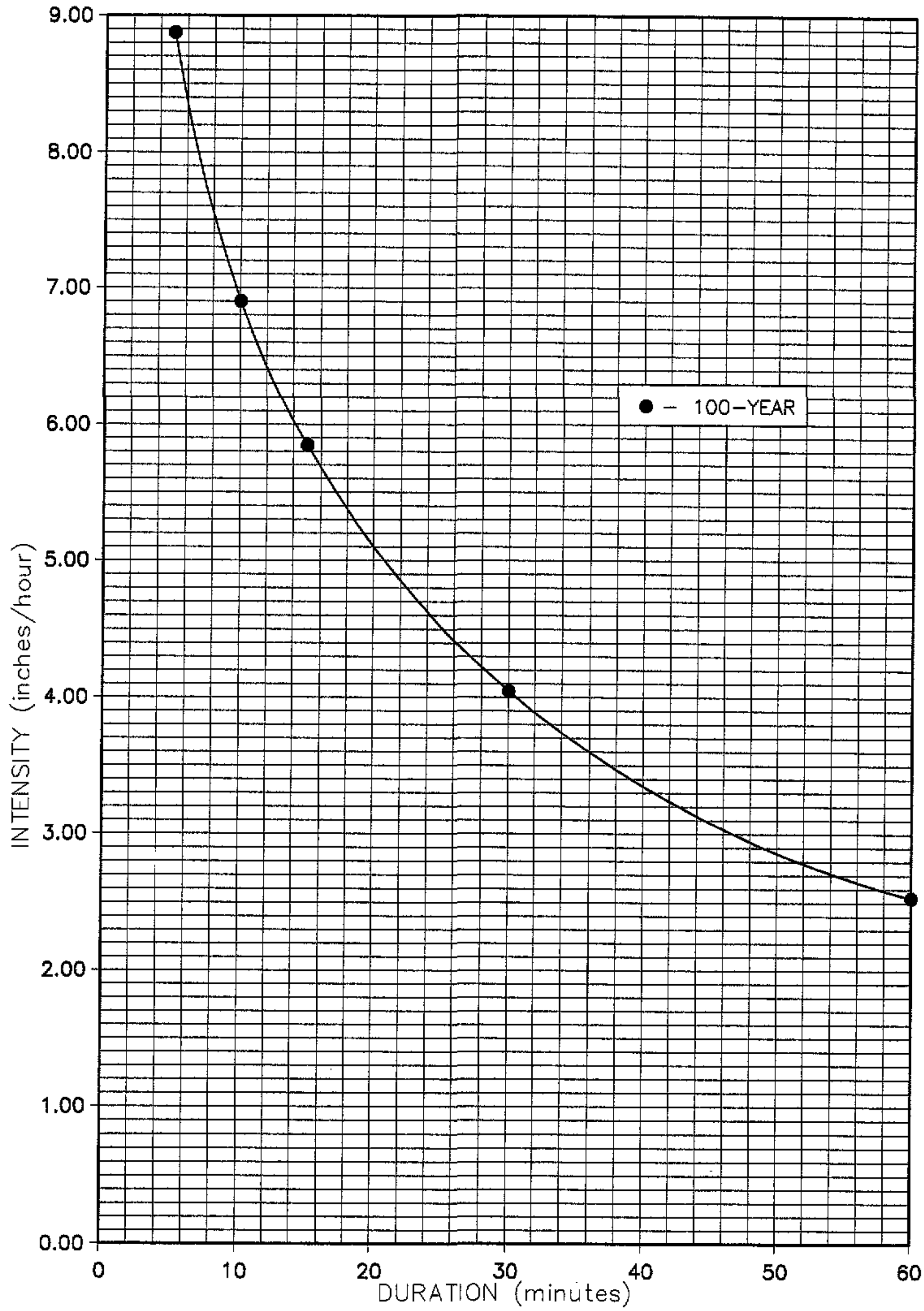


FIGURE CH5-F116

EXAMPLE 2: IDF CURVE RESULTS

CHAPTER 5

HYDROLOGY

SECTION 2.0

RUNOFF

CHAPTER 5
HYDROLOGY

SECTION 2.0
RUNOFF

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

LIST OF STANDARD FORMS

CHAPTER 5
HYDROLOGY

SECTION 2.0
RUNOFF

2.1 INTRODUCTION

This section presents the criteria and methodologies for determining the storm runoff peaks and volumes to be used in the preparation of storm drainage studies, plans, and facilities design in the State of Colorado. The practical analysis guidelines for the three deterministic hydrologic methods (Rational Method, HEC-1 (or HEC-HMS), and CUHP) are provided in this section. For more in-depth discussions of the modeling methods, theories, and procedures, the users of this manual are referred to the following program users manuals.

The practical analysis guidelines for the three deterministic hydrologic methods: Rational Method, HEC-1 or HEC-HMS, and CUHP are provided in this section.

- US Army Corps of Engineers, Hydrologic Engineering Center, HEC-1, Flood Hydrograph Package, User's Manual, June 1998
- Urban Drainage and Flood Control District (UDFCD), Users Manual, Colorado Urban Hydrograph Procedure Computer Program – CUHP, PC Version, 1995

2.2 RATIONAL METHOD

The Rational Method has been widely used for the sizing of storm sewers and for determining rainfall-runoff design values for small drainage basins with an area less than 160 acres. Even though this method has frequently come under academic criticism for its simplicity, no other practical drainage design method has evolved to such a level of general acceptance by practicing engineers. The Rational Formula method, when properly understood and applied, can produce satisfactory results for determining peak discharge estimates. The limit of application of the Rational Method is approximately 160 acres. The assumptions used in the Rational Method become less valid for larger areas. For drainage basins equal to or larger than 160 acres, more rigorous rainfall-runoff analysis should be performed utilizing HEC-1, HEC-HMS, or CUHP program.

The Rational Method has been widely used for the sizing of storm sewers and for determining rainfall-runoff design values for small drainage basins with an area less than 160 acres.

2.2.1 METHODOLOGY

The Rational Formula method is based on the following equation:

$$Q = CIA \qquad \text{(Eq. CH5-200)}$$

Q is defined as the maximum rate of runoff in cubic feet per second (actually, Q has units of acre inches per hour, which is approximately equal to the units of cubic feet per second). C is a runoff coefficient and represents the runoff-producing conditions of the subject land area. I is the average intensity of rainfall in inches per hour for a duration equal to the time of concentration. A is the contributing basin area in acres. The time of concentration is defined as the time required for water to flow from the hydraulically most distant part of the drainage area to the point under consideration.

2.2.2 ASSUMPTIONS

The basic assumptions made when applying the Rational Formula method are as follows:

1. The computed maximum rate of runoff to the design point is a function of the average rainfall rate during the time of concentration to that point.

2. The maximum rate of rainfall occurs during the time of concentration, and the design rainfall depth during the time of concentration is converted to the average rainfall intensity for the time of concentration.

3. The maximum runoff rate occurs when the entire area is contributing flow. However, this assumption has been modified from time to time when local rainfall/runoff data was used to improve calculated results.

2.2.3 LIMITATIONS ON METHODOLOGY

The Rational Formula method can adequately approximate the peak rate of runoff from a rainstorm in a given small basin. The critics of the method usually are unsatisfied with the fact that the answers are only approximations. A shortcoming of the Rational Formula method is that only one point on the runoff hydrograph is computed (the peak runoff rate), therefore, the estimated total runoff volume using the triangular hydrograph is not very accurate.

2.2.4 TIME OF CONCENTRATION – RATIONAL METHOD

As previously mentioned, the time of concentration is defined as the time required for runoff to flow from the hydraulically most distant part of the drainage basin to the desired point in the basin.

The time of concentration is defined as the time required for runoff to flow from the hydraulically most distant part of the drainage basin to the desired point in the basin.

The time of concentration consists of two components, the initial or overland flow time (usually as sheet flow) and the time of travel in a concentrated form (i.e., in a storm sewer, gutter, swale, channel, etc.). The initial flow time, t_i, is a function of the slope, surface cover, travel distance, soil, depression storage, and antecedent rainfall. The concentrated travel time, t_t, is a function of the hydraulic properties (i.e., surface roughness, slope, area, etc.) of the conveyance feature and the length of travel path. The time of concentration, for both urban and non-urban areas, is represented by the following equation:

$$t_c = t_i + t_t \qquad \text{(Eq. CH5-201)}$$

Where t_c = time of concentration (minutes)
t_i = initial or overland flow time (minutes)
t_t = concentrated travel time (minutes)

To aid in the computation of t_c, Standard Form CH5-SF201 has been developed to organize the computation.

The time of concentration, for both urban and non-urban areas, is represented by the following equation: $t_c = t_i + t_t$

2.2.4.1 TIME OF CONCENTRATION IN NON-URBANIZED BASINS

Non-urbanized areas are defined as drainage basins whose impervious area is less than 20% of the total area of the basin. The initial flow time for non-urbanized watersheds can be calculated using the following equation:

The initial travel time equation was originally developed by the Federal Aviation Administration (FAA, 1970) for use with the Rational Formula method. However, the equation is also valid for computation of the initial or overland flow time for the SCS Unit Hydrograph method using the appropriate flow runoff coefficient.

$$t_i = \frac{1.8(1.1 - R)\sqrt{L}}{\sqrt[3]{S}} \qquad \text{(Eq. CH5-202)}$$

Where t_i = initial or overland flow time (minutes)
R = flow runoff coefficient, for Rational Method, use $R = C_5$
C_5 = runoff coefficient for 5-year frequency (Table CH5-T201)
L = length of overland flow (feet) (500 feet maximum)
S = average basin slope along flow path (percent)

The overland flow distance is the flow length where the runoff flows as sheet flow. The maximum overland travel distance is 500 feet. Usually after a 500-foot flow length (although it may be less), the sheet runoff will concentrate into swales, gutters, and etc. and must be considered using the travel time equation.

After the runoff concentrates and flows in swales, gutters, or channels, the flow time should be determined using the travel time equation:

$$t_t = \frac{L}{60V} \qquad \text{(Eq. CH5-203)}$$

Where t_t = travel time (minutes)
L = concentrated flow length (feet)
V = flow velocity (feet per second)

The minimum time of concentration value in non-urban watersheds should be 10 minutes.

If necessary, the concentrated flow path in a given basin can be divided into multiple reaches, and the travel time for each reach should be computed separately and then combined to estimate the total travel time for the basin. The flow velocity, for preliminary calculations, can be estimated using Figure CH5-F201.

The total time of concentration (t_c) is then the sum of the initial flow time (t_i) and the travel time (t_t). The minimum time of concentration value, in non-urban watersheds, should be 10 minutes, and if the calculated value of t_c is less than 10 minutes, then 10 minutes should be used.

2.2.4.2 TIME OF CONCENTRATION IN URBANIZED BASINS

2.2.4.2.1 FIRST DESIGN POINT

The time of concentration at the first design point in urbanized areas should be estimated using two different methods. The first method utilizes the same procedure as previously discussed in Section 2.2.4.1 with the exception that the maximum overland flow length should be 300 feet. In an urban setting, overland flow occurs from the back of the lot to the street, in parking lots, in greenbelt areas, etc. and the length until the runoff concentrates is usually less than in a non-urban environment.

The second method of determining the time of concentration to the first design point is using the following equation:

$$t_c = \frac{L}{180} + 10 \qquad \text{(Eq. CH5-204)}$$

Where t_c = time of concentration to the first design point in an urban basin (minutes)
L = total basin length

The minimum time of concentration value for an urbanized watershed should not be less than 5 minutes.

This equation was developed by UDFCD using the rainfall and runoff data collected in the Denver region and essentially represents a calibration of the Rational Method for the Denver metro area.

The minimum time of concentration value using these two methods should be used to determine the rainfall intensity, which will be described in a subsequent section. The minimum time of concentration for an urbanized area should not be less than 5 minutes, and if the calculations provide a value less than 5 minutes, then 5 minutes should be used.

2.2.4.2.2 AFTER FIRST DESIGN POINT

The time of concentration calculated at the second design point and all subsequent design points should be calculated by adding the travel time to the downstream design points to the time of concentration calculated for the first design point. This relationship is represented by the following equation:

$$t_{c_n} = t_{c_1} + t_{t_2} + \ldots + t_{t_n} \qquad \text{(Eq. CH5-205)}$$

Where t_{cn} = total time of concentration at the n^{th} design point
t_{c1} = time of concentration at the first design point
t_{t2} = travel time from the first design point to the second design point
t_{tn} = travel time from the n-1 design point to the n^{th} design point

A common error in estimating the time of concentration is to not check the peak runoff resulting from only a portion of the basin. Sometimes the lower portion of the basin will produce a higher peak runoff than if the basin as a whole was considered. If this is not considered, it could result in conveyance facilities that are under-designed. This error occurs most frequently in long watersheds and in basins where the upper portion of the basin contains grassed areas (slow runoff velocities) and the downstream portion is developed urban land.

2.2.5 RAINFALL INTENSITY

The rainfall intensity, I, is the average rainfall rate in inches per hour for the period of maximum rainfall of a given frequency and duration. A time-intensity-frequency curve of a given drainage basin for various frequency events can be developed following the procedures outlined in the Rainfall Section, Chapter 5, Section 1. The rainfall intensity for a given design storm event can be determined from the time-intensity-frequency curve using the calculated time of concentration.

2.2.6 RUNOFF COEFFICIENT

The runoff coefficient, C, represents the integrated effects of infiltration, evaporation, retention, flow routing, and interception, all of which affect the time distribution and peak rate of runoff. Determination of the coefficient requires judgment and understanding on the part of the engineer. Table CH5-T201 presents the recommended values of C for various recurrence frequency storms. The values are presented for different surface characteristics as well as for different aggregate land uses.

A composite runoff coefficient can be computed on the basis of the percentage of different types of surfaces in the drainage basin. A composite C analysis will result in more accurate peak flow estimation. The runoff coefficients in Table CH5-T201 vary with recurrence frequency and therefore, further adjustments of the C factor are not needed.

2.2.7 APPLICATION OF THE RATIONAL FORMULA METHOD

The first step in applying the Rational Formula method is to obtain a topographic map and define the boundaries of all the relevant drainage basins. Basins to be defined include all basins tributary to the area of study and sub-basins within the study area. A field check and possibly field surveys should be made for each basin. At this stage of planning, the possibility for the diversion of transbasin waters should be identified. An example of how to apply the Rational Formula method is presented at the end of this chapter.

2.2.8 MAJOR STORM ANALYSIS

The major storm drainage basin does not always coincide with the minor storm drainage basin. This is often the case in urban areas where a low flow will stay next to a curb and follow the lowest grade, but when a large storm occurs, the water will be deep enough so that part of the water will overflow street crowns and flow into a new sub-basin.

When analyzing the major runoff occurring on an area that has a storm sewer system sized for the minor storm, care must be used when applying the Rational Formula method. Common application of the Rational Method assumes that all the runoff is collected by the storm sewer. For the minor storm design, the time of concentration is dependent upon the flow travel time in the pipe system. However, during the major storm runoff, the pipes will probably be at capacity and would not carry the additional water flowing to the inlets. This additional water then flows overland past the inlets, generally at a lower velocity than the flow in the storm sewers.

If a separate time of concentration analysis is made for the pipe flow and surface flow, a time lag between the surface flow peak and the pipe flow peak will occur. This lag, in effect, will allow the pipe to carry a larger portion of the major storm runoff than would be predicted using the minor storm time of concentration. The basis for this increased benefit is that the excess water from one inlet will flow to the next inlet downhill, using the overland route. If that inlet is also at capacity, the water will often continue on until capacity is available in the storm sewer. The analysis of this aspect of the interaction between the storm sewer system and major storm runoff is complex. The simplified approach of using the minor storm time of concentration for all frequency analysis is acceptable for use in the State of Colorado.

2.3 SCS UNIT HYDROGRAPH METHOD

The SCS Unit Hydrograph method uses the unit hydrograph theory as a basis for runoff computations.

The SCS Unit Hydrograph method was developed for the Soil Conservation Services (SCS) by Mr. Victor Mockus. The SCS Unit Hydrograph was derived from a large number of natural unit hydrographs from watersheds varying widely in size and geographic location. The SCS

Unit Hydrograph has been in use for many years and has produced satisfactory results for many applications.

2.3.1 METHODOLOGY

The SCS Unit Hydrograph method uses the unit hydrograph theory as a basis for runoff computations. The unit hydrograph theory computes rainfall excess hydrographs for a unit amount of rainfall excess applied uniformly over a sub-basin for a given unit of time (or unit duration). The rainfall excess hydrographs are then transformed to a sub-basin hydrograph by superimposing each excess hydrograph lagged by the unit duration.

The shape of the SCS Unit Hydrograph is based on studies of various natural unit hydrographs. The basic governing parameters of this curvilinear hydrograph are as follows:

1. The time-to-peak, T_p, of the unit hydrograph approximately equals 0.2 times the time-of-base, T_b.
2. The point of inflection of the falling leg of the unit hydrograph approximately equals 1.7 times T_p.

The unit hydrograph theory computes rainfall excess hydrographs for a unit amount of rainfall excess applied uniformly over a sub-basin for a given unit of time.

For ease of calculation, an equivalent triangular unit hydrograph was derived from the natural curvilinear unit hydrograph. From the triangular unit hydrograph, equations for the peak discharge, Q_p, time-to-peak, T_p, and the time of concentration, t_c were developed based on a single lag factor (TLAG). The discharge hydrograph is then determined for the SCS Unit Hydrograph method based on the storm excess precipitation applied to the unit hydrograph whose parameters are determined by TLAG. TLAG is defined and discussed below.

2.3.2 ASSUMPTIONS

The basic assumptions made when applying the SCS Unit Hydrograph method (and all other unit hydrograph methods) are as follows:

1. The effects of all physical characteristics of a given drainage basin are reflected in the shape of the storm runoff hydrograph for that basin.
2. At a given point on a stream, discharge ordinates of different unit graphs of the same unit time of rainfall excess are mutually proportional to respective volumes.
3. A hydrograph of storm discharge that would result from a series of bursts of excess rain or from continuous excess rain of variable intensity may be constructed from a series of overlapping unit graphs each resulting from a single increment of excess rain of unit duration.

2.3.3 LAG TIME

Input data for the Soil Conservation Service dimensionles s unit hydrograph method consists of a single parameter, TLAG, which is equal to the lag (in hours) between the center of mass of rainfall excess and the peak of the unit hydrograph.

Input data for the Soil Conservation Service dimensionless unit hydrograph method (SCS, 1985) consists of a single parameter, TLAG, which is equal to the lag (in hours) between the center of mass of rainfall excess and the peak of the unit hydrograph.

For small drainage basins (less than one square mile) with basin slopes less than ten percent, the lag time may be related to the time of concentration, t_c, by the following empirical relationship:

$$TLAG = 0.6\ t_c \qquad \text{(Eq. CH5-206)}$$

The t_c can be computed as presented previously in Section 2.2.4 with the exception that the flow runoff coefficient, R, should be calculated using the following equation. The equation was developed by converting curve numbers (CN) to typical C_5 runoff coefficients.

$$R = 0.0132{*}CN - 0.39 \qquad \text{(Eq. CH5-207)}$$

For larger drainage basins (greater than one square mile) and basins with a basin slope equal to or greater than ten percent, the lag time is generally governed mostly by the concentrated flow travel time, not the initial overland flow time. In addition, as the basin gets increasingly larger, the average flow velocity (and associated travel time) becomes more difficult to estimate. Therefore, for these basins, the following lag equation is recommended for use in computing TLAG:

$$TLAG = 22.1\ K_n\ (L\ L_c/S^{0.5})^{0.33} \qquad \text{(Eq. CH5-208)}$$

Where K_n = Roughness factor for the basin channels, Table CH5-T203
L = Length of longest watercourse (miles)
L_c = Length along longest watercourse measured upstream to a point opposite the centroid of the basin (miles)
S = Representative (average) slope of the longest watercourse (feet per mile)

This lag equation is based on the United States Bureau of Reclamation's analysis of the above parameters for several drainage basins in the southwest desert, Great Basin, and Colorado Plateau area (USBR, 1989). Since the SCS and the USBR define lag differently, this equation was developed by modifying the USBR's S-graph lag equation to correspond to the SCS's definition of the dimensionless unit hydrograph lag equation.

In order to obtain comparable results between the t_c calculation and the TLAG calculation, it is recommended that either method be used as a check of the other method for drainage areas around one square mile in size.

2.3.3.1 ROUGHNESS FACTOR

The selection of a proper roughness factor for use in the lag time calculation is highly subjective. Therefore, in order to obtain more consistent lag time and runoff analysis results, the roughness factor, K_n, shall be determined using the factors presented in Table CH5-T203. For partially developed basins, the roughness factor should be interpolated in relationship to the percent of each land use in the basin.

2.3.4 UNIT STORM DURATION

The typical unit storm duration should be 5 minutes unless conditions warrant otherwise.

The minimum unit duration, Δt, is dependent on the time of concentration of a given basin. If the basin is large (i.e., > one square mile), a larger unit duration may be used. If the basin is small (i.e., < one square mile) a smaller unit duration should be used. The unit duration, Δt, should be $\leq .25\ T_p$, where T_p is the time-to-peak of the unit hydrograph. For the State of Colorado, the typical unit storm duration should be 5 minutes unless conditions warrant otherwise.

2.3.5 PRECIPITATION LOSSES

Precipitation loss calculations are required for the SCS Unit Hydrograph method. Land surface interception, depression storage, and infiltration are referred to as precipitation losses. Interception and depression storage are intended to represent the surface storage of water by trees or grass, in local depressions in the ground surface, in cracks and crevices in parking lots or roofs, or in a surface area where water is not free to move as overland flow. Infiltration represents the movement of water to areas beneath the land surface.

The SCS Curve Number method has been used widely by the practicing engineers because the necessary data can be relatively easily obtained and the method can be easily applied to practical applications.

Two important factors should be noted about the precipitation loss computations to be used for the SCS Unit Hydrograph methods. First, precipitation which does not contribute to the runoff process is considered to be lost from the system. Second, the equations used to compute the losses do not provide for soil moisture or surface storage recovery.

The precipitation loss component of the SCS Unit Hydrograph method is considered to be sub-basin average (uniformly distributed over an entire sub-basin). In some instances, there are negligible precipitation losses for a portion of a sub-basin. This would be true for an area containing a lake, reservoir or impervious area. In this case, precipitation losses will not be computed for a specified percentage of the area labeled as impervious.

There are several methods that can be used to calculate the precipitation loss. These methods include the Initial and Uniform Loss Rate, Exponential Loss Rate, Holtan Loss Rate, Horton Loss Rate, Green-Ampt and SCS Curve Number method to name a few. The SCS Curve Number method is recommended because there is a lack of data for use of other methods. The SCS Curve Number method has been used widely by the practicing engineers because the necessary data can be relatively easily obtained and the method can be easily applied to practical applications. In the SCS Curve Number method, an average precipitation loss is determined for a computation interval and subtracted from the rainfall hyetograph. The resulting precipitation excess is used to compute an outflow hydrograph for a sub-basin.

2.3.5.1 SCS CURVE NUMBER

The SCS Curve Number Method uses a soil cover complex number (CN) for computing excess precipitation. The curve number is related to hydrologic soil group (A, B, C, or D), land use, treatment class (cover), and antecedent moisture condition.

For the State of Colorado, an AMC-II condition shall be used for determining storm runoff.

The Soil Conservation Service (SCS) and U.S. Department of Agriculture has instituted a soil classification system for use in Soil Survey maps across the country. Based on experimentation and experience, the agency has been able to relate the drainage characteristics of soil groups to a curve number, CN (SCS, 1985). The SCS provides information on relating soil group type to the curve number as a function of soil cover, land use type and antecedent moisture conditions.

Precipitation loss is calculated based on supplied values of CN and the initial surface moisture storage capacity, IA. CN and IA are related to a total runoff depth for a storm by the following relationships:

$$Q = (P\text{-}IA)^2/((P\text{-}IA) + S) \qquad \text{(Eq. CH5-209)}$$

$$S = (1000/CN) - 10 \qquad \text{(Eq. CH5-210)}$$

Where Q = Accumulated excess (inches)
P = Accumulated rainfall depth (inches)
IA = Initial surface moisture storage capacity (inches)
S = Current available soil moisture storage deficit (inches)

$$IA = .2S \qquad \text{(Eq. CH5-211)}$$

This relation is based on empirical evidence established by the Soil Conservation Service and is the default value in the HEC-1 program (HEC, 1990). Since the SCS method gives total excess for a storm (the difference between rainfall and precipitation loss), the incremental excess for a time period is computed as the difference between the accumulated excess at the end of the current period and the accumulated excess at the end of the previous period.

The SCS Curve Number Method uses a soil cover complex number (CN) for computing excess precipitation. The curve number CN is related to hydrologic soil group (A, B, C, or D), land use, treatment class (cover), and antecedent moisture condition. The soil group is determined from published soil maps for the area. These maps are usually published by the SCS. Land use and treatment class are usually determined during investigations in conjunction with aerial photographs. The procedures for determining land use and treatment class are found in Chapter 8 of the National Engineering Handbook, Section 4 (SCS, 1985). The antecedent moisture condition of the watershed is explained as follows:

The amount of rainfall in a period of 5 to 30 days preceding a particular storm is referred to as antecedent rainfall, and the resulting condition of the watershed in regard to potential runoff is referred to as an antecedent moisture condition. In general, the heavier the antecedent rainfall, the greater the direct runoff that occurs from a given storm. The effects of infiltration and evapo-transpiration during the antecedent period are also important, as they may increase or lessen the effect of antecedent rainfall. Because of the difficulties of determining antecedent storm conditions from data normally available, the conditions are reduced to three cases, AMC-I, AMC-II and AMC-III.

For the State of Colorado, an AMC-II condition shall be used for determining storm runoff.

Having determined the soil group, land use and treatment class, and the antecedent moisture condition, CN values can be determined from Table CH5-T202.

There will be areas to which the values in Table CH5-T202 do not apply. The percentage of impervious area for the various types of residential areas or the land use condition for the pervious portions may vary from the conditions assumed in Table CH5-T202. A curve for each pervious CN can be developed to determine the composite CN for any density of impervious area. Figure CH5-F202 has been developed assuming a CN of 98 for the impervious area. The curves in Figure CH5-F202 can help in estimating the increase in runoff as more and more land within a given area is covered with impervious material.

There are a number of methods available for computing the percentage of impervious area in a watershed. Some methods include using U.S. Geological Survey topographic maps, land use maps, aerial photographs, and field reconnaissance. Care must be exercised when using methods based on such parameters as population density, street density, and age of the development as a means of determining the percentage of impervious area. The available data on runoff from urban areas is not yet sufficient to validate widespread use of these methods. Therefore, the CN shall be based on Table CH5-T202 or Figure CH5-F202 in this Manual. A CN computation example is included in Section 2.5.2 of this chapter.

2.3.6 <u>SUB-BASIN SIZING</u>

The determination of the peak rate of runoff at a given design point is affected by the discretization of sub-basins in the subject basin. Typically, the more discrete the analysis of a given basin (more sub-basins), the larger the peak flow rate as compared to analysis of the basin with no sub-basins. Therefore, in order to obtain more consistent results between different designers as well as between different runoff models (i.e. Rational Formula

Method vs. SCS method), the following guidelines are recommended for basin discretization:

1. For drainage basins up to 100 acres in size, the maximum sub-basin size should be approximately 20 acres.

2. For drainage basins over 100 acres in size, increasingly larger sub-basins may be used as long as the land use and surface characteristics within each sub-basin are homogeneous. In addition, the sub-basin sizing should be consistent with the level of detail needed to determine peak flow rates at various design points within a given basin.

2.3.7 ROUTING OF HYDROGRAPHS

Whenever a large or a non-homogeneous basin is being investigated, the basin should be divided into smaller and more homogeneous sub-basins and the storm hydrograph for each sub-basin should be calculated. The user then must route and combine the individual sub-basin hydrographs to develop a network of storm hydrographs for the entire watershed. There are several methods available for use in flow routing which include:

a. Muskingum
b. Convex
c. Direct Translation
d. Storage-Discharge (Modified Puls)
e. Kinematic Wave
f. Diffusion Wave
g. Dynamic Wave
h. Muskingum-Cunge

The most commonly used routing techniques are Muskingum, Muskingum-Cunge (an approximate diffusion router), and Kinematic Wave (a finite-difference technique) methods.

The Muskingum-Cunge method provides reasonable results over a wide range of channel flow conditions and is relatively easy to use. The Muskingum-Cunge technique should be used for channels with standard prismatic shapes or channels with irregular cross section shapes. In some instances, an error message will occur with Muskingum-Cunge method that terminates the program computations. In this instance the Muskingum method should be used to route flows.

The Muskingum-Cunge technique should be used for channels with standard prismatic shapes or channels with irregular cross section shapes.

The Muskingum method should be used to route flows over a wide shallow floodplain. The Muskingum weighting factor, X coefficient ($0 \leq X \leq 0.5$), should be selected carefully to represent the routing reach conditions. The "X" coefficient of 0.2 to 0.3 is recommended for an average well-established natural channel, and 0.5 for a concrete lined channel.

The Kinematic Wave method should only be used in relatively short reaches such as those encountered in an urban environment. Numerical errors introduced when solving the Kinematic Wave technique may cause a greater attenuation of the peak flow than actually occurs. The Kinematic Wave technique can only be used for specific types of channel shapes (i.e., trapezoidal, rectangular, etc.).

The Muskingum method should be used to route flows over a wide shallow floodplain.

The Kinematic Wave method should only be used in relatively short reaches such as those encountered in an urban environment.

The reader is referred to the HEC-1 User's Manual for details on the development of Muskingum, Muskingum-Cunge and Kinematic Wave techniques and details on the parameters and procedures needed for their use in HEC-1 program.

Since the HEC-1 program computes hydrograph lagging based on internally selected computation interval, the user should always check that the peak generated from the internally selected computation interval are comparable to the result peaks shown in the output at the user determined intervals.

2.3.8 RESERVOIR ROUTING OF HYDROGRAPHS

The methodology for manual computation of reservoir routing is presented in this section. This method is computerized and is part of the HEC-1 program. The input requirements are explained in the HEC-1 Users Manual.

2.3.8.1 MODIFIED PULS ROUTING METHOD

The procedure for the original Puls Method was developed in 1928 by L.G. Puls of the U.S. Army Corps of Engineers. The method was modified in 1949 by the Bureau of Reclamation simplifying the computational and graphic requirements. The Modified Puls method is also referred to as the Storage-Indication or Goodrich Reservoir Routing Method. The differences, if any, are mainly in the form of the equation and means of initializing the routing. The procedures presented herein were obtained from Hydrology for Engineers (LINSLEY, 1975).

The principle of mass continuity for a channel reach can be expressed by the equation:

$$(I-D)t = \Delta S \qquad \text{(Eq. CH5-211)}$$

Where I is the inflow rate, D is the discharge rate, t is the time interval, and ΔS is the change in storage. If the average rate of flow during a given time period is equal to the average of the flows at the beginning and end of the period, the equation can be expressed as follows:

$$(I_1 + I_2)\, t/2 - (D_1 + D_2)\, t/2 = S_2 - S_1 \qquad \text{(Eq. CH5-212)}$$

Where the subscripts 1 and 2 refer to the beginning and end of time period t. Rearranging the equation gives the following form used for the Modified Puls method:

$$I_1 + I_2 + (2S_1/t - D_1) = (2S_2/t + D_2) \qquad \text{(Eq. CH5-213)}$$

Reservoir routing using the Modified Puls method may be analyzed using the HEC-1 computer program. The user is referred to the HEC-1 documentation for the required input parameters.

2.4 COLORADO URBAN HYDROGRAPH PROCEDURE (CUHP)

The Colorado Urban Hydrograph Procedure (CUHP) is a synthetic unit hydrograph methodology developed and calibrated for the Denver/Boulder Metro area. Therefore, the CUHP method should only be used for urban areas with similar hydrologic characteristics as Denver Metro area. The computer version of CUHP can be used to compute hydrographs from drainage basins larger than 90 acres. The procedures for developing hydrographs using CUHP, as outlined in the Urban Drainage and Flood Control District (UDFCD) Drainage Criteria Manual, Volume 1, "Runoff", shall be followed.

The Urban Drainage and Flood Control District's UDSWM program can be used to route the sub-basin hydrographs generated by CUHP through conveyance elements and storage facilities located within a drainage basin. UDSWM is the runoff block of EPA's SWMM (Storm Water Management Model, Version 2), as modified by the U.S. Army Corps of Engineers. UDSWM provides channel, pipe, and reservoir routing, and has been calibrated to work with CUHP. UDSWM can add and combine the hydrographs from sub-basins and conveyance elements as the flow proceeds downstream.

An example problem for the use of CUHP is provided at the end of this section. For detailed discussions of the program capabilities and model input parameters, please refer to the following program manuals.

- Colorado Urban Hydrograph Procedure Computer Program, PC Version of CUHP, User Manual, February, 2001
- Urban Drainage Storm Water Management Model (UDSWM), Users Manual, February 2001

2.5 EXAMPLES

2.5.1 EXAMPLE: RATIONAL FORMULA METHOD

Problem: Determine the 5-year flows at the design points within Rose Subdivision shown in Figure CH5-F203. The flow sequences are as follows: Design Point 1 flows to Design Point 2. Design Point 2 flows to Design Point 3. Design Point 5 flows to Design Point 6. Design Points 3 and 6 flow to Design Point 4. Design Point 4 flows into the proposed detention basin represented by Design Point 7 and Design Point 7 finally flows to Design Point 8 located in Doe Creek.

Solution:

Step 1: Estimate the flow runoff coefficients for each sub-basin in Rose Subdivision. Rose Subdivision is a single-family residential area with an average lot size of one-third acre. The flow runoff coefficient, R, is equal to the 5-year runoff coefficient, C_5, which are provided in Table CH5-T201.

$$R_A = R_B = R_C = R_D = R_E = R_F = R_G = C_5 = 0.45$$

Step 2: Calculate the initial overland flow time, t_i, for each sub-basin in Rose Subdivision. For this example, assume the lot depth in each sub-basin is 150 feet and slopes at a grade of 1.5% to the street.

$$t_{i_A} = t_{i_B} = t_{i_C} = t_{i_D} = t_{i_E} = t_{i_G} = \frac{1.8(1.1 - C_5)L^{1/2}}{S^{1/3}}$$

$$= \frac{1.8(1.1 - 0.45)(150)^{1/2}}{(1.5)^{1/3}} = 12.5\text{Minutes}$$

Step 3: Compute the travel time of the runoff in the street gutter to the designated design point using Figure CH5-F201. Only the calculation for the travel time to Design Point 1 is shown in the example. The results of the remaining travel time calculations are shown in Table CH5-T204.

Assuming the runoff combines and flows down the street at a 2.5 % grade, Figure CH5-F201 estimates the runoff velocity in the street to be:

V_A = 3.4 feet per second (fps)

The gutter flow length in sub-basin A is:

L_A = 900 feet

The travel time will be:

$$t_{t_A} = \frac{L}{60\ V} = \frac{900}{60\ *\ 3.4} = 4.4\text{ Minutes}$$

Step 4: Calculate the time of concentration using Equations CH5-201 and CH5-204 at Design Point 1. Select the smaller time estimated by the two equations as the final time of concentration at each design point.

$$t_{c_1} = t_i + t_t = 12.5 + 4.4 = 16.9\text{Minutes}$$

$$t_{c_1} = \frac{L}{180} + 10 = \frac{1050}{180} + 10 = 15.8\text{ Minutes}$$

Since Equation CH5-204 gives the smaller value, the time of concentration at Design Point 1 is:

$$t_{c_1} = 15.8 \text{ Minutes}$$

Step 5: Estimate the time of concentration at downstream design points. The flow calculated at each design point is used to estimate the flow velocity in the downstream pipe, gutter, swale, or channel.

This flow velocity is then used to calculate the time of travel to the next downstream design point. Table CH5-T204 shows the use of Standard Form CH5-SF201 and presents the results of the remaining calculations to determine the time of concentration at each design point.

Step 6: Determine the 5-year runoff coefficient (C_5) at each design point from Table CH5-T201.

$$C_{5_1} = C_{5_2} = C_{5_3} = C_{5_4} = C_{5_5} = C_{5_6} = C_{5_7} = C_{5_8} = 0.45$$

(Note: A composite runoff coefficient may need to be calculated if the drainage area flowing to the design point contains more than one land use or surface characteristic).

Step 7: Determine the 5-year rainfall intensity (I_5) at each design point using the time of concentration calculations in Steps 4 and 5 and the time-intensity-frequency curve for Rose subdivision. The detailed procedures for development of a site-specific time-intensity-frequency curve have been provided in Chapter 5, Section 1–"Rainfall". For the purpose of this example, use Figure CH5-F204 for Rose subdivision.

$$I_{5_1} = 3.10 \text{ Inches/hour}$$
$$I_{5_2} = 3.08 \text{ Inches/hour}$$
$$I_{5_3} = 2.97 \text{ Inches/hour}$$
$$I_{5_4} = 2.95 \text{ Inches/hour}$$
$$I_{5_5} = 3.10 \text{ Inches/hour}$$
$$I_{5_6} = 2.97 \text{ Inches/hour}$$
$$I_{5_7} = 2.92 \text{ Inches/hour}$$

Step 8: Calculate the 5-year peak flow (Q_5) at each design point using Equation CH5-200.

$$Q_{5_1} = C_{5_1} * I_{5_1} * A_1 = 0.45 * 3.10 * 4.13 = 5.8cfs$$
$$Q_{5_2} = 0.45 * 3.08 * 5.94 = 8.2cfs$$
$$Q_{5_3} = 0.45 * 2.97 * 8.26 = 11.0cfs$$
$$Q_{5_4} = 0.45 * 2.95 * 14.72 = 19.5cfs$$
$$Q_{5_5} = 0.45 * 3.10 * 3.36 = 4.7cfs$$
$$Q_{5_6} = 0.45 * 2.97 * 4.65 = 6.2cfs$$
$$Q_{5_7} = 0.45 * 2.92 * 15.5 = 20.4cfs$$

Step 9: The 100-year peak flow at each design point was not estimated in this example problem but may be obtained by repeating Steps 6 through 8 using 100-year runoff coefficients and rainfall intensities.

APPLICATION: The results from the Rational Formula Method can be used to design the drainage system in an urban environment.

2.5.2 EXAMPLE: SCS UNIT HYDROGRAPH METHOD (HEC-1)

Problem: Determine the current conditions 100-year, 24-hour runoff hydrograph on Doe Creek immediately upstream of John Boulevard and Rose Subdivision (see Figure CH5-F203).

Solution:

Step 1: Measure the drainage area of the basin. For this example, assume the drainage area is:

DA = 3.34 square miles = 2140 acres

Step 2: Estimate the average curve number of the basin. Assume the basin can be divided into the following land uses.

Land Use	Soil Type	CN	Area (Acres)
Herbaceous (Fair Cond.)	B	71	840
Herbaceous (Fair Cond.)	C	81	500
Residential (1/2 ac. lots)	B	70	800

CN_{Ave} = (70*800+81*500+71*840)/2140=72.96

Use CN = 73

CHAPTER 5
HYDROLOGY

SECTION 2.0
RUNOFF

Step 3: Measure the length of the longest watercourse (L).

Assume, L = 22,100 feet = 4.19 miles

Step 4: Measure the length along Doe Creek from the John Boulevard Bridge to the point opposite the centroid of the basin (L_c).

Assume L_c = 2.05 miles

Step 5: Calculate the average slope of Doe Creek.

Assume, Elevation of furthest upstream point = 7,276 feet
Elevation at John Boulevard = 4,920 feet

Slope = (7,276-4,920)/4.19 = 563 feet/mile

Step 6: Estimate the average roughness factor, K_n for Doe Creek using Table CH5-T203.

Land Use	K_n	Area
Herbaceous	0.08	1,340
Residential (1/2 ac. lots)	0.07	800

$$K_n = (0.08*1340+0.07*800)/2{,}140 = 0.076$$

Step 7: Calculate the lag time (TLAG) for the SCS dimensionless unit hydrograph using Equation CH5-208.

$$TLAG = 22.1*K_n*(L*L_c/S^{0.5})^{0.33}$$

$$TLAG = 22.1*0.076*(4.19*2.05/563^{0.5})^{0.33} = 1.20 \text{ hours}$$

Step 8: Input the necessary information into HEC-1 program and run HEC-1 to obtain the 100-year, 24-hour storm hydrograph at John Boulevard Bridge. The HEC-1 program will require KK, BA, LS, PH, and UD cards to model a subdivision. The rainfall distribution information was obtained from Section 1.7.1, Chapter 5. The results are provided in Table CH5-T205.

2.5.3 EXAMPLE: COLORADO URBAN HYDROGRAPH PROCEDURE (CUHP)

Problem: Determine the future conditions hydrograph for the major storm event (100-year) on Smith Creek immediately upstream of Mead Boulevard.

Solution:

Step 1: Determine the drainage boundary of Smith Creek at Mead Boulevard and measure the drainage area. For this example, assume the drainage area is:

DA = 2,140 acres = 3.34 square miles

Step 2: Measure the length of the longest watercourse.

For this example, assume L = 22,100 feet = 4.19 miles

Step 3: Measure the length along Smith Creek from Mead Boulevard to the point opposite the centroid of the basin.

For this example, assume L_c = 10,800 feet = 2.05 miles

Step 4: Determine the length-weighted, corrected basin average slope (ft/ft) of Smith Creek using Equation RO-9 and Figure RO-10 from the UDFCD Drainage Criteria Manual.

Assume the basin can be divided into the following land uses.

Reach	Slope (ft./ft.)	Corrected Slope Using Figure RO-10 (ft./ft.)	Length (feet)
REACH 1 (Upstream)	0.012	0.012	5,000
REACH 2 (Middle)	0.008	0.008	5,000
REACH 3 (Downstream)	0.005	0.005	12,100

$$SLOPE = \left(\frac{5{,}000 * (0.012)^{0.24} + 5{,}000 * (0.008)^{0.24} + 12{,}100 * (0.005)^{0.24}}{22{,}100} \right)^{4.17}$$

Slope = 0.0069 ft/ft

Step 5: Estimate the percent of the Smith Creek drainage area that will be impervious in the future conditions.

Land Use	Area (acres)	Percent Impervious	
Future Residential (¼ acre lots)	1,530	38	From Table CH5-T201
Commercial	120	70	From Table CH5-T201
Single Family (3 Units/Acre)	490	30	From Table CH5-T201

The composite percent impervious area is:

$$\%\text{Impervious} = \frac{1{,}530 * 38 + 120 * 70 + 490 * 30\%}{2{,}140} = 38$$

Step 6: Calculate the amount of depression storage on the pervious area in the drainage basin using Table RO-6 from the UDFCD Drainage Criteria Manual. For this example, assume that the entire pervious area will be covered with grass. If the cover type varies, a composite depression loss value should be estimated.

Land Use	Area (acres)	Percent Impervious	Depression Losses (inches)
Future Residential (¼ acre lots)	1,530	62	0.35
Commercial	120	30	0.35
Single Family (3 Units/Acre)	490	70	0.35

Step 7: Calculate the quantity of depression storage on the impervious area in the drainage basin using Table RO-6 from the UDFCD Drainage Criteria Manual.

Land Use	Area (acres)	Percent Impervious	Depression Losses (inches)
Future Residential (¼ acre lots)	1,530	38	0.05
Commercial	120	70	0.1
Single Family (3 Units/Acre)	490	30	0.05

The composite value for the amount of depression storage on the impervious area is:

$$= \frac{1{,}530*38*0.05+120*70*0.1+490*30*0.05}{1{,}530*38+120*70+490*30} = 0.055$$

Step 8: Determine the initial and final infiltration rates and Decay Coefficient used in Horton's Equation in CUHP. The values can be obtained using Table RO-7 from the UDFCD Drainage Criteria Manual.

SCS Soil Group	Area (acres)	Initial Infiltration Rate (inches/hour)	Final Infiltration Rate (inches/hour)	Decay Coefficient
B	1040	4.5	0.6	0.0018
C	1100	3.0	0.5	0.0018

The composite value of the initial infiltration rate is:

$$f_i = \frac{1{,}040*4.5+1{,}100*3.0}{2{,}140} = 3.73 in/hr$$

CHAPTER 5
HYDROLOGY

SECTION 2.0
RUNOFF

The composite value of the final infiltration rate is:

$$f_o = \frac{1{,}040 * 0.6 + 1{,}100 * 0.5}{2{,}140} = 0.55 in / hr$$

The composite Decay Coefficient is 0.0018.

Step 9: Determine the Smith Creek watershed rainfall data to be used in CUHP. Since the drainage area is between 90 acres and 10 square miles, a 100-year, 2-hour storm distribution without depth-area adjustments should be used. The detailed procedures for development of a site-specific CUHP rainfall distribution data are provided in Chapter 5, Section 1-Rainfall. For this example, use 100-year, 1-hour rainfall depth of 3.06 inches. CUHP program can generate the standard 2-hour storm distribution from a 1-hour point rainfall value based on the procedures outlined in the "Rainfall' section.

Step 10: Input the necessary information into the CUHP program and run CUHP to obtain the 100-year, 2-hour storm hydrograph at Mead Boulevard Bridge. The peak flow is 2,984 cfs. The output from CUHP is presented in Table CH5-T206.

RATIONAL FORMULA METHOD RUNOFF COEFFICIENTS

Land Use or Surface Characteristics	Aver. % Impervious Area	Runoff Coefficients, C 5-Year (C_5)	10-Year (C_{10})	100-Year (C_{100})
Business/Commercial:				
Downtown Areas	85	.82	.84	.85
Neighborhood Areas	70	.65	.70	.80
Residential:				
(Average Lot Size)				
⅛ Acre or Less (Multi-Unit)	65	.60	.68	.78
¼ Acre	38	.50	.55	.65
⅓ Acre	30	.45	.50	.60
½ Acre	25	.40	.45	.55
1 Acre	20	.35	.40	.50
2 Acre	12	.30	.35	.40
Industrial:	72	.68	.72	.82
Others:				
Schools	50	.50	.60	.70
Railroad Yard Area	20	.25	.35	.45
Open Space:				
Parks, Cemeteries	7	.18	.25	.45
Playgrounds	13	.20	.30	.50
Undeveloped Areas:				
Grass, Sandy Soil	0	.05	.05	.20
Grass, Clayey Soil	0	.15	.25	.50
Streets/Roads:				
Paved	100	.88	.90	.93
Gravel (packed)	40	.45	.50	.60
Drives/Walks:	96	.87	.88	.89
Roofs:	90	.85	.90	.90

Notes:

1. Composite runoff coefficients shown for Residential, Industrial, and Business/Commercial Areas assume irrigated grass landscaping for all previous areas. For development with landscaping other than irrigated grass, the designer must develop project specific composite runoff coefficients from the surface characteristics presented in this table.

Chapter 2 | Estimating Runoff | Technical Release 55
Urban Hydrology for Small Watersheds

Table 2-2a Runoff curve numbers for urban areas [1]

Cover description		Curve numbers for hydrologic soil group			
Cover type and hydrologic condition	Average percent impervious area [2]	A	B	C	D
Fully developed urban areas (vegetation established)					
Open space (lawns, parks, golf courses, cemeteries, etc.) [3]:					
Poor condition (grass cover < 50%)		68	79	86	89
Fair condition (grass cover 50% to 75%)		49	69	79	84
Good condition (grass cover > 75%)		39	61	74	80
Impervious areas:					
Paved parking lots, roofs, driveways, etc. (excluding right-of-way)		98	98	98	98
Streets and roads:					
Paved; curbs and storm sewers (excluding right-of-way)		98	98	98	98
Paved; open ditches (including right-of-way)		83	89	92	93
Gravel (including right-of-way)		76	85	89	91
Dirt (including right-of-way)		72	82	87	89
Western desert urban areas:					
Natural desert landscaping (pervious areas only) [4]		63	77	85	88
Artificial desert landscaping (impervious weed barrier, desert shrub with 1- to 2-inch sand or gravel mulch and basin borders)		96	96	96	96
Urban districts:					
Commercial and business	85	89	92	94	95
Industrial	72	81	88	91	93
Residential districts by average lot size:					
1/8 acre or less (town houses)	65	77	85	90	92
1/4 acre	38	61	75	83	87
1/3 acre	30	57	72	81	86
1/2 acre	25	54	70	80	85
1 acre	20	51	68	79	84
2 acres	12	46	65	77	82
Developing urban areas					
Newly graded areas (pervious areas only, no vegetation) [5]		77	86	91	94
Idle lands (CN's are determined using cover types similar to those in table 2-2c).					

1 Average runoff condition, and $I_a = 0.2S$.
2 The average percent impervious area shown was used to develop the composite CN's. Other assumptions are as follows: impervious areas are directly connected to the drainage system, impervious areas have a CN of 98, and pervious areas are considered equivalent to open space in good hydrologic condition. CN's for other combinations of conditions may be computed using figure 2-3 or 2-4.
3 CN's shown are equivalent to those of pasture. Composite CN's may be computed for other combinations of open space cover type.
4 Composite CN's for natural desert landscaping should be computed using figures 2-3 or 2-4 based on the impervious area percentage (CN = 98) and the pervious area CN. The pervious area CN's are assumed equivalent to desert shrub in poor hydrologic condition.
5 Composite CN's to use for the design of temporary measures during grading and construction should be computed using figure 2-3 or 2-4 based on the degree of development (impervious area percentage) and the CN's for the newly graded pervious areas.

Table 2-2b Runoff curve numbers for cultivated agricultural lands [1]

Cover description			Curve numbers for hydrologic soil group			
Cover type	Treatment [2]	Hydrologic condition [3]	A	B	C	D
Fallow	Bare soil	—	77	86	91	94
	Crop residue cover (CR)	Poor	76	85	90	93
		Good	74	83	88	90
Row crops	Straight row (SR)	Poor	72	81	88	91
		Good	67	78	85	89
	SR + CR	Poor	71	80	87	90
		Good	64	75	82	85
	Contoured (C)	Poor	70	79	84	88
		Good	65	75	82	86
	C + CR	Poor	69	78	83	87
		Good	64	74	81	85
	Contoured & terraced (C&T)	Poor	66	74	80	82
		Good	62	71	78	81
	C&T+ CR	Poor	65	73	79	81
		Good	61	70	77	80
Small grain	SR	Poor	65	76	84	88
		Good	63	75	83	87
	SR + CR	Poor	64	75	83	86
		Good	60	72	80	84
	C	Poor	63	74	82	85
		Good	61	73	81	84
	C + CR	Poor	62	73	81	84
		Good	60	72	80	83
	C&T	Poor	61	72	79	82
		Good	59	70	78	81
	C&T+ CR	Poor	60	71	78	81
		Good	58	69	77	80
Close-seeded or broadcast legumes or rotation meadow	SR	Poor	66	77	85	89
		Good	58	72	81	85
	C	Poor	64	75	83	85
		Good	55	69	78	83
	C&T	Poor	63	73	80	83
		Good	51	67	76	80

1 Average runoff condition, and I_a=0.2S
2 Crop residue cover applies only if residue is on at least 5% of the surface throughout the year.
3 Hydraulic condition is based on combination factors that affect infiltration and runoff, including (a) density and canopy of vegetative areas, (b) amount of year-round cover, (c) amount of grass or close-seeded legumes, (d) percent of residue cover on the land surface (good ≥ 20%), and (e) degree of surface roughness.

Poor: Factors impair infiltration and tend to increase runoff.

Good: Factors encourage average and better than average infiltration and tend to decrease runoff.

Table 2-2c Runoff curve numbers for other agricultural lands [1]

Cover description — Cover type	Hydrologic condition	Curve numbers for hydrologic soil group — A	B	C	D
Pasture, grassland, or range—continuous forage for grazing. [2]	Poor	68	79	86	89
	Fair	49	69	79	84
	Good	39	61	74	80
Meadow—continuous grass, protected from grazing and generally mowed for hay.	—	30	58	71	78
Brush—brush-weed-grass mixture with brush the major element. [3]	Poor	48	67	77	83
	Fair	35	56	70	77
	Good	30 [4]	48	65	73
Woods—grass combination (orchard or tree farm). [5]	Poor	57	73	82	86
	Fair	43	65	76	82
	Good	32	58	72	79
Woods. [6]	Poor	45	66	77	83
	Fair	36	60	73	79
	Good	30 [4]	55	70	77
Farmsteads—buildings, lanes, driveways, and surrounding lots.	—	59	74	82	86

1 Average runoff condition, and $I_a = 0.2S$.
2 *Poor:* <50%) ground cover or heavily grazed with no mulch.
Fair: 50 to 75% ground cover and not heavily grazed.
Good: > 75% ground cover and lightly or only occasionally grazed.
3 *Poor:* <50% ground cover.
Fair: 50 to 75% ground cover.
Good: >75% ground cover.
4 Actual curve number is less than 30; use CN = 30 for runoff computations.
5 CN's shown were computed for areas with 50% woods and 50% grass (pasture) cover. Other combinations of conditions may be computed from the CN's for woods and pasture.
6 *Poor:* Forest litter, small trees, and brush are destroyed by heavy grazing or regular burning.
Fair: Woods are grazed but not burned, and some forest litter covers the soil.
Good: Woods are protected from grazing, and litter and brush adequately cover the soil.

Table 2-2d Runoff curve numbers for arid and semiarid rangelands [1]

Cover description		Curve numbers for hydrologic soil group			
Cover type	Hydrologic condition [2]	A [3]	B	C	D
Herbaceous—mixture of grass, weeds, and low-growing brush, with brush the minor element.	Poor		80	87	93
	Fair		71	81	89
	Good		62	74	85
Oak-aspen—mountain brush mixture of oak brush, aspen, mountain mahogany, bitter brush, maple, and other brush.	Poor		66	74	79
	Fair		48	57	63
	Good		30	41	48
Pinyon-juniper—pinyon, juniper, or both; grass understory.	Poor		75	85	89
	Fair		58	73	80
	Good		41	61	71
Sagebrush with grass understory.	Poor		67	80	85
	Fair		51	63	70
	Good		35	47	55
Desert shrub—major plants include saltbush, greasewood, creosotebush, blackbrush, bursage, palo verde, mesquite, and cactus.	Poor	63	77	85	88
	Fair	55	72	81	86
	Good	49	68	79	84

1 Average runoff condition, and $I_a = 0.2S$. For range in humid regions, use table 2-2c.
2 Poor: <30% ground cover (litter, grass, and brush overstory).
Fair: 30 to 70% ground cover.
Good: > 70% ground cover.
3 Curve numbers for group A have been developed only for desert shrub.

LAG EQUATION ROUGHNESS FACTORS

LAND USE	RANGE OF AVERAGE IMPERVIOUS AREA	K_n
Developed Areas	70 - 85	.05
Commercial/Industrial/Office/Business	30 - 65	.05
High and Medium Density Residential		
Low Density Residential	20 - 25	.07
Rural Residential	10 - 15	.08
Irrigated Grass (Golf course/Parks/ Cemeteries)	0 - 5	.10
Undeveloped Areas		
Rock Outcroppings	-	.04
Irrigated Agriculture	-	.10
Rangelands:		
Herbaceous (grasses)	-	.08
Mixed grass and shrub	-	.09
Heavy shrub/brush	-	.10
Forest (Evergreen)	-	.15

0:/2080/FIGURES, CH5-T202A.DWG, 4/9/02 MAS

DEVELOPMENT ROSE SUBDIVISION

CALCULATED BY__________ DATE____

SUB-BASIN DATA			INITIAL/OVERLAND TIME (t_i)			TRAVEL TIME (t_t)				t_c ($t_i + t_t$)	t_c URBANIZED BASINS CHECK		FINAL t_c	REMARKS
DESIG: (1)	R (2)	AREA Ac (3)	LENGTH Ft (4)	SLOPE % (5)	t_i Min (6)	LENGTH Ft (7)	SLOPE % (8)	VEL. FPS (9)	t_t Min (10)	t_c Min (11)	TOTAL LENGTH Ft (12)	t_c=(L/180)+10 Min (13)	Min (14)	
A	.45	2.32	150	1.5	12.5	900	2.5	3.4	4.4	16.9	1050	15.8	15.8	
B	.45	1.81	150	1.5	12.5	600	2.5	3.4	2.9	15.4	750	14.2	14.2	
DP1		4.13											15.8	
C	.45	1.81	150	1.5	12.5	600	2.5	3.4	2.9	15.4	750	14.2	14.2	
DP2		5.94				40	2.5	3.4	0.2				16.0	DP1 TO DP2
D	.45	2.32	150	1.5	12.5	600	2.5	3.4	2.9	15.4	750	14.2	14.2	
DP3		8.26				260	2.5	3.4	1.3				17.3	DP2 TO DP3
DP5	.45	3.36	150	1.5	12.5	900	2.5	3.4	4.4	16.9	1050	15.8	15.8	SUB-BASIN F
G	.45	1.29	150	1.5	12.5	300	2.5	3.4	1.5	14.0	450	12.5	12.5	
DP6		4.65				300	2.5	3.4	1.5				17.3	DP5 TO DP6
E	.45	1.81	150	1.5	12.5	600	2.5	3.4	2.9	15.4	750	14.2	14.2	
DP4		14.72				40	2.5	3.4	0.2				17.5	DP3 TO DP4 DP6 TO DP4
H	.45	0.78	150	1.5	12.5	–	–	–	–	12.5	150	10.8	10.8	
DP7		15.5				100	2.5	3.4	0.5				18.0	DP4 TO DP7

$$t_i = 1.8\ (1.1 - R)\ L^{1/2} / S^{1/3}$$

TIME OF CONCENTRATION

COLORADO STATEWIDE
DRAINAGE AND FLOODPLAIN MANAGEMENT CRITERIA MANUAL

WRC ENGINEERING, INC
VERSION: AUGUST 2002
REFERENCE:

TABLE CH5-T204
EXAMPLE: TIME OF CONCENTRATION

```
                           HEC-1 INPUT

LINE           ID.......1.......2.......3.......4.......5.......6.......7.......8.......9......10

               *DIAGRAM
   1           ID

   2           ID      COLORADO STATEWIDE DRAINAGE AND FLOODPLAIN MANAGEMENT CRITERIA MANUAL
   3           ID      HEC-1 EXAMPLE RUN

   4           ID      100-YEAR, 24-HOUR STORM EVENT
   5           ID      PREPARED BY WRC ENGINEERING INC.
   6           ID      FILE NAME: CH5-T205.DAT
   7           ID

   8           IT       5                   300

   9           IO       5

  10           JR     PREC    0.995

  11           KK     DOE1    DOE CREEK BASIN #1

  12           BA     3.34

  13           PH             0.001    0.74    1.46    2.56    2.92    3.18    3.6    4.3    5.0
  14           LS               73

  15           UD      1.2

  16           ZZ


                 SCHEMATIC DIAGRAM OF STREAM NETWORK
INPUT
 LINE     (V) ROUTING          (--->) DIVERSION OR PUMP FLOW

  NO.     (.) CONNECTOR        (<---) RETURN OF DIVERTED OR PUMPED FLOW

  11        DOE1


   PEAK FLOW AND STAGE (END-OF-PERIOD) SUMMARY FOR MULTIPLE PLAN-RATIO ECONOMIC COMPUTATIONS
                    FLOWS IN CUBIC FEET PER SECOND, AREA IN SQUARE MILES
                               TIME TO PEAK IN HOURS

                                                  RATIOS APPLIED TO PRECIPITATION
OPERATION        STATION     AREA     PLAN        RATIO 1
                                                     1.00

HYDROGRAPH AT
+                   DOE1      3.34      1    FLOW     1742.
                                             TIME     13.33


*** NORMAL END OF HEC-1 ***
```

O:/2080/FIGURES. CH5-SF201.DWG, 4/5/02 MAS

1 U.D.F.C.D. CUHP RUNOFF ANALYSIS EXECUTED ON DATE 1/ 7/2002 AT TIME 13:51

CUHPF/PC RELEASE 2A (32-BIT VER) SEPTEMBER 10, 1998

PRINT OPTION NUMBER SELECTED FOR THIS BASIN IS 2

Example – Smith Creek at Mead Blvd

BASIN ID: Smith1 -- BASIN COMMENT: Smith Creek at mead

AREA (SQMI)	LENGTH OF BASIN (MI)	DIST TO CENTROID (MI)	IMPERV. AREA (PCT)	SLOPE (FT/FT)	UNIT DURATION (MIN)
3.34	4.19	2.05	38.00	.0069	5.00

COEFFICIENT (REFLECTING TIME TO PEAK)	COEFFICIENT (RELATED TO PEAK RATE OF RUNOFF)
.094	.591

THIS BASIN USES TRADITIONAL DRAINAGE PRACTICES

FRACTION OF PERVIOUS AREA RECEIVING IMPERVIOUS DRAINAGE (DEFAULT)	FRACTION OF IMPERVIOUS AREA DIRECTLY CONNECTED TO DRAINAGE SYSTEM (DEFAULT)
R= .18	D= .76

CALCULATED UNIT HYDROGRAPH

TIME TO PEAK (MIN)	PEAK RATE OF RUNOFF (CFS/SQMI)	UNIT HYDROGRAPH PEAK (CFS)	VOLUME OF RUNOFF (AF)
54.82	433.75	1448.73	178.13

WIDTH AT 50 = 69. MIN. WIDTH AT 75 = 36. MIN. K50 = .35 K75 = .45

RAINFALL LOSSES INPUT W/ BASIN DATA

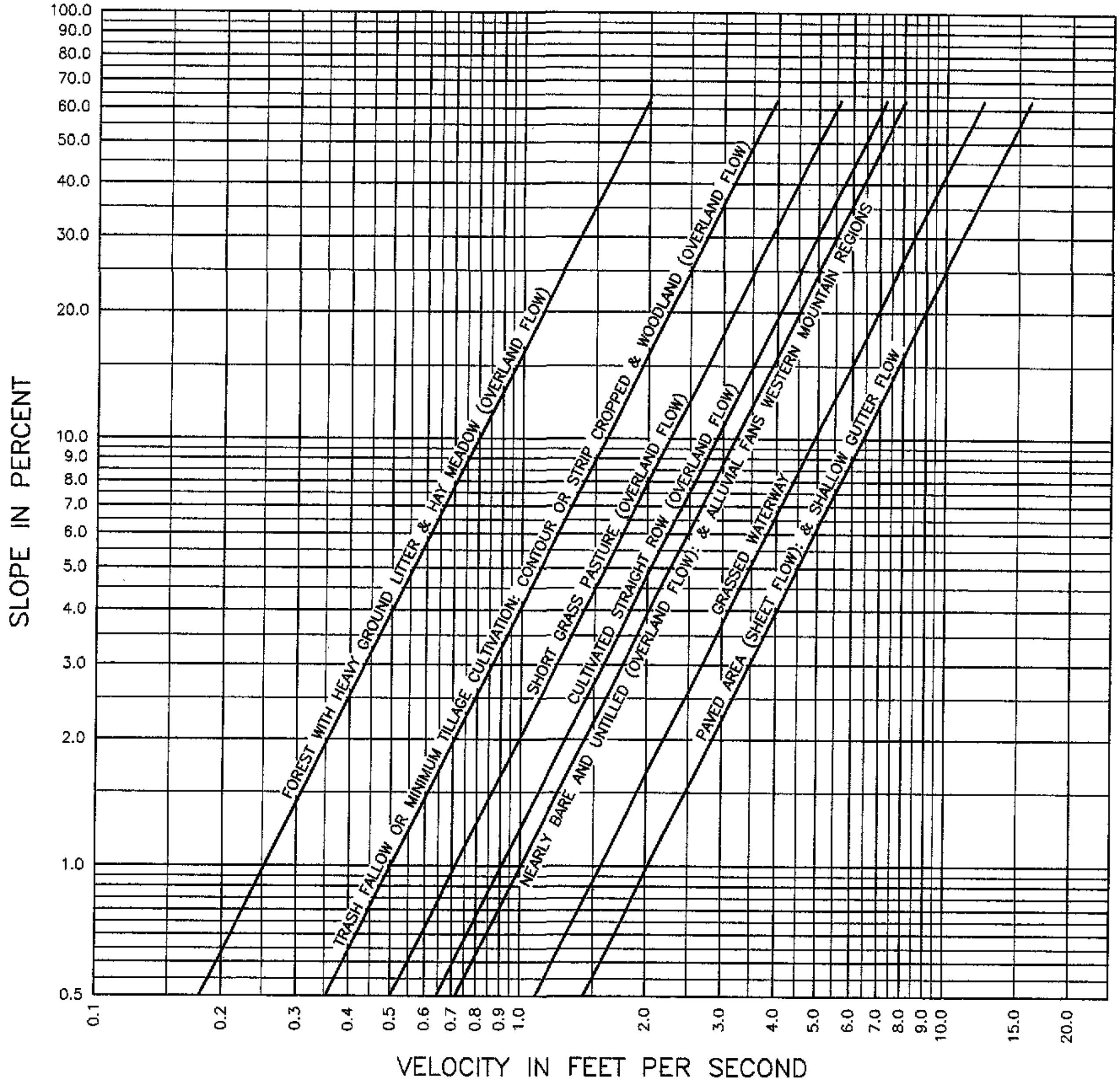
MAX. PERVIOUS RET. = .35 IN. MAX. IMPERVIOUS RET. = .05 IN.
INFILTRATION = 3.73 IN./HR. DECAY = .00180/SECOND FNINFL = .55 IN./HR.

1 BASIN ID: Smith1 -- BASIN COMMENT: Smith Creek at mead

**** STORM NO. = 1 **** DATE OR RETURN PERIOD = 100-year,

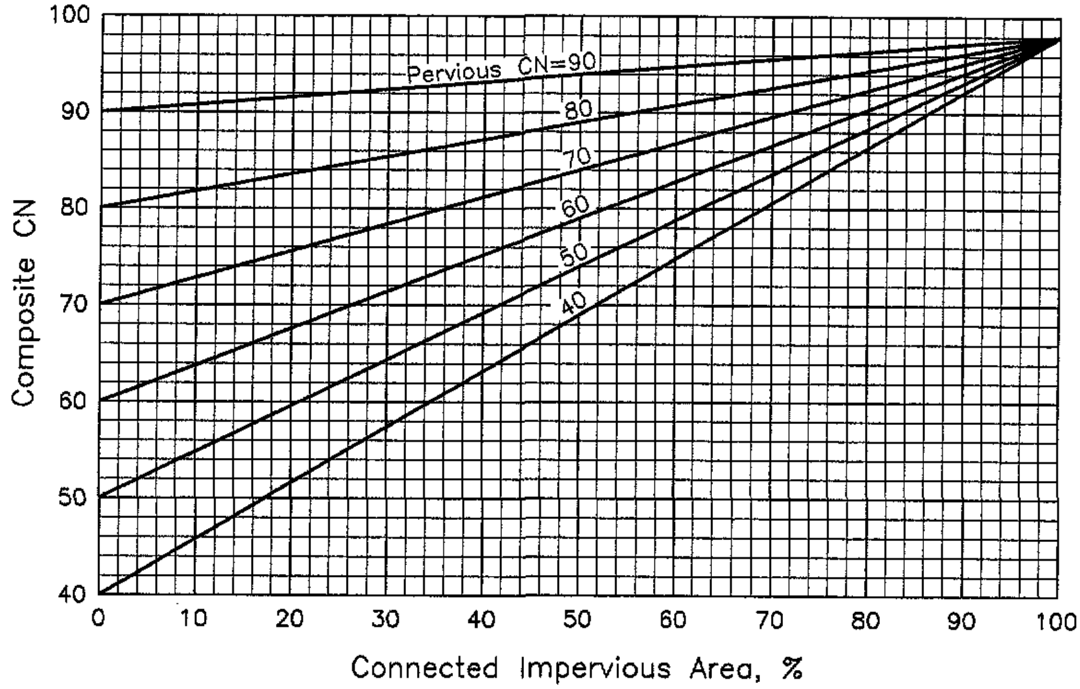
TOTAL PRECIP. = 3.54 (1-HOUR RAIN = 3.06) EXCESS PRECIP. = 2.466 INCHES
VOLUME OF EXCESS PRECIP = 439.27 ACRE-FEET
PEAK Q = 2984. CFS TIME OF PEAK = 85. MIN.
INFILT.= 3.73 IN/HR DECAY = .00180 FNINF = .55 IN/HR
MAX.PERV.RET.= .35 IN. MAX.IMP.RET.= .05 IN.

COLORADO STATEWIDE
DRAINAGE AND FLOODPLAIN MANAGEMENT CRITERIA MANUAL



O:/2080/FIGURES, CH5-F201.DWG, 4/8/02 MAS

VERSION: AUGUST 2002	REFERENCE: Soil Conservation Service, 1985 (Modified)	**FIGURE CH5-F201** TRAVEL TIME VELOCITY
WRC ENGINEERING, INC.		



NOTE:

1. REFER TO TABLE 701 FOR CN VALUES FOR VARIOUS LAND USE CONDITIONS.

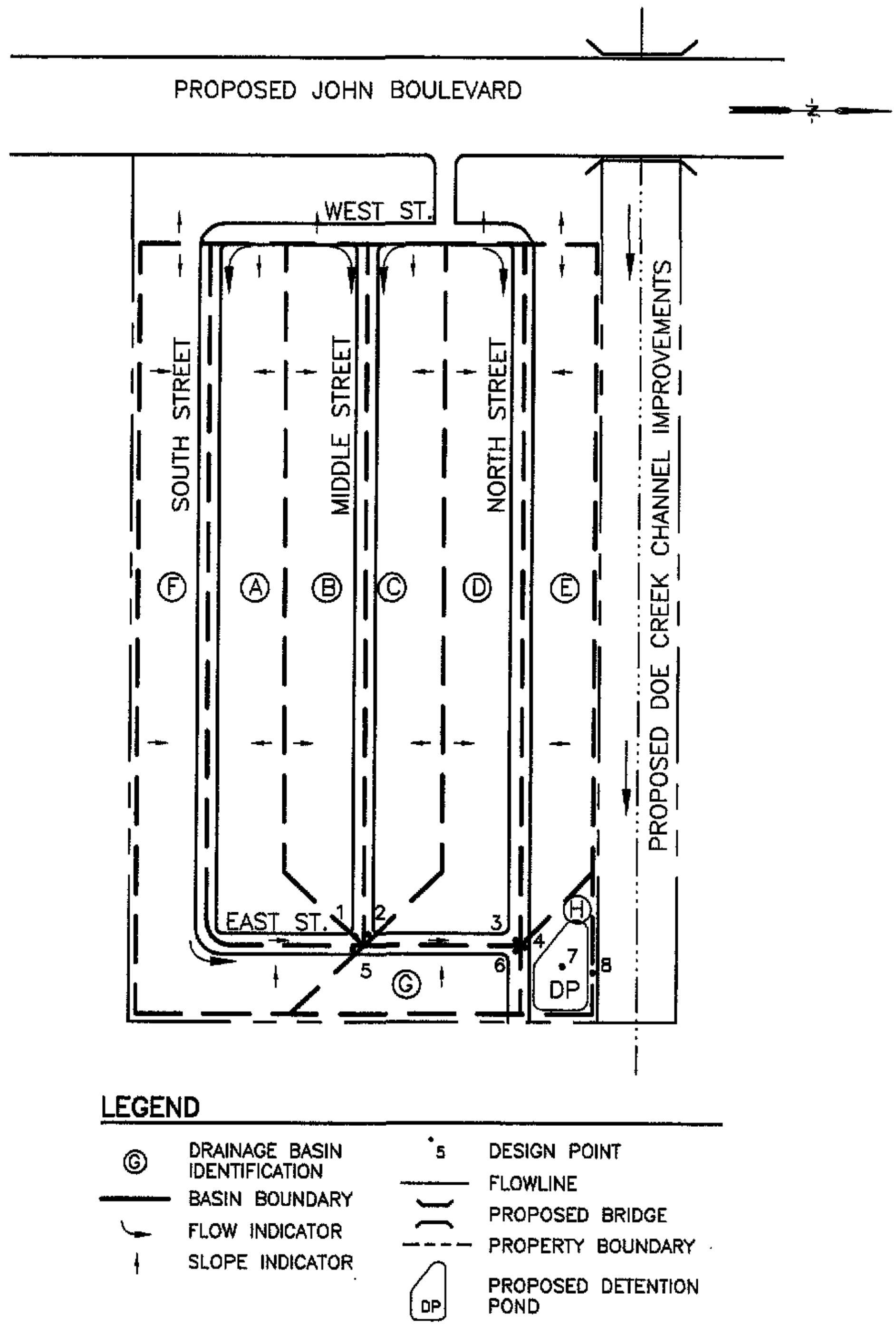


FIGURE CH5-F203
EXAMPLE: PROPOSED ROSE SUBDIVISION LAYOUT

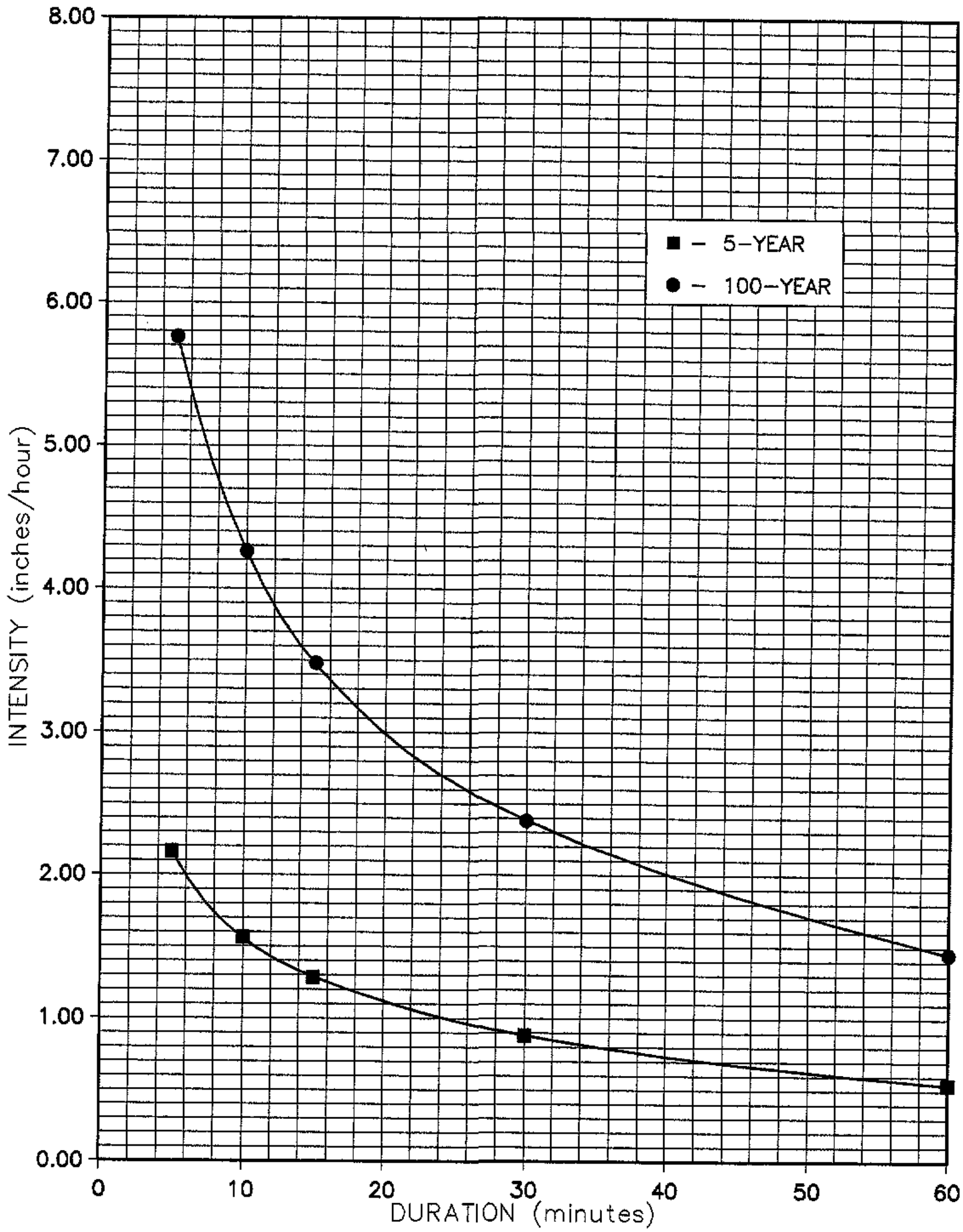


FIGURE CH5-F204

EXAMPLE: ROSE SUBDIVISION IDF CURVES

O:/2080/FIGURES, CH5-SF201.DWG, 4/5/02 MAS

DEVELOPMENT ______________________

CALCULATED BY______________ DATE____

SUB-BASIN DATA			INITIAL/OVERLAND TIME (t_i)			TRAVEL TIME (t_t)				t_c ($t_i + t_t$)	t_c URBANIZED BASINS CHECK		FINAL t_c	REMARKS
DESIG: (1)	R (2)	AREA Ac (3)	LENGTH Ft (4)	SLOPE % (5)	t_i Min (6)	LENGTH Ft (7)	SLOPE % (8)	VEL. FPS (9)	t_t Min (10)	t_c Min (11)	TOTAL LENGTH Ft (12)	t_c=(L/180)+10 Min (13)	Min (14)	

$$t_i = 1.8\ (1.1 - R)\ L^{1/2} / S^{1/3}$$

WRC ENGINEERING, INC

VERSION: AUGUST 2002

REFERENCE:

FIGURE CH5-SF201

TIME OF CONCENTRATION

COLORADO STATEWIDE

DRAINAGE AND FLOODPLAIN MANAGEMENT CRITERIA MANUAL

Chapter 6

CHAPTER 6

HYDRAULIC ANALYSIS AND DESIGN

CHAPTER 6

HYDRAULIC ANALYSIS AND DESIGN

SECTION 1.0

OPEN CHANNELS

CHAPTER 6
HYDRAULIC ANALYSIS AND DESIGN

SECTION 1
OPEN CHANNELS

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

CHAPTER 6
HYDRAULIC ANAYLSIS AND DESIGN

SECTION 1.0
OPEN CHANNELS

1.1 INTRODUCTION

Presented in this section are the technical criteria and design standards for hydraulic evaluation and design of natural and artificial open channels. Discussions and hydraulic standards are provided for various channel types anticipated to be encountered or used in the State of Colorado.

The information presented in this section should be considered to be the minimum hydraulic standards upon which channel evaluation and design should be based. Additional analyses may be necessary for unique or unusual channel and site conditions. The users of this manual are encouraged to review the related textbooks and other technical literatures on the subject for more in-depth discussions. The following is a short list of some of the related publications.

- Chow, V. T., Open Channel Hydraulics, McGraw-Hill, 1970
- Brater and King, Handbook of Hydraulics, McGraw-Hill Book Co., 6th Ed., 1976.
- Dave Rosgen, illustrated by Hilton Lee Silvey, Applied River Morphology, 1996
- US Army Corps of Engineers, HEC-2 User Manual, Version 3.0, January 2001
- US Army Corps of Engineers, HEC-RAS User Manual, Version 4.6, February 1991
- US Army Corps of Engineers, Hydraulic Design of Flood Control Channels, EM 1110-2-1601, July 1991
- US Army Corps of Engineers, River Hydraulics, EM 1110-2-1416, October 1993

1.2 CHANNEL TYPES

Open channels can be categorized as either natural or artificial. Natural channels include all watercourses that are carved and shaped naturally by the erosion and sediment transport process. Artificial channels are those constructed or developed by human efforts. Essentially, open channels in Colorado can be separated into the following six (6) different types:

1.2.1 NATURAL CHANNELS

A natural channel is a watercourse formed naturally by the erosion and sediment transport process. In general, a natural channel continually changes its position and shape as a result of hydraulic forces acting on its bed and banks. If feasible, natural channels should be kept undisturbed and new developments should be placed sufficiently away from the channel banks.

1.2.2 GRASS-LINED CHANNELS

Grass-lined channels may be considered to be the most desirable artificial channels from an aesthetic viewpoint. The channel storage, lower velocities, and the sociological benefits create significant advantages over other types of channels. The grass cover can stabilize the channel side slopes, check erosion of the channel surface, and control the movement of soil particles along the channel bottom. Low flow areas may need to be concrete or rock lined to minimize erosion and maintenance problems.

1.2.3 CONCRETE-LINED CHANNELS

Concrete-lined channels are defined as rectangular or trapezoidal channels in which reinforced concrete is used for lining of the channel banks and bottom. Concrete-lined channels will be permitted only where ROW restrictions due to existing developments prohibit use of other channel types and will be approved on a case-by-case basis only. Special attentions should be taken to provide safety measures (i.e. fence) around the concrete channels.

1.2.4 RIPRAP-LINED CHANNELS

Riprap-lined channels are defined as channels in which riprap is used for lining of the channel banks and the channel bottom. Riprap is a popular choice for erosion protection because the initial installation costs are often less than alternative methods for preventing erosion. However, the designer needs to bear in mind that there are additional costs associated with riprap erosion protection since riprap installations require periodic inspection and maintenance.

Riprap-lined channels will be permitted in areas of existing development where ROW is limited and such limitation prohibits the use of bio-engineered channels. Situations for which riprap lining might be appropriate are: 1) where major flows, such as the 100-year flood are found to produce channel velocities in excess of allowable non-eroding values; 2) where channel side slopes must be steeper than 3:1; 3) for low flow channels, and 4) where rapid changes in channel geometry occur such as channel bends and transitions.

1.2.5 OTHER CHANNEL LINERS

A variety of artificial channel liners are on the market, all intended to protect the channel from erosion at higher velocities. These include gabion, interlocked concrete blocks, concrete revetment mats formed by injecting concrete into double layer fabric forms, and various types of synthetic fiber liners. As with rock and concrete liners, all of these types are best considered for helping to solve existing urban flooding problems and are not recommended for new developments. Each type of liner has to be scrutinized for its merits, applicability, how it meets other community needs, its long term integrity, and maintenance needs and costs.

1.2.6 WETLAND VEGETATION BOTTOM CHANNELS

This type of channel is a subset of "grass-lined" channels, designed to encourage the development of wetlands or certain types of riparian vegetation in the channel bottom. The potential benefits associated with a wetland bottom channel include habitat for aquatic, terrestrial, and avian wildlife and possible water quality enhancement as the base flows move through the marshy vegetation.

1.3 NATURAL CHANNEL SYSTEMS

In general, a natural channel system continually changes its position and shape as a result of hydraulic forces acting on its bed and banks. These changes may be slow or rapid and may result from natural environmental changes or from changes caused by human activities. When a natural channel is modified locally, the change frequently causes alteration in channel characteristics both upstream and downstream. The response of a natural channel to human-induced changes often occurs in spite of attempts to control the natural channel environment.

When a natural channel is modified locally, the change frequently causes alteration in channel characteristics both upstream and downstream.

Natural and human-induced changes in natural channels frequently set in motion responses that can be propagated for long distances.

Natural and human-induced changes in natural channels frequently set in motion responses that can be propagated for long distances. In spite of the complexity of these responses, all natural channels are governed by the same basic forces but to varying degrees. It is necessary that a natural channel system design be based on adequate knowledge of: (1) geologic factors, including soil conditions; (2) hydrologic factors, including possible changes in flow and runoff, and the hydrologic effects of changes in land use; (3) geometric characteristics of the stream, including the probable geometric alterations that developments will impose on the channel; (4) hydraulic characteristics such as depth, slope, velocity of streams, sediment transport, and the changes that may be expected in these characteristics over space and time; and (5) ecological/biological changes that will result from physical changes that may in turn induce or modify physical changes.

Effects of development in natural channels, flood control measures, and constructed channel structures have proven the need for considering the immediate, delayed, and far-reaching effects of alterations imposed on natural channel systems. Variables affecting natural channels are numerous and interrelated. Their nature is such that, unlike rigid-boundary hydraulic problems, it is not possible to isolate and study the role of each individual variable. Because of the complexity of the processes occurring in natural flows that influence the erosion and deposition of material, a detached analytical approach to the problem may be difficult and time consuming. Most relationships describing natural channel processes have been derived empirically. The major factors affecting natural channel geometry are: (1) stream discharge; (2) sediment load; (3) longitudinal slope; (4) characteristics of bed and bank material; (5) bank and bed resistance to flow; (6) vegetation or lack there of; (7) geology, including type of sediment; and (8) constructed improvements.

1.3.1 CHANNEL MORPHOLOGY

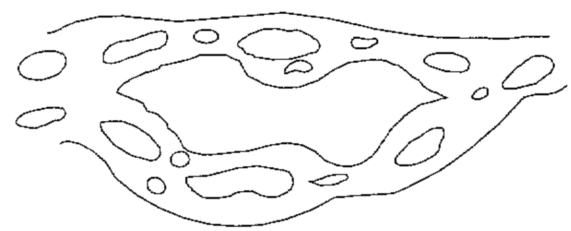
When seeking to utilize or modify a natural channel, an understanding of the mechanism of its morphology is important. Without incorporating thorough understanding of the geomorphic conditions of the stream and the watershed, alterations to channels or to their watersheds can lead to unexpected instabilities, bring about unwanted erosion or aggradation, and cause significant damage to fluvial systems.

The morphology of a stream is a result of the variables that determine the quantity of water and sediment it carries, including the geology, soils and vegetation of the stream and watershed, the hydrology and dominant discharge of the system, and the slope of the stream. The following is a short discussion of some fundamentals of fluvial geomorphology. The users of this manual are encouraged to review the related textbooks and other technical literatures on the subject for more detailed discussions. The following is a short list of some of the related publications.

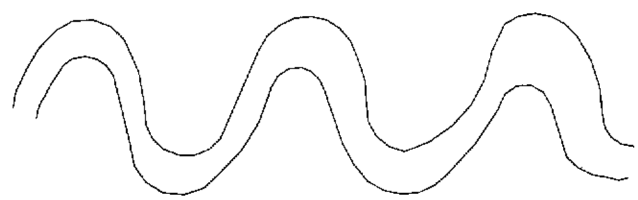
- Dave Rosgen, illustrated by Hilton Lee Silvey, Applied River Morphology, 1996
- Lane, E. W., 14957. A study of the shape of channels formed by natural streams flossing in erodible material: M.R. D. Sediment Series No. 9, US. Army Engineer Division, Missouri River, Corps of Engineers, Omaha, NE.
- Ritter, Dale F, 1986. Process Geomorphology. Wm C. Brown Publishers, Dubuque, Iowa.
- Simons Li and Associates, 1982. Engineering Analysis of Fluvial Systems.

There are three general principles governing the geomorphology of a natural stream system. First, riverine systems are dynamic. Erosion and aggradation can occur over a relatively short period of time (as sudden as one storm event) and can result from unstable conditions brought about by changing hydrologic or sediment-supply conditions (either natural or anthropomorphic). However, because all systems are dynamic, normal progression of a stream is not always a result or a symptom of instability. Second, the responses resulting from changes to a channel or its watershed are complex. Morphologic responses can be anticipated but cannot always be quantitatively predicted, even by the most trained engineers. Additionally, short reaches of streams cannot be looked at individually; a change to a short stretch or even to a single area of the stream can cause unwanted or unexpected alterations upstream or downstream of the change. Third, most geomorphic boundaries within a riverine system can be classified as thresholds. Gradual changes to a channel or its watershed will not always bring about gradual responses. Instead, gradual changes may build-up to a threshold so that a small-scale occurrence, such as a moderately large flood, will seemingly cause a catastrophic result. (SLA, 1982)

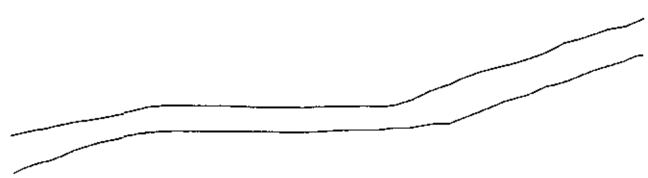
Natural streams can be classified generally into three prevailing patterns. These patterns, straight, meandering and braided, are characteristics of the responses of a system to its prevailing discharge and load.



Braided Channel



Meandering Channel



Straight Channel

Straight and meandering streams are two manifestations of similar dynamics. The thalwegs in both shift from bank to bank and sediment deposition and erosion within the channel bottom establish a series of riffles and pools. Straight channels have relatively straight banks; meandering streams have sinuous banks. Straight channels are fairly rare; most natural channels have some degree of sinuosity. Although meandering and straight streams can be in quasi-equilibrium, their thalwegs, meanders and riffle-pool sequences migrate in predictable patterns if left untouched. Braided systems, unlike meandering and straight, do not have a single trunk; they have a network of branches and series of islands. The single branches usually meander to some degree. Braided channels convey low to medium flows in the series of branches; large flows intermingle into a single floodplain. Meandering and straight systems are generally more stable than braided. Braided channels tend to carve new channels and deposit islands at a relatively fast pace and be horizontally unstable. The divisions between the three classifications are imprecise and relatively indistinct. A given stream can have reaches of each classification, and given reaches can include characteristics of one or more pattern. (Ritter, 1986)

Any change to a variable of a natural stream system, such as the slope or dominant discharge, can change the morphology and/or the existing stream pattern according to the three principles outlined above. These changes can be somewhat predicted; much work has been done to establish relationships between the variables and characteristics of natural streams. Two general relationships for predicting morphological responses to changes in riverine variables are as follows:

$$Q \cong \frac{b, d, \lambda}{S} \qquad \text{(Eq. CH6-100)}$$

and

$$Qs \cong \frac{b, \lambda, S}{d, P} \quad \text{(SLA, 1982)} \qquad \text{(Eq. CH6-101)}$$

Where Q = Average discharge
Qs = Sediment supply
B = Channel width
d = Channel depth
λ = Meander wavelength
S = Bed slope
P = Sinuosity

An increase in mean annual discharge will generally cause an increase in channel depth, width, and meander wavelength and a decrease in bed slope. An increase in sediment supply will generally cause an increase in channel width, meander wavelength and bed slope and a decrease in sinuosity and channel width. Because the average flow rate is usually directly related to sediment supply, these relationships can become complex when both flow and sediment supply increase or flow increases and sediment supply decreases, or vice-versa. Additionally, changes to one or more channel morphology characteristics can cause changes to other characteristics. An increase in slope can cause a decrease in channel depth or a decrease in meander wavelength. Further complicating these relationships are variables such as the average grain-size and type of sediment, the percentage of sediment carried as bed load, and the geology of the valley, all which can affect the responses of the stream and contribute to unexpected or seemingly counter-intuitive results.

A general relationship between slope, mean annual discharge and the tendency of a system to be meandering or braided has been established by Lane (1957). They found that if a stream's $SQ^{1/4} \leq 0.0017$, it tends to be meandering. If $SQ^{1/4} \geq 0.01$, systems tend toward a braided pattern. Streams that have $SQ^{1/4}$ between 0.0017 and 0.01 are in an intermediate zone and can be either braided or meandering with a greater tendency to respond to flow and slope alterations with a change in river pattern. These relationships are complicated and not absolute.

Some specific examples of man-induced changes to the natural stream/river systems that could cause undesired responses by channel morphology are as follows:

1) **Change in Flow:** As demonstrated in the above relationships, a decrease in flow due to diversion or reservoir routing change can cause a decrease in channel width, depth, and sinuosity and an increase in slope; an increase in flow due to development can have the opposite effect. In addition to these changes, the corresponding decrease or increase of average stage of the main stem of a river can have significant effects on the streams' tributaries. If the average stage decreases, the tributaries' energy slopes will increase, increasing the ability of the tributary to transport sediment, which can cause degradation of its channel, commonly referred to as headcutting. Similarly, an increase in stage in the main stem can lead to aggradation within its tributaries. Both of these scenarios can do serious damage to the tributary channel and increase its horizontal instability. Headcutting can cause bank destabilization and failure. Aggradation can cause increased flooding potential and rerouting of the channel.

2) **Channelization:** The channelization of a natural stream to allow increased conveyance often straightens channels and cuts off meanders causing an increase in slope through the improved stretch. This can increase velocities and degradation through the stretch and then decrease slopes and increase aggradation downstream of the stretch. The increase in slope and average discharge can also cause a meandering system to tend toward a braided configuration that can lead to further horizontal and vertical instabilities. In addition, by lowering the average stage, channelization will affect the stream's tributary channels in the same manner as the first example.

3) **Construction of Dams**: The construction of both large and small-scale dams can have far-reaching effects on a stream system. Without a design-approach that will allow frequent flows to travel through the dam unadulterated, some suspended sediment and most bedload will be deposited upstream of the dam. This will decrease slopes and change channel configuration upstream and release clear water and potentially cause scour and degradation in the downstream reach. This can upset any equilibrium that was established within the system prior the construction and may even potentially cause failure of the dam itself.

4) **Construction of Bridges:** The construction of bridges and culverts, in addition to the well-documented local scour issues, can cause more regional channel morphology problems. An undersized bridge or culvert can decrease velocity and increase average stage upstream of the bridge, causing deposition and affecting the tributaries' channels. Scour around the bridges can cause an increase in sediment supply in the channel, leading to deposition downstream.

There are many additional examples of morphological problems that can be caused by man-made changes on a natural stream system. Any substantial modification to a natural channel system should be evaluated carefully to determine the potential adverse impacts on the stream system both upstream and downstream of the proposed modification.

1.3.2 CHANNEL RESTORATION

The practice of channel restoration is becoming more common in Colorado and the rest of the United States as the negative effects of urbanization, channelization, and other hydraulic "improvements" have taken their toll on the sediment balance, channel stability, biological habitats, and the aesthetic and recreational benefits of the impacted rivers and streams.

Channel/river restoration projects typically involve re-connection of the floodplain back to its channel, establishment of wetland areas around the channel, restoration of meanders, point-bars and riffle-pool sequences, and re-creation of the chemical and biological complexity that exists in the natural channel system.

Although, it may not be feasible to restore the disturbed stream/river systems back to its original conditions, channel restoration projects can help expedite the natural channel recovering process and help to re-create an environment that closely resembles the original configuration of the stream system. Channel/river restoration projects typically involve re-connection of the floodplain back to its channel, establishment of wetland areas around the channel, restoration of meanders, point-bars and riffle-pool sequences, and re-creation of the chemical and biological complexity that exists in the natural channel system. Benching, allowing for a low-flow meandering channel with terraced banks above the low-flow channel, is a popular technique that allows for expansive riparian plant and wildlife habitat, recreation opportunities, and unique flood control options. Channel restoration usually involves a significant degree of both planting and seeding native, wetland, and self-sustainable vegetations within the channel and along the banks.

A design team comprised of hydraulic engineers, fluvial geomorphologists, biologists and botanists who are highly knowledgeable of the system should be involved in the channel restoration design process. Furthermore, due to the advantage of irregular alignments and channel cross sections, the construction phase must be carefully managed and overseen to ensure that the design is fully incorporated into the final improvement.

1.4 OPEN-CHANNEL HYDRAULICS

An open channel is a conduit in which water flows with a free surface (non-pressurized flow). The hydraulics of an open channel can be very complex, encompassing many different flow conditions from steady-state uniform flow to unsteady, rapidly varying flow. Most of the problems in storm water drainage involve uniform, gradually varying or rapidly varying flow states. Examples of these flow conditions are illustrated in Figure CH6-F101. The calculations for uniform and gradually varying flow are relatively straight forward and are based upon similar assumptions (i.e., parallel streamlines). Rapidly varying flow computations (i.e., hydraulic jumps and flow over spillways), however, can be very complex, and the solutions are generally empirical in nature.

Presented in this section are the basic equations and computational procedures for uniform, gradually varying and rapidly varying flow. The user is encouraged to review the many hydraulics textbooks available for more detailed discussions.

1.4.1 UNIFORM FLOW

Open-channel flow is said to be uniform if the depth of flow is the same at every section of the channel. For a given channel geometry, roughness, discharge and slope, the only possible depth for maintaining uniform flow is the normal depth. For uniform flow in a prismatic channel (i.e., uniform cross section), the water surface will be parallel to the channel bottom.

Uniform flow rarely occurs in nature and is difficult to achieve in a laboratory, because not all of the parameters remain exactly the same. However, channels are designed assuming uniform flow as an approximation, which is adequate for planning and design purposes.

The computation of uniform flow and normal depth shall be based upon the Manning formula as follows:

$$Q = \left(\frac{1.49}{n}\right) A R^{2/3} S^{1/2} \qquad \text{(Eq. CH6-102)}$$

Where Q = Flow rate (cubic feet per second (cfs))
n = Roughness coefficient
A = Area (square feet (sf))
P = Wetted perimeter (feet)
R = A/P = Hydraulic radius (feet)
S = Slope of the energy grade line (feet/feet)

For prismatic channels, the energy gradeline (EGL) slope, hydraulic gradeline (HGL) slope, and the bottom slope are assumed to be the same for uniform, normal depth flow conditions.

Presented in Table CH6-T101 are equations for calculating many of the parameters required for hydraulic analysis of different channel sections.

Table CH6-T102 provides a list of Manning roughness coefficient values for many types of conditions that may occur in the State of Colorado. These parameters and the Manning equation may also be readily computed using hand-held calculators and personal computers.

1.4.2 UNIFORM CRITICAL FLOW ANALYSIS

The critical state of uniform flow through a channel is characterized by several important conditions.

1. The specific energy is a minimum for a given discharge.
2. The discharge is a maximum for a given specific energy.
3. The specific force is a minimum for a given discharge.
4. The velocity head is equal to half the hydraulic depth in a channel of small slope.
5. The Froude Number is equal to 1.0.

If the critical state of uniform flow exists throughout an entire reach, the channel flow is critical and the channel slope is at critical slope, S_c. A slope less than S_c will cause sub-critical flow. A slope greater than S_c will cause super-critical flow. A flow at or near the critical state is unstable. Factors creating minor changes in specific energy, such as channel debris, will cause a major change in depth.

A flow at or near the critical state is unstable. Factors creating minor changes in specific energy, such as channel debris, will cause a major change in depth.

The criteria of minimum specific energy for critical flow results in the definition of the Froude Number (F_r) as follows:

$$F_r = \frac{V}{(gD)^{0.5}} \qquad \text{(Eq. CH6-103)}$$

Where F_r = Froude Number
V = Velocity (feet per second (fps))
g = Acceleration of gravity (feet per second squared)
A = Channel flow area (square feet)
T = Top width of flow area (feet)
D = A/T = Hydraulic depth (feet)

The Froude Number for a given channel section and flow can be easily computed using the above equation. The critical depth in a given trapezoidal channel section with a known flow rate can be determined using the following methodology. First, the section factor, Z, is computed.

$$Z = \frac{Q}{g^{0.5}} \qquad \text{(Eq. CH6-104)}$$

Where Z = Section factor
Q = Flow rate (cfs)
g = Acceleration of gravity (feet per second squared)

Utilizing values for Z, the channel bottom width, b, and the side slope, z, the critical depth in the channel, y, can be determined from Figure CH6-F102. For other prismatic channel shapes, Equation CH6-104 above can be used with the section factors provided in Table CH6-T101 to determine the critical depth.

Since flows at or near critical depth are unstable, all channels shall be designed with Froude Numbers and flow depths as follows:

Flow Condition	Froude Number (F_r)	Flow Depth
Sub-Critical	<0.8	$>1.1d_c$
Super-Critical	>1.13	$<0.9d_c$

Where d_c = critical depth

All channel design submittals shall include the calculated Froude Number and critical depth for each unique reach of channel to identify the flow state and verify compliance with the MANUAL.

1.4.3 GRADUALLY VARYING FLOW

The most common occurrence of gradually varying flow in storm drainage is the backwater created by culverts, storm sewer inlets, or channel constrictions. For these conditions, the flow depth will be greater than normal depth in the channel, and the water surface profile must be computed using backwater techniques.

Backwater computations can be made using the methods presented in Chow, 1959. Many computer programs are available for computation of backwater curves. The most general and widely used programs are US Army Corps of Engineers' HEC-2 and HEC-RAS. These programs are recommended for floodwater profile computations for channel and floodplain analyses.

For prismatic channels, the backwater calculation can be computed manually using the Direct Step Method as described in Chow, 1959. The Direct Step Method is also available in many hand-held and personal computer software programs. For an irregular non-uniform channel, the Standard Step Method is used which is a more tedious and iterative process. For these channels, the use of HEC-2 or HEC-RAS is recommended.

1.4.4 RAPIDLY VARYING FLOW

Rapidly varying flow is characterized by very pronounced curvature of the flow streamlines. The change in curvature may become so abrupt that the flow profile is virtually broken, resulting in a state of high turbulence. There are mathematical solutions to some specific cases of rapidly varying flow, but empirical solutions are generally relied on for most rapidly varying flow problems.

The most common occurrence of rapidly varying flow in storm drainage applications involves weirs, orifices, hydraulic jumps, non-prismatic channel sections (transitions, culverts and bridges), and non-linear channel alignments (bends). Each of these flow conditions require extensive and detailed calculations to properly identify the flow capacities and depths of flow in the given section. The designer should be cognizant of the design requirements for each of the above conditions and must include all necessary calculations as part of the design submittal documents. The designer is referred to the many hydraulic references for the proper calculation methods to use in the design of rapidly varying flow facilities.

1.4.5 TRANSITIONS

Channel transitions occur in open channel design whenever there is a change in channel slope or shape and at junctions with other open channels or storm sewers. The goal of a good transition design is to minimize the loss of energy as well as minimize surface disturbances from cross-waves and turbulence. Special cases of transitions where excess energy is dissipated by design are drop structures and hydraulic jumps. Channel drop structures are discussed in Chapter 6, Section 6.

Transitions in open channels are generally designed for the following four flow conditions:

1. Sub-critical flow to sub-critical flow.
2. Sub-critical flow to super-critical flow.
3. Super-critical flow to sub-critical flow (Hydraulic Jump).
4. Super-critical flow to super-critical flow.

For definition purposes, conditions 1 and 2 will be considered as sub-critical transitions and are later discussed in Section 1.8.1. Conditions 3 and 4 will be considered as super-critical transitions and are later discussed in Section 1.8.2.

1.5 OPEN CHANNEL DESIGN

Adequate drainage facilities in developed areas are essential to preserve and promote the general health, welfare, and economic well being of the region. All new open channels shall be designed, as a minimum, to safely confine and convey the estimated 100-year flood flows.

The design standards for major and minor drainageways are included in this section. A major drainageway is defined as a channel/drainageway with a contributing tributary area of 160 acres or more. The design standards presented in this chapter are minimum standards, and the channel designer is reminded that the ultimate responsibility for a safe and stable channel design lies solely with the engineer responsible for the design. Thus, the execution of this responsibility may require additional analysis and stricter standards than are presented in this chapter. In addition, unique or unusual site conditions may require additional design analysis be performed to verify the suitability of the proposed channel design for the project site.

All new open channels shall be designed, as a minimum, to safely confine and convey the estimated 100-year flood flows. A major drainageway is defined as a channel/ drainageway with a contributing tributary area of 160 acres or more.

1.5.1 CHANNEL TYPE SELECTION

As discussed previously in Section 1.2, open channels can be generally separated into the following six (6) different channel types.

- Natural Channels
- Grass-lined Channels
- Concrete-lined Channels
- Riprap-lined Channels
- Wetland Vegetation Bottom Channels
- Other Channel Liners

The selection of a channel type appropriate for the conditions that exist at the project site should be based on the following multi-disciplinary factors including hydraulic, structural, environmental, sociological, maintenance, and regulatory factors. In general, the use of concrete-lined and riprap-lined channels is discouraged.

Hydraulic Factors

1. Slope of thalweg
2. Right-of-way
3. Capacity needed
4. Basin sediment yield
5. Topography
6. Ability to drain adjacent lands

Structural Factors

1. Cost
2. Availability of material
3. Areas for wasting excess excavated material
4. Seepage and uplift forces
5. Shear stresses
6. Pressures and pressure fluctuations
7. Momentum transfer

Environmental Factors

1. Neighborhood character
2. Neighborhood aesthetic requirements
3. Need for new green areas
4. Street and traffic patterns
5. Municipal or county policies
6. Wetland mitigation
7. Wildlife habitat
8. Water quality enhancement

Sociological Factors

1. Neighborhood social patterns
2. Neighborhood children population
3. Pedestrian traffic
4. Recreational needs

Maintenance Factors

1. Life expectancy
2. Repair and reconstruction needs
3. Maintainability
4. Proven performance
5. Accessibility

Regulatory Factors

1. Federal Regulations
2. State Regulations
3. Local Regulations

1.5.2 MAXIMUM PERMISSIBLE VELOCITIES

The design of open channels shall be based on maximum permissible velocities. This method of design assumes that a given channel section will remain stable up to the stated maximum permissible velocity provided that the channel is designed in accordance with the provisions of this MANUAL. Presented in Table CH6-T103 are the maximum permissible velocities for natural, improved, unlined, and lined channels. These values shall be used for all channel designs in the State of Colorado. If a higher velocity is desired, the design engineer must demonstrate that the higher velocity would not endanger the health or safety of the public and would not increase maintenance of the channel section. For natural and improved unlined channels, a geotechnical report shall be submitted identifying the existing

The design of open channels shall be based on maximum permissible velocities. Presented in Table CH6-T103 are the maximum permissible velocities for natural, improved, unlined, and lined channels.

and/or proposed soil material classification used for the maximum permissible velocity determination. Additional analysis may be required for natural channels or improved unlined channels to verify that the channel will remain stable based on the stated maximum permissible velocities.

The stated maximum permissible velocities are based on flow studies conducted by various governmental agencies and private individuals using non-clear water conditions. The application of these velocities to actual site conditions are subject to proper design and competent construction of the channel sections. The design engineer shall be responsible for designing the channel section so it will remain stable at the final design flow rate and velocity. For channels constructed in part or in whole from fill materials, the design engineer shall be responsible for designing the channel based upon the characteristics of the fill material.

1.6 NATURAL CHANNEL DESIGN

Presented in this section are the typical natural open channel sections that are encountered in Colorado. A graphical illustration of the typical design sections is presented in Figure CH6-F103. The selection of a design section for a natural channel is generally dependent on the value of developable land versus the cost to remove the land from a floodplain. The costs for the removal depend on the rate of flow, slope, alignment, and depth of the channel as well as material and fill costs for construction of the encroachment. The design sections discussed herein vary from no encroachment to the level of encroachment at which point an improved channel (unlined or lined) becomes more economical or is required to adequately protect the proposed development. The design standards of natural channels are the same for both major and minor drainage-ways.

For natural channel sections, the engineer shall identify through stable channel (normal depth) calculations the stability or instability of the channel to contain the major (100-year) storm flows. If this analysis demonstrates that either bank erosion outside of the designated flow path (easement and/or right-of-way) or channel degradation is likely to occur, then an analysis of the magnitude and extent of the erosion may be necessary. In such a condition, the design engineer shall meet with the local official to determine: a) what additional analysis shall be prepared to estimate the potential extent of lateral and vertical channel movement, b) what is the potential risk to the proposed development from channel degradation and/or bank failure, c) what solutions and/or remedies are available which can mitigate the potential risk to the proposed development, and d) what improvements and/or reduction in encroachment in or adjacent to the subject channel will be required to allow approval of the subject development.

Some general design considerations and evaluation techniques for natural channels are as follows:

1. The channel and overbank areas shall have adequate conveyance capacity for the major (100-year) storm runoff.

2. Natural channel segments with a calculated flow velocity greater than the allowable flow velocity shall be analyzed for erosion potential with a suitable methodology using standard engineering practice. Additional erosion protection may be required.

3. The water surface profiles shall be defined so that the 100-year floodplain can be delineated.

4. Filling of the floodplain fringe may reduce valuable storage capacity and may increase downstream runoff peaks.

5. Erosion control structures, such as drop structures or check dams, may be required to control flow velocities for both the minor storm and major storm events.

6. Plan and profile information (i.e., HEC-2 output) for both existing and proposed floodplain site conditions shall be prepared.

7. The engineer shall verify, through stable channel (normal depth) calculations, the suitability of the floodplain to contain the flows. If this analysis demonstrates erosion outside of the designated flow path (easement and/or ROW), an analysis of the equilibrium slope and degradation or aggregation depths is required and suitable improvements identified.

With many natural channels, erosion control structures may need to be constructed at regular intervals to decrease the thalweg slope and to minimize erosion. However, these channels should be left in as near a natural state as possible. For that reason, extensive modifications should not be pursued unless they are found to be necessary to avoid excessive erosion with substantial deposition downstream.

The usual rules of freeboard depth, curvature, and other rules, which are applicable to artificial channels, do not apply for natural channels. There are significant advantages that occur if the designer incorporates into his planning the overtopping of the channel and localized flooding of adjacent areas, which remain undeveloped for the purpose of being inundated during the major runoff peak.

There are significant advantages that occur if the designer incorporates into his planning the overtopping of the channel and localized flooding of adjacent areas, which remain undeveloped for the purpose of being inundated during the major runoff peak.

If a natural channel is to be modified or encroached upon for a development, then the applicant shall meet with the agencies with jurisdiction over the channel to discuss the design concept and to obtain the requirements for planning, design analysis, and documentation.

1.6.1 NATURAL UNENCROACHED CHANNELS

CHAPTER 6
HYDRAULIC ANALYSIS AND DESIGN

SECTION 1.0
OPEN CHANNELS

Natural unencroached channels are defined as channels where overlot grading from the development process does not encroach into the 100-year floodplain of a given channel. Although the development does not alter the flow carrying capacity of the floodplain, it is necessary to ensure that the development is protected from movement of the floodplain boundaries due to erosion and scour. Therefore, the designer needs to identify the locations susceptible to erosion and scour and provide a design that reinforces these locations to minimize potential damage to the proposed development. For natural channels with velocities that exceed stable velocities, erosion protection may include the construction of buried grade control/check structures to minimize head-cutting and subsequent bank failures.

1.6.2 NATURAL ENCROACHED CHANNELS

Natural encroached channels are defined as channels where the development process has encroached into the 100-year floodplain fringe. This definition includes both excavation and/or fill in the floodplain fringe. The designer must prepare a design that will minimize damage to the development from movement of the floodplain boundaries due to erosion and scour. Consideration of erosion protection is similar to that for unencroached channels with emphasis on protection of the fill embankment.

1.6.3 BANK-LINED CHANNELS

Bank-lined channels are channels where the banks will be lined but the channel bottom will remain in a natural state with minimal regrading. The concerns with bank-lined channels are to minimize scour of the channel bottom at the bank lining interface as well as maintaining a stable natural channel. The designer must prepare a design that addresses scour depths at the lining interface to assure that the lining extends below this depth to avoid undermining of the lining.

1.6.4 PARTIALLY LINED CHANNELS

Partially lined channels are defined as channels in which half of the channel is lined and other half is left in a natural or unimproved condition. The concerns with partially lined channels are twofold. First, the improvement and lining of one side of the channel will cause changes to the hydraulic parameters of the unlined section which could increase erosion and scour in the unlined section. Second, floods which occur during the temporary condition may damage the improved channel section and require avoidable costly repairs.

Partially lined channels will only be allowed if:

a) The bottom paving is bonded, or there is another mechanism in place to pay for the bottom paving once the channel is completed.

b) Erosion in the unlined section is addressed to the satisfaction of the local official.

c) Scour below the lining is addressed to the satisfaction of the local official.

The analysis and design must show that the proposed temporary channel does not adversely impact the hydraulic parameters and stability of the unlined section in a significant way.

1.7 DESIGN STANDARDS FOR MAJOR ARTIFICIAL DRAINAGEWAYS

Presented in this section are the typical improved channel design sections. A graphical illustration of the typical design sections is presented in Figure CH6-F104. The selection of a channel section and lining type is generally dependent on physical and economic channel restrictions (i.e. value of developable land), the slope of the proposed channel alignment, the rate of flow to be conveyed by the channel, and the comparative costs of the lining materials. The channel sections and linings discussed herein provide a range of options from which an appropriate channel may be selected. Specific hydraulic design standards that are applicable to all improved channels (i.e. transition, freeboard, etc.) are presented later in Section 1.8.

Within this section, six types of improved channels will be discussed: unlined channels, grass-lined channels, wetland bottom channels, riprap-lined channels, concrete-lined channels, and channels with other types of linings.

1.7.1 PERMANENT UNLINED CHANNELS

Permanent unlined channels are improved channels, which are constructed to the shape of vegetation-lined channels but are not re-vegetated. The cost of construction of these channels is relatively low for areas with flat slopes and where the design flow rates and velocities are small. The designer must adequately address potential erosion problem areas (i.e. bends, transitions, structures) as well as the overall stability of the unlined channel and the effect that possible future natural re-vegetation may have on the channel hydraulics. The stability of the channel shall be analyzed as if the channel was a natural channel using the design standards in Section 1.6 of this Chapter.

1.7.2 GRASS-LINED CHANNELS

Grass-lined channels may be considered to be the most desirable artificial channels from an aesthetic viewpoint.

CHAPTER 6
HYDRAULIC ANALYSIS AND DESIGN

SECTION 1.0
OPEN CHANNELS

The channel storage, lower velocities, and the sociological benefits create significant advantages over other types of channels. The designer must give full consideration to flow hydraulics for which calculations shall be submitted for review and approval by the local official.

The satisfactory performance of a grass-lined channel depends on constructing the channel with the proper shape and preparing the area in a manner to provide conditions favorable to vegetative growth. Between the time of seeding and the actual establishments of the grass, the channel is unprotected and subject to considerable damage unless special protection is provided. Channels subject to constant or prolonged flows require special supplemental treatment, such as grade control structures, stone centers, or subsurface drainage capable of carrying such flows. After establishment, the protective vegetative cover must be maintained.

A maintenance agreement and/or bond may be required to cover maintenance of grass-lined channels. In addition, the grass-lined channels may not be allowed on project sites where insufficient precipitation exists to maintain grass lining without irrigation.

1.7.2.1 LONGITUDINAL CHANNEL SLOPES

Grass-lined channel slopes are dictated by maximum permissible velocity requirements. Where the natural topography is steeper then desirable, drop structures may be utilized to maintain design velocities.

1.7.2.2 ROUGHNESS COEFFICIENT

The Manning's roughness coefficient used in the channel design shall be obtained from Figure CH6-F105 assuming a mature channel (i.e., substantial vegetation with minimal maintenance).

1.7.2.3 LOW FLOW AND TRICKLE CHANNELS

Low flows or base flows, from urban areas must be given specific attention. Waterways that are normally dry prior to urbanization will often have a continuous flow after urbanization because of lawn irrigation return flows, both overland and from ground water in-flow. Since continuous flow over grass will destroy a grass stand and may cause the channel profile to degrade, trickle channels or low flow channels are required on all urban grass-lined channels. Concrete trickle channels are preferred because of their ease of maintenance. Other types are

Trickle channels or low flow channels are required on all urban grass-lined channels. Concrete trickle channels are preferred because of their ease of maintenance. Trickle channels may not be practical on larger major drainageways, streams and rivers, or in channels located on sandy soils where a low flow channel may be the more appropriate choice.

acceptable if they are properly designed. Trickle channels may not be practical on larger major drainageways, streams and rivers, or in channels located on sandy soils where a low flow channel may be the more appropriate choice.

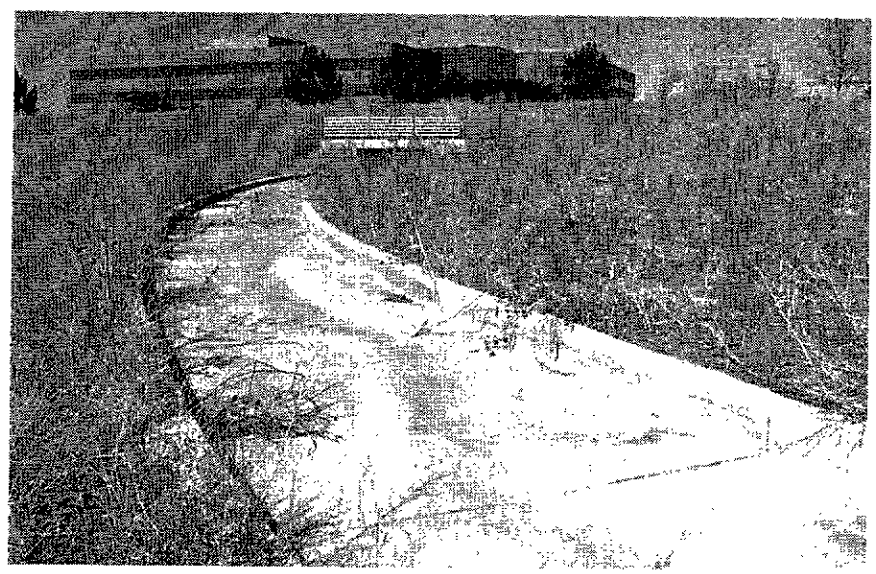
Trickle channels are used for channels with a 100-year design flow less than or equal to 200 cfs. The trickle channel's capacity should be a minimum of 5.0 percent of the 100-year design flow rate or 5 cfs, whichever is greater.

Low-flow channels will be used in channels with a 100-year flow greater than 200 cfs. The low-flow channel will have the capacity to carry the 2-year flow event with no freeboard.

a) Trickle Channels

Trickle channels are used for channels with a 100-year design flow less than or equal to 200 cfs. The trickle channel's capacity should be a minimum of 5.0 percent of the 100-year design flow rate or 5 cfs, whichever is greater. The flow capacity of the main channel should be determined without considering the flow capacity of the trickle channel. Care must be taken to ensure that low flows enter the trickle channel without flow paralleling the trickle channel or bypassing the inlets.

i) Concrete Trickle Channel: To prevent erosion, silting, and excessive plant growth, concrete trickle channels are preferred. The concrete trickle channel shall have a minimum depth of 6 inches. A Manning's roughness coefficient value of 0.015 will be used to design the concrete trickle channel. The trickle channel shall be a minimum 6-inches thick with, as a minimum, #4 reinforcement at 12-inches each direction. Figure CH6-F106 shows a typical cross-section of a concrete trickle channel.



ii) Riprap Trickle Channel: The riprap trickle channel shall have a minimum depth of 12 inches. Manning's roughness coefficient will be determined by CH6-106. Figure CH6-F107 is a typical cross-section of a riprap trickle channel.

b) Low Flow Channels

Low-flow channels will be used in channels with a 100-year flow greater than 200 cfs. The low-flow channel will have the capacity to carry the 2-year flow event with no freeboard. Low-flow channels are used to contain relatively frequently occurring flows within a recognizable channel section. The flow capacity of the main channel should include the flow in the low flow channel. Figure CH6-F108 illustrates an example of a low-flow channel.

Low-flow channels shall have a minimum depth of 12 inches. The riprap-lined side slopes of the low-flow channel will be 2.5:1 to 3:1. The main channel depth limitation does not apply to the low-flow channel area of the total channel cross-section.

1.7.2.4 BOTTOM WIDTH

The following design factors should be considered in selecting an appropriate channel bottom width.

- Constructability
- Channel stability and maintenance
- Multi-use purpose
- Trickle/low flow channel width

1.7.2.5 FLOW DEPTH

Typically, the maximum design depth of flow (outside the low flow channel area) should not exceed 5.0 feet for a 100-year flow of 1,500 cfs or less. For greater flows, excessive depths should be avoided to minimize high velocities and for public safely considerations.

1.7.2.6 SIDE SLOPES

Side slopes shall not be designed steeper than 3 horizontal to 1 vertical. The use of 4 horizontal to 1 vertical side slope is recommended.

1.7.2.7 GRASS LINING

The grass lining for channels shall be seeded or sodded with a grass species adapted to the local climate and will flourish without irrigation. Flowering plants (i.e. Honeysuckle) and weeds shall not be used for grass-lined channels.

1.7.2.8 ESTABLISHING VEGETATION

Channel vegetation is usually established by seeding. In the more critical sections of some channels, it may be desirable to provide immediate protection by transplanting a complete sod cover.

Jute, plastic, paper mesh, hay mulch may be used to protect the entire width and side slopes of a waterway until the vegetation becomes established. All seeding, planting, and sodding should conform to local agronomic recommendations.

1.7.2.9 CHANNEL BEND PROTECTION

The potential for erosion increases along the outside bank of a channel bend due to the acceleration of flow velocities on the outside part of the bend. Thus, it is often necessary to provide erosion protection in natural or grass-lined channels which otherwise would not need protection.

In erosion resistant soils, no extra protection is required along bends where the radius is greater than 2 times the top width of the 100-year water surface, but in no case less than 100 feet. Channel bends with radii smaller than stated above require erosion protection. If erosion protection is provided, the minimum radius is 1.2 times the top width and in no case less than 50 feet. Erosion protection should extend downstream from the end of the bend a distance that is equal to the length of the bend measured along the channel centerline.

1.7.3 WETLAND BOTTOM CHANNEL

Under certain circumstances, such as when existing wetland areas are affected or natural channels are modified, the Corps of Engineers Section 404 permitting process may mandate the use of channels with wetland vegetation in their bottoms. In other cases, a wetland bottom channel may better suit individual site needs if used to mitigate wetland damages somewhere else or if used to enhance urban runoff quality. These types of channels are in essence grass-lined channels; with the exception that wetland type vegetation is encouraged to grow in their bottom. The easiest way to achieve this is to eliminate the concrete lined trickle/low-flow channel from the channel bottom and to limit the channel longitudinal slope so that low flows have low velocities.

There are potential benefits associated with a wetland bottom channel. These include habitat for aquatic, terrestrial, and avian wildlife and possible water quality enhancement as the base flows move through the marshy vegetation.

The down side of this practice is that the channel bottom is "boggy" and can become overgrown. This more abundant bottom vegetation traps sediments, thereby reducing channel flow carrying capacity as the bottom fills with sediments. Depending on the sediment loads being carried by the flows, the channel bottom will eventually have to be dredged to restore its flood carrying capacity or the channel section must be over-designed to compensate for the sediment deposition within the channel. Wetland bottom channels can provide habitat for mosquito breeding, and because the abundant vegetation can dislodge during a flood, an increased potential exists for blockage of roadway crossing structures.

The design of channels with wetland bottoms can be a complicated and iterative process. In order to simplify the design procedure for this manual, assumptions have been made concerning how the flow depth in a channel interacts with the wetland vegetation and affects the channel roughness and the rate of sediment deposition on the bottom.

1.7.3.1 LONGITUDINAL CHANNEL SLOPE

The longitudinal channel slope should be set so the maximum permissible velocity criteria provided in Table CH6-T103 is not violated. To prevent channel degradation, the channel slope should be determined assuming there is no wetland vegetation on the bottom (i.e., "New Channel"). In addition to the velocity requirements, the Froude Number for the New Channel condition shall be less than 0.7.

1.7.3.2 ROUGHNESS COEFFICIENTS

The channel must be designed for two flow roughness conditions. A Manning's roughness coefficient assuming there is no growth in the channel bottom is used to set the channel slope. The required channel depth including freeboard is determined assuming Mature Channel conditions.

The channel must be designed for two flow roughness conditions. As previously mentioned, a Manning's roughness coefficient assuming there is no growth in the channel bottom is used to set the channel slope. This is referred to as the New Channel condition. The Mature Channel condition assumes that wetland vegetation in the channel bottom has been established. The required channel depth including freeboard is determined assuming Mature Channel conditions.

A composite Manning's roughness coefficient should be used for the New Channel condition design and the Mature Channel condition design. The composite Manning's roughness coefficient is determined by the following equation (Chow, 1959):

$$n_c = \frac{(n_0^2 P_0 + n_w^2 P_w)^{0.5}}{(P_o + P_w)} \qquad \text{(Eq. CH6-105)}$$

Where n_c = Manning's roughness coefficient for the composite channel (Dimensionless)
n_o = Manning's roughness coefficient for areas above the wetland area (Dimensionless)
n_w = Manning's roughness coefficient for the wetland area (Dimensionless)
P_o = Wetland perimeter of channel cross-section above the wetland area (feet)
P_w = Wetland perimeter of the wetland channel bottom (feet)

For grass-lined areas above the wetland area, use a Manning's roughness coefficient, n_o, of 0.035. Manning's roughness coefficients for the wetland area (N_w) can be obtained from Figure CH6-F109.

1.7.3.3 LOW-FLOW CHANNEL

Trickle channels are not permitted in wetland bottom channels. Low-flow channels may be used when the 100-year flow exceeds 1,000 cfs. The design of the low flow channel should be according to Section 1.7.2.3 of this Chapter.

1.7.3.4 BOTTOM WIDTH

The following design factors should be considered in selecting an appropriate channel bottom width.

- Wetland mitigation requirements
- Constructability
- Channel stability and maintenance
- Multi-use purpose
- Low flow channel width

1.7.3.5 FLOW DEPTH

Typically, the maximum design depth of flow (outside the low flow channel area) should not exceed 5.0 feet for a 100-year flow of 1,500 cfs or less. For greater flows, excessive depths should be avoided to minimize high velocities and for public safely considerations.

1.7.3.6 SIDE SLOPES

Side slopes shall not be designed steeper than 3 horizontal to 1 vertical.

1.7.3.7 GRASS LINING

The side slopes may be grass-lined according to the guidelines provided previously in Sections 1.7.2.7 and 1.7.2.8.

1.7.3.8 CHANNEL BEND PROTECTION

Channel bends shall be designed according to the criteria discussed previously in Section 1.7.2.9.

1.7.3.9 CHANNEL CROSSINGS

Whenever a wetland bottom channel is crossed by a road, railroad or a trail requiring a culvert or a bridge, a drop structure should be provided immediately downstream of such a crossing. This will help reduce the silting-in of the crossing with sediments. A 1-foot to 2-foot drop is recommended. The designer shall determine the hydraulics of the crossing and the drop structure and design the structures to ensure the stability of the channel.

1.7.3.10 LIFE EXPECTANCY

Wetland vegetation bottom channels are expected to fill with sediment over time. This occurs because the bottom vegetation traps some of the sediments carried by the flow. The life expectancy of such a channel will depend primarily on the land use of the tributary watershed and could range anywhere from 20 to 40 years before major channel dredging is needed. However, life expectancy can be dramatically reduced, to as little as two to five years, if land erosion in the tributary watershed is not controlled. Therefore, land erosion practices need to be strictly controlled during new construction within the watershed and all facilities need to be built to minimize soil erosion in the watershed to maintain a reasonable economic life of a wetland bottom channel.

1.7.4 RIPRAP-LINED CHANNELS

Riprap-lined channels are defined as channels in which riprap is used for lining of the channel banks and the channel bottom, if required. Riprap used for erosion protection at transitions and bends is also considered as a riprap-lined channel and those portions shall be designed in accordance with the riprap-lined channel and transition design standards. The design standards presented in this section are the minimum hydraulic design parameters.

Riprap has proven to be an effective means to deter erosion along channel banks, in channel beds, upstream and downstream from hydraulic structures, at bends, at bridges, and in other areas where erosive tendencies exist. Riprap is a popular choice for erosion protection because the initial installation costs are often less than alternative methods for preventing erosion. However, the designer needs to bear in mind that there are additional costs associated with riprap erosion protection since riprap installations require periodic inspection and maintenance.

Channel linings constructed from loose riprap or grouted riprap to control channel erosion have been found to be cost effective where channel reaches are relatively short (less than 3 miles). Situations for which riprap lining might be appropriate are: 1) where major flows, such as the 100-year flood are found to produce channel velocities in excess of allowable non-eroding values; 2) where channel side slopes must be steeper than 3:1; 3) for low flow channels, and 4) where rapid changes in channel geometry occur such as channel bends and transitions. Design criteria applicable to these situations are presented in the following sections.

1.7.4.1 LONGITUDINAL CHANNEL SLOPE

Riprap-lined channel slopes are dictated by the maximum permissible velocity requirements (Table CH6-T103). Where topography is steeper than desirable, drop structures could be utilized to maintain design velocities.

1.7.4.2 ROUGHNESS COEFFICIENTS

The Manning's roughness coefficient, n, for hydraulic computations may be estimated for loose riprap using the following equation.

$$n = .0395\,(d_{50})^{1/6} \qquad \text{(Eq. CH6-106)}$$

Where d_{50} = mean stone size (feet)

This equation (Anderson, 1968) does not apply to grouted riprap (n= .023 to .030) or to very shallow flow (hydraulic radius is less than or equal to 2 times the maximum rock size) where the roughness coefficient will be greater than indicated by the formula.

1.7.4.3 LOW FLOW CHANNEL

The design criteria for the low-flow channel are discussed in the previous Section 1.7.2.3.

1.7.4.4 BOTTOM WIDTH

The following design factors should be considered in selecting an appropriate channel bottom width.

- Constructability
- Channel stability and maintenance
- Multi-use purpose
- Trickle/low flow channel width

1.7.4.5 FLOW DEPTH

As preliminary criteria, the design depth of flow for the major (100-year) storm runoff flow should not exceed 7.0 feet in areas of the channel cross-section outside the low-flow or trickle channel.

1.7.4.6 SIDE SLOPES

Due to stability, safety, and maintenance considerations, riprap-lined side slopes shall be 2 horizontal to 1 vertical or flatter.

1.7.4.7 TOE PROTECTION

Where only the channel sides are to be lined, additional riprap is needed to provide for long-term stability of the lining. In this case, the riprap blanket should extend a minimum of 3 feet below the proposed channel bed, and the thickness of the blanket below the proposed channel bed should be increased to a minimum of 3 times d_{50} to accommodate possible channel scour during floods. If the velocity exceeds the permissible velocity requirements of the soil comprising the channel bottom, a scour analysis should be performed to determine if the toe requires additional protection.

1.7.4.8 BEGINNING AND END OF RIPRAP-LINED CHANNEL

At the upstream and downstream termination of a riprap lining, the thickness should be increased 50 percent for at least 3 feet to prevent undercutting. Depending on the site-specific conditions, concrete cutoff walls at both ends may be necessary.

1.7.4.9 LOOSE RIPRAP LINING

Rock having a minimum specific gravity of 2.65 is preferred; however, in no case shall the specific gravity of the individual stones be less than 2.50.

Classification and gradation for riprap are shown in Table CH6-T104 and are based on a minimum specific gravity of 2.50 for the rock.

Loose riprap, or simply riprap, refers to a protective blanket of large loose angular stones that are usually placed by machine to achieve a desired configuration. The term loose riprap has been introduced to differentiate loose stones from grouted riprap.

Many factors govern the size of the rock necessary to resist the forces tending to move the riprap. For the riprap itself, this includes the size and weight of the individual rock, the shape of stones, the gradation of the particles, the blanket thickness, the type of bedding under the riprap, and slope of the riprap layer. Hydraulic factors affecting riprap include the velocity, current direction, eddy action, and waves. Figure CH6-F110 provides typical cross-sections for riprap-lined channels.

Experience has shown that riprap failures generally result from undersized individual rocks in the maximum size range, improper gradation of the rock which reduces the interlocking of individual particles and improper bedding for the riprap which allows leaching of channel particles through the riprap blanket.

a) Riprap Material

Rock used for loose riprap, grouted riprap, or wire enclosed riprap should be hard, durable, angular in shape, and free from cracks, overburden, shale and organic matter. Neither breadth nor thickness of a single stone should be less than 1/3 of its length and rounded stone should be avoided. Rock having a minimum specific gravity of 2.65 is preferred; however, in no case shall the specific gravity of the individual stones be less than 2.50.

Classification and gradation for riprap are shown in Table CH6-T104 and are based on a minimum specific gravity of 2.50 for the rock. Because of its relatively small size and weight, riprap Class 150 must be buried with native topsoil and revegetated to protect the rock from vandalism.

Riprap lining requirements for a stable channel lining are based on the following relationship which resulted from model studies by Smith and Murray (Smith, 1965)

$$d_{50} = \frac{0.05 \ V^2 \ S^{0.34}}{(S_s - 1)^{1.332}} \qquad \text{(Eq. CH6-107)}$$

Where d_{50} = Rock size for which 50 percent of riprap by weight is smaller (feet)
V = Mean channel velocity (fps)
S = Longitudinal channel slope (feet/feet)
S_s = Specific gravity of rock (minimum S_s = 2.50) (dimensionless)

The riprap blanket thickness should be at least 2.0 times d_{50} and should extend up the side slopes to an elevation of the design water surface plus the calculated freeboard and superelevation.

b) Bedding Requirements

Long term stability of riprap erosion protection is strongly influenced by proper bedding conditions. A large percentage of all riprap failures is directly attributable to bedding failures.

Properly designed bedding provides a buffer of intermediate sized material between the channel bed and the riprap to prevent movement of soil particles through the voids in the riprap. Three types of bedding are in common use: a generic single-layer granular bedding, a granular bedding based on the T-V methodology, and filter fabric.

1) Granular Bedding - Generic Design

The gradation of a single layer bedding specification is based on the assumption that said bedding will generally protect the underlying soil from displacement during a flood event. The single layer bedding design does not require any soil information, but in order to be effective covering a wide range of soil types and sizes, this method requires a greater thickness than the T-V method.

A single 12-inch layer of said granular bedding can be used except at drop structures. At drop structures, filter fabric must be added below the 12-inch layer of granular bedding.

2) Granular Bedding - T-V Design

The T-V (Terzughi-Vicksburg) design establishes an optimum granular bedding gradation for a specific channel soil. Since this method designs the granular bedding for a particular soil, the allowable granular bedding thickness may be much less than the generic design.

The specifications for the T-V reverse filter method relate the gradation of the protective layer (filter) to that of the bed material (base) by the following inequalities:

$$D_{15(filter)} \leq 5d_{85(base)} \quad \text{(Eq. CH6-108)}$$

$$4d_{15(base)} \leq D_{15(filter)} \leq 20d_{15(base)} \quad \text{(Eq. CH6-109)}$$

$$D_{50(filter)} < 25d_{50(base)} \quad \text{(Eq. CH6-110)}$$

Where the capital "D" refers to the filter grain size and the lower case "d" to the base grain size. The subscripts refer to the percent by weight which is finer than the grain size denoted by either "D" or "d". For example, 15 percent of the filter material is finer than $D_{15(filter)}$ and 85 percent of the base material is finer than $d_{85(base)}$.

When the T-V method is used, the thickness of the resulting layer of granular bedding may be reduced to six inches. However, if a gradation analysis of the existing soils shows that a single layer of T-V Method designed granular bedding can not bridge the gap between the riprap specification and the existing soils, then two or more layers of granular bedding shall be used. The design of the bedding layer closest to the existing soils shall be based on the existing soil gradation. The design of the upper bedding layer shall be based on the gradation of the lower bedding layer. The thickness of each of the two or more layers shall be four inches.

3) Filter Fabrics

Filter fabric is not a complete substitute for granular bedding. Filter fabric provides filtering action only perpendicular to the fabric and has only a single equivalent pore opening between the channel bed and the riprap. Filter fabric has a relatively smooth surface which provides less resistance to stone movement. As a result, it is recommended that the use of filter fabric in place of granular bedding be restricted to slopes no steeper than 2.5 horizontal to 1 vertical, and that such filter fabric only replace the bottom layer in a multi-layer T-V Method granular bedding design. The granular bedding shall be placed on top of the filter fabric to act as a cushion when placing the riprap. Tears in the fabric greatly reduce its effectiveness so that direct dumping of riprap on the filter fabric is not allowed and due care must be exercised during construction. Nonetheless, filter fabric has proven to be an adequate replacement for granular bedding in many instances. Filter fabric provides adequate bedding for channel linings along uniform mild sloping channels where leaching forces are primarily perpendicular to the fabric.

At drop structures and sloped channel drops, where seepage forces may run parallel with the fabric and cause piping along the bottom surface of the fabric, special care is required in the use of filter fabric. Seepage parallel with the fabric may be reduced by folding the edge of the fabric vertically downward about 2 feet (similar to a cutoff wall) at 12-foot intervals along the installation, particularly at the entrance and exit of the channel reach. Filter fabric has to be lapped a minimum of 12 inches at roll edges with upstream fabric being placed on top of downstream fabric at the lap.

Fine silt and clay has been found to clog the openings in filter fabric. This prevents free drainage which increases failure potential due to uplift. For this reason, a granular filter is often a more appropriate bedding for fine silt and clay channel beds.

1.7.4.10 GROUTED RIPRAP LINING

Grouted riprap provides a relatively impervious channel lining which is less subject to vandalism than loose riprap. Grouted riprap requires less routine maintenance by reducing silt and trash accumulation and is particularly useful for lining low-flow channels and steep banks. The appearance of grouted riprap is enhanced by exposing the tops of individual stones and by cleaning excess grout from the projecting rock with a wet broom prior to curing. Figure CH6-F111 provides a typical cross-section for a grouted riprap lining.

a) Riprap Material

The rock used for grouted riprap is different from the standard gradation of riprap in that the smaller rock has been reduced to allow greater penetration by the grout. The riprap specifications are shown on Table CH6-T106. Riprap smaller than Class 400 should not be grouted.

b) Bedding Material

The bedding material will be the same as for loose riprap.

c) Cutoff Trench

As the riprap layer is placed, a cutoff trench should be excavated around the rock section at the top of the slope and at the upstream and downstream edges. The trench should be, at a minimum, the full depth of the riprap and bedding layer and at least 1-foot wide. This trench is filled with grout to prevent water from undermining the grouted rock mass.

d) Grout

After the riprap has been placed to the required thickness and the trench excavated, the rock is sprayed with clean water which cleans the rock and allows better adherence by the grout. The rock is then grouted using a low pressure (less than 10 psi) grout pump with a 2" maximum diameter hose. Using a low pressure grout pump allows the work crew time to move the hose and vibrate the grout. Vibrating the grout with a pencil vibrator assures complete penetration and filling of the voids. After the grout has been placed and vibrated, a small hand broom or gloved hand is used to smooth the grout and remove any excess grout from the rock. The finished surface is sealed with a curing compound.

The grout should consist of 6 sacks (564 pounds) of cement per cubic yard, and the aggregate should consist of 30% of 3/8-inch coarse gravel and 70% natural sand. The grout should contain 7.5% +/- 1.5% air entrainment, have a 28-day compressive strength of at least 2,000 p.s.i., and have a slump of 7 inches +/- 2 inches. Fiber reinforcement should be used such as 1.5 pounds per cubic yard of Fibermesh or an approved equivalent amount. A maximum of 25% flyash maybe substituted for the cementations material.

1.7.4.11 CHANNEL BEND PROTECTION

When riprap protection is required for a straight channel, increase the rock size by one category (e.g., Class 300 to Class 400) through bends. The minimum radius for a riprap-lined bend is 1.2 times the top width and in no case less than 50 feet. Riprap protection should extend downstream from the end of the bend a distance that is equal to the length of the bend measured along the channel centerline.

1.7.4.12 TRANSITION PROTECTION

Scour potential is amplified by turbulent eddies in the vicinity of rapid changes in channel geometry such at transitions and bridges. For these locations, the riprap lining thickness shall be increased by one size category.

Protection should extend upstream from the transition entrance at least 5 feet and extend downstream from the transition exit at least 10 feet. See Section 1.8 for further discussions on transitions.

1.7.4.13 CONCRETE CUTOFF WALLS

Transverse concrete cutoff walls may be required for riprap lined channels where a resulting failure of the riprap lining could seriously affect the health and safety of the public. The designer shall consult with the local officials prior to design of riprap lined-channels to determine if concrete cutoff walls are required as well as their sizing and spacing, if required.

1.7.4.14 RIPRAP-LINED CHANNELS ON STEEP SLOPES

Achieving channel stability on steep slopes usually requires some type of channel lining. The only exception is a channel constructed in durable bedrock.

On mild slopes, the water velocity is slow enough and the depth of flow is large enough (relative to the riprap size) that a reasonable estimate of the resistance to flow can be made. On steep channels, the riprap size required to stabilize the channel is on the same order of magnitude or greater than the flow depth, which invalidates the Manning's relation. Since the resistance to flow is now unknown, an estimate of the velocity needed for the design of the riprap cannot be accurately estimated.

A graphically based methodology was developed for the U.S. Department of Interior, Office of Surface Mining Reclamation and Enforcement (SIMONS, 1989) to design riprap-lined channels on steep slopes (supercritical flow). This methodology was based on a study by BRUTHURST, 1979 that analyzed the hydraulics of mountain rivers where roughness elements are on the same order of magnitude as the depth of flow. Using the resistance equation developed by Bathurst, the velocity can be estimated for a given riprap size. The velocity is then used to predict the stability of the riprap.

This procedure shall be used for all riprap lined channels whose depth of flow is equal to or less than d_{50} as computed initially using Equation CH6-107.

a) Rock Size

Five sets of design curves (Figures CH6-F112 through CH6-F116) have been developed from Bathurst's relationship to simplify riprap design for steep channels. The design curves were developed for channels with 2 to 1 side slopes and bottom widths of 0 feet, 6 feet, 10 feet, 14 feet, and 20 feet. The curves were terminated at the point where flow velocity exceeded 15 fps. A median rock diameter could be determined that would be stable at higher flows and velocities; however, rock durability at velocities greater than 15 fps becomes of greater concern.

For a given flow, channel slope, and channel width, Figures CH6-F112 through CH6-F116 will provide the median riprap size. When the channel slope is not provided by one of the design curves, linear interpolation is used to determine the riprap size. This is done by extending a horizontal line at the given flow through the curves with slopes bracketing the design slope. A curve at the design slope is then estimated by visual interpolation. The design D_{50} size is then chosen at the point that the flow intercepts the estimated design curve. Linear interpolation can also be used to estimate the D_{50} size for bottom widths other than those supplied in the figures.

For practical engineering purposes, the D_{50} size specified for the design should be given in 0.25-foot increments. The final minimum design size is determined using Table CH6-T107.

b) Riprap Gradation For Steep Slopes

Lack of proper riprap gradation is one of the most common causes of riprap failure. With the proper rock gradation, the voids formed by large stones are filled with smaller sizes in an interlocking fashion that prevents jets of water from contacting the underlying soil and ultimately eroding the soil supporting the riprap layer.

Table CH6-T108 provides ratios used to determine the D_{10}, D_{20}, and D_{max} rock sizes from the D_{50} rock size determined in the previous section. It is important to establish a smooth gradation from the largest to the smallest sizes to prevent large voids between rocks.

c) Riprap Thickness For Steep Slopes

For riprap linings on steep slopes, a thickness of 1.25 times the median rock size is recommended. The maximum resistance to the erosive forces of flowing water occurs when all rock is contained within the riprap layer thickness. Oversize rocks that protrude above the riprap layer reduce channel capacity and reduce riprap stability.

d) Riprap Placement On Steep Slopes

Improper placement is another major cause of failure in riprap-lined channels. To prevent segregation of rock sizes, riprap should never be placed by dropping it down the slope in a chute or pushing it down with a bulldozer. Rock can be dumped directly from trucks from the top of the embankment, and draglines with orange peel buckets, backhoes, and other power equipment can also be used to place riprap with minimal handwork.

e) Freeboard

Figures CH6-F112 through CH6-F116 also provide the depth of flow for a given flowrate, channel slope, and channel dimensions. The required freeboard is given by Equation CH6-115 for subcritical flow or CH6-122 for supercritical flow. The velocity can be estimated by dividing the flow rate by the area of flow.

f) Bedding Requirements on Steep Slopes

Either a granular bedding material or filter fabric may be used on steep slopes according to the requirements previously specified in Section 1.7.4.9.

1.7.5 CONCRETE-LINED CHANNELS

Concrete-lined channels are defined as rectangular or trapezoidal channels in which reinforced concrete is used for lining of the channel banks and channel bottom. The cost of concrete channels generally can be more economical than other lining types in an urban environment due to their greater flow carrying capacity resulting in less land area requirements. Special attentions should be taken to provide safety measures (i.e. fence) around the concrete channels (Section 1.10.2, Chapter 6).

The following sections present design parameters for concrete-lined channels. The design parameters presented do not relieve the designer of performing appropriate engineering analyses.

1.7.5.1 LONGITUDINAL CHANNEL SLOPE

The maximum slope of concrete-lined channels is determined by the maximum permissible velocity requirements (Table CH6-T103). Concrete-lined channels have the ability to accommodate super-critical flow conditions and thus can be constructed to almost any naturally occurring slope.

1.7.5.2 ROUGHNESS COEFFICIENTS

The Manning's roughness coefficient for concrete-lined channels is as shown in Table CH6-T102. For concrete-lined channels with subcritical flow, check the Froude Number using a roughness coefficient of 0.011.

1.7.5.3 LOW FLOW CHANNEL

The bottom of the concrete channel shall be constructed with a defined low flow channel but shall be adequately sloped to confine the low flows to the middle or one side of the channel. Low flows are defined in Section 1.7.2.3, Chapter 6.

1.7.5.4 BOTTOM WIDTH

There are no bottom width requirements for concrete-lined channels.

1.7.5.5 FLOW DEPTH

There are no flow depth requirements for concrete-lined channels.

1.7.5.6 SIDE SLOPES

Concrete-lined channels may have side slopes that are vertical or flatter.

1.7.5.7 CONCRETE LINING SECTION

a) Thickness

All concrete lining shall have a minimum thickness of 6 inches for flow velocities less than 30 fps and a minimum thickness of 7 inches for flow velocities of 30 fps and greater.

b) Concrete Joints

The following design standards, found to work in similar conditions, are suggested. Alternatives will be considered on a case-by-case basis.

- Channels shall be continuously reinforced without transverse joints. Expansion/ contraction joints (without continuous reinforcement) shall only be installed where the new concrete lining is connected to a rigid structure or to an existing concrete lining which is not continuously reinforced. The design of the expansion joint shall be coordinated with the local officials.

- Longitudinal joints, where required, shall be constructed on the sidewalls at least one foot vertically above the channel invert.

- All joints shall be designed to prevent differential movement.

- Construction joints are required for all cold joints and where the lining thickness changes. Reinforcement shall be continuous through the joint and the concrete lining shall be thickened at the joint.

c) Concrete Finish

The surface of the concrete lining shall be provided with a wood float finish unless the design requires additional finishing treatment. Excessive working or wetting of the finish shall be avoided if additional finishing is required.

d) Concrete Curing

It is suggested that concrete-lined channels be cured by the application of a liquid membrane-forming curing compound (white pigmented) upon completion of the concrete finish. All curing shall be completed in accordance with the standard specifications of the local government agency.

e) Reinforcement Steel

- Steel reinforcement shall be a minimum grade - 40 deformed bars. Wire mesh shall not be used.

- Ratio of longitudinal steel area to concrete cross sectional area shall be greater than .0905 but not less than a #4 rebar placed at a 12-inch spacing. The longitudinal steel shall be placed on top of the transverse steel.

- Ratio of transverse steel area to concrete cross sectional area shall be greater than .0025 but not less than a #4 rebar placed at a 12-inch spacing.

- Reinforcing steel shall be placed near the center of the section with a minimum clear cover of three inches adjacent to the earth.

- Additional steel shall be added as needed. If a retaining wall structure is used, the structure must be designed by a registered structural engineer with structural design calculations submitted for review and approval.

f) Earthwork

As a minimum, the following areas shall be compacted to at least 90 percent of maximum density as determined by ASTM 1557 (Modified Proctor). Additional requirements may be required by the geotechnical report.

- The 12 inches of subgrade immediately beneath concrete lining (both channel bottom and side slopes).
- Top 12 inches of maintenance road.
- Top 12 inches of earth surface within 10 feet of concrete channel lip.
- All fill material.

g) Bedding

A geotechnical report shall be submitted which addresses the required bedding necessary for the specific concrete section under consideration.

h) Underdrain and Weepholes

The necessity for longitudinal underdrains and weepholes shall be addressed in a geotechnical report submitted for the specific concrete channel section under consideration.

i) Concrete Cutoffs

A transverse concrete cutoff shall be installed at the beginning and end of the concrete-lined section of channel and at a maximum spacing of 90 feet. The concrete cutoffs shall extend a minimum of three feet below the bottom of the concrete slab and across the entire width of the channel lining. Longitudinal cutoffs, a minimum of 3 feet in depth, at top lining are required to ensure integrity of the concrete lining.

If the channel is continuously reinforced without transverse joints then a concrete cutoff is required to be incorporated into the expansion/concrete joint.

1.7.5.8 SPECIAL CONSIDERATION FOR SUPERCRITICAL FLOW

Supercritical flow in an open channel in an urbanized area creates hazards which the designer must take into consideration. Careful attention must be taken to insure against excessive waves which may extend down the entire length of the channel from only minor obstructions. Imperfections at joints may rapidly cause a

There should not be a drastic reduction in cross section shape and diligent care should be taken to minimize the change in wetted area of the cross-section at bridges and culverts.

deterioration of the joints, in which case a complete failure of the channel can readily occur. In addition, high velocity flow entering cracks or joints creates an uplift force by the conversion of velocity head to pressure head which can damage the channel lining.

Generally, there should not be a drastic reduction in cross section shape and diligent care should be taken to minimize the change in wetted area of the cross-section at bridges and culverts. Bridges and other structures crossing the channel must be anchored satisfactorily to withstand the full dynamic load which might be imposed upon the structure in the event of major debris plugging.

The concrete lining must be protected from hydrostatic uplift forces, which are often created by a high-water table or momentary inflow behind the lining from localized flooding. Generally, an underdrain will be required under and/or adjacent to the lining. The underdrain must be designed to be free draining. With supercritical flows, minor downstream obstructions do not create any backwater effect. Backwater computation methods are applicable for computing the water-surface profile or the energy gradient in channels having a supercritical flow; however, the computations must proceed in a downstream direction. The designer must take care to insure against the possibility of unanticipated hydraulic jumps forming in the channel.

1.7.6 OTHER CHANNEL LININGS

Other channel linings include all channel linings that are not discussed in the previous sections. These include composite-lined channels, which are channels in which two or more different lining materials are used (i.e. riprap bottom with concrete side slope lining). They also include gabions, soil cement linings, synthetic fabric and geotextile linings, preformed block linings, reinforced soil linings, and floodwalls (vertical walls constructed on both sides of an existing floodplain). The wide range of composite combinations and other lining types does not allow a discussion of all potential linings in this MANUAL. For those linings not discussed in this MANUAL, supporting documentation will be required to support the use of the desired lining. A guideline of some of the items which must be addressed in the supporting documentation is as follows:

a. Structural integrity of the proposed lining.

b. Interfacing between different linings.

c. The maximum velocity under which the lining will remain stable.

d. Potential erosion and scour problems.

e. Access for operations and maintenance.

f. Long term durability of the product under the extreme meteorological and soil conditions.

g. Ease of repair of damaged section.

h. Past case history (if available) of the lining system in other arid areas.

i. Potential groundwater mitigation issues (i.e. weepholes, underdrains, etc.)

These linings will be allowed on a case by case basis. The local community and/or the CWCB may reject the proposed lining system in the interests of operation, maintenance, and protecting the public safety.

1.8 ADDITIONAL HYDRAULIC DESIGN STANDARDS

Presented in this section are the hydraulic design standards for design of improved channels. The standards included herein are those standards that are the same for all improved channels. Standards which are specific to a lining type are included in the discussion for the specific lining under consideration.

1.8.1 SUBCRITICAL FLOW DESIGN STANDARDS

The following design standards are to be used when the design runoff in the channel is flowing in a Subcritical condition (F_r>0.8). Furthermore, all subcritical channels (F_r>0.8) must be designed with the limits as stated in Section 1.4.2, Chapter 6.

1.8.1.1 TRANSITIONS

For the purposes of this manual, subcritical transitions occur when transitioning one sub-critical channel section to another subcritical channel section (expansion or contraction) or when a subcritical channel section is steepened to create a super critical flow condition downstream (i.e. sloping spillway entrance). Several typical subcritical transition sections are presented in Figures CH6-F117 and CH6-F118. The warped transition section, although most efficient, should only be used in extreme cases where minimum loss of energy is required since the section is very difficult and costly to construct. Conversely, the square-ended transition should only be used when either a straight-line transition or a cylinder-quadrant transition cannot be used due to topographic constraints or utility conflicts.

a) Transition Energy Loss

The energy loss created by a contracting section may be calculated using the following equation:

$$H_t = K_{tc}\left(\frac{V_2^2}{2g} - \frac{V_1^2}{2g}\right) \qquad \text{(Eq. CH6-111)}$$

Where H_t = Energy loss (feet)
K_{tc}= Transition coefficient - contraction
V_1 = Upstream velocity (feet per second)
V_2 = Downstream velocity (feet per second)

CHAPTER 6
HYDRAULIC ANALYSIS AND DESIGN

SECTION 1.0
OPEN CHANNELS

g = Acceleration of gravity (feet per second squared)

K_{tc} values for the typical transition sections are presented in Figure CH6-F118.

Similarly, the energy loss created by an expanding transition section may be calculated using the following equation:

$$H_t = K_{te}\left(\frac{V_1^2}{2g} - \frac{V_2^2}{2g}\right) \qquad \text{(Eq. CH6-112)}$$

Where H_t = Energy loss (feet)
K_{te}= Transition coefficient - expansion
V_1 = Upstream velocity (feet per second)
V_2 = Downstream velocity (feet per second)
g = Acceleration of gravity (feet per second squared)

K_{te} values for the typical transition sections are also presented in Figure CH6-F118.

The energy loss in a contracting transition for straight-line or warped transitions is allowed to be partially or totally accommodated by sloping the transition channel bottom from the transition entrance to the exit.

b) Transition Length

The length of the transition section should be long enough to keep the streamlines smooth and nearly parallel throughout the expanding (contracting) section. Experimental data and performance of existing structures have been used to estimate the minimum transition length necessary to maintain the stated flow conditions. Based on this information, the minimum length of the transition section shall be as follows:

$$L_t \geq 0.5 L_c(\Delta T_w) \qquad \text{(Eq. CH6-113)}$$

Where L_t = Minimum transition length (feet)
L_c = Length coefficient (dimensionless)
ΔT_w = Difference in the top width of the normal water surface upstream and downstream of the transition (feet)

For an approach flow velocity less than 12 feet per second, L_c = 4.5. This represents a 4.5 (length) to 1.0 (width) wall expansion or contraction with the angle of expansion or contraction of 12.5 degrees from the channel centerline. For an approach flow velocity equal to or greater than 12 feet per second, L_c = 10.0. This represents a 10.0 (length) to 1.0 (width) expansion or contraction with the angle of expansion or contraction of about 5.75 degrees from the channel centerline.

The transition length equation is not applicable to cylinder-quadrant or square-ended transitions.

1.8.1.2 SUPERELEVATION IN BENDS

Superelevation in bends is estimated from the following equations:

$$S_e = \frac{C V^2 T_w}{rg} \qquad \text{(Eq. CH6-114)}$$

Where r = Radius of curvature (feet)
C = Superelevation coefficient (=0.5 for subcritical flow)
S_e = Superelevation water surface increase (feet)
T_w = Top width of the design water surface (feet)
V = Mean design velocity (feet per second)
g = Acceleration of gravity (feet per second squared)

Superelevation shall be limited to a maximum of 1.0 feet, and the radius of curvature shall conform to the requirements provided in Section 1.7.2.9, Chapter 6.

1.8.1.3 FREEBOARD

All subcritical channels shall be constructed with a minimum freeboard determined as follows:

$$F_b = 0.5 + \frac{V^2}{2g} \qquad \text{(Eq. CH6-115)}$$

Where F_b = Freeboard height (feet)
V = Mean design velocity (feet per second)
g = Acceleration of gravity (feet per second squared)

In no case shall the freeboard be less than 1.0 foot. All channel linings must extend to the freeboard height plus the increase in water surface elevation due to superelevation.

1.8.2 SUPERCRITICAL FLOW DESIGN STANDARDS

The following design standards are to be used when the design runoff in the channel is flowing in a supercritical condition (F_r>1.13). Furthermore, all supercritical channels must be designed within the limits as situated in Section 1.4.2, Chapter 6

1.8.2.1 SUPER CRITICAL TRANSITIONS

The design of supercritical flow in a transition is much more complicated and requires more special attention than a subcritical transition design due to the potential damaging effects of the oblique jump which is created by the transition. The oblique jump results in cross waves and higher flow depths which can cause severe damage if not properly accounted for in the design. A simpler design analysis is to force a hydraulic jump (supercritical flow to subcritical flow). However, hydraulic jumps must also be carefully designed to assure the jump will remain where the jump is designed to occur. Hydraulic jumps shall not be designed to occur in an erodible channel section but only within energy dissipation or drop structure. The design guidelines of these structures are presented in Chapter 6, Section 6.

a) Contracting Transitions

Presented in Figure CH6-F119 is an example of a supercritical contracting transition. As shown in this figure, the upstream flow is contracted from width b_1 to b_3 with a wall diffraction angle of θ. The oblique jump occurs at the points A and B where the diffraction angles start. Wave fronts generated by the oblique jumps on both sides propagate toward the centerline with a wave angle β_1. Since the flow pattern is symmetric, the centerline acts as if there was a solid wall that causes a subsequent oblique jump and generates a backward wave front toward the wall with another angle β_2. These continuous oblique jumps result in turbulent fluctuations in the water surface.

To minimize the turbulence, the first two wave fronts are designed to meet at the center and then end at the exit of the contraction. Using the contraction geometry, the length of the transition shall be as follows:

$$L_t = \frac{b_1 - b_3}{2\tan\theta} \qquad \text{(Eq. CH6-116)}$$

Where L_t = Transition length (feet)
b_1 = Upstream top width of flow (feet)
b_3 = Downstream top width of flow (feet)
θ = Wall angle as related to the channel centerline (degrees)

Using the continuity principle,

$$\frac{b_1}{b_3} = \left(\frac{Y_3^{1.5}}{Y_1}\right)\left(\frac{F_3}{F_1}\right) \qquad \text{(Eq. CH6-117)}$$

Where Y_1 = Upstream depth of flow (feet)
Y_3 = Downstream depth of flow (feet)
F_1 = Upstream Froude Number
F_3 = Downstream Froude Number

Also, by the continuity and momentum principals, the following relationship between the Froude Number, wave angle, and wall angle is found to be:

$$\tan\theta = \frac{\tan\beta_1[(1+8F_1^2\sin^2\beta_1)^{1/2}-3]}{2\tan^2\beta_1+(1+8F_1^2\sin^2\beta_1)^{1/2}-1} \quad \text{(Eq. CH6-118)}$$

Where β_1 = Initial wave angle (degrees)

Equations CH6-116, CH6-117, CH6-118 can be used by trial and error to determine the transition length and wall angle. However, Figure CH6-F120 is provided to allow a quicker trial and error solution than by using the equations. The procedure to determine the transition length and wall angle between two predetermined channel sections using Figure CH6-F120 is as follows:

Step 1: Determine the upstream and downstream channel flow conditions including flow depths, velocities, and Froude numbers.

Step 2: If either or both sections are trapezoidal, convert the trapezoidal flow parameters to equivalent rectangular flow parameters by calculating an equivalent flow width equal to the flow area divided by the flow depth. This computed flow width is used for all calculations.

Step 3: Compute Y_3/Y_1

Step 4: Assume a trial wall angle, θ

Step 5: Using θ and F_1, read the values of F_2 and Y_2/Y_1 for Section 1 from Figure CH6-F120. Then, replacing F_1 with F_2 read a second F_2 (really F_3) and second Y_2/Y_1 (really Y_3/Y_2) from Figure CH6-F120 for Section 2.

Step 6: Compute the first trial value of Y_3/Y_1 by multiplying the Y_2/Y_1 for Section 1 by the Y_2/Y_1 (really Y_3/Y_2) for Section 2.

Step 7: Compare the first trial Y_3/Y_1 to the actual Y_3/Y_1 (Step 3). If the trial value Y_3/Y_1 is larger than the actual Y_3/Y_1, assume a smaller θ and redo Steps 5 through 7. If the trial value Y_3/Y_1 is smaller than the actual Y_3/Y_1, assume a larger θ and redo Steps 5 through 7.

Step 8: Repeat the trial and error procedure until the computed Y_3/Y_1 is within the five percent of the actual Y_3/Y_1 .

Step 9: Compute the transition length using Equation CH6-121 and the last assumed value of θ.

Figure CH6-F120 can also be used to determine the wave angle, β, or may be used with the equations to determine the required downstream depth or width parameter if a certain transition length is desired or required.

To minimize the length of the transition section, Y_3/Y_1 should generally be between 2 and 3. However, F_3 shall not be less than 1.7 for all transition designs. For further discussion on oblique jumps and supercritical contractions, refer to Chow, 1959.

b) Expanding Transitions

The goal of a properly designed expansion transition is to expand the flow boundaries at the same rate as the natural flow expansion. Based on experimental and analytical data results, the minimum length of a supercritical expansion shall be as follows:

$$L_t \geq 1.5(\Delta T_w) F_{r1} \quad \text{(Eq. CH6-119)}$$

Where L_t = Minimum transition length (feet)
ΔT_w = Difference in the top width of the normal water surface upstream and downstream of the transition
F_{r1} = Upstream Froude number

1.8.2.2 SUPERELEVATION IN BENDS

Bends in supercritical channels create cross waves and superelevated flow in the bend section as well as further downstream from the bend. In order to minimize these disturbances, the radius of curvature in the bend shall not cause superelevation of the water surface exceeding two feet. Equation CH6-114 can be modified to determine the allowable radius of curvature of a channel for a given superelevation value. In no case shall the radius of curvature be less than 50 feet.

$$r = \frac{C(V^2 T_w)}{(S_e\ g)} \quad \text{(Eq. CH6-120)}$$

C shall equal 1.0 for all trapezoidal channels and for rectangular channels without transition curves. For rectangular channels with transition curves, C shall equal 0.5.

1.8.2.3 CIRCULAR TRANSITION CURVES

When a designer desires to reduce the required amount of freeboard and radius of curvature in a rectangular channel, a circular transition curve may be used. The length of the transition curve measured along the channel centerline shall be determined as follows:

$$L_c = \frac{0.32 T_w V}{y^{0.5}} \quad \text{(Eq. CH6-121)}$$

Where L_c = Length of transition curve (feet)

T_w = Top width of design water surface (feet)
V = Mean design velocity (feet per second)
y = Depth of design flow (feet)

The radius of the transition curves should be twice the radius of the main bend. Transition curves shall be located both upstream and downstream of the main bend.

1.8.2.4 FREEBOARD

In supercritical channels, adequate channel freeboard above the designed water surface shall be provided and shall not be less than that determined by the following:

$$F_b = 1.0 + 0.025V(d)^{1/3} \qquad \text{(Eq. CH6-122)}$$

Where F_b = Freeboard height (feet)
V = Velocity (feet per second)
d = depth of flow (feet)

Freeboard shall be in addition to superelevation, standing waves, and/or other water surface disturbances.

The channel lining side slopes shall be extended, as a minimum, to the freeboard elevation.

1.8.2.5 SLUG FLOW

Slug flow is a series of shallow-water shock waves that occur in steep super critical channels. The resulting wave heights may easily overtop channel linings using the typical freeboard requirements presented in this MANUAL or damage the channel lining. Therefore, all channels shall be designed to avoid the occurrence of slug flow. To avoid slug flow when the Froude Number is greater than 2.0, the channel slope shall be as follows:

$$S \le \frac{12}{R_e} \qquad \text{(Eq. CH6-123)}$$

Where S = Channel slope (feet per feet)
R_e =Reynolds Number = (VR/v) (Eq. CH6-124)
V = Mean design velocity (feet per second)
R = Hydraulic radius (feet)
v = Kinematic viscosity of water (feet squared per second)

Theoretically, slug flow will not occur with $F_r < 2.0$.

1.9 DESIGN STANDARDS FOR MINOR ARTIFICIAL DRAINAGEWAYS

A minor drainageway is defined as a channel/drainageway with a contributing tributary area of less than 160 acres. Additional flexibility and less stringent standards may be allowed for minor drainageways. Only the differences in a channel type's design as a minor drainageway versus that of a major drainageway are presented in this section.

A minor drainageway is defined as a channel/ drainageway with a contributing tributary area of less than 160 acres. Only the differences in a channel type's design as a minor drainageway versus that of a major drainageway are presented in this section.

1.9.1 GRASS-LINED CHANNELS

1.9.1.1 FREEBOARD

For swales and drainageways with a 100-year flow of equal to or less than 10 cfs, the minimum freeboard requirements is 6 inches.

1.9.1.2 CURVATURE (HORIZONTAL

The minimum radius for channels with a 100-year runoff of 20 cfs or less shall be 25 feet.

1.9.1.3 TRICKLE CHANNEL

For 100-year runoff peaks of 20 cfs or less, trickle channel requirements will be evaluated for each case. Trickle channels help preserve swales crossing residential property. Factors to be considered when establishing the need for trickle channels are: drainage slope, flow velocity, soil type, and upstream impervious area.

1.9.2 WETLAND BOTTOM CHANNELS

1.9.2.1 CURVATURE (HORIZONTAL)

The minimum radius for channels with a 100-year runoff of 20 cfs or less shall be 25 feet.

1.9.3 CONCRETE-LINED CHANNELS

1.9.3.1 FREEBOARD

For swales and drainageways with a 100-year flow of equal to or less than 10 cfs, the minimum freeboard requirements is 6 inches.

1.9.3.2 CURVATURE (HORIZONTAL)

The minimum radius for channels with a 100-year runoff of 20 cfs or less shall be 25 feet.

1.9.3.3 TRICKLE CHANNEL

For 100-year runoff peaks of 20 cfs or less, trickle channel requirements will be evaluated for each case. Trickle channels help preserve swales crossing residential property. Factors to be considered when establishing the need for trickle channels are: drainage slope, flow velocity, soil type, and upstream impervious area.

1.9.4 RIPRAP-LINED CHANNELS

1.9.4.1 FREEBOARD

For swales and drainageways with a 100-year flow of equal to or less than 10 cfs, the minimum freeboard requirements is 6 inches.

1.9.4.2 CURVATURE (HORIZONTAL)

The minimum radius for channels with a 100-year runoff of 20 cfs or less shall be 25 feet.

1.9.4.3 TRICKLE CHANNEL

For 100-year runoff peaks of 20 cfs or less, trickle channel requirements will be evaluated for each case. Trickle channels help preserve swales crossing residential property. Factors to be considered when establishing the need for trickle channels are: drainage slope, flow velocity, soil type, and upstream impervious area.

1.10 CHANNEL APPURTENANCES

Presented in this section are the design standards for appurtenances to improved channels. All improved channels shall be designed to include these appurtenances.

1.10.1 MAINTENANCE ACCESS ROAD

A maintenance access road with a minimum passage width of 12 feet shall be provided along the entire length of all improved channels with 100-year design capacity equal to or greater than 50 cfs. For such channels with less than 50 feet in top width, one maintenance access shall be provided as part of the channel improvements. For channels with greater than 50 feet in top width, the maintenance road shall be located in or within 10 feet horizontal distance from the bottom of the channel or on both sides at the channel top.

A maintenance access road with a minimum passage width of 12 feet shall be provided along the entire length of all improved channels with 100-year design capacity equal to or greater than 50 cfs.

For channels with the maintenance access road at or near the channel bottom, ramps to said road shall be provided at a maximum 10 percent slope. Said ramps shall slope down in the down gradient direction of the channel.

1.10.2 SAFETY REQUIREMENTS

The following safety requirements are required for concrete-lined channels. Similar safety requirements may be required for all other channels:

a. A six-foot high galvanized-coated chain link or comparable fence shall be installed to prevent unauthorized access. The fence shall be located at the edge of the ROW or on the top of the channel lining. Gates, with top latch, shall be placed at major access points or 1,320-foot intervals, whichever is less.

b. Ladder-type steps shall be installed not more than 1,200 feet apart and shall be staggered on alternating sides of the channel to provide a ladder every 600 feet. The bottom rung shall be placed approximately 12 inches vertically above the channel invert.

1.10.3 CULVERT OUTLET PROTECTION

If the flow velocity at a culvert or storm sewer outlet exceeds the maximum permissible velocity for the local soil or channel lining, channel protection is required. This protection usually consists of an erosion resistant reach, such as riprap, to provide a stable reach at the outlet in which the exit velocity is reduced to a velocity allowable in the downstream channel.

The following basin sizing procedure shall be used for culvert sizes less than or equal to 36-inches in diameter or equivalent open area and outlet velocities less than 15 fps. For larger culverts or outlet velocities greater than 15 fps, the outlet protection design provided for in USDOT, 1983 shall be used.

1.10.3.1 BASIN CONFIGURATION

The length of the outlet protection (L_a) is determined using the following empirical relationships that were developed for the U.S. Environmental Protection Agency (USEPA, 1976):

$$L_a = \frac{1.8Q}{D_o^{3/2}} + 7D_0 \,, \quad \text{for TW} < \frac{D_o}{2} \qquad \text{(Eq. CH6-125)}$$

and

$$L_a = \frac{3Q}{D_o^{3/2}} + 7D_o \,, \quad \text{for TW} \geq \frac{D_o}{2} \qquad \text{(Eq. CH6-126)}$$

Where D_o = Maximum inside culvert width (ft)
Q = Pipe discharge (cfs)
TW = Tailwater depth (ft)

Where there is no well defined channel downstream of the apron, the width, W,of the outlet and of the apron (as shown in Figure CH6-F121) should be as follows:

CHAPTER 6
HYDRAULIC ANALYSIS AND DESIGN

SECTION 1.0
OPEN CHANNELS

$$W = 3D_o + 0.4L_a \text{ , for TW} \geq \frac{D_o}{2} \qquad \text{(Eq. CH6-127)}$$

and

$$W = 3D_o + L_a \text{ , for TW} < \frac{D_o}{2} \qquad \text{(Eq. CH6-128)}$$

The width of the apron at the culvert outlet should be at least 3 times the culvert width.

Where there is a well-defined channel downstream of the apron, the bottom width of the apron should be at least equal to the bottom width of the channel and the lining should extend at least one foot above the tailwater elevation and at least two-thirds of the vertical conduit dimension above the invert.

The apron side slopes should be 2:1 or flatter, and the bottom grade should be level.

1.10.3.2 ROCK SIZE

The median stone diameter, d_{50} is determined from the following equation:

$$d_{50} = 0.02\frac{(Q)^{4/3}}{TW(D_o)} \qquad \text{(Eq. CH6-129)}$$

Existing scour holes may be used where flat aprons are impractical. Figure CH6-F122 shows a general design of a scour hole. The stone diameter is determined using the following equations:

$$d_{50} = \frac{0.0125(Q)^{4/3}}{TW(D_o)} \text{ , for } Y = \frac{D_o}{2} \qquad \text{(Eq. CH6-130)}$$

Also,

$$d_{50} = \frac{0.0082(Q)^{4/3}}{TW(D_o)} \text{ , for } Y = D_o \qquad \text{(Eq. CH6-131)}$$

Where Y = depth of scour hole below culvert invert

The other riprap requirements are as indicated in the previous sections for channel lining.

1.10.4 LOW FLOW GRADE CONTROL STRUCTURES

1.10.4.1 INTRODUCTION

With the advent of floodplain management programs, developers and local governments frequently decided to preserve the floodplain. Since urbanization causes more frequent and sustained flows, the trickle/low flow channel becomes more susceptible to erosion even though the overall floodplain may remain stable and able to resist major flood events.

Erosion of the low flow channel, if left uncontrolled, can cause degradation and destabilizaton of the entire floodplain. Low flow check structures are designed to provide control points and establish stable bed slopes within the base flow channel. The check structures can be small versions of the drop structures described in Chapter 6, Section 6 or in many instances simply control sills across the floodplain. Low flow check structures are not appropriate in instances such as completely incised floodplains or very steep channels.

1.10.4.2 DROP STRUCTURE GRADE CONTROL STRUCTURES

The grouted sloping boulder drop structure and the vertical riprap drop structure designs can be adapted for use as check structures. The analysis steps are the same with the additional consideration of 1) stable bed slope for the unlined trickle or low flow channel and 2) potential overflow erosion during submergence of the check structure and where flow converges back from the main channel sides or below the check structure.

The basic design steps for this type of structure include the following:

a. Determine a stable slope and configuration for the low flow zone. For unlined channels, discharges from full floodplain flow to the dominant discharge should first be considered. The dominant discharge is more fully explained in sediment transport texts such as Simons, Li and Associates (1982).

b. The configuration of the low flow zone, and number and placement of the check structures has to be reviewed. Typically, the floodplain slope is steeper, often on the order of critical conditions. If the checks are widely spaced, the trickle channel depth can be quite deep downstream of the check, leading to concentration of higher flows into the trickle channel and the check. A good rule of thumb is to not have the trickle channel more than 2 feet deep at the crest of the check, or more than 4 feet deep below the check structure (relative to the overbank).

c. A hydraulic analysis should be performed using the discharge that completely fills the check structure at its crest (the primary design flow).

d. The secondary design flow is that flow which causes the worst condition for lateral overflow around the abutments and back into the basin or trickle channel below. The goal is to have the check structure survive such an event with minimal or reasonable damage to the floodplain below. The best approach is to estimate unit discharges, velocities and depths along overflow paths. The unit discharges can be estimated at the crest or critical section for the given total flow. Estimating the overflow path around the check abutment is difficult and requires practical judgment. Slopes can be derived for the anticipated overflow routes and protective measures devised such as grouted rock.

e. Seepage control is also important, as piping and erosion through or around these structures is a frequent problem. It is advisable to provide a cutoff which extends laterally at least 5 to 10 feet into undisturbed bank at minimum and has cutoff depth appropriate to the profile dimensions of the check.

1.10.4.3 CONTROL SILL GRADE CONTROL STRUCTURES

Another type of check structure that can be used to stabilize low flow channels within wide, relatively stable floodplains is the control sill shown in Figure CH6-F123. The sill can be constructed by filling an excavated trench with concrete, if soil conditions are acceptable for trenching, or forming a simple wall if a trench will not work.

The sill crosses the low flow channel and should extend a significant distance into the adjacent floodplain on both sides. The top of the sill conforms to the top of the ground at all points along its length. Riprap or other erosion control methods can then be added as erosion occurs.

The basic design steps are:

a. Determine a stable slope as described above.

b. Determine spacing of the sills based on the difference in slope between the natural and projected stable slope and the amount of future drop to be allowed (not to exceed 3 feet).

1.11 EXAMPLE APPLICATION

1.11.1 EXAMPLE

Problem:

An open channel is to be constructed for Doe Creek downstream of John Boulevard and north of Rose Subdivision. Assume the following conditions for this problem.

Q_{100} = 191 cfs
Invert elevation downstream of John Boulevard = 4,918

Invert elevation downstream of Rose Subdivision = 4,917
Channel improvement length = 900 feet

Due to aesthetics and sufficient right-of-way, a grass-lined channel shall be constructed.

Side Slope = z = 3
Bottom Width = b = 10 feet
n = 0.035 for grass-lined channel

Since the 100-year, 24-hour flow is less than 200 cfs, a trickle channel shall be constructed in the proposed channel bottom.

Solution:

Step 1: Determine the depth of water during a 100-year flow event.

$$\text{Slope} = \frac{4918 - 4917}{900} = 0.0011 \text{ feet/ feet}$$

The Manning Equation can be re-written so that the depth of flow, y, in a trapezoidal channel is on one side of the equation.

$$\frac{(by + zy^2)^{5/3}}{(b + 2y(1 + z^2)^{1/2})^{2/3}} = \left(\frac{Q}{S^{1/2}}\right)\left(\frac{n}{1.49}\right)$$

Solving by trial and error,

Y= 3.7 feet

Step 2: Calculate the water velocity in the proposed channel during a 100-year flow event using the Manning Equation.

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

$$= \left(\frac{1.49}{0.035}\right) * (.0011)^{1/2} * \left(\frac{(10 + 3 * 3.7) * 3.7}{10 + 2 * 3.7 * (1 + 3^2)^{1/2}}\right)^{2/3}$$

$$= 2.5 \text{ fps}$$

Since the water velocity of the proposed channel (2.5 fps) is less than the maximum permissible water velocity in a grass-lined channel, a grass-lined channel can be used at this location.

Step 3: Design the trickle channel.

Assume dimensions for a concrete trickle channel:

Bottom width = 5 feet
Depth = 1 foot
Side Slopes = vertical

The capacity of the trickle channel is:

$$Q = \left(\frac{1.49}{n}\right)(S^{1/2})(R^{2/3})(A)$$

$$Q = \left(\frac{1.49}{n}\right)(S^{1/2})\left(\frac{by}{b+2y}\right)^{2/3}(by)$$

$$Q = \left(\frac{1.49}{0.015}\right) * (.0011)^{1/2} * \left((5*1)/(5+(2*1))\right)^{2/3} * (5*1)$$

$$Q = 13.16 \text{ cfs}$$

Step 4: Verify that trickle channel has sufficient capacity.

The minimum capacity of the trickle channel is:

$$\text{Min. } Q_{T_c} = 0.05 * Q_{100}$$

$$\text{Min. } Q_c = 9.6 \text{ cfs}$$

Since the capacity of the proposed trickle channel (13.2 cfs) is greater than the required capacity (9.6 cfs), the proposed trickle channel is adequate.

Step 5: Determine the freeboard required for the proposed channel.

$$F_b = 0.5 + \frac{V^2}{2g}$$

$$F_b = 0.5 + \frac{(2.5)^2}{2 * 32.2} = 0.6 \text{ feet},$$

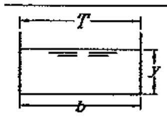
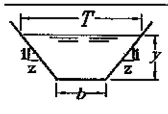
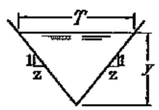
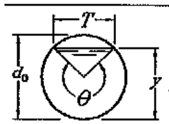
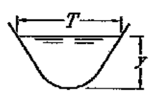
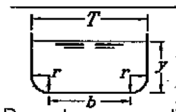
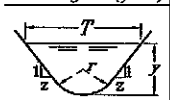
but minimum = 1.0 feet

Therefore use F_b = 1.0 feet.

Step 6: The cross-section of the proposed channel is shown in Figure CH6-F124.

O:/2080/FIGURES, CH6-F124.DWG, 4/10/02 MAS

GEOMETRIC ELEMENTS OF CHANNEL SECTIONS

SECTION	AREA, A	WETTED PERIMETER, P	HYDRAULIC RADIUS, R	TOP WIDTH, T	HYDRAULIC DEPTH, D	SECTION FACTOR, Z
 Rectangle	by	$b+2y$	$\frac{by}{b+2y}$	b	y	$by^{1.5}$
 Trapezoid	$(b+zy)y$	$b+2y\sqrt{1+z^2}$	$\frac{(b+zy)y}{b+2y\sqrt{1+z^2}}$	$b+2zy$	$\frac{(b+zy)y}{b+2zy}$	$\frac{[(b+zy)y]^{1.5}}{\sqrt{b+2zy}}$
 Triangle	zy^2	$2y\sqrt{1+z^2}$	$\frac{zy}{2\sqrt{1+z^2}}$	$2zy$	$\frac{y}{2}$	$\frac{\sqrt{2}}{2}zy^{2.5}$
 Circle	$\frac{1}{8}(\theta-\sin\theta)d_0^2$	$\frac{1}{2}\theta d_0$	$\frac{1}{4}\left(1-\frac{\sin\theta}{\theta}\right)d_0$	$(\sin\frac{1}{2}\theta)d_0$ or $2\sqrt{y(d_0-y)}$	$\frac{1}{8}\left(\frac{\theta-\sin\theta}{\sin\frac{1}{2}\theta}\right)d_0$	$\frac{\sqrt{2}}{32}\frac{(\theta-\sin\theta)^{1.5}}{(\sin\frac{1}{2}\theta)^{0.5}}d_0^{2.5}$
 Parabola	$\frac{2}{3}Ty$	$T+\frac{8}{3}\frac{y^2}{T}$ *	$\frac{2T^2y}{3T^2+8y^2}$ *	$\frac{3}{2}\frac{A}{y}$	$\frac{2y}{3}$	$\frac{2}{9}\sqrt{6}\,Ty^{1.5}$
 Round-cornered rectangle (y>r)	$\left(\frac{\pi}{2}-2\right)r^2+(b+2r)y$	$(\pi-2)r+b+2y$	$\frac{(\pi/2-2)r^2+(b+2r)y}{(\pi-2)r+b+2y}$	$b+2r$	$\frac{(\pi/2-2)r^2}{b+2r}+y$	$\frac{[(\pi/2-2)r^2+(b+2r)y]^{1.5}}{\sqrt{b+2r}}$
 Round-bottom triangle	$\frac{T^2}{4z}-\frac{r^2}{z}(1-z\cot^{-1}z)$	$\frac{T}{z}\sqrt{1+z^2}-\frac{2r}{z}(1-z\cot^{-1}z)$	$\frac{A}{P}$	$2[z(y-r)+r\sqrt{1+z^2}]$	$\frac{A}{T}$	$A\sqrt{\frac{A}{T}}$

*Satisfactory approximation for the interval $0<x\leq 1$, where $x=4y/T$. When $x>1$, use the exact expression $P=(T/2)[\sqrt{1+x^2}+1/x\ \ln(x+\sqrt{1+x^2})]$

WRC ENGINEERING, INC

VERSION: AUGUST 2002

REFERENCE:
Chow, V.T. Open-Channel Hydraulics, McGraw Hill Book Company, 1959.

TABLE CH6-T101
GEOMETRIC ELEMENTS OF CHANNEL SECTIONS

COLORADO STATEWIDE
DRAINAGE AND FLOODPLAIN MANAGEMENT CRITERIA MANUAL

TYPICAL ROUGHNESS COEFFICIENTS FOR OPEN CHANNELS

TYPE OF CHANNEL AND DESCRIPTION	MINIMUM	NORMAL	MAXIMUM
EXCAVATED OR DREDGED			
a. Earth, straight and uniform			
1. Clean, recently completed	0.016	0.018	0.020
2. Clean, after weathering	0.018	0.022	0.025
3. Gravel, uniform section, clean	0.022	0.025	0.030
4. With short grass, few weeds	0.022	0.027	0.033
b. Earth, winding and sluggish			
1. No vegetation	0.023	0.025	0.030
2. Grass, some weeds	0.025	0.030	0.033
3. Dense weeds or aquatic plans in deep channels	0.030	0.035	0.040
4. Earth bottom and rubble sides	0.028	0.030	0.035
5. Stony bottom and weedy banks	0.025	0.035	0.040
6. Cobble bottom and clean sides	0.030	0.040	0.050
c. Dragline-excavated or dredged			
1. No vegetation	0.025	0.028	0.033
2. Light brush on banks	0.035	0.050	0.060
d. Rock cuts			
1. Smooth and uniform	0.025	0.035	0.040
2. Jagged and irregular	0.035	0.040	0.050
e. Channels not maintained, weeds and brush			
1. Dense weeds, high as flow depth	0.050	0.080	0.120
2. Clean bottom, brush on sides	0.040	0.050	0.080
3. Same as above, but highest state of flow	0.045	0.070	0.110
4. Dense brush, high state	0.080	0.100	0.140

TYPICAL ROUGHNESS COEFFICIENTS FOR OPEN CHANNELS

TYPE OF CHANNEL AND DESCRIPTION	MINIMUM	NORMAL	MAXIMUM
NATURAL STREAMS			
Minor Streams (top width at flood stage <100 ft)			
a. Streams on plain			
1. Clean, straight, full stage, no rifts or deep pools	0.025	0.030	0.033
2. Same as above, but more stones and weeds	0.030	0.035	0.040
3. Clean, winding, some pools and shoals	0.033	0.040	0.045
4. Same as above, but some weeds and stones	0.035	0.045	0.050
5. Same as above, but lower stages, and more ineffective slopes and sections	0.040	0.048	0.055
6. Same as 4, but more stones	0.045	0.050	0.060
7. Sluggish reaches, weedy, deep pools	0.050	0.070	0.080
8. Very weedy reaches, deep pools, or floodways with heavy stand of timber and underbrush	0.075	0.100	0.150
b. Mountain streams, no vegetation in channel, banks usually steep, trees and brush along banks submerged at high stages			
1. Bottom: gravel, cobbles, and few boulders	0.030	0.040	0.050
2. Bottom: cobbles with large boulders	0.040	0.050	0.070
Floodplains			
a. Pasture, no brush			
1. Short grass	0.025	0.030	0.035
2. High grass	0.030	0.035	0.050
b. Cultivated areas			
1. No crop	0.020	0.030	0.040
2. Mature row crops	0.025	0.035	0.045
3. Mature field crops	0.030	0.040	0.050
c. Brush			
1. Scattered brush, heavy weeds	0.035	0.050	0.070
2. Light brush and trees, in winter	0.035	0.050	0.060
3. Light brush and trees, in summer	0.040	0.060	0.080
4. Medium to dense brush, in winter	0.045	0.070	0.110
5. Medium to dense brush, in summer	0.070	0.100	0.160
d. Trees			
1. Dense willows, summer, straight	0.110	0.105	0.200
2. Cleared land with tree stumps, no sprouts	0.030	0.040	0.050
3. Same as above, but with heavy growth of sprouts	0.050	0.060	0.080
4. Heavy stand of timber, a few down trees, little undergrowth, flood stage below branches	0.080	0.100	0.120
5. Same as above, but with flood stage reaching branches	0.100	0.120	0.160
Major streams (top width at flood state 100 ft). The n value is less than that for minor streams of similar description, because banks offer less effective resistance.			
a. Regular section with no boulders or brush	0.025	--	0.060
b. Irregular and rough section	0.035	--	0.100

TYPICAL ROUGHNESS COEFFICIENTS FOR OPEN CHANNELS

TYPE OF CHANNEL AND DESCRIPTION	MINIMUM	NORMAL	MAXIMUM
LINED OR BUILT-UP CHANNELS			
a. Concrete			
1. Trowel finish	0.011	0.013	0.015
2. Float finish	0.013	0.015	0.016
3. Gunite, good section	0.016	0.019	0.023
4. Gunite, wavy section	0.018	0.022	0.023
b. Concrete bottom float finished with side of			
1. Dressed stone in mortar	0.015	0.017	0.020
2. Random stone in mortar	0.017	0.020	0.024
3. Dry rubble or riprap	0.020	0.030	0.035
c. Gravel bottom with sides of			
1. Formed concrete	0.017	0.020	0.025
2. Random stone in mortar	0.020	0.023	0.026
3. Dry rubble or riprap	0.023	0.033	0.036
d. Asphalt			
1. Smooth	0.013	0.013	--
2. Rough	0.016	0.016	--
e. Grassed	0.030	0.040	0.050

MAXIMUM PERMISSIBLE MEAN CHANNEL VELOCITY

MATERIAL / LINING	MAXIMUM PERMISSIBLE MEAN VELOCITY (fps)
NATURAL & IMPROVED UNLINED CHANNELS	
Erosive Soils:	
Loams, Sands, Noncolloidal Silts	3.0
Less Erosive Soils:	
Clays, Shales, Cobbles, Gravel	5.0
FULLY LINED CHANNELS	
Unreinforced Vegetation	5.5
Loose Riprap	10.0
Grouted Riprap	15.0
Gibbons	15.0
Soil-Cement	15.0
Concrete	35.0

NOTES:

1. For composite lined channels, use the lowest of the maximum mean velocities for the materials used in the composite lining.
2. Deviations from the above values are only allowed with appropriate engineering analysis and/or suitable agreements for maintenance responsibilities.
3. Maximum permissible velocities based upon non-clear water conditions.

CLASSIFICATION AND GRADATION OF LOOSE RIPRAP

RIPRAP CLASS DESIGNATION	% SMALLER THAN GIVEN SIZE BY WEIGHT	RIPRAP GRADATION (Inches)	d_{50}* (Inches)
Class 150	100	10	6**
	35 - 50	6	
	0 - 15	2	
Class 300	100	20	12
	35 - 50	12	
	0 - 15	4	
Class 400	100	26	16
	35 - 50	16	
	0 - 15	6	
Class 550	100	37	22
	35 - 50	22	
	0 - 15	8	
Class 700	100	45	28
	35 - 50	28	
	0 - 15	10	
Class 900	100	57	35
	35 - 50	35	
	0 - 15	14	

*d_{50} = mean stone size

** Bury Class 150 riprap with native top soil and re-vegetate to protect from vandalism

GRADATION FOR GRANULAR RIPRAP BEDDING

RIPRAP DESIGNATION	GRANULAR BEDDING SIEVE SIZE (MM)	GRANULAR BEDDING PERCENT PASSING BY WEIGHT
Class 150	37.5	100
	19	35 - 100
	12.5	15 - 80
	9.5	5 - 60
	4.75	0 - 35
	1.18	0 - 5
Class 300	100	100
	37.5	30 - 100
	25	15 - 80
	12.5	0 - 50
	4.75	0 - 20
	2.36	0 - 5
Class 400	125	100
	50	30 - 100
	37.5	20 - 80
	19	0 - 45
	6.3	0 - 20
	4.75	0 - 10
Class 550	150	100
	75	35 - 100
	50	15 - 80
	25	0 - 50
	12.5	0 - 30
	6.3	0 - 10
Class 700	200	100
	75	25 - 85
	50	5 - 70
	19	0 - 40
	9.5	0 - 15
	6.3	0 - 5
Class 900	250	100
	100	25 - 90
	75	15 - 75
	25	0 - 35
	12.5	0 - 15
	6.3	0 - 5

CLASSIFICATION AND GRADATION OF ROCK FOR GROUTED RIPRAP

RIPRAP DESIGNATION	% SMALLER THAN GIVEN SIZE BY WEIGHT	INTERMEDIATE ROCK DIMENSION (Inches)
Class 400	100	26
	35 - 50	16
	0 - 5	12
Class 550	100	37
	35 - 50	22
	0 - 5	16
Class 700	100	45
	35 - 50	28
	0 - 5	20
Class 900	100	57
	35 - 50	35
	0 - 5	28

REFERENCE:
Draft State of Nevada, Department of Transportation, Standard Specifications for Road and Bridge Construction, 1996

TABLE CH6-T106
CLASSIFICATIONS AND GRADATION OF ROCK FOR GROUTED RIPRAP

DESIGN D_{50} VALUES

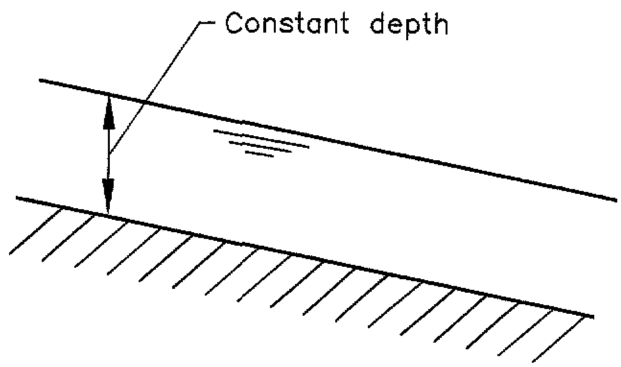
D_{50} DETERMINED FROM DESIGN CURVE (FT)	MINIMUM DESIGN D_{50} (FT)
< 0.25	0.25
0.26 - 0050	0.50
0.51 - 0.75	0.75
0.76 - 1.00	1.00
1.01 - 1.25	1.25
1.26 - 1.50	1.50
1.51 - 1.75	1.75
1.76 - 2.00	2.00
2.01 - 2.25	2.25
2.26 - 2.50	2.50
2.51 - 2.75	2.75
2.76 - 3.00	3.00

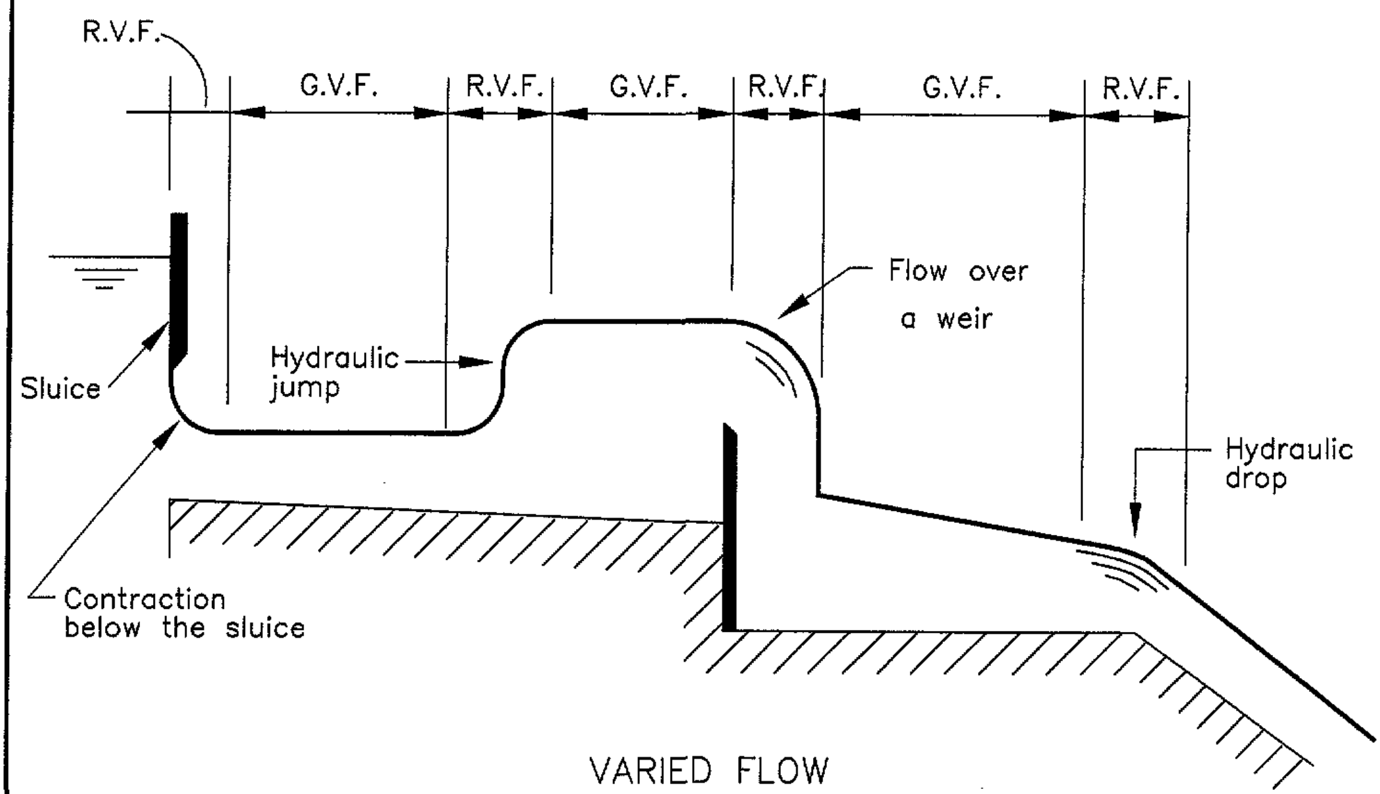
RIPRAP GRADATION FOR STEEP SLOPES

$$\frac{D_{MAX}}{D_{50}} = 1.25$$

$$\frac{D_{50}}{D_{20}} = 2.0$$

$$\frac{D_{50}}{D_{10}} = 3.0$$





G.V.F. – Gradually Varying Flow
R.V.F. – Rapidly Varying Flow

O:/2080/FIGURES, CH4-F101.DWG, 4/5/02 MAS

COLORADO STATEWIDE
DRAINAGE AND FLOODPLAIN MANAGEMENT CRITERIA MANUAL

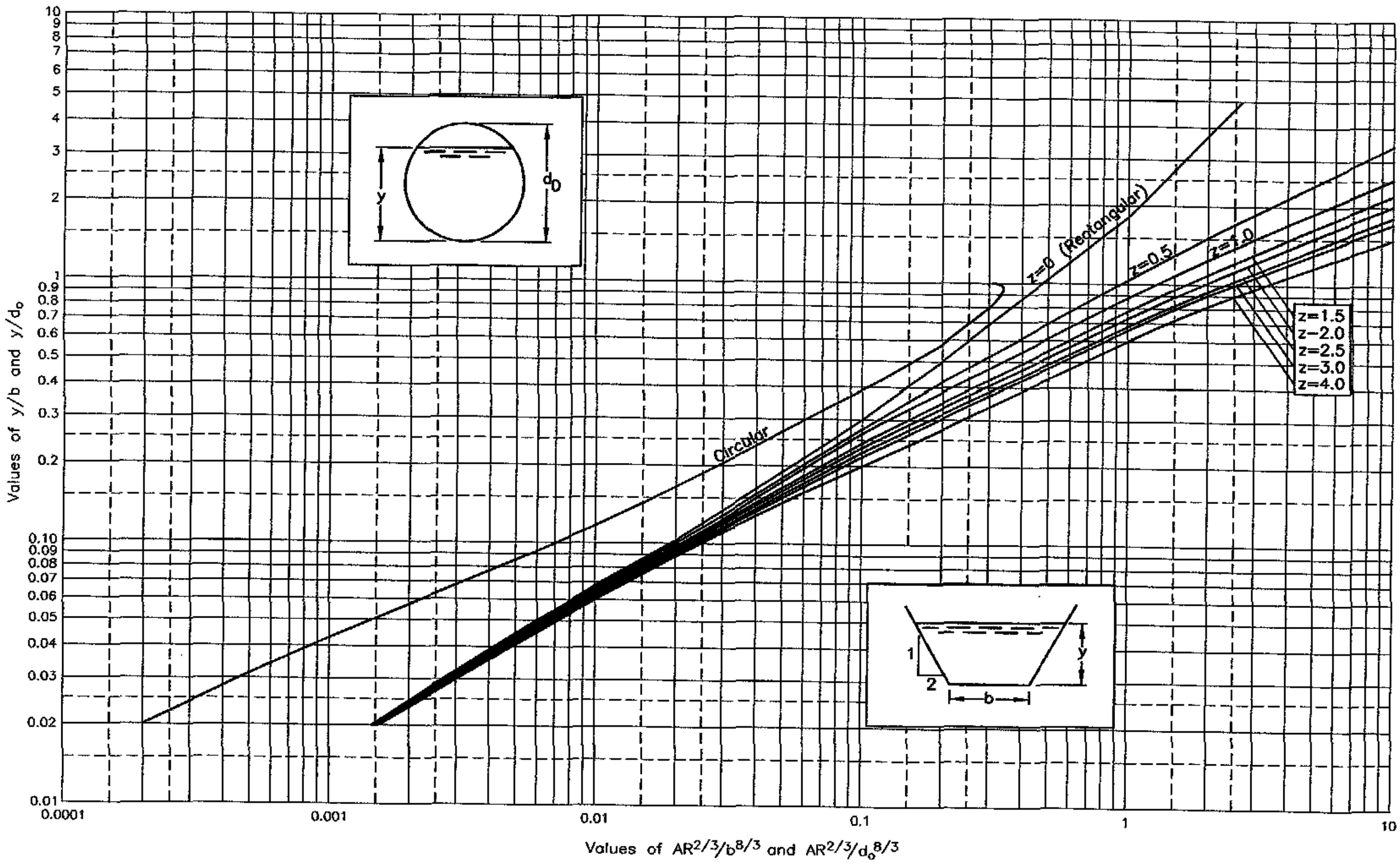
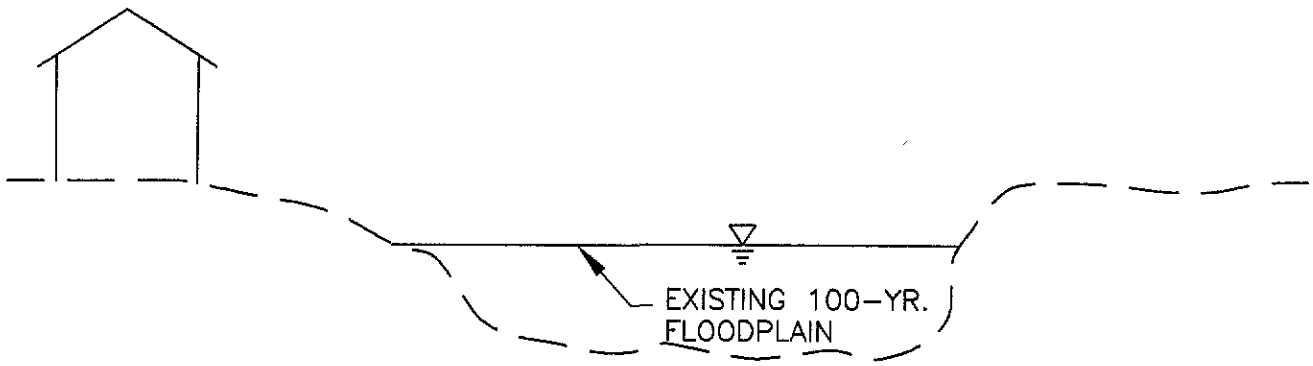


FIGURE CH6-F102
CRITICAL DEPTH FOR TRAPEZOIDAL AND CIRCULAR SECTIONS

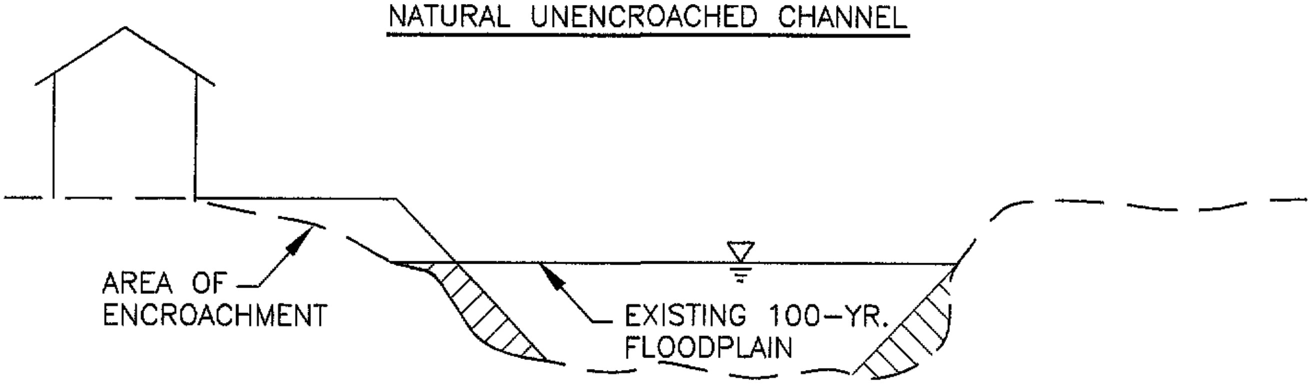
REFERENCE:
Chow, V.T., Open-Channel Hydraulics
McGraw Hill Book Company 1959

VERSION: AUGUST 2002

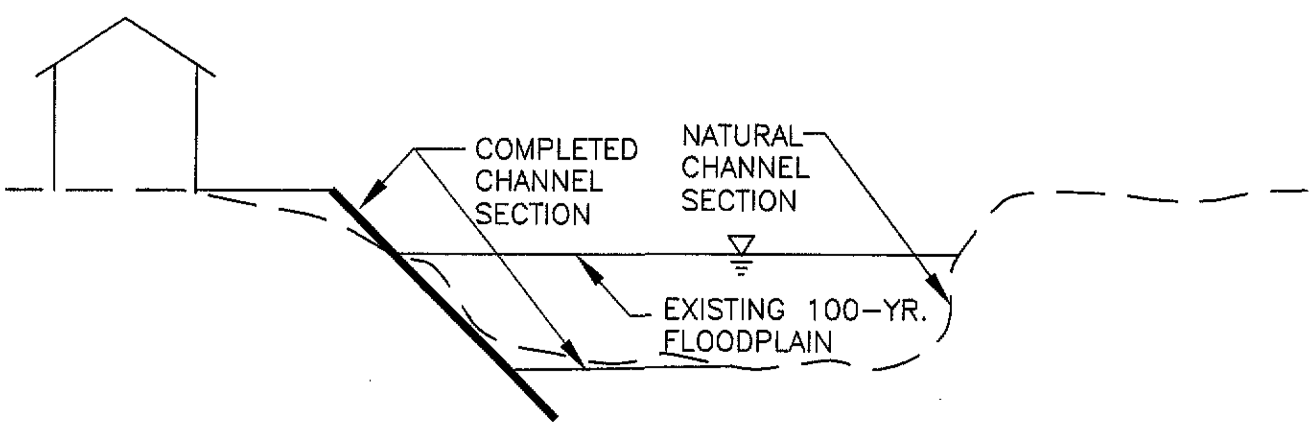
WRC ENGINEERING, INC



NATURAL UNENCROACHED CHANNEL



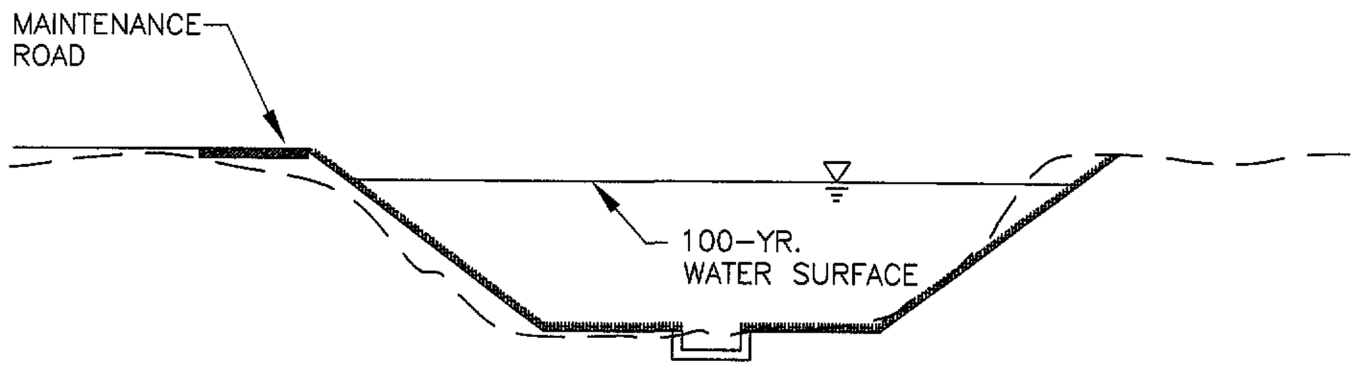
NATURAL ENCROACHED CHANNEL



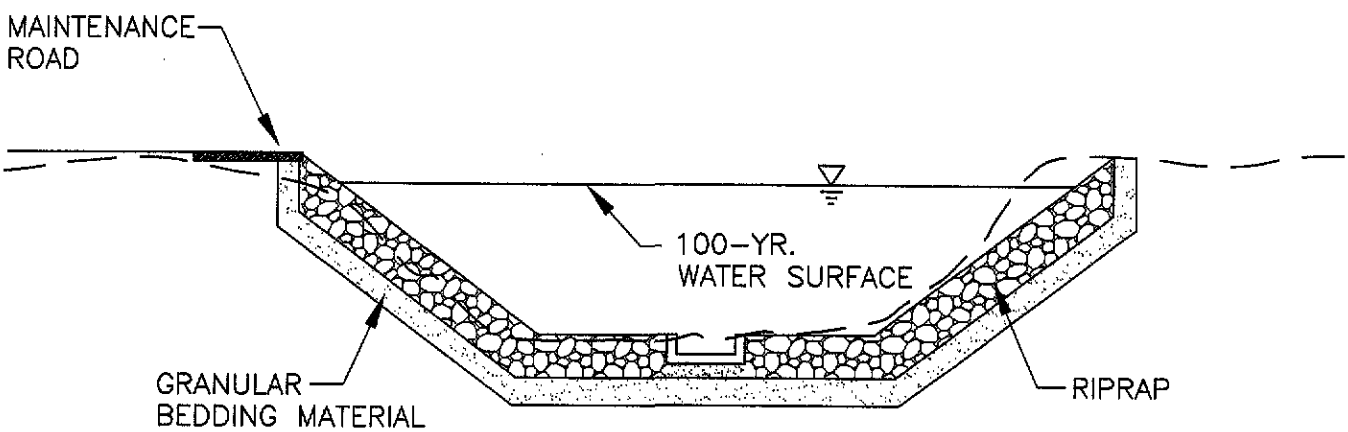
BANK LINED AND TEMPORARY UNLINED CHANNEL

COLORADO STATEWIDE

DRAINAGE AND FLOODPLAIN MANAGEMENT CRITERIA MANUAL



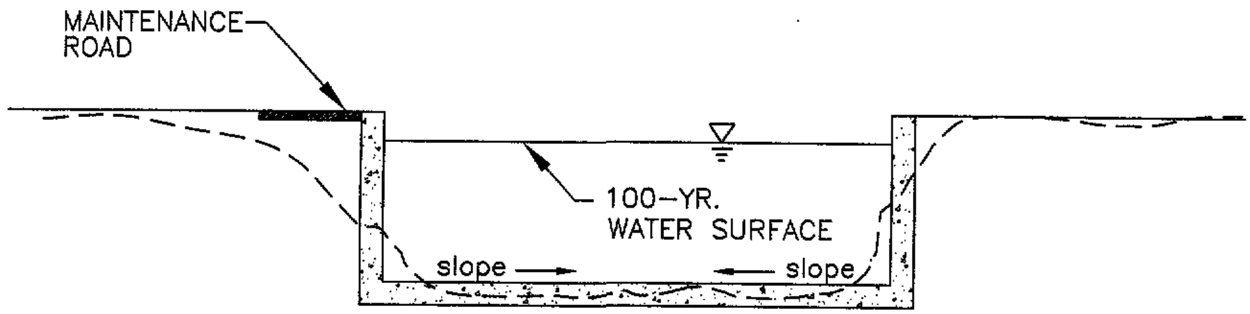
UNLINED AND GRASS–LINED CHANNEL



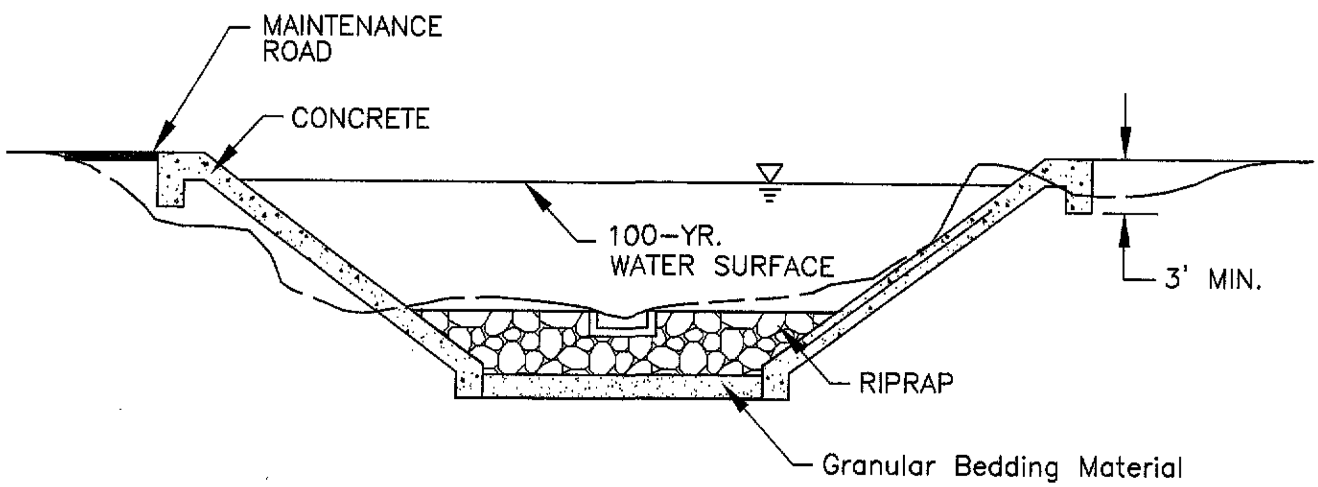
RIPRAP–LINED CHANNEL

O:/2080/FIGURES. CH6-F104A.DWG. 4/8/02 MAS

VERSION: AUGUST 2002	REFERENCE:	FIGURE CH6-F104A
WRC ENGINEERING, INC.		TYPICAL OPEN-CHANNEL DESIGN SECTIONS (IMPROVED CHANNELS)

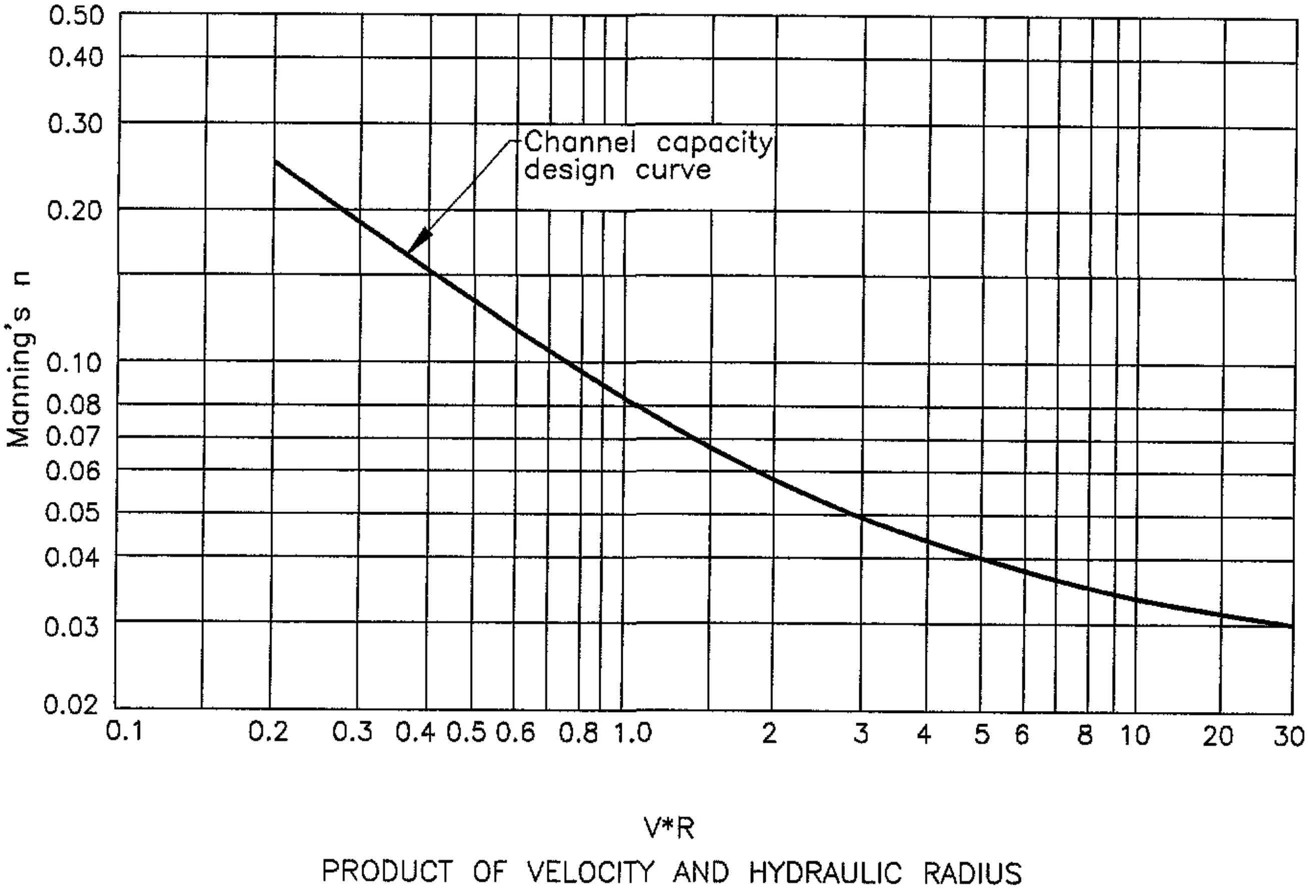


CONCRETE-LINED CHANNEL



COMPOSITE-LINED CHANNEL

NOTE:
REFER TO SECTION 805.7 FOR DISCUSSION OF DESIGN CONSIDERATIONS.



From "Handbook of Channel Design For Soil and Water Conservation," U.S. Department of Agriculture, Soil Conservation Service, No. SCS–TP–61 March, 1947, Rev. June, 1954

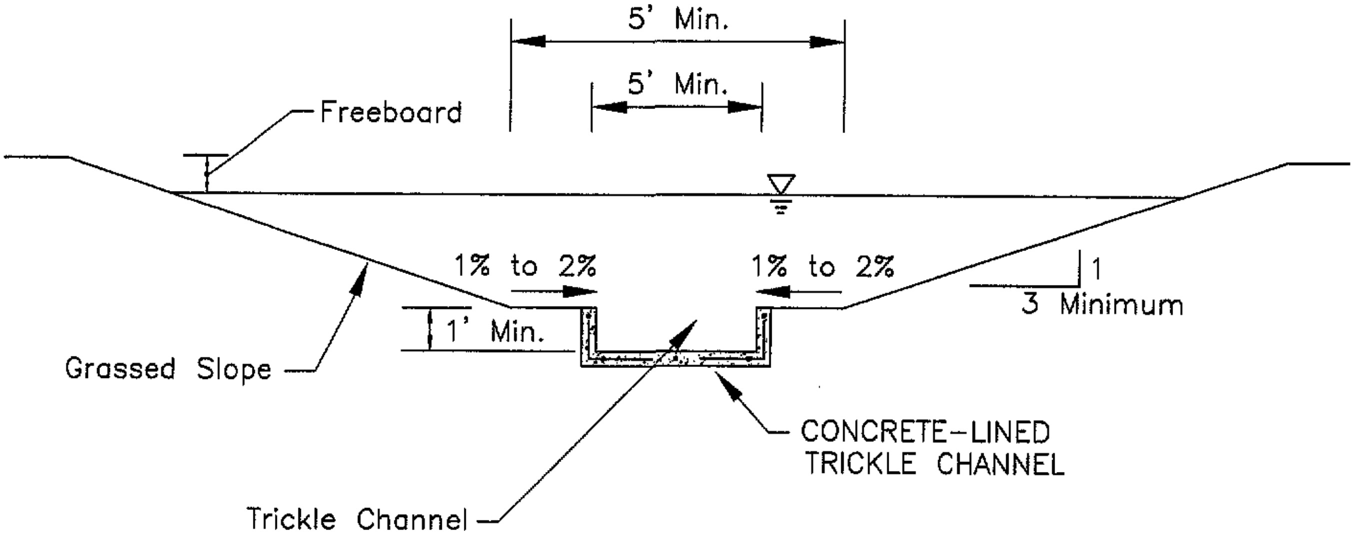


FIGURE CH6-F106
TYPICAL CROSS-SECTION OF
CONCRETE-LINED TRICKLE CHANNEL

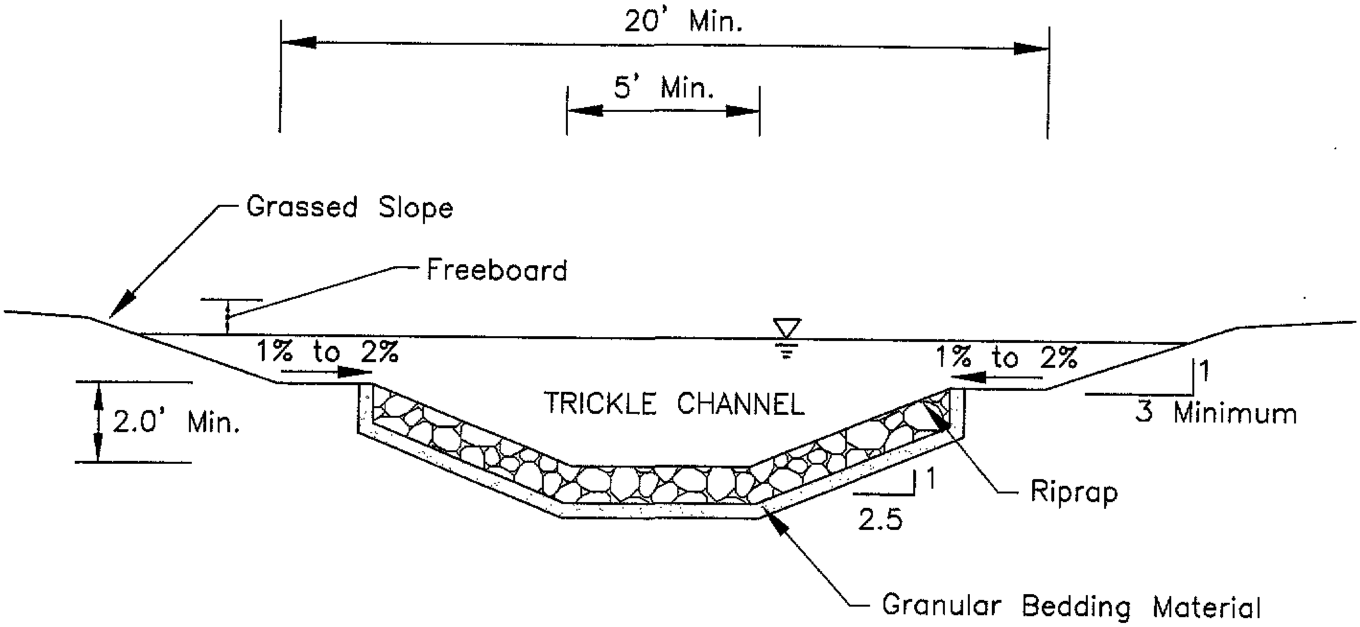
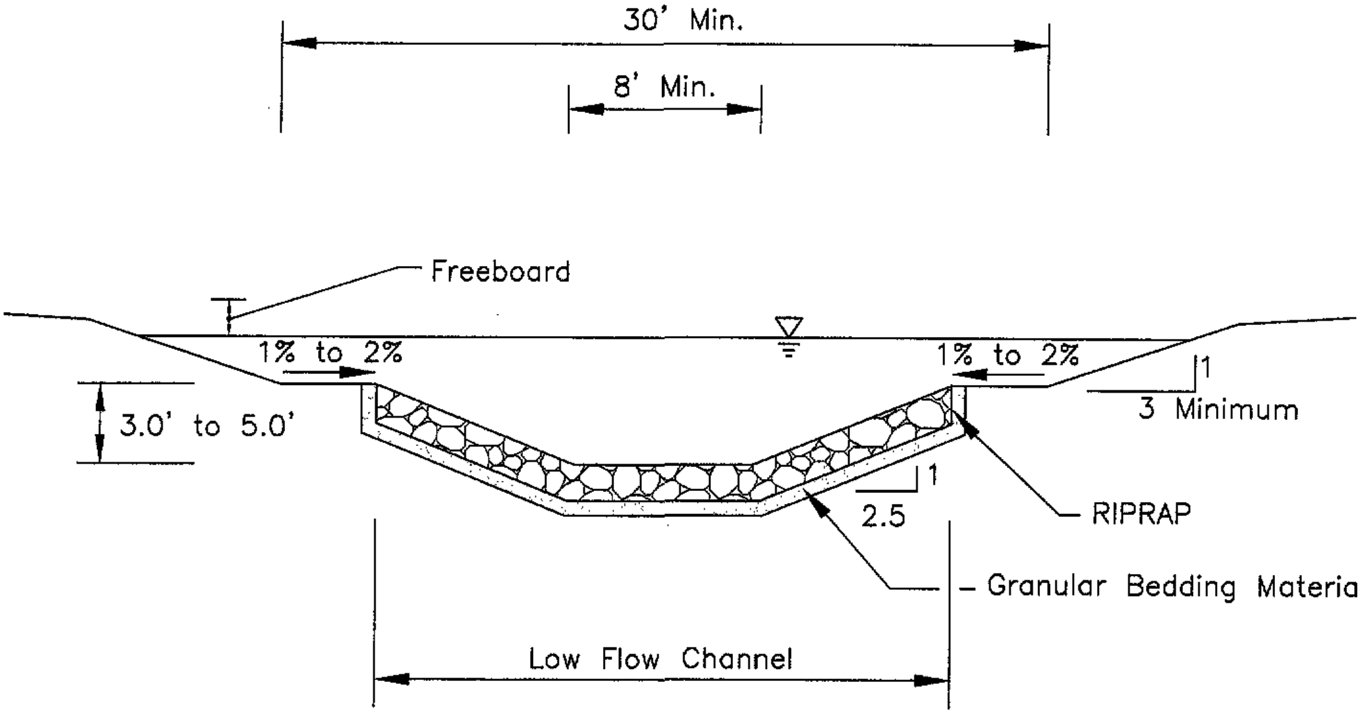
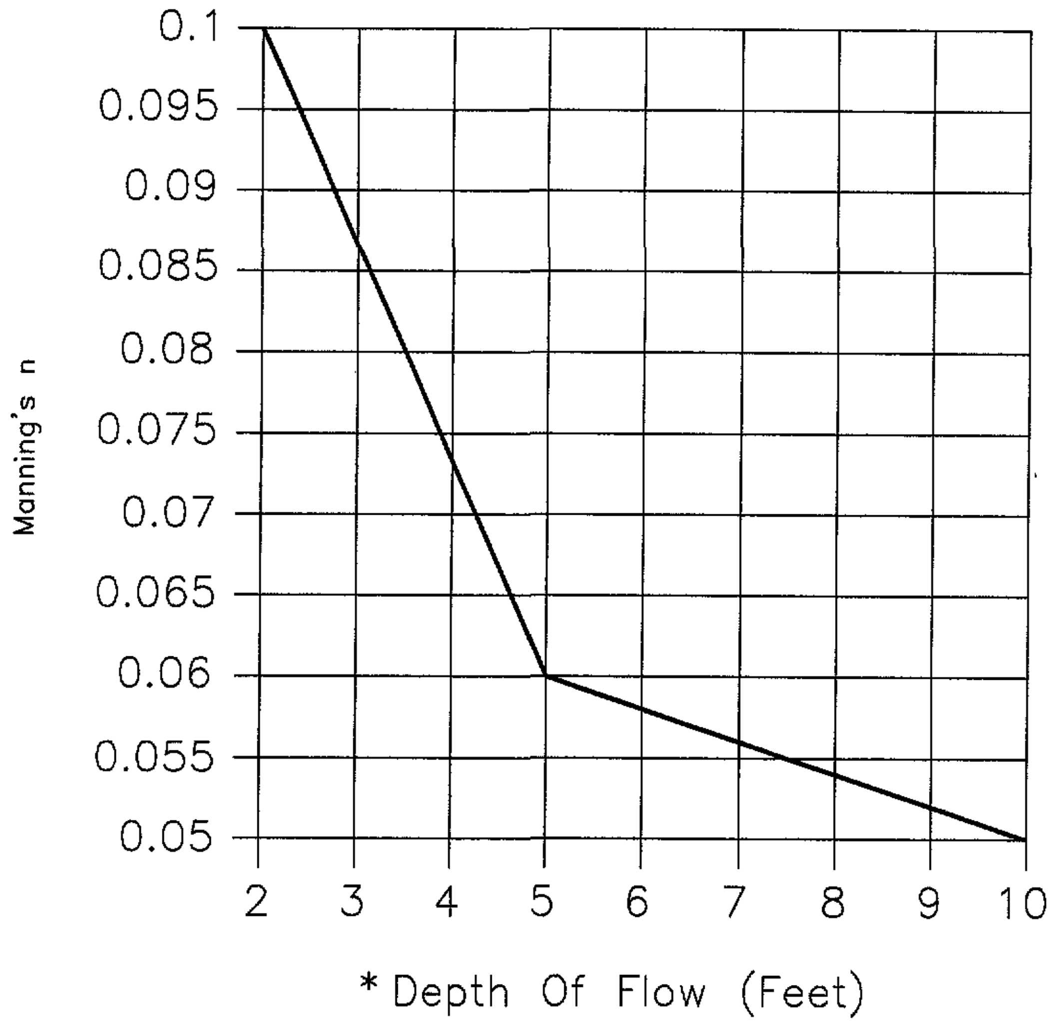


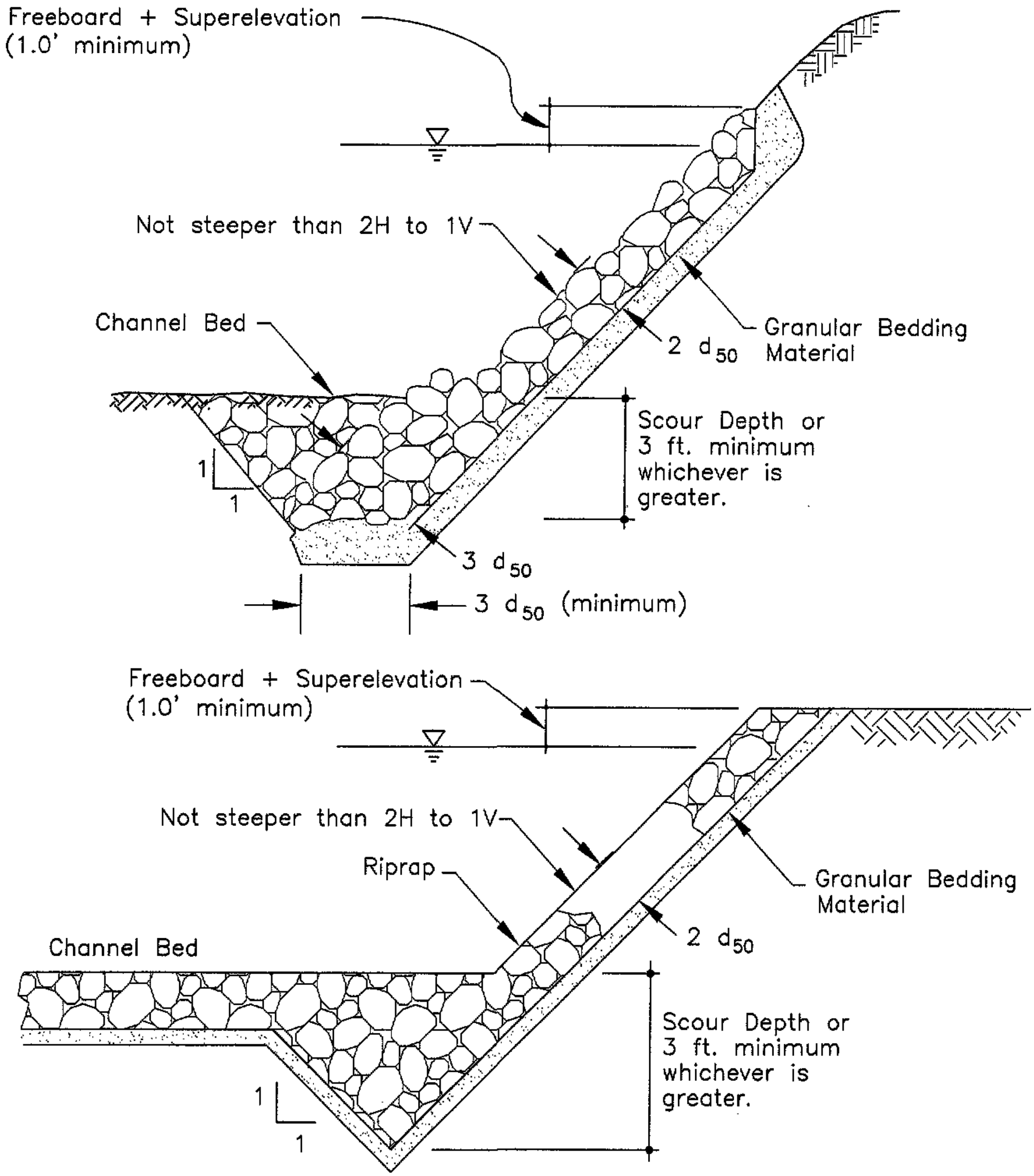
FIGURE CH6-F107
TYPICAL CROSS-SECTION OF RIPRAP-LINED TRICKLE CHANNEL



VERSION: AUGUST 2002	REFERENCE:	FIGURE CH6-F108
WRC ENGINEERING, INC		TYPICAL CROSS-SECTION OF LOW-FLOW CHANNEL

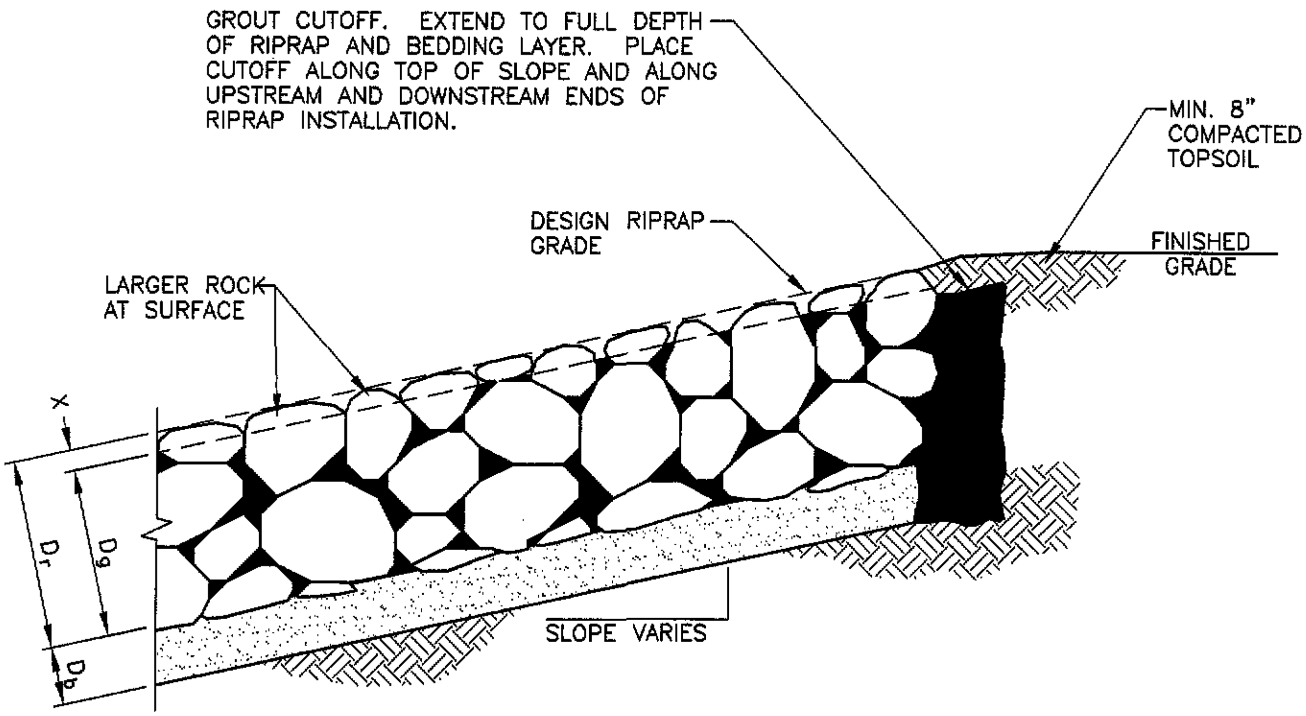


* Use normal depth, ignoring all backwater effects



c:/2080/FIGURES, CH6-F110.DWG, 4/9/02 MAS

VERSION: AUGUST 2002	REFERENCE: Modified from H.E.C. No. 11, USDOT, 1989	**FIGURE CH6-F110** TYPICAL CROSS-SECTIONS FOR RIPRAP-LINED CHANNELS
WRC ENGINEERING, INC.		



LEGEND:

D_b = DEPTH OF BEDDING MATERIAL

D_g = DEPTH OF GROUT LAYER

D_r = DEPTH OF RIPRAP LAYER

X = DEPTH FROM RIPRAP SURFACE TO GROUT SURFACE

NOTES:

1. FINAL PLACEMENT OF RIPRAP TO BE APPROVED BY ENGINEER PRIOR TO GROUTING.
2. BEFORE GROUTING, CLEAN ALL DIRT AND MATERIALS FROM ROCK THAT COULD PREVENT THE GROUT FROM BONDING TO ROCK.
3. PLACE GROUT BY INJECTION METHODS AND USE A PENCIL VIBRATOR TO FILL VOIDS TO THE SPECIFIED GROUT DEPTH. CLEAN EXCESS GROUT FROM ALL EXPOSED SURFACES. PROVIDE A BROOM FINISH FOR GROUT SURFACE.
4. THE CONTRACTOR SHALL CONTROL GROUT MIX AND PLACEMENT PROCEDURES TO ACHIEVE THE SPECIFIED THICKNESS, PENETRATION AND GRADE OF THE GROUT LAYER.

O:/2080/FIGURES, CH6-F112.DWG, 4/9/02 MAS

WRC ENGINEERING, INC

VERSION: AUGUST 2002

REFERENCE:
Simons, Li and Assoc., 1989

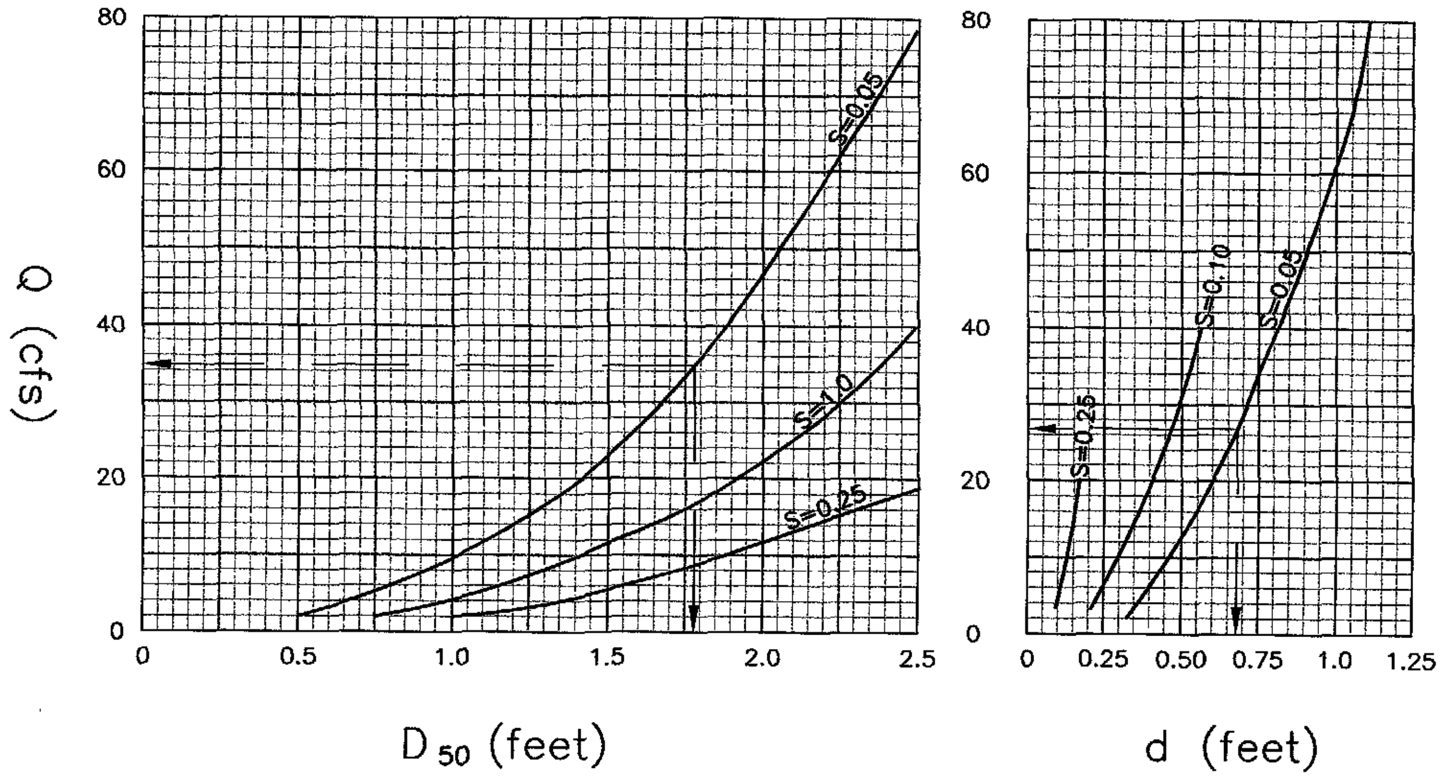
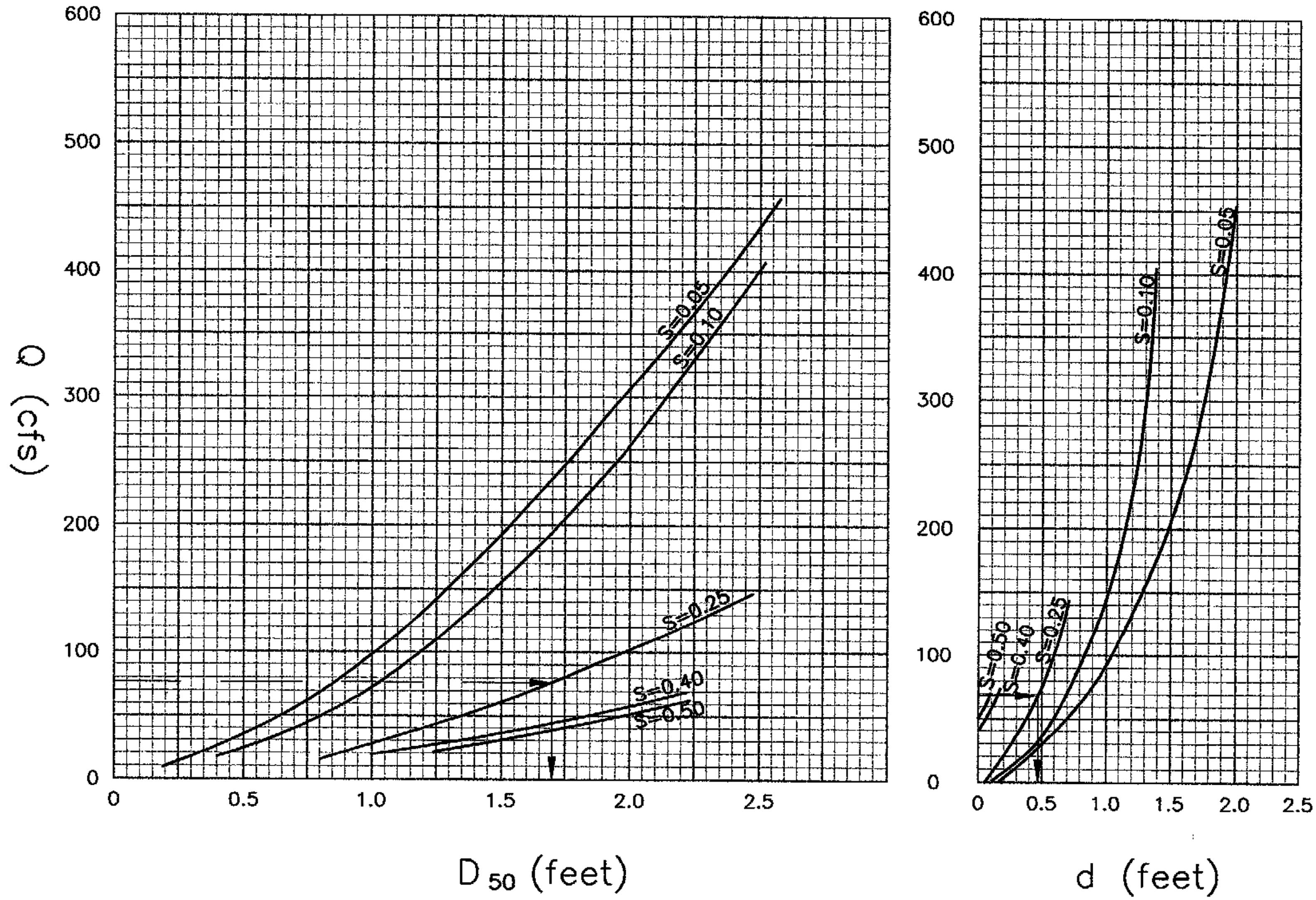


FIGURE CH6-F112
STEEP SLOPE RIPRAP DESIGN, TRIANGULAR CHANNEL, 2:1 SIDE SLOPES

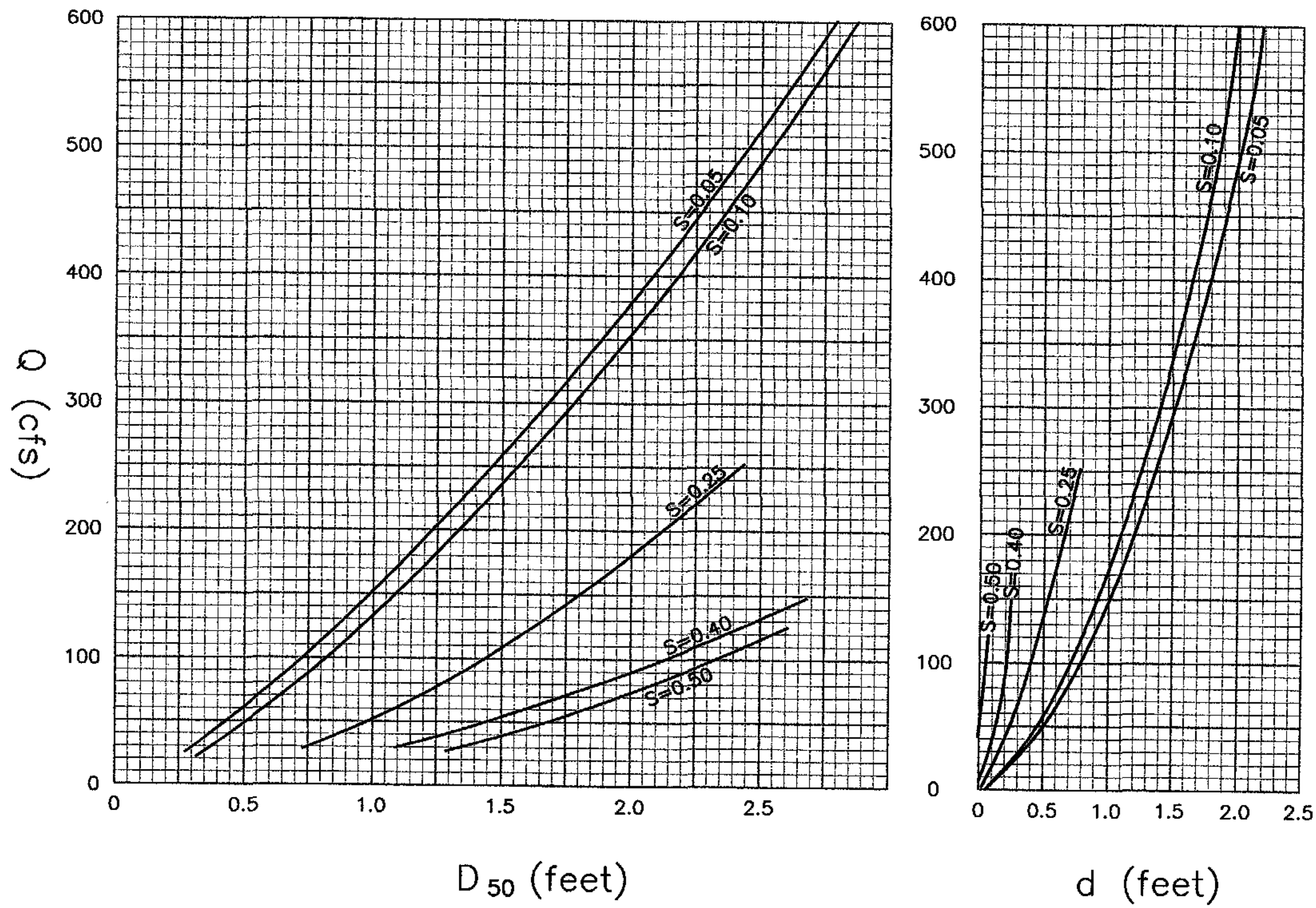


WRC ENGINEERING, INC

VERSION: AUGUST 2002

REFERENCE:
Simons, Li and Assoc., 1989

FIGURE CH6-F113
STEEP SLOPE RIPRAP DESIGN, TRAPEZOIDAL CHANNEL, 2:1 SIDE SLOPES, 6-FOOT BOTTOM WIDTH



WRC ENGINEERING, INC

VERSION: AUGUST 2002

REFERENCE:
Simons, Li and Assoc., 1989

FIGURE CH6-F114
STEEP SLOPE RIPRAP DESIGN, TRAPEZOIDAL CHANNEL, 2:1 SIDE SLOPES, 10-FOOT BOTTOM WIDTH

O:/2080/FIGURES, CH6-F115.DWG, 4/9/02 MAS

COLORADO STATEWIDE
DRAINAGE AND FLOODPLAIN MANAGEMENT CRITERIA MANUAL

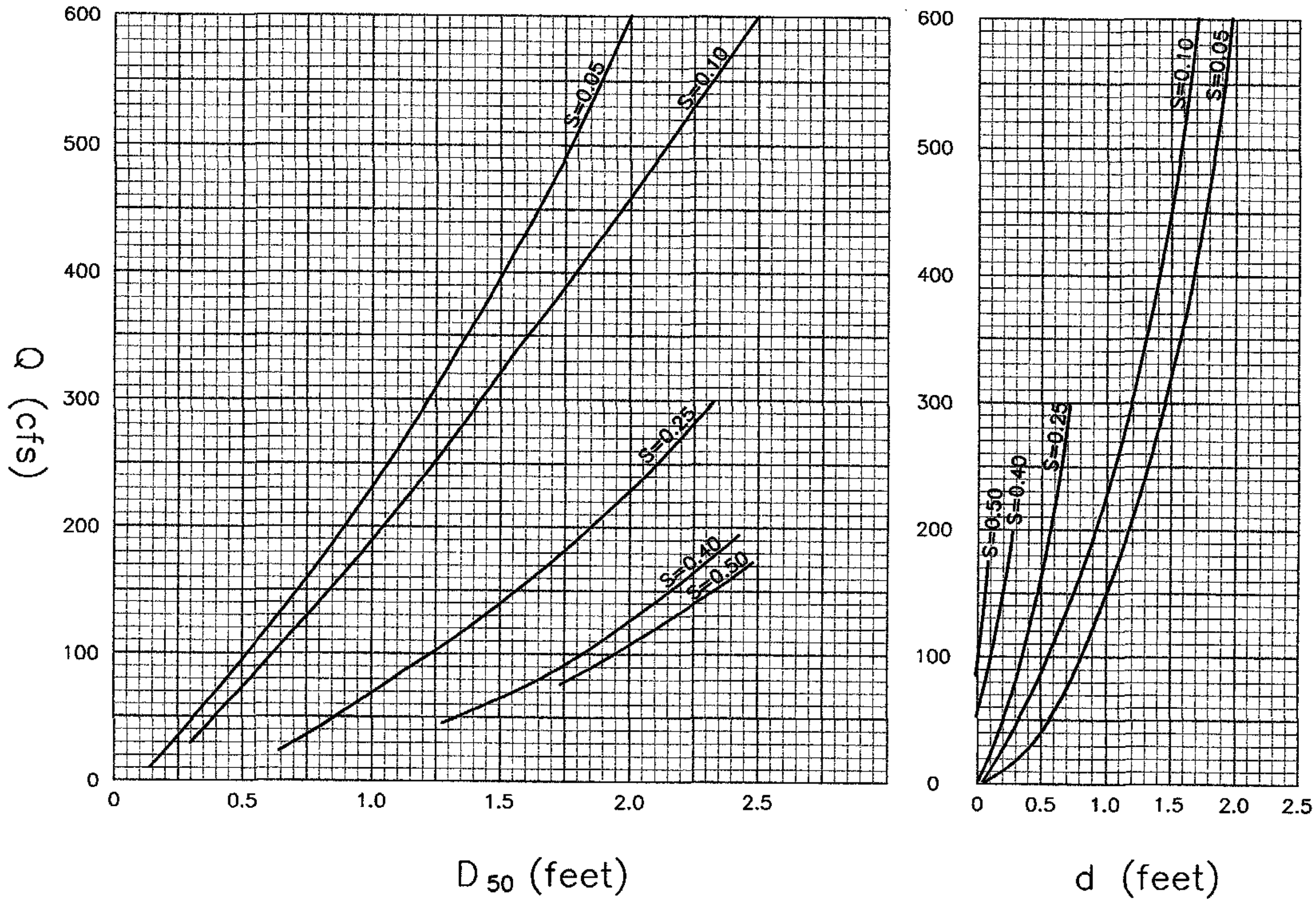


FIGURE CH6-F115
STEEP SLOPE RIPRAP DESIGN, TRAPEZOIDAL CHANNEL, 2:1 SIDE SLOPES, 14-FOOT BOTTOM WIDTH

REFERENCE:
Simons, Li and Assoc., 1989

VERSION: AUGUST 2002

WRC ENGINEERING, INC

O:/2080/FIGURES, CH6-F116.DWG, 4/9/02 MAS

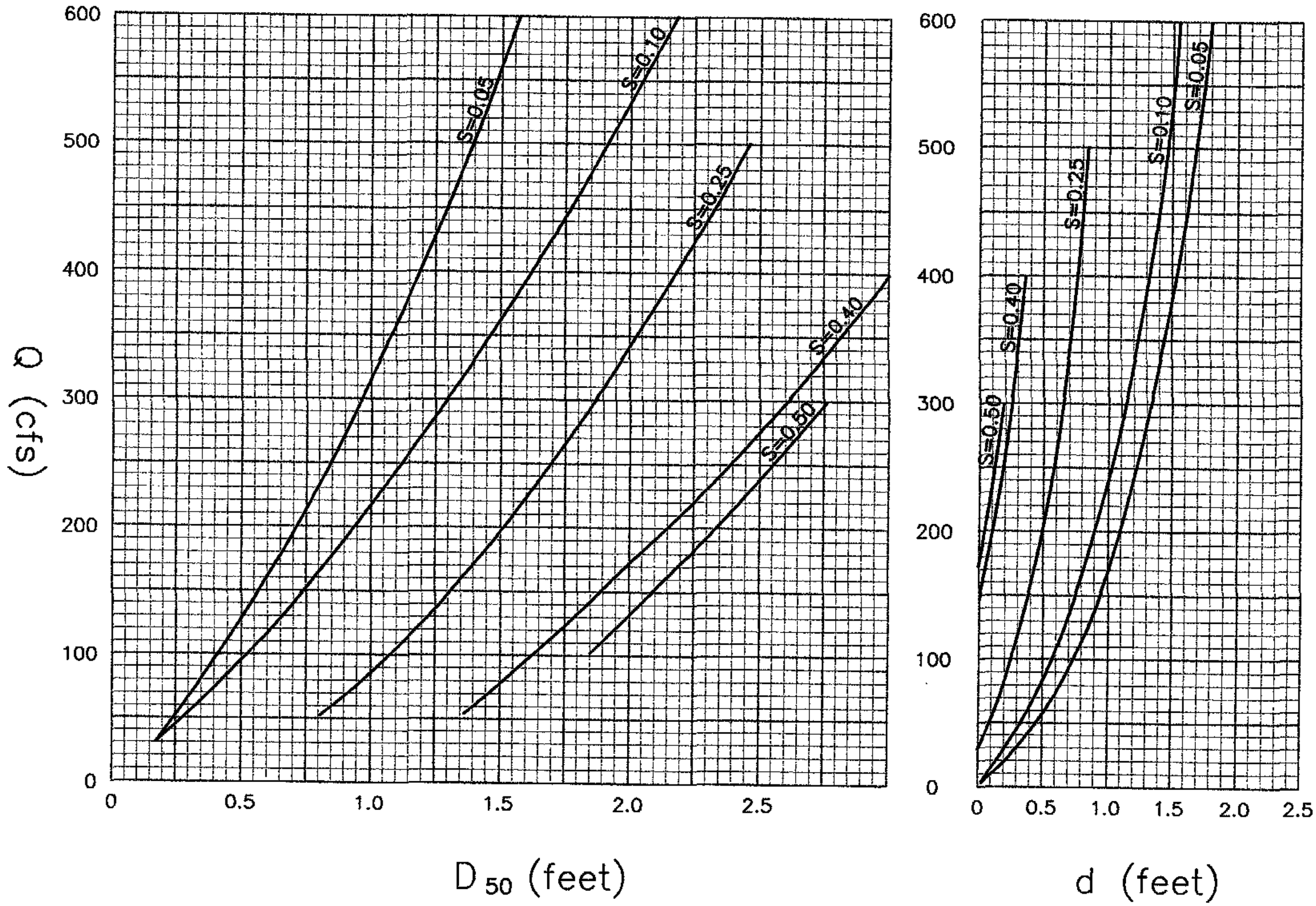


FIGURE CH6-F116

STEEP SLOPE RIPRAP DESIGN, TRAPEZOIDAL CHANNEL, 2:1SIDE SLOPES, 20-FOOT BOTTOM WIDTH

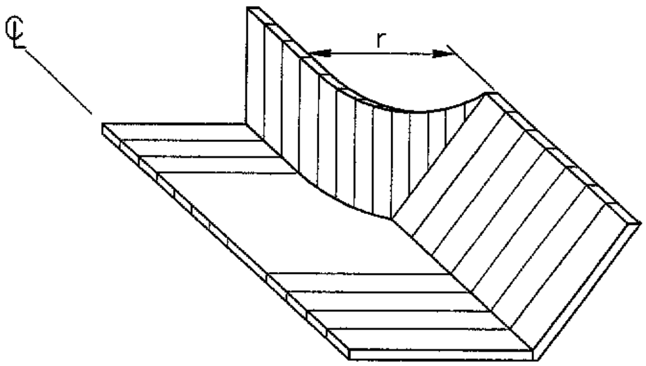
REFERENCE:

Simons, Li and Assoc., 1989

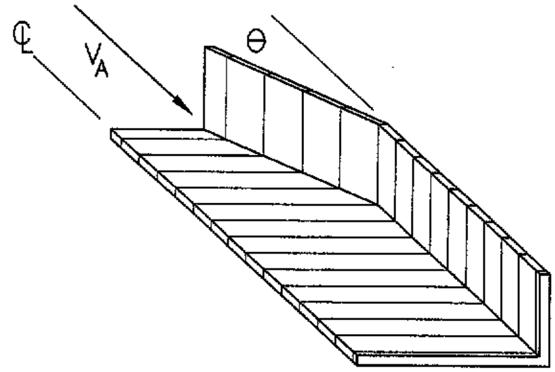
VERSION: AUGUST 2002

WRC ENGINEERING, INC

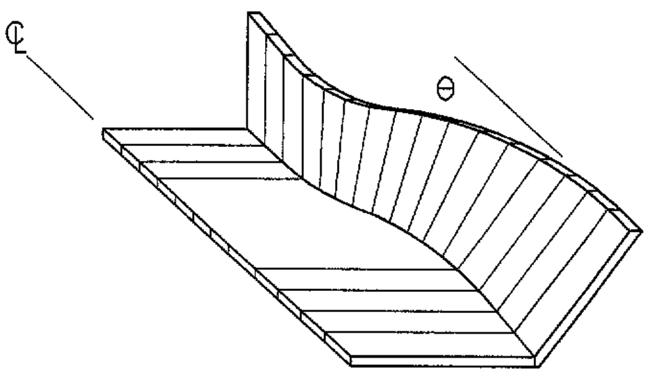
$$\text{Froude Number} = \sqrt{\frac{V_A}{g y_A}}$$



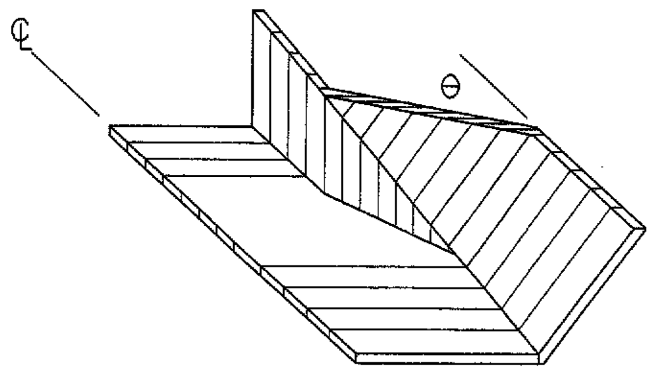
CYLINDRICAL QUADRANT



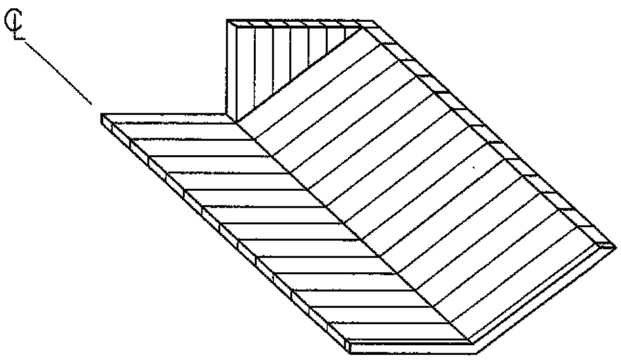
STRAIGHT LINE



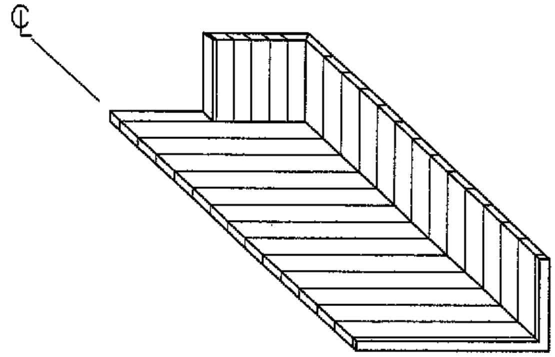
WARPED



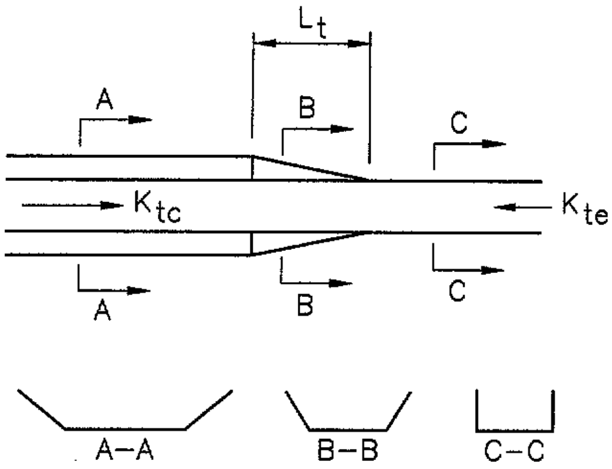
WEDGE



SQUARE–ENDED



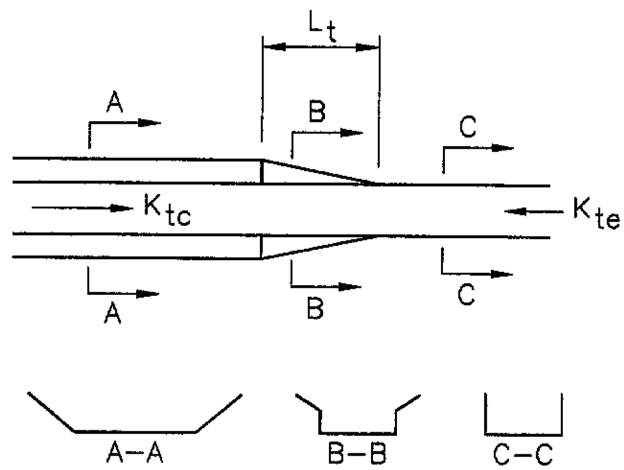
SQUARE–ENDED



WARPED TRANSITION

K_{tc} (CONTRACTION)=0.1

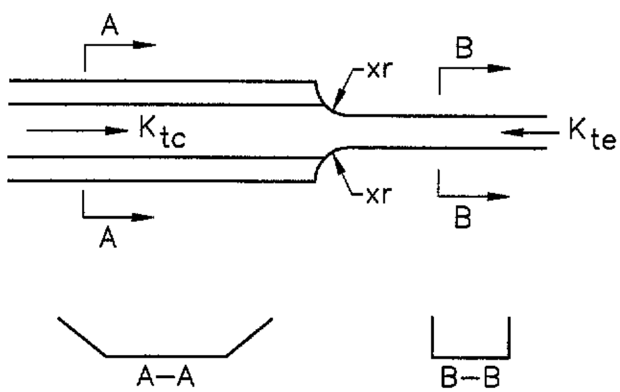
K_{te} (EXPANSION)=0.2



STRAIGHT–LINE TRANSITION

K_{tc} (CONTRACTION)=0.3

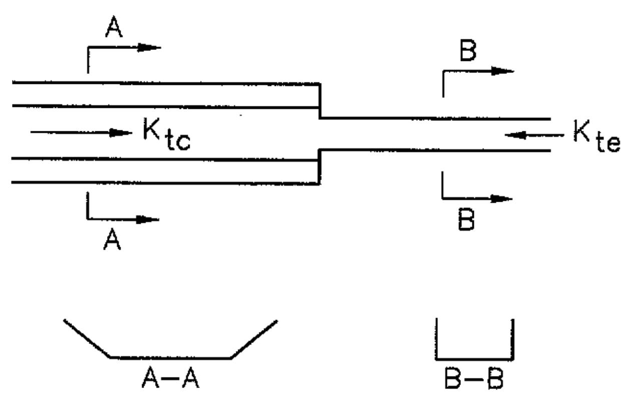
K_{te} (EXPANSION)=0.5



CYLINDER–QUADRANT

K_{tc} =0.15

K_{te} =0.25



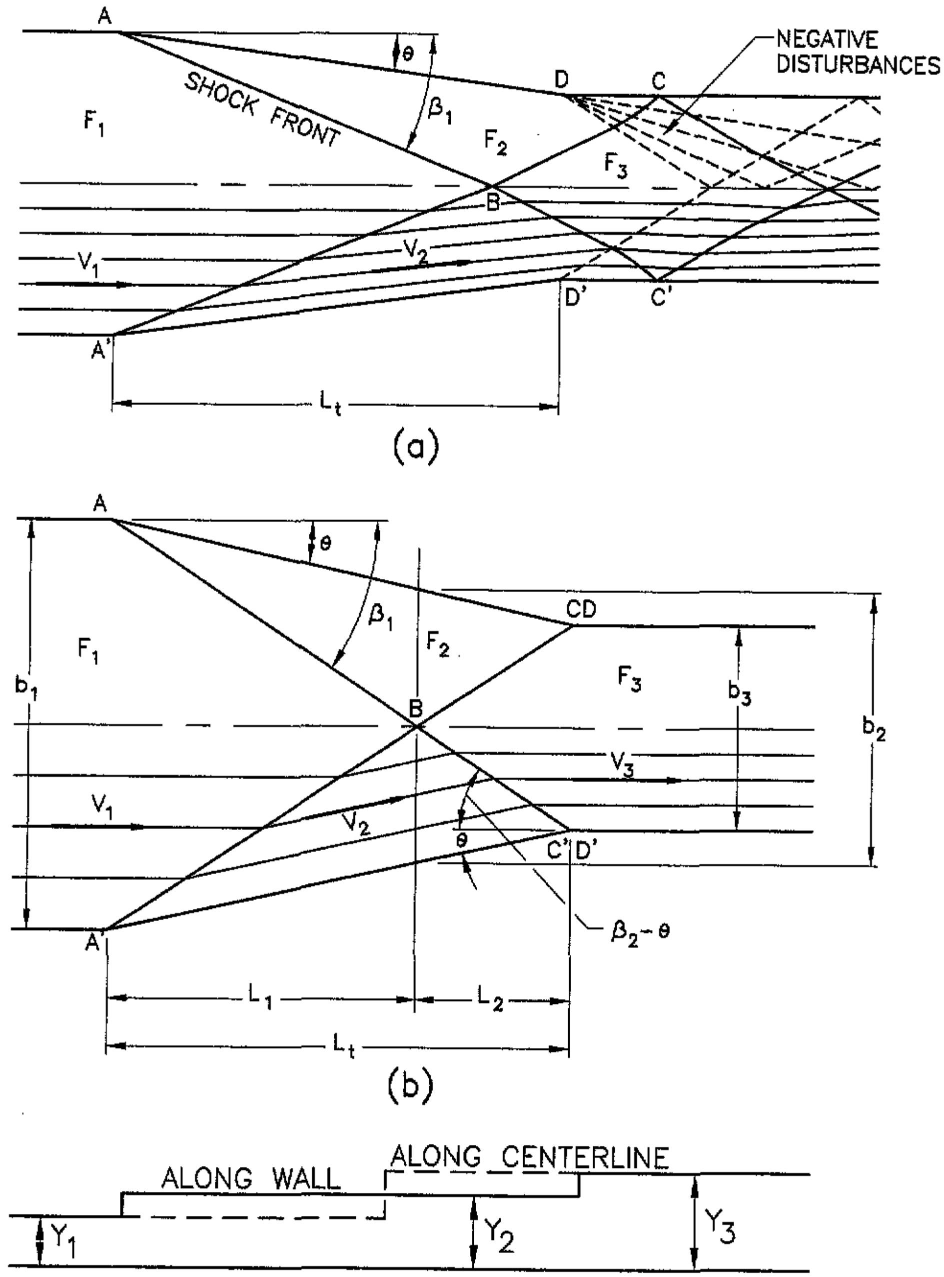
SQUARE–ENDED TRANSITION

K_{tc} =0.30

K_{te} =0.75

VERSION: AUGUST 2002	REFERENCE: USACE, Hydraulic Design Of Flood Control Channels, EM-1110-02-1601, July 1970	FIGURE CH6-F118 TYPICAL CHANNEL TRANSITION SECTIONS AND ENERGY LOSS COEFFICIENTS
WRC ENGINEERING, INC.		

O:/2080/FIGURES, CH6-F118.DWG, 4/9/02 MAS



NOTES:

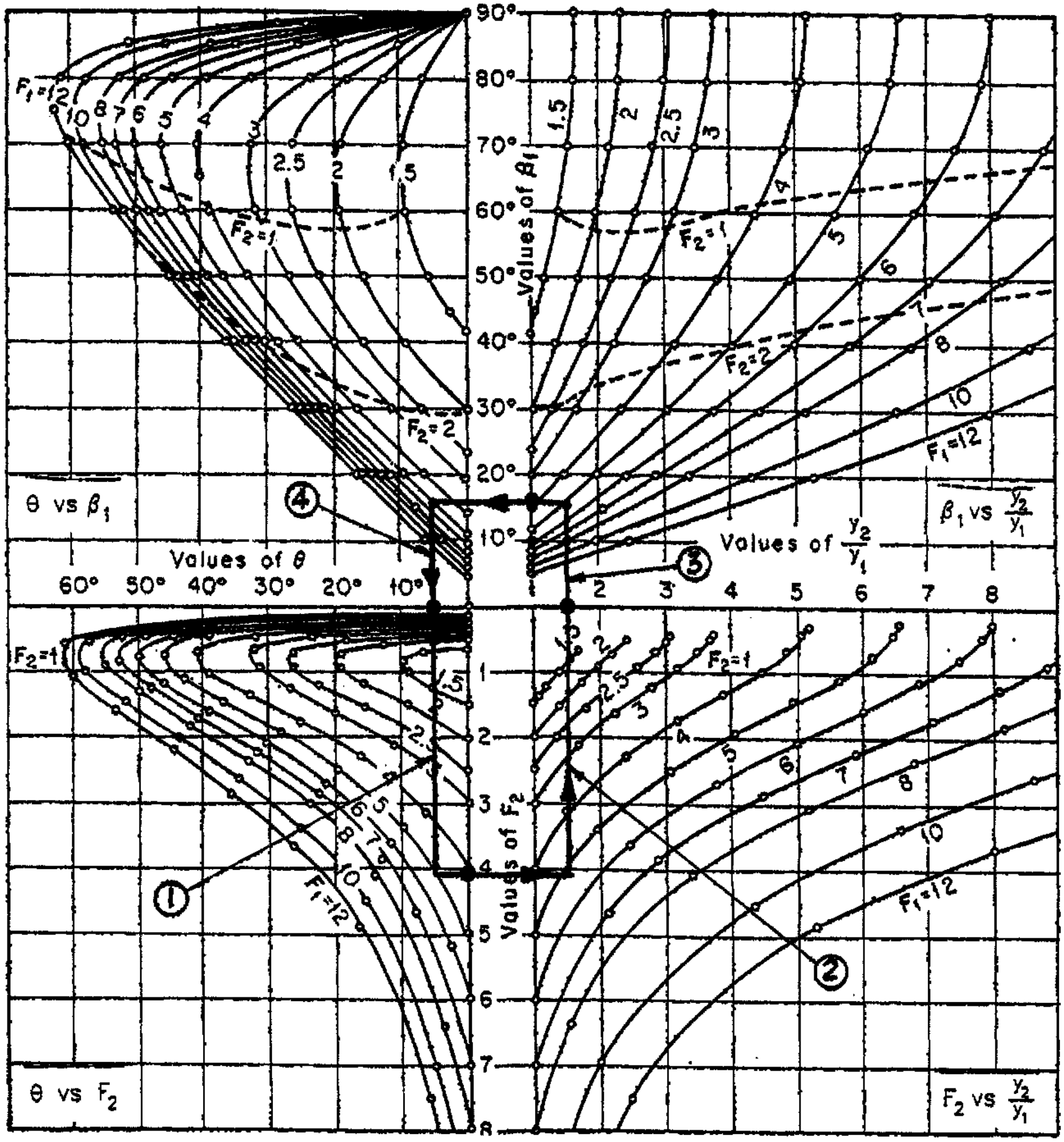
(a) GENERAL DISTURBANCE PATTERNS

(b) MINIMUM DOWNSTREAM DISTURBANCE

(c) SCHEMATIC PROFILE

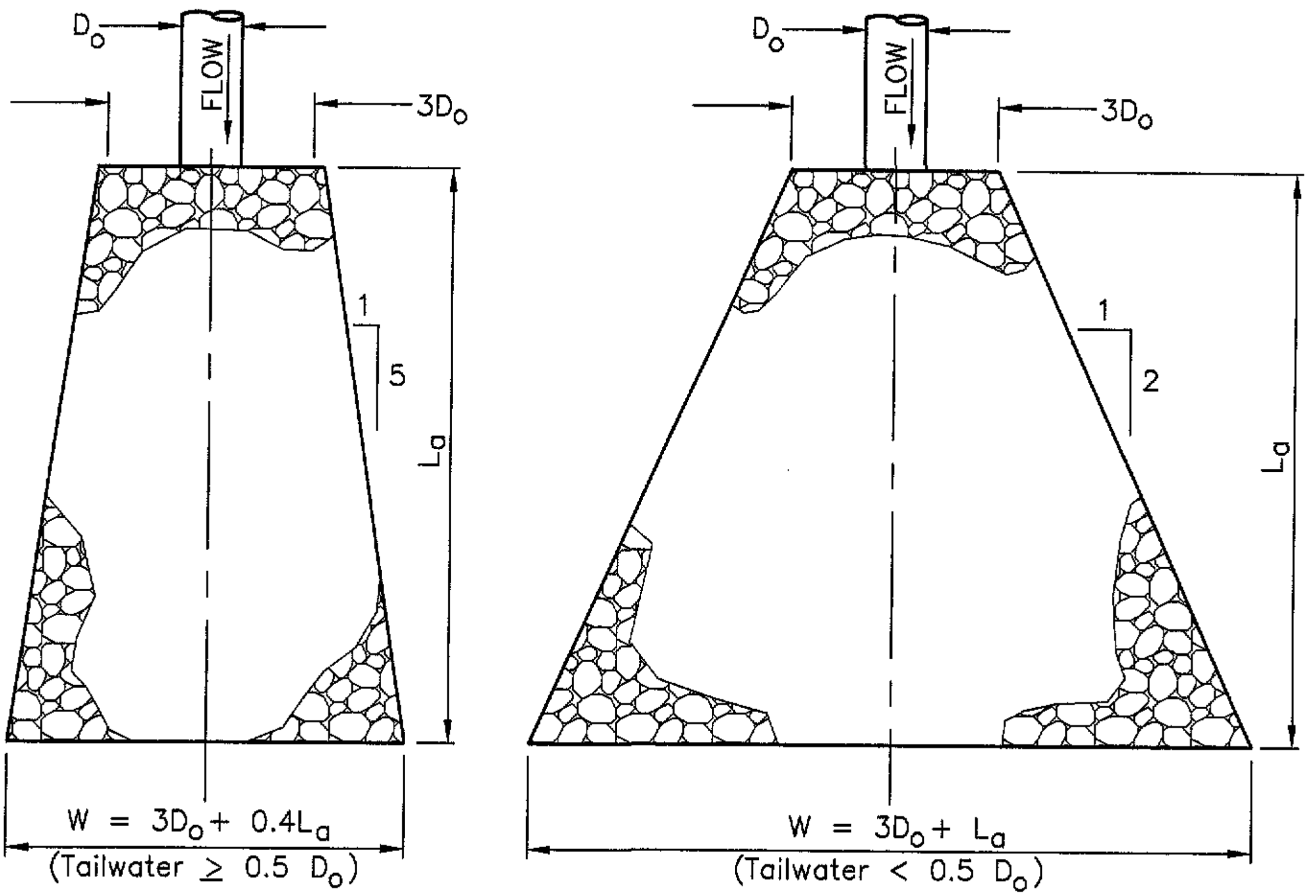
O:/2080/FIGURES, CH6-F119.DWG, 4/10/02 MAS

VERSION: AUGUST 2002 WRC ENGINEERING, INC.	REFERENCE: Chow, V.T., Open-Channel Hydraulics, McGraw Hill Book Company, 1959	**FIGURE CH6-F119** TYPICAL CONTRACTING TRANSITION FOR SUPERCRITICAL FLOW



EXAMPLE: For $\theta = 5°$ and $F_1 = 5.0$

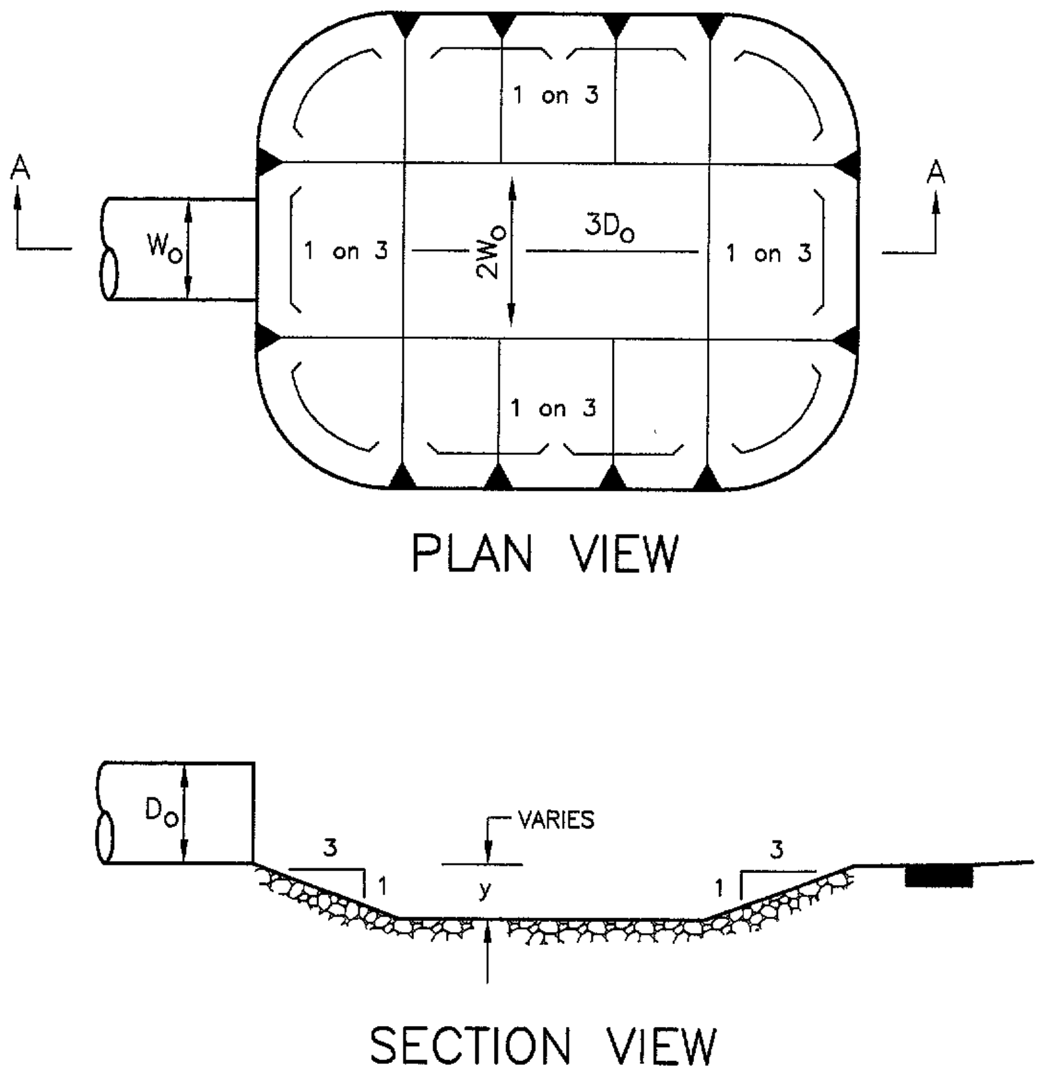
1. Read $F_2 = 4.1$
2. Read $y_2/y_1 = 1.5$
3. Read $\beta_1 = 16°$
4. Read $\theta = 5°$ (check)



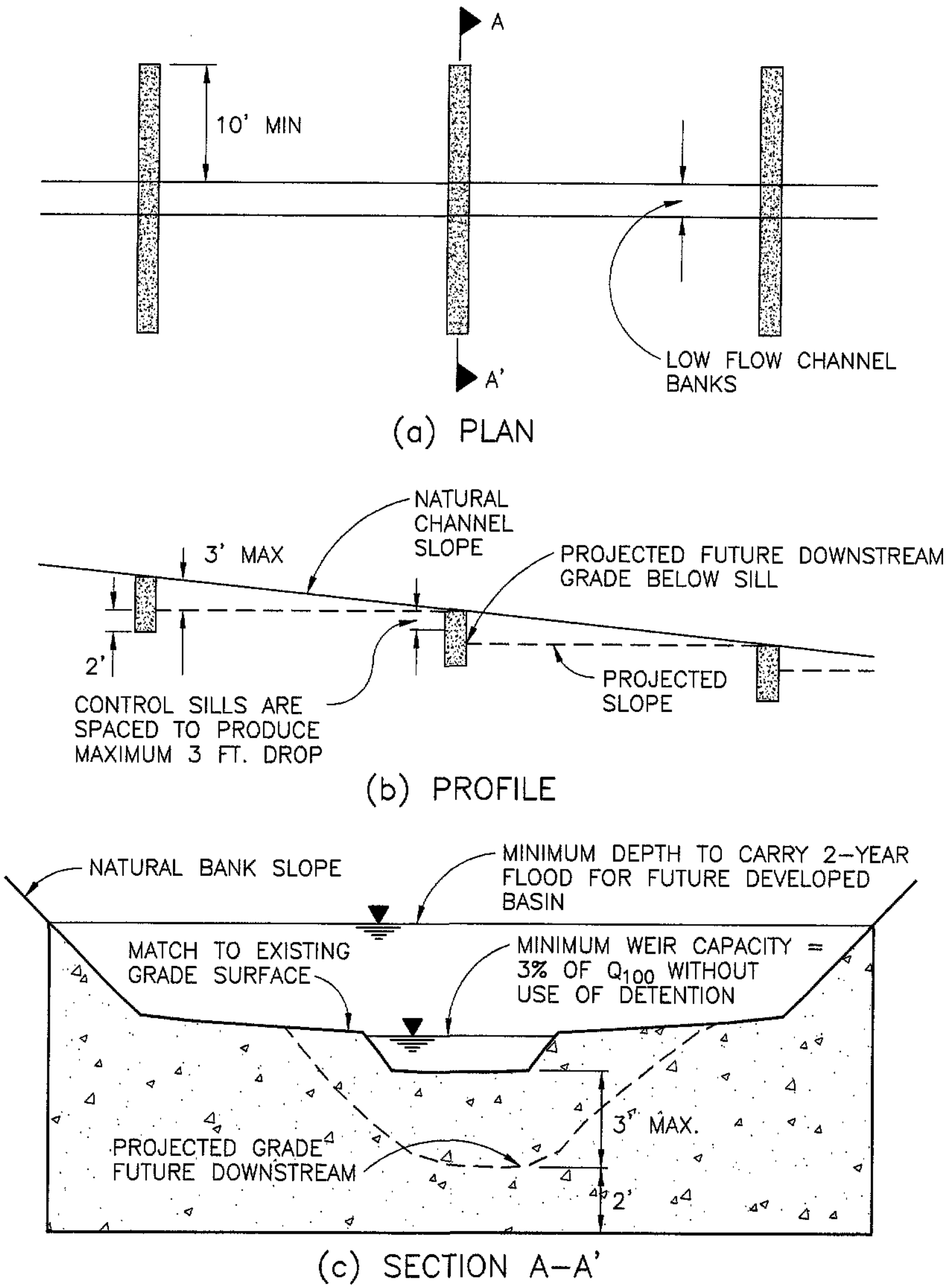
c:/2080/FIGURES, CH6-F121.DWG, 4/10/02 MAS

REFERENCE:
U.S. EPA, 1976

FIGURE CH6-F121
CONFIGURATION OF CULVERT OUTLET PROTECTION

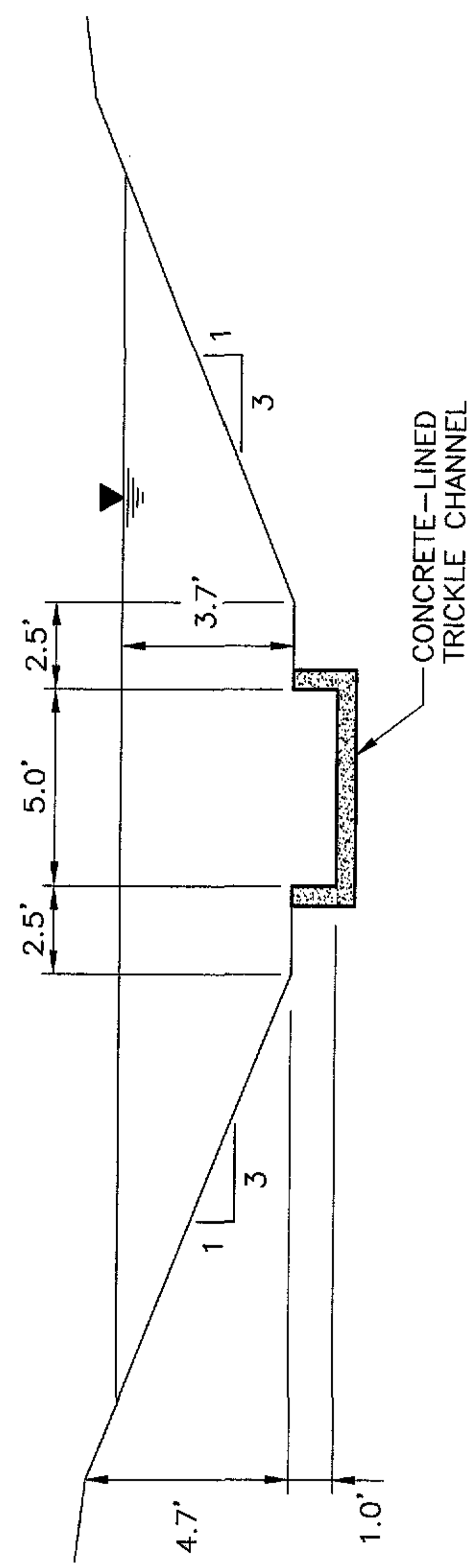

A
A
W_o
1 on 3
1 on 3
1 on 3
1 on 3
$2W_o$
$3D_o$
PLAN VIEW
D_o
VARIES
3
1
y
3
1
SECTION VIEW

COLORADO STATEWIDE
DRAINAGE AND FLOODPLAIN MANAGEMENT CRITERIA MANUAL



o:/2080/FIGURES, CH6-F123.DWG, 4/10/02 MAS

VERSION: AUGUST 2002	REFERENCE:	FIGURE CH6-F123
WRC ENGINEERING, INC.	UDFCD, 1990	CONTROL SILL GRADE CONTROL STRUCTURE



G:/2080/FIGURES, CH6-F124.DWG, 4/10/02 MAS

VERSION: AUGUST 2002	REFERENCE:	**FIGURE CH6-F124**
WRC ENGINEERING, INC.		EXAMPLE: CROSS-SECTION OF DOE CREEK

CHAPTER 6

HYDRAULIC ANALYSIS AND DESIGN

SECTION 2.0

BRIDGES AND CULVERTS

CHAPTER 6
HYDRAULIC ANALYSIS AND DESIGN

SECTION 2.0
BRIDGES AND CULVERTS

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

LIST OF STANDARD FORMS

CHAPTER 6
HYDRAULIC ANALYSIS AND DESIGN

SECTION 2.0
BRIDGES AND CULVERTS

2.1 INTRODUCTION

Culverts and bridges are widely used to convey surface water through or beneath roadways, railroads, other embankments, and engineered structures. The size, material, alignment, and support structures of bridges or culverts directly affect the flow conveyance capacity of the overall drainage system.

Inadequately designed culverts or bridges can force flows out of the conveyance system, and the flows may take an alternate path and cause damage away from the channel. Undersized structures can also cause increased flow depths upstream of the crossing location. All new and replacement culverts and bridges should be designed to not adversely impact surrounding properties by increasing the water surface elevations and/or by diverting flows out of the channel to a different flow path. Placement of culverts and bridges within the designated floodway may be allowed only if it can be proven through a detailed hydraulic analysis that it will not increase the 100-year water surface elevation.

All new and replacement culverts and bridges should be designed to not adversely impact surrounding properties by increasing the water surface elevations and/or by diverting flows out of the channel to a different flow path.

The primary distinction between a culvert and a bridge is the change in flow conveyance area from the upstream channel cross-section. A culvert is usually designed to allow the upstream water surface elevation to be greater than the top of the culvert, while bridge design generally provides freeboard between the design floodwater surface and the low chord of the bridge.

2.2 DESIGN STANDARDS FOR CULVERTS

All culverts shall be designed, at a minimum, to with-stand an HS-20 loading.

All culverts within the State of Colorado shall be designed using the following standards. The analysis and design shall consider the design flow rate, culvert size and material, culvert length and slope, upstream channel and entrance configuration, downstream channel and outlet configuration, and erosion protection. Maintenance access for culvert maintenance and cleaning shall be provided at all culvert locations.

Culverts must be structurally designed to withstand the design loads including earth, pavement, and traffic loads. The structural design of culverts shall conform to those methods and criteria recommended by the manufacturer for the culvert type and for the conditions found at the installation site. The minimum standards set forth in the current American

Association of State Highway and Transportation Officials (AASHTO) Standard Specifications for Highway Bridges shall be adhered to. All culverts shall be designed, at a minimum, to withstand an HS-20 loading. For large structures or where groundwater is a problem, the design shall include necessary provisions to resist hydrostatic uplift forces that could result in failure of the culvert structure.

2.2.1 CULVERT SIZING CRITERIA

In most instances, culverts have direct impacts on the resulting water surface elevations and the flow conveyance capacity of the overall drainage system. Therefore, it is imperative that culverts are properly sized to convey the design flows at or below the required water surface elevations. Larger culverts do not encroach into the channel cross-section as much as smaller culverts and will cause a smaller rise in water surface elevations. The trade-off is that larger culverts are more expensive to construct than small culverts.

All new and replacement culvert structures, including street overflow sections where permitted, are recommended to be designed to confine and convey the 100-year flows. As a general rule, a 10 % bulking/clogging factor shall be added to the estimated 100-year peak flow rate to account for sediment and debris loads

2.2.1.1 DESIGN FREQUENCY

All new and replacement culvert structures, including street overflow sections where permitted, are recommended to be designed to confine and convey the 100-year flows.

Sediment and debris loads associated with a 100-year flood event shall be considered in the culvert design. As a general rule, a 10 % bulking/clogging factor shall be added to the estimated 100-year peak flow rate. For drainage-ways with known substantial sediment deposition problems, sediment and debris loads shall be determined using historic flood/debris information documented by CWCB or local officials. Where appropriate, sediment/debris trap basins shall be constructed upstream of the culvert structure.

2.2.1.1 ALLOWABLE CROSS STREET FLOW

The maximum allowable flow overtopping limits during a 100-year event for various street classifications are outlined below.

Street Classification	Max. Depth at the Street Crown (Ft.)	Max. Flow Velocity (fps)
Local	1 ft.	6 fps
Collector	1 ft.	6 fps
Arterial	No Overflow	No Overflow
Freeway & Highway	No Overflow	No Overflow

The minimum guidelines for the design of street overflow section are outlines below.

- Using the allowable overflow limits specified above, the allowable overflow for a 100-year event should be determined based on the street classification and profile. In most instances, the roadway overtopping section may be treated as a broad-crested weir.

- The culvert is then sized for the difference between the 100-year peak flow rate and the allowable flow over the street.

- If the resulting culvert size is smaller than what is required by the Colorado Department of Transportation (CDOT) as summarized in the following table, adjust the culvert size to comply with the CDOT criteria.

Cross Drainage Type	Design Storm Frequency
Multilane Roads-	
including Interstate	
In Urban areas	100 years
In Rural areas	50 years
Two-Lane Roads	
In Urban areas	100 years
In Rural areas	
Q50 ≥ 4000 cfs	50 years
Q50 < 4000 cfs Design ADT > 750	25 years
Q50 < 4000 cfs Design ADT < 750	10 years

- If only a small increase in culvert size is required to prevent overtopping during a 100-year event, then the larger culvert is recommended.

- Street overflow will not be allowed if the street in question is the only excess for an area during a 100-year flood event.

In all cases, culverts should be adequately sized and designed to not adversely impact adjacent properties by increasing the water surface elevations and/or by diverting flows out of the channel to a different flow path.

2.2.1.3 MINIMUM CULVERT SIZE

The minimum culvert size shall be 18-inch diameter for a round pipe or shall have a minimum flow conveyance area of 2.2 square feet for other pipe shapes. The minimum inside dimension for elliptical or arched pipes shall be no less than 12 inches.

Culverts under driveways of single-family residences shall be sized to convey flows equivalent to the roadside ditch capacity and be a minimum of 15-inch diameter round pipe or equivalent.

2.2.2 CONSTRUCTION MATERIALS

Culverts can be constructed with many different types of materials for variety of sizes and shapes. Culverts used in the State of Colorado shall be constructed with reinforced concrete, PVC, HDPE, or corrugated metal.

Corrugated metal pipe culverts are available in round or arch cross-sections. Sections of corrugated metal can also be bolted together to form several other cross sectional shapes, such as elliptical and pear shapes. Corrugations also come in various dimensions, which affect the hydraulics of the pipe flow. The wall thickness of CMP should be determined based on many factors including, design loads, cover depth, culvert size, and corrugated dimension. Please refer to the Handbook of Steel Drainage and Highway Construction Products published by The American Iron and Steel Institute for the design standards. Site-specific soil tests are required for the placement of corrugated metal pipes (CMP). If soil tests identify the presence of corrosive soil conditions, appropriate pipe coatings will be required.

Reinforced Concrete Box Culverts (RCBC) can be constructed (cast-in-place) for generally any rectangular cross-section with the only limitations being the physical site constraints and the structural requirements. Pre-cast reinforced concrete box and pipe culverts and are also available in several standard dimensions.

The Colorado Water Conservation Board (CWCB) may allow other materials to be used for the construction of culverts. Design and material testing documentations must be submitted for review and approval by CWCB. Supporting documentations must demonstrate that the subject pipe material has a design life similar to the approved materials and that the interior lining, if any, will maintain the design Manning's roughness coefficient ("n") value for the life of the pipe material. Typical Manning's "n" values for different culvert materials and shapes are provided in Table CH6-T202.

Typical Manning's "n" values for different culvert materials and shapes are provided in Table CH6-T202.

2.2.3 VELOCITY LIMITATIONS AND INLET/OUTLET PROTECTION

All culverts shall be designed to provide a minimum flow velocity of 3 fps at the culvert outlet for the 5-year storm event condition. In addition, the culvert slope shall be a minimum of 0.25 percent.

Design flow velocities through the culvert structure should be determined, at a minimum, for 5- and 100-year storm events. If the flow velocity is too slow, sediment deposition may occur within the culvert decreasing the effective conveyance area of the culvert and increasing the frequency of required maintenance. All culverts shall be designed to provide a minimum flow velocity of 3 fps at the culvert outlet for the 5-year storm event condition. In addition, the culvert slope shall be a minimum of 0.25 percent.

If the flow velocity exiting the culvert is too high, channel erosion and scour at the outlet will take place, possibly jeopardizing the integrity of the culvert and roadway embankment. The design criteria of outlet erosion protections for natural and unlined channels are as follows:

Outlet Velocity (fps)	Required Outlet Protection
Less than 5	Minimum riprap protection (Section 1.10.3, Chapter 6)
Between 5 and 15	Riprap protection (Section 1.10.3, Chapter 6) or Energy dissipater (Section 6, Chapter 6)
Greater than 15	Energy dissipater (Section 6, Chapter 6)

For lined channels, the outlet discharge velocity must not exceed the maximum allowable channel design velocity. Otherwise, additional outlet erosion protection measures shall be provided as outlined above.

Headwalls and wingwalls or flared-end sections shall be provided for all culverts at both inlets and outlets. Guardrails and/or handrails shall also be provided in conformance with the local building codes and roadway design safety requirements. Street overflow sections, when used, shall be designed to adequately confine and convey the 100-year flows into the downstream channel. Adequate erosion protection measures shall be provided to prevent degradation of the roadway and embankments.

2.2.4 HEADWATER CRITERIA

The extent of impacts on adjacent properties from the 100-year backwater created by culvert installations shall be analyzed for all culverts. Culverts should be designed to properly convey the design flows at or below the required water surface elevations. Ponding at the culvert entrance will not be allowed if such ponding will cause property or roadway damage, saturation of fills, significant upstream deposits of debris, or inundation of existing or future facilities.

The maximum headwater for the 100-year design flow shall be 1.5 times the culvert height for all culverts taller than 36" with standard inlet and outlet configurations. The maximum headwater for culverts with a height of 36" or less shall be 5 feet.

The maximum headwater for the 100-year design flows shall be 1.5 times the culvert height for all culverts taller than 36" with standard inlet and outlet configurations. The maximum headwater for culverts with a height of 36" or less shall be 5 feet.

If site conditions are such that the maximum headwater limits cannot be met, additional engineering analysis shall be performed. The additional analysis is necessary to determine scour potential, embankment stability and any other factors that may influence the long-term stability of the structure. Additional erosion protection around the culvert inlet or other design considerations shall be included as appropriate to ensure the long-term stability of the culvert and approaches.

Culverts that do not include a street overtopping section shall have a minimum of 1-foot freeboard from the hydraulic grade line at the culvert entrance to the edge of pavement elevation. Levees shall not be used to provide increased headwater at culvert inlets.

2.2.5 ALIGNMENT

Alignment of the culvert with respect to the natural channel is very important for proper hydraulic performance. Culverts may pass beneath the roadway normal to the centerline or they may pass at an angle (skewed). Whenever possible, culverts should be aligned with the natural channel. This reduces inlet and outlet flow transition problems.

Where the natural channel alignment would result in an exceptionally long culvert, modification of the natural alignment may be necessary. Since such modifications will change the natural stability of the channel, proposed modifications should be thoroughly investigated. Although the economic factors are important, the hydraulic effectiveness of the culvert must be given major considerations. Improper culvert alignment may cause erosion to adjacent properties or siltation within the culvert. Culvert alignment considerations are shown in Figure CH6-F201.

Roadway alignment also affects the culvert design. The vertical alignment of roadways may define the maximum culvert diameter that can be used. Low vertical clearance may require the use of elliptical or arched culverts, or the use of a multiple-barrel culvert system. All culverts shall have a minimum of 1.5 foot of cover from top of asphalt (or gravel for gravel road) to outside top of pipe. Culverts with less than 1.5 feet of cover will require additional structural analysis and other provisions (i.e. full depth concrete paving to compensate for the loss of proper cover.

2.2.6 MULTIPLE-BARREL CULVERTS

If the available embankment fill height limits the size of culvert necessary to convey the flood flows, multiple culverts can be used. If each barrel of a multiple-barrel system is of the same type and size, and constructed such that all hydraulic parameters are equal, the total flow should be assumed to be equally divided among each of the barrels.

2.2.7 TRASH RACKS/SAFETY GRATES

Trash racks or safety grates may be necessary at the upstream inlet of some culverts. During the culvert design, engineering judgments shall be used to determine if trash racks or safety grates should be included. Factors that may influence whether or not trash racks or safety grates should be used include the following:

- Tributary Land Use (urban, rural, forest)
- Location (urban/rural)
- Design flow rate
- Size of culvert

- Anticipated debris loading
- Performance of nearby existing structures

Trash racks shall be used on any size or length of culvert where the horizontal or vertical alignment does not allow for an unobstructed view through the culvert. Trash racks/safety grates should be hinged at the top to permit the grate to be lifted and allow the culvert and grate to be cleaned. The grate/rack should slope at 2:1 to 5:1 (horizontal to vertical) to permit the debris to float up the grate as the water level rises. The bar spacing should prevent a child from passing through the openings. The net open area through the rack/grate below the design water surface shall be at least four times the design flow area of the culvert.

2.2.8 AIR VENTS

All culverts greater than 48 inches in diameter for which both the inlet and outlet are sealed by water under less than full flow conditions shall include an air vent pipe to prevent air accumulation/partial vacuums. Said vent shall have a diameter equal to or greater than one-sixth of the culvert pipe diameter.

2.3 CULVERT HYDRAULICS

This section presents the general procedures for hydraulic design and evaluation of culverts. The user is assumed to possess a basic working knowledge of culvert hydraulics and is encouraged to review the textbooks and other technical literature on the subject. The following is a short list of some of the culvert hydraulics publications.

- U.S. Department of Transportation, Federal Highway Administration, Hydraulic Design of Highway Culverts, Hydraulic Design Series No. 5, September 1985.

- U.S. Department of Transportation, Federal Highway Administration, Hydraulic Charts for the Selection of Highway Culverts, Hydraulic Engineering Circular No. 5, December 1965.

- U.S. Department of Transportation, Federal Highway Administration, Capacity Charts for the Hydraulic Design of Highway Culverts, Hydraulic Engineering Circular No. 10, November 1972.

- U.S. Department of Transportation, Federal Highway Administration, Hydraulic Design of Improved Inlets for Culverts, Hydraulic Engineering Circular No. 13, 1972.

The two categories of flow in culverts are inlet control and outlet control. Under inlet control, the flow through the culvert is controlled by the headwater of the culvert and the inlet geometry. Under outlet control, the flow through the culvert is controlled primarily by culvert slope, roughness, and the tailwater elevation.

When designing a culvert, the designer must evaluate both inlet and outlet control conditions for the given design constraints (e.g. headwater depth, flow capacity, etc.). The control condition that produces the greater energy loss for the design conditions determines the appropriate control to use for culvert design. Culvert hydraulic calculations shall be performed using rating nomographs and/or culvert hydraulic analysis programs.

2.3.1 INLET CONTROL CONDITION

Inlet control for culverts may occur in two ways (see Figure CH6-F202):

1. Unsubmerged - The headwater is not sufficient to submerge the top of the culvert and the culvert invert slope is supercritical. The culvert entrance acts like a weir (Condition A, Figure CH6-F202).

2. Submerged - The headwater submerges the top of the culvert but the pipe does not flow full. The culvert inlet acts like an orifice (Condition B and C, Figure CH6-F202).

   The inlet control rating for typical culvert shapes and inlet configurations are presented in Figures CH6-F203 to CH6-F206. Additional nomographs are available in the U.S. Department of Transportation's Hydraulic Design Series Number 5 (USDOT, 1985). These nomographs were developed empirically by pipe manufacturers, Bureau of Public Roads, and the Federal Highway Administration. The nomographs shall be used rather than the orifice and weir equations, due to the uncertainty in estimating the orifice and weir coefficients.

2.3.2 OUTLET CONTROL CONDITION

Outlet control will govern if the headwater and/or tailwater is deep enough, the culvert slope is relatively flat, and the culvert is relatively long. There are three types of outlet control culvert flow conditions:

1. The headwater submerges the culvert top, and the culvert outlet is submerged by the tailwater. The culvert will flow full (Condition A, Figure CH6-F202).

2. The headwater submerges the top of the culvert and the culvert is unsubmerged by the tailwater (Condition B or C, Figure CH6-F202).

3. The headwater is insufficient to submerge the top of the culvert. The culvert slope is subcritical and the tailwater depth is lower than the pipe critical depth (Condition D, Figure CH6-F202).

The factors affecting the capacity of a culvert in outlet control include the headwater elevation, the inlet geometry and associated losses, the culvert material friction losses, and the tailwater condition.

The capacity of the culvert is calculated using the conservation of energy principal (Bernoulli's Equation). An energy balance exists between the total energy of the flow at the culvert inlet and at the culvert outlet, which includes the inlet losses, the friction losses, and the velocity head (see Figure CH6-F207). The equation is then expressed as:

$$H = h_e + h_f + h_v \quad \text{(Eq CH6-200)}$$

Where H = Total energy difference, inlet through outlet (ft)
h_e = Entrance head losses (ft)
h_f = Friction losses (ft)
h_v = Velocity head = $V^2/2g$ (feet) (Eq CH6-201)

For inlet losses, the governing equation is:

$$h_e = k_e\,(V^2/2g) \quad \text{(Eq CH6-202)}$$

Where k_e is the entrance loss coefficient. Typical entrance loss coefficients recommended for use are given in Table CH6-T201.

Friction loss is the energy required to overcome the roughness of the culvert and is expressed as follows:

$$h_f = (29n^2L/R^{1.33})(V^2/2g) \quad \text{(Eq CH6-203)}$$

Where n = Manning's coefficient (see Table CH6-T202)
L = Length of culvert (ft)
R = Hydraulic radius (ft)
V = Velocity of flow (fps)
G = Gravitational acceleration constant (32.2 ft/s^2)

Substituting equivalent terms from equations CH6-201, CH6-202, and CH6-203 into equation CH6-200 and simplifying the terms results in the following equation:

$$H = [k_e + (29n^2L/R^{1.33}) + 1]\,V^2/2g \quad \text{(Eq CH6-204)}$$

Equation CH6-204 can be used to calculate the culvert capacity directly when the culvert is flowing under outlet control conditions A or B as shown on Figure CH6-F202. The actual headwater (Hw) is calculated by adding H to the tailwater elevation (see Figure CH6-F207). For conditions C or D in Figure CH6-F202, the hydraulic grade line at the outlet is approximated by averaging the critical depth and the culvert diameter. This value is used to compute headwater depth (Hw) if it is greater than the tailwater depth (Tw). This is an approximate method and is more fully described in HDS No. 5. Estimates of critical depth for box culverts, circular pipe, and elliptical pipe can be obtained from Figures CH6-F208, CH6-F209, and CH6-F210 respectively.

A series of outlet control nomographs for various culvert shapes have been developed by pipe manufacturers, Bureau of Public Roads, and the Federal Highway Administration. The nomographs are presented in Figures CH6-F211 to CH6-F214. Additional nomographs are available in HDS No. 5. When rating a culvert, either the outlet control nomographs or Equation CH6-204 can be used to calculate the headwater requirements.

When using the outlet control nomographs for corrugated metal pipe, the data must be adjusted to account for the variation in the "n" value between the nomographs and the culvert being evaluated. The adjustment is made by calculating an equivalent length according to the following equation:

$$L^1 = L\,(n^1/n)^2 \qquad \text{(Eq CH6-204)}$$

Where L^1 = Equivalent length
L = Actual length
n = Manning's "n" value shown on Figures CH6-F211 to CH6-F214
n^1 = Actual "n" value of the culvert

2.3.3 HYDRAULIC DATA

The hydraulic data provided in Table CH6-T201 and CH6-T202 shall be used in the hydraulic design of all culverts. The design capacity of culverts shall be calculated using the computation sheet provided as Standard Form CH6-SF201. Manning's roughness coefficients ("n") used for velocity and capacity calculations shall be those presented in Table CH6-T202. Alternatively, computer programs may be used for hydraulic analysis. However the designer should thoroughly review the modeling results to determine if the analysis has properly modeled the hydraulic conditions.

2.4 DESIGN STANDARDS FOR BRIDGES

All bridges shall be designed in accordance with the "Standard Specifications for Highway Bridges" by AASHTO. Hydraulic design and analysis shall be in accordance with the following criteria.

2.4.1 BRIDGE SIZING CRITERIA

All new bridges shall be designed to pass the 100-year estimated peak flows. Additionally, the design water surface elevation within the bridge shall be at least 2 feet below the bridge low chord or appropriate measures should be taken to avoid floatation of the bridge due to debris blockage. Additional freeboard may be necessary for various special hydraulic conditions.

If possible, replacement bridges shall also be designed to pass the 100-year estimated peak flows as discussed above. If site-specific conditions do not allow a replacement bridge to be designed to convey the 100-year flows, the design engineer shall coordinate with the appropriate agencies to determine the acceptable bridge design capacity. Hydraulic analyses must be performed to demonstrate that the bridge placement will not adversely affect adjacent properties.

All new bridges shall be designed to pass the 100-year estimated peak flows. The design water surface elevation within the bridge shall be at least 2 feet below the bridge low chord or appropriate measures should be taken to avoid floatation of the bridge due to debris blockage. If possible, replacement bridges shall also be designed to pass the 100-year estimated peak flows as discussed above.

2.4.2 VELOCITY LIMITATIONS

The velocity limitation through a bridge opening is controlled by the scour potential and subsequent channel erosion protection measures provided. The 100-year design flow velocity through the bridge and approaches shall not exceed the allowable velocity for the channel lining type as discussed in Section 1.5.2, Chapter 6. If the design velocity through the bridge is greater than the maximum allowable velocity of the natural channel, appropriate channel protection measures shall be provided.

2.5 BRIDGE HYDRAULICS

2.5.1 HYDRAULIC ANALYSIS

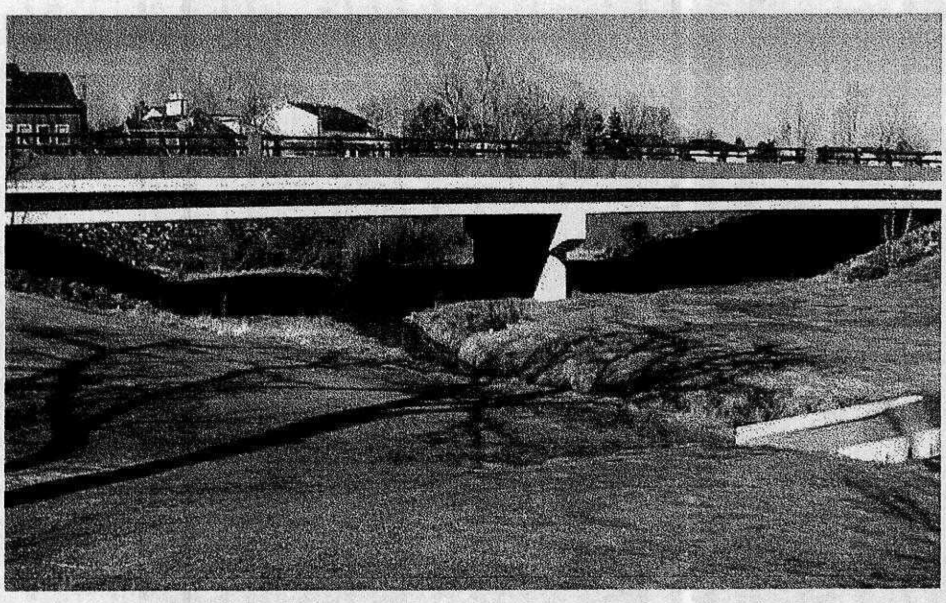
The procedures for analysis and design as outlined in the following publications shall be used for the hydraulic design and scour analysis of all bridges.

- U.S. Department of Transportation, Federal Highway Administration, Hydraulics of Bridge Waterways, Hydraulic Design Series No. 1, 1978.

- U.S. Department of Transportation, Federal Highway Administration, Evaluating Scour at Bridges, Hydraulic Engineering Circular No. 18, 1993.

- U.S. Department of Transportation, Federal Highway Administration, Stream Stability at Highway Structures, Hydraulic Engineering Circular No. 20, 1991.

This analysis shall be supplemented by an appropriate backwater analysis using HEC-RAS or HEC-2 to verify the resulting hydraulic performance of the bridge. The extent of the bridge backwater shall be shown on a topographic map.



2.5.2 INLET AND OUTLET CONFIGURATION

The design of all bridges shall include adequate wingwalls of sufficient length to minimize abutment erosion and to provide slope stabilization from the embankment to the channel. Erosion protection on the inlet and outlet transition slopes shall be provided to protect the channel from the erosive forces of eddy currents.

2.6 EXAMPLE APPLICATION

2.6.1 EXAMPLE: CULVERT SIZING

Problem: Determine the culvert size necessary to convey the 100-year, 24-hour peak flow in Doe Creek beneath John Boulevard. The results of this analysis are provided in Table CH6-T203.

Top of road elevation	4928 feet
Culvert inlet elevation	4920 feet
Culvert outlet elevation	4918 feet
Culvert length	200 feet
Inlet	Groove end with headwall and wingwalls at 45 degrees
Outlet	Groove end with headwall and wingwalls at 45 degrees
Flow	191 cfs
Tailwater Depth	4 feet

Solution:

Step 1: Assume a pipe diameter or box culvert dimensions and determine the headwater to depth ratio for inlet control conditions.
Assuming a 5-foot diameter reinforced concrete pipe (RCP), the headwater to depth ratio, is 1.38 (see Figure CH6-F215).

Step 2: Calculate the headwater assuming inlet control conditions. Multiply the pipe diameter times the headwater to depth ratio.

$$\text{Headwater} = HW_I = D*HW/D = 5*1.38 = 6.9 \text{ feet}$$

Step 3: Estimate the critical depth, d_c, from Figure CH6-F209 (see Figure CH6-F216).

$$d_c = 3.9 \text{ feet}$$

Step 4: Since the tailwater depth is less than the culvert diameter, compute the estimated water depth at the culvert outlet assuming the tailwater does not control the outlet conditions.

$$\text{Outlet Depth} = (d_c + D)/2 = (3.9 + 5.0)/2 = 4.5 \text{ feet}$$

Step 5: Determine the flow depth at the culvert outlet, h_o. The estimated depth is the maximum value of the tailwater depth and the water depth assuming no tailwater.

h_o = 4.5 feet

Step 6: Estimate the head, H, for outlet control conditions from Figure CH6-F212.

H = 2.6 feet (see Figure CH6-F217).

Step 7: Calculate the headwater depth for outlet control conditions.

$HW_o = H + h_o + LS_o = 2.6 + 4.5 - 2.0 = 5.1$

Step 8: Determine if the culvert is under inlet control or outlet control and provide the resulting headwater depth and elevation.

Since HW_I is greater than HW_o, the culvert is under inlet control.

HW= 6.9

Step 9: Calculate the outlet velocity by an appropriate method, and determine the type of outlet protection needed.

V = 10.0 fps

Riprap protection or an energy dissipater is necessary.

HYDRAULIC DATA FOR CULVERTS
CULVERT ENTRANCE LOSSES

TYPE OF ENTRANCE	ENTRANCE COEFFICIENT, K_E
Pipe	
Headwall	
Grooved edge	0.20
Rounded edge (0.15D radius)	0.15
Rounded edge (0.25D radius)	0.10
Square edge (cut concrete and CMP)	0.40
Headwall & 45° Wingwall	
Grooved edge	0.20
Square edge	0.35
Headwall with Parallel Wingwalls Spaced 1.25D apart	
Grooved edge	0.30
Square edge	0.40
Beveled edge	0.25
Projecting Entrance	
Grooved edge (RCP)	0.25
Squared edge (RCP)	0.50
Sharp edge, thin wall (CMP)	0.90
Sloping Entrance	
Mitered to conform to slope	0.70
Flared-end Section	0.50
Box, Reinforced Concrete	
Headwall Parallel to Embankment (no wingwalls)	
Square edge on 3 edges	0.50
Rounded on 3 edges to radius of 1/12 barrel dimension	0.20
Wingwalls at 30° to 75° to barrel	
Square edge at crown	0.40
Crown edge rounded to radius of 1/12 barrel dimension	0.20
Wingwalls at 10° to 30° to barrel	
Square edge at crown	0.50
Wingwalls parallel (extension of sides)	
Square edge at crown	0.70

TYPE OF CHANNEL & DESCRIPTION	MINIMUM	NORMAL	MAXIMUM
Brass, smooth	0.009	0.010	0.013
Steel:			
Lockbar and welded	0.010	0.012	0.014
Riveted and spiral	0.013	0.016	0.017
Cast Iron:			
Coated	0.010	0.013	0.014
Uncoated	0.011	0.014	0.016
Wrought Iron:			
Black	0.012	0.014	0.015
Galvanized	0.013	0.016	0.017
Corrugated Metal:			
Sub-drain	0.017	0.019	0.021
Storm Drain	0.021	0.024	0.030
Lucite	0.008	0.009	0.010
Glass	0.009	0.010	0.013
Cement:			
Neat, surface	0.010	0.011	0.013
Mortar	0.011	0.013	0.015
Concrete:			
Culvert, straight and free of debris	0.010	0.011	0.013
Culvert with bends, connections, and some debris	0.011	0.013	0.014
Finished	0.011	0.012	0.014
Sewer with manholes, inlet, etc., straight	0.013	0.015	0.017
Unfinished, steel form	0.012	0.013	0.014
Unfinished, smooth wood form	0.012	0.014	0.016
Unfinished, rough wood form	0.015	0.017	0.020
Wood:			
Stave	0.010	0.012	0.014
Laminated, treated	0.015	0.017	0.020
Clay:			
Common drainage tile	0.011	0.013	0.017
Vitrified sewer	0.011	0.014	0.017
Vitrified sewer with manholes, inlet, etc.	0.013	0.015	0.017
Vitrified subdrain with open joint	0.014	0.016	0.018
Brickwork:			
Glazed	0.011	0.013	0.015
Lined with cement mortar	0.012	0.015	0.017
Sanitary sewers coated with sewage slime with bends and connections	0.012	0.013	0.016
Paved invert, sewer, smooth bottom	0.016	0.019	0.020
Rubble masonry, cemented	0.018	0.025	0.030

0:/2080/FIGURES, CH6-T202.DWG, 4/5/02 MAS

WRC ENGINEERING, INC

VERSION: AUGUST 2002

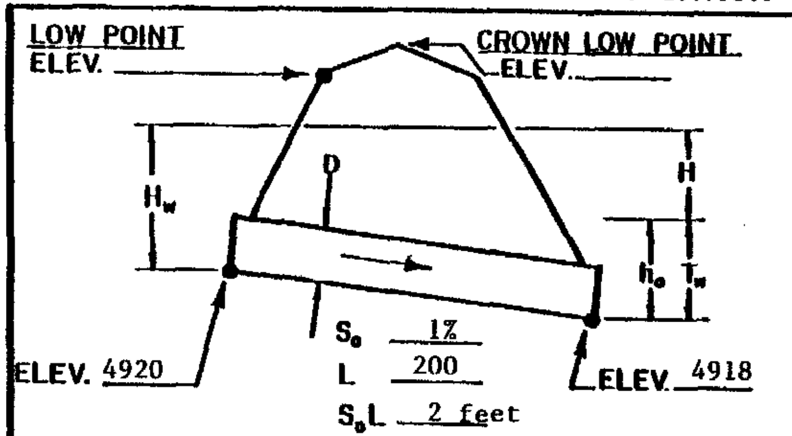
REFERENCE: WRC ENGINEERING, INC.

COLORADO STATEWIDE

DRAINAGE AND FLOODPLAIN MANAGEMENT CRITERIA MANUAL

CULVERT RATING

PROJECT: JOHN BOULEVARD LOCATION: DOE CREEK STATION: ______



CULVERT DATA

TYPE: 5 foot Dia. RCP n: ______

INLET: Groove end w/headwall Q_{FULL}: ______

K_e: 0.2 V_{FULL}: ______

OUTLET CONTROL EQUATIONS

(1) $H_w = H + h_o - LS_o$

(2) For $T_w < D$; $h_o = \frac{d_c + D}{2}$ or T_w (whichever is greater)

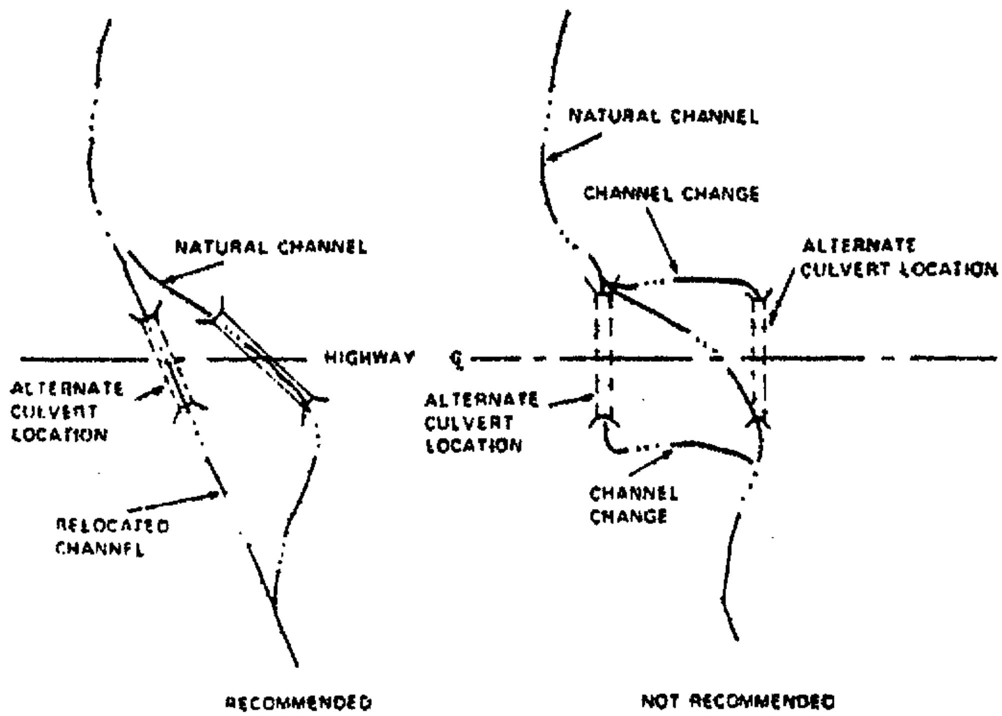
$T_w \geq D$; $h_o = T_w$

(3) For Box Culvert: $d_c = 0.315(Q/B)^{2/3} \leq D$

Q	STORM EVENT	INLET CONTROL		OUTLET CONTROL						TYPE OF CONTROL	CONTROL HEADWATER ELEVATION	OUTLET VELOCITY
		$\frac{H_w}{D}$	H_w	H	T_w	$T_w < D$: d_c	$T_w < D$: $\frac{d_c + D}{2} = h$	$T_w \geq D$: h_o	H_w			
1	2	3	4	5	6	7	8	9	10	11	12	13
191	100-YR	1.38	6.90	2.60	4.0	3.9	4.5	4.5	5.1	Inlet	4926.9	10

TABLE CH6-T203

EXAMPLE: CULVERT DESIGN



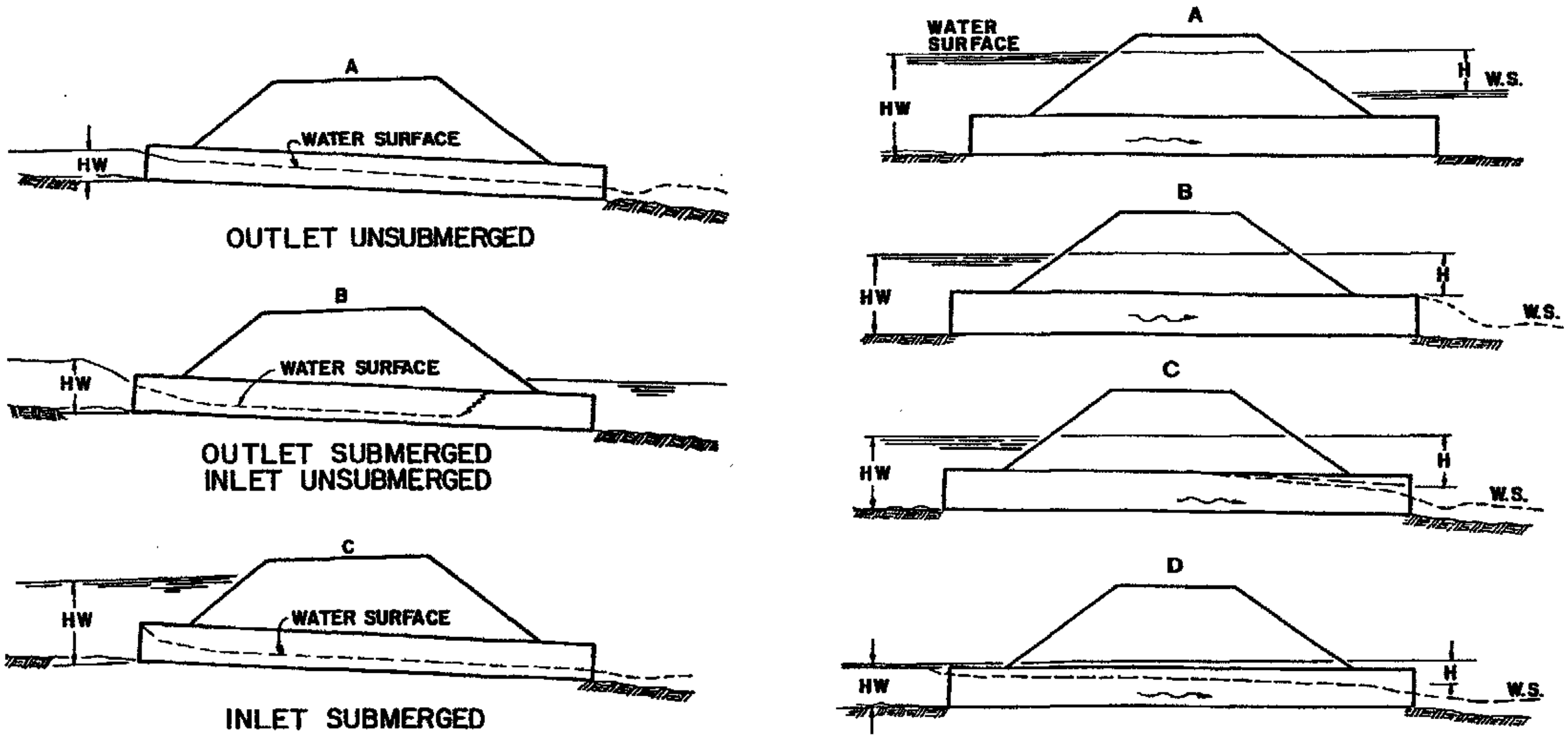
VERSION: AUGUST 2002	REFERENCE: HYDRAULIC DESIGN OF HIGHWAY CULVERTS FHWA-NHI-01-020	FIGURE CH6-F201
WRC ENGINEERING, INC.		CULVERT ALIGNMENT

O:/2080/FIGURES, CH6-F202.DWG, 4/10/02 MAS

CULVERT FLOW TYPES

INLET CONTROL

OUTLET CONTROL



WRC ENGINEERING, INC

VERSION: AUGUST 2002

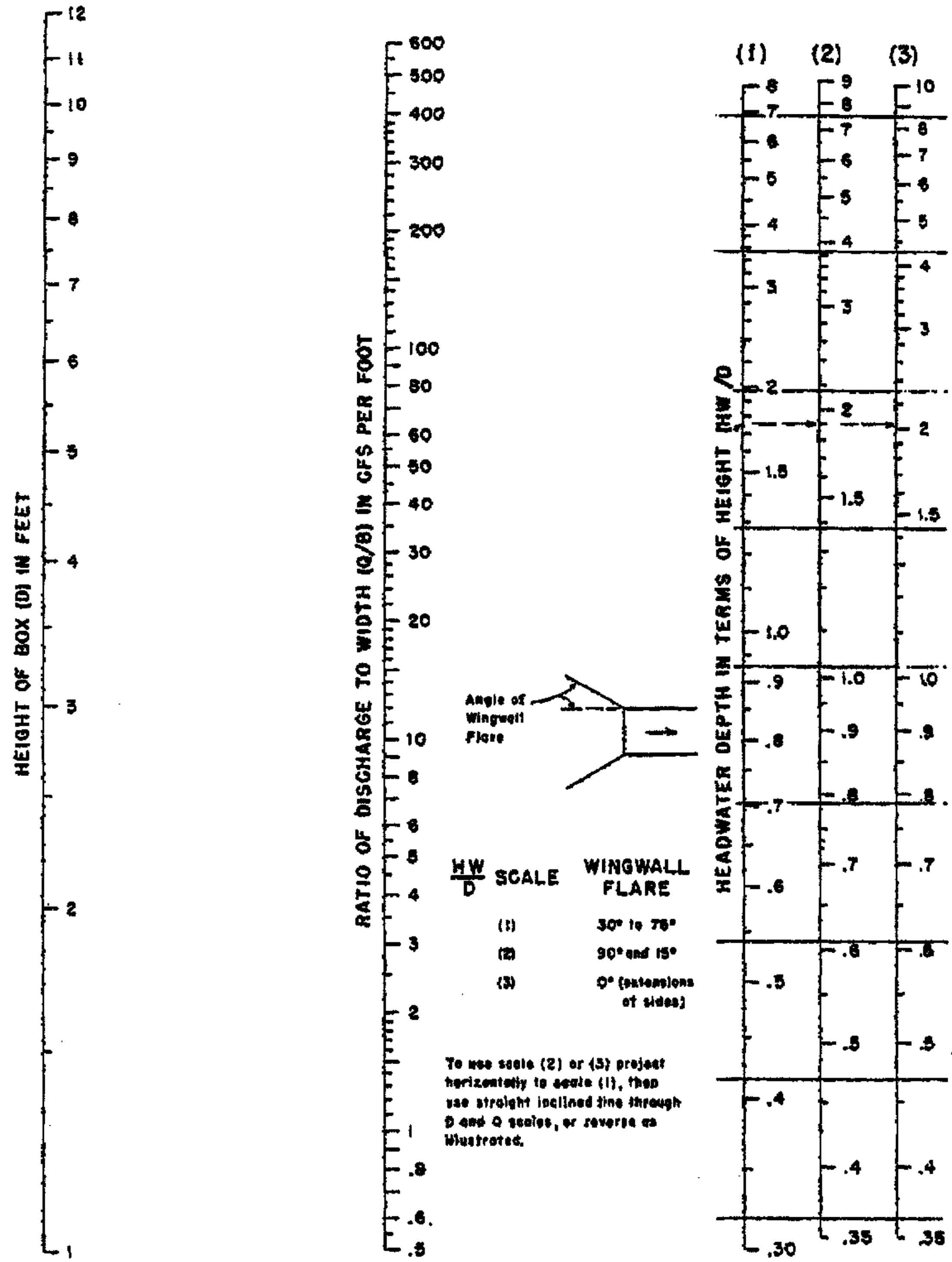
REFERENCE:
HYDRAULIC DESIGN OF HIGHWAY CULVERTS
FHWA-NHI-01-020

FIGURE CH6-F202

CULVERT FLOW TYPES

COLORADO STATEWIDE

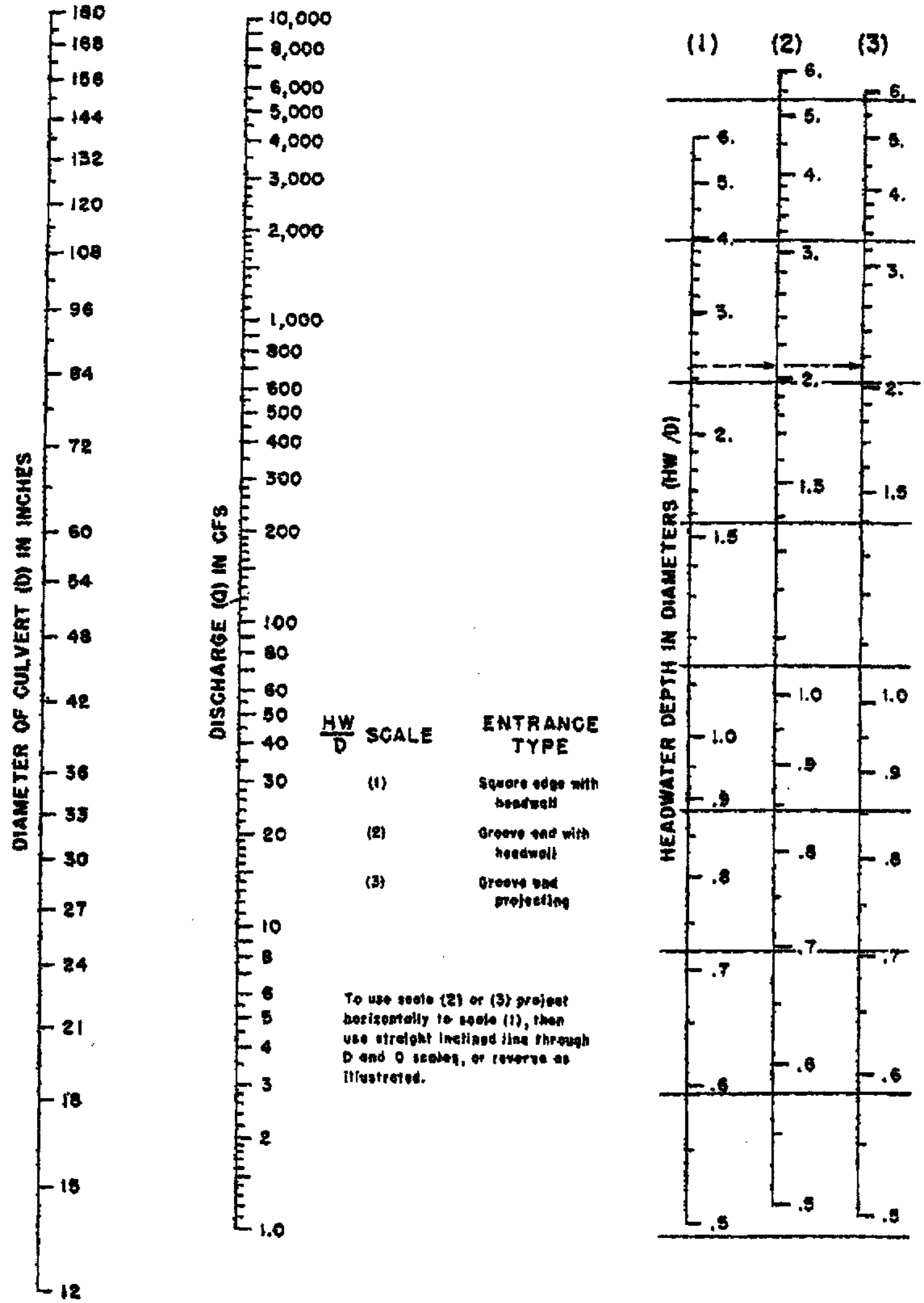
DRAINAGE AND FLOODPLAIN MANAGEMENT CRITERIA MANUAL



REFERENCE:
HYDRAULIC DESIGN OF HIGHWAY CULVERTS
FHWA-NHI-01-020

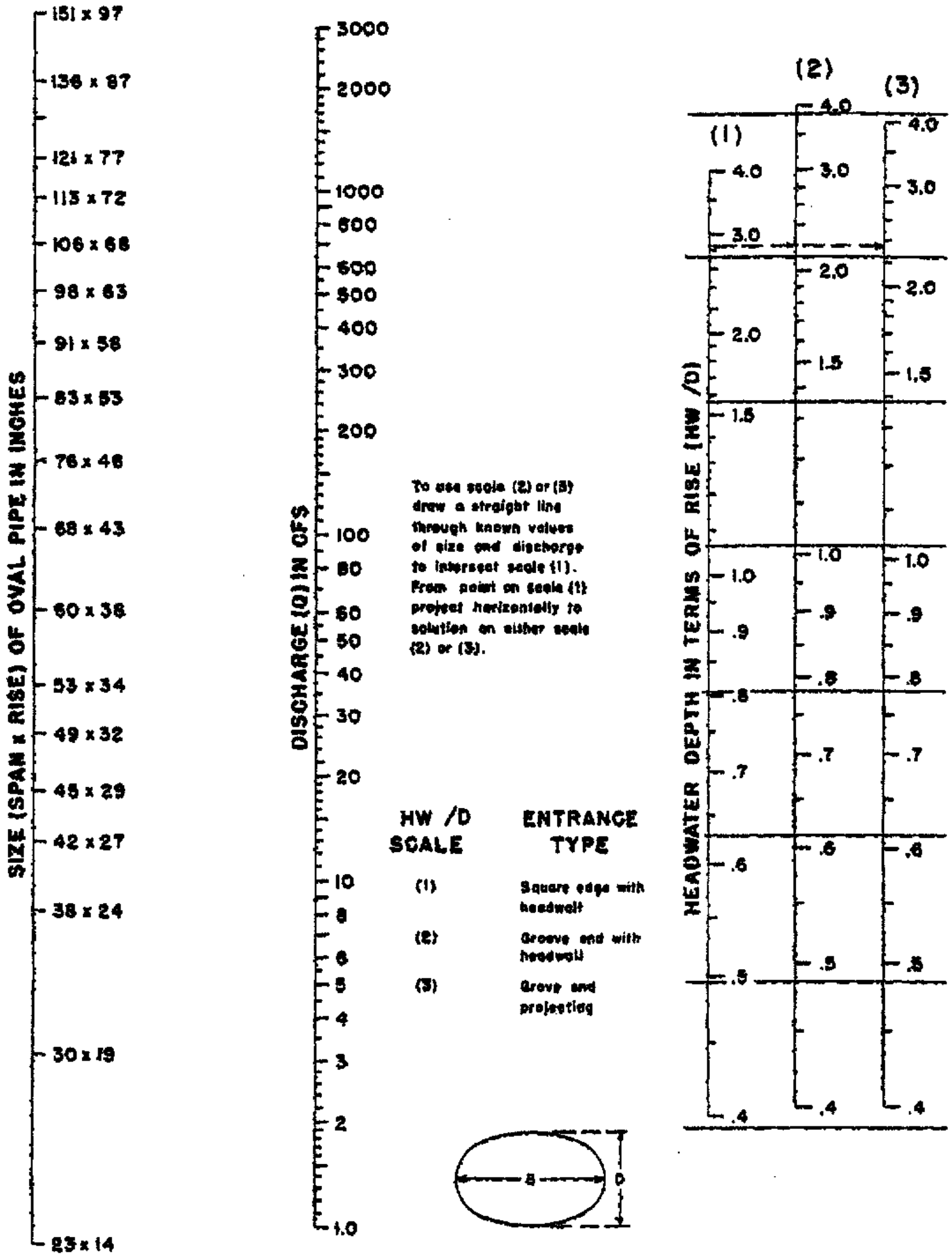
FIGURE CH6-F203
NOMOGRAPH-INLET CONTROL BOX CULVERT

COLORADO STATEWIDE
DRAINAGE AND FLOODPLAIN MANAGEMENT CRITERIA MANUAL

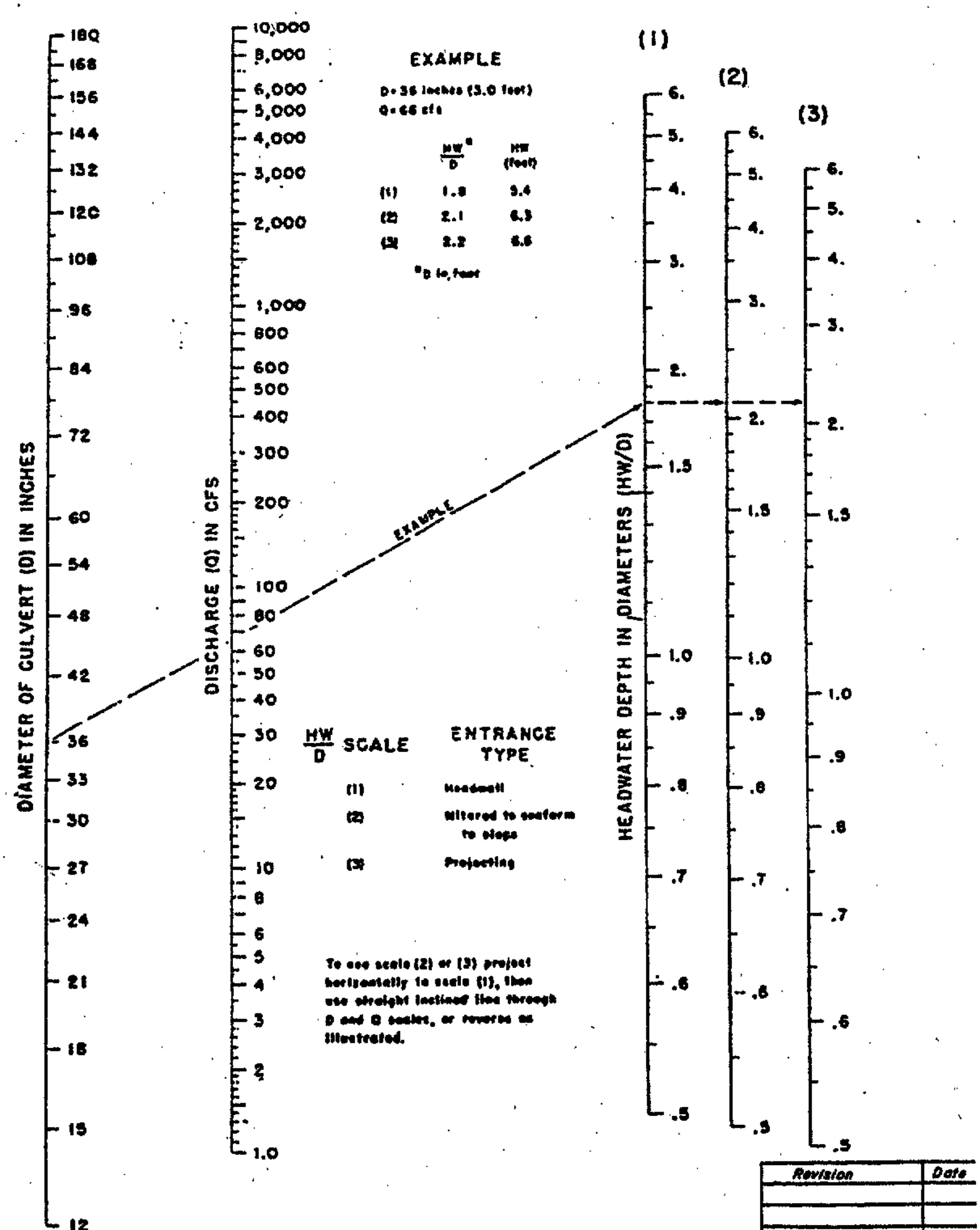


VERSION: AUGUST 2002	REFERENCE: HYDRAULIC DESIGN OF HIGHWAY CULVERTS FHWA-NHI-01-020	FIGURE CH6-F204
WRC ENGINEERING, INC.		NOMOGRAPH-INLET CONTROL RCP

O:/2080/FIGURES, CH6-F204.DWG, 4/10/02 MAS



O:/2080/FIGURES, CH6-F205.DWG, 4/10/02 MAS



Revision	Date

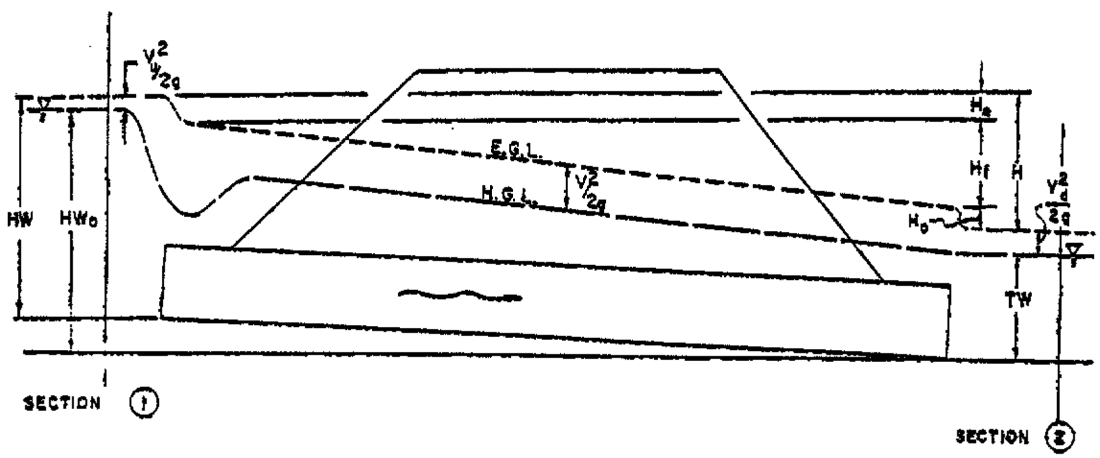
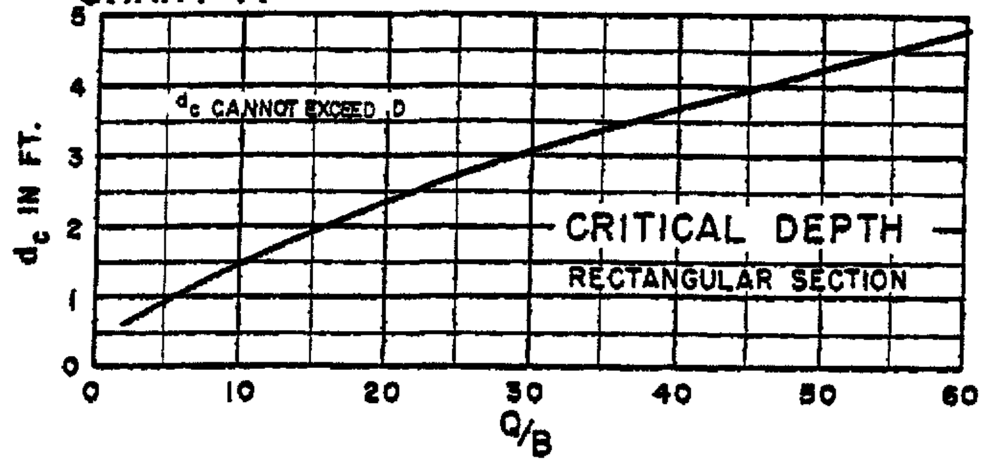
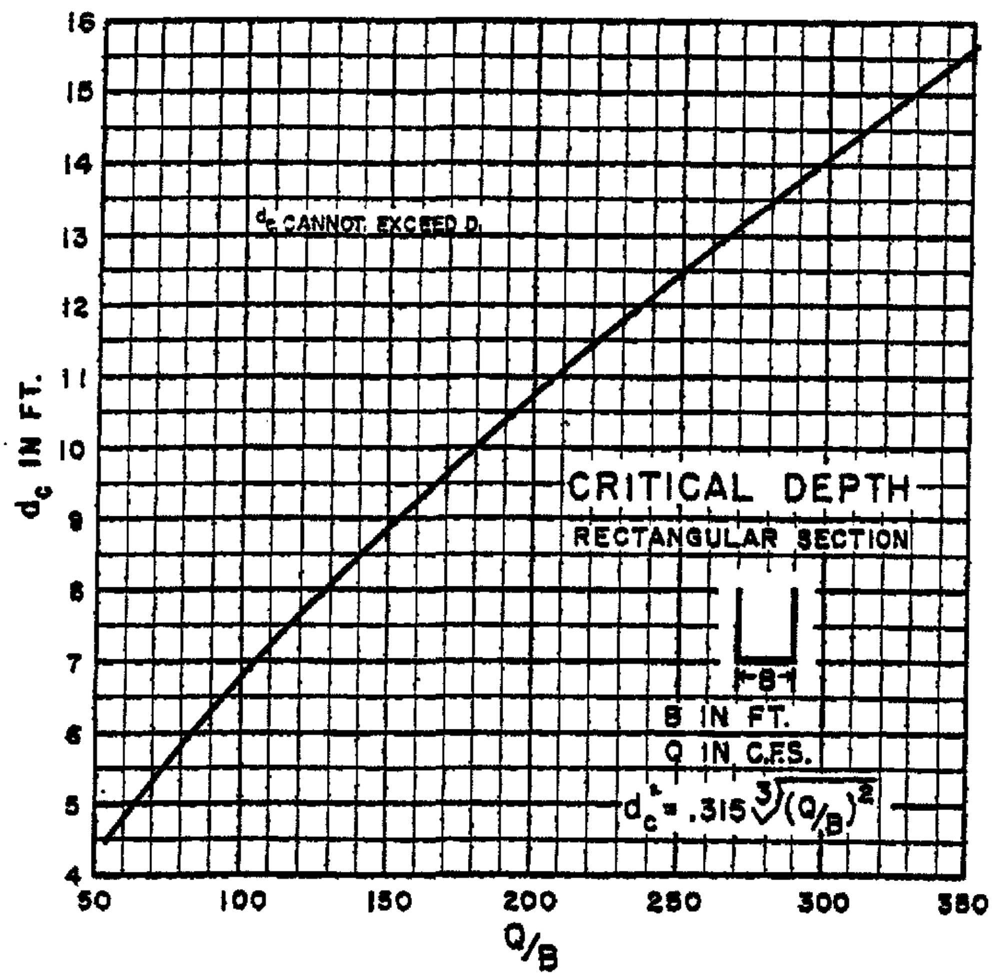


FIGURE CH6-F207
OUTLET CONTROL CULVERT HYDRAULICS

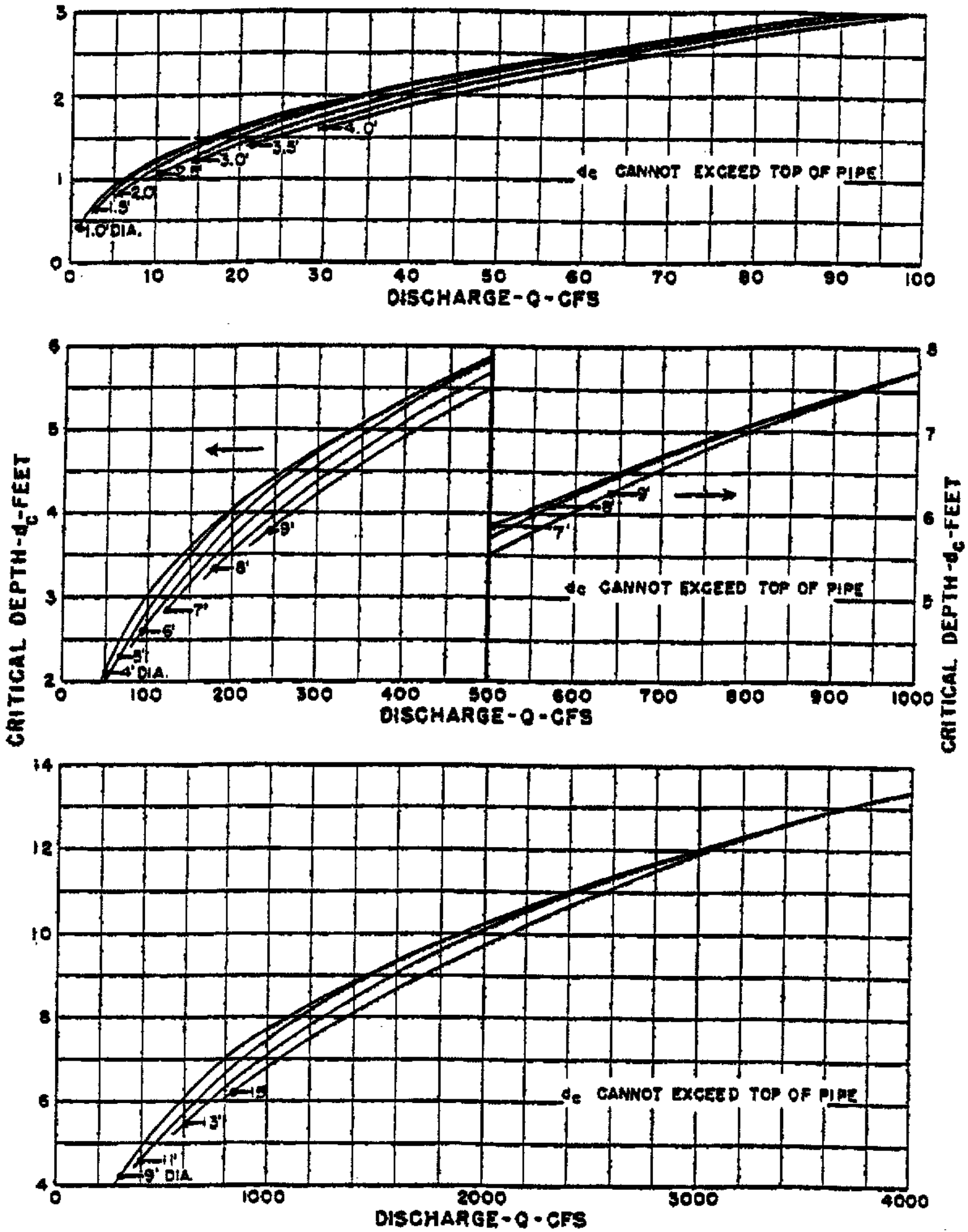
CHART 14





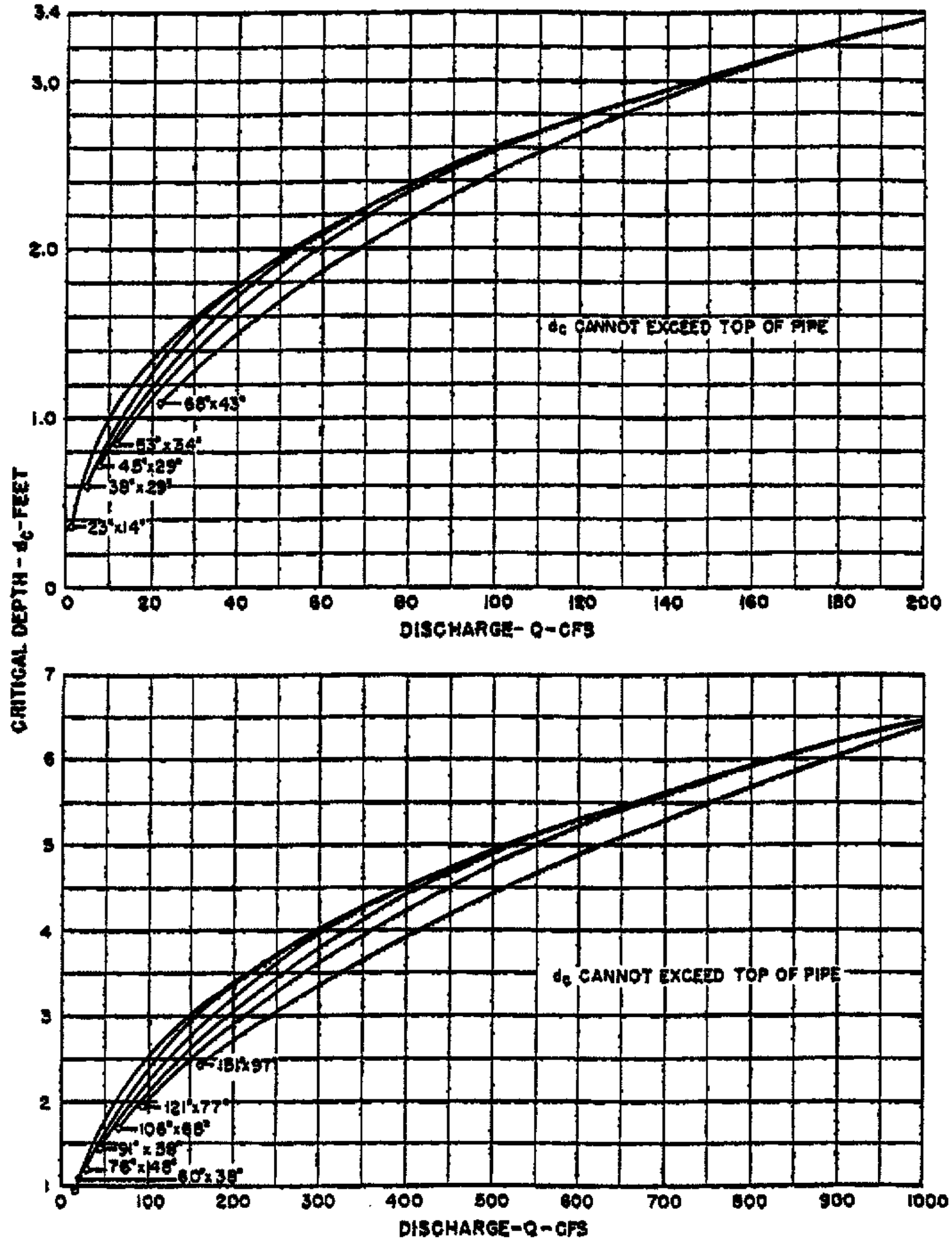
BUREAU OF PUBLIC ROADS JAN. 1963

CHART 4



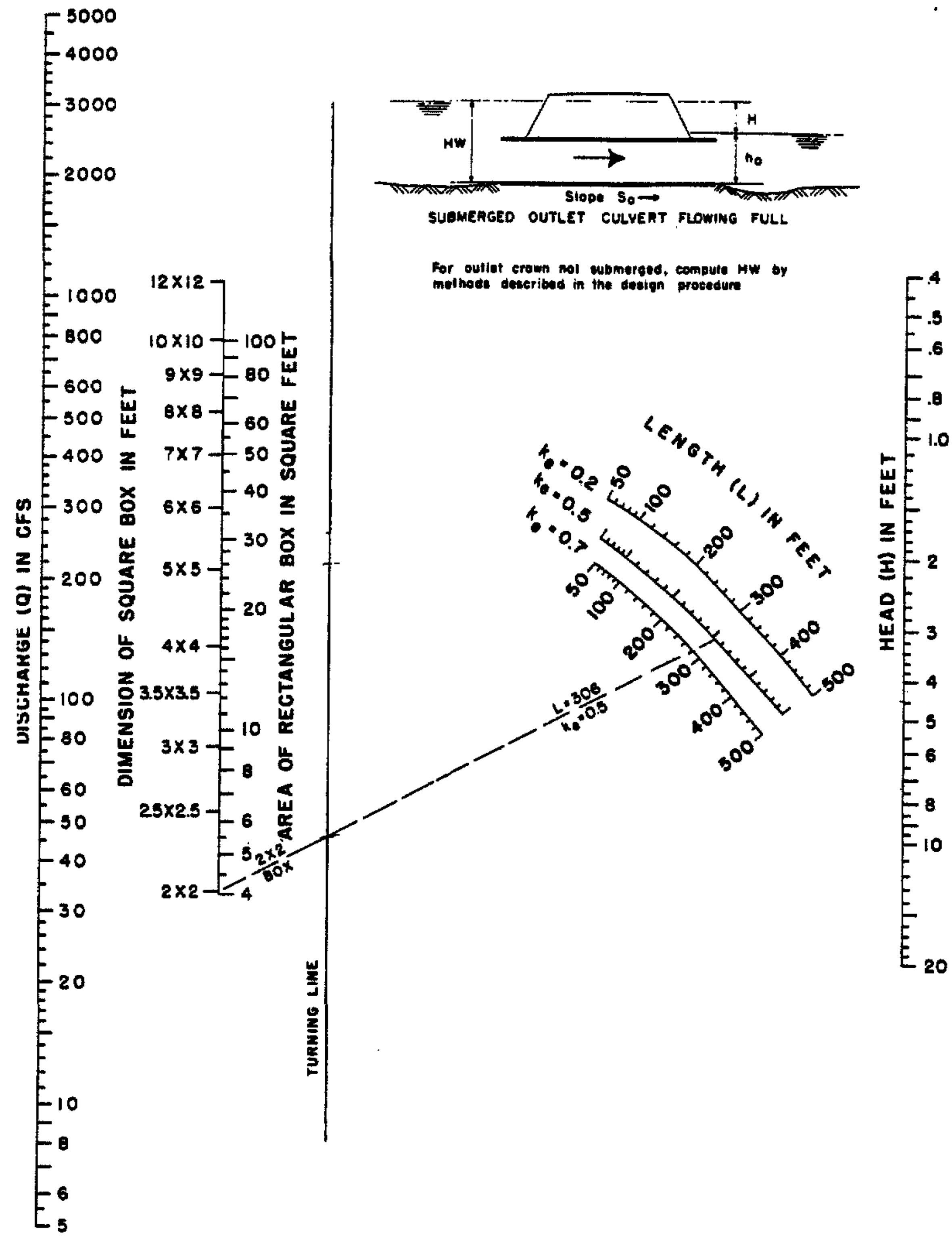
BUREAU OF PUBLIC ROADS
JAN. 1964

CHART 31



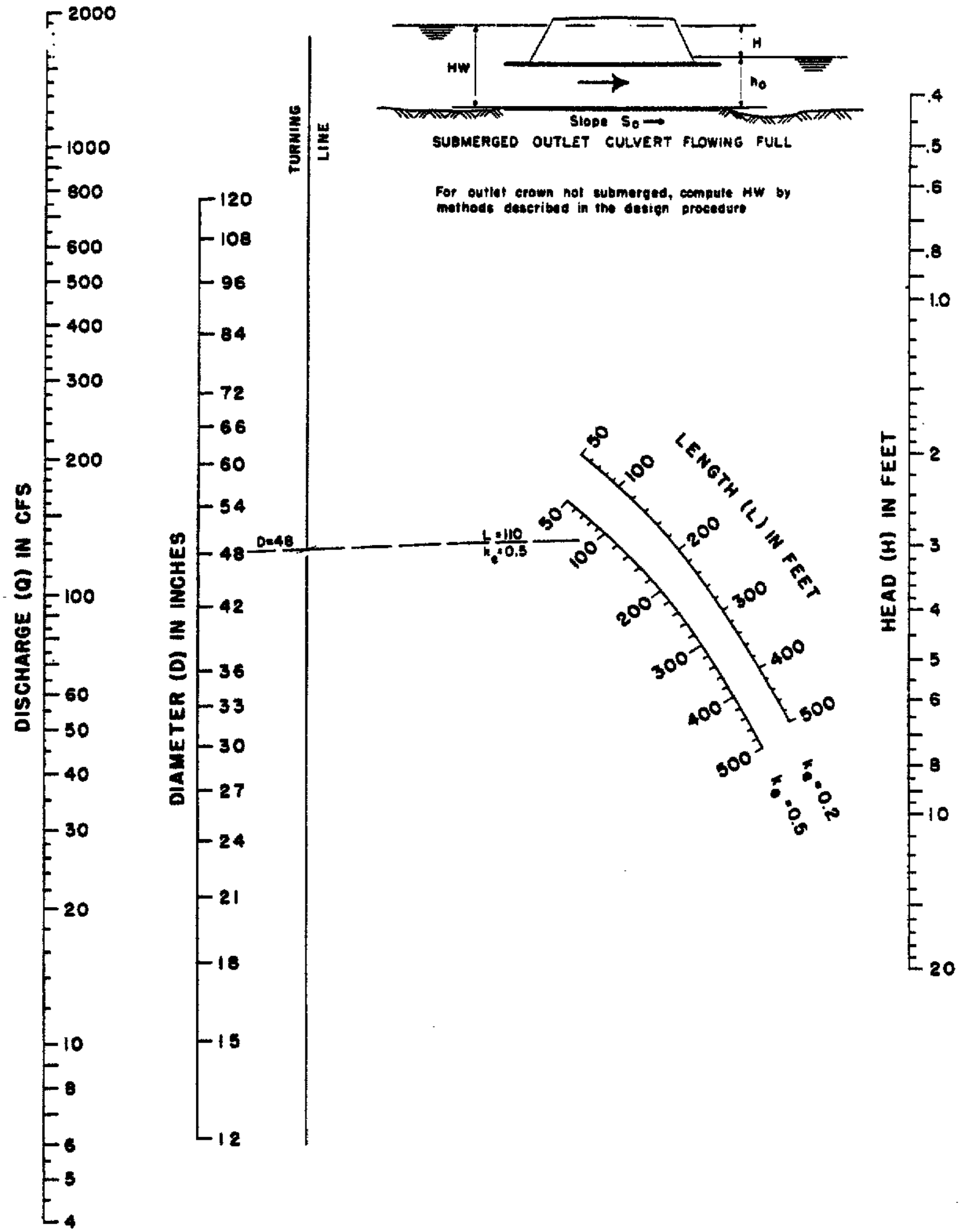
BUREAU OF PUBLIC ROADS
JAN. 1964

VERSION: AUGUST 2002	REFERENCE: HYDRAULIC DESIGN OF HIGHWAY CULVERTS FHWA-NHI-01-020	FIGURE CH6-F210 NOMOGRAPH-CRITICAL DEPTH ELLIPTICAL PIPE
WRC ENGINEERING, INC		

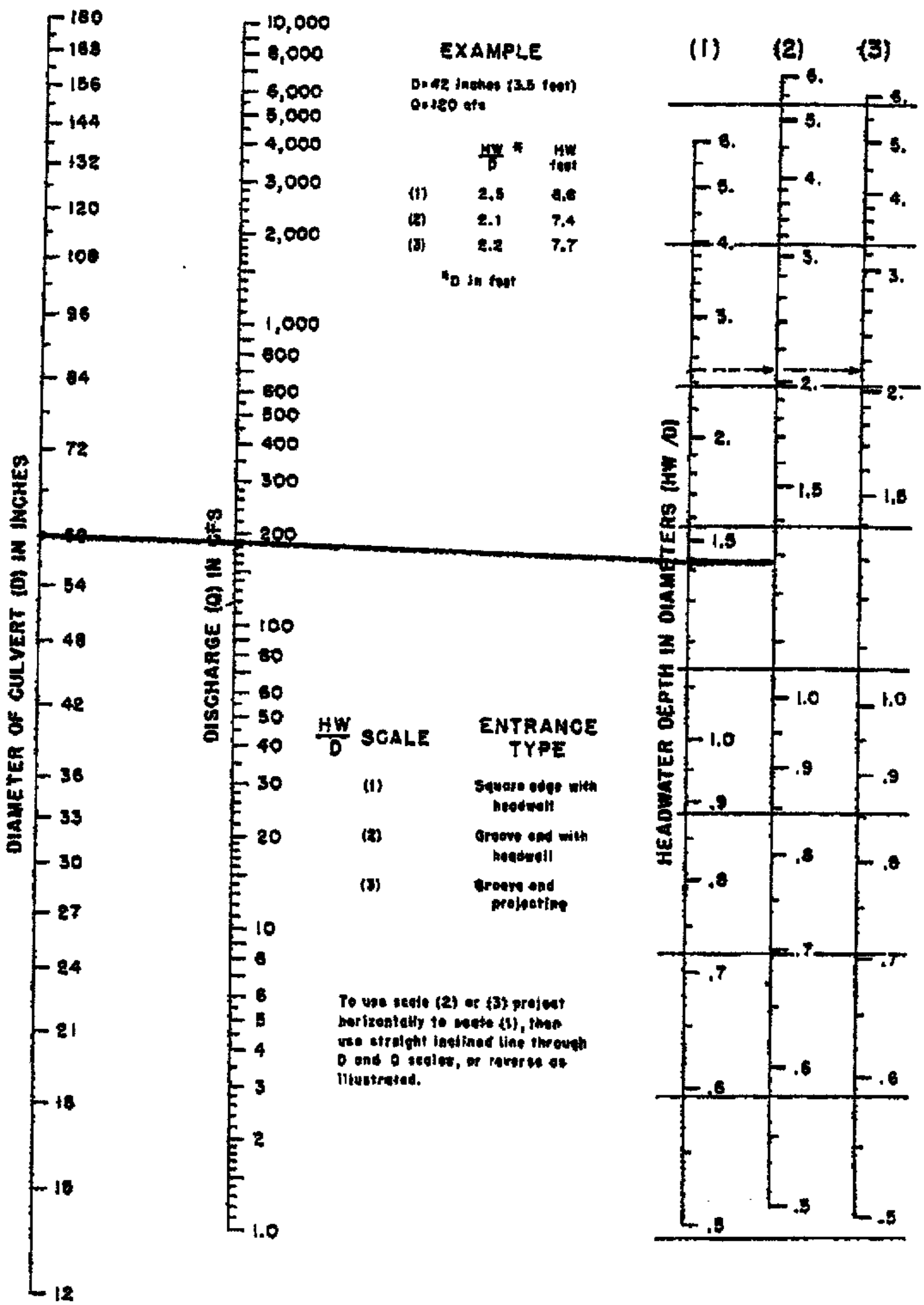


VERSION: AUGUST 2002	REFERENCE: HYDRAULIC DESIGN OF HIGHWAY CULVERTS FHWA-NHI-01-020	FIGURE CH6-F211 NOMOGRAPH-OUTLET CONTROL BOX CULVERT
WRC ENGINEERING, INC.		

O:/2080/FIGURES, CH6-F211.DWG, 4/10/02 MAS

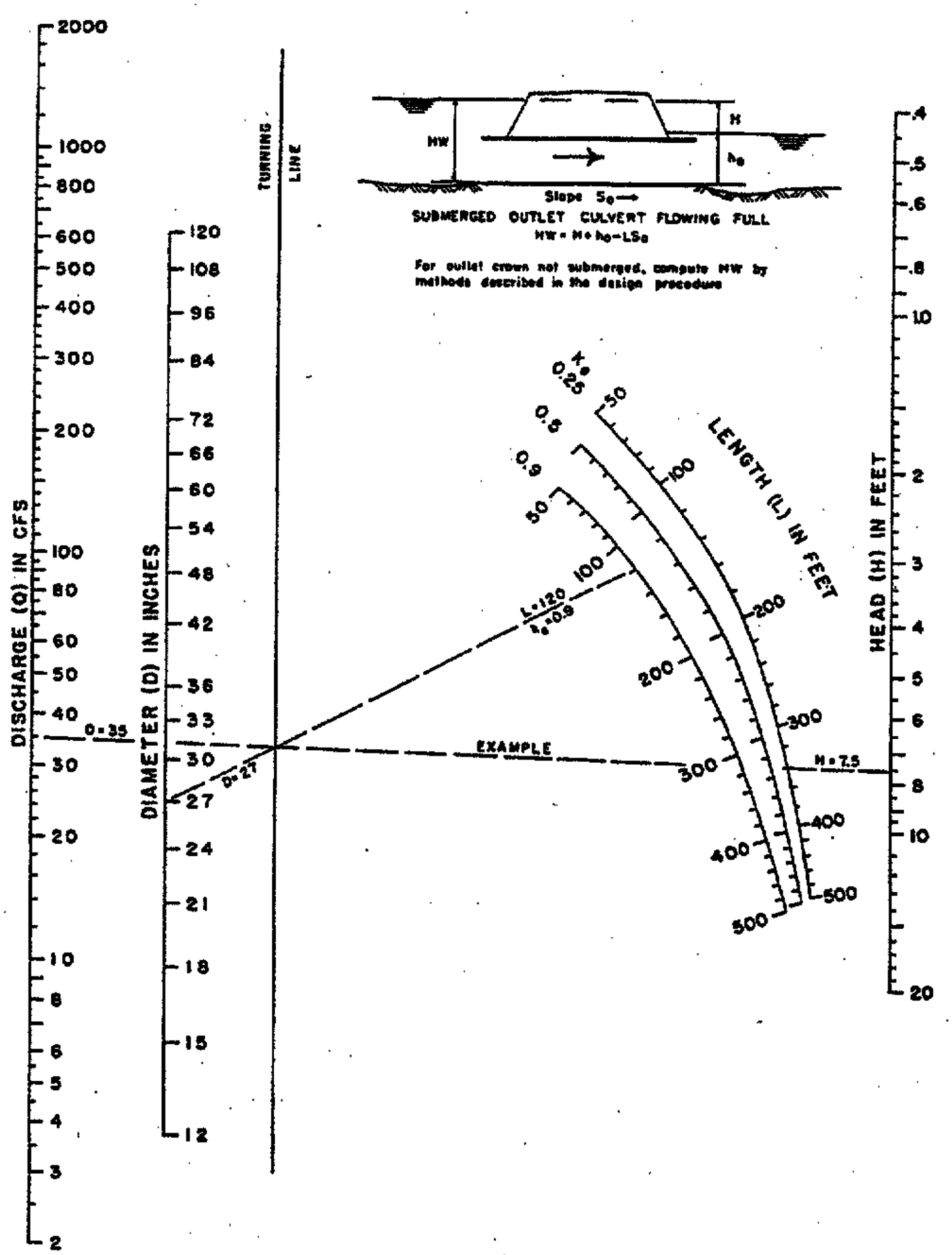


c:/2080/FIGURES, CH6-F212.DWG, 4/10/02 MAS



O:/2080/FIGURES, CH6-F213.DWG, 4/10/02 MAS

(n=0.024)

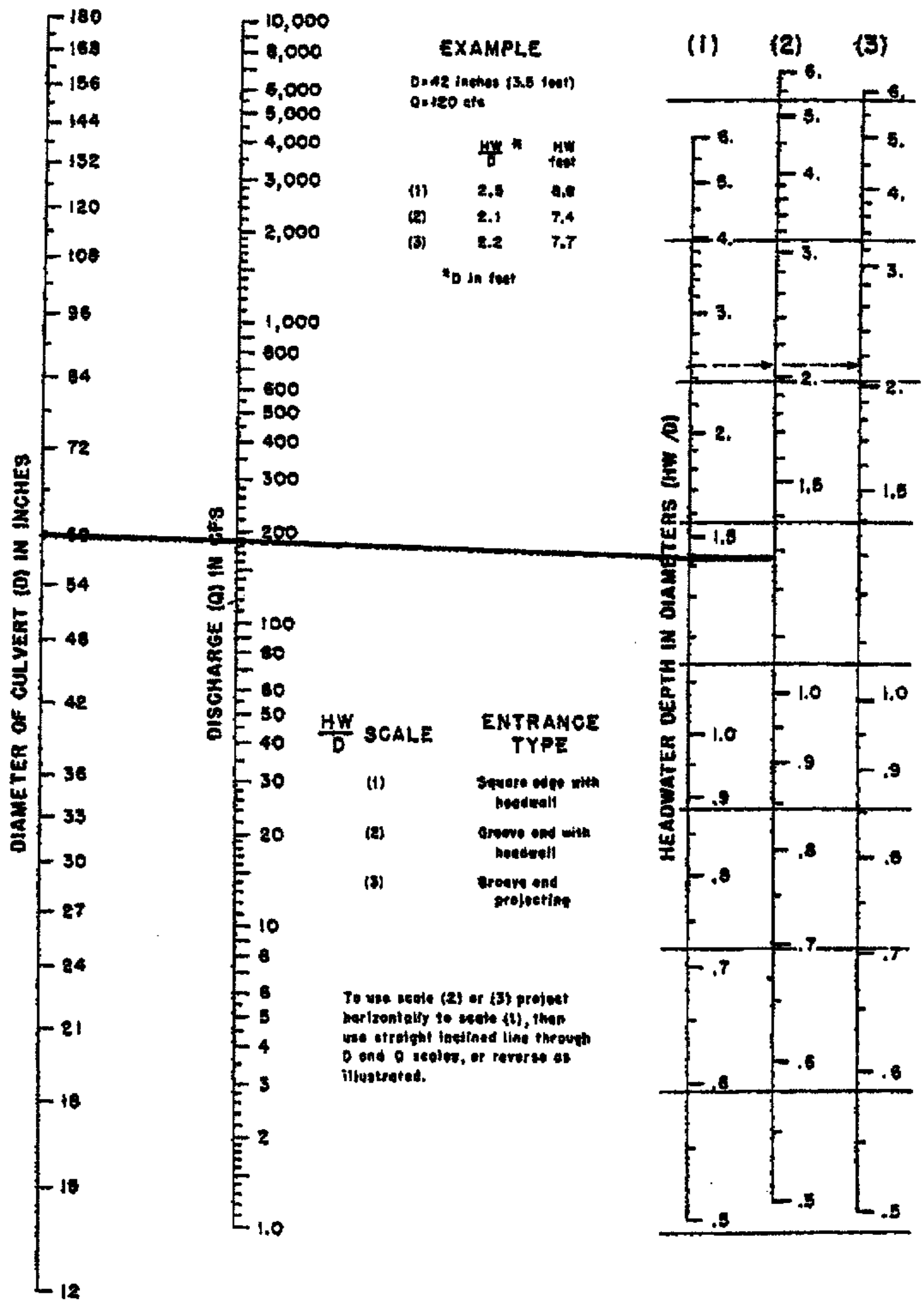


REFERENCE:
HYDRAULIC DESIGN OF HIGHWAY CULVERTS
FHWA-NHI-01-020

FIGURE CH6-F214

NOMOGRAPH-OUTLET CONTROL CMP

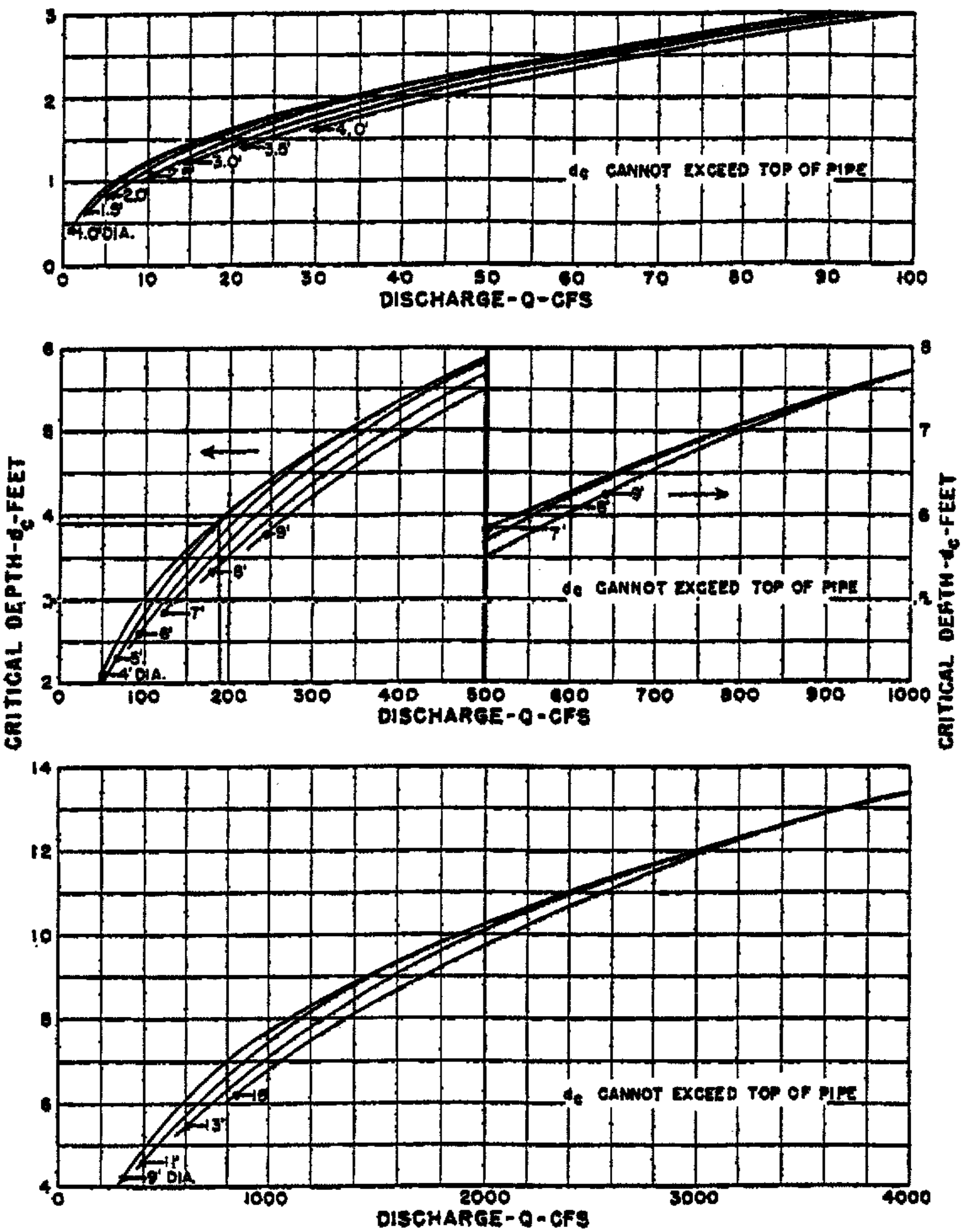
COLORADO STATEWIDE
DRAINAGE AND FLOODPLAIN MANAGEMENT CRITERIA MANUAL



VERSION: AUGUST 2002	REFERENCE: HYDRAULIC DESIGN OF HIGHWAY CULVERTS FHWA-NHI-01-020	FIGURE CH6-F215
WRC ENGINEERING, INC.		EXAMPLE: NOMOGRAPH-INLET CONTROL 5-FOOT DIAMETER RCP

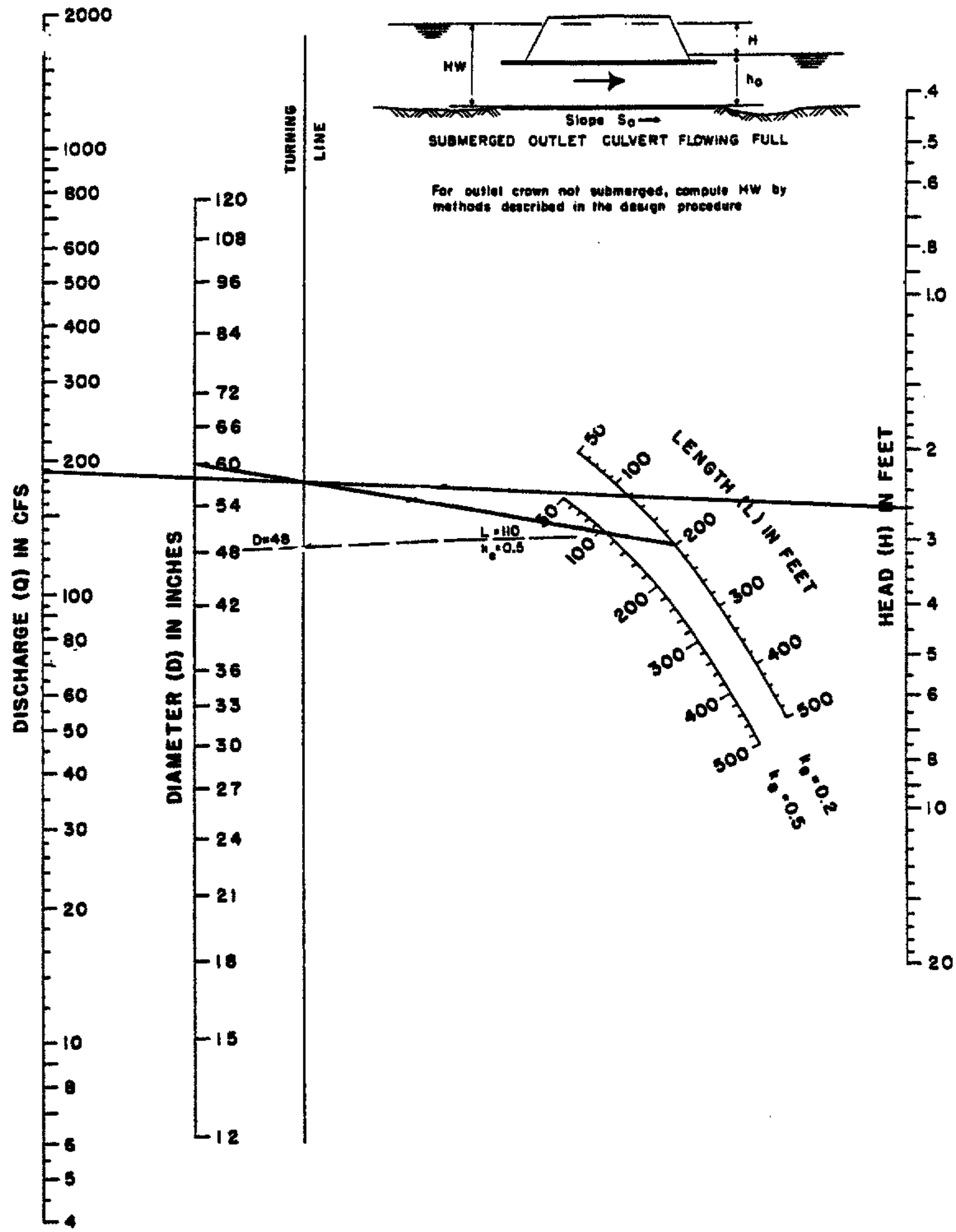
O:/2080/FIGURES, CH6-F215.DWG, 4/10/02 MAS

CHART 4



BUREAU OF PUBLIC ROADS
JAN. 1964

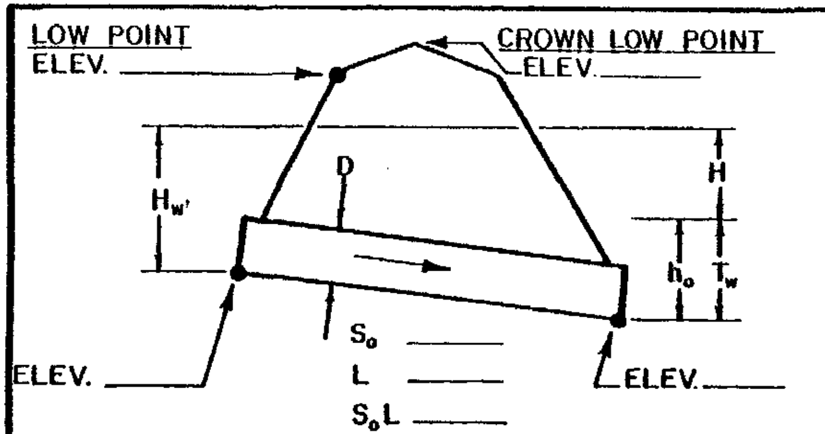
O:/2080/FIGURES, CH6-F216.DWG, 4/10/02 MAS



VERSION: AUGUST 2002	REFERENCE: HYDRAULIC DESIGN OF HIGHWAY CULVERTS FHWA-NHI-01-020	FIGURE CH6-F217 EXAMPLE: NOMOGRAPH-OUTLET CONTROL 5-FOOT DIAMETER RCP
WRC ENGINEERING, INC.		

WRC ENGINEERING, INC.
VERSION: AUGUST 2002
REFERENCE: WRC ENGINEERING, INC.

PROJECT: ______________ LOCATION: ______________ STATION: ________



CULVERT DATA

TYPE: ______________ n: ________
INLET: ______________ Q_{FULL}: ________
K_e: ______________ V_{FULL}: ________

OUTLET CONTROL EQUATIONS

(1) $H_w = H + h_o - LS_o$

(2) For $T_w < D$; $h_o = \frac{d_c + D}{2}$ or T_w (whichever is greater)

$T_w \geq D$; $h_o = T_w$

(3) For Box Culvert: $d_c = 0.315(Q/B)^{2/3} \leq D$

Q	STORM EVENT	INLET CONTROL		OUTLET CONTROL						TYPE OF CONTROL	CONTROL HEADWATER ELEVATION	OUTLET VELOCITY
						$T_w < D$		$T_w \geq D$				
		$\frac{H_w}{D}$	H_w	H	T_w	d_c	$\frac{d_c+D}{2}=h$	h_o	H_w			
1	2	3	4	5	6	7	8	9	10	11	12	13

FIGURE CH6-SF201
STANDARD CULVERT DESIGN FORM

CHAPTER 6

HYDRAULIC ANALYSIS AND DESIGN

SECTION 3.0

DAMS AND RESERVOIRS

CHAPTER 6
HYDRAULIC ANALYSIS AND DESIGN

SECTION 3.0
DAMS AND RESERVOIRS

TABLE OF CONTENTS

CHAPTER 6
HYDRAULIC ANAYLSIS AND DESIGN

SECTION 3.0
DAMS AND RESERVOIRS

3.1 INTRODUCTION

A dam is a man-made embankment that allows temporary or permanent impoundment of water above the natural ground, and a reservoir is a body of water (pond or lake) stored by a dam or a depression of natural ground. Dams and reservoirs can serve a single or multiple purposes including flood control, raw water supply (agricultural, municipal, and industrial), recreation, hydropower, environmental enhancement, water conservation, fish and wildlife, and others. Larger dams and reservoirs are usually designed to serve multiple purposes. Dams that serve a single purpose may include flood control dams, diversion dams, erosion control dams, and others.

This section is intended to provide practical guidelines for determining flood attenuation/storage benefits of dams and reservoirs for the purpose of determining downstream flow rates and associated floodplain boundaries. For detailed discussions on the design and analysis requirements for the jurisdictional and non-jurisdictional dams and reservoirs, please refer to the following publications:

- Office of State Engineer, State of Colorado, Rules and Regulations for Dam Safety and Dam Construction, September 30, 1988
- Office of State Engineer, State of Colorado, Dam Safety Project Review Guide, Third Revision June 1, 2000
- U.S. Department of Interior, Bureau of Reclamation, Design of Small Dams, 3rd Edition, 1987

3.2 STATE DAM SAFETY PROGRAM

Although properly designed dams and reservoirs can provide many great benefits to communities, the problems of dam safety and the related hazard of the emergency spillways have been brought to the attention of the public by many dam failures nationwide. In order to enhance the safety of dams in the State of Colorado, the authority was granted to the State Engineer (Colorado Dept. of Natural Resources, Division of Water Resources) to implement the dam safety program. The state dam safety program is administered through the implementation of "Rules and Regulations for Dam Safety and Dam Construction" (Dam Safety Rules) by the Dam Safety Branch of the Division of Water Resources. The Dam Safety Rules apply to all

> *Dams within the State of Colorado are classified as either "Jurisdictional Dams" or "Non-jurisdictional Dams" by the State Engineer's office based on the height of the embankment above the natural ground, the surface area of the reservoir, or the total reservoir storage capacity.*

dams within the State of Colorado that are constructed and/or operated for the purpose of storing water temporarily or permanently.

Dams within the State of Colorado are classified as either "Jurisdictional Dams" or "Non-jurisdictional Dams" by the State Engineer's office based on the height of the embankment above the natural ground, the surface area of the reservoir, or the total reservoir storage capacity. All existing and new dams meeting the criteria outlined below are classified as "Jurisdictional Dams" and those that don't meet the criteria are classified as "Non-jurisdictional Dams".

" A "Jurisdictional Dam" is a dam which impounds water above the elevation of the natural surface of the ground creating a reservoir with a capacity of more than 100 acre-feet, or creates a reservoir with a surface area in excess of 20 acres at the high-water line, or exceeds 10 feet in height measured vertically from the elevation of the lowest point of the natural surface of the ground where that point occurs along the longitudinal centerline of the dam up to the flowline crest of the emergency spillway of the dam. (Dam Safety Rule 4.A. (6))"

3.2.1 CLASSFICATION OF DAMS

Dams are categorized by the State Engineer into four classes based on the potential damages to properties and human lives resulting from failure of a dam assuming the reservoir is full to the crest of the emergency spillway.

- Class I Dam – A dam for which loss of human life is expected in the event of failure of the dam (Dam Safety Rule 4.A. (5)).
- Class II Dam - A dam for which significant damage is expected to occur, but no loss of human life is expected in the event of failure of the dam. Significant damage is defined as damage to structures where people generally live, work, or recreate, or public or private facilities exclusive of unpaved roads and picnic areas. Damage means rendering the structures uninhabitable or inoperable (Dam Safety Rule 4.A. (5)).
- Class III Dam – A dam for which loss of human life is not expected, and damage to structures and public facilities will not be significant in the event of failure of the dam (Dam Safety Rule 4.A. (5)).
- Class IV Dam – A dam for which no loss of human life is expected, and which damage will occur only to the dam owner's property in the event of failure of the dam (Dam Safety Rule 4.A. (5)).

Dams are also categorized as minor, small, intermediate, or large structures depending on the height of embankments or the storage capacity. Dams may be re-categorized, if developments occur within the dam failure hazard areas or as modifications to existing dam structures occur.

An applicant proposing to construct or modify a jurisdictional dam is required to obtain approval from the State Engineer's Office based on their submittal guidelines, prior to beginning construction.

3.3 JURISDICTIONAL DAMS

An applicant proposing to construct or modify a jurisdictional dam is required to obtain approval from the State Engineer's Office based on their submittal guidelines, prior to beginning construction. The design, analysis, construction, maintenance, and submittal guidelines for jurisdictional dams are specified in the State Engineer's Dam Safety Rules. The spillway capacity requirement and the recommended hydrologic analysis method for jurisdictional dams are also specified.

The list and description of structures exempted from the State rules and regulations are provided in Rule 18, Dam Safety Rules. The applicant should verify the current Dam Safety Rules as the rules may change from time to time.

3.4 NON-JURISDICTIONAL DAMS

Although smaller than jurisdictional dams, depending on the location of the dam structure, a non-jurisdictional dam failure can result in substantial damages to properties and even loss of human lives. The applicant proposing to construct a non-jurisdictional dam should notify the State Engineer's Office at least 10-days prior to construction using the forms provided by the State. It should be noted that the State Engineer might require that a non-jurisdictional or exempted dam to be designed based on the same design guidelines for a jurisdictional dam, if the site-specific conditions warrant such requirements. It is recommended that the project engineer coordinate with the State Engineer's Office during the early phase of the design to determine the appropriate design criteria for the dam.

In general, non-jurisdictional dams are not required to comply with the jurisdictional dam design criteria provided in the State Engineer's Dam Safety Rules. However, all non-jurisdictional dams should be designed and constructed to safely collect and store the design flows without structural failures. Emergency spillways should be provided to control and confine the overflows. The design elements including, but not limited to, protection of embankment slopes, primary and emergency spillways, stability of embankment and foundation, seepage, compaction of fill, potential settlement, and maintenance access should be addressed. It is the design engineer's responsibility to design the dam to withstand the hydraulic, seismic and other loadings and to ensure the stability of the dam. The readers of this manual are encouraged to review the following publications in addition to the State Dam Safety Rules for detailed dam design guidelines:

- U.S. Department of Interior, Bureau of Reclamation, Design of Small Dams, 3rd Edition, 1987
- U.S. Army Corps of Engineers, Engineering and Design, EM 1110-2-1603, Hydraulic Design of Spillways, January 1990
- U.S. Army Corps of Engineers, Engineering and Design, EM 1110-2-2300, Earth and Rock-Fill Dams – General Design and Construction Considerations, September 1986

3.4.1 HYDROLOGIC ANALYSIS

For detailed discussions on the rainfall-runoff analysis methods and procedures, please refer to Chapter 5, Hydrology.

3.4.2 SPILLWAYS

Emergency spillways should be provided to control and confine the overflows. Spillways should be sized, as a minimum, to handle the 100-year peak flows with a minimum freeboard of one foot.

3.4.3 DAM EMBANKMENT

The minimum top width of a fully compacted earthen dam embankment should be 12 feet and the side slopes shall not be steeper than 3H:1V. Steeper embankment side slopes may be allowed only if the design engineer can demonstrate the stability of embankments and foundations based on acceptable engineering analyses. However, under no circumstances, should an embankment side slope steeper than 2H:1V be used. All dam structures should be designed to minimize required maintenance and to allow access by equipment and workers to perform maintenance.

Spillways should be sized, as a minimum, to handle the 100-year peak flows with a minimum freeboard of one foot.

The minimum top width of a fully compacted earthen dam embankment should be 12 feet and the side slopes shall not be steeper than 3H:1V.

3.5 FLOOD CONTROL DAMS

Dams and reservoirs can be designed to help reduce the downstream flooding by capturing and storing a portion of or the entire design storm runoff from the upstream watersheds. Dams and reservoirs designed to provide flood protection for the downstream area must have the required floodwater storage capacity reserved, and the dam operation plan should clearly identify the flood control regulation purpose of the dam.

3.5.1 DETENTION DAMS

The majority of flood control dams are designed to detain flood flows and limit the peak outflows to the downstream receiving drainage facilities. The main purpose of a detention dam facility is to temporarily impound runoff behind the dam and reduce the downstream flow rate by allowing flows to be discharged through the primary spillway (usually a culvert) at a controlled outflow rate. The controlled outflow rate is usually determined based on either the downstream receiving facility conveyance capacity or a limit on the increase in flows over pre-development conditions. However, unless an agreement can be reached with the downstream water rights holders, flood detention dam outlets should be sized to drain the stored floodwater within 24 hours of a storm event.

Unless an agreement can be reached with the downstream water rights holders, flood detention dam outlets should be sized to drain the stored floodwater within 24 hours of a storm event.

The controlled detention dam outlet capacity has direct influence on the required size of the detention dam. For a given design storm event, the smaller the outlet capacity, the larger the required storage capacity of a dam. For detailed discussions on the design requirements of detention basins, please refer to Chapter 6, Section 5.

3.5.2 RETENTION DAMS

Depending on the flow conveyance capacity of the downstream drainage facility and site-specific conditions, it may be necessary to design a retention dam to capture and store the entire design storm runoff. Retention dams can be designed to either permanently or temporarily store the runoff from the upstream drainage basins.

A retention dam can be designed with a controlled outlet gate to capture the entire runoff, and later release the stored water at a controlled rate when the downstream facility can safely convey the outflows from the dam. Retention dams designed to permanently store the storm runoff are not desirable for the flood control purposes since the storage capacity available for back-to-back storm events will be minimized.

The design and construction of retention dams should not adversely impact the water rights of downstream users, unless an agreement can be reached with all impacted downstream users. Further, retention dams must have valid storage rights that can be exercised to store water when such rights are in priority.

3.6 ROADWAY AND RAILROAD EMBANKMENTS

Intentionally or unintentionally, some roadway and railroad embankments are used to store flood flows behind the embankments during storm events. Depending on the topography of the site, size of culverts, and the total runoff from the upstream drainage basins, the depth and/or the amount of water temporarily impounded behind the embankment may exceed the non-jurisdictional dam size limit.

The use of roadway and railroad embankments for flood detention purposes is exempted from the State Engineer's Dam Safety Rules. However, if the embankment height or the storage capacity meets the State's definition of "Jurisdictional Dams", the project engineers should coordinate with the State Engineer's Office during the early phase of the design to determine the appropriate design criteria for the roadway/railroad embankment.

3.7 AREAS PROTECTED BY DAMS AND RESERVOIRS

Properly designed and maintained dams and reservoirs can significantly reduce the downstream flooding problems by capturing and storing a portion of or the entire design storm runoff from the upstream watersheds. The flood attenuation and storage benefits of a dam or a reservoir should be included in the hydrologic and hydraulic analysis of the downstream drainageway, if the dam/reservoir is:

The flood attenuation and storage benefits of a dam or a reservoir should be included in the hydrologic and hydraulic analysis of the downstream drainageway, if the dam or reservoir is:

- *Owned, operated, and maintained by a public agency*
- *Designed and operated, either in whole or in part, for flood control purposes*

- Owned, operated, and maintained by a public agency
- Designed and operated, either in whole or in part, for flood control purposes

Dams and reservoirs constructed for other purposes (i.e. gravel pits, water supply reservoirs, etc.) may provide flood protections for the downstream areas inadvertently. However, the available flood storage capacity of these dams cannot be relied upon, since the flood storage availability cannot be guaranteed. Dams and reservoirs not specifically designed and operated, either in whole or in part, for flood control purposes should not be included in the hydrologic and hydraulic analysis of the downstream drainageway unless such a dam/reservoir aggravates downstream flooding conditions. The downstream peak flow rates and floodplain boundaries should be determined assuming such a dam/reservoir does not exist.

However, if adequate assurances have been obtained to preserve the flood routing capabilities of such a dam, then the delineation of the floodplain below the dam may, but need not, be based on the assumption that the reservoir formed by the dam will be filled to the elevation of the dam's emergency spillway. The project engineer should coordinate with appropriate government agencies and CWCB in determining whether a non-flood control dam should be included in the analysis or not.

3.7.1 STORAGE ROUTING METHOD

The flow attenuation effect of a dam/reservoir can be determined using the Modified Puls Routing Method. The Modified Puls Routing Method can be used in HEC-1, HEC-HMS, and UDSWM computer programs to route hydrographs through dams and reservoirs. Only the storage specifically reserved for the flood attenuation purposes should be included in the analysis.

Detailed discussions on the Modified Puls routing method and the use of HEC-1 and CUHP computer programs are provided in Chapter 5, Section 2.

CHAPTER 6

HYDRAULIC ANALYSIS AND DESIGN

SECTION 4.0

LEVEES

CHAPTER 6
HYDRAULIC ANALYSIS AND DESIGN

SECTION 4.0
LEVEES

TABLE OF CONTENTS

LIST OF FIGURES

CHAPTER 6
HYDRAULIC ANAYLSIS AND DESIGN

SECTION 4.0
LEVEES

4.1 INTRODUCTION

A levee is a man-made embankment that can provide flood protection from occasional flood events up to and including the duration and magnitude of the design storm event. Typically, levees are designed to provide flood protection from an estimated 100-year storm event and only for a short period of time. Levees are normally not designed to provide flood protection for a prolonged period.

The use of levees for flood control and flood mitigation projects is not encouraged by the CWCB. The CWCB does not endorse the use of levees as a form of floodplain reduction for areas along streams where new development is planned.

The use of levees for flood control and flood mitigation projects is not encouraged by the CWCB, unless other mitigation alternatives are not viable or cost effective. Setback levees should be designed whenever possible to maintain the natural channel and some natural floodplain areas. The CWCB does not endorse the use of levees as a form of floodplain reduction for areas along streams where new development is planned.

Presented in this section are the general criteria and standards for the hydraulic analysis and design of earthen levees. There are many factors, which must be considered in the design of earth levee systems and these factors differ substantially from one project site to another. The site-specific geological, hydraulic, environmental, and other design factors should be identified and incorporated into the levee design. The following is a short list of some of the levee design factors that must be considered:

- Design peak flow rate, duration of flood, and water surface elevations
- Flow velocity
- Embankment height and freeboard
- Opening closures (culverts, etc.)
- Interior drainage
- Embankment erosion protection
- Embankment and foundation stability
- Under and through seepage
- Settlement
- Other site-specific factors
- Operations and maintenance

4.1.1 LEVEE FAILURE

Throughout the United States, levees are often used to protect properties within and adjacent to the natural floodplains. Properly designed levee systems can effectively provide the designed flood protection for many communities. However, levees are rarely designed to provide flood protection for storm events greater than a 100-year event. Since flood events greater than the 100-year event can and do occur (with less probability), the levees designed to provide a 100-year flood protection can be overtopped, increasing the possibility of levee failure.

Due to the lack of adequate levee maintenance, improperly designed levee embankments and foundations, and the occurrence of storm events greater than the design event, many levee failures have occurred throughout the United States. The most common reasons of levee failures are:

- Embankment erosion and scour
- Levee overtopping
- Seepage and piping

Levee failures usually result in great flood losses for many communities. The importance of proper levee design practices, adequate ongoing maintenance and operations, and early flood warning programs cannot be over emphasized.

4.2 EMBANKMENT AND FOUNDATION DESIGN

The primary purpose of levees is to provide flood protection from flood events that occur infrequently for short durations of time. Therefore, the levee embankment and foundation are typically designed to withstand the continuous hydraulic forces for periods up to just a few days. If the site and flood conditions require the earthen levee to withstand the hydraulic loading for an extended period, the levee embankment and foundation should be designed in accordance with the design criteria outlined for earthen dams (Chapter 6, Section 3).

The minimum top width of 12 feet for a fully compacted earthen levee embankment is recommended and shall not be less than 10 feet under any conditions. The earthen levee embankment side slopes shall not be steeper than 3H:1V.

The design of all levee embankments and foundations should be in accordance with the guidelines established by Federal Emergency Management Agency (FEMA) and the US Amy Corps of Engineers. Levee design elements including, but not limited to, closures of openings, protection of embankment slopes, stability of embankments and foundations, compaction of fill, and potential settlement should be addressed. The readers of this manual are referred to the following levee design publications for detailed design guidelines:

- US Army Corps of Engineers, Engineering and Design, Design and Construction of Levees, Engineer Manual, EM 1110-2-1913, April 2001.
- US Army Corps of Engineers, Engineering and Design, Settlement Analysis, Engineer Manual, EM 1110-1-1902, September 1990.

- Federal Emergency Management Agency, NFIP Laws and Regulations, Title 44, Part 65, Section 65.10, Mapping of Areas Protected by Levee Systems, revised October 1999.

The minimum top width of 12 feet for a fully compacted earthen levee embankment is recommended and shall not be less than 10 feet under any conditions. The earthen levee embankment side slopes shall not be steeper than 3H:1V. Embankment side slopes flatter than 3H:1V may be necessary depending on the site-specific design conditions. Steeper embankment side slopes may be allowed only if the design engineer can demonstrate the stability of embankments and foundations based on appropriate engineering analyses. However, under no circumstances, should an embankment side slope steeper than 2H:1V be used.

4.2.1 EMBANKMENT PROTECTION

Levee embankments should be protected against erosion and scour problems associated with a 100-year flood event. The following is a list of some of the general factors that should be addressed in the design of embankment protections:

- Flow velocities
- Channel migration
- Sediment and debris loading
- Embankment and foundation materials
- Duration and depth of flooding
- Embankment alignments
- Transitions and bends
- Embankment widths and side slopes

If possible, environmentally friendly erosion protection measures (e.g., grass cover or grass cover with a geo-mat under layer, etc.) should be used. Please refer to Chapter 6, Section 1 for detailed discussions on the allowable maximum flow velocities of various materials and the design procedures for erosion protection measures.

It is important to evaluate the flow velocities associated with smaller storm events since these events may produce higher flow velocities, especially where flows are constricted by structures including culverts and bridges.

4.2.2 SETTLEMENT

Potential levee settlement should be evaluated and addressed during the levee design, especially when the embankment and foundation materials contain highly compressible soils. The detailed settlement analysis procedures can be found in the Army Corps of Engineers, Settlement Analysis, Engineer Manual, EM 1110-1-1902, dated September 1990.

The estimated settlement amount should be incorporated into the top of the levee grade to ensure the required freeboard will be maintained after settlement has occurred.

4.3 DESIGN TOP OF LEVEE ELEVATION

Levees in the State of Colorado shall be designed to safely confine and convey, at a minimum, peak flows associated with a 100-year flood event. The final design top of levee elevations should be set to include the required minimum freeboard.

Levees in the State of Colorado shall be designed to safely confine and convey, at a minimum, peak flows associated with a 100-year flood event. The detailed procedures for determination of the 100-year peak flow rate for a design point are provided in Chapter 5 - Hydrology.

Once the design 100-year hydrograph has been determined, appropriate hydraulic analyses should be performed to establish the 100-year design water surface profile based on the proposed levee alignments and channel configurations. It may be necessary to perform several iterations of hydraulic modeling in order to refine the levee alignment and design. The Army COE hydraulic computer programs HEC-RAS and HEC-2 are recommended for determination of the design water surface profiles.

The top of levee embankment grades should be set sufficiently above the calculated design water surface elevations to account for the uncertainties in design peak flow rates, water surface elevations, settlements, and other unforeseen site conditions. The final design top of levee elevations should be set to include the required minimum freeboard. A deterministic risk and uncertainty analysis can be performed to directly account for hydraulic and design uncertainties and to set the top of levee grades instead of utilizing the required freeboard.

4.3.1 FREEBOARD

The following levee freeboard requirements should be used for levees within the State of Colorado. The freeboard criteria are consistent with the FEMA requirements at the time of publication.

"Levees must provide a minimum freeboard of three feet above the water-surface level of the base flood (100-year). An additional one foot above the minimum is required within 100-feet in either side of structures (such as bridges) riverward of the levee or wherever the flow is constricted. An additional one-half foot above the minimum at the upstream end of the levee, tapering to not less than the minimum at the downstream end of the levee, is also required. (FEMA NFIP 44CFR65.10)"

If the site conditions prevent conformance to the above minimum levee freeboard requirements, lesser freeboard may be allowed in accordance with the FEMA criteria set forth in NFIP 44CFR65.10. A CLOMR (see Chapter 4, Section 3) may be required before the use of lesser freeboard is accepted. However, freeboard of less than 2 feet will not be allowed under any circumstances.

For levees of small drainage-ways with the 100-year design peak flow rate of 100 cfs or less, the minimum required levee freeboard should be 2 feet.

4.3.2 RISK AND UNCERTAINTY ANALYSIS

In place of utilizing the freeboard requirements outlined above, a deterministic risk and uncertainty analysis may be performed to directly account for uncertainties associated with hydrology, hydraulic analysis, and embankment and foundation design. The analysis can be used to directly establish the design top of levee profile.

Currently, the Army Corps of Engineers does not use the freeboard concepts for design of their levee projects. The readers of this manual are referred to the following publications for detailed discussions on the risk and uncertainty analysis.

- US Army Corps of Engineers, Water Resources Support Center, Institute for Water Resources, Guidelines for Risk and Uncertainty Analysis in Water Resources Planning, Volumes I and II, March 1992.

4.4 INTERIOR DRAINAGE

Interior drainage systems should be designed to minimize human intervention, and backup systems should be provided to the extent feasible.

The areas protected by levees may still experience flooding from other sources including runoffs from local drainage basins and backwater through levee openings. Since the levee embankments are usually higher than the adjacent protected areas, the runoff from the local interior drainage basins cannot surface drain into the channel/river on the other side of the levee. Also, during a flood event, underground storm drain outlets will be closed to prevent backflows from the channel/river, again preventing discharge of local runoffs into the channel.

Interior drainage systems should be provided to drain flows from the local drainage basins into the channel/river during flood events. An interior drainage system associated with a levee system may include, but is not limited to, temporary flow retention areas with controlled outlets, various pump stations, gravity outlets to a downstream channel location, or a combination thereof. Interior drainage systems should be designed to minimize human intervention, and backup systems should be provided to the extent feasible. If human intervention is necessary, the necessary procedures and responsibilities should be clearly defined in the officially adopted maintenance and operations plan for the levee system. If the areas protected by the levee and interior drainage systems are to be removed from the flood hazard designation, the guidelines provided in the FEMA National Flood Insurance Program (NFIP) regulation 44CFR65.10 should be followed.

Interior drainage systems should be adequately sized to handle the flows from the local contributing drainage basins for the following two scenarios:

- Sized to handle expected flows from the contributing drainage basins during a 100-year flood event of the river/channel. The expected flows from the interior contributing basins should be determined based on the joint probability of the interior and exterior flooding.
- Sized to handle 100-year flows from the interior contributing basins (in combination with the other drainage facilities including storm drains & etc.)

with expected water levels on the other side of the levee (channel/river). The flows and associated water surface elevations of the receiving channel/river should be determined based on the joint probability of the interior and exterior flooding.

4.4.1 CLOSURES

All levee openings including culverts shall be adequately designed to not adversely impact the embankment stability and shall be provided with closure devices that can prevent flood flows from flowing though the openings and inundate the areas protected by the levee system.

Culvert closure devices can be classified as automatic or manual. Automatic closure devices include flap gates, Tideflex check valves, and other devices not requiring human intervention. Manual closure devices include slide type gates, sluice gates, and other devices that require human intervention.

Automatic closure devices should be used for openings where the water level can rise in a short time and for situations where the gates cannot be easily accessed. The flap-gates should not be used to provide opening closures where debris can easily prevent the flap gates to close completely. Manual closure devices may be used where flood flows rise slowly allowing ample time for safe operations. If the site conditions warrant, a secondary emergency gate may be necessary to minimize the risk of backflows through the opening.

4.5 OPERATIONS AND MAINTENANCE

In order for levees to be recognized as providing flood protections, levees must be designed in accordance with the guidelines set forth in this section. In addition, the following levee ownership and operations and maintenance requirements should be followed.

4.5.1 OWNERSHIP

Levees owned, operated, or maintained by a private party will not be recognized as providing flood protection. Levees for which the local, state, or federal government has responsibility for operations and maintenance may be considered as providing flood protection provided that the other criteria outlined in this section are satisfied.

4.5.2 CERTIFICATION

A levee must be certified by a federal agency or the CWCB that the levee meets the structural and freeboard requirements outlined in this section. Existing levees with noticeable structural defects, lack of freeboard, or lack of maintenance should not be considered as providing flood protections.

4.5.3 HUMAN INTERVENTION

Levees that require human intervention during or shortly before a flood event (i.e., sandbagging, earthfill, flashboards, etc.) in order to increase the levee heights to the required 100-year design top of the levee grades (including freeboard) should not be considered as providing the 100-year flood protection. Human intervention necessary for the operation of opening closures and mechanical functions of internal drainage systems (i.e., manual backup of pumping stations & etc.) may be considered only if the operation procedures are clearly defined in an officially adopted operations manual.

4.5.4 OPERATIONS AND MAINTENANCE PLAN

For levee systems to be considered as providing the designed 100-year flood protection, comprehensive operations and maintenance plans should be prepared, followed and officially adopted by local, state, or federal agencies. The operations and maintenance plan criteria outlined in the FEMA NFIP regulations 44CFR65.10 (www.fema.gov) should be followed.

Levees should be inspected periodically and after storm events, and any considerable damage should be repaired promptly.

4.6 FLOODPLAIN DELINEATION OF AREAS PROTECTED BY LEVEES

The natural floodplain areas protected from a 100-year event by levees may be designated as Zone X only if the levee systems meet the FEMA design, operations, and maintenance standards and requirements. These FEMA standards can be found in the National Flood Insurance Program (NFIP) regulations 44CFR65.10 (www.fema.gov).

Areas inundated by the interior drainage behind the levees should be defined, and if necessary, the 100-year water surface elevations, flooding limits and depths, special hazard zones should be clearly identified. For more detailed discussions on the floodplain delineation of areas impacted by levees, please refer to Chapter 4, Section 2.

4.7 SETBACK LEVEES

Properly designed levee systems can effectively provide the designed flood protection for many communities and allow existing developments to be removed from the floodplains. Levees have been used because they usually cost less and require relatively small amounts of land when compared to other flood control options. Also, there may be site-specific constraints that prevent the use of other flood control options. However, when and if the levees fail, the resulting flooding can be devastating for many communities.

Levees should be used only if other reasonable and safer flood control methods (i.e., relocation, channel modification/improvement, fill, elevation, acquisition, etc.) cannot be utilized due to the site-specific constraints or if other methods were determined economically impractical. If levees are to be used, setback levees should be used where possible. Setback levees are less susceptible to failures because levees are placed substantially away from the channel, allowing flood flows to spread out thereby reducing the flow velocity acting on the levee embankments. Setback levees can also allow some natural channel migration to occur without impacting the levee embankment and foundation and usually results in less environmental impacts.

4.8 FLOODWALLS

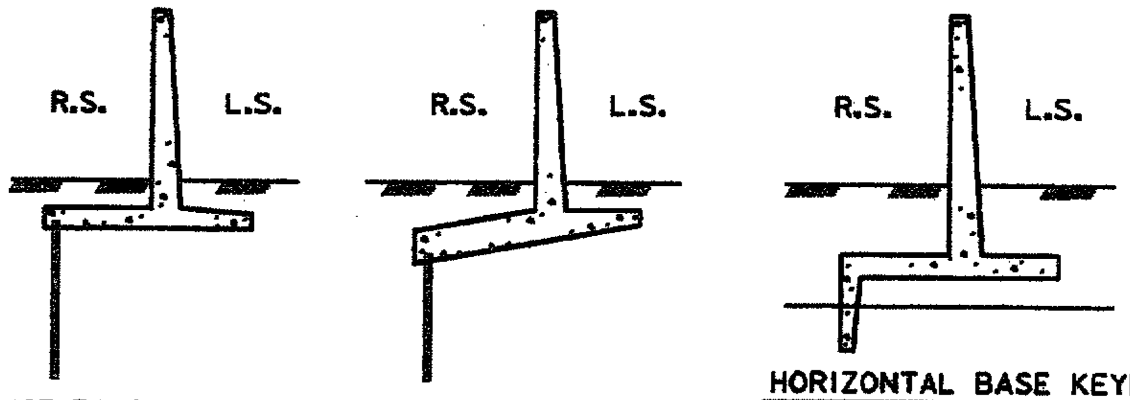
When the right-of-way necessary for the construction of new levees or enlargement of existing levees is not available or too expensive, floodwalls may be used in place of earthen levees. Floodwalls are considerably more expensive to design and construct compared to earthen levees and therefore floodwalls are rarely used outside of urban areas.

Most commonly used floodwall types are cantilever T-type and cantilever I-type walls and they are shown schematically on Figure CH6-F401. Floodwalls should be structurally designed to withstand the hydraulic forces and other loadings. The top of floodwall grades should be determined following the same guidelines as the earthen levee as outlined in Section 4.3.

If floodwalls are used to confine flood flows and remove areas out of natural floodplains, a CLOMR should be obtained prior to the construction to allow local and state agencies and FEMA to review and comment on the design prior to the wall construction.

For detailed discussions on the design of floodwalls, readers are referred to the following publications:

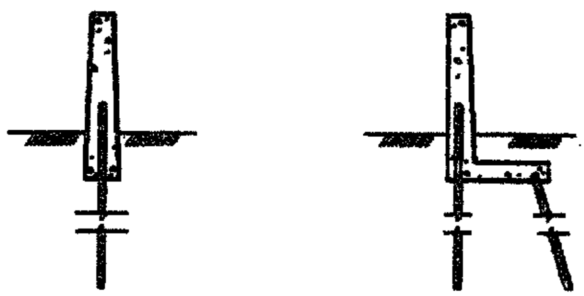
- US Army Corps of Engineers, Engineering and Design, Retaining and Flood Walls, Engineer Manual, EM 1110-2-2502, Sept. 1989



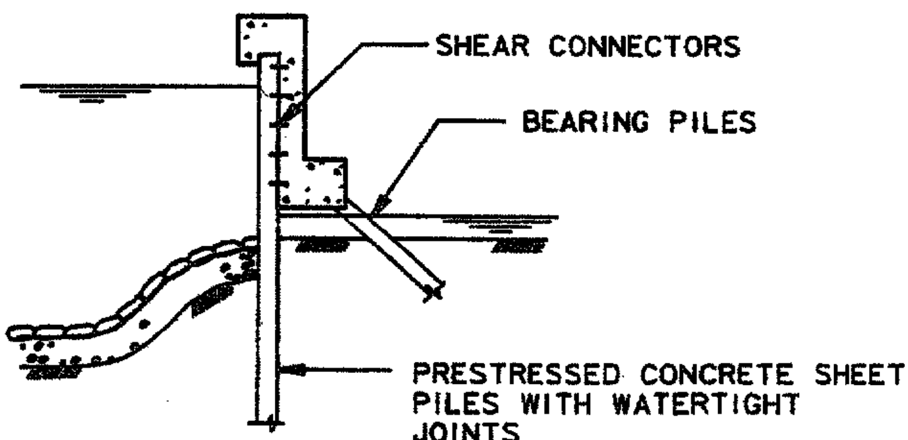
HORIZONTAL BASE | SLOPED BASE | HORIZONTAL BASE KEYED INTO STRONGER SOIL

NOTE: R.S.= RIVER SIDE (OR SEAWARD, UNPROTECTED SIDE)
L.S.= LAND SIDE (OR PROTECTED SIDE)

INVERTED T-TYPE CANTILEVER WALLS



CANTILEVER I-TYPE SHEET PILE WALLS *



BRACED SHEET PILE COASTAL FLOOD WALL *

* Analysis of this type of wall is beyond the scope of this manual.

Chapter 7

CHAPTER 7

STORM SEWER SYSTEMS

This Chapter will be completed in Phase 2 of the Criteria Manual project.

Chapter 8

CHAPTER 8

UNIQUE HYDRAULIC CONDITIONS

This Chapter will be completed in Phase 2 of the Criteria Manual project.

Chapter 9

CHAPTER 9

WATER QUALITY

This Chapter will be completed in Phase 2 of the Criteria Manual project.

Chapter 10

CHAPTER 10

PERMIT REQUIREMENTS FOR FLOOD HAZARD REDUCTION PROJECTS

This Chapter will be completed in Phase 2 of the Criteria Manual project.

Chapter 11

CHAPTER 11

FLOOD HAZARD MITIGATION AND RELIEF

This Chapter will be completed in Phase 2 of the Criteria Manual project.

Chapter 12

CHAPTER 12

INFORMATION RESOURCES

SECTION 1.0 REFERENCES **CH12-100**

This Chapter will be completed in Phase 2 of the Criteria Manual project.

CHAPTER 12

INFORMATION RESOURCES

SECTION 1.0

REFERENCES

CHAPTER 12
INFORMATION RESOURCES

SECTION 1.0
REFERENCES

TABLE OF CONTENTS

CHAPTER 12
INFORMATION RESOURCES

SECTION 1.0
REFERENCES

1.1 HYDROLOGIC ANALYSIS

CHOW, 1964 - Chow, V. T., Handbook of Applied Hydrology, McGraw-Hill Book Company, 1964.

FAA, 1970 - Airport Drainage, Advisory Circular 150/5320-5B, U. S. Department of Transportation, Federal Aviation Administration, Washington, D.C., 1970.

HMR 49, 1977 - Hansen E.M., Schwarz F.K., and Riedal J.T., Probable Maximum Precipitation Estimates, Colorado River and Great Basin Drainages, Hydrometeorological Report m. 49, U.S. Department of Commerce, September 1977.

HOGGAN, 1989 - Hoggan, D.H., Computer Assisted Floodplain Hydrology and Hydraulics, McGraw-Hill Book Company, 1989.

IAC, 1982 - Guidelines for Determining Flood Flow Frequency - Bulletin # 17B, Interagency Advisory Committee on Water Data, U. S. Department of the Interior, Geological Survey, March, 1982.

LADPW, 1989 - Hydrology Manual, Los Angeles County Department of Public Works, Alhambra, California, December 1971, revised March 1989.

LINSLEY, 1975 - Hydrology for Engineers, 2nd Edition, Linsley, Kohler, and Paulhus, McGraw-Hill Company, New York, 1975.

NOAA, 1984 - U.S. Department of Commerce, National Oceanic and Atmospheric Administration, Depth-Area Ratios in the Semi-Arid Southwest United States, NOAA Technical Memorandum NWS Hydro-40, August 1984.

NOAA, 1973 - U.S. Department of Commerce, National Oceanic and Atmospheric Administration, Precipitation-Frequency Atlas of the Western United States, Volume III – Colorado, 1973

SCS, 1983 – TR-20 Computer Program for Project Formulation Hydrology, U. S. Department of Agriculture, Soil Conservation Service, Washington, D.C., May 1983.

SCS, 1985 - National Engineering Handbook, Section 4, Hydrology, U.S. Department of Agriculture, Soil Conservation Service, Washington, D.C., March 1985.

UDFCD, 2001 - Urban Drainage and Flood Control District, Colorado Urban Hydrograph Procedure Computer Program, User Manual, February 2001.

UDFCD, 2001 - Urban Drainage and Flood Control District, Urban Drainage Storm Water Management Model (UDSWM), Users Manual, February 2001.

USACE, 1998 – U.S. Army Corps of Engineers, Hydrologic Engineering Center, HEC-1, Flood Hydrograph Package, User's Manual, June 1998.

USACE, 1979 - U.S. Army Corps of Engineers, "Introduction and Application of Kinematic Wave Routing Techniques Using HEC-1, Training Document m. 10", Davis, California, May 1979.

USBR, 1989 - Flood Hydrology Manual, U. S. Department of the Interior, Bureau of Reclamation, Washington, D.C., 1989.

WRC Engineering, Inc., 1990 - Clark County Regional Flood Control District, Hydrologic Criteria and Drainage Design Manual, Las Vegas, Nevada, 1990.

1.2 HYDRAULIC ANALYSIS

ASCE, 1975 - American Society of Civil Engineers, Sedimentation Engineering, Manuals and Reports on Engineering Practice m. 54, ASCE, New York, 1975.

BRATER, 1976 - Brater and King, Handbook of Hydraulics, McGraw Hill Book Company, 1976.

BRUTHURST, 1979 - J.C. Bruthurst, R.M. Li, and D.B. Simons, "Hydraulics of Mountain Rivers", Civil Engineering Department, Colorado State University, CER78-78JCB-RML-DBS55, 1979.

CEDEGREN, 1967 - H.R. Cedegrin, Seepage Drainage and Flow Nets, John Wiley and Sons, Inc., New York, 1967.

CHOW, 1959 - Chow, V. T., Open Channel Hydraulics, McGraw-Hill Book Company, 1959.

COOKE, 1973 - Cooke, R. U. and Warren, A., Geomorphology in Deserts, University of California Press, Berkeley and Los Angeles, California, 1973.

FEMA, 1990 - Federal Emergency Management Agency, FAN, An Alluvial Fan Flooding Computer Program & User's Manual, September 1990.

FRENCH, 1985 - French, R. H., Open-Channel Hydraulics, McGraw-Hill Book Company, New York, 1985.

GOLDMAN, 1986 - S. J. Goldman, K. Jackson, and T. A. Bursztyusky, Erosion and Sediment Control Handbook, McGraw Hill, 1986.

HOGGAN, 1989 - Hoggan, D.H., Computer Assisted Floodplain Hydrology and Hydraulics, McGraw-Hill Book Company, 1989.

LANE, 1935 - E.W. Lane, Security from Under Seepage, Transactions ASCE, Vol. 100, 1935.

LINSLEY, 1964 - Linsley, R. K., and Franzini, J. B., Water Resources Engineering, McGraw-Hill Book Company, USA, 1964.

Rosgen, 1996 – Dave Rosgen, illustrated by Hilton Lee Silvey, Applied River Morphology, 1996.

SIMONS, 1982 - Simons, Li and Associates, Engineering Analysis of Fluvial Systems, 1982.

TAYLOR, 1967 - D.W. Taylor, Fundamentals of Soil Mechanics, John Wiley and Sons, 1967.

USACE, 1990B - HEC-2, Water Surface Profiles, Davis, California, September 1990.

USACE, 1990 – Settlement Analysis, EM 1110-1-1902, U. S. Army Corps of Engineers, September 1990.

USACE, 1993 – River Hydraulics, EM 1110-2-1416, U. S. Army Corps of Engineers, October 1993.

USDA, 1976 - Sedimentation Deposition in U. S. Reservoirs, Summary of Data Reported through 1975, U. S. Department of Agriculture, Washington, D.C., 1976.

USDOT, 1978 - Hydraulics of Bridge Waterways, Hydraulic Design Series m. 1, U. S. Department of Transportation, Federal Highway Administration, Washington, D.C., March, 1978.

USDOT, 1991A - Stream Stability at Highway Structures, Hydraulic Engineering Circular m. 20, U.S. Department of Transportation, Federal Highway Administration, McLean, Virginia, February, 1991.

USDOT, 1991B - Evaluating Scour at Bridges, Hydraulic Engineering Circular m. 18, U.S. Department of Transportation, Federal Highway Administration, McLean, Virginia, February, 1991.

1.3 HYDRAULIC DESIGN

AISI, 1971 - Handbook of Steel Drainage and Highway Construction Products, American Iron and Steel Institute, Washington, D.C., 1971.

ANDERSON, 1968 - Anderson, A.G., Paintal, A.S., and Davenport, J.T., Tentative Design Procedure for Riprap Lined Channels, University of Minnesota, St. Anthony Falls Hydraulics Laboratory, Project Design Report m. 96, 1968.

FORTIER, 1926 - S. Fortier and F. C. Scobey, Permissible Canal Velocities, Transactions - American Society of Civil Engineers, Volume 89, pp 940-956, 1926.

LACFCD, 1982 - Design Manual, Hydraulic, Los Angeles County Flood Control District, March 1982.

McLAUGHLIN WATER ENGINEERS, LTD., Evaluation of and Design Recommendations for Drop Structures in the Denver Metropolitan Area, December 1986.

PETERKA, 1978 - A. J. Peterka, Hydraulic Design of Stilling Basins and Energy Dissipators, EM m. 25, U. S. Department of the Interior, Bureau of Reclamation, Denver, Colorado, 1978.

SIMONS, 1981 - Design Guidelines and Criteria for Channels and Hydraulic Structures on Sandy Soils, Simons, Li, and Associates, prepared for Urban Drainage and Flood Control District and City of Aurora, Colorado, June 1981.

SIMONS, 1989 - Simons, Li and Associates, Surface Mining Water Diversion Design Manual, (September 1982), U.S. Department of Interior, Office of Surface Mining Reclamation and Enforcement, 1989.

SMITH, 1965 - Smith, C.D., and Murray, D.G., Cobble Lined Drop Structures, 2nd Canadian Hydro-Technical Conference, Burlington, Ontario, 1965.

STEVENS, 1981 - Stevens, Michael, A., Hydraulic Design Criteria for Riprap Chutes and Vertical Drop Structures, prepared for Urban Drainage and Flood Control District, 1981.

UD&FCD, 1990 - Grouted Riprap and Boulder Installations, Supplement to Urban Storm Drainage Criteria Manual, Denver Urban Drainage and Flood Control District, 1982.

USACE, 1986 – Earth and Rock-Fill Dams – General Design and Construction Considerations, EM 1110-2-2300, U. S. Army Corps of Engineers, September 1986.

USACE, 1989 – Retaining and Flood Walls, EM 1110-2-2502, U. S. Army Corps of Engineers, Sept, 1989.

USACE, 1991 - Hydraulic Design of Flood Control Channels, EM 1110-2-1601, U. S. Army Corps of Engineers, July 1991.

USACE, 1990 - Hydraulic Design of Spillways, EM 1110-2-1603, U. S. Army Corps of Engineers, January 1990.

USACE, 2001- Design and Construction of Levees, EM 1110-2-1913, U. S. Army Corps of Engineers, April 2001.

USBR, 1967 - Canals and Related Structures, Design Standards m. 3, U. S. Department of the Interior, Bureau of Reclamation, Denver, Colorado, 1967.

USBR, 1974 - Design of Small Canal Structures, U. S. Department of the Interior, Bureau of Reclamation, Denver, Colorado, 1974.

USBR, 1987 - Design of Small Dams, U. S. Department of the Interior, Bureau of Reclamation, Washington, D.C., 1987.

USDCM, 1969 - Urban Storm Drainage Criteria Manual, Denver Regional Council of Governments, Denver, Colorado, March 1969 (with current revisions).

USDOT, 1971 - Debris Control Structures, EPD-86-106, HEC-9, U. S. Department of Transportation, Federal Highway Administration, Washington, D.C., 1971.

USDOT, 1965 – Hydraulic Charts for the Selection of Highway Culverts, Hydraulic Engineering Circular No. 5, U.S. Department of Transportation, Federal Highway Administration, December 1965.

USDOT, 1972 – Capacity Charts for the Hydraulic Design of Highway Culverts, Hydraulic Engineering Circular No. 10, U.S. Department of Transportation, Federal Highway Administration, November 1972.

USDOT, 1972 – Hydraulic Design of Improved Inlets for Culverts, Hydraulic Engineering Circular No. 13, U.S. Department of Transportation, Federal Highway Administration, 1972.

USDOT, 1983 - Hydraulic Design of Energy Dissipators for Culverts and Channels, Hydraulic Engineering Circular m. 14, U.S. Department of Transportation, Federal Highway Administration, Washington, D.C., September 1983.

USDOT, 1985 - Hydraulic Design of Highway Culverts, Hydraulic Design Series m. 5, U. S. Department of Transportation, Federal Highway Administration, McLean, Virginia, September 1985.

USDOT, 1989 - Design of Riprap Revetment, Hydraulic Engineering Circular m. 11, U.S. Department of Transportation, Federal Highway Administration, McLean, Virginia, March 1989.

WRC Engineering, Inc., 1990 - Clark County Regional Flood Control District, Hydrologic Criteria and Drainage Design Manual, Las Vegas, Nevada, 1990.

1.4 FLOODPLAIN MANAGEMENT AND ADMINISTRATION

FEMA, 1999 - Federal Emergency Management Agency, NFIP Regulations, Title 44, Chapter 1, Part 65, Identification and Mapping of Special Hazard Areas, revised October 1999.

FEMA, 1993 - Federal Emergency Management Agency, Flood Insurance Study Guidelines and Specifications for Study Contractors, March 1993.

FEMA, 2000 - Federal Emergency Management Agency, Guidelines for Determining Flood Hazards on Alluvial Fans, February 23, 2000

FEMA - Federal Emergency Management Agency, Appeals, Revisions and Amendments to Flood Insurance Maps, A guidebook for Local Officials (FIA-12)

FEMA, 1999 - Federal Emergency Management Agency, NFIP Regulations, Title 44, Chapter 1, Parts 60, 65, 70, and 72, revised October 1999.

FEMA, 1993 - Federal Emergency Management Agency, Openings in Foundation Walls for Buildings Located in Special Flood Hazard Areas in Accordance with the National Flood Insurance Program, Technical Bulletin 1-93, 1993.

FEMA, 1993 - Federal Emergency Management Agency, Flood-Resistant Materials Requirements for Buildings Located in Special Flood Hazard Areas in Accordance with the National Flood Insurance Program, Technical Bulletin 2-93, 1993.

FEMA, 1993 - Federal Emergency Management Agency, Non-Residential Floodproofing – Requirements and Certification for Buildings Located in Special Flood Hazard Areas in Accordance with the National Flood Insurance Program, Technical Bulletin 3-93, 1993.

FEMA, 1993 - Federal Emergency Management Agency, Elevator Installation for Buildings Located in Special Flood Hazard Areas in Accordance with the National Flood Insurance Program, Technical Bulletin 4-93, 1993.

FEMA, 1993 - Federal Emergency Management Agency, Below-Grade Parking Requirements for Buildings Located in Special Flood Hazard Areas in Accordance with the National Flood Insurance Program, Technical Bulletin 6-93, 1993.

FEMA, 1993 - Federal Emergency Management Agency, Wet Floodproofing requirements for Buildings Located in Special Flood Hazard Areas in Accordance with the National Flood Insurance Program, Technical Bulletin 7-93, 1993.

1.5 WATER QUALITY

USEPA, 1976 - U.S. Environmental Protection Agency, Erosion and Sediment Control, Surface Mining in the Eastern U.S., EPA-625/3-76-006, Washington, D.C., 1976.

Washoe County Department of Public Works, Construction Activities/Best Management Practice Handbook, June 1994.

1.6 STORM SEWER SYSTEMS

HOPKINS, 1956 - The Design of Storm-Water Inlets, John Hopkins University, Department of Sanitary Engineering and Water Resources, Baltimore, Maryland, June 1956.

IZZARD, 1977 - Izzard, Carl F., Hydraulic Capacity of Curb Opening Inlets, Flood Hazard News, UD&FCD, Denver, Colorado, June 1977.

1.7 WEBSITES

Agency Name	Website	Available Information
Colorado Water Conservation Board (CWCB)	www.cwcb.state.co.us/	CWCB Rules and Regulations and Colorado Water Resources Information
Federal Emergency Management Agency (FEMA)	www.fema.gov/nfip/	FEMA Forms, Computer Programs, and Publications
Federal Highway Administration (FHWA)	www.fhwa.dot.gov/bridge/hydpub.htm	FHWA Hydraulics Publications
Hydrologic engineering Center (HEC), USACE	www.hec.usace.army.mil/software/software_distrib/index.html	HEC Hydrologic and Hydraulic Computer Programs and User's Manuals

Urban Drainage and Flood Control District (UDFCD)	www.udfcd.org/	UDFCD Computer Programs and Publications
U.S. Army Corps of Engineers (USACE)	www.usace.army.mil/inet/usace-docs/	USACE Publications