CLARK COUNTY REGIONAL FLOOD CONTROL DISTRICT

HYDROLOGIC CRITERIA AND DRAINAGE DESIGN MANUAL

ADOPTED

AUGUST 12, 1999

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Amendments and Revisions

This MANUAL has been prepared using current, state-of-the-art technology and procedures. Due to the dynamic nature of urban storm drainage, amendments and revisions will be required from time to time as the state-of-the-art changes and experience is gained in the use of this MANUAL.

Users of this MANUAL are encouraged to submit their comments and revisions. This information should be addressed to:

Mr. Gale Wm. Fraser, II, P.E.
General Manager/Chief Engineer
Clark County Regional Flood Control District
600 South Grand Central Parkway
Suite 300
Las Vegas, Nevada 89106-4511

Comments and revisions may also be faxed to (702) 455-3870. For information purposes, the CCRFCD maintains a website at: http://www.ccrfcd.org/.

A list of MANUAL holders will be maintained by the CCRFCD. To receive copies of amendments or revisions, please complete the form below and submit it to the address shown.

Return to:

Mr. Gale Wm. Fraser, II, P.E.
General Manager/Chief Engineer
Clark County Regional Flood Control District
600 South Grand Central Parkway
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Re: Hydrologic Criteria and Drainage Design Manual

NAME: ______________________

COMPANY: ______________________

MAILING ADDRESS ______________________

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DATE MANUAL RECEIVED: ______________________

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CLARK COUNTY REGIONAL FLOOD CONTROL DISTRICT
HYDROLOGIC CRITERIA AND DRAINAGE DESIGN MANUAL

SECTION 100
GENERAL PROVISIONS

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Section 100
General Provisions

101 TITLE

These criteria and design standards with all future amendments and revisions shall be known as the "CLARK COUNTY REGIONAL FLOOD CONTROL DISTRICT HYDROLOGIC CRITERIA AND DRAINAGE DESIGN MANUAL" (herein referred to as the MANUAL). This MANUAL is part of the "Uniform Regulations for the Control of Drainage" (herein referred to as the REGULATIONS) as adopted by the Board of Directors (herein referred to as the BOARD) for the Clark County Regional Flood Control District (herein referred to as CCRFCD).

102 ADOPTION AUTHORITY

The CCRFCD is organized under Chapter 543 of the Nevada Revised Statutes (NRS). The CCRFCD BOARD (NRS 543.320) has adopted REGULATIONS as mandated in NRS 543.595(1) as well as a "Policies and Procedures Manual" (herein referred to as POLICIES AND PROCEDURES) as mandated in NRS 543.340(4).

Chapter 15.0 of the POLICIES AND PROCEDURES states that a "Hydrologic and Design Criteria Manual" will be developed by the CCRFCD. This chapter also states that the MANUAL shall be used for the development and design of all Flood Control Facilities located within the CCRFCD.

The CCRFCD has complied with the provisions of Chapter 15.0 by preparation of this MANUAL and has afforded all entities governed by the REGULATIONS the opportunity to participate in the development of the MANUAL.

This BOARD has enabled this MANUAL by adoption pursuant to Section 1.010(B) of the REGULATIONS.

103 JURISDICTION

These criteria and design standards shall apply to all areas within the boundaries of Clark County (NRS 543.240(1)).

104 PURPOSE

The purpose of the MANUAL is to provide a minimum standard for analysis and design of storm drainage facilities within the CCRFCD. Provision of the minimum standard assures that all drainage facilities are consistent in design and construction, and provides an integrated system which acts to protect the public health, safety, comfort, convenience, welfare, property and commerce.

105 ENFORCEMENT RESPONSIBILITY

Each entity is charged with enforcement of the MANUAL within its jurisdictional boundaries for Local and Regional Flood Control Facilities. Section 1600, Local Entity Criteria, contains the agency's addenda where more restrictive requirements are detailed. In addition to the entity's responsibility, the CCRFCD is also charged with enforcement of the MANUAL for all Regional Flood Control Facilities.

106 VARIANCE PROCEDURES

Variances to this MANUAL may only be requested for the following reasons:

1. Unusual situations where strict compliance with the MANUAL may not act to protect the public health and safety.

2. Unusual situations which require additional analysis outside the scope of this MANUAL for which the additional analysis shows that strict compliance with the MANUAL may not act to protect the public health and safety.

3. Unusual hydrologic and/or hydraulic conditions which cannot be adequately addressed by strict compliance with the MANUAL.

Conditions which are created by improper site planning (i.e., lack of adequate space allocations) shall not be considered as grounds for a variance request.

If the subdivider (developer, builder, etc.) believes that a variance to the minimum standards in this MANUAL is warranted based on the reasons listed above they shall request a variance from the minimum standards.

Variances from this MANUAL for all Regional Flood Control Facilities shall be made in accordance with Section 13 of the REGULATIONS. Variances from this MANUAL for all Local Flood Control Facilities shall be made in accordance with the local governing entity's variance procedure under which the REGULATIONS and this MANUAL are adopted.

107 INTERPRETATION

In the interpretation and application of the provisions of this MANUAL, the following shall govern:

1. The provisions shall be regarded as the minimum requirements for the protection of the public health, safety, comfort, convenience, welfare, property, and commerce of the residents of CCRFCD. This MANUAL shall therefore be regarded as remedial and shall be liberally construed to further its underlying purposes.

2. Whenever a provision of this MANUAL and any other provisions of the REGULATIONS or any provisions in any law, ordinance, resolution, rule, or regulation of any kind, contain any restrictions covering any of the same subject matter, whichever restrictions are more restrictive or impose higher standards of requirements shall govern.

108 REVIEW AND APPROVAL

All drainage plans, reports, construction drawings and specifications shall be reviewed in accordance with the provisions of this MANUAL. This review and approval shall not relieve the owner, engineer, or designer from responsibility of ensuring that the calculations, plans, specifications, and construction drawings are in compliance with the provisions of this MANUAL.

The owner, developer, engineer, and designer must also understand that the local entities and CCRFCD do not and will not assume liability for the drainage facilities designed and/or certified by the engineer. In addition, the local entities and CCRFCD cannot guarantee that drainage design review and approval will absolve the owner, developer, engineer, designer, and/or their successors and/or assigns of future liability for improper design.

109 IMPLEMENTATION

109.1 Development of the MANUAL

The CCRFCD has developed this comprehensive MANUAL for use by the entities, and by consulting engineers. This MANUAL shall be used for the development and design of all Master Plan Flood Control Facilities and any other facilities dedicated to a public entity for ownership and maintenance.

Respective entities have been afforded the opportunity to participate in the development of this MANUAL and will be given the opportunity to participate in subsequent updates.

109.2 Updates

The MANUAL will be updated from time to time as determined by the Chief Engineer.

109.3 Adoption

The BOARD of the CCRFCD should adopt the MANUAL and all subsequent updates thereto. The entities should also adopt the MANUAL and all subsequent updates thereto.

109.4 Reconciliation of Pre- and Post-MANUAL Studies

1. Developments for which final detailed drainage reports (i.e., additional reports or analysis are not required prior to construction) or construction drawings have been approved are exempt from the provisions of this MANUAL and/or subsequent revisions. An exception to this exemption is Local Master Planned Facilities which are addressed in 109.4 (4).

2. Developments for which a preliminary drainage analysis (i.e., additional analysis or reports are required prior to construction) has been approved are exempt from the provisions of this MANUAL if a final drainage report and/or analysis is submitted for review within 180 days of the initial adoption of this MANUAL and/or subsequent revisions.

3. Developments for which drainage reports and/or analysis have not been submitted by the time of the initial adoption of this MANUAL shall be analyzed in conformance with the provisions of this MANUAL.

4. Developments for which an overall Local Master Drainage Plan has been approved shall be addressed as follows:

   (a) For new construction affected by facilities designed and constructed (or under construction) at the time of initial MANUAL and/or revision adoption based on an approved master plan shall be analyzed using flow rates and volumes calculated per the requirements of this MANUAL. If these facilities pass the revised peak flows and volumes within the freeboard limits of the facility, then the facility shall be considered to have adequate capacity. If not, then the owner or developer shall submit a plan which discusses the impact of flows exceeding the capacity of the originally designed system and proposed solutions to minimize these impacts.

   (b) Facilities planned but not under construction at the time of initial MANUAL adoption shall also be analyzed as discussed in 109.4 (4) (a) above. However, if the facility does not have adequate capacity including freeboard, the facility shall be redesigned in accordance with the requirements of the MANUAL.

(c) Local facilities for specific subdivisions within the master planned area for which a separate detailed drainage report and/or analysis is required shall be addressed as discussed in 1 and 2 above.

110 ACRONYMS AND ABBREVIATIONS

The following acronyms and abbreviations are used within the contents of this MANUAL:

A	Area
BFE	Base Flood Elevation
BMP	Best Management Practices
CAC	Clark County Regional Flood Control District Citizens Advisory Committee
CAP	Corrugated Aluminum Pipe
CAPA	Corrugated Aluminum Pipe Arch
CCPW	Clark County Public Works
CCRFCD	Clark County Regional Flood Control District
CEC	Consulting Engineers Council
CLOMA	Conditional Letter of Map Amendment
CLOMR	Conditional Letter of Map Revision
CLV	City of Las Vegas
CMP	Corrugated Metal Pipe
CMPA	Corrugated Metal Pipe Arch
CN	Curve Number
CNLV	City of North Las Vegas
COH	City of Henderson
CSP	Corrugated Steel Pipe
CSPA	Corrugated Steel Pipe Arch

EGL	Energy Grade Line
FAA	Federal Aviation Administration
ft	feet, foot
FEMA	Federal Emergency Management Agency
FIRM	Flood Insurance Rate Map
fps	feet per second
HDS	Hydraulic Design Series
HEC	Hydraulic Engineering Circular
HERCP	Horizontal Elliptical Reinforced Concrete Pipe
HGL	Hydraulic Grade Line
HOA	Home Owners Association
hr	hour(s)
in	inch(es)
LOMA	Letter of Map Amendment
LOMR	Letter of Map Revision
mi	mile(s)
min	minute(s)
NFIP	National Flood Insurance Program
NGVD 29	National Geodetic Vertical Datum of 1929
NOAA	National Oceanic and Atmospheric Administration
NPDES	National Pollutant Discharge Elimination System
NRS	Nevada Revised Statutes
NWS	National Weather Service
NURP	Nationwide Urban Runoff Program

NDOT	Nevada Department of Transportation
NDEP	Nevada Division of Environmental Protection
NRCS	National Resources Conservation Service (formerly SCS, Soil Conservation Service)
NRS	Nevada Revised Statutes
NAVD 88	North American Vertical Datum of 1988
PE	Professional Engineer (Licensed by the State of Nevada)
PMF	Probable Maximum Flood
PMR	Physical Map Revision
RCBC	Reinforced Concrete Box Culvert
RCP	Reinforced Concrete Pipe
ROW	Right-of-Way
RTC	Regional Transportation Commission of Clark County
SFHA	Special Flood Hazard Area
SNHBA	Southern Nevada Home Builders Association
SPP	Structural Plate Pipe
SPPA	Structural Plate Pipe Arch
sq mi	square mile(s)
SWPPP	Storm Water Pollution Prevention Plan
TAC	Clark County Regional Flood Control District Technical Advisory Committee
t_c	time of concentration
t_i	initial inlet or overland flow time
t_p	time-to-peak
t_t	travel time
TRC	Clark County Regional Flood Control District

	Technical Review Committee
USACE	United States Army Corps of Engineers
USBR	United States Bureau of Reclamation
USEPA	United States Environmental Protection Agency
USGS	United States Geological Survey

111 GLOSSARY

The following glossary is provided as an aid in the understanding of some of the terms and abbreviations included in this MANUAL. Section numbers where the term is first mentioned are indicated in parenthesis.

All Weather Access: (Section 203.1) The ability to access a site during a Major Storm.

BOARD: (Section 101) The Clark County Regional Flood Control District's Board of Directors.

Local Flood Control Facilities: (Section 303.4) All facilities which collect and convey stormwater from the local area into a Regional Flood Control Facility.

Local Off-Site (Public) Flood Control Facilities: (Section 303.4) All facilities which are dedicated to the public and collect and convey stormwater from the Local Off-Site Flood Control Facilities to a Regional Flood Control Facility.

Local On-Site (Private) Flood Control Facilities: (Section 303.4) All facilities which are privately owned and maintained and collect and convey stormwater from a single unified development or parcel to the Local Off-Site Flood Control Facilities. These facilities serve only the development or parcel in question.

Major Storm: (Section 304.2) The design storm having a recurrence interval of 100 years which may cause major damage to public property and possible loss of life.

MANUAL: (Section 101) The Clark County Regional Flood Control District's Hydrologic Criteria and Drainage Design Manual.

Minor Storm: (Section 304.2) The design storm having a recurrence interval of 10 years are typically small rain storms of very short duration and lower intensity which cause minor problems and inconvenience to the general public.

POLICIES AND PROCEDURES: (Section 102) The Clark County Regional Flood Control District's Policies and Procedures Manual, 1989 and all future amendments (Latest 11/12/90).

Regional Flood Control Facilities: (Section 303.4) All facilities which are included in the Regional Master Plan(s) as adopted by the Clark County Regional Flood Control District.

REGULATIONS: (Section 101) The Clark County Regional Flood Control District's Uniform Regulations for the Control of Drainage, 1988 and all future amendments.

Regional Flood Control Significance: Facilities land alteration, portions of the natural drainage system and regulatory actions which impact implementation of the Master Plan or lie within Special Flood Hazard Areas.

STANDARD SPECIFICATIONS: (Section 206) The Uniform Standard Specifications for Public Works' Construction, Off-Site Improvements, Clark County Area, Nevada, 1986 and all future amendments.

STANDARD DRAWINGS: (Section 206) The Uniform Standard Drawings for Public Works' Construction, Off-Site Improvements, Clark County Area, Nevada, 1988 and all future amendments.

CLARK COUNTY REGIONAL FLOOD CONTROL DISTRICT
HYDROLOGIC CRITERIA AND DRAINAGE DESIGN MANUAL

SECTION 200
DRAINAGE PLANNING AND SUBMITTAL

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Section 200
Drainage Planning and Submittal

201 SUBMITTAL AND REVIEW PROCESS

The purpose of the submittal and review process is to determine whether or not the specific drainage plan for a given project meets the regional and local policy requirements (Section 300) for drainage design in the Clark County area. These requirements include overall facility planning to assure an integrated and coordinated design as well as design standards to assure consistent design and analysis. Presented in **Table 201** are the Drainage Study Submittal Requirements for all land development and disturbance processes in the CCRFCD. The submittal and review process requirements are tailored to provide the minimal amount of information necessary for each development process and size of development in order to minimize the cost of drainage report preparation as well as to minimize the time necessary for local entity review. The submittal and review process does not, however, relieve the design engineer of the responsibility to provide a correct and safe drainage design nor the developer to properly construct the designed drainage facilities.

By reviewing and approving drainage designs for given developments, neither CCRFCD nor the local entities will assume liability for improper drainage design nor guarantee that the final drainage design review will absolve the developer or designer of future liability for improper design.

All land development and land disturbance processes which are within the jurisdiction of the MANUAL shall submit the required forms, reports, drawings, and/or specifications required for the appropriate drainage study as determined by **Table 201.** This table outlines the specific submittal requirements for the more typical land development or land disturbance processes. For processes not covered in the **Table 201** submittal requirements, the property developer shall contact the governing local entity to determine the submittal requirements for the process being considered.

Two copies of the required studies and attachments shall be submitted to the local entity for review. If the proposed development or land disturbance process is determined by the local entity to have regional significance, the local entity will submit one copy of the study to the CCRFCD. Additional copies, as necessary, shall be submitted as requested by the local entity. All submitted reports should be clearly and cleanly reproduced. Photostatic copies of charts, tables, nomographs, calculations, or any other referenced material should be legible. Washed out, blurred, or unreadable portions of the report are unacceptable and could warrant resubmittal of the report.

For regionally-significant projects, coordination meetings are encouraged between the developer, the developer's engineer, the entity, and CCRFCD.

A checklist of required items for each submittal process is presented on the Drainage Submittal Checklist (**Standard Form 2**). This checklist will be used by the local entity to initially determine if the minimum submittal requirements have been met. If the submittal does not meet the minimum requirements, the submittal will be returned to the submitting party with the deficiencies noted. These deficiencies must be corrected and resubmitted before the submittal will be accepted for review. The checklist shall be included with all drainage study submittals with the first section completed by the engineer.

202 DRAINAGE STUDY INFORMATION FORM

A Drainage Study Information Form (**Standard Form 1**) shall be included as the first page of all drainage study submittals including addenda. The purpose of the Drainage Study Information Form is to provide each entity a set of basic information regarding the subject development. This basic information will be used by the local entities to:

a) Assist in determining the need to involve the CCRFCD in the review process.

b) Catalog the submittal for filing, distribution, and retrieval purposes.

c) Provide a sharing of information between the local entities when a proposed development may impact the facilities of an adjacent entity.

The Drainage Study Information Form shall be directly bound into and at the front of the submittal drainage study. The Drainage Study Information Form shall contain the seal and signature of the professional engineer who fills out the form.

Note: The Drainage Study Information Form (Standard Form 1) is mandatory for building permits that may obstruct drainage.

203 CONCEPTUAL DRAINAGE STUDY

A Conceptual Drainage Study is a short letter type report which addresses existing and proposed drainage conditions from sites which generally have minor impact on the overall local and regional drainage facilities. The Conceptual Drainage Study documents the existing drainage conditions of the property as well as presents the details of the proposed drainage system. The Conceptual Drainage Study shall address all hydrologic criteria, with preliminary hydraulics. Detailed hydraulics shall be addressed in the Technical Drainage Study. In some cases, the Drainage Study Information Form (**Standard Form 1**) may provide sufficient information to serve as the Conceptual Drainage Study. The Conceptual Drainage Study shall contain a brief narrative letter, a calculation appendix (if required), and a drainage plan in accordance with the following outline:

203.1 Letter Contents

I. Introduction

A. **Standard Forms 1 and 2**
B. Project Name, Type of Study, Study Date
C. Preparer's Name, Seal, and Signature
D. Description of Project
E. Existing Site Conditions
F. General Location Map (8 ½" x 11" is suggested)

II. Existing and Proposed Hydrology/Hydraulics

A. Discuss existing and proposed drainage basin boundaries

B. Present existing and proposed minor and major storm flow calculations (if required)

C. Discuss existing drainage patterns and areas of inundation (if applicable)

III. Proposed Drainage Facilities

A. Discuss routing of flow in and/or around site and location of drainage facilities

B. Discuss mitigation measures (if applicable)

C. Discuss floodplain modifications (if applicable)

D. Present preliminary calculations for proposed facilities and typical sections for stormwater conveyance, if applicable.

IV. Conclusions

A. Compliance with MANUAL

B. Ability to provide emergency all weather access

C. Compliance with Federal Emergency Management Agency (FEMA) (if applicable)

D. Discuss effect of development on adjacent properties

1. Flow rates
2. Discharge location
3. Discharge velocity
4. Inundation limits
5. Summary table for II and III
6. List of facilities required

V. Exhibits

A. Drainage Plan (Section 203.2)
B. Watershed Maps
C. Cross Section Location Maps

VI. Calculations Appendix (if required)

A. Runoff calculations (existing and proposed)

B. Street and drainage facility capacity calculations, existing and proposed flood limit calculations

C. Detention calculations (if applicable)

203.2 Drainage Plan

An 8 ½" x 11" or larger, legible drainage plan which covers the development area shall be submitted and bound with the Conceptual Drainage Study. The plan shall contain, as a minimum, the following:

1. Locate and label development boundary.

2. Locate and label adjacent streets.

3. Locate and label known 100-year floodplains.

4. Locate and label existing and/or planned CCRFCD facilities.

5. Locate and label existing and/or planned local flood control facilities.

6. Show flow paths.

7. Identify design inflow points and design outflow points and corresponding minor and major storm flow rates.

Note: The drainage plan stated above is preferred; however, multiple exhibits containing the same information may be submitted.

203.3 Parcel Map Studies

Parcel map studies will be required according to the individual entities' processes as described in Section 1600. In general, a parcel map study for division of property for future sale or final design with no intention to proceed with any ground disturbing activities will contain Items I, II.A and C, V.B of Section 203.1 and Items 1 through 4 and 6 of Section 203.2. A Technical Drainage Study, as described in Section 204, will be required to support approval of final design.

204 TECHNICAL DRAINAGE STUDY

The Technical Drainage Study discusses at a detailed level the existing site hydrologic conditions and the proposed drainage plan to accommodate or modify these site drainage conditions in the final development plan for the site. The Technical Drainage Study addresses both on-site and off-site drainage analysis and improvements necessary to mitigate the impact of the proposed development on adjacent properties in accordance with current State of Nevada Drainage Law.

The Technical Drainage Study shall be in accordance with the following outline and contain the applicable information listed. **Standard Form 2** includes a drainage study criteria checklist and should be submitted along with the Technical Drainage Study. When the requested information is not applicable, signify with "N/A."

204.1 Study Contents

I . TITLE PAGE

- A. **Standard Forms 1 and 2**
- B. Project Name, Type of Study, Study Date
- C. Preparer's Name, Seal and Signature

II. GENERAL LOCATION AND DEVELOPMENT DESCRIPTION

- A. Location of Property

  1. City, County, State Highway and local streets within and adjacent to the subdivision

  2. Township, range, section,1/4 section

  3. Drainage basin(s) encompassing the development

  4. Location of development in relationship to the drainage basin's Regional Flood Control Facilities

5. Names of surrounding developments

6. General location map (8 ½" x 11" is suggested)

B. Description of Property

1. Area in acres

2. Existing site conditions (vegetation, buildings, drainage structures, etc.)

3. General site topography

4. Existing irrigation facilities such as ditches and canals

5. General project description and proposed land use

Ill . DRAINAGE BASIN DESCRIPTION

A. Off-Site Drainage Description

1. Discuss off-site flows which enter property at the following discrete points:

   a. Upstream Local Facilities runoff
   b. Upstream Regional Facilities runoff

2. Discuss off-site flows which enter property at non-discrete points.

3. Discuss existing and proposed land use types and level of development in upstream basin, as defined by the local entity(ies).

4. Hydrologic soil groups, vegetation, slope.

5. Natural and manmade conveyances in the watershed.

B. On-Site Drainage Description

1. Discuss historic on-site drainage patterns of the property (flow directions through site and at property line).

2. Discuss historic drainage patterns of upstream runoff.

3. Discuss historic discharge points at downstream property lines.

C. Master Planning Information

1. Identify currently adopted master plan(s) which include the subject site.

2. Discuss proposed Master Plan Flood Control Facilities on subject site (if applicable).

3. Discuss upstream Master Plan Flood Control Facilities which would affect runoff on subject site (if applicable).

D. Floodplain Information

1. Identify all FEMA regulated floodplains which overlay on the subject site.

2. Identify all calculated floodplains, including a proposed conditions or “with-project” floodplain.

E. Previous Drainage Studies

1. Identify any previous drainage studies for the site.

2. Identify any previous drainage studies which affect the site.

IV. PROPOSED DRAINAGE FACILITIES

A. General Description

1. Discuss proposed Local (On-Site) Drainage System plan and layout.

2. Discuss proposed Local (Off-Site) Drainage System plan from the Local (On-Site) Drainage System to the Regional Flood Control System.

3. Discuss proposed Regional Flood Control System design (only where the Regional Flood Control System passes through the subject site).

B. Compliance with Regulations and Adopted Plans

1. Discuss compliance with all Master Planned Flood Control Facilities (as applicable) and discuss all proposed deviations from the adopted Master Plans.

2. Discuss compliance with FEMA floodplain regulations and all proposed modifications to or verifications of the FEMA regulated floodplain through the subject site.

3. Discuss compliance with rules and regulations for developments on alluvial fans (if applicable).

4. Discuss compliance with previously approved drainage studies for the subject site.

5. Discuss compliance with BMPs as discussed in Section 1500.

6. Identify individually all requests for variances from the requirements of the drainage criteria and variances from the local entities' development code.

7. Discuss compliance with Uniform Regulations.

8. Discuss compliance with the MANUAL.

C. Hydrologic Analyses

Hydrologic analyses shall be completed for the following conditions. Calculations for all conditions shall be bound in the report:

1. Existing off-site and on-site

2. Existing off-site and developed on-site

3. Developed off-site and on-site

4. Design rainfall computation discussion.

5. Design runoff computation discussion.

6. Discuss peak flow rates from off-site areas and facilities.

7. Discuss flow split areas and analysis.

8. Hydrologic parameters.

9. Routing schematic.

10. Calculations for parking lots and Low Impact Development LID impervious areas (if required) per section 1502.3.

D. Facility Design Calculations

1. Discuss design calculations for the Proposed Drainage System

   a. Street flow calculations
   b. Storm sewer, inlets, and ditch flow calculations
   c. Channel and culvert flow calculations
   d. Other hydraulic structure flow calculations
   e. Detention storage and outlet design calculations
   f. BMP design calculations for parking lots and LIDs (if required)

2. Discuss design calculations for the Local (Off-site) Drainage System

   a. Alluvial fan analysis and calculations (when required)

3. Discuss Floodplain/Floodway calculations as related to FEMA requirements

4. Discuss maintenance access and potential maintenance requirements. Provide maintenance procedures for privately maintained facilities, with projected annual maintenance costs for incorporation into homeowners association.

5. Discuss easement requirements for the proposed drainage facilities

6. Discuss phasing of all drainage facilities

V. CONCLUSIONS

1. Compliance with Drainage Laws

2. Compliance with Master Plans

3. Compliance with FEMA requirements

4. Compliance with MANUAL

5. Compliance with REGULATIONS

6. Effectiveness of proposed drainage facilities to control storm runoff

7. Impact of proposed development on off-site property and facilities

VI . REFERENCES

1. Provide references for all drainage reports, plans, and technical information used in preparing the drainage report.

VII. APPENDICES

A. Hydrologic Computations

1. Watershed boundaries

2. Soils information

3. Land use information

4. Design rainfall calculations

5. Basin parameter calculations

6. Routing schematic

7. Runoff calculations at design points

a. Minor and major storm flows

b. Flows for historic and fully developed basin conditions

8. Hydrographs at property line discharge points, when appropriate

9. Input data listing for all computerized hydrologic calculations, maps with all parameters

B. Hydraulic Calculations

1. Street and ditch capacities

2.* Inlet and storm sewer capacities (including Energy Grade Line (EGL) and Hydraulic Grade Line (HGL) calculations), with inlet and outlet condition assumptions
3.* Channel and culvert capacities

4.* Floodplain/Floodway calculations

5. Detention area/storage/discharge rating curves and calculations

6. BMP hydraulic capacities

7. Input data listing for all computerized hydraulic calculations

8. Plots of all cross sections

9. Map with cross section locations

204.2 Drainage Plan

A detailed drainage plan(s) for the subject site shall be submitted with the Technical Drainage Study. The plan(s) shall be on a 24" x 36" drawing at an appropriate legible and microfilmable scale (a scale of 1" = 20' to 1" = 200' is recommended). A reference to all hydraulic calculations shall be a part of this plan. The following information shall be shown on this drawing, except that the off-site drainage basin boundaries may be shown at an appropriate legible scale on an exhibit.

1. Property lines and streets (roads) including right-of-way (ROW) widths within 100 feet of the property

2. Existing contours and proposed elevations sufficient to analyze drainage patterns extending 100 feet past property lines

3. Existing drainage facilities and structures, including ditches, storm sewers, channels, street flow directions, and culverts. All pertinent information such as material, size, shape, slope, and location shall also be included.

4. Limits of existing floodplains based on Flood Insurance Rate Maps (FIRMs), if available. Also, existing and proposed floodplains based on best available data (existing floodplain studies) should be shown, if available.

5. Proposed on-site drainage basin boundaries and sub-boundaries. Include off-site boundary intersections with on-site boundaries and off-site boundaries if not shown elsewhere.

6. Proposed future on-site and off-site flow concentration points, directions, and paths

7. Proposed street and ditch flow paths and slopes

8. Proposed storm sewer locations, type, size, and slope. Include inlet types, sizes and locations, and manhole locations. Proposed channel alignment with typical cross section. Include major storm flow limits.

10. Proposed culvert locations, type, size, slope, and headwater pool.

11. Proposed Local (On-Site) Drainage System outlet(s) to the Local (Off-Site) Drainage System.

12. Proposed BMP locations, types and sizes for parking lots and LIDs (if required).

13. Alignment of Local (Off-Site) Drainage System from Local (On-Site) Drainage System to Regional Flood Control System. If extent of Local (Off-Site) Drainage System is too large to include on the Drainage Plan, include a separate drawing showing entire drainage path of the Local (Off-Site) Drainage System.

14. Miscellaneous proposed drainage facilities (i.e., hydraulic structures, etc.)

15. Table of minor and major storm peak flows including tributary area at critical design points

16. Maintenance easement widths and boundaries.

17. Legend for all symbols used on drawing.

18. Scale, North Arrow, and Title Block.

204.3 Calculations Exemption

The report requirements for a Technical Drainage Study may be reduced at the request of the applicant if there is uncertainty over the final characteristics of the proposed drainage facilities or at the request of the local entity. The Technical Drainage Study shall identify all areas where the uncertainty exists. Hydrology and hydraulic calculations based upon assumptions may be provided with less detail. The areas where the assumptions and details are not provided must be identified so that they can be completed in the required detail as part of the Hydrologic/Hydraulic Calculations Addendum, if required. However, no construction permits will be issued until these details are provided in an Addendum.

Areas where assumptions are made and where the level of detail is limited shall be identified so that they can be completed in full detail as part of the Hydrologic/Hydraulic Calculations Addendum, if required.

205 HYDROLOGIC/HYDRAULIC CALCULATIONS ADDENDUM

The purpose of the Hydrologic/Hydraulic Calculations Addendum is to provide all detailed hydrologic and hydraulic calculations which were exempted from the Technical Drainage Study requirements. This addendum shall be prepared in accordance with the following outline and contain the applicable information listed.

I. TITLE PAGE

- A. **Standard Form 1**
- B. Project Name, Type of Study, Study Date
- C. Preparer's Name, Seal and Signature

II. HYDROLOGIC CALCULATIONS

- A. Calculations exempted from the Technical Drainage Study

III. HYDRAULIC CALCULATIONS

- A. Calculations exempted from the Technical Drainage Study

IV. REVISED DRAINAGE PLAN

A revised drainage plan for the subject site shall be included in this Addendum. The revised plan shall show the correct peak flows and facility capacities as computed in the enclosed calculations.

206 IMPROVEMENT PLANS

Where drainage improvements are to be constructed, the final construction plans (on 24" x 36" mylar) shall be submitted. Approval of the final construction plans (including details) by the local entity and/or CCRFCD is a condition of issuing construction permits. The plans for the drainage improvements will include:

1. Storm sewers, inlets, outlets and manholes with pertinent elevations, dimensions, type, and horizontal control indicated

2. Culverts, end sections, and inlet/outlet protection with dimensions, type, elevations, and horizontal control indicated

3. Channels, ditches, and swales (including side/rear yard swales) with lengths, widths, cross-sections, grades and erosion control (i.e., riprap, concrete, grout) indicated

4. Checks, channel drops, erosion control facilities

5. Detention pond grading, trickle channels, outlets, and landscaping

6. Other drainage related structures and facilities (including underdrains , sump pump lines and BMPs)

7. HGL's for minor (storm sewer) and major (channels) storm runoff including flow rates. To avoid confusion, EGL's do not need to be shown on the original plans, but they should be plotted on a second (paper) copy of the plans and included with the Drainage Study for review.

8. Maintenance access considerations

9. Overlot grading and erosion and sedimentation control facilities

10. Drainage easements and ROW with horizontal distance to improvements

The information required for the plans shall be in accordance with sound engineering principles, this MANUAL, and the uniform STANDARD DRAWINGS and STANDARD SPECIFICATIONS. Construction documents shall include geometric, dimensional, structural, foundation, bedding, hydraulic, landscaping, and other details as needed to construct the drainage facility. The approved drainage plan shall be included as part of the construction documents for all facilities affected by the drainage plan. Construction plans shall be signed and sealed by a registered professional civil engineer in the State of Nevada as being in accordance with the approved drainage report/drawings.

207 NPDES PERMITS

Non-point sources of pollution are diffuse sources which are distributed throughout the watershed and contribute to receiving waters at multiple locations. They are contrasted with point sources which contribute pollution to receiving waters at a single definable point. The United States Environmental Protection Agency (USEPA) has adopted regulations to control non-point pollutants from entering the environment through storm drainage facilities. Locally, the Nevada Division of Environmental Protection (NDEP) administers a municipal stormwater discharge permitting program for the Las Vegas Valley area. The local National Pollutant Discharge Elimination System (NPDES) municipal stormwater permit is issued jointly to CCRFCD; the Cities of Las

Vegas, North Las Vegas, and Henderson; and Clark County. These co-permittees have joined in a cooperative, multi-jurisdictional effort to comply with the permit requirements and address other regional stormwater quality issues.

In addition to mandating general municipal stormwater permits, USEPA's stormwater management program established permitting requirements for construction and industrial sites. NDEP administers construction site and industrial site stormwater permitting programs for Nevada. The emphasis of this portion of the program is on implementing BMPs to control non-point source pollution generated from active construction sites and industrial operations. NDEP issues permits, collects fees associated with permit application and approval, and is responsible for permit monitoring and enforcement.

NDEP is working with local jurisdictions in Las Vegas Valley to distribute information related to the construction and industrial permits as part of the permitting process of each entity.

207.1 Construction Permits

Currently, construction permits are required by NDEP for construction sites disturbing 5 acres of area or more. The construction permits require developing and implementing: (1) a “Notice of Intent” to Discharge; (2) a request for inclusion in the Stormwater General Permit No. GNV0022241; and (3) a Storm Water Pollution Prevention Plan (SWPPP) for the construction area. The SWPPP commits the contractor to implement BMPs to control sediment production and discharge of other pollutants from the site. An erosion control plan is required to prevent migration of sediment from the construction site into the drainage system. An application form and fee are also required; these must be submitted to NDEP.

207.2 Industrial Permits

Industrial permits are required by NDEP for all industries engaged in activities with a high potential for contributing non-point source pollution to the drainage system. The industrial categories requiring permits from NDEP include: mining; chemical products; paper, wood, and lumber products; metal industries; electronic equipment; etc. As with the construction permits, the industrial permits also require the development of a SWPPP to manage stormwater generated from areas directly related to manufacturing, processing, or raw material storage areas at an industrial plant. An application form and fee are also required; these must be submitted to NDEP.

208 NEVADA DEPARTMENT OF TRANSPORTATION CRITERIA

The Nevada Department of Transportation’s (NDOT's) drainage guidelines and criteria are summarized in a publication entitled "Nevada Department of Transportation, Terms and Conditions Relating to the Drainage Aspects of Right-of-Way Occupancy Permits." In this publication, NDOT defines minimum design return frequencies for drainage facilities such as culverts and channels.

The design frequencies range from the 10- to the 50-year event, based on various roadway classifications.

Other design criteria such as design frequencies for roadway surface drainage facilities (curb/gutter, drop inlets, storm drains) are also presented.

In their guidelines, NDOT also lists acceptable design references, including hydrologic and hydraulic publications and computer programs.

If a project requires an NDOT ROW permit, then either an NDOT Drainage Information Form or a drainage report may need to be submitted to NDOT along with the permit application. It is possible that a single drainage report could be prepared for submittal to the entity, NDOT, and CCRFCD.

The engineer is referred to the NDOT drainage guidelines if a project involves an NDOT ROW permit.

NDOT was issued their own NPDES stormwater permit by NDEP. Drainage projects affecting NDOT ROW must comply with the provisions of the NDOT stormwater permit.

209 MASTER DRAINAGE STUDY

Master drainage studies are utilized to establish the off-site and on-site flows for larger sized land development projects. They may be prepared when requested by the project developer or when required by the appropriate government entity during zoning actions or when specified in the entities' policy.

A Master Drainage Study will quantify the peak flows from the on-site and off-site basins. The pattern for on-site drainage routing will be established along with street hydraulic calculations. In general, the on-site basins are established based on the proposed collector/arterial street system. The need for other drainage improvements, i.e., storm sewers, open channels, etc., will be outlined as required to satisfy drainage criteria and policies.

In general, this study will be prepared in accordance with the standards of Section 204, as noted with an asterisk (*). Detailed grading or improvement plans are not required. Latitude shall be given to the requirements of the Master Drainage Study versus a Technical Drainage Study since the detail of design may not be known at the time of preparation.

The following sub-sections of Section 204 as noted with an asterick (*) are not required to be included in a Master Drainage Study, Other sub-sections, as determined through coordination with appropriate local Government entity, may also be omitted.

204.1 Study Contents

Section III.D.2, Section IV.B.5, Section IV.D.1.b through e, Section IV.D.2 through 5, Section VI, Section VII.B.2 through 5 and 7 through 8.

204.2 Drainage Plan

Items 12 & 15

If the requirements for the Technical Drainage Study outlined in Section 204 are met and all necessary grading and improvement plans are included in the Master Drainage Study, then the Master Drainage Study for the entire project can be utilized for overall grading of this project, construction of interim and perimeter streets, and drainage facilities.

In addition, the Master Drainage Study can be utilized for an entire project as well as a Technical Drainage Study for initial units of the project when the requirements of Section 204 are met and appropriate grading and improvement plans are provided.

DRAINAGE STUDY SUBMITTAL REQUIREMENTS

Land Development and/or Land Disturbance Process	Required Drainage Submittals*
Rezoning:	A [1, 2]
Parcel Map:	A [1]
Subdivisions: Tentative Map Final Map	 B [2] B [3]
Planned Unit Developments: Tentative Map Final Map	 B [2] B [3]
Commercial/Industrial Approvals	A [1]
Building Permit	A [1, 5]
Clearing, Grading, Filling and/or Excavation	A [1]
Other: Development Master Drainage Plans Transportation Studies Floodplain Modification Study (LOMA, LOMR, etc.)	 B [3] B [3] B [3, 4]

* Submittal Types: A - Conceptual Drainage Study
B - Technical Drainage Study

Notes:

1. A Technical Drainage Study may be required if requested by the local entity.
2. If the local entity does not perceive a flooding hazard with the proposed development, then the Land Development and/or Land Disturbance Process may be approved subject to review and approval of the Drainage Study and acceptance of conditions of approval by the owner.
3. A Hydrologic/Hydraulic Calculations Addendum is required only when uncertainty over the final characteristics within a proposed development does not allow the preparation of final hydraulic/hydrologic calculations with the Technical Drainage Study. This requirement may be waived at the discretion of the local entity and/or the CCRFCD.
4. All floodplain Modification Studies shall be prepared in accordance with the REGULATIONS and FEMA requirements.
5. See Section 202.

Revision	*Date*

REGIONAL FLOOD CONTROL DISTRICT

REFERENCE:

TABLE 201

CLARK COUNTY REGIONAL FLOOD CONTROL DISTRICT
HYDROLOGIC CRITERIA AND DRAINAGE DESIGN MANUAL

SECTION 300
DRAINAGE POLICY

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Drainage Policy

301 INTRODUCTION

NRS 543.340(4) gives the BOARD of the CCRFCD the authority to adopt written policies for administering the District and for operating and maintaining its projects and improvements. The CCRFCD has prepared and adopted POLICIES AND PROCEDURES for the operation of the District. The policies presented in this MANUAL relate specifically to the engineering aspects of drainage. The reader should become familiar with the policies of the District contained in the POLICIES AND PROCEDURES.

In urban areas it is necessary to provide an adequate drainage system in order to preserve and promote the general health, welfare, and economic well being of the region. Drainage is a regional feature that affects all governmental jurisdictions and all parcels of property. This characteristic of drainage requires coordination between different entities and cooperation from both the public and private sectors.

Master planning of Regional Flood Control Facilities as identified in the CCRFCD's Master Plan has been provided by the CCRFCD through coordination and cooperation with all impacted entities. Periodic updates, as required by NRS 543.596 or allowed by NRS 543.5965, to the CCRFCD's Master Plan must be provided by the CCRFCD and will be coordinated with the effected entities.

All Flood Control Facilities not identified in the CCRFCD's Master Plan are considered to be Local Flood Control Facilities of the entity where the facilities are located. The planning of Local Flood Control Facilities will be provided by the entity where the facility is located. If the Local Flood Control Facility impacts other entities and/or Regional Flood Control Facilities then the planning must be coordinated with the impacted entities.

When planning drainage facilities, certain underlying principles provide direction for the effort. These principles are made operational through a set of policy statements. The application of the policy is in turn facilitated by technical criteria and data. When considered in a comprehensive manner, on a regional and local level with public and private involvement, drainage facilities can be provided in developing areas in a manner that will provide the required flood protection.

302 BASIC PRINCIPLES

302.1 Stormwater Drainage System

The Stormwater Drainage System is an integral part of the total urbanization process. The planning of drainage facilities must be included in the urbanization process. The first step is to include drainage planning with all regional and local development master plans.

Drainage systems require space to accommodate their conveyance and storage functions. When the space requirements are considered, the provision for adequate drainage becomes a competing use for space along with other land uses. If adequate provision is not made in a land use plan for the drainage requirements, stormwater runoff will conflict with other land uses and may result in water damage, and may impair or even disrupt the functioning of other urban systems.

THE POLICY OF THE CCRFCD SHALL BE TO CONSIDER STORMWATER DRAINAGE AN INTEGRAL PART OF THE OVERALL URBAN SYSTEM, AND TO REQUIRE STORM DRAINAGE PLANNING FOR ALL DEVELOPMENTS TO INCLUDE THE ALLOCATION OF SPACE FOR DRAINAGE FACILITY CONSTRUCTION AND MAINTENANCE, WHICH MAY ENTAIL THE DEDICATION OF ROW AND/OR EASEMENTS.

302.2 Multi-Purpose Resource

Stormwater runoff is a resource that has the potential, although limited in a desert environment, of being utilized for different beneficial uses. These uses, however, must be compatible with adjacent land uses and applicable water laws.

THE POLICY OF THE CCRFCD SHALL BE TO CONSIDER STORMWATER RUNOFF AS A RESOURCE AND RECOGNIZE ITS POTENTIAL FOR OTHER USES.

302.3 Water Rights

A drainage design must be planned and constructed with proper recognition given to the existing water rights and applicable water laws. When the drainage system interferes with existing water rights, the value and use of the water rights are affected.

THE POLICY OF THE CCRFCD SHALL BE TO RECOGNIZE THE EXISTENCE OF VESTED WATER RIGHTS.

302.4 Jurisdictional Cooperation

Since drainage considerations and problems are regional in nature, and do not respect jurisdictional boundaries, drainage planning must emphasize regional jurisdictional cooperation, unified standards, and similar drainage requirements in accomplishing the goals.

THE POLICY OF THE CCRFCD SHALL BE TO PURSUE A JURISDICTIONALLY UNIFIED DRAINAGE EFFORT TO PROMOTE AN INTEGRATED DRAINAGE PLAN.

303 REGIONAL AND LOCAL PLANNING

303.1 Reasonable Use of Drainage

Drainage Law (Section 400) recognizes that downstream properties should not be unreasonably burdened with increased flow rates or unreasonable changes in manner of flow from upstream properties. The law also recognizes that drainage problems should not be transferred from one location to another (basin transfers). However, drainage law also acknowledges that downstream properties cannot block natural runoff through their site and must accept runoff from upstream properties.

Drainage planners have long realized that the development process alters historic or natural drainage paths and sets the stage for violation of the above stated drainage laws. However, strict compliance with the above laws can produce drainage systems which may be a detriment to the general public. Therefore, drainage planners follow a "Reasonable Use of Drainage" philosophy to provide for economic and efficient drainage systems within the limits of drainage laws.

Briefly stated, "Reasonable Use of Drainage" is defined for planning purposes, as providing an economic and hydraulically efficient drainage system which is demonstrated not to adversely impact downstream properties within reason. This "Reasonable Use of Drainage" therefore allows development to occur while preserving the rights of adjacent property owners.

THE POLICIES OF CCRFCD REGARDING THE "REASONABLE USE OF DRAINAGE" AS RELATED TO THE STATED DRAINAGE LAWS ARE PRESENTED IN THE FOLLOWING SECTIONS.

303.1.1 Increase in Rate of Flow

The process of development will generally increase the rate of flow to downstream properties due to increases in impervious area from buildings, streets, and parking lots. Mitigation of these increases are generally accomplished through detention and/or retention facilities.

THE POLICY OF THE CCRFCD SHALL BE TO MINIMIZE THE INCREASE IN THE RATE OF FLOW FROM DEVELOPING PROPERTIES UNLESS DOWN STREAM FACILITIES EXIST TO ACCOMMODATE THE INCREASED FLOW RATES.

The CCRFCD's policies for detention and retention are presented in Section 303.6.

303.1.2 Change in Manner of Flow

The process of development will tend to concentrate existing natural sheet flow into point flows at property lines. These point flows are generally associated with outlets from gutter flow, storm sewers, and detention facilities. Discharge of point flows on undeveloped downstream property can cause increased erosion at the discharge point and further downstream. Mitigation of these point flows can be accomplished through energy dissipators or flow spreaders.

THE POLICY OF CCRFCD SHALL BE TO REQUIRE THAT POINT FLOWS BE DISCHARGED TO DOWNSTREAM PROPERTIES AT NON-EROSIVE VELOCITIES AND DEPTHS OF FLOW.

303.1.3 Diversion of Drainage

The process of development can alter the historic or natural drainage paths. When these alterations result in a local on-site drainage system that discharges back into the natural drainageway or wash at or near the historic location, then the alterations (intra-basin transfer) are generally acceptable. However, when flows from the local on-site drainage system do not return to the historic drainageway or wash, then inter-basin transfer may result. These inter-basin transfers are generally not acceptable. Planning and design of drainage systems should not be based on the premise that problems can be transferred from one location or basin to another. Every reasonable attempt shall be made to mitigate non-stormwater nuisance flows.

THE POLICY OF THE CCRFCD SHALL BE TO MAINTAIN THE FLOW OF STORM RUNOFF WITHIN ITS NATURAL DRAINAGE PATH UNLESS REASONABLE USE IS DEMONSTRATED OTHERWISE.

303.2 Regional Master Planning

As mandated in NRS 543.590, the CCRFCD has prepared and the BOARD has adopted a Regional Master Plan. The NRS further mandates that each entity hold a public hearing to consider adopting the Regional Master Plan as a component of its Local Master Plan pursuant to Chapter 278 of NRS.

THE POLICY OF THE CCRFCD SHALL BE TO FOLLOW THE ADOPTED REGIONAL MASTER PLAN WHICH SETS FORTH THE MOST CURRENTLY EFFECTIVE STRUCTURAL AND REGULATORY MEANS FOR CORRECTING EXISTING FLOODING WITHIN AN AREA TAKING INTO ACCOUNT THE POSSIBLE EFFECTS OF FUTURE DEVELOPMENT. THE REGIONAL MASTER PLAN HAS INCORPORATED, IN SO FAR AS POSSIBLE, PLANNING COMPLETED OR UNDERTAKEN BY THE LOCAL ENTITIES AND DEVELOPERS. THE CCRFCD RECOGNIZES THE NEED TO MODIFY AND/OR REVISE PORTIONS OF THE ADOPTED MASTER PLAN FROM TIME TO TIME TO REFLECT CHANGES DESIRED BY THE LOCAL ENTITIES AS LONG AS THE INTENT AND INTEGRITY OF THE REGIONAL MASTER PLAN ARE NOT COMPROMISED.

THE CCRFCD WILL ALSO REVIEW THE REGIONAL MASTER PLAN ANNUALLY TO ADDRESS THE REQUIREMENTS LISTED IN THE "POLICIES AND PROCEDURES."

303.3 Local Master Planning

The CCRFCD's Master Plan provides a plan to handle the regional flows within an area. The CCRFCD's Master Plan depicts the main arteries of the necessary drainage system. Local Flood Control Facilities, as planned by the entities and/or developers, are an integral part of the total drainage system required to preserve and promote the general health, welfare, and economic well being of the area.

THE POLICY OF THE CCRFCD SHALL BE TO ENCOURAGE THE LOCAL ENTITIES TO DEVELOP LOCAL MASTER PLANS FOR LOCAL FLOOD CONTROL FACILITIES WHICH ARE COMPATIBLE WITH THE REGIONAL MASTER PLAN. THESE LOCAL MASTER PLANS SHALL ALSO SET FORTH SITE REQUIREMENTS FOR NEW DEVELOPMENT AND IDENTIFY THE REQUIRED PUBLIC IMPROVEMENTS.

303.4 Drainage Improvements

Drainage improvements, as defined in local development drainage plans or local and regional plans, are classified as either Local Flood Control Facilities or Regional Flood Control Facilities. The Local Flood Control Facilities consist of curb and gutter, inlets and storm sewers, culverts, bridges, swales, ditches, channels, detention areas, and other drainage facilities required to convey the minor and major storm runoff to the Regional Flood Control

Facilities. These Local Flood Control Facilities are further defined as on-site (private) facilities and off-site (public) facilities. The on-site (private) facilities serve a specific development and are privately owned and maintained. The off-site (public) facilities are facilities which are dedicated to the public and connect on-site (private) facilities to the Regional Flood Control Facilities. These off-site (public) facilities may actually be constructed within the specific development to pass flow through from upstream properties. The Regional Flood Control Facilities consist of channels, storm sewers, bridges, detention areas, and other facilities which carry runoff from on-site and off-site facilities to an ultimate outfall location.

When drainage plans identify that off-site (public) facilities improvements are justified, mechanisms for funding the improvements are required. The funding for off-site (public) facilities improvements which serve only a single development should be obtained from that development. This funding is provided by having these off-site (public) facilities improvements designed and constructed by the subject development.

THE POLICY OF THE CCRFCD SHALL BE THAT ALL NEW DEVELOPMENT DESIGN AND CONSTRUCT THE REQUIRED DRAINAGE IMPROVEMENTS AS SET FORTH BELOW:

1. Local On-Site (Private) Flood Control Facilities.

2. Local Off-Site (Public) Flood Control Facilities are required to provide adequate conveyance capacity from the Local On-Site (Private) Flood Control Facilities to the Regional Flood Control Facilities or for pass through of upstream off-site runoff. Oversizing of the Local (Public) Off-Site Flood Control Facilities to accommodate future development may be required by the local entity. The local entity may also require payment to a local (Public) off-site facilities fund in lieu of construction of these facilities by the developer.

3. Regional Flood Control Facilities passing through or directly adjacent to the subject development. The CCRFCD may participate in funding of these regional improvements within the limits of the most current POLICIES AND PROCEDURES if the improvements are designed and constructed in accordance with the Regional Master Plan and this MANUAL.

303.5 Drainage Planning Submittal and Review

Review and approval of drainage plans, studies, and construction drawings and specification by the local entities and CCRFCD is required to obtain a final drainage system which is consistent and integrated in analysis, design, and level of protection. The degree of review and approval required depends on the nature of the drainage improvement under consideration.

THE POLICY OF THE CCRFCD IS TO REQUIRE THAT ALL DRAINAGE PLANS, STUDIES, AND CONSTRUCTION DRAWINGS AND SPECIFICATIONS BE REVIEWED AND APPROVED AS SET FORTH BELOW:

1. The entity shall be responsible for review and approval of all Local Flood Control Facilities, except when the proposed facilities impact the areas listed below, they shall also be approved by the CCRFCD.

   a. Identified Flood Hazard Areas in accordance with FEMA regulations

   b. Regional Flood Control Facilities

   c. Facilities with Regional Significance as defined in the REGULATIONS

2. Local entities which construct flood control facilities must comply with the CCRFCD's Master Plan. Local entities may request an amendment or obtain a variance to the CCRFCD's Master Plan pursuant to NRS.

3. State Agencies shall consider and, when applicable, comply with the CCRFCD's Master Plan when planning and designing their flood control facilities.

303.6 Floodplain Management

The foremost goal of many successful businesses is to obtain the greatest return for the least cost. When applied to land development, this goal translates into obtaining the largest developable land area using the most economic measures. Thus, existing floodplain land becomes more valuable if the land can be removed from the floodplain for development. The purpose of floodplain management is to provide the guidance, conditions, and restrictions for development in floodplain areas while protecting the public's health, safety, welfare, and property from danger and damage.

To provide impetus for proper floodplain management, the United States government, acting through the FEMA’s National Flood Insurance Program (NFIP), has established regulations for development in floodplain areas. Compliance with these regulations allows property owners to obtain lower cost flood insurance premiums and/or eliminates the requirement for the owner to obtain flood insurance as a condition for obtaining government supported loans. Therefore, there is a benefit to the CCRFCD population for remaining in compliance with the NFIP's regulations. The guidance, conditions, and restrictions for development in floodplain areas as presented in the REGULATIONS are used to meet the above stated floodplain management objectives.

THE POLICY OF THE CCRFCD SHALL BE TO REGULATE FLOODPLAINS IN ACCORDANCE WITH THE DISTRICT'S ADOPTED UNIFORM REGULATIONS FOR THE CONTROL OF DRAINAGE AND THE REGULATIONS OF THE NFIP.

Since FEMA policies can change, their regulations are not specifically cited in the MANUAL. It is incumbent upon the local engineering community to keep abreast of FEMA's regulations. FEMA has adopted the 100-year flood (1 percent chance of annual occurrence) as the base flood for floodplain management purposes and delineates the 100-year floodplain on their maps. For certain stream courses studied by FEMA by detailed methods, a floodway may also be depicted. The floodway is a portion of the floodplain and is defined as the channel itself plus any adjacent land areas which must be kept free of encroachment in order to pass the base flood without increasing water surface elevations by more than a designated height. **Figure 301** depicts a typical floodplain and floodway along with some of the terminology found in FEMA's regulations. The following subsections discuss in general terms some other terms and issues having to do with FEMA's regulations.

303.6.1 FEMA Map Revisions and Amendments

FEMA has a number of different procedures for requesting changes to their Flood Insurance Rate Maps (FIRMs). Since the FIRMs for Clark County and the incorporated jurisdictions are effective (dated August 16, 1995), changes to them are handled either as a Letter of Map Revision (LOMR), Letter of Map Amendment (LOMA), or publication of a revised map, also known as a "physical map revision" (PMR). FEMA may issue a conditional letter of map revision (CLOMR or CLOMA) if the request is based on a proposed project. If exclusion from a Special Flood Hazard Area (SFHA) is due to elevating a structure on fill, the map revision is designated LOMR-F or CLOMR-F.

Any change to the Special Flood Hazard Information is handled as a **map revision.** These changes can include changes to floodplain boundaries, floodway boundaries, flood insurance risk zones, base flood elevations, flood depths, and other information shown on the maps. All changes must be based on existing conditions, although a conditional determination may be requested for proposed projects, such as modifications to stream channels and floodplains, and proposed elevation on fill of structures or parcels of land.

Due to the scale of FEMA's maps, individual structures or legally-described parcels of land may be inadvertently included in a SFHA. Excluding an individual structure or parcel of land from the SFHA is handled as a **map amendment**. The map amendment process is not applicable to requests involving changes to Special Flood Hazard Information on the maps and cannot be based on new topography or hydrologic or hydraulic conditions.

The following subsections briefly discuss the various types of map revisions and amendments and how to request them. More detailed information is available from CCRFCD.

303.6.1.1 Map Revisions

Any information on FEMA's FIRMs may be changed, subject to FEMA review. However, FEMA generally only revises effective maps if changes affect 100-year flood information. Map revisions can either be a PMR or LOMR. In general, FEMA only processes PMRs only when changes involve a large area of land or increased flood risk. Otherwise, LOMRs are issued if changes involve small areas within a community.

In Clark County, the CCRFCD and the individual agencies are responsible for ensuring all obligations are met to allow the County and incorporated cities to participate in the NFIP. Although private parties may request a map revision, it is strongly suggested that such requests be submitted or coordinated with CCRFCD. If FEMA receives any map revision requests in Clark County from a private party without the concurrence of CCRFCD, the requestor will be asked for evidence that the request was first submitted to CCRFCD.

Map revisions typically fall into three categories – those based on the effects of physical changes in the floodplain, those based on the use of better data, and those based on the use of alternative methodology.

If a structure or parcel of land is elevated on fill, a map revision may be issued to remove the structure from a SFHA if both the lowest adjacent grade to the structure and the lowest floor are at or above the base flood elevation (BFE) or when the placement of fill has elevated a legally-defined parcel of land at or above the BFE. Requests for such map revisions are called LOMR-F or CLOMR-F, depending if the structure is existing or proposed.

Because each request for a map revision is unique, the required forms, supporting data, and submittal fees vary widely. Requestors should contact CCRFCD for a FEMA Application/Certification Forms and Instructions for Amendments and Revisions to National Flood Insurance Program Maps.

303.6.1.2 Map Amendments

Typically, the scale of FEMA's maps do not allow the floodplain delineations to be shown in the detail required to determine whether an individual structure or legally-described parcel is within the SFHA. Similarly, existing small areas of high ground may be shown within the SFHA because they are too small to be shown to scale. FEMA has developed the map amendment process to allow property owners or lessees to request that FEMA determine whether a specific structure or parcel is in the SFHA and, if necessary, issue a Letter of Map Amendment.

It is important to note that a LOMA should only be requested on the basis of an inadvertent inclusion in the SFHA and not due to recent alterations of topography or significant changes to the flooding information shown on the FEMA map. In either of these cases, the request should be submitted as a map revision rather than a map amendment.

Although less common than conditional map revisions, a conditional map amendment (CLOMA) can be requested if an individual intends to build a structure(s) on a single or multiple lots, but not on fill, and wants FEMA to determine whether the structure will be excluded from the SFHA shown on the effective maps.

As with map revisions, because each request for a map amendment is unique, the required forms, supporting data, and submittal fees vary widely. Requestors should contact CCRFCD for a FEMA Application/Certification Forms for Amendments and Revisions to National Flood Insurance Program Maps.

303.6.2 Levee Freeboard Criteria

If a flood control levee is proposed within a FEMA SFHA and a map revision will be requested based on the levee providing protection against the 100-year flood, FEMA's levee criteria shall be used in order for FEMA to credit the levee. It is noted that FEMA's levee policy requires that the levee be maintained by a governmental agency and certified according to the Code of Federal Regulations in order to be recognized as a providing flood protection. Therefore, any levees in FEMA 100-year SFHAs shall be coordinated with the local jurisdiction and CCRFCD.

Local levees are those which are not maintained by a governmental agency. For local levees, freeboard shall be provided per Section 700 of the MANUAL. A sediment study shall be performed per Section 700 of the MANUAL. Non-erosive velocities shall be maintained, or erosion protection shall be provided to ensure the integrity of the levee during storm events.

303.6.3 Vertical Control (NGVD 29 vs. NAVD 88)

The base flood elevations and elevation reference marks on the current FEMA Flood Insurance Rate Maps for Clark County and incorporated areas are based on the National Geodetic Vertical Datum of 1929 (NGVD 29). The National Geodetic Survey has determined that the national vertical control network needs to be readjusted and FEMA is in the process of converting all National Flood Insurance Program maps to the new national datum, called the North American Vertical Datum of 1988 (NAVD 88).

In addition, the Clark County Department of Public Works, Surveyor's Office, has based the official Clark County Vertical Control on NAVD 88 and requires that all new maps, plans, reports, and other documents submitted for review reflect elevations referenced to NAVD 88. Copies of the Clark County benchmark information may be obtained from the County Surveyor's Office. However, FEMA requires that requests for map revisions and amendments use the reference datum on the applicable, effective FIRM map which, in Clark County, is NGVD 29. Therefore, as of publishing, actual work should be done in NAVD 88 and a conversion factor to NGVD 29 should accompany any submittal to FEMA.

Additional information regarding the conversion from NGVD 29 to NAVD 88 is available from CCRFCD or the County Surveyor.

303.7 Stormwater Runoff Detention

The value of stormwater runoff detention as part of the urban system has been explored by many individuals, agencies, and professional societies. Detention is considered a viable method to reduce urban drainage costs. Temporarily detaining stormwater runoff can significantly reduce downstream flood hazards as well as pipe and channel requirements in urban areas. Storage also provides for sediment and debris collection which helps to keep downstream channels and streams cleaner.

THE POLICY OF THE CCRFCD SHALL BE TO REQUIRE LOCAL DETENTION STORAGE FOR NEW DEVELOPMENTS WHEN THE DEVELOPMENT IN CREASES FLOWS AND DOWNSTREAM CONVEYANCE CAPACITIES OF THE LOCAL DRAINAGE SYSTEM ARE DEMONSTRATED NOT TO BE CAPABLE OF HANDLING NON-DETAINED FLOWS.

THE CAPACITY OF DOWNSTREAM CONVEYANCE SYSTEMS SHALL BE ANALYZED IN ACCORDANCE WITH THIS MANUAL AND SHALL BE BASED ON RUNOFF FROM THE DEVELOPMENT AS FULLY IMPROVED. LOCAL DETENTION IS ALSO REQUIRED WHEN DESIGNATED IN LOCAL MASTER PLANS TO REDUCE THE PEAK RUNOFF RATE IN REGIONAL FACILITIES.

EXEMPTIONS TO THE DETENTION POLICY MAY BE GRANTED BY THE LOCAL ENTITY FOR THE FOLLOWING:

1. DEVELOPMENTS OF LESS THAN 2 ACRES WITH AN IMPERVIOUS DENSITY OF 50 PERCENT OR LESS.

2. ADDITIONS TO BUILDINGS PROVIDED THE IMPERVIOUS DENSITY OF THE ENTIRE PROPERTY DOES NOT INCREASE BY MORE THAN 10 PERCENT OR THE TOTAL IMPERVIOUS AREA DOES NOT INCREASE BY MORE THAN ONE ACRE, WHICHEVER IS LESS.

3. DEVELOPMENTS WHICH DISCHARGE DIRECTLY TO A REGIONAL FLOOD CONTROL FACILITY PROVIDED THE REGIONAL FACILITY IS COMPLETED PER THE ADOPTED MASTER PLAN.

4. LOCATIONS WHERE A LOCAL DETENTION FACILITY IS PLANNED TO SERVE SEVERAL DEVELOPMENTS. FOR THIS EXEMPTION, THE LOCAL ENTITY MAY REQUIRE PAYMENT TO A LOCAL DETENTION FACILITIES FUND IN LIEU OF CONSTRUCTION OF THE DETENTION FACILITY BY THE DEVELOPER.

ALL EXEMPTIONS ARE SUBJECT TO APPROVAL OF THE LOCAL ENTITY. ADDITIONAL ANALYSIS MAY BE REQUESTED BY THE LOCAL ENTITY TO DEMONSTRATE THE BENEFITS OBTAINED BY GRANTING OF THE EXEMPTION.

303.8 Stormwater Runoff Retention

Stormwater runoff retention has been used to eliminate the need for constructing outlet structures and for ease of construction. However, problems with past retention basins including soil expansion, siltation, and lack of infiltration capacity have created a nuisance to the general public.

THE POLICY OF THE CCRFCD SHALL BE TO DISCOURAGE THE USE OF RETENTION FACILITIES EXCEPT WHERE THERE ARE NO DOWNSTREAM FACILITIES TO CONVEY SITE RUNOFF. SPECIAL FACILITIES (I.E., LEACHFIELDS, ETC.) MAY BE REQUIRED BY THE LOCAL ENTITY BEFORE A RETENTION BASIN IS ALLOWED.

303.9 Water Quality

In January 1984, the United States Environmental Protection Agency (USEPA) presented to Congress the results of the Nationwide Urban Runoff Program (NURP). The purpose of this study was to characterize and evaluate the quality of runoff from and due to urbanized areas. The results of the study showed that the process of urbanization decreases the quality of runoff from the natural conditions. In addition, USEPA has adopted regulations to control pollutants from entering the environment through storm drainage facilities. These regulations are administered locally through a municipal stormwater discharge permitting program by the Nevada Division of Environmental Protection (NDEP). The local National Pollutant Discharge Elimination System (NPDES) stormwater permit is issued jointly to CCRFCD, City of Las Vegas, City of North Las Vegas, City of Henderson, and Clark County. These co-permittees have joined in a cooperative, multi-jurisdictional effort to comply with the permit requirements and address other regional stormwater quality issues. Details of the NPDES permit are outlined in Section 207 of the MANUAL.

THE POLICY OF THE CCRFCD SHALL BE TO ENCOURAGE THE DESIGN OF DRAINAGE FACILITIES AND OTHER MEASURES WHICH ENHANCE THE QUALITY OF STORM RUNOFF.

303.10 Drainage Facilities Maintenance

An important part of all storm drainage facilities is the continued maintenance of the facilities to ensure they will function as designed. Maintenance of detention facilities involves removal of debris and sediment. Such tasks are necessary to preclude the facility from becoming unhealthy and to retain the effectiveness of the detention basin. Sediment and debris must also be periodically removed from channels and storm sewers. Trash racks and street inlets must be regularly cleared of debris to maintain system capacity. Channel bank erosion, damage to drop structures, crushing of pipe inlets and outlets, and deterioration to the facilities must be repaired to avoid reduced conveyance capability, unsightliness, and ultimate failure.

THE POLICY OF THE CCRFCD SHALL BE TO REQUIRE ALL DRAINAGE FACILITIES BE DESIGNED TO MINIMIZE FACILITY MAINTENANCE AS WELL AS TO PROVIDE EASE OF MAINTENANCE AND INCLUDE MAINTENANCE ACCESS TO THE ENTIRE DRAINAGE FACILITY. A MINIMUM 20-FOOT WIDE DRAINAGE EASEMENT SHALL BE PROVIDED FOR *ALL* PUBLICLY MAINTAINED DRAINAGE FACILITIES. SEE SECTION 1600 FOR LOCAL ENTITY MINIMUM REQUIREMENTS.

THE POLICY OF THE CCRFCD IS TO REQUIRE THE PROPERTY OWNER OR DEVELOPER BE RESPONSIBLE FOR MAINTENANCE OF ALL PRIVATELY OWNED ON-SITE DRAINAGE FACILITIES INCLUDING, BUT NOT LIMITED TO, INLETS, PIPES, CHANNELS, BEST MANAGEMENT PRACTICES (BMPs), AND DETENTION BASINS, UNLESS MODIFIED BY SEPARATE AGREEMENT. SHOULD THE PROPERTY OWNER OR DEVELOPER FAIL TO ADEQUATELY MAINTAIN SAID FACILITIES, THE GOVERNING ENTITY SHALL BE GIVEN THE RIGHT TO ENTER SAID PROPERTY, UPON PROPER NOTICE, FOR THE PURPOSES OF MAINTENANCE. ALL SUCH MAINTENANCE COSTS SHALL BE ASSESSED AGAINST THE OWNER.

THE POLICY OF THE CCRFCD SHALL BE TO ENSURE ALL FLOOD CONTROL FACILITIES ARE PROPERLY MAINTAINED AND TO FUND THE MAINTENANCE OF REGIONAL FLOOD CONTROL FACILITIES. THE LOCAL ENTITIES WILL BE RESPONSIBLE FOR THE MAINTENANCE OF ALL PUBLIC DRAINAGE FACILITIES NOT MAINTAINED BY CCRFCD. FOR PRIVATELY MAINTAINED FACILITIES, THE MAINTENANCE PROCEDURES SHALL BE SUBMITTED AS PART OF THE DRAINAGE STUDY REQUIREMENTS.

304 TECHNICAL CRITERIA

304.1 Stormwater Management Technology

The information presented in this MANUAL represents the current state-of-the-art in stormwater management planning and design. However, the dynamic nature of stormwater runoff technology, information, and criteria will continue to advance the state-of-the-art of stormwater management. Therefore, this MANUAL should be periodically updated to account for advances made in the stormwater management field.

THE POLICY OF THE CCRFCD SHALL BE TO KEEP ABREAST OF THE STATE-OF-THE-ART IN STORMWATER MANAGEMENT AND AMEND AND/OR MODIFY THESE CRITERIA AS NEW TECHNOLOGY IS DEVELOPED AND EXPERIENCE GAINED IN THE USE OF THESE CRITERIA.

304.2 Design Storm Events

The selection of design storm events is based on many factors including public perception, federal regulations, balance of economics versus public safety, physical basin characteristics, and typical storm patterns. Typically, the public perceives only two types of storm events; minor and major. The minor storm events are perceived as typical small rain storms of very short duration and low rainfall intensity which cause few problems or inconvenience to the general public. These storms appear to occur with greater frequency (every year or several years) than the larger major storms. The major storm events are perceived as occurring infrequently and appear to cause major damage to public property and possibly loss of life.

Without properly designed drainage facilities, the minor storms can also cause more damage and inconvenience than the public perception would allow. Therefore, facilities should be designed to minimize public inconvenience for minor storm events and protect public property and life for major storm events.

The federal government has recognized the need to protect the general public from catastrophic damage and destruction associated with major storm events. This recognition has resulted in the issuance of floodplain regulations for the 100-year storm event (major storm).

The CCRFCD, in reviewing the factors stated above for selection of design storm events, has determined that drainage facilities should be designed based on runoff from two storm events; a minor storm event and a major storm event. In addition, the CCRFCD has determined that the general interest of the Clark County area is better served by providing a minor storm system design which accommodates minor storm flows of greater intensity and/or duration than the typical small annual storms.

THE POLICY OF THE CCRFCD SHALL BE TO REQUIRE ALL NEW DEVELOPMENT TO INCLUDE THE PLANNING, DESIGN, AND CONSTRUCTION OF DRAINAGE FACILITIES FOR BOTH THE MINOR AND MAJOR STORM EVENTS AND WILL INCLUDE EMERGENCY FLOW PATHS FOR FLOWS EXCEEDING THE MAJOR STORM. THE MINOR STORM EVENT SHALL HAVE A RECURRENCE INTERVAL OF 10 YEARS. THE MAJOR STORM EVENT SHALL HAVE A RECURRENCE INTERVAL OF 100 YEARS.

304.3 Stormwater Runoff Determination

The stormwater runoff peak, volume, and timing provide the basis for all planning, design, and construction of drainage facilities. The best method for determining stormwater runoff is to measure the runoff from a flood with a known intensity and recurrence interval. Since this approach is seldom practical, various analytical methods have been developed which predict the stormwater runoff from preselected hydrologic conditions (independent of chance). These methods are referred to as deterministic models. Other methods evaluate measured past trends to predict future trends, which are referred to as probabilistic methods (dependent on chance such as a statistical analysis). The general lack of sufficient rainfall/runoff data in the Clark County area presently precludes the use of probabilistic models for normal stormwater runoff calculations.

THE POLICY OF THE CCRFCD SHALL BE TO REQUIRE THE DETERMINATION OF STORMWATER RUNOFF (RATES AND VOLUMES) IN ACCORDANCE WITH THE FOLLOWING:

CONTRIBUTING BASIN AREA	COMPUTATION PROCEDURE
A $\leq$ 150 ACRES	MODIFIED RATIONAL FORMULA METHOD, SCS TR-55, OR HEC-1 (SCS UNIT HYDROGRAPH OR KINEMATIC WAVE)
A > 150 ACRES	HEC-1 (SCS UNIT HYDROGRAPH OR KINEMATIC WAVE)

304.4 Streets

The use of streets to convey stormwater runoff, although naturally occurring, interferes with the primary function of the street for transportation purposes. Streets are, however, an important component in the storm drainage system due to their large stormwater runoff carrying capacity obtained for little or no drainage related costs. In order to balance these two competing street uses, limits on the street carrying capacity are required based on the classification of the street related to emergency usage during flood events. This classification generally follows the streets' traffic classification. For example, the wider and more frequently used arterial streets are restricted further in stormwater runoff flow depths and capacity than local or collector streets.

At street intersections or sag points between intersections, two alternatives exist for positive conveyance of gutter flow past or through these locations. These two options are:

a) Provide storm inlets and storm sewer to pass the gutter flow under the street or road.

b) Provide a concrete valley gutter across the street and/or intersection.

The selection of which alternative to use for a given location is dependent on the street classification (i.e., arterial versus local) and on the flow direction past the intersection or sag point. The drainage planner should consult with the appropriate local entity in selection of the appropriate alternative.

Bubbler laterals have been used in the past to convey stormwater runoff under streets without constructing additional storm sewers from the downstream inlets. Although these facilities work well theoretically, problems exist in the actual field installations. These problems include the gradual siltation of the structures over time as well as stagnation of water trapped in the laterals. Therefore, bubbler facilities are prohibited unless special site conditions warrant otherwise and permission is obtained from the local entity and/or CCRFCD.

THE POLICY OF THE CCRFCD SHALL BE TO ALLOW THE USE OF STREETS FOR STORM DRAINAGE WITH SPECIFIC LIMITATIONS. THE ALLOWABLE STREET CAPACITY FOR THE MINOR STORM EVENT FOR DIFFERENT STREET ROW WIDTHS SHALL BE DETERMINED AS FOLLOWS:

	ROW WIDTH		MINOR STORM STREET CAPACITY LIMITATIONS (See Note)
1)	Greater than or equal to 80 feet	A.	A 12-foot wide dry lane shall be maintained in each direction (center turning lane cannot be used for a dry lane)
		B.	The depth of flow at intersections with other streets with ROW widths greater than or equal to 80 feet shall be curb height (typically 6 inches) or less.
		C.	The product of the flow depth (feet) in the gutter flow line times the average flow velocity (feet per second (fps)) shall be less than or equal to 6.
2)	Less than 80 Feet	A.	The depth of flow in the gutter flow line shall be less than or equal to 1 foot.
		B.	The product of the flow depth (feet) in the gutter flow line times the average flow velocity (fps) shall be less than or equal to 6.

Temporary streets will be treated as minor streets (less than 80 feet wide).

THE ALLOWABLE STREET CAPACITY FOR THE MAJOR STORM EVENT SHALL BE DETERMINED AS FOLLOWS:

MAJOR STORM STREET CAPACITY LIMITATIONS (See Note)

A. The product of the flow depth (feet) in the gutter flowline times the average flow velocity (fps) shall be less than or equal to 8.

B. The depth of flow in the gutter flowline shall be less than or equal to 2 feet.

C. The transverse street flow on through traffic streets shall be at a depth less than or equal to 1 foot and a depth times velocity less than or equal to 8. This shall not be used to determine the allowable transverse flow depth for a culvert.

D. In Special Flood Hazard Areas and Areas of Interim Delineation:

1. The residential finished floor (new construction) shall be a minimum of 18 inches above the water surface elevation in the street.

2. The non-residential finished floor (new construction) shall be a minimum of 18 inches above the water surface elevation in the street or be floodproofed to 18 inches above the water surface elevation in the street.

E. In Non-Special Flood Hazard Areas for new construction (residential and non-residential):

1. The finished floor shall be set at a vertical distance above the gutter flowline of at least twice the depth of flow in the gutter flowline up to a maximum of 18 inches above the water surface elevation in the street.

Note: If any of the above conditions cannot be met, the maximum depth of flow in the gutter flowline shall be less than or equal to curb height (typically 6 inches). Also, other criteria such as the Federal Housing Administration (FHA) regulations may impose standards more restrictive than cited.

Figure 302 depicts typical lot grading and drainage.

304.5 Culverts and Bridges

Culverts and bridges are required where natural or manmade channels are crossed by roads and streets. The amount of channel flow which crosses over the road should be minimized to protect the road embankment and pavement from erosion damage as well as to protect vehicles and pedestrians from dangerous flow depths and velocities.

THE POLICY OF THE CCRFCD SHALL BE TO ALLOW CULVERT/BRIDGE CROSSINGS UNDER STREETS WITHIN THE FOLLOWING LIMITATIONS:

ROW Width	Minimum Capacity (Recurrence Interval)
Greater than or equal to 80 feet	100-Year (No Overflow)
Less than 80 feet	100-Year (See Note)

Note: An overflow section is allowed if the product of the depth in feet and the velocity in fps does not exceed 6. Also the maximum depth of flow in the overflow section shall not exceed 1 foot measured at the street gutter, or the lowest point on the street cross section.

304.6 Floodproofing

Floodproofing can be defined as those measures which reduce the potential for flood damages to properties within a floodplain. The floodproofing measures can range from elevating structures to intentional flooding of non-critical building spaces (i.e., basement) to minimize structural damages. Floodproofing measures are only a small part of good floodplain management which encourages wise floodplain development to minimize the adverse effects of floods.

Floodproofing can be divided into measures required for protection of existing structures or future structures. For any future construction, the floodproofing requirements are controlled by the floodplain regulations (see Section 303.6) which generally restrict the types of structures within a floodplain.

THE POLICY OF THE CCRFCD SHALL BE TO ENCOURAGE THE FLOODPROOFING OF EXISTING STRUCTURES LOCATED WITHIN A DESIGNATED FLOODPLAIN AREA WHICH ARE NOT BUILT IN CONFORMANCE TO THE ADOPTED FLOODPLAIN REGULATIONS.

304.7 Alluvial Fans

Alluvial fans are a common and dominant feature in the arid west. These fans, consisting of sand and fine sediment, are subject to radical changes in shape, direction, depth, and flow carrying capacity during storm events. These erratic changes increase the potential flood hazards of developing on alluvial fan areas and require additional analysis and design to provide safe and effective facilities to accommodate these hazards.

THE POLICY OF THE CCRFCD SHALL BE TO REQUIRE ADDITIONAL ANALYSIS OF RUNOFF FLOW PATTERNS AND THE EFFECT OF NATURAL RADICAL CHANGES OF THESE PATTERNS ON THE DESIGN OF DRAINAGE FACILITIES FOR PROPOSED DEVELOPMENTS ON AN ALLUVIAL FAN.

THE POLICY OF THE CCRFCD SHALL BE TO KEEP ABREAST OF THE LATEST TECHNOLOGY ON ALLUVIAL FANS AND TO DISPERSE THIS INFORMATION AS IT BECOMES AVAILABLE.

304.8 Multiple Use Facilities

In urban areas the creative use and identification of open space opportunities is important. In many cases linear and block flood control facilities can provide opportunities for trails, parks, environmental preserves and many other recreational uses.

THE POLICY OF CCRFCD SHALL BE TO ENCOURAGE EARLY PLANNING TO IDENTIFY AND TAKE ADVANTAGE OF MULTIPLE USE OPPORTUNITIES AFFORDED BY FLOOD CONTROL FACILITIES INCLUDED ON THE MASTER PLAN. TO THAT END MASTER PLANS, DESIGNS AND CAPITAL IMPROVEMENT PROGRAMS WILL BE MADE AVAILABLE TO ENTITY PLANNING PERSONNEL FOR THEIR USE.

Inasmuch as multiple uses do not contribute to the mission of CCRFCD:

THE POLICY OF CCRFCD SHALL BE TO RESTRICT THE USE OF CCRFCD FUNDS TO IMPLEMENTATION OF THE FACILITIES INCLUDED ON THE FLOOD CONTROL MASTER PLAN. CCRFCD FUNDING IS NOT AVAILABLE FOR THE INSTALLATION, OPERATION, MAINTENANCE OR REHABILITATION OF RECREATIONAL OR OTHER NON-DRAINAGE RELATED FACILITIES LOCATED IN OR ALONG FLOOD CONTROL STRUCTURES.

That being said, certain considerations can be made during facility planning and design to better accommodate multiple uses. If a Master Plan Facility is to incorporate multiple uses, then the following policy statements shall be met:

A. Basic Policies:

1. Public safety and the proper functioning of the flood control facilities are of the highest concern and cannot be compromised by other uses.

2. Multiple use facilities within or along alignments of future proposed flood control improvements shall be designed in such a way as to not adversely affect implementation of the master plan.

3. Multiple use flood control facilities shall be clearly signed to identify them as areas subject to flooding, and as areas that shall not be used during rainfall or flood events. Signs shall include a 24-hour emergency telephone number for response personnel from the appropriate entity.

Signs shall be bilingual (English and Spanish) and pictorial. Signs shall be spaced, at a minimum, at all ingress and egress points for flood control facilities and at appropriately spaced intervals around the perimeter of detention basins. Signage and maintenance of signage shall be the sole responsibility of the entity.

4. Multiple use facilities and appurtenant amenities including vegetation shall be designed to not interfere with maintenance, operation and design capacity of the flood control facility. Estimates of full growth of mature trees shall be considered to ensure canopies will not interfere with high profile maintenance equipment and root systems will not reduce design life of flood control facilities. Levees, levee like structures, detention basins and debris basin embankments shall maintain a root free tree zone across the levee/embankment and extending to 15' from the toe of the levee/embankment slope.

5. The establishment of wetlands, passive vegetation zones, or other desirable habitat will require the entity to coordinate with and obtain approval by appropriate local, state and federal agencies, as well as the development of a workable habitat management plan that allows for the periodic maintenance of the drainage facilities.

6. If lighting is provided, the entity will ensure that electrical service access, switches, luminaries, etc., shall be located above the water surface elevation of the 100-year pool or be designed to automatically shut off if inundated.

7. Multiple use flood control facilities shall have emergency operation plans developed that clearly state the conditions under which these facilities will be closed to the public, as well as an implementation plan to ensure that the facilities are not being used under those conditions. The development, approval and implementation of these emergency operation plans are not the responsibility of CCRFCD. The emergency operation plan shall be prepared by the entity prior to opening facility to the public.

B. Specific Policies. Each entity will be responsible for compliance with the following provisions:

1. Detention Basins

a. The use of benching or other means to create areas which would not be subject to flooding during more frequent events is encouraged. Consideration must be given to storm water flow across these benched areas to lower lying areas. The following requirements shall apply:

I. The use of those portions of detention basins subject to flooding during a 10-year or more frequent event, other than as wildlife habitat or passive uses, is not allowed.

II. In general, multiple use facilities (i.e. recreation zones, picnic areas, soccer fields and ball fields) may be suitable uses for the lower elevated benches above the water surface elevation of the 10-year pool.

III. Play areas and court games shall be located on higher elevated benches above the water surface elevation of the 25-year pool.

Tot lots, parking areas and rest rooms (including portable facilities), concession stands, habitable structures, and swimming pools shall be located above the water surface elevation of the 100-year pool.

b. Detention basin embankments shall have gentle slopes (3H:1V or flatter) on the impoundment area side to allow the public to exit multiple use facilities.

c. Picnic tables, benches, trash receptacles, sport goals, and other amenities located in flood control facilities shall be securely fastened in place.

2. Linear Facilities

a. Due to hazardous conditions associated with untreated urban runoff, high velocity and fast rising flood waters prevalent in improved channels, all multiple use facilities shall be designed to keep users out of the designed flow conveyance area. However, while discouraged, if there are instances where a trail is proposed to pass under a transverse crossing, at a minimum, the trail shall be elevated above the 10-year storm water surface elevation. The trail system shall be designed to allow users to exit the trail at roadway crossings without passing below the road during adverse weather conditions.

b. A fence or other structure that prevents user from accessing the channel in accordance with local, state and federal requirements shall be designed and installed in a manner that does not impair normal operation and maintenance activities, or emergency response and rescue activities.

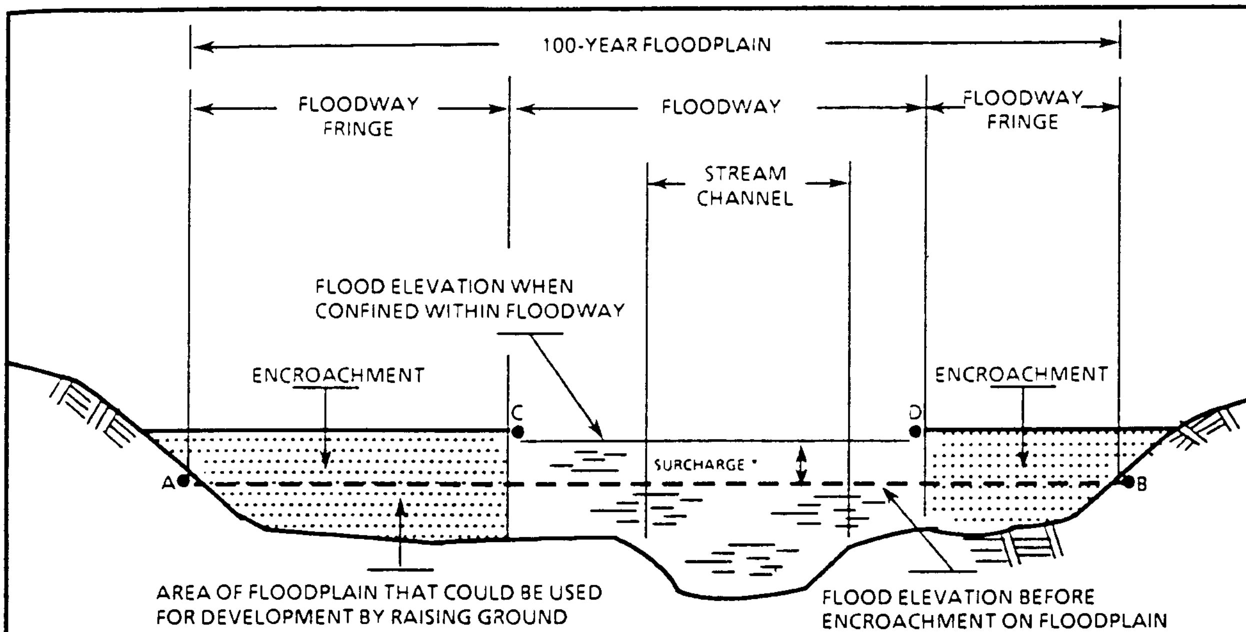
3. Natural Watercourses

Natural watercourses are subject to hazards associated with untreated urban runoff, high velocity and fast rising flood waters. Depending on natural watercourse characteristics, construction and maintenance of multiple use facilities (trails, ATV uses, golf course, etc.) within the flow conveyance poses different and unique challenges. Based upon utilizing the best available information, the owner should consider if depth of flow, cross-sectional characteristics, ability to warn the public and proposed use of the natural watercourse will allow the user to safely evacuate the hazard.

C. Effective Date

The policy is effective for all multiple use facilities after the date approved by the Board of Directors. Those multiple use facilities that exist prior to adoption of these policies, as a minimum, shall be brought into compliance at the time of reconstruction or replacement of the flood control facility.

FEMA FLOODPLAIN/FLOODWAY SCHEMATIC



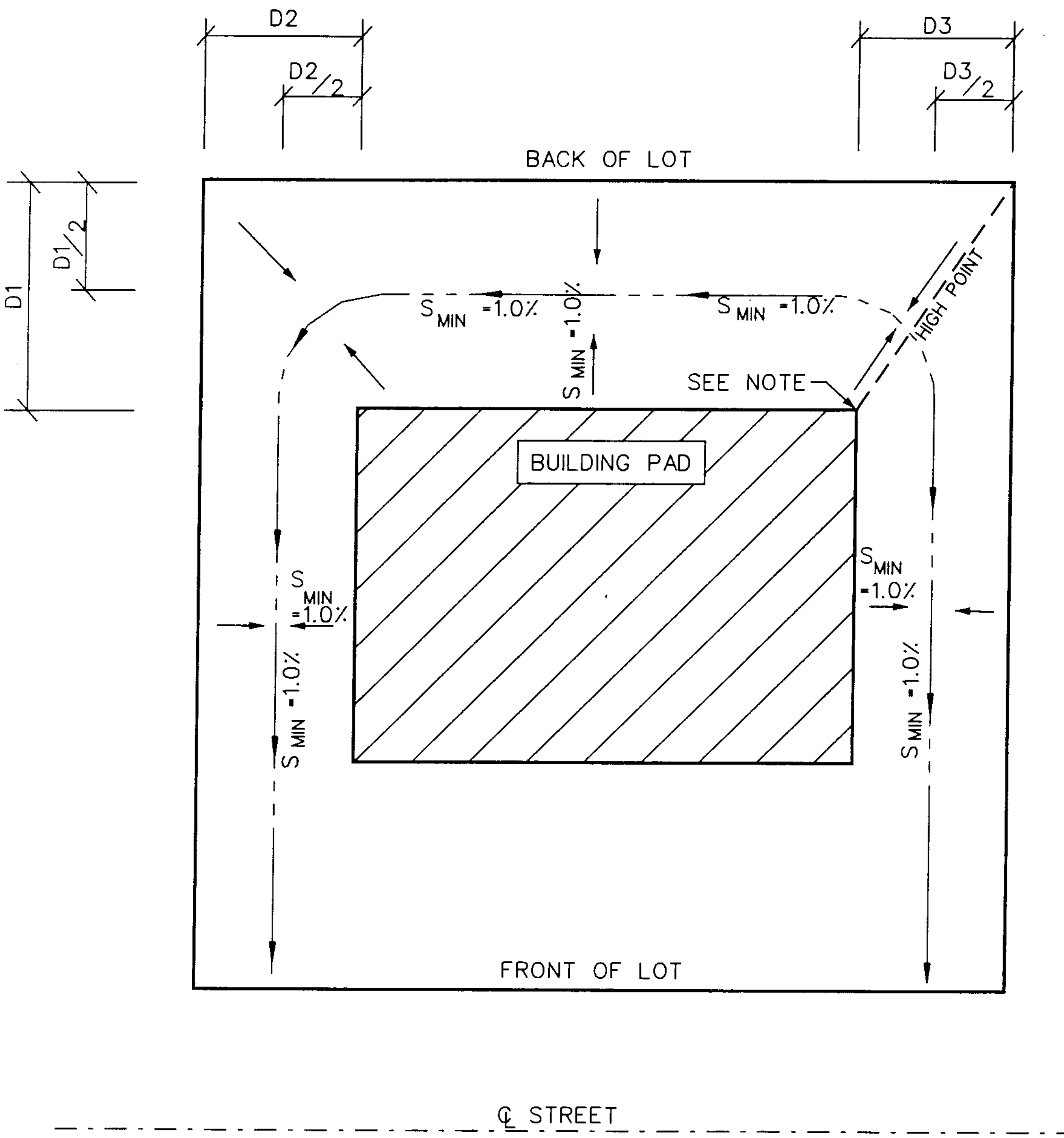
LINE AB IS THE FLOOD ELEVATION BEFORE ENCROACHMENT.
LINE CD IS THE FLOOD ELEVATION AFTER ENCROACHMENT.
*SURCHARGE IS NOT TO EXCEED 1.0 FOOT (FIA REQUIREMENT) OR LESSER AMOUNT IF SPECIFIED BY STATE.

Revision	Date

REFERENCE: FEMA FIS, Clark County and Incorporated Areas, August 1995

FIGURE 301

TYPICAL LOT DRAINAGE



NOTE: MINIMUM CLEARANCE OF 8 INCHES REQUIRED BETWEEEN BACKYARD HIGHPOINT AND FINISHED FLOOR ELEVATION AT BACK OF BUILDING PAD.

D - DISTANCE FROM THE PROPERTY LINE TO THE BUILDING PAD

S_{MIN} - MINIMUM SLOPE OF LAND

Revision	Date

REFERENCE:

FIGURE 302

CLARK COUNTY REGIONAL FLOOD CONTROL DISTRICT
HYDROLOGIC CRITERIA AND DRAINAGE DESIGN MANUAL

SECTION 400
DRAINAGE LAW

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Section 400
Drainage Law

401 INTRODUCTION

The materials contained in this chapter are not intended to be an exhaustive presentation of each area of law which is discussed. The purpose is to familiarize the design professional with these areas to enable them to better perform engineering duties and tasks. These materials should not be used in place of a consultation with an attorney and no liability is being assumed with respect to the use of these materials for such purpose.

An important lesson which has been learned in Southern Nevada is that water does not respect arbitrary jurisdictional boundaries. Water does not respect the various rights and liabilities of adjacent land owners as it flows through depressions, gulleys, and washes seeking ultimate terminus in Lake Mead. However, engineers are presented with the enormous task of attempting to control the drainage of water while at the same time maintaining the integrity of natural flow paths and existing legal relationships arising from land ownership. The goal of maintaining both natural flow paths and existing legal relationships is not easily achieved. However, this goal can be more easily achieved if the engineer is familiar with the basic legal framework against which legal relationships will be adjudicated.

This chapter discusses the historical evolution of water drainage law in Nevada. Unlike other states such as California and Colorado, there is not a great body of Nevada case law which discusses every identifiable issue with respect to water drainage law. There are many “gray” areas in Nevada law, but the engineer can avoid major legal obstacles by being more familiar with those cases which have been expressly decided by the Nevada Supreme Court. Relevant statutes will also be discussed.

402 HISTORICAL E\/OLUTION OF SURFACE WATER DRAINAGE LAW

Prior to a specific discussion of Nevada law, it is important for the engineer to be aware of the development of the historical principles and theories involved in drainage law. There are three common early doctrines which were followed in the United States: The doctrines were the common enemy doctrine, civil law rule, and the rule of reasonable use. Each theory will be briefly examined prior to an in depth analysis of Nevada law.

402.1 The Common Enemy Doctrine

The common enemy doctrine is a harsh rule which is still followed in some states. The common enemy doctrine has not been specifically recognized by the Nevada Supreme Court.

Stated in its extreme form, the common enemy doctrine provides that as an incident to their right to use their own property as they please, each landowner has an unqualified right, by operations on their own land, to fight off surface waters as they see fit without being required to take into account the consequences to other land owners, who have the duty and right to protect themselves as best they can. See 93 ALR 3d 1193.

Surface water was thus regarded as a common enemy which each property owner could fight off or control by any means such as retention, diversion, repulsion or altered conveyance. Thus, there was no cause of action even if some injury occurred to the adjoining parcel.

All jurisdictions originally following this harsh rule have either modified the rule or adopted the civil law rule or reasonable use Rule 5 Water and Water Rights, Sections 450.6.451.2 (RE Clark ED. 1972).

As previously mentioned, the Nevada Supreme Court has not specifically recognized or adopted this theory.

402.2 Civil Law Rule

Courts later recognized the rule of water drainage law which is basically diametrically opposed to the common enemy doctrine. The civil law rule recognizes a natural servitude for natural drainage between adjoining lands, so that the lower owner must accept the surface water which naturally drains onto their land, but, on the other hand, the upper owner has no right to change the natural system of drainage to increase the burden on the lower parcel. This rule caused problems with allowing development because virtually almost any development has a tendency to increase the flow either in quantity or velocity. According to the civil law rule, if the quantity or velocity of water flow were increased, the natural flow on the downstream property would be changed and would be in violation of the civil law rule. Thus, with the evolution of drainage law the courts sought to modify the law to consider the competing interests of adjoining land owners and allocate the burden of risk attendant to development.

The civil law rule analyzes drainage problems in terms of property law concepts such as servitudes and easements. It did not consider tort law analysis of what is "reasonable".

The Nevada Supreme Court specifically recognized the civil law rule as early as 1885 in the case of Boynton v. Longley, 19 Nev. 69, 6 Pac. 437

(1885). This case will be discussed in detail in the analysis of Nevada Drainage Law.

402.3 Reasonable Use Rule

The rule of reasonable use was developed as an alternative between the civil law rule and the common enemy doctrine. The courts attempted to balance the hardships created in attempting to control surface waters and relevant factors in the relationship between the competing rights/liabilities of adjoining land owners. The rule was apparently developed to provide flexibility in avoiding harsh results which often occurred in applying both the common enemy doctrine and the civil law rule to various factual situations.

Under the reasonable use rule, a property owner can legally make reasonable use of their land, even though the flow of surface waters is altered and causes some harm to others. However, liability occurs when the property owners' harmful interference with the flow of surface water is "unreasonable". A balancing test is utilized to determine whether a landowners use of their property is unreasonable. The analysis involves three basic questions: (1) was there reasonable necessity for the property owner to alter the drainage to make use of their land? (2) was the alteration done in a reasonable manner? (3) does the utility of the actor's conduct reasonably outweigh the gravity of harm to others? See, <u>Restatement Torts, 822-831,833 (1939)</u>.

The Nevada Supreme Court has recognized consideration of at least five factors in determining whether a property owner's conduct was reasonable. As one can see from this analysis, it becomes very difficult to predict how a jury would rule in relationship to any particular set of facts because the standard for determination is reasonableness, and each jury will have its own standard for determining reasonable conduct.

The reasonable use rule does not utilize property law concepts of servitude and easement. It substitutes a tort analysis of "reasonable" conduct. The positive aspect of this rule is that it accommodates development and allows for alterations of surface flow if done in a responsible manner. The negative aspect of this rule is the uncertainty created by the vague standard regarding "reasonable" conduct. One engineer's "reasonable" design for handling surface waters may be perceived by a different engineer in a court of law as "unreasonable."

403 NEVADA DRAINAGE LAW

The Nevada Supreme Court initially adopted the civil law rule of drainage in 1885. The civil law rule was later changed when the Nevada Supreme Court adopted the reasonable use rule for surface water drainage in 1980. However, it is important for the engineer to be familiar with both cases in order to understand the evolution of Nevada Drainage Law and its underlying public policy considerations.

403.1 Civil Law Rule

In 1885 the Nevada Supreme Court was presented with a novel question. Can an upper land owner drain artificially collected waters onto their neighbor's lower parcel? This question had never been presented before because most property owners usually complained of lack of water rather than an excess of water.

In Boyton v. Longley, 19 Nev. 69, 6 Pac. 437 (1885) an upper land owner used an irrigation ditch to collect water from the Truckee River and irrigate his farm. The irrigation water naturally flowed onto an adjacent parcel. The lower land owner sued to recover damages for his land and crops allegedly caused by the waste water.

The upper land owner made several arguments as follows: irrigation was necessary to cultivate his land, the lower land owed a servitude to the upper parcel to receive water which naturally flowed on to it, he had been irrigating his land for five years, and therefore, had obtained a prescriptive easement across the lower piece of land. The lower land owner argued there was not a natural right to discharge water from artificial sources.

In ruling in favor of the lower land owner, the Nevada Supreme Court noted as follows:

> "...As to the flow of water caused by the fall of rain, the melting of snow, or natural drainage of the ground, the prevailing doctrine is that when two tracts of land are adjacent and one is lower than the other, the owner of the upper tract has an easement in the lower land to the extend of the water naturally flowing from the upper land to and upon the lower tract, and that damage that may be occasioned to the lower land thereby is damnum absque injuria (injury without damage). Water seeks its level and naturally flows from a higher to a lower plane; hence the lower surface, or inferior heritage is doomed by nature to bear a servitude to the higher surface, or superior heritage, in this: that it must receive the water that naturally falls on and flows from the level. The proprietors of the lower land cannot complain of this - this expression of the law only applies to waters which flow naturally from springs, from storms of rain or snow, or the natural moisture of land. Wherever courts have had occasion to discuss this question

> they have generally declared that the servitude of the lower land cannot be augmented, or made more burdensome by the acts of industry of man."
>
> 19 Nevada at 69, 72-73.

The court observed that in order to cultivate their respective lands, both parties had to obtain irrigation water by bringing it from points remote and distant from their lands. Without the "reasonable use" of the water, the lands were comparatively worthless.

The Nevada Supreme Court held that the upper land owner, while they have the unqualified right to make reasonable use of the water for irrigation, must use, manage, and control the water as to not injure an adjacent parcel. Central to the court's holding is the concept that a land owner should not be permitted to make his land more valuable by an act which renders the land of a lower land owner less valuable. This policy consideration would later be utilized by the Nevada Supreme Court almost 95 years later when the reasonable use rule was adopted.

Thus, until the reasonable use rule was adopted in 1980, Nevada Drainage Law consisted of a property law analysis of natural easements for upper parcels to drain water over lower parcels.

403.2 Reasonable Use Rule

Approximately 95 years passed in Nevada before the Nevada Supreme Court was presented with the opportunity to change drainage law in Nevada. The court adopted the reasonable use rule in 1980 when presented with a modern factual situation which opened the door for Nevada to join the majority of jurisdictions in the western states by adopting the reasonable use rule. The case in which the reasonable use rule was adopted was controversial not only when it was decided, but remains somewhat controversial for all of the questions it does not answer.

The Nevada Supreme Court changed Nevada drainage law in County of Clark v. Powers, 96 Nev. 497, 611 P.2d 1072 (1980). Land owners had filed an action against the County and various developers because their activities allegedly had altered the drainage of surface waters in their area. The plaintiffs settled with the individual developers prior to trial, and proceeded to trial against the County and the County Flood Control District under theories of inverse condemnation, nuisance, and trespass. The trial court adopted the reasonable use rule and entered an award for the plaintiffs.

The Nevada Supreme Court found that during the 1950's and early 1960's, the plaintiffs had acquired their parcels and developed them for residential use. Prior to major development in the area, the land immediately west of the two plaintiffs' parcels was sufficiently porous to absorb and dissipate most rain waters. Heavy rains, however, would collect in the low surrounding areas and would follow the natural terrain entering the plaintiffs' properties at the approximate border between the two properties. These waters would flow, if at all, at a slow velocity and would be naturally dissipated and absorbed. Flooding was rare. The "ephemeral stream" on the plaintiffs' property paralleled a wash which ran to the south of the plaintiffs' parcel.

The court found that starting in 1967 the development of the lands west of the plaintiffs' parcel resulted in the alteration, diversion, channeling and acceleration of rain, nuisance, and flood waters on to respondents' properties. The court found the County had actively participated in the development of these lands, both by its own planning, design, engineering, and construction activities and by its adoption of the similar activities of various private developers as part of the County's Master Plan for the drainage and flood control of the area.

The facts as determined at trial established various roads and intersections had been elevated, waters were collected and diverted from a grocery store site, and channeled those waters to a drainage pipe maintained by the County. Similarly, the streets, curbs, and gutters were specifically designed to divert and channel waters onto the plaintiffs' parcels which normally would have drained to the wash. The court held that the cumulative effect of the development activities was to increase and accelerate the flow of waters through the ephemeral stream between the plaintiffs' parcels, to divert waters normally draining into the wash onto the plaintiffs' properties, and to alter and divert the natural course of the ephemeral stream. The property was subjected to temporary but frequent and inevitable flooding.

The County argued that the civil law rule should be maintained. The Nevada Supreme Court felt that the question of which law to apply to surface water drainage entailed a judgment concerning the proper allocation of costs incident in the transformation of rural or semi-rural areas into urban and suburban communities. In making its judgment the court identified three central principles from prior decisions: one, the law of water rights must be flexible, taking notice of the varying needs of various localities; two, a land owner may make reasonable use of his land as long as he does not injure his neighbor; and three, a land owner should not be permitted to make his land more valuable at the expense of the estate of a lower land owner.

The court found that the civil law rule was ill suited to the complexities of urban growth and expansion, and found the reasonable use rule to be more predictable and suited to modern development. The court held that in effecting a reasonable use of land for legitimate purpose, a land owner or user, acting in good faith, may drain surface waters and cast them on a neighbor's land if:

a) The injurious flow of water is reasonably necessary for drainage;

b) Reasonable care is taken to avoid unnecessary injury;

c) The benefit to the drained land outweighs the gravity of harm inflicted upon the flooded land;

d) The drainage is accompanied, where practicable, by the reasonable improvement and aiding of normal and natural systems of drainage in accordance with their reasonable caring capacity; and

e) Where no natural systems of drainage are available, the drainage is accomplished by the use of a reasonable, artificial system of drainage.

The reasonable use rule was adopted by the court because it felt that the economic costs incident to the expulsion of surface waters in the transformation of rural and semi-rural areas into urban and suburban communities should not be borne solely by adjoining land owners. Rather, land owners, developers, and local officials, should take into account the costs of development of the community prior to the implementation of their plans. The court found that absent such prior planning, of reasonable use rule allows for a more equitable allocation of the incidental economic costs than did the civil law rule.

The County also argued it had statutory immunity for damages which were caused by "urbanization." The Nevada Supreme Court rejected the concept of limited sovereign immunity, and held as follows:

> "we...chose to follow the view, adopted in a majority of jurisdictions, that a governmental entity's substantial involvement in the development of private lands which unreasonably injures the property of others is actionable." 96 Nev.a505.

The NRS which confer immunity from suit for discretionary acts of County employees, were not argued at the trial court level and therefore were specifically not considered on appeal. It remains an open question regarding the effect discretionary immunity might have played in this case. Similarly, the factual situation included both the acts of private developers and the County. It is therefore impossible to determine whether the court focused its decision regarding County liability on the fact that a County constructed and maintained drainage pipe was related to the plaintiffs' flood problems.

Although the Powers case changed Nevada law, it leaves many questions unanswered such as: (1) Is the governmental entity liable for mere approval of a private development; (2) What constitutes "substantial involvement" in the development of private land? (3) Is a governmental entity liable if it fails to detect design or construction deficiencies in a private design? and (4) Is a governmental entity liable for privately designed flood control improvements which are later dedicated to the entity? The Powers case is the controlling legal precedent in this State. Engineers should be aware of the balancing test set forth in the decision, as well as the underlying factual situation. The balancing test should be considered when an engineer is designing or approving alternate methods of handling water drainage. As previously mentioned, one engineer's "reasonable" drainage approach could be a juror's "unreasonable" diversion.

403.3 Surface Waters - Private Development

Engineers and developers working in the private sector are presented with similar liability exposure as governmental entities, but do not enjoy the same statutory protections. A brief discussion of each liability theory is important for the engineer to have a sense of the potential exposure he or she faces when proceeding with a design project.

403.3.1 Negligence

Negligence has been defined by Black's Law Dictionary as "the omission to do something which a reasonable man, guided by those ordinary considerations which ordinarily regulate human affairs, would do, or the doing of something which a reasonable and prudent man would not do."

Placing the negligence definition into an engineering context, the reasonable and prudent man standard becomes a reasonable professional and prudent professional standard. Thus, a professional engineer who fails to act within the standard of care of his engineering profession may be held liable for negligence. The applicable standard of care is established in court by expert testimony.

The concept of negligence is composed of the traditional elements of duty, breach of the duty, the breach resulting in the proximate cause of damage, or injury. Prosser, Torts 143 (4th Ed. 1971). Nevada courts and courts across the nation have broadly interpreted the duty element as a duty being owed to all persons who may foreseeably be affected by the work being performed.

In order for the engineer to determine if he faces potential negligence exposure, it is helpful to analyze the project and its relation to the engineering activities which are being performed. The engineer should attempt to determine what the standard of care in his profession is in relationship to the particular engineering task being performed and then make a realistic evaluation as to whether or not the services he has rendered would meet that standard. For example, using the Rational Method for a watershed in excess of 100 acres, rather than the HEC-1 computer model required by this MANUAL, may fall below the standard of care and could result in potential liability. The engineer should always strive to use the best information available and also strive to use methods which are state of the art and widely accepted by the engineering profession.

Complying with legally required procedures (i.e., contained in this MANUAL) have been held by the courts to be a minimum standard of care.

Unfortunately, an engineer cannot always be guaranteed that by merely following the computer models and design procedures contained in this MANUAL he will be insulated from negligence liability. It is possible that in a particular area of design the engineer's standard of care could require a higher standard of engineering activities. However, following the requirements of this MANUAL will go a long way toward establishing that an engineer has met the accepted standard of care.

403.3.2 Breach of Express/Implied Warranty

The liability theory can be based on either an implied warranty or an express warranty. Under this particular theory of liability an engineer does not face the same exposure as a developer who actually sells a finished product.

The courts have generally held that an implied warranty normally requires privacy of contract between the person bringing the action and the party who allegedly breached the implied warranty. An implied warranty only goes to the product and may not be imputed to one who has provided services as opposed to the product. Thus, a private engineer who has designed plans does not face the same liability exposure as a developer who has sold a completed product.

403.3.3 Fraud/Misrepresentation

Fraud is a much less commonly-pled liability theory because it is much harder to prove. The Court requires “clear and convincing" evidence that fraudulent conduct has occurred.

Fraud in the general sense is deemed to be any conduct which is calculated to deceive another person or entity and results in damage.

The essential elements required to sustain fraud action are the representations made as a statement of fact (not genuine) which was untrue and known to be untrue by the party making it, or else recklessly made; that the statement was made with intent to deceive and for the purpose of inducing the other party to act upon it, and the person did in fact rely on it and was induced to act to his detriment. AmJur.2d, Fraud & Deceit, Sections 2, 12.

An example of fraudulent conduct would be an engineer or developer telling a potential home purchaser that the home was not located in a floodplain when the engineer/developer knew for a fact that the statement was false. If the purchaser relies on that representation and purchases a home in the subdivision, then a potential case of fraud exists.

403.3.4 Trespass

Trespass is an injury to possession. It is an intrusion which invades a person's protected interested in exclusive possession. A trespass action requires active conduct on the part of the wrongdoer. Liability can be imposed for intentional, negligent or ultrahazardous activity. The only "intent" required is that the act constituting the trespass is voluntarily done. An act may constitute a trespass despite the fact that its consequences were unintended. 75 AmJur.2d, Trespass, Section 8.

In general, one is subject to liability for trespass to real property if one intentionally enters land in the possession of another or causes a thing or force to do so. A landowner who sets in motion a force which, in the usual course of events, will damage the property of another is guilty of trespass on such property. **Burt v. Beautiful Savior Lutheran Church**, 809 P.2d 1064 (Colo.Ap.1990).

Nevada has defined trespass as an injury to an estate, or use thereof, by one who is a stranger to the title of the injured property. **Price v. Ward,** 25Nev.203,58Pac.849 (1899).

An example of this liability theory would be damage to real property caused by waters escaping from a drainage channel or damaging a subdivision as a result of an improperly designed drainage system. The damage occurs when the water flows on the person's property and in turn damages the real property, personal property and possessory interest of the landowner. Such damage easily occurs once water begins to flow onto a property and into the front door of someone's home. The landowner need not prove that the engineer or developer intentionally flooded the property, but merely that the act of designing and constructing the flood control improvement were done voluntarily. As previously noted, the liability for trespass can be based on negligent conduct.

Flooding of a person's property because of improper construction of highway embankments constitutes trespass. **Viestenz v. Arthur** TP, 54 N.W.2d.572 (ND 1952). Where the defendant's affirmative act results in the flooding of the plaintiff's land and the destruction of crops, the defendant has constituted Trespass. **Western Union Tel. Co. v. Bush**, 89 S.W. 2d 723 (Ark. 1935). However, floods resulting solely from a severe storm do not constitute trespass. **Hughes v. King's County**, 714 P.2d 316, (Wash. Ap. 1986).

Pursuant to the trespass liability theory, damages generally constitute the difference in value of the land both before and after the act. Damages can also include the loss of use of the land, discomfort and annoyance to the property owner, cost of repair, and lost crops.

A warning sign for dual use facilities is shown in **Figure 401** and shall be posted not more than 200 feet on each side of the facility, upon or near the boundary. Warning signs shall be mounted on a metal fence post, and shall be painted with fluorescent orange paint. **Figure 402** depicts a warning sign for flood channels.

403.3.5 Nuisance

The "nuisance" liability theory applies to that class of wrongs that is covered by the unreasonable, unwarrantable, or unlawful use by a person of his property, or from his improper, indecent or unlawful conduct, which operates as an obstruction or injury to the right of another or to the public, and producing such material annoyance, inconvenience, discomfort or hurt that the law will presume consequential damage. **Bliss v. Grayson**, 24 Nev.422, 56 Pac.231 (1899).

The term "nuisance" is incapable of an exact and exhaustive definition which will fit all cases because the factual situations are seldom alike. Nevertheless, "nuisance" has been defined as a distinct civil wrong, and is used to describe the wrongful invasion of a legal right or interest. "Nuisance" includes everything that endangers life, health, or obstructs the reasonable and comfortable use of property. 58 AmJur.2d, Nuisance, Section 1.

Nuisance and trespass are analogous in some respects. However, there is a distinction between them, the difference being that trespass is an invasion of the person's interest in the exclusive possession of his land (as by entry on it) while a nuisance is an interference with the use and enjoyment of the land, and doesn't require interference with the possession. The requisites that an interference be substantial and unreasonable, in order to constitute a nuisance, have been said to distinguish an action for nuisance from that of trespass. In this regard, an action for trespass can be maintained without a showing of damage because it is the unauthorized entry upon the land that creates the trespass and the presumed damage.

A claim of nuisance is more than a claim of negligence. Negligent acts do not in themselves constitute a nuisance; rather, negligence is merely one type of conduct upon which liability for nuisance may be based.

This liability theory primarily involves the annoyance and inconvenience which people experience once their property has been flooded. The flood clean-up process and associated odors, filth, and insect infestation would fall within this theory. In many ways, this theory closely tracks claims for emotional distress and can provide dramatic testimony for a jury. Even something as minor as increased flow in an irrigation ditch has been deemed a nuisance in Nevada. **Thomas v. Blaisdell**, 25 Nev.223, 58 Pac.903 (1899).

403.3.6 Strict Liability

Nevada has recognized that an end user of a "product" has established a cause of action in strict liability against a manufacturer or distributor when "his injury is caused by a defect in the product, and the user proves that such defect existed when the product left the hands of the defendant." Shoshone Coca-Cola Bottling v. Dolinski, 82 Nev. 439, 443, 420, P.2d 855, 858 (1966).
California has applied the strict liability theory to the sale of homes and defective lots. The Nevada Supreme Court noted in Elley v. Steven, 104 Nev.Adv.Op. 62, N.2 (1988) that courts are divided about whether a home is a product under strict liability theory. In that case of Nevada Supreme Court was presented with this issue but was able to decide the case without ruling on the applicability of the strict liability theory. As the law currently stands in Nevada, the strict liability theory does not apply to homes. However, this theory could be extended to a situation where a "product" is sold by someone in the regular course of its business.

403.3.7 Punitive Damages

The above liability theories can support both an award of compensatory damages and punitive damages. Compensatory damages are to compensate a person for specific damages such as property repair or replacement costs. However, the private developer faces a possible award of punitive damages which can be unrelated to the actual damages suffered by the land owner.

> NRS 42.010 provides as follows:
>
> "In an action for the breach of an obligation not arising from contact, where the defendant: (1) has been guilty of oppression, fraud or malice, expressly implied; or (2) caused an injury by the operation of a motor vehicle in violation of NRS 484.379 or 484.3795 after willfully consuming or using alcohol or another substance, knowing that he would thereafter operate the motor vehicle.

> The plaintiff, in addition to actual damages, may recover damages for the sake of example and by way of punishing the defendant."

The concept of punitive damages rests upon a presumed public policy to punish a wrongdoer for his act and to deter others from acting in similar fashion. The punitive damage allowance should be in an amount that will promote the public interest without financially annihilating the defendant. Nevada Cement Company v. Lemler, 89 Nev. 447, 514 P.2d 1180 (1973).

Since the purpose of punitive damages is to punish and deter culpable conduct, the award lies in the discretion of the court or jury and need not bear a fixed relationship to the compensatory damages awarded. Randano v. Turk, 86 Nev. 123, 466 P.2d 218 (1970).

The "malice" contemplated in NRS 42.010 is malice in fact and which the malice is established. Malice in fact sufficient to support an award of damages may be established by a showing that the wrongful conduct was willful, intentional and done in reckless disregard of its possible results. Nevada Credit Rating Bureau Inc. v. Williams, 88 Nev. 601, 503 P.2d 9 (1972).

In Village Development Company v. Filice, 90 Nev. 305 P.2d 83 (1974), the Nevada Supreme Court was presented with a case involving a claim for damages arising from the destruction of a home constructed in an undisclosed floodplain and the subsequent claim for punitive damages. The lot purchaser brought an action to recover damages from the developer of a lot which was situated in an undisclosed floodplain of a mountain stream. The District Court awarded compensatory and punitive damages and the developer appealed. The Nevada Supreme Court found the developer was aware that a stream which crossed the plaintiff's lot usually was quite narrow but varied radically under various storm conditions of given return frequencies. Despite knowledge of the developer's officers regarding the extent of the floodplain, the developer did not impose any building restriction other than requiring that building plans be submitted to an architectural control committee. Knowing of the flood hazard, the developer assumed the plaintiff would build on the highest possible site on the lot, but never advised the lot purchasers of its thoughts regarding a proper building site. In short, the court found that the corporation's highest management personnel failed to warn of the danger although they well knew the plaintiffs were planning to build in the floodplain. Plans were submitted to the architectural control committee and approved without warning.

The court held that there was ample evidence to support a jury instruction regarding negligence and that the resulting award under that theory was proper. After carefully reviewing the record the court found that although there was ample evidence of negligence and unconscionable irresponsibility, there was insufficient evidence to support a finding of "oppression, fraud or malice express or implied." The court noted it had previously sustained punitive damage awards when the evidence showed the wrong was willful. Here, the evidence was insufficient to meet the requirement that more must be shown than malice in law, and that there should be substantial evidence of malice in fact.

The above case indicated how the private developer can face punitive damage exposure. Although in the case above the developer escaped punitive damage exposure, it could easily have faced punitive exposure if representations had been made to the purchaser such as the property was not located in the floodplain, or that flooding was not likely in that area. If an area is located in a floodplain that fact should be fully disclosed to the purchaser and proper engineering procedures consistent with the standard of care should be followed.

403.4 Surface Waters - Governmental Entity Liability

The liability of a governmental entity with respect to surface waters is treated differently in some respects than the liability of a private developer even though the same liability theories can be asserted. The State legislature has conferred various statutory defenses, immunities, and damage limitation in view of the burden regarding land development which has been placed upon the governmental entities. Governmental entity tort liability is controlled by Chapter 41 of the NRS which was adopted in 1965.

403.4.1 Sovereign Immunity

The principle of sovereign immunity can be traced back to ancient times in England when a person could not sue the King. This concept has carried through the common law and has appeared in statutory provisions in most states. NRS 41.031 contains a waiver of sovereign immunity which is expressly limited by several other statutes containing specific defenses. The purpose of the limited waiver of sovereign immunity is to compensate the victims of governmental negligence in circumstances like those in which victims of private negligence would be compensated. Harrigan v. City of Reno, 86 Nev. 678, 475 P.2d 94 (1970).

The legislative intent in enacting NRS 41.031 was to waive the immunity of governmental units and agencies from liability for injuries caused by their negligent conduct, thus putting them on equal footing with private persons committing torts. Jimenez v. State, 98 Nev. 204, 644 P.2d 1023 (1982).

In close cases where the issue of whether the allegations of conduct fall within the parameters of a waiver of sovereign immunity, courts must favor a waiver of immunity; only when it is concluded that a discretionary act alone is involved will the court find immunity. Haablom v. State Director of Motor Vehicles. 93 Nev. 599. 571 P.2d 1172 (1977).

NRS 41.031 initially provided for a special claims procedure when a person wanted to sue the State. However, the Nevada Supreme Court eliminated this requirement in 1973.

403.4.2 NRS 41.032 - Discretionary Immunity

NRS 41.032 provides that no action may be brought under the limited waiver of immunity statute or against an officer or employee of the State or any of its agencies or political subdivisions which is based upon the following:

a. An act or omission of an officer or employee, exercising due care, in the execution of a statute or regulation, whether or not such statute or regulations is valid, if the statute or regulation has not been declared invalid by a court of competent jurisdiction; or

b. Based upon the exercise of performance or the failure to exercise or perform a discretionary function or duty on the part of the State or any of its agencies or political subdivisions or any officer or employee of any of these, whether or not the discretion involved is abused.

The discretionary function immunity initially was a very strong defense for governmental agencies. However, over the years various interpretations of the statute by the Nevada Supreme Court have eroded its effectiveness.

The Nevada Supreme Court has analyzed discretionary immunity in terms of the type of functions the governmental entity is performing at the time. The governmental (discretionary) function is the initial decision to act. A discretionary function can be categorized as a decision to build a freeway, flood control channel, or parking structure. Once the discretionary decision to act has been made, then the governmental entity shifts into the operational function which usually involves construction and design. The governmental entity is obligated to use due care when acting in the operational function area.

The discretionary immunity cases can generally be divided into the following areas: road/street, police protection, and miscellaneous. One case from each area will briefly be discussed to give the engineer a sense of the analysis which is engaged in by the Supreme Court.

In the case of <u>State v. Webster,</u> 88 Nev. 690, 504 P.2d 1316 (1972) horses wandered onto a frontage road and then onto a newly constructed controlled access freeway near Carson City. An accident later resulted when a car struck the horses. The plaintiffs sued the State on the theory that the State was negligent for not providing a fence to keep animals off the freeway. The Nevada Supreme Court held that the governmental (discretionary) function was the decision to build a controlled access freeway, rather than continuing an old two lane highway. Once the discretionary decision regarding construction was made, the State was obligated to use due care to make the freeway meet standards of reasonable safety. The court held the State was negligent for failing to install a cattle guard.

In Parker v. Mineral County, 102 Nev.Ad.Op. 131 (1986) a person cutting firewood saw another person lying on the side of a rural road who apparently needed help. After the person on the ground has refused assistance, the firewood cutter reported the incident to a Sheriff's deputy who said they would take care of the situation. No one responded to the report and the person by the side of the road later died of exposure. The police department was sued for failure to respond to the call. The Nevada Supreme Court, in upholding a ruling in favor of Mineral County Sheriff's Department, held that personal deliberation, decision and judgment are requirement of a discretionary act. In deciding not to respond to the call the County officials exercised their personal judgment as to how their limited resources should be utilized to best promote the public good. Such a decision could not be second guessed by the court.
In Esmeralda County v. Grogan, 94 Nev. 923 (1978) the Nevada Supreme Court held that the decision to grant, revoke, or withhold a liquor license is a discretionary act.

403.4.3 NRS 41.033 - Failure to Inspect

NRS 41.033 provides that an action may not be brought against the State under the waiver of sovereign immunity or against an officer or an employee of the State based upon the following:

a. Failure to inspect any building, structure, or vehicle, or to inspect the construction of any street, public highway or other public work to determine any hazards, deficiencies or other matters, whether or not there is a duty to inspect; and

b. Failure to discover such hazard, deficiency or other matter, whether or not an inspection is made.

An initial reading of this statute would seem to confirm broad protection for the governmental entity. However, subsequent interpretations of this statute by the Nevada Supreme Court eroded its effectiveness.

The protection provided by this statute can only be obtained if the government entity does not have actual notice of a hazard or dangerous condition. For instance, in Crucil v. Carson City, 95 Nev. 583, 600 P.2d 216 (1979) it was held that where the City allegedly had knowledge of a downed stop sign in an intersection and failed to act reasonable after discovering it, that NRS 41.033 did not provide immunity against such suit.

The State's protection under NRS 41.033 can also be altered by contract. In 1975 the City of North Las Vegas was sued when a person was electrocuted while working on a billboard and touched a high voltage line. Approximately 20 years before the accident the City had signed a franchise agreement with Nevada Power in which the City agreed to inspect the power lines in return for a certain percentage of the gross revenues attributable to the citizens of North Las Vegas. The court held the agreement imposed a contractual duty to inspect the power lines which superseded any protection provided by NRS 41.033.

In Butler v. Bogdanovich, 101 Nev. 499 (1985) a person built a home that was inspected and approved by the County. Several years later the plaintiffs purchased the home and found approximately 25 substantial building code violations and sued the County. The Nevada Supreme Court held that if the County had knowledge of the defects, the County owed a duty to the plaintiffs to take action as a result of the discovery of the deficiencies. The court held sovereign immunity would not bar actions based upon a public entity's failure to act reasonably after learning of a hazard. This case highlights the effect of actual notice eliminating certain sovereign immunity defenses.

403.4.4 Limitation of Tort Damage Awards

NRS 41.035 generally provides two important limitations on the types of damage claims which can be awarded against a governmental entity.

The first limitation on damages awards limits a person's recovery in tort against a governmental entity to a maximum of $50,000. The stated damage limitation applies to an individual for each cause of action which may be asserted against the State, regardless of how many actions he or she may have even if more than one action arose from a single event. State v. Webster, 88 Nev. 690, 504 P.2d 1316 (1972).

The second important damage limitation prevents an award of punitive damages against the State. This is a very important distinction between governmental and private liability. A private developer may be held liable in punitive damages which can range far in excess of any compensatory damages which are awarded to a plaintiff, while a governmental entity is protected from such damages. However, government entities can be sued in inverse condemnation while a private developer is protected from such an action.

403.4.5 Inverse Condemnation - Eminent Domain

The subject of Eminent Domain is extremely complex. However, a brief overview of this area is necessary for the engineer.

Article 1, Section 8 of the Nevada State Constitution provides in pertinent part that private property shall not be taken for public use without just compensation having been first made or secured, except in cases of war, riot, fire or great public peril, in which case compensation will be made later. Private property cannot be taken for a private use and can only be taken for a public use by a specific act of the governmental entity.

Eminent domain and inverse condemnation are basically the same concept but from a different perspective. If a governmental entity needs to obtain land for the construction of a flood control project, then the land is obtained by filing an eminent domain proceeding in which the land is condemned and the land owner is paid “just compensation" for the land. If a land owner claims that the property has been taken for a public use without just compensation being first made, then an inverse condemnation action is filed by the land owner seeking compensation from the governmental entity for the land.

Chapter 37 of the NRS governs eminent domain actions. Specifically, NRS 37.010(3) and (5) provides that the right of eminent domain may be exercised for the public purpose of "draining any County" or "for draining and reclaiming lands." Thus, obtaining property for flood control purposes has been specifically recognized by State statutes. Chapter 37 contains the statutes governing the acquisition and valuation process.

Chapters 340 and 342 of the NRS also contain additional information regarding eminent domain procedures and acquisition of real property. Of particular interest is NRS 342.280 which provides that no public body shall intentionally make it necessary for an owner to institute legal proceedings to prove the fact of the taking of his real property.

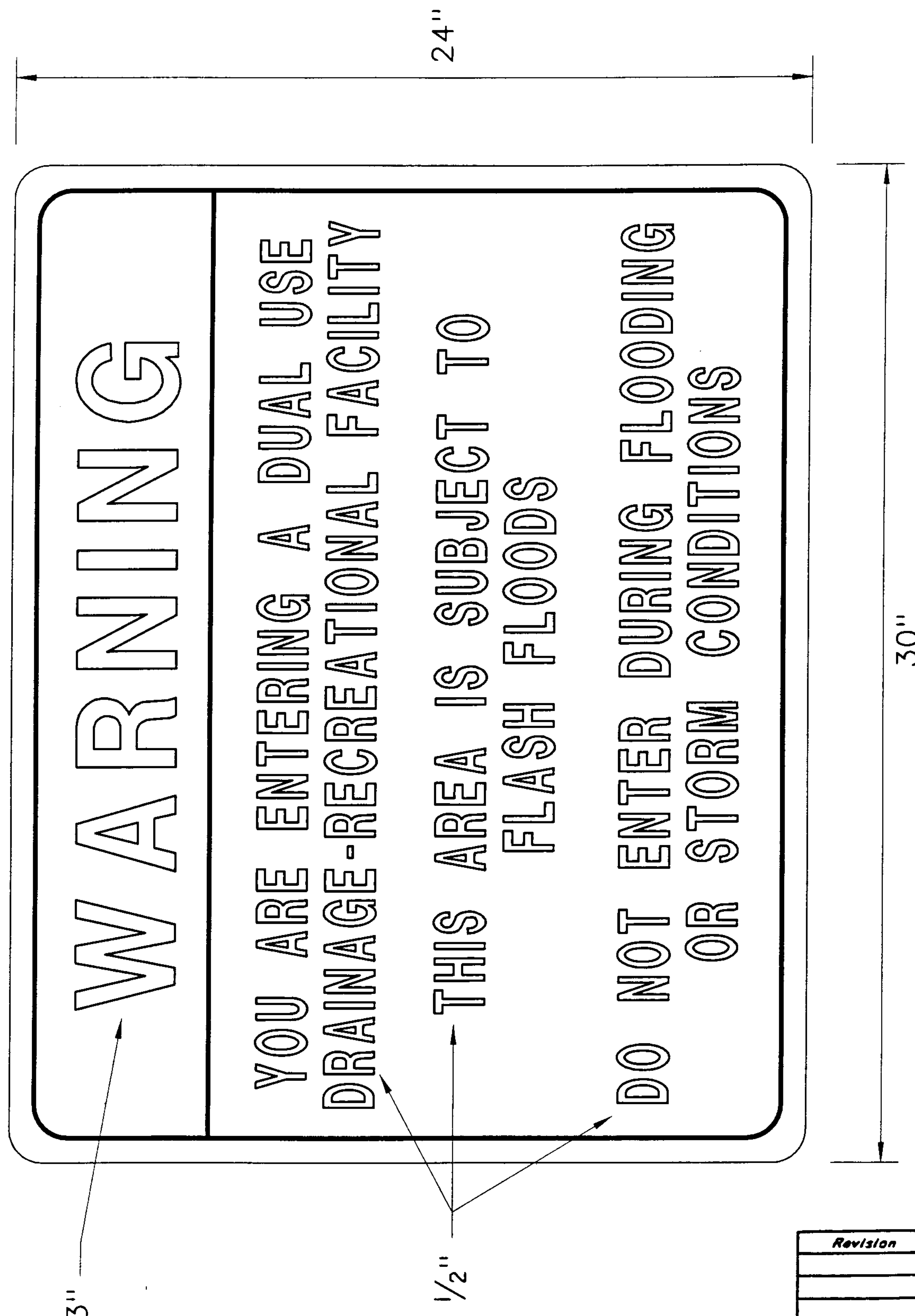
The courts have generally upheld the concept that drainage improvements are public purposes. A public drainage ditch has been held to be for a public purpose under eminent domain, and therefore required compensation for private property taken or damaged in the construction thereof. Eminent domain, 26 AmJur.2d Section 44. The courts quite generally have come to consider drainage district acts with favor as being for public purpose, whether exercised for the benefit of public health or for the reclamation or utilization of lands for agricultural purposes.

The Nevada Supreme Court specifically recognized the inverse condemnation theory in County of Clark v. Powers, supra. In that case the plaintiffs' properties were repeatedly flooded as a result of development activities of upstream developers. The court found the property no longer had a practical use other than as a flood control channel. The court noted in a footnote on page 501 of the decision as follows:

> "It has long been established that a taking occurs where real estate is actively invaded by superinduced additions of water...so as to effectively destroy or impair its usefulness" Pumpelly v. Green Bay Company, 80 U.S. (13 Wall.) 166, 181, (1871), and the result is no different when property is subjected to intermittent, but inevitable flooding which causes substantial injury, United States v. Cress, 243 U.S. 316, 328 (1917).

Thus, private property which is subject to intermittent but inevitable flooding can be "taken" as a result of governmental flood control projects. However, each of the cases is highly dependent upon its factual situation. Inverse condemnation liability extends to "just compensation" for the highest and best use of the property. The previously mentioned $50,000 damage limitation applies only to tort actions and does not apply to inverse condemnation actions. Additionally, the sovereign immunity defenses such as discretionary immunity and failure to inspect immunity are not available to the governmental entity because the right to just compensation for private property taken for a public use cannot be abridged or impaired by statute. Alper v. Clark County, 93 Nev. 569, 571 P.2d 810 (1977) cert. denied, 436 U.S. 905, 98 S.Ct. 2235, 56 L.Ed. 2d 402 (1978).

WARNING SIGN - DUAL USE FACILITY

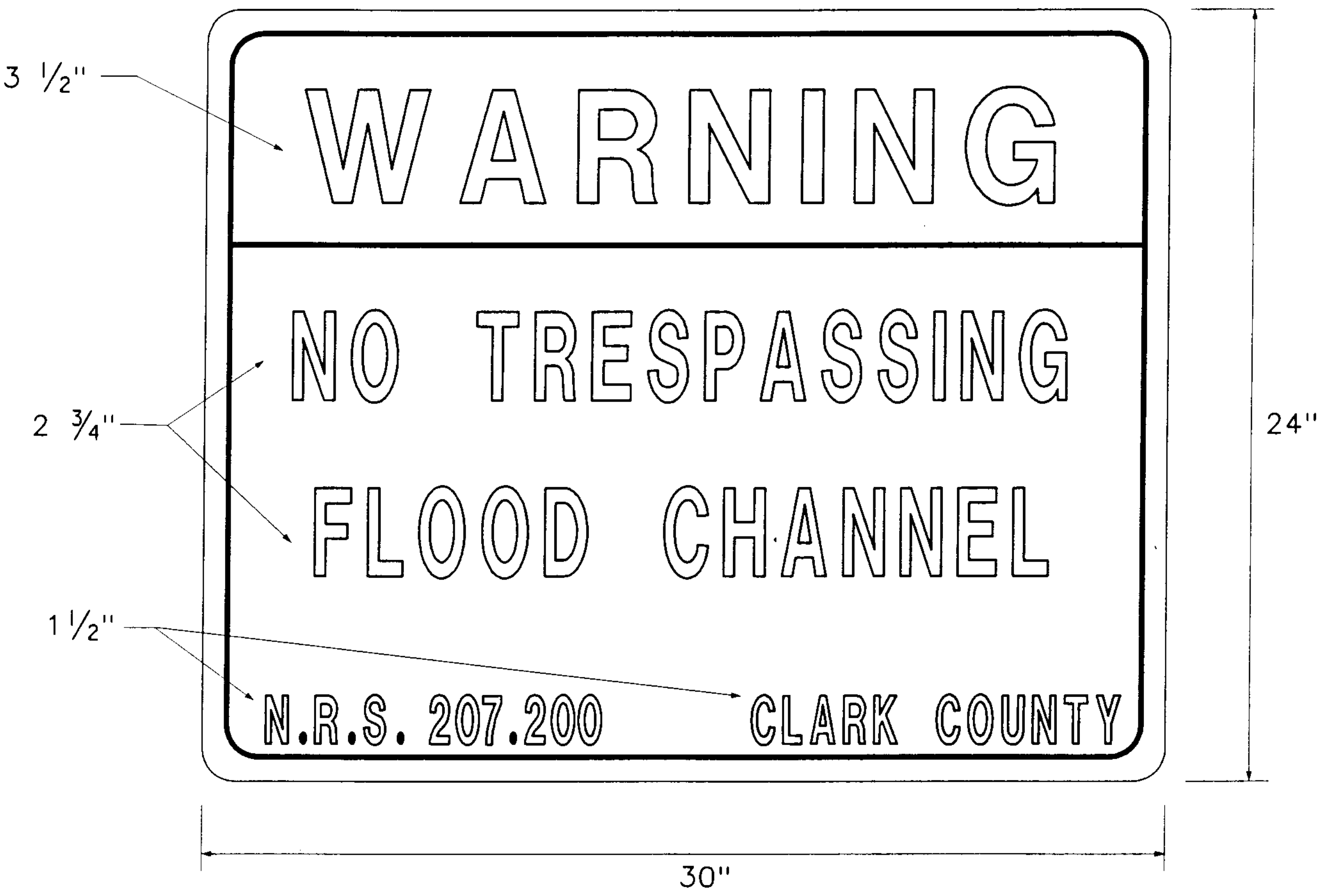


Revision	Date

REFERENCE:

FIGURE 401

WARNING SIGN - FLOOD CHANNEL



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REFERENCE:

CLARK COUNTY REGIONAL FLOOD CONTROL DISTRICT
HYDROLOGIC CRITERIA AND DRAINAGE DESIGN MANUAL

SECTION 500
RAINFALL

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Section 500
Rainfall

501 INTRODUCTION

Presented in this section are the design rainfall data for the minor and major storm events as designated in Section 304.2 of this MANUAL. This data is used to determine storm runoff in conjunction with the runoff models designated in Section 304.3. All hydrologic analyses within the jurisdiction of this MANUAL shall utilize the rainfall data presented herein for calculating storm runoff.

The rainfall data published by the National Oceanic and Atmospheric Administration (NOAA) in the NOAA Atlas 2, "Precipitation - Frequency Atlas of the Western United States, Volume Vll - Nevada" (NOAA, 1973) and their subsequent modification by the United States Army Corps of Engineers (USACE), Los Angeles District (1988) shall be used to develop point rainfall values for Clark County. The depth-area ratios developed by the USACE, Los Angeles District (1988) based on NOAA Technical Memorandum NWS HYDRO40 (NOAA, 1984) and area data are used to reduce point rainfall to area rainfall. The distribution of design rainfall is based on studies conducted by the USACE, Los Angeles District (1988).

Calculation methods and procedures are presented herein to compute rainfall depths and intensities for return frequencies of 2-, 5-, 10-, 25-, 50-, and 100-year and durations of 5-, 10-, 15-, and 30-minutes, and 1-, 2-, 3-, 6-, and 24-hours. Some of these values are not used to determine runoff for the analysis required by this MANUAL, but are included for informational purposes.

In cases where probable maximum precipitation analyses are required, methodology outlined in a publication by NOAA and the USACE entitled “Hydrometeorological Report No. 49, Probable Maximum Precipitation Estimates, Colorado River and Great Basin Drainages” (NOAA and USACE, 1977) shall be used.

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

502 RAINFALL DEPTH-DURATION-FREQUENCY RELATIONS

502.1 Rainfall Depth - Duration - Frequency Maps

The NOAA Atlas 2 Rainfall Depth - Duration - Frequency Maps are reproduced for the Clark County area at the end of this section. Maps are presented for the 6- and 24-hour durations for the 2-, 5-, 10-, 25-, 50-, and

100-year recurrence frequencies as shown in **Figures 501** through **512**. The data obtained from these maps shall be modified according to procedure stated in Section 502.3 (except for SCS TR-55 as described in 502.3).

502.2 Rainfall Depths for Durations From 1- to 6-Hours

The refined rainfall values for the 6- and the 24-hour durations for 2- and 100-year recurrence intervals are utilized in calculation of rainfall values for 1-hour duration and 2- and 100-year recurrence intervals. The following equations shall be used to derive the 2-year, 1-hour and 100-year, 1-hour rainfall:

$$Y_2 = -0.011 + 0.942 \left[(x_1)(x_1 / x_2) \right] \quad (501)$$

$$Y_{100} = 0.494 + 0.755 \left[(x_3)(x_3 / x_4) \right] \quad (502)$$

Where:

Y_2 = 2-yr, 1-hr estimated value (in)
Y_{100} = 100-yr, 1-hr estimated value (in)
x_1 = 2-yr, 6-hr value from **Standard Form 3** (in)
x_2 = 2-yr, 24-hr value from **Standard Form 3** (in)
x_3 = 100-yr, 6-hr value from **Standard Form 3** (in)
x_4 = 100-yr, 24-hr value from **Standard Form 3** (in)

The 2- and 100-year, 1-hour rainfall (Y_2 and Y_{100}) values are then plotted on **Standard Form 3** and a straight line connecting these points is drawn. The 5-, 10-, 25-, and 50-year, 1-hour rainfall values are then read from the graph.

The 2- and 3-hour duration rainfall for the various recurrence intervals may then be calculated using the following equations:

$$(2\text{-hr}) = 0.341\,(6\text{-hr}) + 0.659\,(1\text{-hr}) \quad (503)$$

$$(3\text{-hr}) = 0.569\,(6\text{-hr}) + 0.431\,(1\text{-hr}) \quad (504)$$

Where:

2-hr = 2-hr 'x'-yr estimated value (in)
3-hr = 3-hr 'x'-yr estimated value (in)
1-hr = 1-hr 'x'-yr previously determined (in)
6-hr = 6-hr 'x'-yr previously determined (in)

These point rainfall values shall be modified as stated in the following Section 502.3.

502.3 Adjustments to NOAA Atlas 2

Recent analysis of rainfall data in the Clark County area (WRC, 1989 and USACE, 1988) indicates that the NOAA Atlas 2 values do not necessarily reflect the trend of observed and recorded rainfall values which have occurred following publication of the Atlas in 1973. Therefore, the rainfall values in Section 502.2 are to be adjusted to reflect the current trend of rainfall values based on the latest available information for the Clark County area. This adjustment consists of increasing the rainfall depths for durations of 6-hours or less by multiplying the values previously obtained by the appropriate factors presented in **Table 501.**

The said adjustment shall not be used when developing the design rainfall for use with SCS TR-55. The 24-hour design rainfall for TR-55 shall be used directly as developed in Section 502.1.

503 DEPTH-AREA REDUCTION FACTORS

The NOAA Atlas 2 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. Standard precipitation analysis methods require adjusting point precipitation depths downward in order to estimate the average depth of rainfall over the entire storm area. This is normally performed using depth-area reduction curves relating to a point precipitation reduction factor to storm area and duration.

In previous hydrologic studies in Southern Nevada, three methods have been used for adjusting point-precipitation depths to areally-averaged depths. All early studies used the depth-area reduction curves presented in the NOAA Atlas 2 (NOAA, 1973). These curves were developed through investigations of storms throughout the Western United States. In fact, the NOAA Atlas 2 for each state in the West contains the same family of depth-area reduction curves. Most of the recent studies have adopted depth-area reduction factors from a relatively new publication known as "Hydro 40" (NOAA, 1984), which developed factors applicable specifically to Arizona and New Mexico.

The USACE, Los Angeles District (1988) used slightly different depth-area reduction factors than those presented in "Hydro 40" for areas greater than 30 square miles. These factors were based on analysis of thunderstorms in the greater Las Vegas area. For areas up to 30 square miles the depth-area reduction factors are almost the same as those in "Hydro 40".

The 6-hour USACE, Los Angeles District (1988) depth-area reduction factors are to be used for all rainfall analysis in the Clark County area. The USACE, Los Angeles District depth-area reduction curve is shown in **Figure 514**. The depth-area reduction factors for the 6-hour storm are also tabulated in **Table 502**.

For areas greater than 200 square miles, the ability of the thunderstorm generating mechanisms (i.e., available moisture, strong convective currents, etc.) to sustain a thunderstorm much greater than 200 square miles in diameter is greatly reduced. Therefore, only a portion of an entire drainage basin could be subject to precipitation from the thunderstorm event. Analysis of this effect on runoff peaks and volumes is complicated by the necessity to determine the "storm centering" which produces the greatest peak flow and/or volume at the selected design point. In order to obtain a consistent method of analysis for these areas, the designer shall consult the local entity (and/or the CCRFCD if suggested by the local entity) to determine the appropriate method of analysis and design rainfall area reduction factors for the specific location and basin under consideration.

504 DESIGN STORMS

504.1 General

The design storm within the jurisdiction of the MANUAL shall be a 6-hour duration storm. The 6-hour duration storm is to be used for all HEC-1 runoff modeling in the Clark County area. The exception to the 6-hour design storm duration is when the SCS TR-55 method is used to compute runoff values. For SCS TR-55, a Type II rainfall distribution shall be used in conjunction with the 24-hour rainfall depth as described in TR-55.

504.2 6-Hour Design Storm Distribution

Three different 6-hour storm distributions are to be used as design storms in the Clark County area. The three design storm distributions, labeled SDN 3, SDN 4, and SDN 5, are graphically presented in **Figure 515** and tabularized in **Table 503.** For drainage areas less than 8 square miles in size, use SDN 3. For drainage areas greater than or equal to 8 and less than 12 square miles in size, use SDN 4. For drainage areas greater than or equal to 12 square miles in size, use SDN 5.

505 TIME-INTENSITY-FREQUENCY CURVES FOR RATIONAL METHOD

505.1 General

Procedures stated in Section 502 to obtain and modify the NOAA Atlas 2 rainfall depths must first be done before proceeding with development of time-intensity-frequency curves to be used with Rational Method.

505.2 Time-Intensity-Frequency Curves

To develop time-intensity-frequency curves for the Rational Method of runoff analysis, take the 1-hour adjusted point depth(s) obtained from Section 502 and multiply by the factors in **Table 504.** These point values are then converted to intensities. An example showing the development of time-intensity-frequency curves is given in Section 507.

506 RAINFALL DATA FOR McCARRAN AIRPORT RAINFALL AREA

506.1 General

This section presents the point rainfall data to be used for the McCarran Airport Rainfall Area. The data presented is applicable to those studies that have their contributing drainage basin within the area presented in **Figure 513.**

506.2 Rainfall Depth-Duration-Frequency

Presented in **Table 505** and **Figure 516** are the rainfall depth-duration-frequency values to be used in the McCarran Airport Rainfall Area as designated in **Figure 513**.

506.3 Time-Intensity-Frequency Data

Presented in **Table 506** and **Figure 517** are the time-intensity-frequency values to be used in the McCarran Airport Rainfall Area as designated in **Figure 513.**

507 EXAMPLE APPLICATIONS

507.1 Introduction

The following examples are a first in a series of example applications pertaining to the use of this MANUAL. The series is set up to lead the reader through the MANUAL'S design/evaluation procedures by building on two different hypothetical design basins within the CCRFCD.

The two example basins are introduced in **Figures 518** and **519** and list general basin parameters. The first design basin, shown in **Figure 518**, is located within Las Vegas Valley and was selected for non-urban, large basin applications. The second design basin, shown in **Figure 519**, is also located within Las Vegas Valley and was selected for urban, small basin applications utilizing the McCarran Airport Rainfall Area data. Basin modifications are presented as the examples progress in each section to emphasize the application modeled.

Even though the basins were selected to represent actual areas within the CCRFCD, minor changes have been made in basin parameters to enhance the use of this MANUAL. Therefore, results obtained from the hypothetical examples should not be construed as representative values for the actual basin locations.

507.2 Example: 6-Hour Design Storm Distribution

Problem: Utilizing the Basin in **Figure 518**, determine the 100-year, 6-hour design storm distribution.

Solution:

Step 1: Determine the NOAA 100-year, 6-hour point rainfall value from **Figure 506**:

For subbasin 1: P = 2.1 in (Area = 4.73 sq mi)

For subbasin 2: P = 2.0 in (Area = 6.14 sq mi)

Step 2: Determine the weighted point rainfall:

$$= \frac{(2.1)\,(4.73) + (2.0)\,(6.14)}{10.87} = 2.04 \text{ in}$$

(One point rainfall value is used for both basins to simplify the example calculation.)

Step 3: Determine the adjusted point rainfall value:

From **Table 501**: NOAA Adjustment Factor = 1.43
From **Table 502**: Depth-Area Reduction Factor = 0.86
Adjusted Rainfall = 2.04 x 1.43 x 0.86 = 2.51 in

Step 4: Compute the 100-year, 6-hour design storm distribution:

For a basin area of 10.87 sq mi use SDN = 4
Multiply the storm distribution percentages (**Table 503** for SDN = 4) by the adjusted rainfall depth of 2.51 in as follows:

Final 100-Year, 6-Hour Design Storm Distribution

Storm Time (min)	Percent of Total Storm Depth (SDN 4 from **Table 503**)	Storm Depth (in)
0	0.00	0.00
5	2.00	0.05
10	5.80	0.15
15	7.50	0.19
20	9.90	0.25
25	12.60	0.32
30	13.70	0.34
35	14.50	0.36
40	14.90	0.37
45	15.10	0.38
50	15.50	0.39
55	15.60	0.39
60	15.90	0.40
.	.	.
.	.	.
.	.	.
330	99.20	2.49
335	99.30	2.49
340	99.40	2.49
345	99.70	2.50
350	99.80	2.50
355	99.90	2.51
360	100.00	2.51

Application: For HEC-1, the total adjusted rainfall depth of 2.51 inches is input on the PB Card (Adjusted Precipitation). The cumulative storm depths as computed above are input on the PC Card (Storm Distribution Number 4).

507.3 Example: Time-Intensity-Frequency Curves

Problem: Derive the 10-year and 100-year time-intensity-frequency curves for a 10 acre subbasin within the basin presented in **Figure 518.** The 10 acre subbasin is located in Section 7, T23S, R62E:

Solution:

Step 1: Determine the 6-hour and 24-hour rainfall depths:

From **Figure 513** the designated basin is not within the McCarran Airport Rainfall Area. Therefore the 2-, 5-, 10-, 25-, 50-, and 100-year, 6-hour and 24-hour point precipitation values are read from **Figures 501** through **512**. For Section 7, T23S, R62E:

Return Period (yr)	6-Hour Depth (in)	24-Hour Depth (in)
2	1.0	1.2
5	1.2	1.6
10	1.45	2.0
25	1.75	2.4
50	1.9	2.8
100	2.1	3.0

Step 2: Determine the 1-hour rainfall depths:

Use **Equations 501** and **502** to find the 2-year and 100-year, 1-hour depths:

$$Y_2 = -0.011 + 0.942 [(X_1) (X_1 / X_2)] \quad (501)$$

$$Y_{100} = 0.494 + 0.755 [(X_3) (X_3 / X_4)] \quad (502)$$

$$Y_2 = -0.011 + 0.942 [(1.0) (1.0 / 1.2)] = 0.77 \text{ in}$$
$$Y_{100} = 0.494 + 0.755 [(2.1) (2.1 / 3.0)] = 1.57 \text{ in}$$

These two points are plotted on **Figure 520** and connected with a straight line to develop 1-hour depths for the other return periods.

Step 3: Determine the adjusted 1-hour rainfall depths:

The adjusted 1-hour rainfall depths are obtained by multiplying the rainfall depths from **Figure 520** by the adjustment factors in **Table 501**:

Return Period (yr)	1-Hour Depth (in)	Adjustment Factor (**Table 501**)	Adjusted 1-Hour Depth (in)
2	0.77	1.00	0.77
5	1.00	1.16	1.16
10	1.15	1.24	1.43
25	1.30	1.33	1.73
50	1.40	1.39	1.95
100	1.57	1.43	2.25

Step 4:Determine the rainfall depths and intensities for durations less than 1-hour:

The values shown above are multiplied by the factors shown in **Table 504** to obtain rainfall depths for durations less than 1-hour. These depths are then converted to intensities. The resulting rainfall depths and intensities for this example are presented in **Table 507.** The resulting time-intensity-frequency curves are shown on **Figure 521** for the 10- and 100-year return period.

Application: The time-intensity-frequency curve is used to determine the rainfall intensities (I) used in the Rational Method (Q = CIA).

PRECIPITATION ADJUSTMENT RATIOS

Recurrence Interval	Ratio to NOAA Atlas 2
2-year	1.00
5-year	1.16
10-year	1.24
25-year	1.33
50-year	1.39
100-year	1.43

NOTE:
1. Multiply the values obtained from the NOAA Atlas 2 by the above ratios to obtain the adjusted precipitation values.
2. NOAA Atlas 2 values for use with TR-55 shall not be adjusted by the above ratios.
3. Tables 505 and 506 require no adjustments.

Revision	Date

SIX HOUR DEPTH-AREA REDUCTION FACTORS

Drainage Area (Square Miles)	Six-Hour Depth-Area Reduction Factor
0.0	1.00
0.5	0.98
1.0	0.97
2.0	0.93
4.0	0.91
6.0	0.90
8.0	0.88
10.0	0.86
20.0	0.79
30.0	0.74
50.0	0.68
100.0	0.60
150.0	0.55
200.0	0.51
300.0	0.46
400.0	0.42
500.0	0.39

NOTES:

1. A graphical representation of these factors is presented in Figure 514.
2. Consult with the local entity and/or the CCRFCD for guidance in using the Depth-Area Reduction Factors for drainage areas greater than 200 square miles.

Revision	Date

SIX-HOUR STORM DISTRIBUTIONS

Storm Time (In Minutes)	Percent of Total Storm Depth SDN3	SDN4	SDN5	Storm Time (In Minutes)	Percent of Total Storm Depth SDN3	SDN4	SDN5
0	0.0	0.0	0.0	185	32.2	37.6	43.0
5	2.0	2.0	2.0	190	35.2	41.5	47.7
10	5.7	5.8	5.9	195	40.9	46.2	51.4
15	7.0	7.5	8.0	200	49.9	53.0	56.1
20	8.7	9.9	11.0	205	59.0	61.0	63.0
25	10.8	12.6	14.4	210	71.0	71.0	71.0
30	12.4	13.7	15.0	215	74.4	73.2	72.0
35	13.0	14.5	16.0	220	78.1	75.6	73.1
40	13.0	14.9	16.8	225	81.2	78.2	75.2
45	13.0	15.1	17.1	230	81.9	79.9	77.9
50	13.0	15.5	18.0	235	83.5	81.3	79.0
55	13.0	15.6	18.2	240	85.1	82.3	79.5
60	13.0	15.9	18.7	245	85.6	83.0	80.4
65	13.3	16.2	19.0	250	86.0	83.5	81.0
70	14.0	16.9	19.7	255	86.8	84.4	82.0
75	14.2	17.2	20.2	260	87.6	85.1	82.6
80	14.8	17.9	21.0	265	88.8	86.4	84.0
85	15.8	18.9	22.0	270	91.0	88.5	85.9
90	17.2	20.1	23.0	275	92.6	90.8	88.9
95	18.1	21.1	24.1	280	93.7	92.4	91.0
100	19.0	22.0	25.0	285	95.0	94.4	93.8
105	19.7	22.8	25.9	290	97.0	96.8	96.6
110	19.9	23.2	26.5	295	97.6	97.3	97.0
115	20.0	24.0	28.0	300	98.2	97.8	97.4
120	20.1	24.6	29.0	305	98.5	98.2	97.9
125	20.4	25.2	30.0	310	98.7	98.4	98.1
130	21.4	26.0	30.5	315	98.9	98.6	98.3
135	22.9	26.9	30.9	320	99.0	98.8	98.5
140	24.1	27.6	31.0	325	99.3	99.1	98.9
145	24.9	28.3	31.7	330	99.3	99.2	99.0
150	25.1	28.6	32.1	335	99.4	99.3	99.2
155	25.6	29.2	32.7	340	99.5	99.4	99.3
160	27.0	30.2	33.3	345	99.8	99.7	99.6
165	27.8	31.2	34.6	350	99.8	99.8	99.7
170	28.1	32.1	36.1	355	99.9	99.9	99.9
175	28.3	33.2	38.1	360	100.0	100.0	100.0
180	29.5	35.2	40.8				

Notes:
1. For drainage areas less than 8 square miles in size, use SDN 3.
2. For drainage areas greater than or equal to 8 square miles and less than 12 square miles in size, use SDN 4.
3. For drainage areas greater than or equal to 12 square miles, use SDN 5.
4. A graphical representation of these values is presented on **Figure 515**.

Revision	Date

REFERENCE:

FACTORS FOR DURATIONS OF LESS THAN ONE-HOUR

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

NOTE: 1. Multiply the 1-hour precipitation depths by the above ratios to obtain the precipitation depths for storm durations of less than 1-hour.

Revision	Date

DEPTH-DURATION-FREQUENCY VALUES FOR McCARRAN AIRPORT RAINFALL AREA

(IN INCHES)

	RECURRENCE INTERVAL					
TIME	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR
5 min.	0.15	0.27	0.35	0.46	0.54	0.63
10 min.	0.25	0.44	0.57	0.74	0.89	1.02
15 min.	0.33	0.57	0.74	0.97	1.15	1.32
30 min.	0.44	0.78	1.01	1.31	1.55	1.79
1 hour	0.52	0.89	1.15	1.50	1.78	2.06
2 hour	0.59	1.01	1.30	1.70	2.01	2.30
3 hour	0.64	1.08	1.39	1.82	2.15	2.48
6 hour	0.72	1.22	1.58	2.05	2.41	2.77
24 hour (TR-55)	1.20	1.60	1.80	2.40	2.70	2.96

NOTE:
1. Refer to Figure 513 for a description and drawing of the area included in the McCarran Airport Rainfall Area.
2. The 24 hour values presented above are for use with TR-55 only.
3. Table 501 adjustments not required.

Revision	Date

TIME-INTENSITY-FREQUENCY VALUES FOR McCARRAN AIRPORT RAINFALL AREA

(IN INCHES PER HOUR)

TIME (min)	RECURRENCE INTERVAL 2-YR	5-YR	10-YR	25-YR	50-YR	100-YR
5	1.80	3.24	4.20	5.52	6.48	7.56
10	1.50	2.64	3.42	4.44	5.34	6.12
15	1.32	2.28	2.96	3.88	4.60	5.28
30	0.88	1.56	2.02	2.62	3.10	3.58
60	0.52	0.89	1.15	1.50	1.78	2.06

NOTE: 1. Refer to Figure 513 for a description and drawing of the area included in the McCarran Airport Rainfall Area.

2. Table 501 adjustments not required.

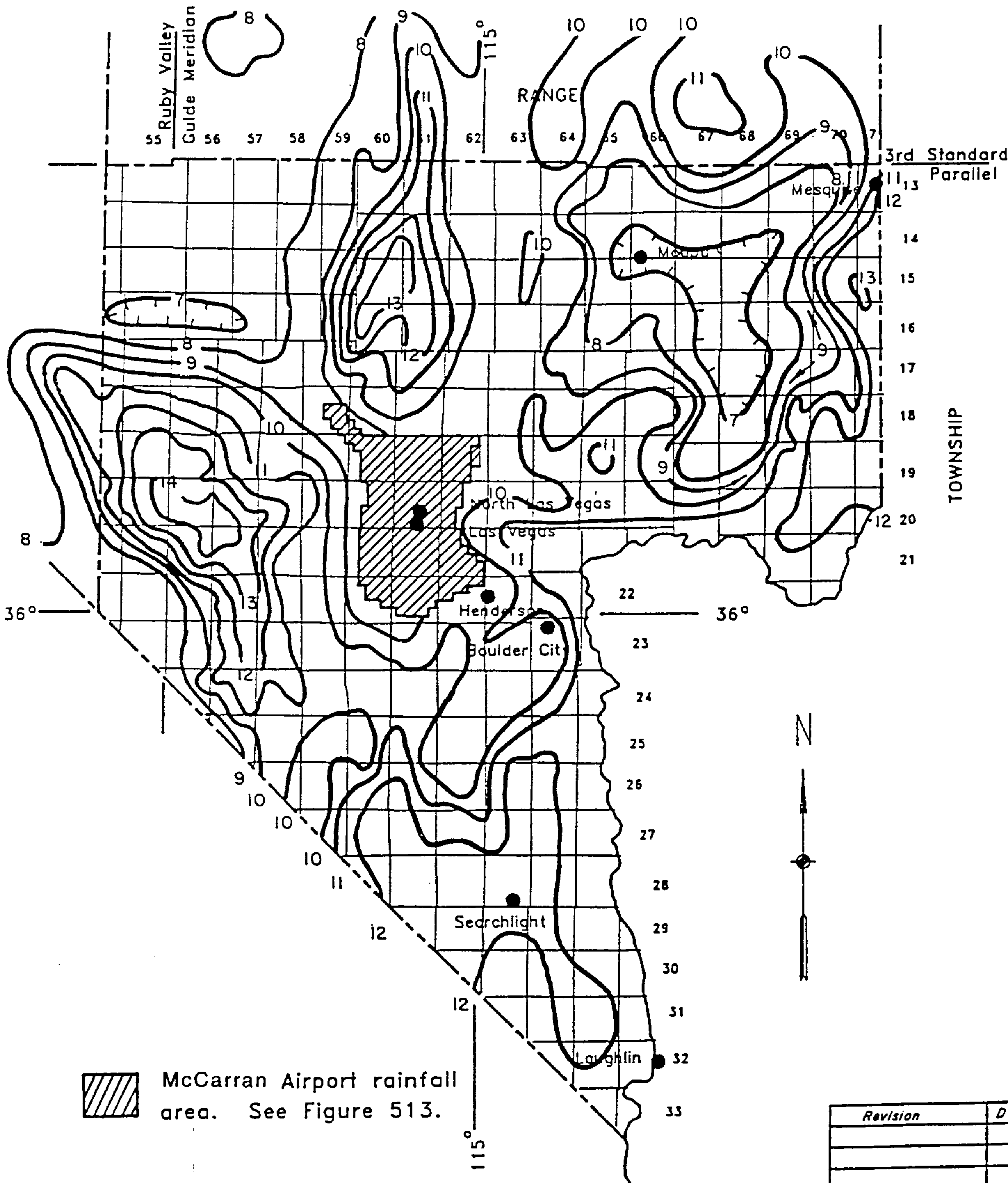
Revision	Date

TIME-INTENSITY-FREQUENCY VALUES FOR EXAMPLE IN SECTION 507.3

Return Period (Yr)	Adjusted 1-Hr Depth (In)	Duration 5 Min. (In)		10 Min. (In)		15 Min. (In)		30 Min. (In)	
		(In)	(In/Hr)	(In)	(In/Hr)	(In)	(In/Hr)	(In)	(In/Hr)
2	0.77	0.22	2.68	0.35	2.08	0.44	1.76	0.61	1.22
5	1.16	0.34	4.04	0.52	3.13	0.66	2.64	0.92	1.83
10	1.43	0.41	4.98	0.64	3.86	0.82	3.26	1.13	2.26
25	1.73	0.50	6.02	0.78	4.67	0.99	3.94	1.37	2.73
50	1.95	0.57	6.79	0.88	5.27	1.11	4.45	1.54	3.08
100	2.25	0.65	7.83	1.01	6.08	1.28	5.13	1.78	3.56

Revision	Date

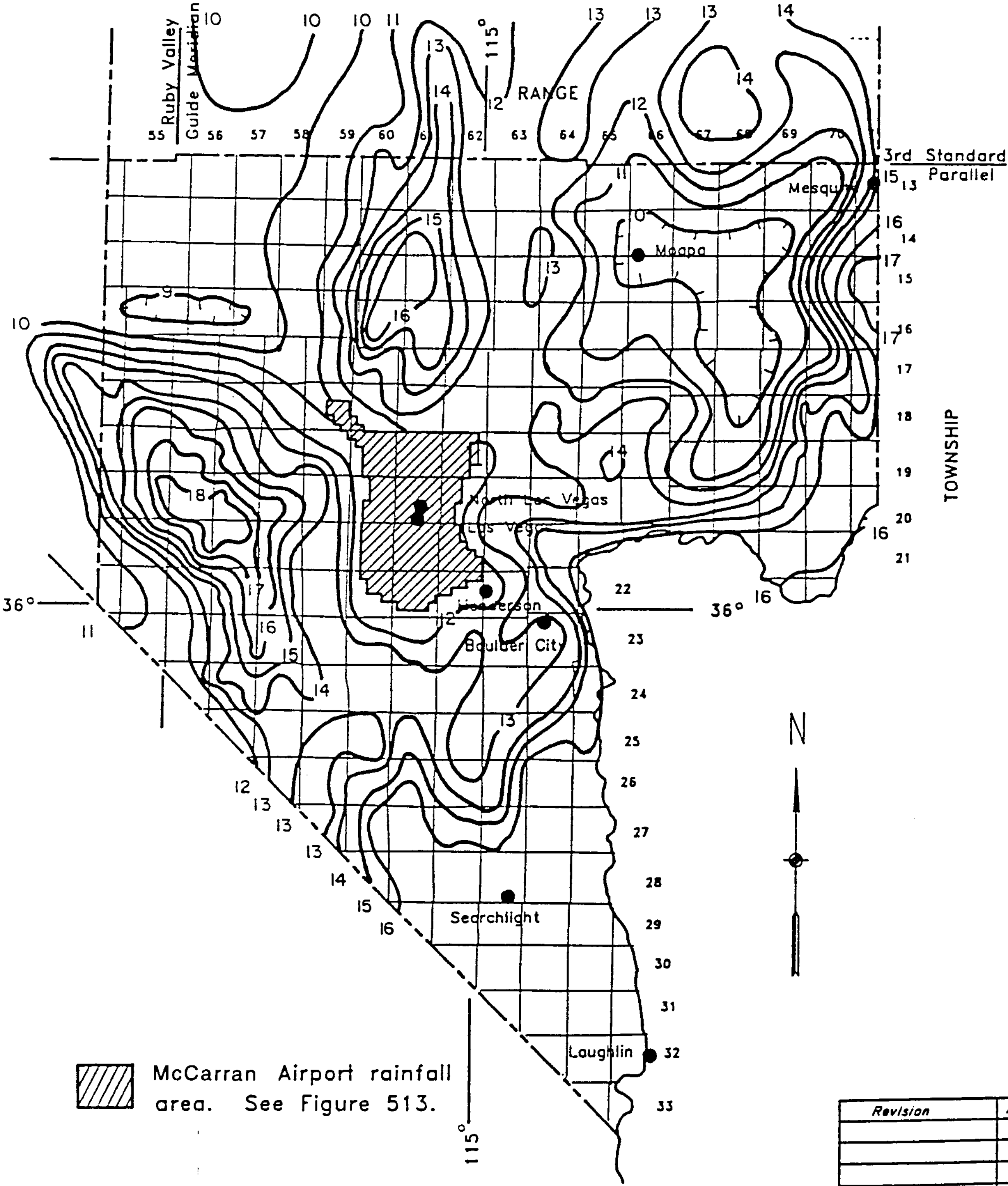
RAINFALL DEPTH-DURATION-FREQUENCY
2-YEAR, 6-HOUR
(DEPTHS IN TENTHS OF INCHES)



Revision	Date

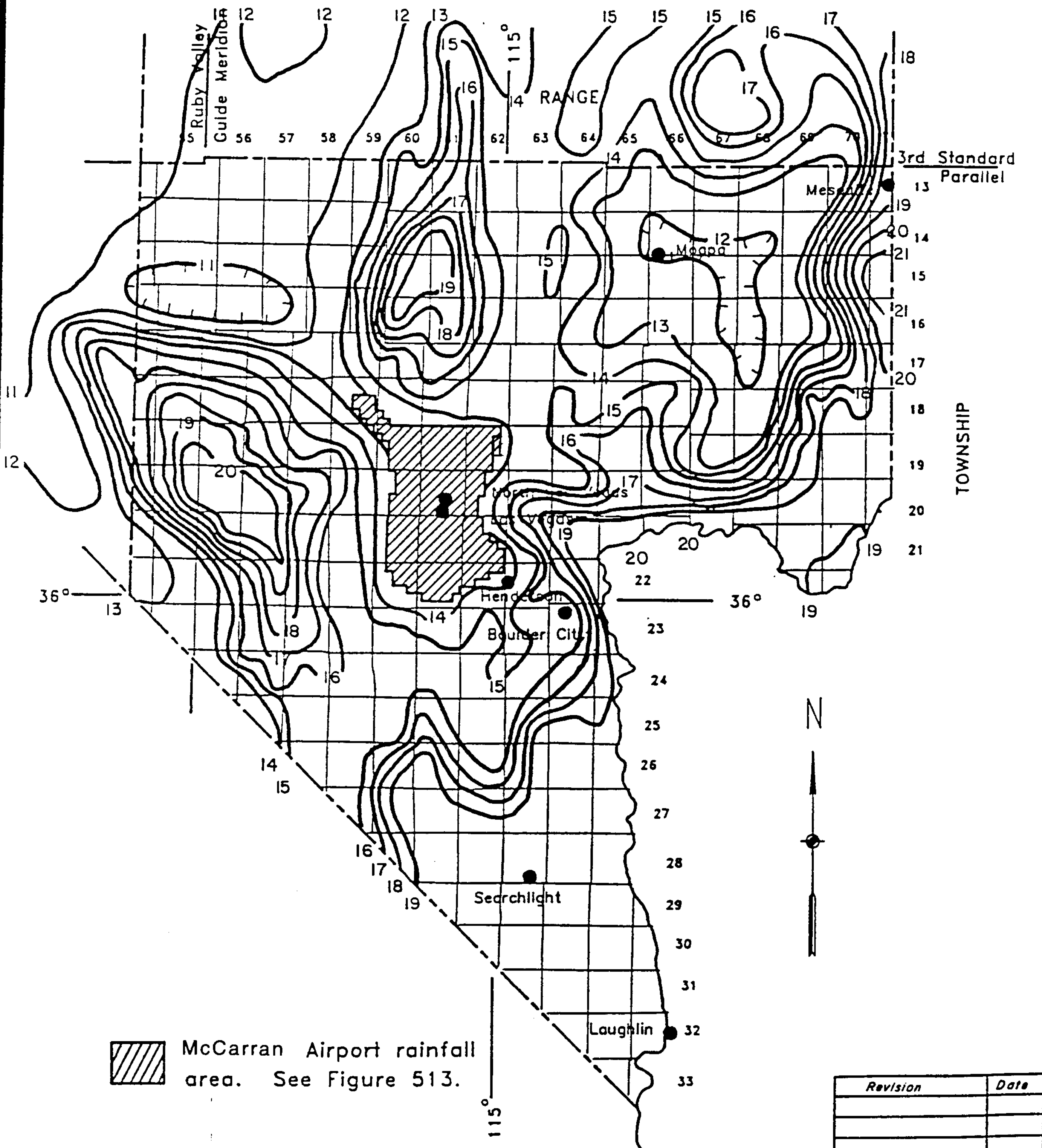
RAINFALL DEPTH-DURATION-FREQUENCY 5-YEAR, 6-HOUR

(DEPTHS IN TENTHS OF INCHES)



Revision	Date

RAINFALL DEPTH-DURATION-FREQUENCY
10-YEAR, 6-HOUR
(DEPTHS IN TENTHS OF INCHES)



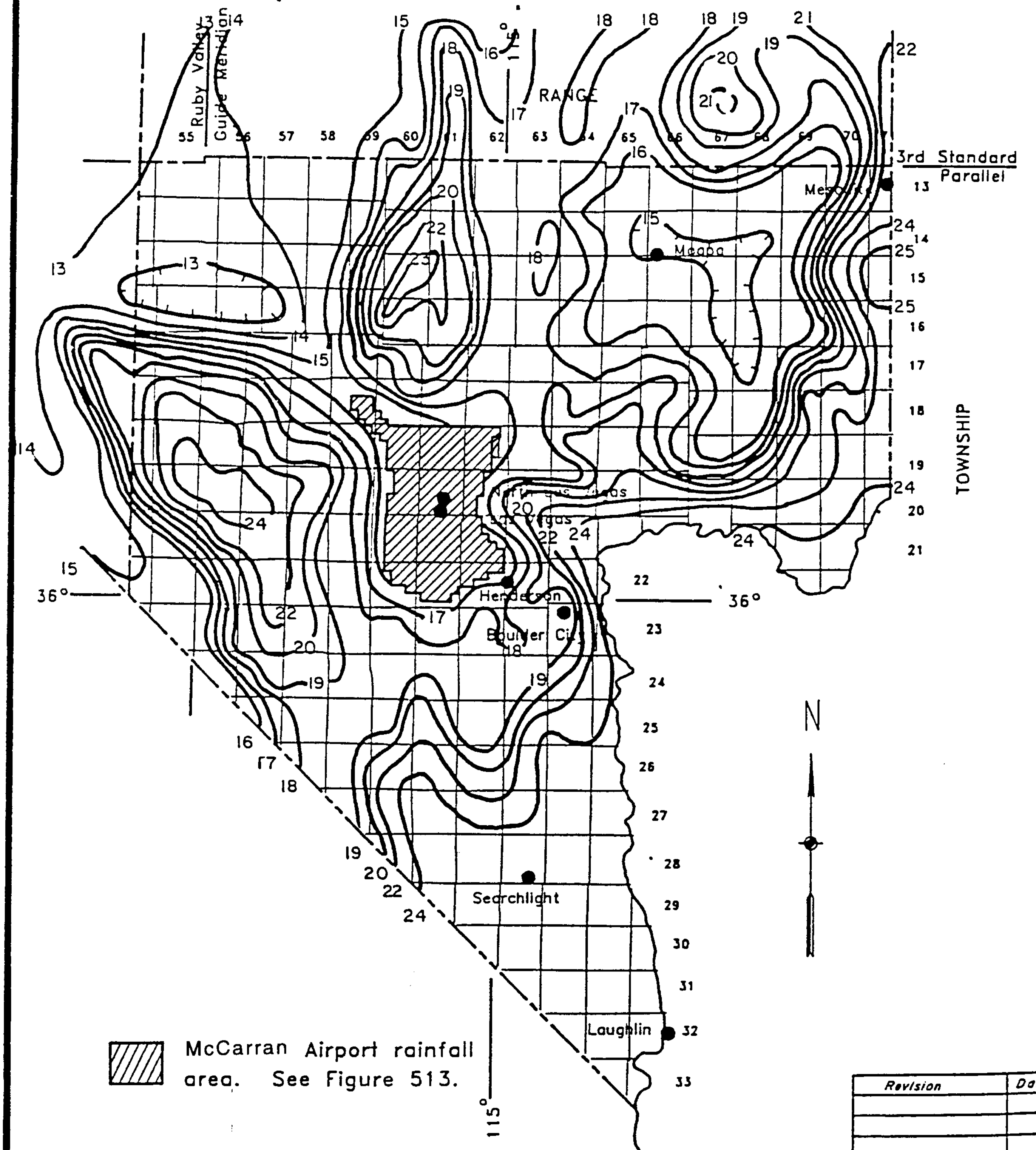
Revision	Date

WRC ENGINEERING

REFERENCE:
NOAA ATLAS 2, VOLUME VII NEVADA, 1973

FIGURE 503

RAINFALL DEPTH-DURATION-FREQUENCY
25-YEAR, 6-HOUR
(DEPTHS IN TENTHS OF INCHES)

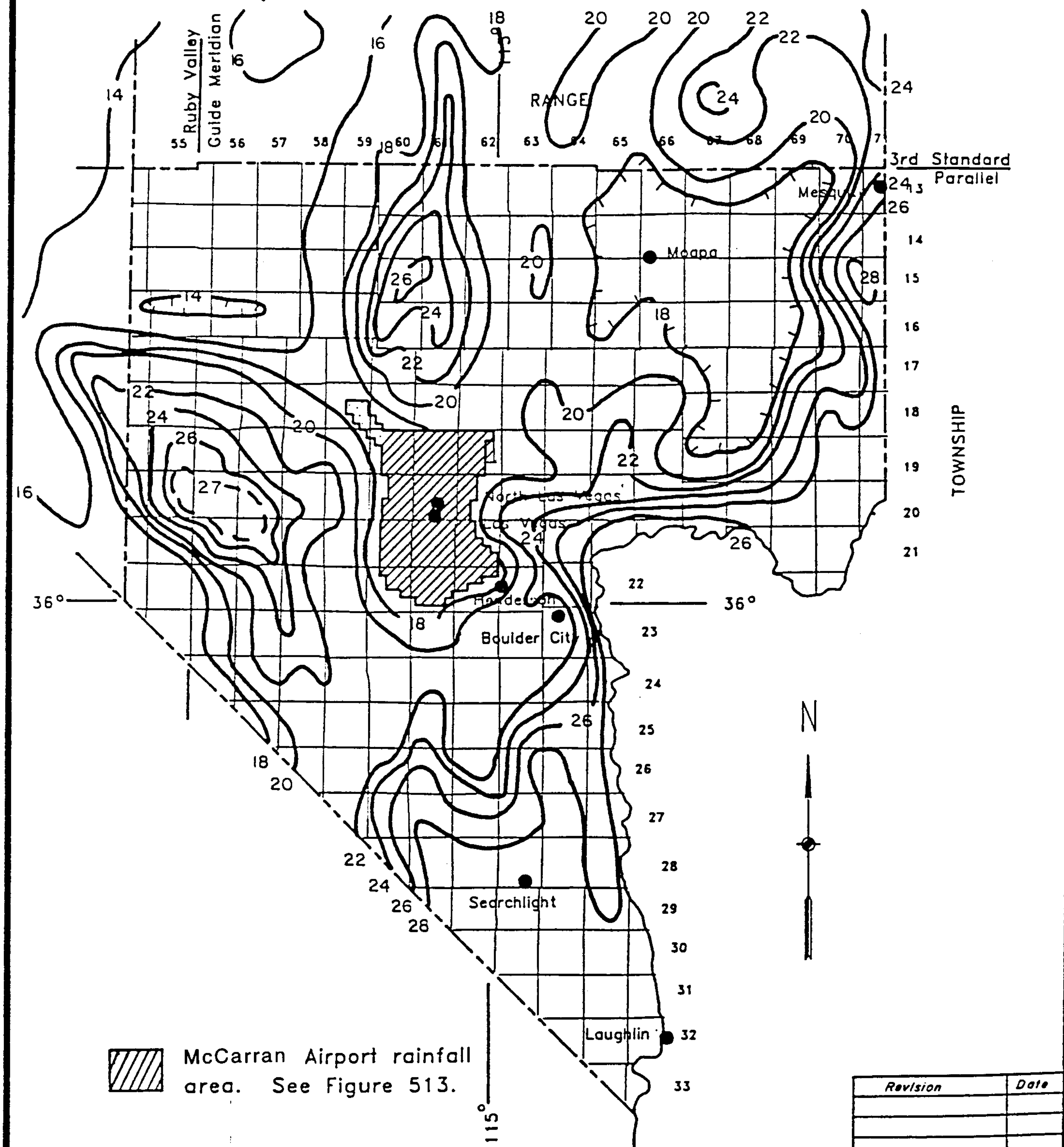


McCarran Airport rainfall area. See Figure 513.

Revision	Date

WRC ENGINEERING	REFERENCE: NOAA ATLAS 2, VOLUME VII NEVADA, 1973	FIGURE 504

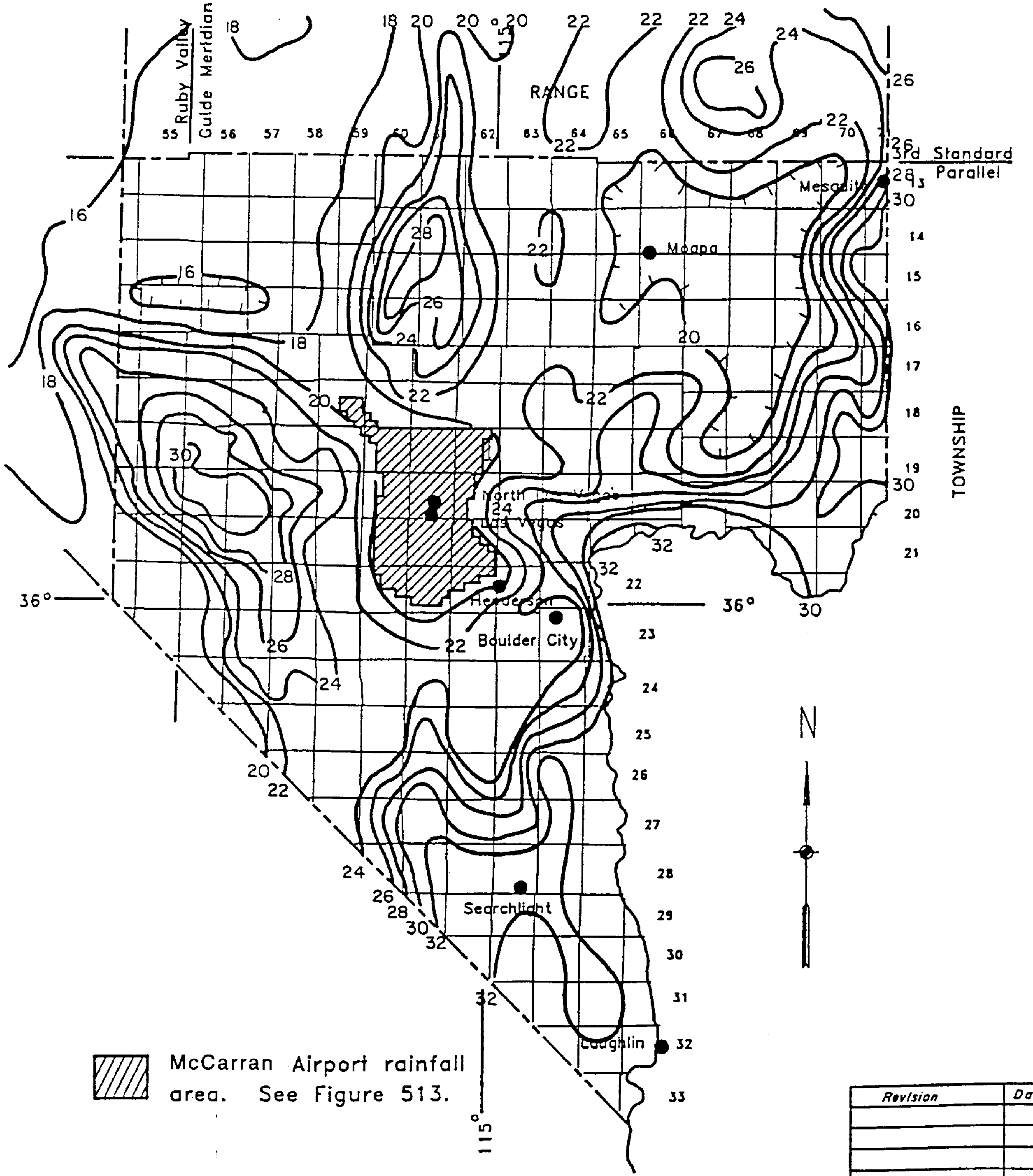
RAINFALL DEPTH-DURATION-FREQUENCY
50-YEAR, 6-HOUR
(DEPTHS IN TENTHS OF INCHES)



Revision	Date

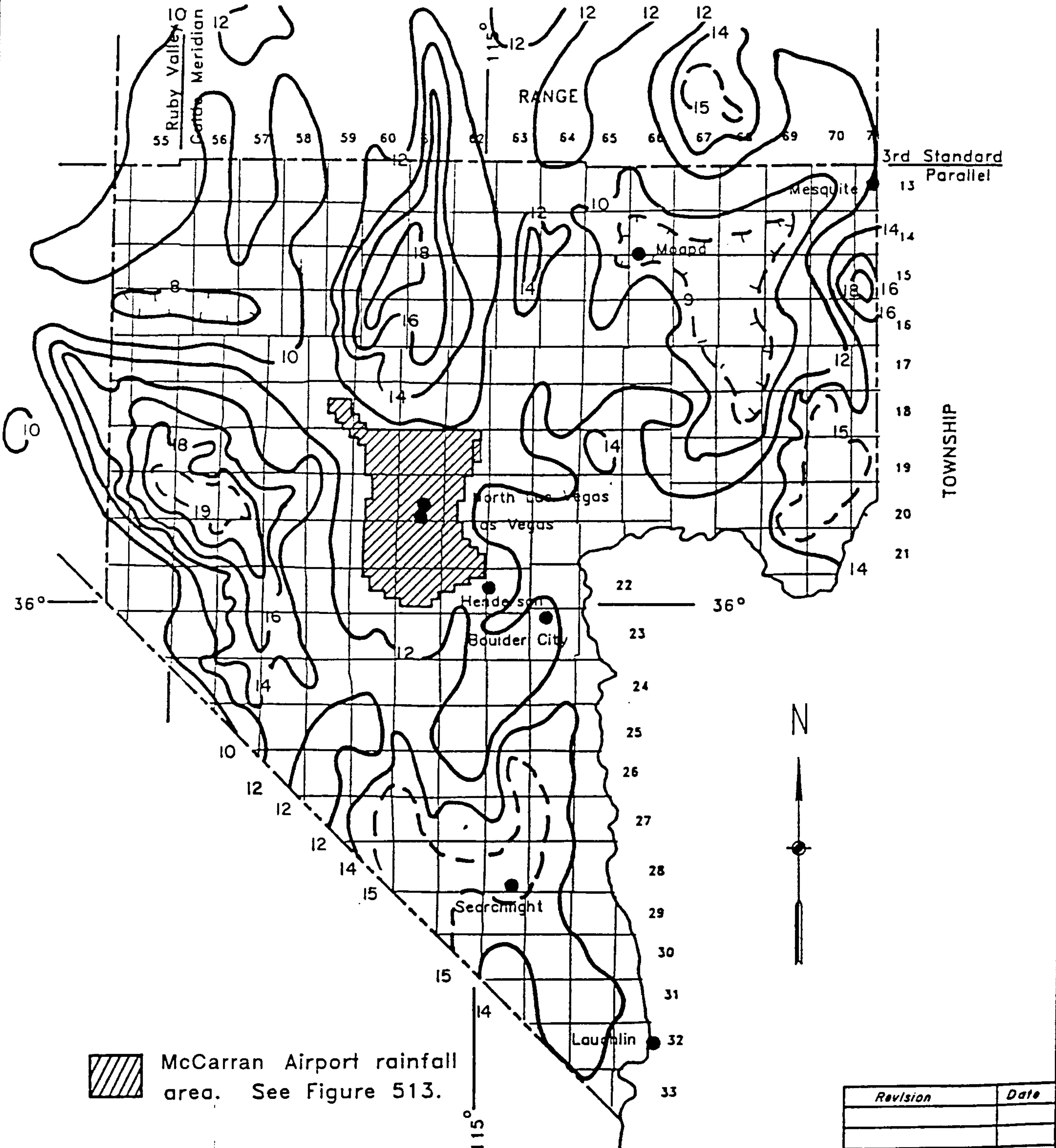
WRC ENGINEERING	REFERENCE: NOAA ATLAS 2, VOLUME VII NEVADA, 1973	FIGURE 505

RAINFALL DEPTH-DURATION-FREQUENCY 100-YEAR, 6-HOUR (DEPTHS IN TENTHS OF INCHES)



Revision	Date

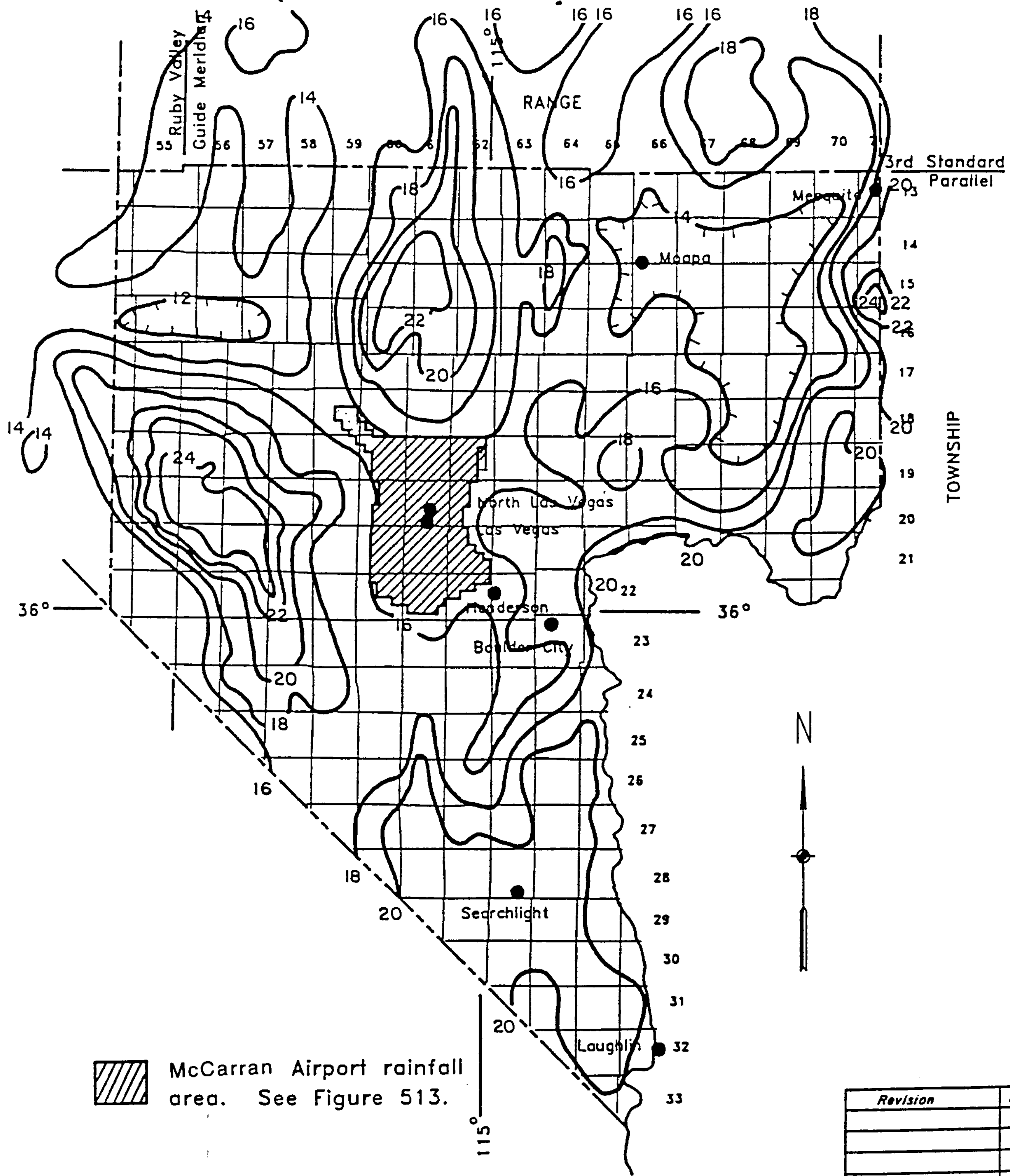
RAINFALL DEPTH-DURATION-FREQUENCY
2-YEAR, 24-HOUR
(DEPTHS IN TENTHS OF INCHES)



Revision	Date

WRC ENGINEERING	REFERENCE: NOAA ATLAS 2, VOLUME VII NEVADA, 1973	FIGURE 507

RAINFALL DEPTH-DURATION-FREQUENCY
5-YEAR, 24-HOUR
(DEPTHS IN TENTHS OF INCHES)



Revision	Date

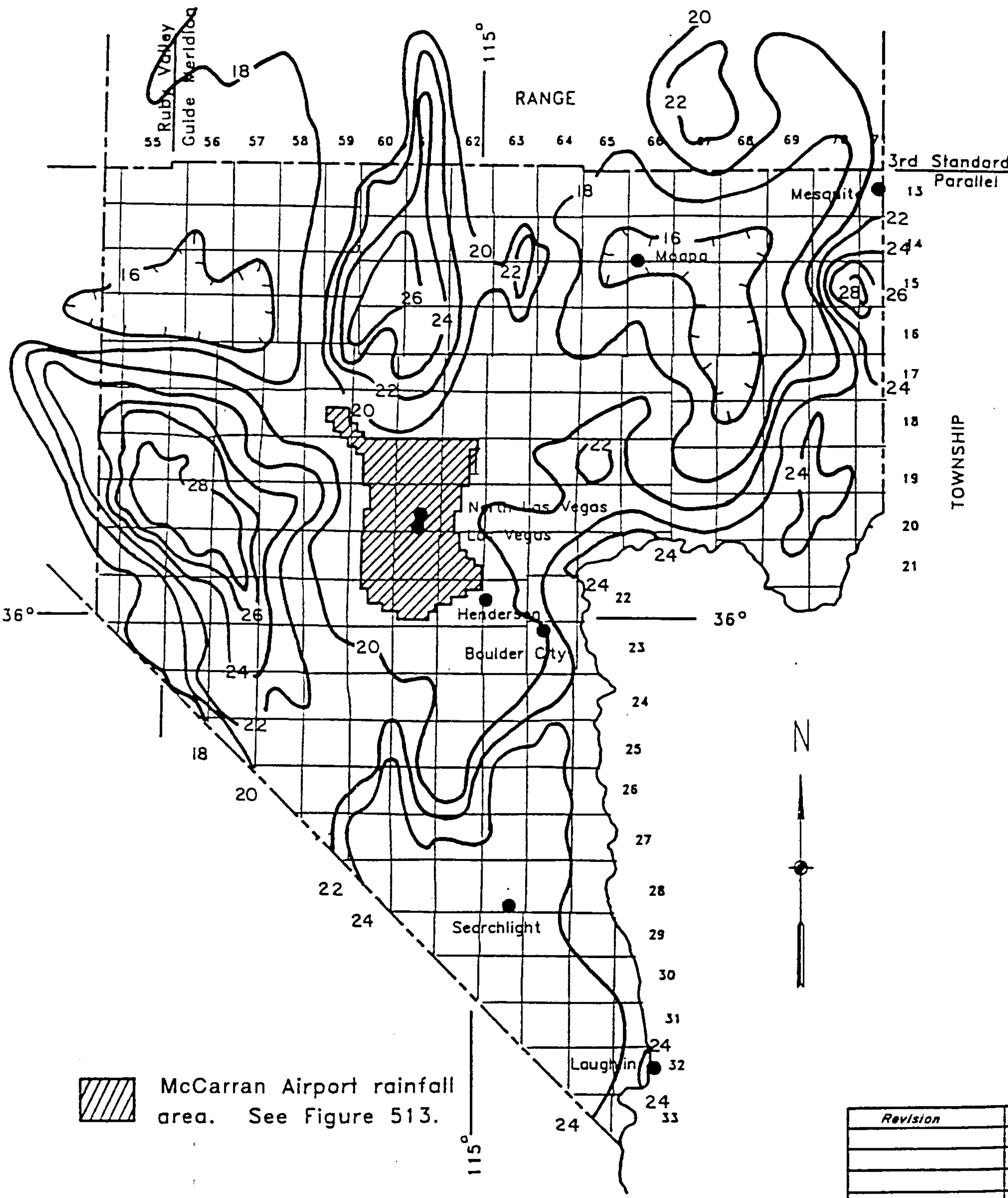
WRC ENGINEERING

REFERENCE:
NOAA ATLAS 2, VOLUME VII NEVADA, 1973

FIGURE 508

RAINFALL DEPTH-DURATION-FREQUENCY 10-YEAR, 24-HOUR

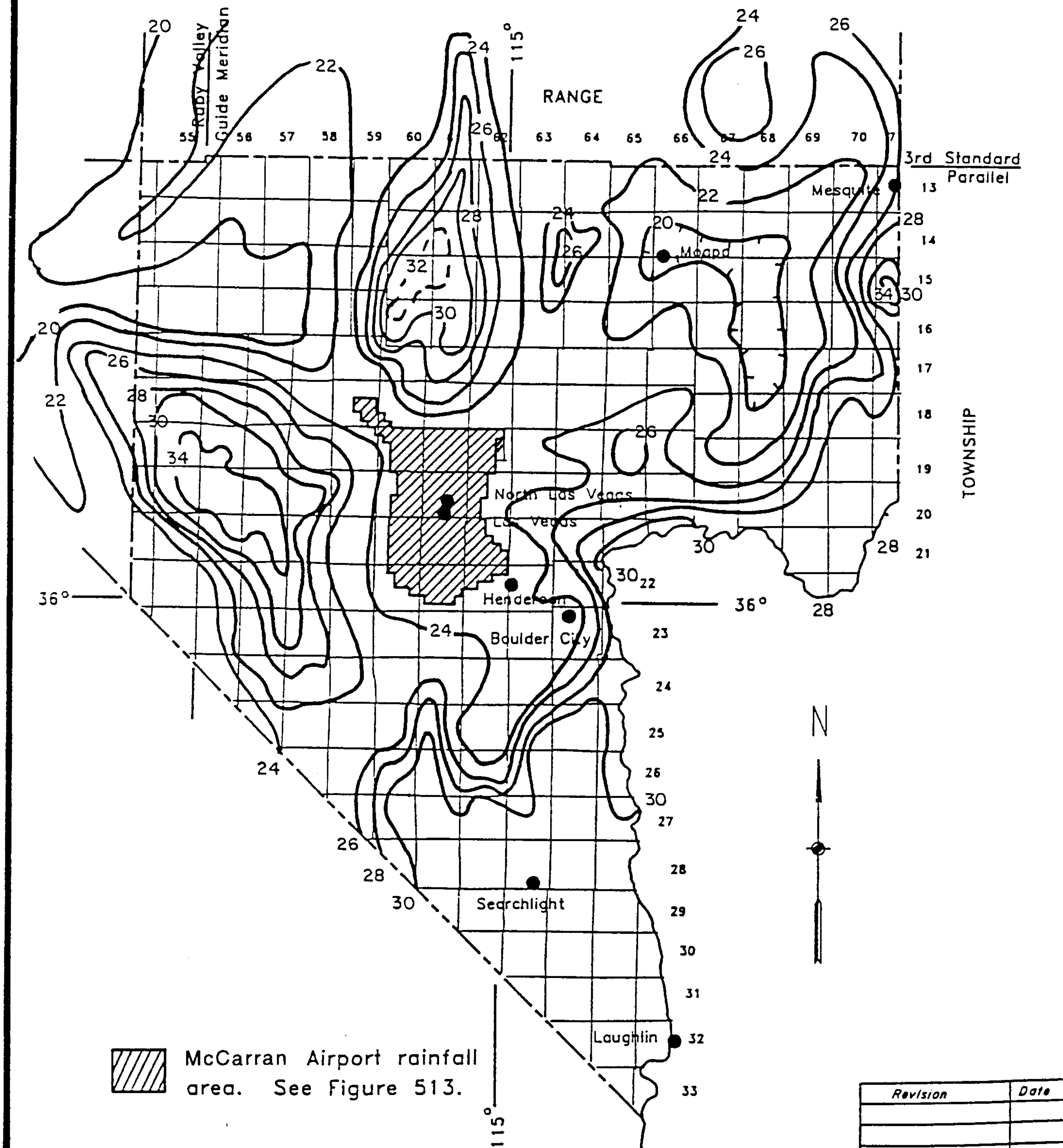
(DEPTHS IN TENTHS OF INCHES)



Revision	Date

WRC ENGINEERING	REFERENCE: NOAA ATLAS 2, VOLUME VII NEVADA, 1973	FIGURE 509

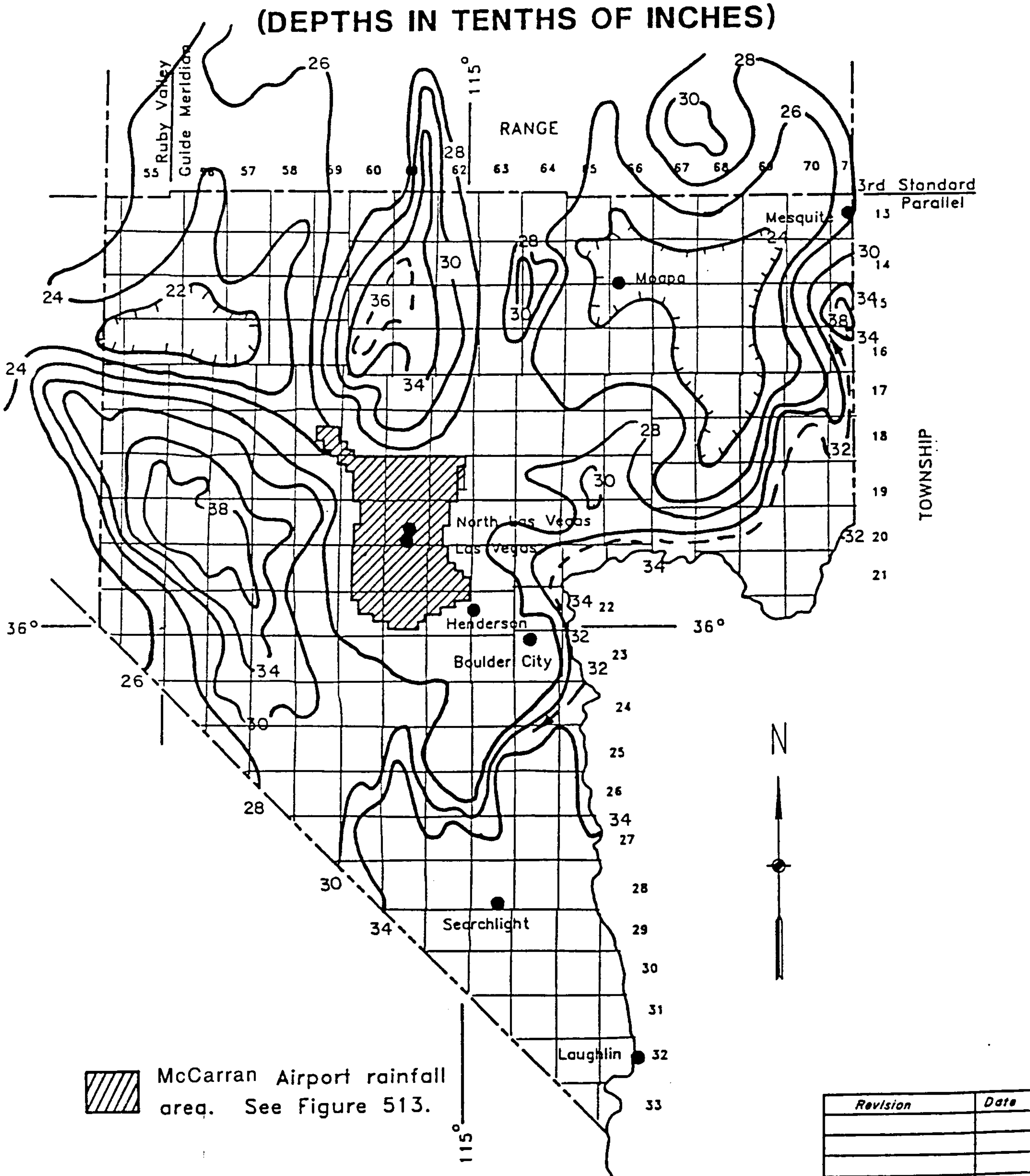
RAINFALL DEPTH-DURATION-FREQUENCY
25-YEAR, 24-HOUR
(DEPTHS IN TENTHS OF INCHES)



Revision	Date

REFERENCE:
NOAA ATLAS 2, VOLUME VII NEVADA, 1973

RAINFALL DEPTH-DURATION-FREQUENCY
50-YEAR, 24-HOUR
(DEPTHS IN TENTHS OF INCHES)

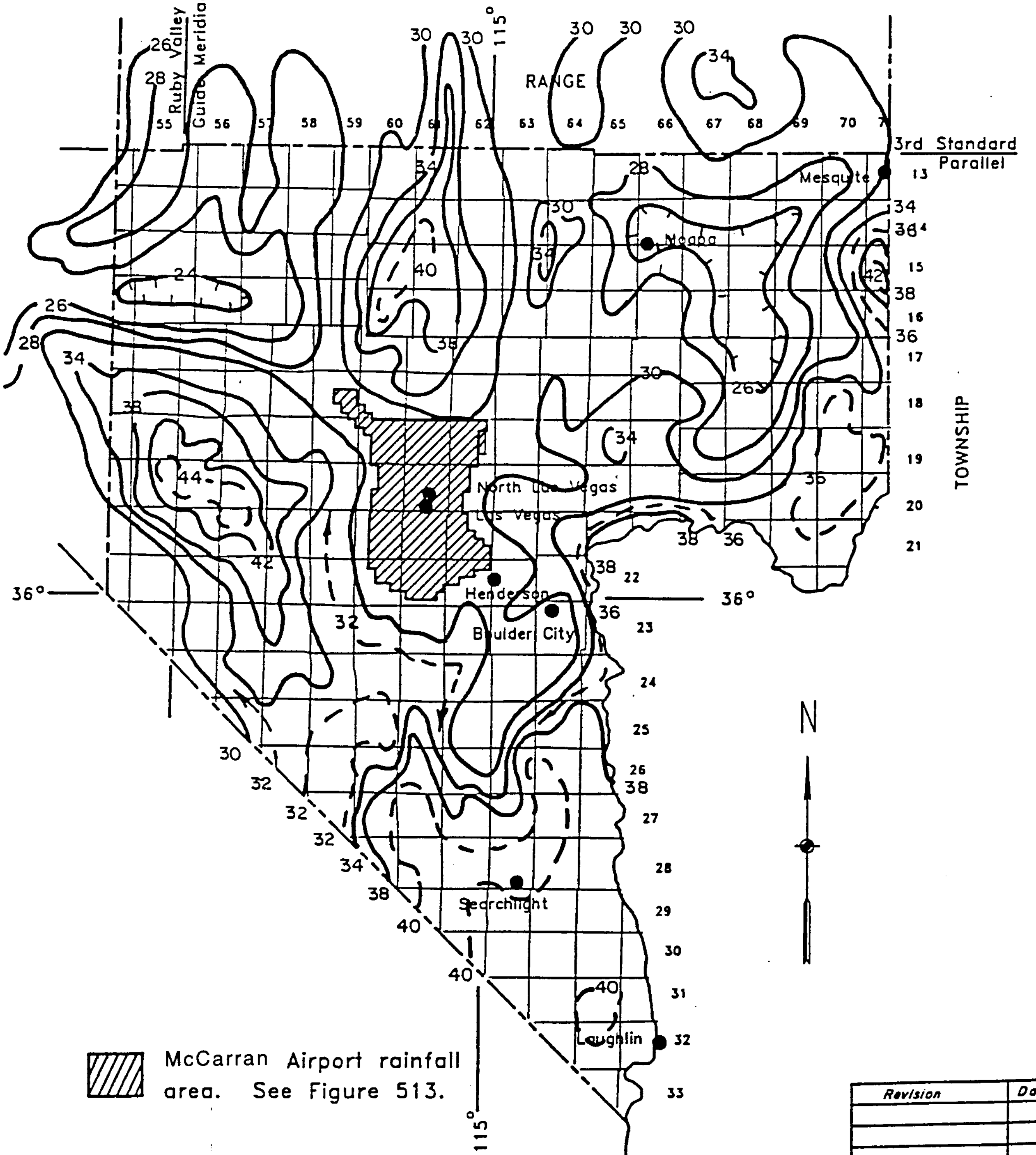


Revision	Date

REFERENCE:
NOAA ATLAS 2, VOLUME VII NEVADA, 1973

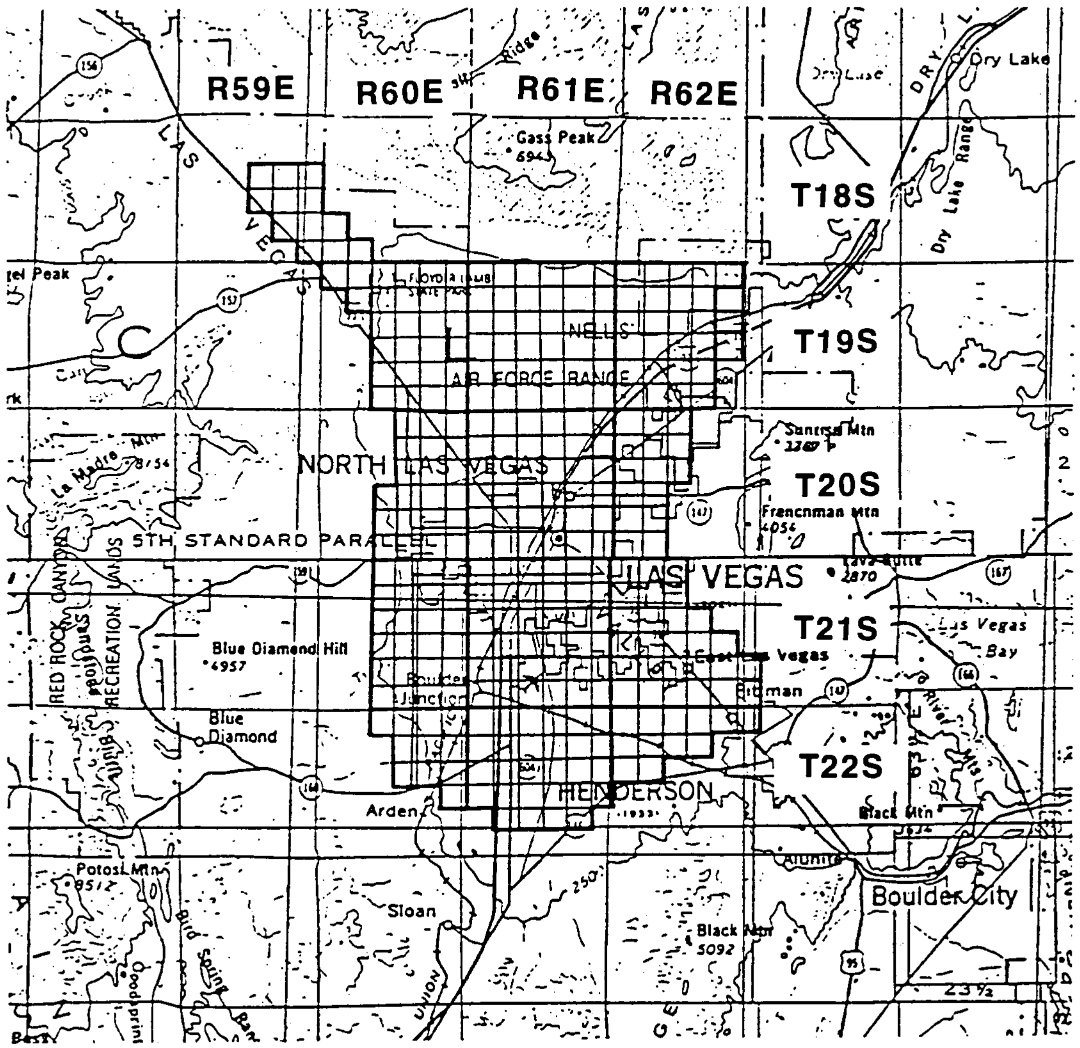
RAINFALL DEPTH-DURATION-FREQUENCY 100-YEAR, 24-HOUR

(DEPTHS IN TENTHS OF INCHES)



Revision	Date

McCARRAN AIRPORT RAINFALL AREA

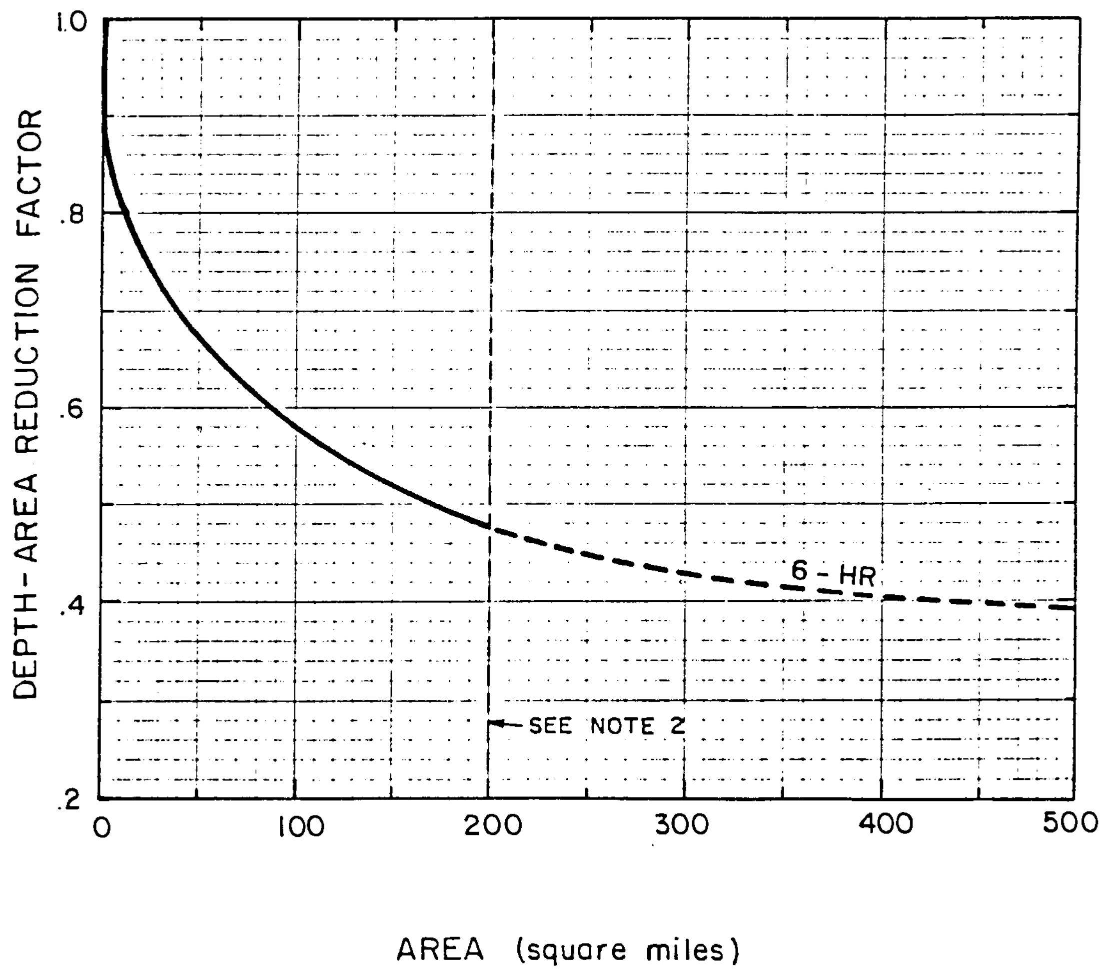


TOWNSHIP	RANGE	SECTIONS	TOWNSHIP	RANGE	SECTIONS
18 South	59 East	13-15,22-26,36	20 South	62 East	4-9,16-20,29-32
18 South	60 East	30-32	21 South	60 East	1-4,9-16,21-28,33-36
19 South	60 East	1-6,8-16,21-28,33-36	21 South	61 East	ALL SECTIONS
19 South	61 East	ALL SECTIONS	21 South	62 East	4-9,15-23, 25-36
19 South	62 East	2-11,14-23,27-34	22 South	60 East	1-4,10-15,24
20 South	60 East	1-3,10-15,21-28,33-36	22 South	61 East	1-24,26-29
20 South	61 East	ALL SECTIONS	22 South	62 East	1-10,17-18

Notes:
1. Refer to Table 505 and Figure 516 Depth-Duration- Frequency values in the McCarran Airport Rainfall Area.
2. Refer to Table 506 and Figure 517 for Time-Intensity-Frequency values on the McCarran Airport Rainfall Area.

Revision	Date

DEPTH-AREA REDUCTION CURVES



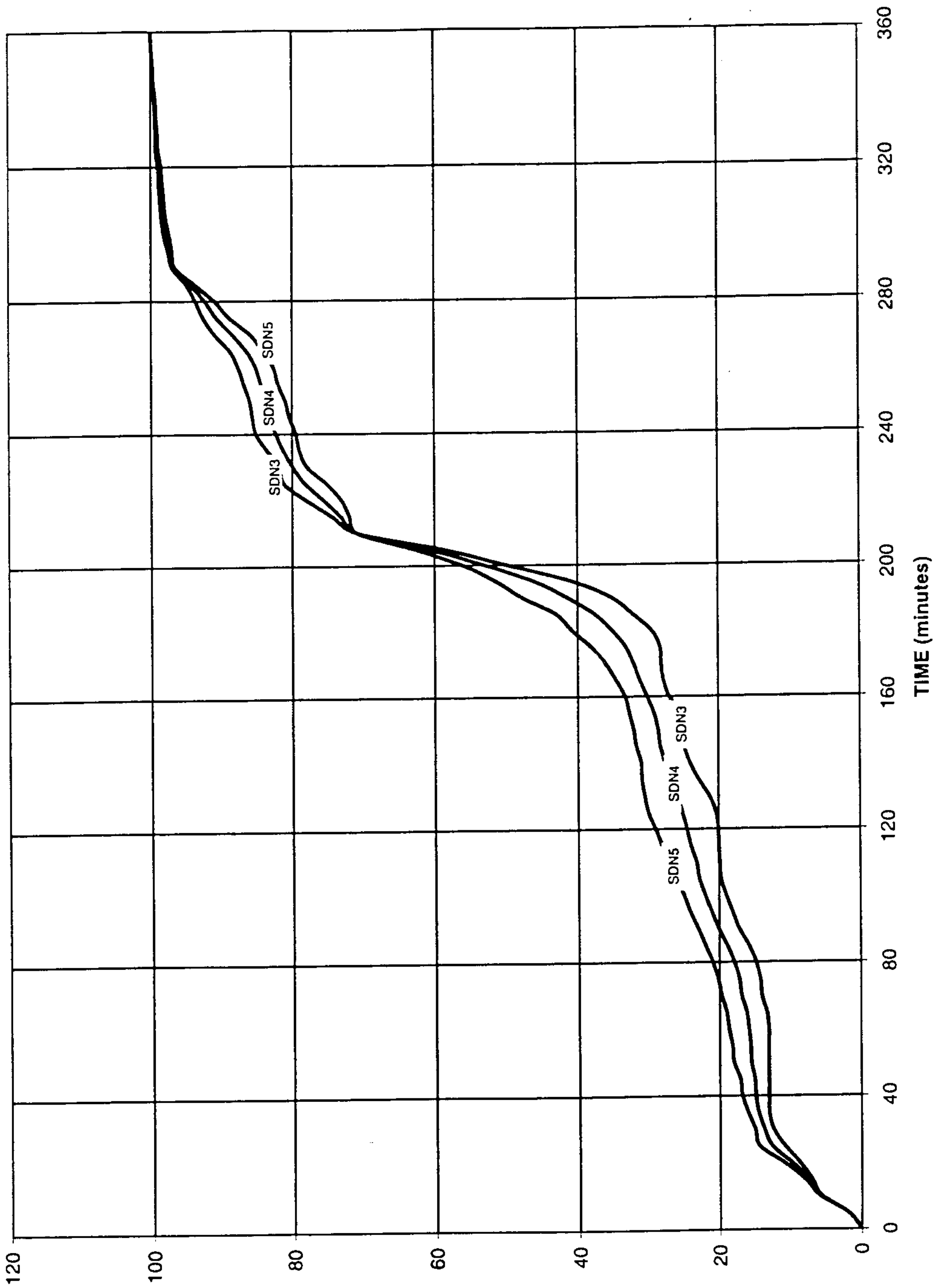
Note:

1. Refer to Table 502 for a tabulation of the 6-Hour Depth-Area reduction factors.

2. Consult with the local entity and/or the CCRFCD for guidance in using the Depth-Area Reduction factors for drainage areas greater than 200 miles.

Revision	Date

SIX-HOUR DESIGN STORM DISTRIBUTIONS

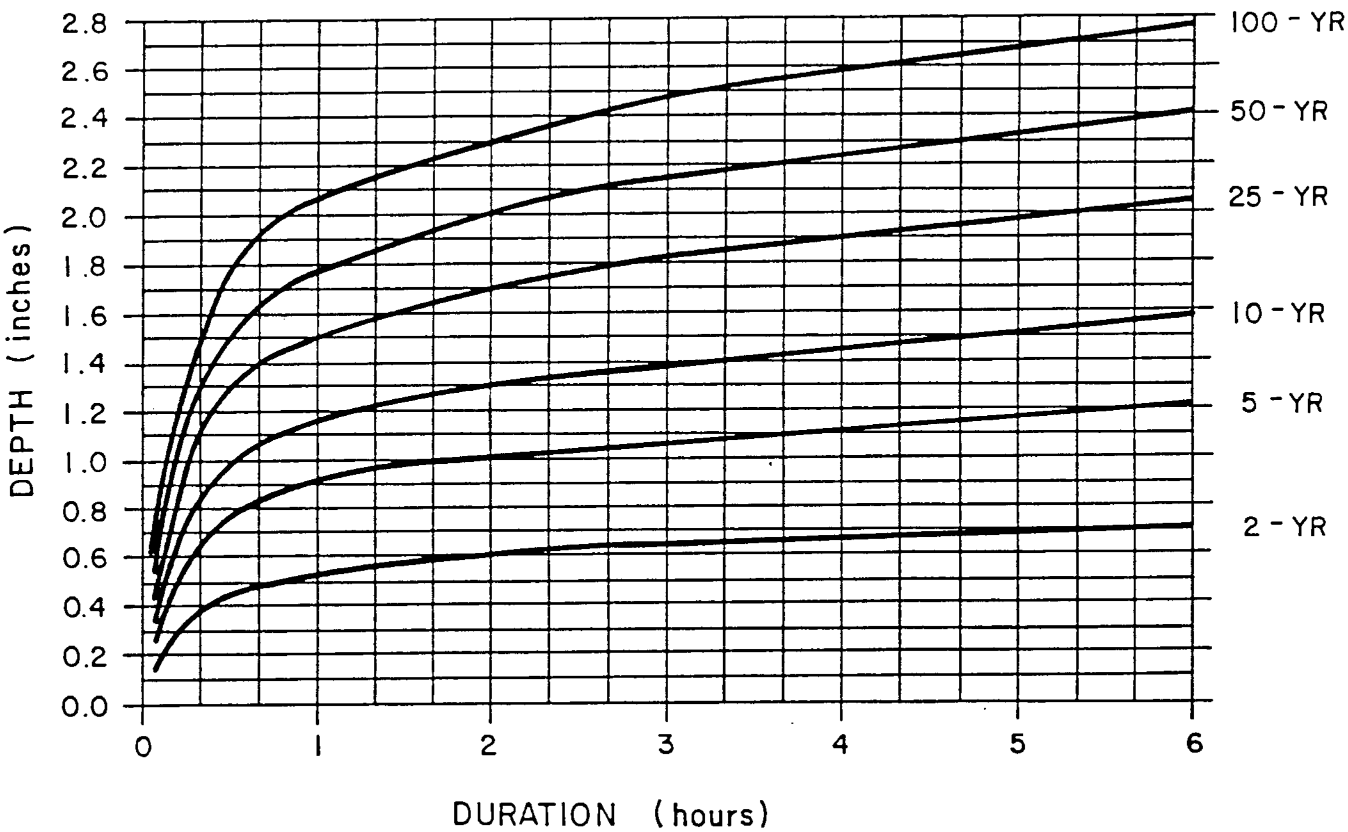


Notes:
1. For drainage areas less than 8 square miles in size, use SDN 3.
2. For drainage areas greater than or equal to 8 square miles and less than 12 square miles in size, use SDN 4.
3. For drainage areas greater than or equal to 12 square miles, use SDN 5.
4. Refer to **Table 503** for a tabulation of storm distribution.

Revision	Date

REFERENCE:

DEPTH-DURATION-FREQUENCY CURVES FOR McCARRAN AIRPORT RAINFALL AREA

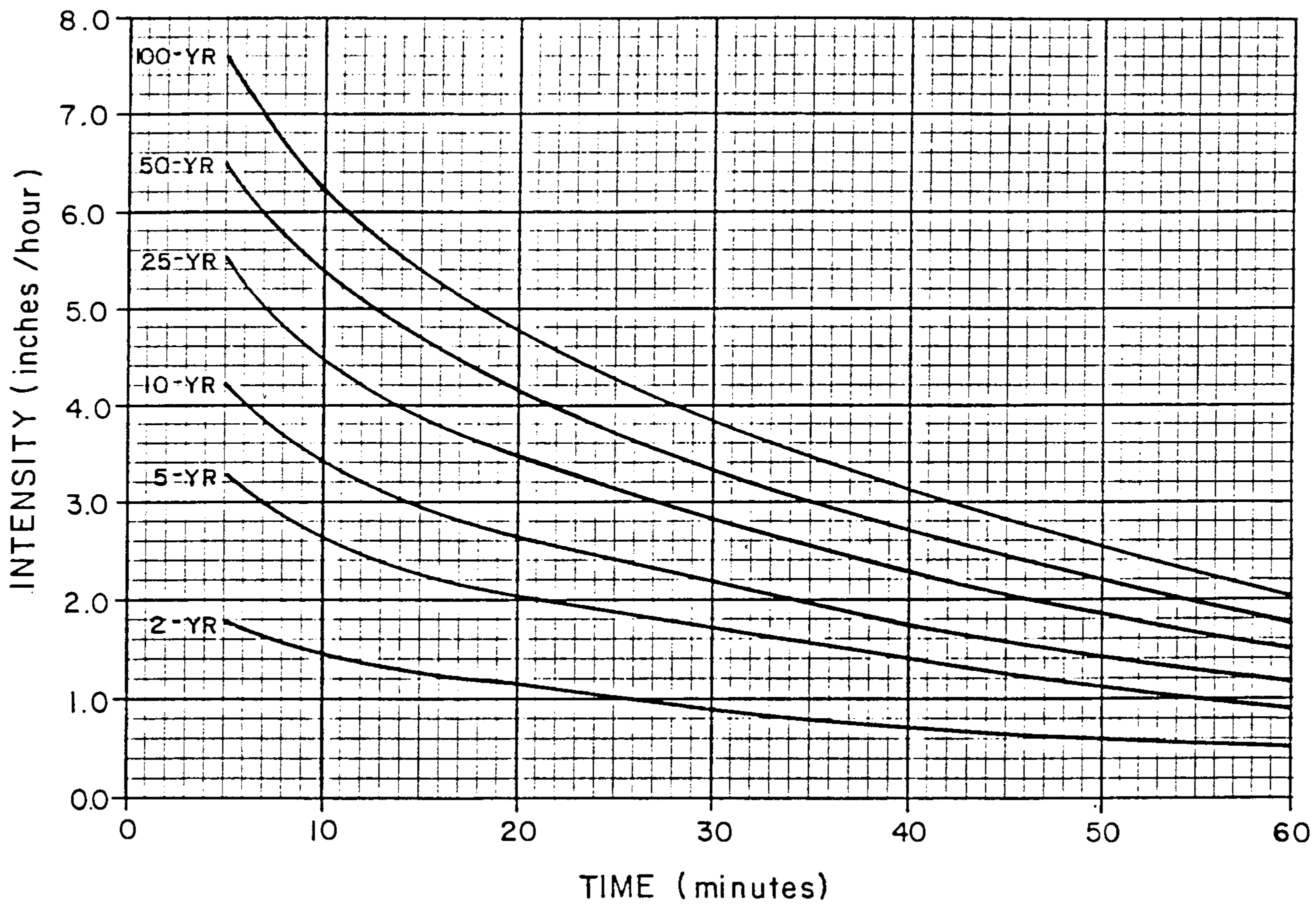


NOTES: 1. Refer to Table 505 for tabulation of depth-duration-frequency values.

Revision	Date

REFERENCE:
USACE, Los Angeles District, 1988

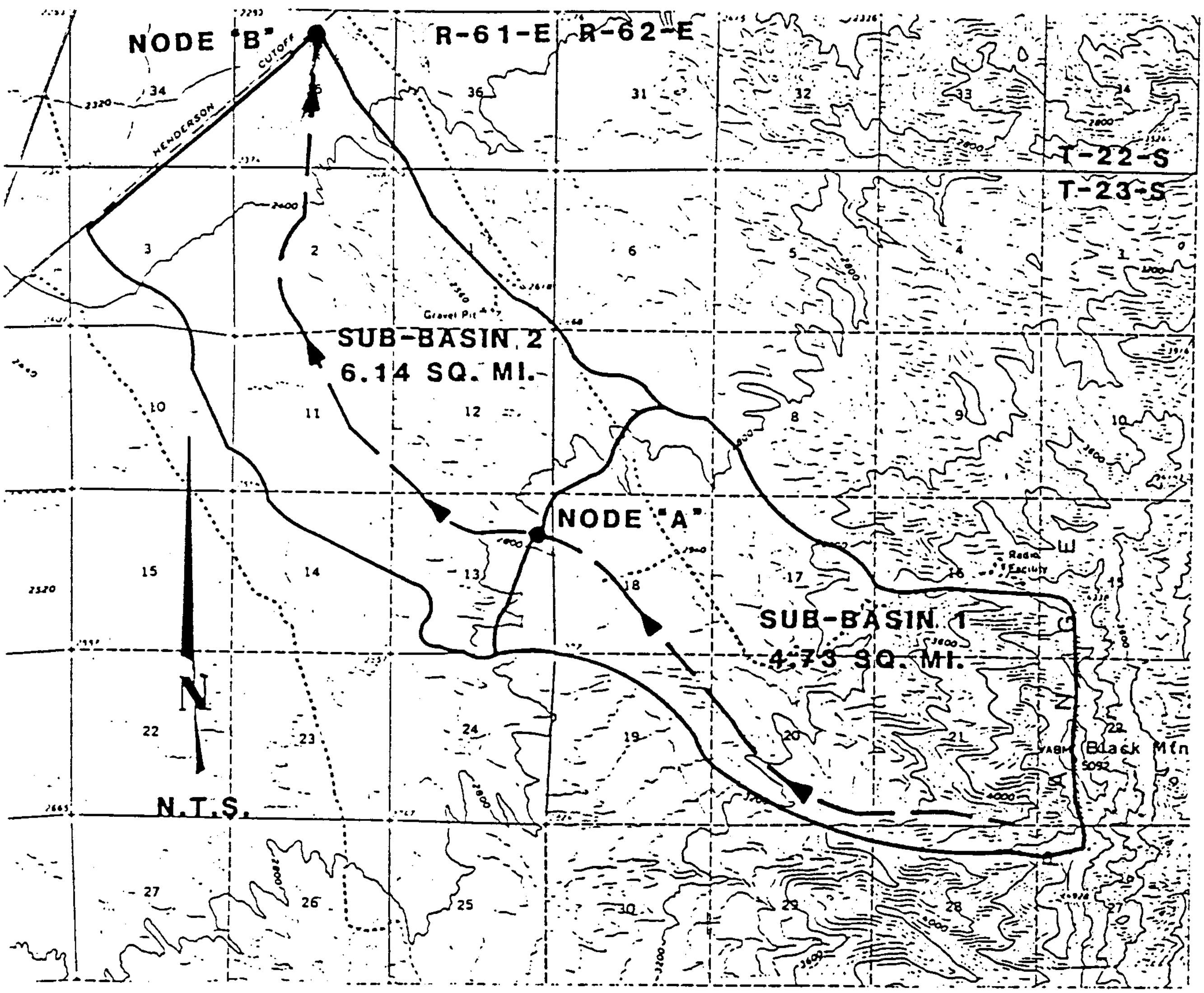
TIME-INTENSITY-FREQUENCY CURVES FOR McCARRAN AIRPORT RAINFALL AREA



NOTE: 1. Refer to Table 506 for tabulation of Time-Intensity-Frequency Values.

Revision	Date

HYPOTHETICAL BASIN FOR NON-URBAN, LARGE BASIN EXAMPLES



DESCRIPTION

The above basin is initially presented in Section 507.1 and will be used for non-urban, large basin examples in this MANUAL.

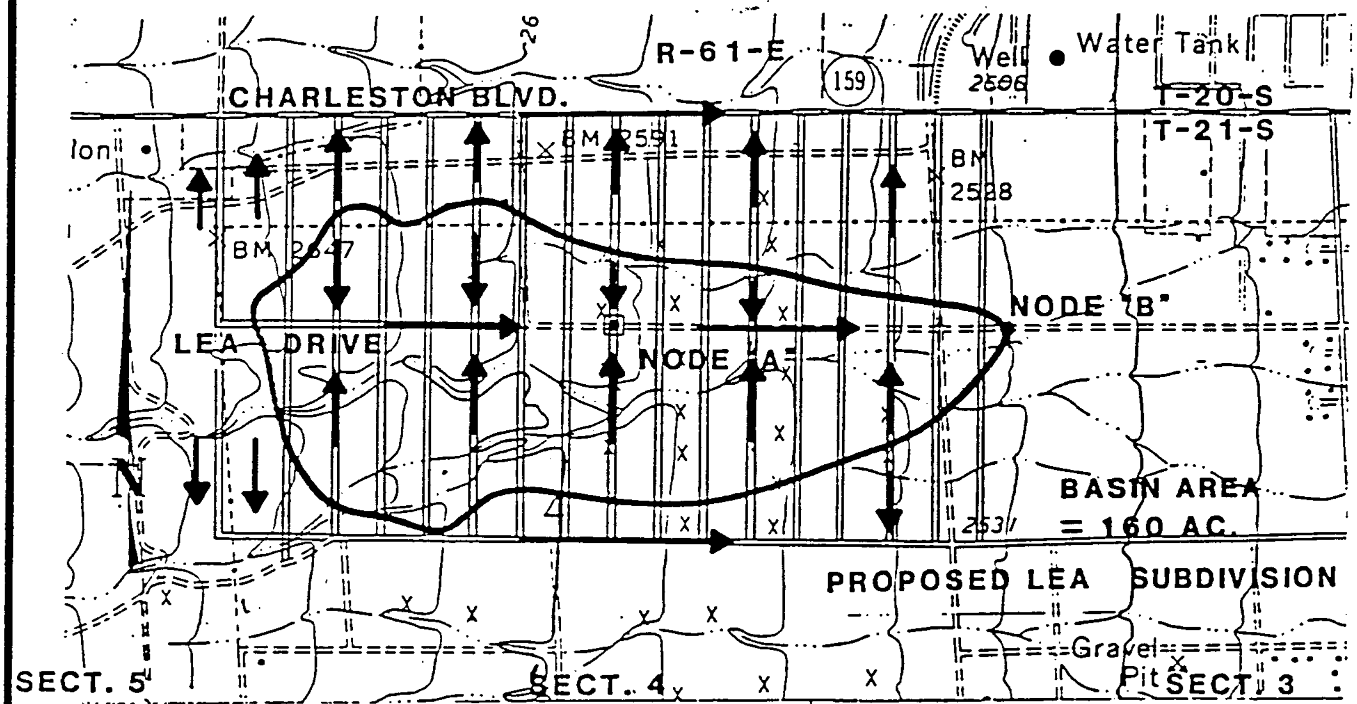
Revision	Date

WRC ENGINEERING

REFERENCE:

FIGURE 518

HYPOTHETICAL BASIN FOR URBAN, SMALL BASIN EXAMPLES



GENERAL DRAINAGE PATTERN

BASIN BOUNDARY

DESCRIPTION

The above basin is initially presented in Section 507.1 and will be used for urban, small basin examples in this MANUAL.

General drainage patterns for the proposed subdivision are shown above. Off-site flows west of the proposed subdivision are diverted north and south around the subdivision.

Although not discussed in the examples, capacities of facilities to accommodate the off-site flow diversions must be addressed.

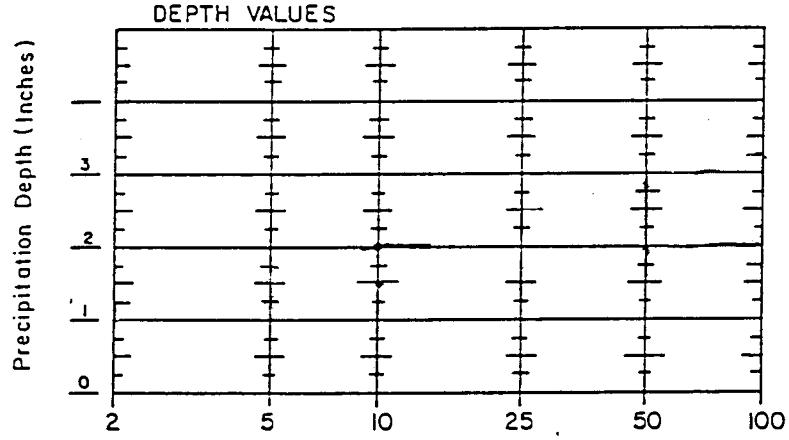
Revision	Date

PRECIPITATION DEPTH VERSUS RETURN PERIOD FOR EXAMPLE IN SECTION 507.3

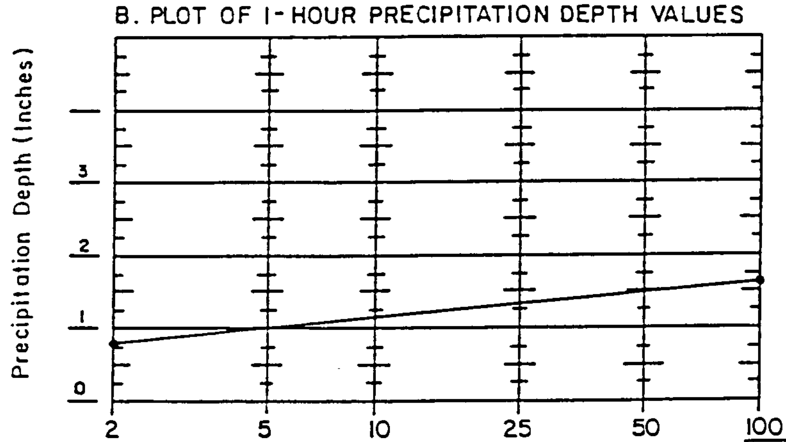
HYDROLOGIC CRITERIA AND DRAINAGE DESIGN MANUAL

PRECIPITATION DEPTH VERSUS RETURN PERIOD

A. PLOT OF 6-HOUR AND 24-HOUR PRECIPITATION DEPTH VALUES



B. PLOT OF 1-HOUR PRECIPITATION DEPTH VALUES



Revision	Date

REFERENCE: NOAA ATLAS 2, VOLUME VII NEVADA, 1973

STANDARD FORM 3

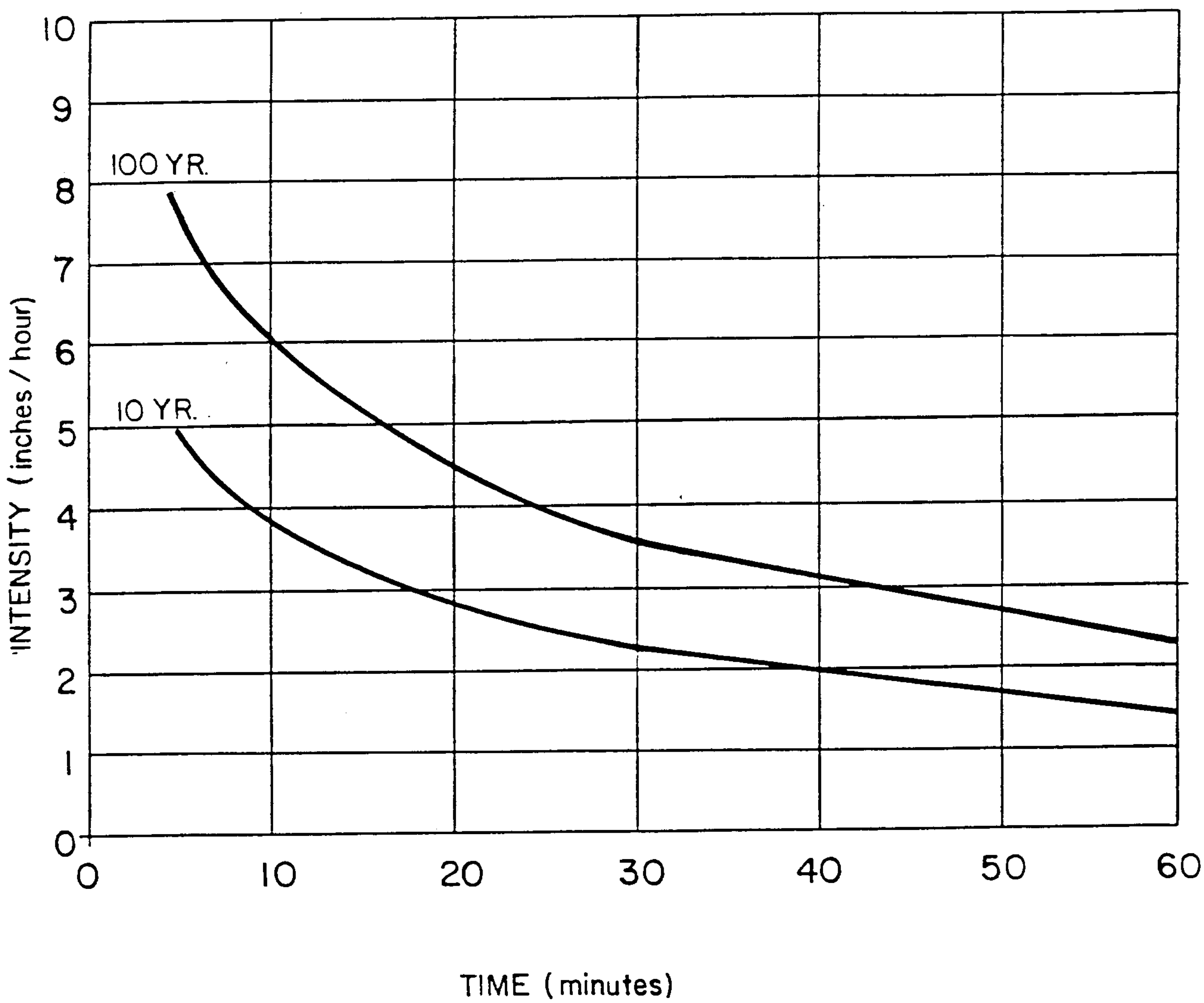
Revision	Date

WRC ENGINEERING

REFERENCE:

FIGURE 520

TIME-INTENSITY-FREQUENCY CURVE FOR EXAMPLE IN SECTION 507.3



Revision	Date

CLARK COUNTY REGIONAL FLOOD CONTROL DISTRICT
HYDROLOGIC CRITERIA AND DRAINAGE DESIGN MANUAL

SECTION 600
STORM RUNOFF

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609 HEC-1 RUN FOR EXAMPLE IN SECTION 611.3

610 HYPOTHETICAL URBAN BASIN FOR EXAMPLES IN SECTION 611.4 and 611.5

611 HEC-1 RUN FOR EXAMPLE IN SECTION 611.5

Section 600
Storm Runoff

601 INTRODUCTION

For the area within the jurisdiction of this MANUAL, four deterministic hydrological models can be used to predict storm runoff (POLICY Section 304.3). These models are the Rational Formula method, the SCS TR-55 method, the SCS Unit Hydrograph method and the Kinematic Wave method. The techniques for these methods are presented in this section. The Rational Formula method, the SCS TR-55 method, and the SCS Unit Hydrograph method may be employed without the use of computers. However, use of computers is required for the Kinematic Wave method. Also use ofcomputers is recommended for the SCS Unit Hydrograph method. For certain circumstances, where adequate recorded stream flow data are available and the drainage area is large (> 10 square miles), a statistical analysis may be required to predict the storm runoff peaks or for calibration of deterministic models (see Section 610).

601.1 Basin Characteristics

The basin characteristics needed for the subject runoff computation methods include the drainage area, the various flow path lengths, slopes, and characteristlcs (i.e., overland, grassed channel, gutter) and land use types. The drainage basin boundary and area can be determined from available topographic maps. A field investigation is recommended to verify drainage boundaries. The land use and flow path characteristics can be obtained from zoning maps, aerial photographs, field investigations, or detailed topographic maps.

602 TIME OF CONCENTRATION

The definition of the time of concentration, t_c, for the purpose of this MANUAL, is the time required for water to flow from the most remote part of the drainage area to the point under consideration. For the Rational Formula method, the time of concentration must be estimated so that the average rainfall rate for a corresponding duration can be determined from the rainfall intensity-duration-frequency curves. For the SCS TR-55 and SCS Unit Hydrograph methods, the time of concentration is used to determine the time-to-peak, t_p, of the unit hydrograph and subsequently, the peak runoff.

Typically, many different time of concentration equations may be used with the various runoff methods discussed in the following sections. However, all these methods have the same definition of the time of concentration. Therefore, to obtain consistent results between all the runoff methods, the time of concentration equations presented in this section shall be used for all small watershed (less than one square mile) runoff calculations. For large watershed calculations, see Section 606.3 for application of the basin lag equation.

For urban areas, the time of concentration consists of an initial time or overland flow time, t_i, plus the time of travel, t_t, in the storm sewer, paved gutter, roadside drainage ditch, or drainage channel. For non-urban areas, the time of concentration consists of an overland flow time, t_i, plus the time of travel in a combined form, such as a small swale, channel, or wash. The latter portion, t_t, of the time of concentration can be estimated from the hydraulic properties of the storm sewer, gutter, swale, ditch, or wash. Initial time, on the other hand, will vary with surface slope, depression storage, surface cover, antecedent rainfall, and infiltration capacity of the soil, as well as distance of surface flow. Thus, the time of concentration for both urban and non-urban areas shall be calculated as follows:

$$t_c = t_i + t_t \qquad (601)$$

In which t_c = Time of Concentration (min)

t_i = Initial, Inlet, or Overland Flow Time (min)

t_t = Travel Time in the Ditch, Channel, Gutter, Storm Sewer, etc. (min)

To aid in the computation of t_c, **Standard Form 4** has been developed to organize the computation. In all drainage studies, t_c calculations should be submitted using **Standard Form 4.**

The initial or overland flow time, t_i, may be calculated using the following equation or **Figure 601**:

$$t_i = 1.8\,(1.1 - K)\,L_o^{1/2} / S^{1/3} \qquad (602)$$

Where t_i = Initial or Overland Flow Time (min)

K = Flow Resistance Coefficient

L_o = Length of Overland Flow, (ft, 500-ft maximum)

S = Average Basin Slope (%)

Equation 602 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, TR-55 and SCS Unit Hydrograph methods using the appropriate flow resistance coefficient.

For the Rational Formula method, the 10-year runoff coefficient, C_{10}, presented in **Table 601** shall be used as the flow resistance coefficient, K. For the SCS TR-55 and SCS Unit Hydrograph methods, K shall be calculated using the following equation:

$$K = 0.0132\ CN - 0.39 \qquad (603)$$

This equation was developed by converting curve number, CN, factors to typical C_{10} runoff coefficients.

The overland flow length, L_o, is generally defined as the length of flow over which the flow characteristics appear as sheet flow or very shallow flow in grassed swales. Changes in land slope, surface characteristics, and small drainage ditches or gullies will tend to force the overland flow into a combined flow condition. Thus, the initial flow time would generally end at these locations.

For longer basin lengths, the initial or overland flow needs to be considered in combination with the travel time, t_t, which is calculated using the hydraulic properties of the swale, ditch, or channel. For preliminary work, the travel time can be estimated with the help of **Figure 602** (SCS, 1985) or the method described in Section 602.1.

The time of concentration is then the sum of the initial flow time, t_i and the travel time, t_t (**Equation 601**). The minimum t_c recommended for non-urban watersheds is 10 minutes.

602.1 Urbanized Basins

Overland flow in urbanized basins can occur from the back of the lot to the street, in parking lots, in greenbelt areas, or within park areas. It can be calculated using the procedure described in Section 602 except travel time, t_t, to the first design point or inlet is estimated using the "Paved Area (Sheet Flow) & Small Upland Gullies" line in **Figure 602**, or an estimated velocity based on Manning's equation, which was used to derive formulas for estimating two travel time velocities. The equation is as follows:

$$V = CS^{1/2} \qquad C = 1.49/n\ R^{2/3} \qquad R = A/P$$

Where:

V = Average Velocity (fps)

S = Average Slope (ft/ft)

n = Manning's Roughness Coefficient

R = Hydraulic Radius (ft)

P = Wetted Perimeter (ft)

A = Area of Flow (sq ft)

Two sets of coefficients were determined, one for developed conditions (Urbanized Areas) and the other for undeveloped (Non-Urbanized Areas) conditions. The following coefficients shall be used for developed areas (Urbanized Areas) For the first developed conditions velocity (V1), the appropriate coefficient (C = 20.2) represents street flow with n = 0.016 and a depth in the gutter of 0.3 feet. For the second developed conditions velocity (V_2), the appropriate coefficient (C=30.6) represents curb height street flow. As may be noted, V_2 is approximately equal to 1.5 times V_1. Typically, drainage areas are delineated such that the channelized flow considered in the calculations of travel times is within streets and other minor water courses and does not include significant lengths of major improved floodway and channels in the travel distance for the time of concentration.

The second set of coefficients shall be used in existing areas (Non-Urbanized Areas). For the first undeveloped conditions velocity (V1), the appropriate coefficient C = 14.8) represents wide channel flow with D = 0.25 feet and n = 0.04. For the second undeveloped conditions velocity (V_2), the appropriate coefficient C = 29.4) represents wide channel flow with D = 0.7 feet and n = 0.04.

The time of concentration for the first design point in an urbanized basin using this procedure should not exceed the time of concentration calculated using **Equation 604**, which was developed using rainfall/runoff data collected in urbanized regions (USDCM,1969).

$$t_c = L / 180 + 10 \qquad (604)$$

Where t_c = Time of Concentration at the First Design Point in an Urban Watershed (min)

L = Watershed Length (ft)

Equation 604 may result in a lesser time of concentration at the first design point and thus would govern in an urbanized watershed. For subsequent design points, the time of concentration is calculated by accumulating the travel times in downstream reaches. The minimum t_c recommended for urbanized areas is 5 minutes.

A common mistake in calculating t_c is to assume travel velocities (for t_t) that are too small. Another common error is to not analyze the portion of basin which would result in the longest computed time of concentration. This error is most often encountered in long basins, or a basin where the upper portion contains grassy park land and the lower developed urban land.

When studying a tract of land proposed for subdivision, the overland flow path should not necessarily be taken perpendicular to the contours since the land will be graded and swales will often intercept the natural contour and conduct the water to the streets, thus increasing the time of concentration.

603 PRECIPITATION LOSSES

Precipitation loss calculations (by hand or computer) are required for the SCS TR-55, SCS Unit Hydrograph, and the Kinematic Wave methods. The calculation methodology for precipitation losses within the CCRFCD is presented in the following section. For the Rational Formula method, the precipitation losses are not computed separately. Therefore, the following methodology does not apply to the Rational Formula method.

603.1 Introduction

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, local depressions in the ground surface, in cracks and crevices in parking lots for 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.

Two important factors should be noted about the precipitation loss computations to be used for the subject rainfall/runoff 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 TR-55 and SCS Unit Hydrograph methods is considered to be a subbasin average (uniformly distributed over an entire subbasin). For the precipitation loss component of the Kinematic Wave method, separate precipitation losses can be specified for each overland flow plane (if two are used). These losses are also assumed to be uniformly distributed over each overland flow plane.

In some instances, there are negligible precipitation losses for a portion of a subbasin. 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, and SCS Curve Number method to name a few. The SCS Curve Number method is recommended for the Clark County area because of lack of data to use other methods and the familiarity of the local consultants in using this method. 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 subbasin.

603.2 SCS Curve Number Method

The National Resources Conservation Service (NRCS), 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 IA. CN and IA are related to a total runoff depth for a storm by the following relationships:

$$Q = (P - La)^2 / ((P - IA) + S) \qquad (605)$$

$$S = (1{,}000 / CN) - 10 \qquad (606)$$

where Q = Accumulated Excess (in)

P = Accumulated Rainfall Depth (in)

IA = Initial Surface Moisture Storage Capacity (in)

S = Currently Available Soil Moisture Storage Deficit (in)

For the CCRFCD area, IA is calculated by using the following equation:

$$IA = 0.2\ S \qquad (607)$$

This relation is based on empirical evidence established by the NRCS, and is the default value in HEC-1 Program (HEC,1988).

Since the SCS method gives total excess for a storm, the incremental excess (the difference between rainfall and precipitation loss) 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.

603.2.1 CN Determination

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 field visits or from aerial photographs. The procedure for determining land use and treatment class are found in Chapter 8 of 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 ofthe 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 evapotranspiration 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 CCRFCD area, 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 602**. This table is reproduced from Table 2-2 in TR-55 (SCS, 1986).

There will be areas to which the values in **Table 602** 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 602**. A curve for each pervious CN can be developed to determine the composite CN for any density of impervious area. **Figure 603** has been developed assuming a CN of 98 for the impervious area. The curves in **Figure 603** can help in estimating the increase inrunoff 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 are not yet sufficient to validate widespread use of these methods. Therefore, the CN to be used in the ClarkCounty area shall be based on **Table 602** or **Figure 603** in this MANUAL. A CN computation example is included in Section 611.

The most common cover type for undeveloped areas in Clark County is "semiarid rangelands, desert shrub - poor condition." The following are CN values determined for this land use and cover condition for each of the four SCS hydrologic soil groups:

	Soil Group A	Soil Group B	Soil Group C	Soil Group D
Semiarid Rangelands - Desert Shrub/Poor Conditions	63	77	85	88

604 MODIFIED RATIONAL FORMULA METHOD

For drainage basins that are not complex and have small drainage areas, the design storm runoff may be analyzed using the Rational Formula method in accordance with Section 304.3. This method was introduced in 1889 and is still being used in many engineering offices in the United States. Even though this method has frequently come under academic criticism for its simplicity, no other practical drainage design method has evolved to a level of general acceptance by the practicing engineer. The Rational Formula method, when properly understood and applied, can produce satisfactory results for determining peak discharge. The Rational Formula method has been modified to match the results from HEC-1 in the Clark County area.

604.1 Methodology

The Modified Rational Formula method is based on the formula:

$$Q = KCIA \qquad (608)$$

Q is defined as the maximum rate of runoff in cubic feet per second (cfs) (actually Q has units of acre inches per hour, which is approximately equal to the units of cfs). C is a runoff coefficient and represents the runoff producing conditions of the subject land area (see Section 604.5). 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. K is a local adjustment factor for the Rational Method.

604.2 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.

604.3 Limitations on Methodology

The Rational Formula method adequately approximates the peak rate of runoff from a rainstorm in a given 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).

A local factor of 0.5 (K) has been developed to match the results from the Rational Method to those obtained using HEC-1.

A disadvantage of the Rational Formula method is that with typical design procedures one normally assumes that all of the design flow is collected at the design point and that there is no "carry over water" running overland to the next design point. However, this is not the fault of the Rational Formula method, but of the design procedure. The problem becomes one of routing the surface and subsurface hydrographs which have been separated by the storm sewer system. In general, this sophistication is not warranted and a conservative assumption is made that the entire routing occurs through the storm sewer system when the system is present.

604.4 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 having a duration equal to the time of concentration.

After the design storm frequency has been selected, a graph should be made showing rainfall intensity versus time. The procedure for obtaining the local data and preparing the graph is explained and illustrated in Section 507 of this MANUAL.

604.5 Runoff Coefficient

The runoff coefficient, C, represents the integrated effects of infiltration, evaporation, retention, flow routing, and interception, all which effect the time distribution and peak rate of runoff. Determination of the coefficient requires judgment and understanding on the part of the engineer. **Table 601** presents the recommended values of C for the various recurrence frequency storms. The values are presented for different surface characteristics as well as for different aggregate land uses.

A composite runoff coefficient is computed on the basis of the percentage of different types of surface in the drainage area. This procedure is often applied to typical "sample" blocks as a guide to selection of reasonable values of the coefficient for an entire area. Suggested coefficients with respect to surface type are also given in **Table 601** under the column labeled "Percent Impervious." Where land use features are known, a composite C analysis will result in more accurate results. The runoff coefficients in **Table 601** vary with recurrence frequency and therefore, further adjustments of the C factor are not needed.

604.6 Application of the Modified Rational Formula Method

The first step in applying the Modified 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 subbasins in 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.

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 flow occurs the water will be deep enough so that part of the water will overflow streetcrowns and flow into a new subbasin. An example of how to apply the Modified Rational Formula method is presented in Section 611.

604.7 Major Storm Analysis

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 Modified Rational Formula method. Normal application of the Modified Rational Formula method assumes that all of the runoff is collected by the storm sewer. For the minor storm design, the time of concentration is dependent upon the flow time in the sewer. However, during the major storm runoff, the sewers will probably be at capacity and could 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 the major storm runoff is complex. The simplified approach of using the minor storm time of concentration for all frequency analysis is acceptable for the CCRFCD area.

605 SCS TR-55 METHOD

The SCS TR-55 method was first developed and documented in January, 1975. Its purpose was to provide a simplified procedure for estimating runoff and peak discharges on small urban and urbanizing watersheds. The method was derived from typical hydrographs prepared by procedures outlined in Chapter 16 of the SCS's National Engineering Handbook 4 (SCS, 1985). The computations were made using the computerized SCS Hydrologic model TR-20 (SCS,1983). The method is similar to the SCS Unit Hydrograph method discussed herein in that an SCS CN is used to determine rainfall excess and the unit hydrograph theory is used to develop a runoff hydrograph. The method differs, however, from the SCS Unit Hydrograph method as follows:

1. Synthetic, 24-hour, regional design rainfall distributions are used.

2. Two peak runoff determination methods are available. The Graphical Peak Discharge method estimates only the runoff peak. The Tabular Hydrograph method produces a runoff hydrograph.

3. The Tabular Hydrograph method uses prerouted hydrographs from the specified sub-basins to produce the estimated runoff hydrograph.

The SCS TR-55 method may be used for runoff calculations in the CCRFCD in accordance with Section 304.3. If a runoff hydrograph is required, then the Tabular Hydrograph method must be used. If a more detailed and accurate analysis is required, then the HEC-1 computer program should be used. The reader is referred to the TR-55 document for a more detailed explanation of the subject method. Additional computation forms and worksheets are also provided in the TR-55 documentation.

605.1 Methodology

605.1.1 Graphical Peak Discharge Method

The Graphical Peak Discharge method determines the peak discharge, q_p, from a basin based on the following equation:

$$q_p = q_u\,A\,Q\,F_p \qquad (609)$$

Where q_p = Peak Discharge (cfs)

q_u = Unit Peak Discharge (cfs per sq mi per in of Runoff Excess)

A = Drainage Area (sq mi)

Q = Runoff (in)

F_p = Pond and Swamp Adjustment Factor

The unit peak discharge, q_u, is determined based on the composite CN, for the basin and the total rainfall, P. CN is determined as presented in Section 603. P is determined as presented in Section 500 for the specific frequency under investigation. With the selected CN and P values, an I_a/P ratio may be determined using the following equation:

$$I_a/P = ((200 / CN) - 2)/P \qquad (610)$$

Where I_a = Initial Abstraction (in)

If the computed I_a/P ratio is less than 0.10, use $I_a/P = 0.10$. If the computed I_a/P ratio is greater than 0.50, use $I_a/P = 0.50$.

Next, the time of concentration, t_c (in hours), for the basin must be computed as described in Section 602. Last, using the computed I_a/P ratio and t_c, q_u is determined directly from **Figure 604**.

The basin runoff, Q, is determined using **Equations 605** and **606** as presented in Section 603.2.

The pond and swamp adjustment factor, F_p, accounts for ponds and swamps not in the t_c flow path. This factor is typically not needed in the Clark County area.

605.1.2 Tabular Hydrograph Method

The Tabular Hydrograph method determines the runoff hydrograph at a given design point by combining prerouted subbasin hydrograph coordinates which are computed from the following equation:

$$q = q_t \, A \, Q \tag{611}$$

Where q = Prerouted Subbasin Hydrograph Coordinate (cfs) at Hydrograph Time t

q_t = Tabular Hydrograph Unit Discharge (cfs per sq mi per in of runoff excess)

A = Drainage Area (sq mi)

Q = Runoff (in)

The Tabular Hydrograph unit discharge, q_t, is determined using **Table 603** with computed I_a/P and t_c values. I_a/P is determined as discussed in Section 605.1.1. The t_c values are determined as discussed in Section 602. When using **Table 603**, the travel time is the total time of concentration at the selected design point. The t_c values in the table refer to the individual subbasin t_c's. The value of Q is determined from Section 603.2 and **Equation 605**.

Once the q's are determined for each subbasin, the final composite hydrograph is obtained by summing the subbasin q's for each time interval. **Standard Form 5** may be used to assist in the tabulation of the basic subbasin data and the composite hydrograph.

The Tabular Hydrograph method only computes a partial composite hydrograph for the times encompassing the expected maximum composite discharge. To estimate the complete composite hydrograph, prepare a table similar to **Standard Form 5** which encompasses all unit hydrograph times from 11 to 26 hours. Using the summed composite hydrograph, apply linear extrapolation to the first two and last two points of the partial hydrograph to obtain the slope of the missing rising and falling legs of the composite hydrograph.

605.2 Limitations on Methodology

The following limitations are applicable to the Graphical Peak Discharge method:

1. The watershed must be hydrologically homogeneous (describable by one CN). Land use, soils, and cover are distributed uniformly throughout the watershed.

2. The watershed may have only one mainstream or, if more than one, the branches must have nearly equal t_c's.

3. Accuracy of the peak discharge estimated by this method will be reduced if I_a/P values are used that are outside the range given in **Figure 604**. The limiting I_a/P values presented in Section 605.1.1 are recommended for use.

4. This method should be used only if the weighted CN is greater than 40.

5. When this method is used to develop estimates of peak discharge for both present and developed conditions of a watershed, use the same method for estimating t_c for both conditions.

The limitations to the Tabular Hydrograph method are stated in TR-55 (SCS, 1986). The user is encouraged to research this document before applying the Tabular Hydrograph method.

605.3 Basin/Subbasin Sizing

The SCS TR-55 method differs from the Rational Formula method, the SCS Unit Hydrograph method, and the Kinematic Wave method, in several areas including the use of a 24-hour storm as an inherent feature of the method. In order to obtain more consistent results between the SCS TR-55 method and the above listed methods, the following basin sizing guidelines should be followed:

1. The maximum basin or subbasin area should be approximately 20 acres.

2. For basin areas greater than 20 acres, either the Tabular Hydrograph method must be used or conservatively, the peak discharge rates as analyzed using the Graphical Peak Discharge method for each subbasin may be directly added to obtain the basin peak discharge rate.

606 SCS UNIT HYDROGRAPH METHOD

The SCS Unit Hydrograph method was developed for the 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. This method may be used for drainage areas within the CCRFCD area in accordance with Section 304.3.

606.1 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 subbasin for a given unit of time (or unit duration). The rainfall excess hydrographs are then transformed to a subbasin 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.

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 in Section 606.3.

606.2 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 unitgraphs each resulting from a single increment of excess rain of unit duration.

606.3 Lag Time

Input data for the SCS 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 1 sq mi) in the Clark County area, the lag time may be related to the time of concentration, t_c, by the following empirical relationship:

$$TLAG = 0.6\, t_c \qquad (612)$$

The t_c is computed as presented in Section 602.

For larger drainage basins (greater than 1 sq mi), the lag time (and t_c) 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:

$$TLAG = 20\, K_n\, (L\, L_c / S^{0.5})^{0.33} \qquad (613)$$

where K_n = Manning's Roughness Factor for the Basin Channels

L = Length of Longest Watercourse (mi)

L_c = Length Along Longest Watercourse Measured Upstream to a Point Opposite the Centroid of the Basin (mi)

S = Representative (Average) Slope of the Longest Watercourse (ft per mi)

This lag equation is based on the United States Bureau of Reclamation's (USBR) analysis of the above parameters for several drainage basins in the Southwest desert, Great Basin, and Colorado Plateau area (USBR, 1989). This equation was developed by converting the USBR's S-graph lag equation to a 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.

606.3.1 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 604**. These factors are based on roughness factor analysis by the USACE (1982) and USBR (1989) as compared to the typical watershed channels found in the Clark County area. The reader is referred to these documents for further discussion on selection of a proper roughness factor.

For partially developed basins, the K_n should be interpolated in relationship to the percent of each land use in the basin.

606.4 Unit Storm Duration

The minimum unit duration,)t, is dependent on the time of concentration of a given basin. If the basin is large (i.e., > 1 sq mi), a larger unit duration may be used. If the basin is small (i.e., < 1 sq mi) a smaller unit duration should be used. The unit duration,)t, should be $\leq$ 0.29 TLAG, where TLAG is the lag time. For the CCRFCD area the maximum unit storm duration should be 5 minutes unless conditions warrant otherwise.

606.5 Subbasin Sizing

The determination of the peak rate of runoff at a given design point is affected by the discretization of subbasins in the subject basin. Typically, the more discrete the analysis of a given basin (more subbasins), the larger the peak flow rate as compared to analysis of the basin with no subbasins. Therefore, in order to obtain more consistent results between different designers as well as between different runoff models (i.e., Rational Formula method, SCS TR-55 methods), the following guidelines are recommended for basin discretization:

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

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

607 KINEMATIC WAVE METHOD

In determining subbasin runoff by the Kinematic Wave method three conceptual elements are used: flowplanes, collector channels, and a main channel, **Figure 605**. The Kinematlc Wave technique transforms rainfall excess into subbasin outflow. Refer to HEC-1 Users' Manual (HEC, 1988) for details on development of the Kinematic Wave equations.

607.1 Basic Concepts

In the Kinematic Wave interpretation of the equations of motion, it is assumed that the bed slope and water surface slope are equal and acceleration effects are negligible. **Figure 605** shows relationship between flow elements.

The momentum equation then simplifies to:

$$S_f = S_o \tag{614}$$

where S_f is the friction slope and S_o is the channel bed slope. Thus flow at any point in the channel can be computed from Manning's formula:

$$Q = \frac{1.486}{n} R^{2/3} S^{1/2} \ A \tag{615}$$

where Q is flow, S is the channel bed slope, R is hydraulic radius, A is cross-sectional area, and n is Manning's resistance factor. **Equation 615** can be simplified to:

$$Q = \alpha A^m \tag{616}$$

where α and m are related to flow geometry and surface roughness.

Since the momentum equation has been reduced to a simple functional relation between area and discharge, the movement of a flood wave is described solely by the continuity equation:

$$\frac{\partial A}{\partial t} + \frac{\partial Q}{\partial x} = q \tag{617}$$

607.2 Solution Procedure

The users of this MANUAL should refer to HEC-1 Users' Manual for the solution procedure and parameters needed for Kinematic Wave method of runoff computation.

608 CHANNEL ROUTING OF HYDROGRAPHS

Whenever a large or a non-homogeneous basin is being investigated, the basin should be divided into smaller and more homogeneous subbasins. The storm hydrograph for each subbasin can then be calculated using the procedures described in Sections 605, 606, or 607. The user then must route and combine the individual subbasin hydrographs to develop a storm hydrograph for the entire watershed. There are several methods commonly used in channel routing which include:

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

The Muskingum, Kinematic Wave, and Muskingum-Cunge methods are recommended for use in the Clark County area. The Kinematic Wave method shall be used in well defined channels, and the Muskingum method shall be used in not so well defined channels. The Muskingum-Cunge method shall be used for defined channels that have cross sections that can be determined from points.

608.1 Muskingum Method

The Muskingum method provides for some of the effects of channel storage and, as a result, the storm hydrograph shape is modified in translation along a channel reach.

The basic equation for the Muskingum method as described by HEC-1 (HEC, 1988) is as follows:

$$O_2 = (C_1 - C_2)I_1 + (1 - C_1)O_1 + C_2I_2 \qquad (618)$$

In which O_2 = Outflow from the reach at the end of the unit time increment (beginning of the next time increment).

O_1 = Outflow from the reach at the beginning of the time increment.

I_1 = Inflow into the reach at the beginning of the time increment.

I_2 = Inflow into the reach at the end of the time increment.

C_1, C_2 = Coefficients found using **Equations 619** and **620**, respectively.

$$C_1 = \frac{2\,)t}{2K\,(1 - X) +)t} \qquad (619)$$

$$C_2 = \frac{)t - 2KX}{2K\,(1 - X) +)t} \qquad (620)$$

In which $K = L/(3{,}600V)$ (621)

K = Muskingum storage time constant in hr.

L = Channel reach length in ft.

V = Translation velocity in fps.

$)t$ = Unit time increment in hr.

X = Muskingum weighting factor.

The velocity used in **Equation 621**, shall be the wave velocity, which can be estimated for various channel shapes as a function of average velocity, V, for steady uniform flow, using Manning's equation. The approximate wave velocities for different channel shapes are provided below:

Channel Shape	Wave Velocity
Wide Rectangular	5/8 V (V from Manning's equation)
Triangular	4/3 V (V from Manning's equation)
Wide Parabolic	11/9 V (V from Manning's equation)

The Muskingum routing parameters requested in the HEC-1 program and their record locations are as follows:

PARAMETER	HEC-1 VARIABLE	RECORD
Weighting Factor (X)	X	RM-Field 3
Storage Constant (K)	AMSKK	RM-Field 2
Time Steps	NSTPS	RM-Field 1

The weighting factor (X) in the Muskingum routing method accounts for the peak flow reduction caused by channel routing. The weighting factor generally varies from 0.0 to 0.5 with 0.0 representing a reservoir type peak reduction and 0.5 representing no peak reduction. Values of this weighting factor in the Clark County area should range from 0.15 for channel reaches with some storage in overbank but few obstructions such as would be found on alluvial fans, to 0.10 for inadequate channels in developed basins where overbank flows would encounter severe obstruction and consequently be significantly attenuated. The reader is referred to the USBR Flood Hydrology Manual (USBR, 1989) for further discussion on the selection of an appropriate Muskingum weighing factor (X).

The storage constant (K) in the Muskingum routing method accounts for the peak flow translation along a channel reach. This constant is therefore directly related to the reach length and the mean channel flow velocity as shown in **Equation 621.** An estimate of the mean channel flow velocity may be obtained using Manning's formula with the hydraullc radius estimated as being equal to the flow depth. The flow depth (and thus channel flow velocity) is estimated based on the channel cross-sectional shape and the design discharge for the selected flood frequency.

The routing procedure may be repeated for several subreaches (designated as NSTPS) so the total travel time through the reach is equal to K. To ensure the method's computational stability and the accuracy of computed hydrograph, the routing reach should be chosen so that

$$\frac{1}{2(1 - X)} \leq \frac{K}{NSTPS(\Delta T)} \leq \frac{1}{2X} \tag{622}$$

Where Δt is the time increment in hours.

608.2 Kinematic Wave Method

Kinematic Wave routing was described as part of the runoff determination in Section 607. The channel routing computation can be utilized independently of the other elements of the subbasin runoff. In this case, the upstream inflow is routed through a reach (independent of lateral inflows) using the previously described numerical methods. The Kinematic Wave method in HEC-1 does not allow for explicit separation of main channel and overbank areas. Theoretically, a flood wave routed by the Kinematic Wave technique through the channel section is translated, but does not attenuate (although a degree of attenuation is introduced by the finite difference solution). Consequently, the Kinematic Wave routing technique is most appropriate in channels where flood wave attenuation is not significant, as is typically the case in urban areas. Otherwise, flood wave attenuation can be modeled empirically by using the Muskingum method or other applicable storage routing techniques.

608.3 Muskingum-Cunge Method

The Muskingum-Cunge routing method is similar to the Muskingum method, but is a physically based method whose parameters are determined from actual channel characteristics. Because it does not require calibration to streamflow data, it is suited for use in ungaged watersheds. One limitation to the use of Muskingum-Cunge routing is that this method does not account for backwater and storage in the channel.

Muskingum-Cunge routing is based on wave diffusion theory and is non-linear in nature. In Muskingum-Cunge routing, the amount of diffusion is matched to physical diffusion determined using physical channel characteristics. This is compared to Muskingum routing which uses the x parameter to control diffusion without any relation to physical channel characteristics. Additional detail about the theory behind the Muskingum-Cunge routing method is presented in the HEC-1 User's Manual (HEC, 1990).

Data required for use with HEC-1 include:

- representative channel cross section,
- reach length,
- Manning's roughness coefficients for main channel and overbanks, and
- channel bed slope.

These data are input into HEC-1 using RD, RX, and RY records. The representative channel cross sections are not limited to the standard prismatic shapes required for kinematic wave routing so eight-point cross sections can be used to define the channel and overbanks as with normal-depth storage routing.

The results obtained using Muskingum-Cunge routing in HEC-1 should be checked for reasonability. The increments of time and distance selected internally by HEC-1 and used in the finite-difference computation can affect the accuracy of the results.

609 RESERVOIR ROUTING OF HYDROGRAPHS

Storage as found in an enlargement of a river or drainage channel and storage in reservoirs may modify the shape of the flood hydrograph. If the reservoir does not have gates, the discharge (D) takes place over an uncontrolled weir or through an uncontrolled orifice in such a way that D is a function of the reservoir level.

Storm runoff detention is required for some new development (POLICY Section 303.7) and therefore detention reservoirs will be required (see Section 1200). In some instances, the sizing of the detention storage will be based upon hydrograph storage routing techniques rather than direct calculation of volume and discharge requirements. The methodology for manual computation of reservoir routing is presented in this section. Thismethod is computerized and is part of the HEC-1 program. The input requirements are explained in the HEC-1 Users' Manual.

609.1 Modified Puls Method

The procedure for the original Puls Method was developed in 1928 by L.G. Puls of the United States Army Corps of Engineers (USACE). The method was modified in 1949 by the USBR simplifying the computational and graphic requirements. The 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 <u>Hydrology for Engineers</u> (LINSLEY,1975).

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

$$(I-D)t = \Delta S \qquad (623)$$

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:

$$(I_1 + I_2)\, t / 2 - (D_1 + D_2)\, t / 2 = S_2 - S_1 \qquad (624)$$

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:

$$I_1 + I_2 + (2S_1 / t - D_1) = (2S_2 / t + D_2) \qquad (625)$$

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.

610 STATISTICAL ANALYSIS

For basins larger than 10 square miles, the preferred method to compute flood flows is generally to use actual records of discharges which have been recorded by gaged streams. The reliability of the statistical or regional approach is generally better than the Rational Formula method, Rainfall-Runoff models, or other deterministic model, provided the period of record is sufficiently long (i.e., 20 years or greater).

Before proceeding with a statistical analysis, the analyst shall contact the CCRFCD to obtain applicable data and criteria for evaluation.

In urban hydrology, the preferred statistical approach is limited (1) by the almost total lack of adequate runoff records in urban areas, (2) by the effects of rapid urbanization, and (3) study areas having satisfactory gaging periods usually have records which represent undeveloped basin conditions. Once urbanization occurs, the records representing natural conditions no longer apply to future conditions. Thus, use of the deterministic methods allowed in the CCRFCD area will generally be required for urban or urbanizing areas.

The statistical analysis has the greatest applicability to natural streams where the basins will remain in a natural state. Such streams include those with large basins where the urbanization effect on runoff will be negligible, and on small streams where the basin primarily consists of undevelopable land or land comprising greenbelt areas.

In the statistical approach to determining the size of flood peaks, the logic involved is that nature over a perlod of years has defined a flood magnitude-frequency relationship that can be derived by study of actual occurrences. A period of record of a particular basin where the floods have been measured and recorded is considered to be a representative period. Floods that occurred during the period can be assumed to occur in a similar future period, that is, the period may be expected to repeat itself.

The purpose of statistical analysis to use the recorded runoff events for a given period of record as a means of extrapolating to a longer period of time. For a 25 year period, the largest record flood is generally considered to have a recurrence interval of about 25 years. At the end of this 25 year period, because the period can be assumed to repeat itself, one could expect the largest flood of record to be equaled or exceeded once more during the next 25 years. For any given year the probability of a flood of any given frequency happening in that year is the same as the probability of it happening in any other year. Thus the 100-year flood has a 1 percent chance of being equaled or exceeded in any given year.

The statistical procedure acceptable for use in the CCRFCD area is the one described by the Interagency Advisory Committee on Water Data (IAC, 1982) that utilizes the Log Pearson Type lll distribution. Any independent statistical analysis of records in the Clark County area should follow the procedure outlined by the IAC, 1982.

611 EXAMPLE APPLICATIONS

611.1 Example: Time of Concentration (Urban)

Problem: Utilize the information in **Figure 606** to determine the Time of Concentration, t_c, for Subbasin G at Node "C" for use in the Rational Formula method:

Solution:

Step 1: Determine the Flow Resistance Coefficient, K:

$K = C_{10}$ for Rational Formula method

From **Table 601**: For Single Family Residential Areas

(1/3 Acre Lots) $C_{10} = 0.50$

For Commercial Areas $C_{10} = 0.70$

Therefore, a composite C_{10} for 6.3 residential acres and 2 commercial acres is:

$$\text{Composite } C_{10} = \frac{(0.50)(6.3) + (0.70)(2)}{8.3} = 0.55$$

Step 2: Initial Overland Flow (t_i):

Assuming shallow sheet flow across the lot with an average lot depth of 150 ft

$$t_i = 1.8\,(1.1 - K)\,L^{1/2} / S^{1/3} \qquad (602)$$

Using **Figure 601** with $K = C_{10} = 0.55$, L = 150 ft, S = 2.5%

$t_i = 8.9$ min

Step 3: Travel Time (t_t)

From **Figure 602** the average travel velocity for a "Paved Area" is:

V = 3.2 fps @ S = 2.5%

Then $t_t = L / 60V$, L = 1,200 ft

$t_t = 1{,}200 / 60\,(3.2) = 6.3$ min

Or using the other method from Section 602.1 (Urbanized Basins), the average travel time is calculated as follows:

$V = CS^{1/2}$ $C = 1.49/n\ R^{2/3}$ $R = A/P$

$C = 20.2$ Assume gutter depth one half full

$S = 2.5\%$ Given above

$V = 20.2*(0.025)^{½}$

$V = 3.2$ fps (This is the same velocity found from **Figure 602**).

Step 4: Calculate the time of concentration using **Equations 601** and **604**. Select the smaller time of the two as the time of concentration at Node "C."

$t_c = t_i + t_t$ (601)

$t_c = 8.9 + 6.3 = 15.2$ min

or

$t_c = L / 180 + 10$, $L = 1{,}200 + 150 = 1{,}350$ ft (604)

$t_c = 1{,}350 / 180 + 10 = 17.5$ min

Since **Equation 601** gives the smaller time of concentration, it controls. Thus, at Node "C" use $t_c = 15.2$ min.

Step 5: If multiple subbasins exist, continue the time of concentration calculations in the downstream direction. The flow calculated at each design point is then 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.

Figure 608 shows the use of **Standard Form 4** for the Time of Concentration Computations in the Example.

Application: Rational Formula Method
SCS TR-55 Method
SCS Unit Hydrograph Method

611.2 Example: Modified Rational Formula Method

Problem: Utilize the information in **Figure 606**, to determine the peak flow from subbasin G at Node "C" for the 100-year storm.

Solution:

Step 1: Determine 100-year Runoff Coefficient (C_{100}):

From **Table 601**: For Single Family Residential: $C_{100} = 0.60$

For Commercial Areas: $C_{100} = 0.80$

Therefore a composite C for 6.3 residential acres and 2 commercial acres is:

$$\frac{(0.60)(6.3) + (0.80)(2)}{8.3} = 0.65$$

Step 2: Determine Rainfall Intensity (I_{100})

From Example 611.1, T_c = 15.2 min

The project is located in Section 4, T21S, R61 E.

Referring to **Figure 513**, McCarran Airport rainfall area data should be utilized. Therefore, from **Figure 517** for t_c = 15.2 min.

I_{100} = 5.4 in / hr

Step 3: Determine Peak Flow (Q_{100})

K = 0.5 Local Adjustment Factor

Q_{100} = KCIA

(0.5) (0.65) (5.4) (8.3) = 15 cfs at Node “C”

611.3 Example: SCS Unit Hydrograph Method

Problem: Utilizing the example basin in **Figure 607**, determine the 100-year, 6-hour runoff hydrograph at Node “B.”

Solution:

Step 1: Determine storm distribution and precipitation for the 100-year, 6-hour storm:

From Example 507.2:

Use SDN = 4 and Adjusted Precipitation = 2.51 in

Step 2: Determine SCS Curve Number (CN):

From **Table 602** (4 of 4) for desert shrub rangeland in poor condition (ground cover < 30%) and soil group D:

CN = 88 for Subbasins 1 and 2

Step 3: Compute basin lag, TLAG, for Subbasins 1 and 2.

a) Determine roughness Factor, K_n, for both basins from **Table 604**:

For Subbasin 1: Use $K_n = 0.045$

For Subbasin 2: Use $K_n = 0.035$

b) Determine Basin Length, L, for both basins:

For Subbasin 1: L = 18,000 ft - 3.41 mi

For Subbasin 2: L = 18,500 ft - 3.50 mi

c) Determine Centroidal Length, L_c, for both basins:

For Subbasin 1: L_c = 8,500 ft = 1.61 mi

For Subbasin 2: L_c = 10,600 ft = 2.01 mi

d) Determine Average Basin Slope, S, for both basins:

For Subbasin 1: S = 5.0% or 264 ft / mi

For Subbasin 2: S = 3.0% or 158 ft / mi

$$TLAG = 20\ K_n\ (LL_c / S^{0.5})^{0.33} \tag{613}$$

For Subbasin 1: $TLAG = 20(0.045)[(3.41)(1.61)/(264)^{0.5}]^{0.33}$

$= 0.63$ hr

For Subbasin 2: $TLAG = 20(0.035)[(3.50)(2.01)/(158)^{0.5}]^{0.33}$
$= 0.59$ hr

Step 4: Determine Muskingum Channel Routing Parameters for routing flow from Node "A" to "B":

Weighting Factor (X): Assume to be 0.13 for this example.

Time Steps (NSTPS): Assume 2 Subreaches for this example.

Storage Constant (K): L = 18,500 ft

$$K = L / 3600V \qquad (621)$$

Estimate Velocity (V) using Mannings Formula:

$V = \frac{1.49}{n} R^{2/3} S^{1/2}$, Using $n = 0.03$

$S = 3\%$

$R = 2$ (assumed full bank flow depth)

$$V = \frac{1.49}{0.03} (2)^{2/3} (0.03)^{1/2} = 13.6 \text{ fps}$$

Then, K = 18,500/3,600 (13.6) = 0.38 hr

Check computational stability using a ΔT 5 of a min time interval:

$$\Delta T = 5 \text{ min} \times \frac{1 \text{ HR}}{60 \text{ min}} = 0.083 \text{ HR}$$

$$\frac{1}{2(1 - X)} \leq \frac{K}{NSTPS * \Delta T} \leq \frac{1}{2X} \qquad (618)$$

$$\frac{1}{2(1 - 0.13)} \leq \frac{0.38}{2 \times 0.083} \leq \frac{1}{2\,(0.13)}$$

$$0.57 \leq 2.29 \leq 3.85$$

Therefore, number of steps (NSTPS) is okay.

Step 5: Utilize HEC-1 Program:

The subbasin characteristics, 100-year storm data and routing parameters for Muskingum routing are input to HEC-1 to determine the 100-year discharge at Nodes "A" and "B." HEC-1 input and partial output are presented in **Figure 609**. The peak 100-year flow at Node “B” is 4,979 cfs.

611.4 Example: SCS TR-55 Graphical Peak Flow Method

Problem: Given the urban basin in **Figure 610** determine the 10-year peak flow at Node "A."

Solution:

Step 1: Determine Time of Concentration (t_c):

Initial Overland Flow (t_i):

Utilizing the results from Example 611.1 (Steps 1 and 2):

t_i = 8.9 min with K = 0.55

L = 150 ft

S = 2.5%

(Note: The actual K factor for the subject basin is higher than for the basin in Example 611.1 but is used here for illustrative purposes)

Travel Time (t_t):

From **Figure 610**, the longest travel length is L = 1,700 + 1,000 = 2,700 ft.

From **Figure 602**, the average travel velocity for a "Paved Area" is:

V = 3.2 fps at S = 2.5%

Then t_t = L / 60V

= 2,700 / 60 (3.2), 14.1 min

Calculate the time of concentration using **Equations 601** and **604.** Select the smaller time of the two as the time of concentration at Node "A."

$$t_c = t_i + t_t \tag{601}$$

$$t_c = 8.9 + 14.1 = 23.0 \text{ min}$$

or

t_c = L / 180 + 10, L = 2,700 + 150 = 2,850 ft (604)

t_c = 2,850 / 180 + 10 = 25.8 min

Therefore, use a time of concentration = 23.0 min = 0.38 hr.

Step 2: Determine a composite CN for the basin area.

From **Figure 610**, use the data given and compute the runoff CN utilizing CN values in **Table 602**.

	Hydrologic Soil Group					
Land Use	B			C		
	Pct.	CN	Product	Pct.	CN	Product
Residential (30 pct. impervious)	20	72	1,440	20	81	1,620
Residential (65 pct. impervious)	6	85	510	6	90	540
Roads w/ open ditches	4	89	356	4	92	368
Roads w/ curbs and sewers	5	98	490	5	98	490
Open land:						
Fair cover	4	69	276	4	79	316
Good cover	4	61	244	4	74	296
Parking lots, plazas, etc.	7	98	686	7	98	686
	50		4,002	50		4,316

Thus,

Weighted CN = (4,002 + 4,316) / 100 = 83

Step 3: Determine the Storm Distribution and Precipitation (P):

For Clark County, the storm distribution utilizes TR-55 Type II Storm, **Figure 604**.

From **Figure 509** read the point precipitation value for the 10-year, 24-hour storm:

P_{10} = 2.0 in

Step 4: Determine Unit Discharge (q_u):

Compute I_a/P:

$$I_a/P = (200 / CN - 2)/P \quad (610)$$

$$= (200 / 83-2)/2$$

$$= 0.20$$

From **Figure 604** q_u, = 570 csm/in (t_c = 0.38 hr)

Step 5: Compute Runoff Depth (Q) from **Equations 605**, **606**, and **607**:

$$Q = \frac{[P - 0.2 (1{,}000 / CN - 10)]^2}{P + 0.8 (1{,}000 / CN - 10)}$$

$$= \frac{[2.0 - 0.2 (1{,}000 / 83 - 10)]^2}{2.0 + 0.8 (1{,}000 / 83 - 10)} = 0.695 \text{ in}$$

Step 6: Compute Peak Discharge (q_p) with Area (A) = 0.125 sq mi and

$$F_p = 1.0$$

$$q_p = q_u\, A\, Q\, F_p \quad (609)$$

$$= (570)\,(0.125)\,(0.695)\,(1.0)$$

= 50 cfs (Peak Flow at Node "A")

611.5 Example: Kinematic Wave Method

Problem: Given the urban basin in **Figure 610**, determine the 100-year peak flow at Nodes "A" and "B." Assume 100-year flows would be adequately conveyed through a main storm sewer line from Node "A" to "B."

Solution:

Step 1: Determine storm distribution and precipitation (P) for the 100-year, 6-hour storm:

The proposed subdivision is located within the designated McCarran Airport rainfall area. Therefore, from **Table 505**: 100-year Depth = 2.77 in.

From **Table 502** for an area of 160 acres (0.25 sq mi) the depth-area reduction factor is 0.99. P_{100} = 0.99 x 2.77 = 2.74 in.

From **Figure 515** for an area of 160 acres (0.25 sq mi) use storm distribution number SDN 3 as given in **Table 503**.

Step 2: Determine SCS CN for Overland Flow:

Using the composite CN calculation in Example 611.4, the breakdown of the CN value and the accompanying percent of the total basin with that CN value is approximately as follows:

Pervious Areas:	CN = 69 for 52% of the basin
Impervious Areas:	CN = 98 for 48% of the basin

Step 3: Kinematic Wave and Kinematic Routing Parameters:

From Detail A of **Figure 610** the Overland Flow

Lengths are:	Pervious L_1	=	150 ft
	Impervious L_2	=	50 ft

Typical overland flow resistance factors are presented in **Table 605** (HEC, 1990). The overland effective roughness parameters are:

Pervious N_1	=	0.25 (Lawns)
Impervious N_2	=	0.10

Equivalent street conveyance for the reaches in Subbasin 1 is assumed to be triangular with 50:1 side slopes and Manning's n of 0.015. The average collector channel length is 900 feet and the main channel length is 2,000 feet. The typical area contributing to the collector channel is about 5 acres.

Conveyance through Subbasin 2 is by storm sewer with sufficient capacity for 100-year flows. Assume 48-in diameter circular sewer and Manning's n = 0.013. From **Figure 610**, the conveyance length is 2,550 feet.

Equivalent street conveyance for the reaches in Subbasin 2 is assumed to be triangular with 50:1 side slopes and a Manning's n = 0.013. The runoff from this basin is assumed not to be collected by the storm sewer system until Node "B." The average collector channel length is 600 feet and the main channel length is 2,200 feet. The typical area contributing to the collector channel is about 5 acres.

Step 4: Utilize HEC-1 Program:

The subbasin characteristics, 100-year storm data and routing parameters for Kinematic routing are input to HEC-1 to determine the 100-year discharge at Nodes "A" and "B." The HEC-1 input and partial output are presented in **Figure 611.** The peak 100-year flow at Node "A" is 166 cfs and at Node "B" is 318 cfs.

611.6 Example: Muskingum-Cunge Routing Wave Method

Problem: Given the urban basin in Example 611.5. For a given 100-year peak flow of 3,106 cfs, assume the 100-year flow would be adequately conveyed through a 100-foot wide street to the next downstream design point.

Solution:

Step 1: Determine street slope S_0 = 0.01 routing reach length Δx = 100 ft, main channel length is 2,000 ft, and time interval, Δt = 3 seconds.

Step 2: The equation used for routing by solving for the unknown flow rate is:

$$Q_{i+1}^{t+1} = C_0 Q_i^{t+1} + C_1 Q_i^t + C_2 Q_{i+1}^t$$

Where

$$C_0 = \frac{\Delta t/k - 2x}{2(1-x) + \Delta t/k}$$

$$C_1 = \frac{\Delta t/k + 2x}{2(1-x) + \Delta t/k}$$

$$C_2 = \frac{2(1-x) - C\,\Delta t/\Delta x}{2(1-x) + \Delta t/k}$$

Because $K = \Delta x / C$

C, moving velocity for wide rectangular channel, is $\frac{5}{3}v$

From the inflow, the peak rate of 3,106 cfs gives:

$$q_0 = \frac{Q}{T}$$

$$= \frac{3{,}106}{100}$$

$$= 31.06\ \text{cfs}/\text{ft}$$

$$V = \frac{Q}{A}$$

$$= \frac{1.49}{n} R^{2/3} S_0^{1/2}$$

$$= 14.78\ \text{fps}$$

$$C = \frac{5}{3}v$$

$$= 24.63\ fps$$

Step 3: The value of x for use in Cunge's formulation is:

$$x = \frac{1}{2}\left(1 - \frac{q_0}{S_0 C \Delta x}\right)$$

$$= \frac{1}{2}\left(1 - \frac{6.36}{0.01 \times 24.63 \times 100}\right)$$

$$= 0.37$$

$$K = \frac{\Delta K}{C}$$

$$= \frac{100}{24.63}$$

$$= 4.06\,\text{sec}$$

and $C_0 = 0.0$

$C_1 = 0.74$

$C_2 = 0.26$

Step 4: Apply hydrograph from Section 611.5

The routing for a portion of the hydrograph is as follows:

t	Distance (n !) x)			
0	0 ft	100 ft	200 ft	300 ft
1	0	0	0	0
2	0	0	0	0
3	0	0	0	0
4	3	2.22	1.64	1.22
5	13	10.20	7.97	6.22
6	25	21.15	17.72	14.73
7	33	29.92	26.75	23.62
8	33	32.20	30.78	28.92
9	25	26.87	27.89	28.16
10	16	18.83	21.18	23.00
11	10	12.29	14.61	16.79
12	6	7.64	9.45	11.36
13	4	4.95	6.12	7.48
14	2	2.77	3.64	4.64

t	Distance (n !) x)			
0	0 ft	100 ft	200 ft	300 ft
15	2	2.20	2.57	3.11
16	3	2.79	2.73	2.83
17	5	4.43	3.99	3.69
18	8	7.07	6.27	5.60
19	14	12.20	10.66	9.34
20	21	18.71	16.62	14.73
21	23	21.89	20.52	19.01
22	22	21.97	21.59	20.92
23	19	19.77	20.25	20.42
24	14	15.50	16.73	17.69
25	10	11.43	12.81	14.08
26	7	8.15	9.36	10.59

RATIONAL FORMULA METHOD RUNOFF COEFFICIENTS AND AVERAGE PERCENT IMPERVIOUS AREA

LAND USE OR SURFACE CHARACTERISTICS	AVERAGE PERCENT IMPERVIOUS AREA	RUNOFF COEFFICIENTS 10-YEAR		RUNOFF COEFFICIENTS 100-YEAR	
		GRASS[1]	DESERT[2]	GRASS[1]	DESERT[2]
Business and Commercial:					
Downtown Areas	95	.88	.88	.89	.89
Neighborhood Areas	70	.70	.75	.80	.83
Residential (Average Lot Size):					
1/8 Acre or less (Multi-Unit)	65	.68	.73	.78	.80
1/4 Acre	38	.55	.62	.65	.74
1/3 Acre	30	.50	.57	.60	.70
1/2 Acre	25	.45	.53	.55	.67
1 Acre	20	.40	.49	.50	.64
2 Acre	12	.35	.45	.40	.60
Industrial:	72	.72	.76	.82	.84
Open Space: (Lawns, Parks, Golf Courses)	5	.10	-	.30	-
Undeveloped Areas: (Natural Vegetation)	0	-	.25	-	.50
Streets and Roads:					
Paved	100	.90		.93	
Gravel	20	.40		.50	
Drives and Walks:	95	.88		.89	
Roofs:	90	.85		.87	

Notes:

1 Grass - Grassed Landscaping or Irrigated Vegetation
2 Desert - Desert Landscaping or Natural Vegetation

Revision	Date

WRC ENGINEERING

REFERENCE: USDCM, DRCOG, 1969 (with modifications)

TABLE 601

RUNOFF CURVE NUMBERS
(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:					
See Table 602A					
Developing urban areas					
Newly graded areas (pervious areas only, no vegetation)[5]		77	86	91	94

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 603.

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 Figure 603 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 603 based on the degree of development impervious area percentage) and the CN's for the newly graded pervious areas.

Revision	Date

RUNOFF CURVE NUMBERS
(CULTIVATED AGRICULTURE 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]Hydrologic condition is based on combination of 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 in rotations, (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.

Revision	Date

REFERENCE: SCS TR-55, USDA, June 1986.

RUNOFF CURVE NUMBERS
(AGRICULTURE LANDS[1])

Cover description		Curve numbers for hydrologic soil group—			
Cover type	Hydrologic condition	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	[4]30	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	[4]30	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.

Revision	Date

RUNOFF CURVE NUMBERS
(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 $I_a = 0.2S$.

[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.

Revision	Date

RUNOFF CURVE NUMBERS - RESIDENTIAL DISTRICTS

Average Lot Size or Usage[1]	Percent Impervious[2]	Curve Number for Hydrologic Soil Groups			
		A	B	C	D
Apartments/Condos	72	81	88	91	93
Townhouses/6,000 sq ft lots or less[3]	69	80	87	90	92
7,000 sq ft lots	63	76	84	89	91
8,000 sq ft lots	58	73	82	88	90
10,000 sq ft lots	38	61	75	83	87
14,000 sq ft lots	30	57	72	81	86
20,000 sq ft lots	25	54	70	80	85
40,000 sq ft lots	20	51	68	79	84
80,000 sq ft lots	12	46	65	77	82

1 Lot size should represent the size of the average lot and not the gross acreage divided by the number of lots.
2 Actual percent impervious value should be compared to selected land use type.
3 In cases where average residential lots are smaller than 6,000 sq ft, commercial/business/industrial land use should be used.

Revision	Date

REFERENCE:

TABULAR HYDROGRAPH METHOD
UNIT DISCHARGES

HYDROGRAPH TIME (HOURS)

TRVL TIME (HR)	11.0	11.3	11.6	11.9	12.0	12.1	12.2	12.3	12.4	12.5	12.6	12.7	12.8	13.0	13.2	13.4	13.6	13.9	14.0	14.3	14.6	15.0	15.5	16.0	16.5	17.0	17.5	18.0	19.0	20.0	22.0	26.0
IA/P = 0.10 • • • TC = 0.1 HR • • •																																
0.0	24	34	53	334	647	1010	623	217	147	123	104	86	76	66	57	51	46	42	38	34	32	29	26	23	21	20	19	18	15	13	12	0
.10	21	29	43	134	267	520	847	701	378	224	157	122	98	75	64	56	50	45	41	36	33	30	27	24	21	20	19	18	16	13	12	0
.20	18	25	35	61	110	215	418	704	702	485	312	209	151	94	73	62	54	49	44	38	34	31	28	23	22	21	19	18	16	14	12	0
.30	17	23	33	56	92	174	337	582	662	545	359	269	190	109	79	65	56	50	45	39	35	32	29	25	22	21	20	18	16	14	12	0
.40	15	20	28	41	51	78	142	272	478	601	563	447	329	172	104	76	63	55	49	42	37	33	29	26	23	21	20	19	17	14	12	0
.50	14	19	26	39	47	69	117	220	392	531	553	482	380	209	121	84	67	57	51	43	38	33	30	27	23	21	20	19	17	14	12	0
.75	12	15	21	29	33	35	49	73	126	224	343	432	464	385	252	156	103	76	62	50	43	36	31	28	25	22	21	19	17	15	12	0
1.0	9	12	15	21	23	26	29	33	40	55	56	148	238	406	434	317	205	130	89	62	30	41	34	30	27	24	22	20	18	16	12	0
1.5	7	9	10	14	15	16	18	20	22	25	29	34	45	101	220	339	373	320	234	131	80	53	40	34	30	27	24	21	19	17	12	2
2.0	4	6	7	9	9	10	11	12	13	15	16	18	20	25	37	72	150	252	336	312	216	109	58	42	34	30	27	24	20	18	13	8
2.5	3	4	5	6	7	7	8	8	9	10	11	12	13	16	19	25	39	75	142	262	305	229	108	58	41	34	30	27	22	19	14	11
3.0	1	2	3	4	4	5	5	6	6	7	7	8	8	10	12	14	17	22	31	76	169	288	236	122	64	43	35	30	24	20	16	11
IA/P = 0.30 • • • TC = 0.1 HR • • •																																
0.0	0	0	0	154	568	936	524	217	172	149	126	107	97	86	76	69	63	58	53	48	46	42	38	34	31	30	28	27	24	20	19	0
.10	0	0	0	19	109	415	762	603	346	230	176	143	119	96	84	74	68	62	57	50	47	44	40	35	32	30	29	27	24	21	19	0
.20	0	0	0	0	13	77	302	609	605	432	297	217	167	115	94	81	73	66	60	53	48	45	41	37	33	31	29	28	25	21	19	0
.30	0	0	0	0	9	54	219	479	563	476	357	263	199	129	99	85	75	68	62	54	49	45	41	37	33	31	29	29	25	21	19	0
.40	0	0	0	0	0	6	38	159	372	500	484	399	309	183	123	96	82	73	66	58	51	46	42	38	34	31	30	28	25	22	19	0
.50	0	0	0	0	0	4	27	115	287	429	465	421	346	213	138	103	86	76	68	59	32	47	43	39	34	32	30	29	25	22	19	0
.75	0	0	0	0	0	0	1	10	46	132	246	339	381	341	243	165	119	94	80	67	58	50	45	41	37	33	31	29	26	23	19	0
1.0	0	0	0	0	0	0	0	1	4	22	69	149	241	357	331	246	170	122	96	76	64	54	47	42	38	34	32	30	27	24	19	0
1.5	0	0	0	0	0	0	0	0	0	0	0	1	4	41	142	258	310	285	224	142	97	71	55	47	43	39	35	32	29	25	20	4
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	10	49	130	221	279	255	182	108	70	55	47	42	38	34	30	27	20	11
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	14	52	119	224	256	193	107	70	55	47	42	38	32	28	22	17
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	9	52	141	240	199	117	74	56	48	43	35	30	24	18
IA/P = 0.50 • • • TC = 0.1 HR • • •																																
0.0	0	0	0	0	70	539	377	196	171	154	134	117	108	99	89	83	77	72	67	61	59	56	51	46	43	42	40	38	34	30	28	0
.10	0	0	0	0	47	375	376	256	199	169	146	126	114	102	92	85	79	73	68	62	59	56	52	47	43	42	40	38	34	30	28	0
.20	0	0	0	0	0	31	260	338	283	227	189	160	138	112	99	90	83	77	72	64	60	57	53	48	44	42	41	39	35	30	28	0
.30	0	0	0	0	0	0	21	180	285	284	246	208	176	131	110	97	88	82	76	68	62	59	54	50	45	43	41	39	36	31	28	0
.40	0	0	0	0	0	0	14	125	232	266	253	223	192	142	115	100	91	83	77	69	63	59	55	50	45	43	41	40	36	31	28	0
.50	0	0	0	0	0	0	9	86	183	239	248	231	205	154	122	104	93	85	79	71	64	59	55	51	46	43	41	40	36	32	28	0
.75	0	0	0	0	0	0	3	31	87	147	190	211	213	184	147	121	103	92	84	75	67	61	57	52	47	44	42	40	37	32	28	0
1.0	0	0	0	0	0	0	0	0	1	13	45	92	141	205	197	165	134	112	98	84	75	65	[illegible]	55	50	46	43	41	38	34	28	0
1.5	0	0	0	0	0	0	0	0	0	0	0	2	9	51	118	170	183	167	143	111	92	77	65	59	54	50	45	43	39	35	28	2
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	15	51	103	148	168	156	127	96	76	65	58	54	49	45	41	37	29	12
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	9	31	69	131	159	140	101	78	66	59	54	50	43	39	31	24
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	11	45	101	151	134	99	77	65	59	54	45	41	33	26

RAINFALL TYPE = II • • • TC = 0.1 HR • • • SHEET 1 OF 10

Revision	Date

NOTE: This Table is only valid for the SCS Type II Rainfall Distribution.

REFERENCE: SCS TR-55, USDA, June 1986.

TABULAR HYDROGRAPH METHOD
UNIT DISCHARGES

TRVL TIME (HR)	11.0	11.3	11.6	11.9	12.0	12.1	12.2	12.3	12.4	12.5	12.6	12.7	12.8	13.0	13.2	13.4	13.6	13.8	14.0	14.3	14.6	15.0	15.5	16.0	16.5	17.0	17.5	18.0	19.0	20.0	22.0	26.0
IA/P = 0.10 * * * TC =0.2 HR * * * IA/P = 0.10																																
0.0	23	31	47	209	403	739	800	481	250	166	128	102	86	70	61	54	49	44	40	35	33	30	27	24	21	20	19	18	16	13	12	0
.10	19	26	39	86	168	325	601	733	565	355	229	161	122	83	69	59	53	47	43	37	34	31	28	25	22	21	19	18	16	14	12	0
.20	17	23	32	49	74	136	262	488	652	594	435	298	207	115	81	67	58	51	46	40	35	32	29	26	23	21	20	19	16	14	12	0
.30	16	22	30	46	64	112	212	396	566	585	485	360	258	139	90	71	60	53	48	41	36	32	29	26	23	21	20	19	16	14	12	0
.40	14	19	25	37	43	57	94	173	322	485	551	507	409	227	129	87	68	58	52	44	38	33	30	27	24	21	20	19	17	14	12	0
.50	13	18	24	35	40	52	80	142	262	410	504	506	441	269	153	98	73	61	53	45	39	34	30	27	24	22	20	19	17	15	12	0
.75	10	13	17	23	26	30	34	40	55	86	150	247	349	438	360	240	151	101	75	57	47	39	33	29	26	23	21	20	18	15	12	0
1.0	9	11	14	19	21	24	26	30	35	44	62	101	167	337	413	353	245	157	104	68	53	42	35	31	28	24	22	20	18	16	12	0
1.5	6	8	10	13	14	15	17	19	21	23	26	30	37	73	166	288	356	337	264	154	91	57	42	35	30	27	24	22	19	17	13	3
2.0	4	5	7	8	9	10	10	11	12	14	15	16	18	23	31	55	114	206	291	324	239	125	63	44	35	31	28	24	20	18	14	9
2.5	3	4	5	6	6	7	7	8	9	9	10	11	12	15	18	22	32	58	111	227	258	246	122	63	43	35	31	27	22	19	15	11
3.0	1	2	3	4	4	4	5	5	6	6	7	7	8	9	11	13	16	19	27	59	138	280	248	137	70	46	36	31	25	21	16	11
IA/P = 0.30 * * * TC =0.2 HR * * * IA/P = 0.30																																
0.0	0	0	0	39	180	545	697	497	276	198	158	130	110	93	81	73	67	61	56	49	46	43	39	35	32	30	29	27	24	21	19	0
.10	0	0	0	2	27	129	407	600	532	361	252	190	150	108	90	79	71	65	59	52	48	44	41	36	32	31	29	28	25	21	19	0
.20	0	0	0	2	19	92	302	501	521	415	306	228	176	119	95	82	73	67	61	53	48	45	41	37	33	31	29	28	25	21	19	0
.30	0	0	0	0	1	13	66	223	408	484	438	350	269	163	114	93	80	72	65	57	51	46	42	38	34	31	30	28	25	22	19	0
.40	0	0	0	0	1	9	47	164	327	431	436	379	306	189	127	98	83	74	67	58	52	47	43	38	34	31	30	28	25	22	19	0
.50	0	0	0	0	0	0	6	33	120	258	374	415	391	271	173	121	95	81	72	62	55	48	44	40	35	32	30	29	26	22	19	0
.75	0	0	0	0	0	0	2	13	50	126	221	302	348	323	240	167	121	96	81	68	59	50	45	41	37	33	31	29	26	23	19	0
1.0	0	0	0	0	0	0	0	0	1	6	24	69	139	285	331	280	204	145	109	82	68	56	48	43	39	35	32	30	27	24	19	0
1.5	0	0	0	0	0	0	0	0	0	0	0	0	1	16	79	186	271	288	247	165	110	76	58	49	44	40	35	32	29	26	20	5
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	3	24	80	163	235	262	202	123	76	58	49	43	39	35	30	27	21	13
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	6	28	77	179	242	207	120	75	57	48	43	39	32	29	22	17
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	4	30	101	207	227	130	80	59	49	44	35	30	24	18
IA/P = 0.50 * * * TC =0.2 HR * * * IA/P = 0.50																																
0.0	0	0	0	0	7	98	371	322	221	182	158	137	120	104	94	86	80	74	69	62	60	57	52	47	44	42	40	39	35	30	29	0
.10	0	0	0	0	4	67	270	305	249	204	174	149	130	108	97	88	82	76	71	64	60	57	53	48	44	42	41	39	35	30	28	0
.20	0	0	0	0	0	3	45	195	268	255	221	189	163	125	106	95	87	80	75	67	62	58	54	49	45	43	41	39	35	31	28	0
.30	0	0	0	0	0	2	31	140	226	245	229	203	176	134	111	98	89	82	76	68	62	59	55	50	45	43	41	39	36	31	28	0
.40	0	0	0	0	0	1	21	101	184	225	228	211	189	144	117	101	91	84	78	69	63	59	55	50	45	43	41	40	36	31	28	0
.50	0	0	0	0	0	0	1	14	72	146	199	218	213	175	137	113	99	89	82	73	66	60	56	52	47	43	42	40	36	32	28	0
.75	0	0	0	0	0	0	0	5	28	71	121	162	186	193	161	133	112	98	88	78	70	62	57	53	48	44	42	41	37	33	28	0
1.0	0	0	0	0	0	0	0	0	0	2	13	38	77	154	186	174	147	122	105	89	78	68	60	56	51	46	43	42	38	34	28	0
1.5	0	0	0	0	0	0	0	0	0	0	0	0	2	22	71	129	163	168	150	120	98	80	67	60	55	51	46	43	40	36	28	4
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	5	25	65	112	146	157	134	103	79	67	60	55	50	46	41	38	29	14
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	9	26	60	117	148	136	101	79	66	59	54	50	43	39	31	24
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	9	40	90	142	130	99	78	66	59	54	45	41	33	26

RAINFALL TYPE = II * * * TC = 0.2 HR * * * SHEET 2 OF 10

NOTE: This Table is only valid for the SCS Type II Rainfall Distribution.

Revision	Date

REFERENCE: SCS TR-55, USDA, June 1986.

TABULAR HYDROGRAPH METHOD
UNIT DISCHARGES

HYDROGRAPH TIME (HOURS)

TRVL TIME (HR)	11.0	11.3	11.6	11.9	12.0	12.1	12.2	12.3	12.4	12.5	12.6	12.7	12.8	13.0	13.2	13.4	13.6	13.8	14.0	14.3	14.6	15.0	15.5	16.0	16.5	17.0	17.5	18.0	19.0	20.0	22.0	26.0
IA/P = 0.10																																
0.0	20	28	41	118	235	447	676	676	459	283	196	146	114	80	66	57	51	46	42	37	33	31	28	24	22	20	19	18	16	13	12	0
.10	19	26	39	99	189	361	571	641	520	362	251	181	136	89	70	60	53	48	43	37	34	31	28	25	22	21	19	18	16	14	12	0
.20	17	23	32	53	83	154	292	478	587	542	422	308	223	127	86	68	58	52	46	40	35	32	29	26	23	21	20	19	16	14	12	0
.30	16	22	30	49	72	127	237	398	524	536	460	359	268	151	97	73	61	53	48	41	36	32	29	26	23	21	20	19	16	14	12	0
.40	14	19	25	37	45	63	105	193	330	459	510	477	398	237	139	92	70	59	52	44	38	34	30	27	24	21	20	19	17	14	12	0
.50	13	18	24	35	42	56	89	158	272	397	472	475	424	274	163	104	76	62	54	46	39	34	30	27	24	22	20	19	17	15	12	0
.75	11	14	19	26	30	34	42	59	95	160	250	339	417	398	299	196	128	89	69	54	45	37	32	29	26	23	21	20	17	15	12	0
1.0	9	11	14	19	21	24	27	30	36	46	68	109	174	328	396	346	248	163	109	70	54	43	35	31	28	24	22	20	18	16	12	0
1.5	6	8	10	13	14	15	17	19	21	23	26	31	38	77	169	282	347	330	264	158	94	58	42	35	31	27	24	22	19	17	13	3
2.0	4	5	7	8	9	10	10	11	12	14	15	16	18	23	32	57	116	205	285	317	239	128	64	44	36	31	28	25	20	18	14	9
2.5	2	4	5	6	6	7	7	8	9	9	10	11	12	15	18	23	33	60	113	223	293	245	125	65	44	35	31	27	22	19	15	11
3.0	1	2	3	4	4	4	5	5	6	6	7	7	8	9	11	13	16	20	27	61	138	275	246	139	72	46	36	31	25	21	16	11
IA/P = 0.30																																
0.0	0	0	0	11	64	251	525	574	454	303	221	173	140	104	88	77	70	64	58	51	47	44	40	36	32	31	29	28	24	21	19	0
.10	0	0	0	0	7	45	183	411	520	476	360	268	205	133	101	85	76	69	62	55	49	45	41	37	33	31	30	28	25	21	19	0
.20	0	0	0	0	5	32	132	318	452	468	396	310	240	151	109	90	78	70	64	56	50	46	42	38	33	31	30	28	25	22	19	0
.30	0	0	0	0	0	3	22	96	244	383	440	411	344	217	142	105	87	76	69	60	53	47	43	39	35	32	30	29	26	22	19	0
.40	0	0	0	0	0	2	16	69	186	317	399	407	365	246	160	115	92	79	71	61	54	48	43	39	35	32	30	29	26	22	19	0
.50	0	0	0	0	0	0	2	11	50	140	258	352	389	327	223	149	110	89	77	66	57	50	45	41	36	33	31	29	26	23	19	0
.75	0	0	0	0	0	0	1	4	20	63	135	219	290	335	281	205	146	110	89	72	62	52	46	42	38	34	31	30	27	23	19	0
1.0	0	0	0	0	0	0	0	0	0	2	9	32	78	216	320	306	243	176	128	90	72	59	49	44	40	56	33	31	28	24	19	1
1.5	0	0	0	0	0	0	0	0	0	0	0	0	2	20	84	185	264	281	246	168	112	77	58	49	44	40	36	32	29	26	20	5
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	12	50	121	200	257	224	141	83	61	50	44	40	36	31	28	21	14
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	3	16	51	145	239	223	137	82	60	50	44	40	33	29	22	17
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	19	74	184	224	146	89	63	51	45	36	31	24	18
IA/P = 0.50																																
0.0	0	0	0	0	1	25	151	299	277	219	187	162	141	113	100	90	84	75	72	65	61	58	53	48	44	42	41	39	35	31	28	0
.10	0	0	0	0	1	17	106	235	263	234	202	175	152	120	104	93	85	79	73	66	61	58	54	49	44	42	41	39	35	31	28	0
.20	0	0	0	0	0	0	12	75	182	236	234	213	188	144	116	101	91	84	78	70	63	59	55	50	45	43	41	40	36	31	28	0
.30	0	0	0	0	0	0	8	52	138	203	224	217	197	154	123	105	94	86	79	71	64	59	55	51	46	43	42	40	36	32	28	0
.40	0	0	0	0	0	0	5	37	105	170	206	213	203	164	131	110	97	88	81	72	65	60	56	51	46	43	42	40	36	32	28	0
.50	0	0	0	0	0	0	0	4	26	78	140	184	203	191	155	126	107	95	86	76	69	62	57	53	48	44	42	41	37	33	28	0
.75	0	0	0	0	0	0	0	1	10	34	73	117	153	184	173	146	122	105	94	82	73	64	58	54	49	45	43	41	37	33	28	0
1.0	0	0	0	0	0	0	0	0	0	0	4	17	42	114	168	178	159	134	114	94	82	70	61	57	52	47	44	42	39	35	28	0
1.5	0	0	0	0	0	0	0	0	0	0	0	0	1	10	44	98	144	163	157	130	105	84	69	61	56	52	47	44	40	36	29	6
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	14	44	87	127	153	141	110	83	69	61	56	51	47	42	38	30	17
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	4	16	42	97	138	145	107	82	68	60	55	51	43	40	32	25
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	5	27	71	127	139	105	81	68	60	55	46	41	33	27

* * * TC = 0.3 HR * * *

RAINFALL TYPE = II

SHEET 3 OF 10

NOTE: This Table is only valid for the SCS Type II Rainfall Distribution.

Revision	Date

TABULAR HYDROGRAPH METHOD
UNIT DISCHARGES

HYDROGRAPH TIME (HOURS)

IA/P = 0.10 • • • TC = 0.4 HR • • • IA/P = 0.10

TRVL TIME (HR)	11.0	11.3	11.6	11.9	12.0	12.1	12.2	12.3	12.4	12.5	12.6	12.7	12.8	13.0	13.2	13.4	13.6	13.8	14.0	14.3	14.6	15.0	15.5	16.0	16.5	17.0	17.5	18.0	19.0	20.0	22.0	26.0
0.0	18	25	36	77	141	271	468	592	574	431	298	216	163	104	77	63	55	49	44	38	34	31	28	25	22	21	20	18	16	14	12	0
.10	18	24	34	67	116	219	385	523	557	473	357	263	196	119	84	67	57	51	46	39	35	32	29	25	22	21	20	19	16	14	12	0
.20	15	20	28	44	59	97	179	316	454	523	489	401	309	178	112	81	65	56	49	42	37	33	30	26	23	21	20	19	17	14	12	0
.30	15	20	27	41	53	82	147	260	389	478	486	429	349	210	129	89	69	58	51	43	38	35	30	27	24	21	20	19	17	14	12	0
.40	13	17	23	33	38	48	71	121	214	331	429	467	442	308	189	120	85	66	56	47	41	35	31	28	24	22	20	19	17	15	12	0
.50	12	16	22	31	36	44	62	102	176	279	379	438	440	339	218	137	94	71	59	49	42	35	31	28	25	22	21	19	17	15	12	0
.75	10	13	17	24	26	30	35	45	65	106	170	251	326	393	341	245	164	112	81	59	48	39	33	30	26	23	21	20	18	15	12	0
1.0	8	10	13	17	19	21	24	27	31	37	50	75	118	251	360	376	292	205	138	85	60	45	36	32	28	25	22	21	18	16	12	1
1.5	6	7	9	12	13	14	15	17	19	21	23	26	31	56	121	224	311	333	293	192	115	66	45	36	31	28	25	22	19	17	13	4
2.0	4	5	6	8	8	9	10	10	11	12	14	15	16	20	27	43	85	159	243	306	264	154	74	47	37	32	28	25	21	18	14	9
2.5	2	3	4	5	6	6	7	7	8	9	9	10	11	13	16	20	27	46	85	184	285	262	147	74	47	37	32	25	22	19	15	11
3.0	1	2	2	3	4	4	4	5	5	6	6	7	7	8	10	12	14	17	23	47	109	227	268	160	83	50	38	32	25	21	16	11

IA/P = 0.30 • • • TC = 0.4 HR • • • IA/P = 0.30

TRVL TIME (HR)	11.0	11.3	11.6	11.9	12.0	12.1	12.2	12.3	12.4	12.5	12.6	12.7	12.8	13.0	13.2	13.4	13.6	13.8	14.0	14.3	14.6	15.0	15.5	16.0	16.5	17.0	17.5	18.0	19.0	20.0	22.0	26.0
0.0	0	0	0	4	26	113	296	480	495	413	306	234	186	127	100	84	74	67	61	54	49	45	41	37	33	31	29	28	25	21	19	0
.10	0	0	0	0	2	18	81	224	395	462	430	347	272	172	121	96	82	73	66	57	51	46	42	38	34	31	30	28	25	22	19	0
.20	0	0	0	0	2	13	59	169	320	414	424	373	305	196	134	103	83	75	67	59	52	47	43	39	34	32	30	29	25	22	19	0
.30	0	0	0	0	0	1	9	42	127	255	361	403	383	274	181	127	99	83	73	63	55	48	44	40	36	32	30	29	26	23	19	0
.40	0	0	0	0	0	1	6	30	94	202	308	372	379	298	203	141	106	87	76	65	56	49	44	40	36	32	31	29	26	23	19	0
.50	0	0	0	0	0	0	0	4	21	70	158	258	334	364	270	187	133	102	85	70	60	51	46	41	37	33	31	30	26	23	19	0
.75	0	0	0	0	0	0	0	2	8	30	76	145	219	321	305	241	177	130	102	78	65	55	47	43	38	34	32	30	27	24	19	0
1.0	0	0	0	0	0	0	0	0	0	1	4	15	42	150	267	308	272	209	154	103	79	62	51	45	41	37	33	31	29	25	19	1
1.5	0	0	0	0	0	0	0	0	0	0	0	0	1	10	51	136	226	274	263	195	131	85	62	51	45	41	36	33	29	26	20	6
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	6	31	86	162	252	239	162	93	64	52	45	41	37	31	28	21	15
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	9	35	112	202	235	155	92	64	52	45	41	33	29	23	18
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	21	76	182	221	148	90	63	51	45	36	31	24	18

IA/P = 0.50 • • • TC = 0.4 HR • • • IA/P = 0.50

TRVL TIME (HR)	11.0	11.3	11.6	11.9	12.0	12.1	12.2	12.3	12.4	12.5	12.6	12.7	12.8	13.0	13.2	13.4	13.6	13.8	14.0	14.3	14.6	15.0	15.5	16.0	16.5	17.0	17.5	18.0	19.0	20.0	22.0	26.0
0.0	0	0	0	0	0	7	59	168	245	257	213	186	163	128	109	96	88	81	75	67	62	58	54	50	45	43	41	39	35	31	28	0
.10	0	0	0	0	0	0	5	41	125	205	240	222	198	154	123	106	94	86	79	71	64	60	56	51	46	43	42	40	36	32	28	0
.20	0	0	0	0	0	0	3	28	93	168	216	220	205	164	131	110	97	88	81	72	65	60	56	51	46	43	42	40	36	32	28	0
.30	0	0	0	0	0	0	0	2	20	69	155	189	209	192	133	126	107	95	86	77	69	62	57	53	48	44	42	41	37	33	28	0
.40	0	0	0	0	0	0	0	1	14	50	106	161	193	202	163	133	112	98	89	78	70	62	58	53	48	44	42	41	37	33	28	0
.50	0	0	0	0	0	0	0	1	9	37	83	135	174	194	171	140	117	102	91	80	71	63	58	54	49	45	43	41	37	33	28	0
.75	0	0	0	0	0	0	0	0	0	3	15	40	76	147	177	169	146	124	107	90	79	68	60	56	51	47	43	42	38	34	29	0
1.0	0	0	0	0	0	0	0	0	0	0	1	7	21	78	141	173	167	146	125	101	86	73	63	58	53	48	45	42	39	35	28	1
1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	5	26	71	121	153	159	139	113	89	72	63	57	53	48	44	40	37	29	7
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	3	16	45	86	138	150	125	93	74	64	58	53	48	42	39	31	20
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	4	17	59	112	143	121	91	73	63	57	53	45	40	32	26
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	11	40	101	138	117	90	73	63	57	48	42	34	27

RAINFALL TYPE = II • • • TC = 0.4 HR • • • SHEET 4 OF 13

NOTE: This Table is only valid for the SCS Type II Rainfall Distribution.

Revision	Date

REFERENCE: SCS TR-55, USDA, June 1986.

TABULAR HYDROGRAPH METHOD
UNIT DISCHARGES

HYDROGRAPH TIME (HOURS)

TRVL TIME (HR)	11.0	11.3	11.6	11.9	12.0	12.1	12.2	12.3	12.4	12.5	12.6	12.7	12.8	13.0	13.2	13.4	13.6	13.8	14.0	14.3	14.6	15.0	15.5	16.0	16.5	17.0	17.5	18.0	19.0	20.0	22.0	26.0
IA/P = 0.10															* * * TC = 0.5 HR * * *														IA/P = 0.10			
0.0	17	23	32	57	94	170	509	467	529	507	402	297	226	140	96	74	61	53	47	41	36	32	29	26	23	21	20	19	16	14	12	0
.10	16	22	30	51	80	140	252	395	484	499	434	343	265	162	105	80	65	55	49	42	36	33	29	26	23	21	20	19	16	14	12	0
.20	14	19	25	38	47	69	116	207	332	434	477	449	378	238	149	101	77	62	53	45	39	34	30	27	24	22	20	19	17	14	12	0
.30	13	18	24	35	43	60	97	170	278	382	446	448	401	270	171	114	83	66	56	46	40	34	31	27	24	22	20	19	17	15	12	0
.40	12	15	21	29	33	40	53	83	141	233	332	408	434	361	243	157	107	79	64	51	43	36	32	28	25	22	21	20	17	15	12	0
.50	11	15	20	28	31	37	48	71	118	194	286	367	412	378	271	178	119	86	68	53	44	37	32	29	25	23	21	20	17	15	12	0
.75	9	11	14	19	21	24	27	31	37	49	74	118	182	319	374	328	244	169	117	76	56	43	35	31	25	25	22	21	18	16	12	1
1.0	7	9	12	16	17	19	21	24	27	32	40	55	83	188	309	359	322	245	172	102	68	49	38	32	29	26	23	21	19	16	12	1
1.5	5	7	8	11	12	13	14	15	17	19	21	23	27	43	89	175	269	322	309	225	140	77	49	38	32	29	25	23	20	17	13	5
2.0	3	4	6	7	8	8	9	10	10	11	12	14	15	18	23	35	65	123	202	297	280	181	88	52	39	33	29	26	21	19	14	10
2.5	2	3	4	5	5	6	6	7	7	8	9	[illegible]	10	12	15	18	24	36	66	150	244	278	171	87	52	39	33	29	23	20	15	11
3.0	1	1	2	3	3	4	4	4	5	5	6	6	7	8	9	11	13	16	20	37	86	198	263	182	96	56	40	33	26	21	16	11
IA/P = 0.30															* * * TC = 0.5 HR * * *														IA/P = 0.30			
0.0	0	0	0	1	9	53	157	314	433	439	379	299	237	159	119	93	81	71	65	56	50	46	42	38	34	31	30	29	25	22	19	0
.10	0	0	0	0	1	6	37	117	248	372	416	391	330	218	150	113	92	79	70	60	53	47	43	39	35	32	30	29	26	22	19	0
.20	0	0	0	0	1	4	26	87	194	313	392	388	349	244	167	122	97	82	72	62	54	48	43	39	35	32	30	29	26	22	19	0
.30	0	0	0	0	0	0	3	19	64	151	259	341	372	316	223	156	117	94	80	67	58	50	45	41	36	33	31	29	26	23	19	0
.40	0	0	0	0	0	0	2	13	47	116	211	298	334	328	265	172	127	100	83	69	59	51	45	41	37	33	31	29	26	23	19	0
.50	0	0	0	0	0	0	0	1	9	34	89	170	255	341	303	225	161	120	96	76	64	56	47	42	38	34	31	30	27	24	19	0
.75	0	0	0	0	0	0	0	1	4	14	41	89	152	270	305	268	207	155	118	87	70	57	48	44	39	35	32	30	27	24	19	0
1.0	0	0	0	0	0	0	0	0	0	0	2	7	22	98	212	295	285	257	181	120	88	67	53	46	42	38	34	31	28	25	19	2
1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	5	30	95	183	249	265	217	152	96	66	53	46	41	37	34	30	26	20	8
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	3	18	59	125	221	245	182	105	69	54	47	42	38	32	28	22	16
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	5	21	84	174	230	172	103	69	54	46	42	34	30	23	18
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	13	56	157	217	163	101	68	53	46	37	31	25	18
IA/P = 0.50															* * * TC = 0.5 HR * * *														IA/P = 0.50			
0.0	0	0	0	0	0	2	26	89	170	217	229	200	179	144	119	104	93	85	78	70	64	59	55	51	46	43	41	40	36	32	28	0
.10	0	0	0	0	0	0	1	18	65	135	190	216	265	170	137	115	101	91	83	74	67	61	56	52	47	44	42	40	36	32	28	0
.20	0	0	0	0	0	0	1	12	47	106	162	198	203	178	145	121	105	94	85	76	68	61	57	52	48	44	42	40	37	32	28	0
.30	0	0	0	0	0	0	0	1	8	34	82	135	177	194	168	139	117	102	92	80	71	63	58	54	49	45	43	41	37	33	28	0
.40	0	0	0	0	0	0	0	0	6	25	63	111	155	189	174	146	122	106	94	82	73	64	58	54	50	45	43	41	37	33	28	0
.50	0	0	0	0	0	0	0	0	4	18	48	90	133	184	177	152	128	110	97	84	74	65	59	55	50	45	43	41	38	33	25	0
.75	0	0	0	0	0	0	0	0	1	7	22	47	80	142	169	164	144	124	108	91	79	68	61	56	51	47	44	42	38	34	28	0
1.0	0	0	0	0	0	0	0	0	0	0	1	3	11	51	112	155	166	154	134	109	91	76	65	59	54	49	45	43	39	35	28	2
1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	2	16	50	97	136	154	145	121	95	75	64	58	54	49	45	41	37	29	10
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	4	18	47	86	134	146	125	94	75	64	58	53	49	42	39	31	21
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	3	11	44	95	140	127	97	77	65	58	54	45	41	33	26
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	7	29	86	135	122	95	76	65	58	49	43	35	27

RAINFALL TYPE = II * * * TC = 0.5 HR * * * SHEET 5 OF 10

NOTE: This Table is only valid for the SCS Type II Rainfall Distribution.

Revision	Date

WRC ENGINEERING

REFERENCE:
SCS TR-55, USDA, June 1986.

TABULAR HYDROGRAPH METHOD UNIT DISCHARGES

HYDROGRAPH TIME (HOURS)

TRVL TIME (HR)	11.0	11.3	11.6	11.9	12.0	12.1	12.2	12.3	12.4	12.5	12.6	12.7	12.8	13.0	13.2	13.4	13.6	13.8	14.0	14.3	14.6	15.0	15.5	16.0	16.5	17.0	17.5	18.0	19.0	20.0	22.0	26.0
IA/P = 0.10															• • • TC =0.75 HR • • •														IA/P = 0.10			
0.0	13	18	24	36	46	68	115	194	294	380	424	410	369	252	172	123	93	74	61	49	41	35	31	27	24	22	20	19	17	15	12	0
.10	13	17	23	34	42	59	97	162	250	337	395	405	381	279	191	135	100	79	65	51	42	36	31	28	25	22	21	19	17	15	12	0
.20	11	15	20	28	32	39	52	82	135	211	295	362	391	351	255	178	127	95	75	57	46	38	32	29	26	23	21	20	17	15	12	0
.30	11	14	19	26	30	36	47	70	113	179	256	326	379	360	277	196	140	103	80	60	48	38	33	29	26	23	21	20	18	15	12	0
.40	10	12	16	22	25	28	33	42	61	96	151	221	291	367	336	255	182	131	98	69	54	42	34	30	27	24	22	20	18	16	12	0
.50	9	12	16	21	24	27	31	39	53	82	128	190	258	358	343	274	200	144	106	74	56	43	35	30	27	24	22	20	18	16	12	0
.75	8	10	13	17	18	21	23	26	31	39	55	82	122	230	314	329	281	217	161	104	72	51	38	33	29	26	23	21	19	16	12	1
1.0	6	8	10	13	14	15	17	19	21	23	27	32	42	89	177	272	319	303	249	163	105	66	45	36	31	27	24	22	19	17	13	3
1.5	4	6	7	9	10	10	11	12	14	15	16	18	20	27	46	90	163	241	295	275	204	119	66	45	35	31	27	24	20	18	13	7
2.0	3	4	5	6	7	7	8	9	9	10	11	12	13	16	20	28	48	89	151	245	274	213	115	65	44	35	30	27	22	19	14	10
2.5	1	2	3	4	4	5	5	6	6	7	7	8	9	10	12	14	17	24	37	86	170	260	219	127	71	47	36	31	24	20	16	11
3.0	1	1	2	3	3	3	4	4	4	5	5	5	6	7	8	10	11	14	17	30	64	157	247	205	122	70	46	36	27	22	17	12
IA/P = 0.30															• • • TC =0.75 HR • • •														IA/P = 0.30			
0.0	0	0	0	0	1	6	30	86	174	266	326	348	328	246	181	138	110	92	79	66	57	49	44	40	36	32	31	29	26	23	19	0
.10	0	0	0	0	0	1	4	22	65	137	225	292	329	303	228	170	131	106	89	73	61	52	46	41	37	33	31	29	26	23	19	0
.20	0	0	0	0	0	0	3	15	48	108	185	256	305	321	245	184	141	112	93	75	63	53	46	42	37	34	31	30	27	23	19	0
.30	0	0	0	0	0	0	2	11	36	84	151	221	277	308	260	199	152	120	98	78	65	54	47	42	38	34	31	30	27	23	19	0
.40	0	0	0	0	0	0	0	1	8	27	65	122	188	286	301	243	197	144	114	87	71	57	48	43	39	35	32	30	27	24	19	1
.50	0	0	0	0	0	0	0	1	6	20	50	98	158	263	292	254	200	155	122	91	74	59	49	44	40	35	32	30	27	24	19	1
.75	0	0	0	0	0	0	0	0	0	2	8	23	51	140	231	269	253	211	167	119	90	68	53	46	42	37	34	31	28	25	19	2
1.0	0	0	0	0	0	0	0	0	0	0	0	1	4	29	96	196	249	261	231	169	120	84	61	50	44	40	36	33	29	26	20	5
1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	1	8	34	91	163	220	241	197	131	83	61	50	44	40	35	31	27	21	12
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	11	36	85	174	226	200	127	82	60	49	44	39	32	29	22	17
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	6	37	105	196	214	135	87	62	51	44	36	31	24	18
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	4	24	96	205	189	130	85	62	50	39	32	26	18
IA/P = 0.50															• • • TC =0.75 HR • • •														IA/P = 0.50			
0.0	0	0	0	0	0	0	2	16	45	92	137	166	185	170	146	125	110	98	89	79	70	63	58	53	48	44	42	41	37	33	28	0
.10	0	0	0	0	0	0	0	1	11	34	73	115	149	180	163	141	122	107	96	84	74	65	59	54	50	45	43	41	38	35	28	0
.20	0	0	0	0	0	0	0	1	8	25	57	96	131	173	166	146	126	111	99	86	76	66	59	55	50	46	43	41	38	34	28	0
.30	0	0	0	0	0	0	0	0	1	5	18	44	79	143	170	160	141	122	108	92	81	69	61	56	52	47	44	42	38	34	28	1
.40	0	0	0	0	0	0	0	0	0	4	14	34	64	127	166	162	145	127	111	95	82	70	62	57	52	47	44	42	38	34	28	1
.50	0	0	0	0	0	0	0	0	0	0	2	10	26	52	139	162	157	140	123	103	88	75	64	58	53	49	45	43	39	35	28	2
.75	0	0	0	0	0	0	0	0	0	0	1	4	12	47	98	139	154	148	135	113	96	80	67	60	55	50	46	43	39	36	29	3
1.0	0	0	0	0	0	0	0	0	0	0	0	0	0	6	30	73	119	146	151	134	113	91	74	63	58	53	48	45	41	37	29	7
1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	9	30	66	105	143	143	117	90	73	63	57	52	48	42	39	30	18
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	11	30	77	121	137	114	88	72	63	57	52	44	40	32	25
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	3	19	55	111	132	111	87	71	62	56	47	42	34	27
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	12	51	112	128	108	86	71	62	51	44	36	27

RAINFALL TYPE = II • • • TC = 0.75 HR • • • SHEET 6 OF 10

NOTE: This Table is only valid for the SCS Type II Rainfall Distribution.

Revision	Date

WRC ENGINEERING

REFERENCE:
SCS TR-55, USDA, June 1986.

TABULAR HYDROGRAPH METHOD
UNIT DISCHARGES

TRVL TIME (HR)	11.0	11.3	11.6	11.9	12.0	12.1	12.2	12.3	12.4	12.5	12.6	12.7	12.8	13.0	13.2	13.4	13.6	13.8	14.0	14.3	14.6	15.0	15.5	16.0	16.5	17.0	17.5	18.0	19.0	20.0	22.0	26.0
IA/P = 0.10														* * * TC =1.0 HR * * *															IA/P = 0.10			
0.0	11	15	20	29	35	47	72	112	168	231	289	329	357	313	239	175	133	103	83	63	50	40	33	29	26	23	21	20	17	15	12	0
.10	10	13	17	24	27	33	42	62	95	144	202	260	306	340	293	222	165	126	98	72	56	43	35	30	27	24	22	20	18	15	12	0
.20	10	13	17	23	26	30	38	54	82	123	176	232	281	332	303	238	179	136	105	76	59	45	35	30	27	24	22	20	18	16	12	1
.30	9	12	16	22	24	28	35	48	70	105	152	205	256	323	310	254	193	146	113	81	61	46	36	31	27	24	22	20	18	16	12	1
.40	8	11	14	19	21	23	27	32	42	61	91	132	181	276	318	294	237	181	138	95	70	51	39	32	28	25	23	21	18	16	12	1
.50	8	10	13	18	20	22	25	30	38	53	78	114	159	253	311	300	251	195	149	102	74	53	40	33	29	25	23	21	18	16	12	1
.75	7	8	11	14	16	17	19	21	25	30	38	53	76	146	228	284	295	256	208	143	99	66	46	36	31	27	24	22	19	17	13	2
1.0	5	7	8	11	12	13	14	16	17	19	22	25	31	57	111	188	256	286	272	208	144	90	56	41	33	29	26	23	20	17	13	4
1.5	4	5	6	8	8	9	10	11	12	13	14	15	17	22	33	59	107	171	231	268	235	157	88	56	41	33	29	25	21	18	14	8
2.0	2	3	4	5	5	6	6	7	7	8	9	9	10	12	13	19	27	44	78	157	231	252	167	96	59	42	34	29	23	20	15	11
2.5	1	2	2	3	4	4	4	5	5	6	6	7	7	8	10	12	15	19	27	58	120	214	241	159	94	59	42	34	26	21	16	11
3.0	0	1	1	2	2	3	3	3	4	4	4	5	5	6	7	8	10	12	14	22	44	113	214	231	152	91	58	42	29	23	17	12
IA/P = 0.30														* * * TC =1.0 HR * * *															IA/P = 0.30			
0.0	0	0	0	0	1	4	16	42	83	137	195	243	271	292	227	178	143	117	98	79	66	55	47	42	38	34	31	30	27	23	19	0
.10	0	0	0	0	0	0	3	12	32	66	113	168	218	279	260	213	169	136	113	88	72	59	49	43	39	35	32	30	27	24	19	1
.20	0	0	0	0	0	0	2	9	24	52	93	143	193	271	271	223	180	145	119	92	75	60	50	44	39	35	32	30	27	24	19	1
.30	0	0	0	0	0	0	1	6	18	41	75	120	169	246	264	234	191	153	125	96	78	62	51	44	40	36	33	31	27	24	19	1
.40	0	0	0	0	0	0	0	1	4	14	32	61	100	190	251	259	222	181	146	109	86	67	53	46	41	37	33	31	28	25	19	2
.50	0	0	0	0	0	0	0	1	3	10	24	49	83	168	237	254	230	191	155	115	90	69	54	47	42	37	34	31	28	25	19	2
.75	0	0	0	0	0	0	0	0	0	1	4	12	25	76	150	213	239	228	198	149	112	82	61	50	44	39	35	32	29	26	20	4
1.0	0	0	0	0	0	0	0	0	0	0	0	1	2	15	51	113	182	226	234	197	150	104	72	56	47	42	38	34	30	27	20	7
1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	4	18	51	104	162	220	210	158	102	71	56	47	42	37	31	28	22	13
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	5	20	49	121	187	209	152	100	70	55	47	41	34	29	23	17
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	7	32	87	171	199	146	98	69	54	46	37	31	24	18
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	13	62	158	192	151	103	73	56	41	34	26	18
IA/P = 0.50														* * * TC =1.0 HR * * *															IA/P = 0.50			
0.0	0	0	0	0	0	0	1	7	21	42	71	101	126	160	154	138	123	110	100	87	77	67	60	55	50	46	43	41	38	34	28	1
.10	0	0	0	0	0	0	0	1	5	15	33	38	87	134	156	149	134	120	108	93	82	71	62	57	52	47	44	42	38	34	28	1
.20	0	0	0	0	0	0	0	1	4	12	26	48	74	123	153	153	137	123	111	95	84	72	63	57	52	47	44	42	38	34	28	1
.30	0	0	0	0	0	0	0	0	3	9	20	38	62	111	143	130	140	127	114	98	86	73	63	58	53	48	45	42	39	35	28	1
.40	0	0	0	0	0	0	0	0	0	2	6	16	31	75	120	145	148	137	123	106	91	77	66	59	54	49	45	43	39	35	29	2
.50	0	0	0	0	0	0	0	0	0	1	5	12	25	64	109	139	146	139	127	108	94	79	67	60	55	50	46	43	39	36	29	3
.75	0	0	0	0	0	0	0	0	0	0	2	5	12	39	78	115	136	140	134	117	101	86	70	62	56	51	47	44	40	36	29	4
1.0	0	0	0	0	0	0	0	0	0	0	0	0	1	7	26	59	96	125	139	133	117	97	78	66	59	54	49	46	41	37	29	8
1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	9	24	54	86	123	133	119	95	77	66	59	54	49	43	39	31	17
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	3	10	25	64	104	129	116	93	76	65	58	53	45	41	33	24
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	10	34	84	125	117	96	78	66	59	49	43	35	27
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	6	32	89	122	114	94	77	66	53	45	37	27

RAINFALL TYPE = II   * * * TC = 1.0 HR * * *   SHEET 7 OF 10

NOTE: This Table is only valid for the
SCS Type II Rainfall Distribution.

Revision	Date

REFERENCE:

SCS TR-55, USDA, June 1986.

TABULAR HYDROGRAPH METHOD
UNIT DISCHARGES

HYDROGRAPH TIME (HOURS)

TRVL TIME (HR)	11.0	11.3	11.6	11.9	12.0	12.1	12.2	12.3	12.4	12.5	12.6	12.7	12.8	13.0	13.2	13.4	13.6	13.8	14.0	14.3	14.6	15.0	15.5	16.0	16.5	17.0	17.5	18.0	19.0	20.0	22.0	26.0
IA/P = 0.10																																
0.0	10	13	18	25	29	38	54	81	118	165	213	256	284	311	266	212	163	129	104	78	61	47	37	31	27	24	22	20	18	16	12	1
.10	10	13	17	23	27	34	47	69	102	143	189	234	267	297	274	226	173	138	111	82	64	48	38	31	27	24	22	20	18	16	12	1
.20	9	11	15	20	22	26	31	42	60	88	124	168	212	280	292	261	212	166	131	95	72	53	40	33	28	25	23	21	18	16	12	1
.30	8	11	14	19	21	24	29	38	53	76	108	148	190	263	288	268	224	177	140	101	76	55	41	34	29	25	23	21	18	16	12	2
.40	8	10	13	18	20	23	27	34	46	66	94	130	170	245	282	273	235	188	149	107	80	58	42	34	29	26	23	21	19	16	12	2
.50	7	9	12	16	17	19	22	25	31	41	58	82	114	190	256	279	262	222	178	127	93	65	46	36	31	27	24	22	19	17	13	2
.75	6	8	10	14	15	17	19	21	25	31	41	56	78	139	207	234	265	245	208	152	110	75	51	39	32	28	25	22	19	17	13	3
1.0	5	6	8	10	11	15	14	15	17	19	22	26	33	60	109	173	230	261	253	208	155	100	64	46	36	30	26	24	20	18	13	5
1.5	3	4	5	7	7	8	9	9	10	11	12	13	15	19	27	45	79	130	186	247	239	180	108	68	48	37	31	27	22	19	14	10
2.0	2	3	4	5	6	6	7	7	8	8	9	10	11	13	16	22	35	59	98	171	236	236	156	95	62	44	35	30	23	20	15	11
2.5	1	2	2	3	4	4	4	5	5	5	6	6	7	8	10	12	14	19	28	58	114	197	226	163	102	65	46	36	26	21	16	11
3.0	0	1	1	2	2	2	2	3	3	3	4	4	4	5	6	7	9	10	13	19	35	88	184	218	169	109	70	49	31	24	18	12
IA/P = 0.30																																
0.0	0	0	0	0	0	2	9	23	50	86	130	174	208	255	235	201	164	136	115	92	76	61	51	44	39	35	32	30	27	24	19	1
.10	0	0	0	0	0	0	1	6	19	40	71	110	153	217	247	227	191	157	131	103	84	66	53	46	41	36	33	31	28	24	19	2
.20	0	0	0	0	0	0	1	4	14	31	58	93	135	202	239	231	199	165	138	108	87	68	55	47	41	37	33	31	28	23	19	2
.30	0	0	0	0	0	0	0	1	3	10	24	46	77	152	210	236	222	190	158	122	97	74	58	49	43	38	34	32	29	25	20	3
.40	0	0	0	0	0	0	0	0	2	8	19	37	64	134	196	232	225	198	166	127	101	77	59	50	43	38	35	32	28	25	20	3
.50	0	0	0	0	0	0	0	0	0	2	6	14	30	82	151	206	228	217	189	146	113	85	64	52	45	40	36	33	29	26	20	5
.75	0	0	0	0	0	0	0	0	0	1	2	7	15	49	105	164	205	218	205	166	129	95	69	55	47	41	37	33	29	26	20	6
1.0	0	0	0	0	0	0	0	0	0	0	0	0	1	9	32	77	134	185	214	203	166	120	83	63	52	45	39	35	30	27	21	10
1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	11	33	72	121	184	203	171	117	82	62	51	44	39	32	29	22	15
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	7	21	67	132	194	174	123	86	64	52	45	35	31	24	18
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	13	46	121	187	166	119	84	63	52	39	32	25	18
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	8	44	129	180	160	116	83	63	44	35	27	18
IA/P = 0.50																																
0.0	0	0	0	0	0	0	1	5	13	26	44	68	91	125	142	142	128	117	107	94	83	72	63	57	52	47	44	42	38	34	28	2
.10	0	0	0	0	0	0	0	0	3	10	20	36	57	100	129	140	136	125	114	100	88	76	65	59	54	49	45	43	39	35	29	3
.20	0	0	0	0	0	0	0	0	2	7	16	30	48	90	122	139	139	127	117	102	90	77	66	60	54	49	45	43	39	35	29	3
.30	0	0	0	0	0	0	0	0	0	2	5	12	24	59	98	126	137	134	125	109	96	82	69	61	56	51	46	44	40	36	29	4
.40	0	0	0	0	0	0	0	0	0	1	4	10	19	51	89	119	134	136	127	112	98	83	70	62	56	51	47	44	40	36	29	5
.50	0	0	0	0	0	0	0	0	0	1	3	7	15	43	79	112	131	135	129	114	100	85	71	63	57	52	47	44	40	36	29	6
.75	0	0	0	0	0	0	0	0	0	0	0	1	3	15	39	71	102	123	130	125	112	94	78	67	60	54	49	46	41	37	29	9
1.0	0	0	0	0	0	0	0	0	0	0	0	0	1	4	17	40	71	101	121	129	121	103	84	71	62	56	51	47	42	38	30	13
1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	3	10	26	51	92	119	125	105	86	72	63	57	52	44	40	32	23
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	3	11	35	72	112	122	103	85	71	63	56	47	42	34	26
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	7	24	66	111	119	1C1	83	71	62	51	44	36	27
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	4	23	71	110	116	99	82	70	55	46	37	27

* * * TC =1.25 HR * * *

RAINFALL TYPE = II

SHEET 8 OF 10

NOTE: This Table is only valid for the SCS Type II Rainfall Distribution.

Revision	Date

WRC ENGINEERING

REFERENCE:
SCS TR-55, USDA, June 1986.

TABULAR HYDROGRAPH METHOD
UNIT DISCHARGES

TRVL TIME (HR)	11.0	11.3	11.6	11.9	12.0	12.1	12.2	12.3	12.4	12.5	12.6	12.7	12.8	13.0	13.2	13.4	13.6	13.8	14.0	14.3	14.6	15.0	15.5	16.0	16.5	17.0	17.5	18.0	19.0	20.0	22.0	26.0
IA/P = 0.10														* * * TC =1.5 HR * * *														IA/P = 0.10				
0.0	9	11	15	21	25	31	41	58	82	112	147	184	216	255	275	236	198	159	129	98	76	57	43	35	30	25	23	21	18	16	12	1
.10	9	10	13	18	20	23	28	37	51	72	98	131	166	226	265	254	226	187	151	113	86	63	46	37	31	26	23	21	19	16	13	2
.20	8	10	13	17	19	22	26	33	45	63	87	116	149	212	259	259	233	197	160	119	90	66	48	38	32	27	24	22	19	16	13	2
.30	7	9	12	16	18	21	24	30	40	55	76	103	134	197	244	255	238	206	169	125	95	68	49	38	32	27	24	22	19	17	13	2
.40	7	8	11	14	15	17	19	23	28	36	49	67	91	151	208	247	252	230	196	146	109	77	54	41	34	29	25	22	19	17	13	3
.50	6	8	10	15	15	16	18	21	26	33	43	59	80	136	194	238	249	235	204	154	115	81	56	42	34	29	25	23	20	17	13	3
.75	5	7	8	11	12	13	14	16	18	21	25	32	42	76	125	179	222	240	233	193	148	102	67	48	38	32	27	24	20	18	13	5
1.0	4	5	7	8	9	10	11	12	13	14	16	18	22	34	59	101	152	201	236	230	193	135	86	59	44	35	30	26	21	18	14	7
1.5	5	4	5	6	6	7	8	8	9	10	11	12	13	16	22	34	58	95	141	203	226	197	131	84	58	43	35	29	23	20	15	10
2.0	1	2	3	4	4	5	5	6	6	7	7	8	9	10	12	16	22	34	56	110	172	218	187	126	82	57	43	34	25	21	16	11
2.5	1	1	2	2	3	3	3	4	4	4	5	5	6	7	8	9	11	14	18	34	69	141	210	190	133	97	60	44	30	23	17	12
3.0	0	0	1	1	2	2	2	2	3	3	3	3	4	5	5	6	8	9	11	16	27	66	149	204	181	128	85	58	35	25	18	12
IA/P = 0.30														* * * TC =1.5 HR * * *														IA/P = 0.30				
0.0	0	0	0	0	0	1	6	15	31	53	80	112	144	195	225	208	196	157	134	108	89	70	56	48	42	37	34	31	28	25	20	2
.10	0	0	0	0	0	0	1	4	12	25	43	68	97	157	198	219	203	178	151	120	98	77	60	50	44	38	35	32	28	25	20	3
.20	0	0	0	0	0	0	0	1	3	9	19	35	57	114	168	201	213	196	171	135	108	84	64	53	46	40	36	33	29	26	20	4
.30	0	0	0	0	0	0	0	1	2	7	15	29	48	100	155	193	210	200	177	140	113	87	66	54	46	41	36	33	29	26	20	5
.40	0	0	0	0	0	0	0	0	2	5	12	23	39	87	141	184	207	202	182	146	117	89	68	55	47	41	36	33	29	26	20	5
.50	0	0	0	0	0	0	0	0	0	1	4	9	18	51	101	153	190	205	197	164	131	99	73	58	49	43	38	34	30	26	20	7
.75	0	0	0	0	0	0	0	0	0	0	2	4	9	30	68	116	160	189	197	179	147	110	80	62	52	45	39	35	30	27	21	8
1.0	0	0	0	0	0	0	0	0	0	0	0	0	1	5	20	49	92	138	175	195	178	137	97	72	57	48	42	37	31	28	21	12
1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	7	21	47	95	145	187	178	133	95	71	57	48	42	34	29	23	16
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	4	13	45	97	162	180	138	99	74	58	49	38	32	25	18
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	8	31	89	161	174	133	97	72	58	42	34	26	18
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	5	29	98	160	169	129	95	71	48	37	28	19
IA/P = 0.50														* * * TC =1.5 HR * * *														IA/P = 0.50				
0.0	0	0	0	0	0	0	0	3	8	16	27	42	59	92	115	128	130	121	112	100	90	78	67	60	55	50	46	43	39	35	29	4
.10	0	0	0	0	0	0	0	2	6	12	22	35	51	84	110	125	128	123	114	102	91	79	68	61	55	50	46	43	39	35	29	4
.20	0	0	0	0	0	0	0	0	1	4	10	18	29	60	91	114	126	128	120	108	97	85	71	63	57	52	47	44	40	36	29	5
.30	0	0	0	0	0	0	0	0	1	3	8	14	24	52	83	108	123	126	122	110	98	85	72	63	57	52	48	44	40	36	29	6
.40	0	0	0	0	0	0	0	0	0	1	2	6	12	31	60	90	112	124	126	116	104	90	75	66	59	54	49	45	41	37	29	8
.50	0	0	0	0	0	0	0	0	0	0	2	4	9	26	53	83	106	121	125	118	106	91	77	67	60	54	49	46	41	37	29	8
.75	0	0	0	0	0	0	0	0	0	0	1	2	5	16	36	62	88	108	119	122	112	97	81	69	62	56	51	47	42	38	30	11
1.0	0	0	0	0	0	0	0	0	0	0	0	0	0	3	10	26	49	75	98	118	121	108	90	76	66	59	54	49	43	39	31	16
1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	3	11	25	45	80	107	118	106	89	75	65	59	53	45	41	32	23
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	4	11	32	63	100	115	104	87	74	65	58	48	42	34	26
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	4	16	48	94	113	105	89	76	66	53	45	36	27
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	3	15	54	96	111	103	88	75	58	48	38	28

RAINFALL TYPE = II * * * TC = 1.5 HR * * * SHEET 9 OF 10

Revision	Date

NOTE: This Table is only valid for the SCS Type II Rainfall Distribution.

TABULAR HYDROGRAPH METHOD
UNIT DISCHARGES

HYDROGRAPH TIME (HOURS)

TRVL TIME (HR)	11.0	11.3	11.6	11.9	12.0	12.1	12.2	12.3	12.4	12.5	12.6	12.7	12.8	13.0	13.2	13.4	13.6	13.8	14.0	14.3	14.6	15.0	15.5	16.0	16.5	17.0	17.5	18.0	19.0	20.0	22.0	26.0
IA/P = 0.10														* * * TC =2.0 HR * * *															IA/P = 0.10			
0.0	7	9	12	16	18	21	27	36	49	64	82	104	127	171	201	226	208	193	171	132	105	79	58	45	36	30	26	23	20	17	13	3
.10	6	8	10	14	15	17	20	25	33	43	57	74	94	139	179	204	218	205	188	150	118	88	63	48	38	32	27	24	20	17	13	4
.20	6	8	10	13	14	16	19	23	29	39	51	66	84	128	169	198	213	207	192	157	123	91	65	49	39	33	28	24	20	17	13	4
.30	6	7	9	12	14	15	18	21	27	35	45	59	76	117	159	191	211	208	196	163	128	95	68	51	40	33	28	25	20	18	13	4
.40	5	6	8	11	12	13	15	17	20	24	31	41	53	87	128	167	197	209	205	180	145	106	73	55	43	35	30	26	21	18	14	5
.50	5	6	8	10	11	13	14	16	18	22	28	37	48	78	118	158	190	208	208	183	151	111	77	57	44	36	30	26	21	18	14	5
.75	4	6	7	9	10	11	12	13	15	18	22	27	35	58	91	129	164	191	202	194	167	125	87	63	48	38	32	27	22	18	14	6
1.0	3	4	6	7	8	8	9	10	11	12	14	16	18	28	46	74	110	147	178	201	193	156	108	76	56	43	35	30	23	19	14	8
1.5	2	3	3	5	5	5	6	6	7	8	8	9	10	12	16	23	36	57	86	137	178	195	160	113	79	58	45	36	26	21	16	11
2.0	1	2	2	3	3	4	4	4	5	5	6	6	7	8	10	12	16	23	35	67	112	169	190	154	110	78	57	44	30	23	17	11
2.5	0	1	1	2	2	2	3	3	3	4	4	4	5	6	7	8	9	12	16	28	52	105	170	185	149	107	76	56	35	26	18	12
3.0	0	0	1	1	1	1	1	2	2	2	2	3	3	3	4	5	6	7	8	12	18	41	99	161	180	152	112	80	45	30	19	12
IA/P = 0.30														* * * TC =2.0 HR * * *															IA/P = 0.30			
0.0	0	0	0	0	0	1	3	8	15	25	38	54	74	115	140	168	185	170	159	131	110	89	70	57	49	42	38	34	29	26	20	5
.10	0	0	0	0	0	0	0	2	6	12	21	32	47	85	124	153	169	180	168	145	120	96	75	60	51	44	39	35	30	26	20	6
.20	0	0	0	0	0	0	0	2	4	10	17	27	41	75	114	146	165	175	170	149	124	99	76	62	52	45	39	35	30	27	21	6
.30	0	0	0	0	0	0	0	0	1	3	7	14	25	49	86	122	151	170	174	160	136	107	82	66	54	47	41	37	31	27	21	8
.40	0	0	0	0	0	0	0	0	1	2	6	11	19	43	77	113	144	165	173	163	140	111	85	67	55	47	41	37	31	27	21	8
.50	0	0	0	0	0	0	0	0	1	2	4	9	16	37	68	104	136	160	171	165	144	114	87	69	56	48	42	37	31	27	21	9
.75	0	0	0	0	0	0	0	0	0	0	1	2	5	15	34	62	96	127	152	167	160	132	100	77	62	52	45	40	32	28	22	11
1.0	0	0	0	0	0	0	0	0	0	0	0	0	0	3	10	24	48	79	111	150	166	153	118	90	71	58	49	43	34	29	23	14
1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	3	10	24	45	88	130	161	148	115	88	70	57	48	37	31	24	17
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	4	10	32	68	122	157	143	113	87	68	56	42	34	26	18
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	4	16	51	114	153	144	116	89	70	49	38	27	19
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	3	15	59	118	150	140	113	88	57	42	29	19
IA/P = 0.50														* * * TC =2.0 HR * * *															IA/P = 0.50			
0.0	0	0	0	0	0	0	0	1	4	8	13	20	28	51	73	92	104	111	112	106	97	86	75	66	60	54	49	46	41	37	30	7
.10	0	0	0	0	0	0	0	1	3	6	11	17	24	45	68	87	101	109	112	107	98	88	76	67	60	55	50	46	41	37	30	8
.20	0	0	0	0	0	0	0	1	2	5	9	14	21	40	62	82	98	107	111	108	100	89	77	68	61	55	50	47	41	37	30	8
.30	0	0	0	0	0	0	0	0	0	2	4	7	12	26	46	67	86	100	108	111	104	93	80	70	63	57	52	48	42	38	30	10
.40	0	0	0	0	0	0	0	0	0	1	3	6	10	22	41	62	81	96	106	110	105	94	81	71	63	57	52	48	42	38	30	11
.50	0	0	0	0	0	0	0	0	0	0	1	2	4	13	27	46	67	95	99	110	108	98	85	74	66	59	54	49	43	39	31	13
.75	0	0	0	0	0	0	0	0	0	0	0	1	2	7	18	33	52	71	88	104	108	102	89	77	68	61	55	50	44	39	31	15
1.0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	5	13	25	43	62	87	103	108	97	84	73	65	59	53	45	41	32	20
1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	5	12	24	48	74	99	106	95	83	72	64	58	48	43	34	25
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	5	17	37	69	99	104	94	82	72	64	52	45	36	27
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	8	27	65	95	102	95	83	73	58	49	38	28
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	8	32	68	95	101	93	82	64	52	40	28
RAINFALL TYPE = II														* * * TC = 2.0 HR * * *															SHEET 10 OF 10			

NOTE: This Table is only valid for the SCS Type II Rainfall Distribution.

Revision	Date

REFERENCE:
SCS TR-55, USDA, June 1986.

LAG EQUATION ROUGHNESS FACTORS

WATERSHED CHARACTERISTICS	ROUGHNESS FACTOR, K_n
Urbanized Areas: Water courses in the drainage area consist of street, storm sewer, and improved channels.	.015
Natural Areas: Water courses in the drainage area are well defined, unimproved channels or washes. Watershed has minimal vegetation.	.030
Natural Areas: Water courses in the drainage area are not well defined, and consist of many small rills and braided wash areas. Runoff from area combines slowly into channels. Includes mountainous channels with large boulders and flow restrictions.	.050

Revision	Date

RESISTANCE FACTOR FOR OVERLAND FLOW

Surface	N Value	Source
Asphalt/Concrete*	0.05 - 0.15	a
Bare Packed Soil Free of Stone	0.10	c
Fallow - No Residue	0.008 - 0.012	b
Conventional Tillage - No Residue	0.06 - 0.12	b
Conventional Tillage - With Residue	0.16 - 0.22	b
Chisel Plow - No Residue	0.06 - 0.12	b
Chisel Plow - With Residue	0.10 - 0.16	b
Fall Disking - With Residue	0.30 - 0.50	b
No Till - No Residue	0.04 - 0.10	b
No Till (20 - 40 percent residue cover)	0.07 - 0.17	b
No Till (60 - 100 percent residue cover)	0.17 - 0.47	b
Sparse Rangeland With Debris:		
0 Percent Cover	0.09 - 0.34	b
20 Percent Cover	0.05 - 0.25	b
Sparse Vegetation	0.053 - 0.13	f
Short Grass Prairie	0.10 - 0.20	f
Poor Grass Cover On Moderately Rough	0.30	c
Bare Surface		
Light Turf	0.20	a
Average Grass Cover	0.4	c
Dense Turf	0.17 - 0.80	a, c, e, f
Dense Grass	0.17 - 0.30	d
Bermuda Grass	0.30 - 0.48	d
Dense Shrubbery and Forest Litter	0.4	d

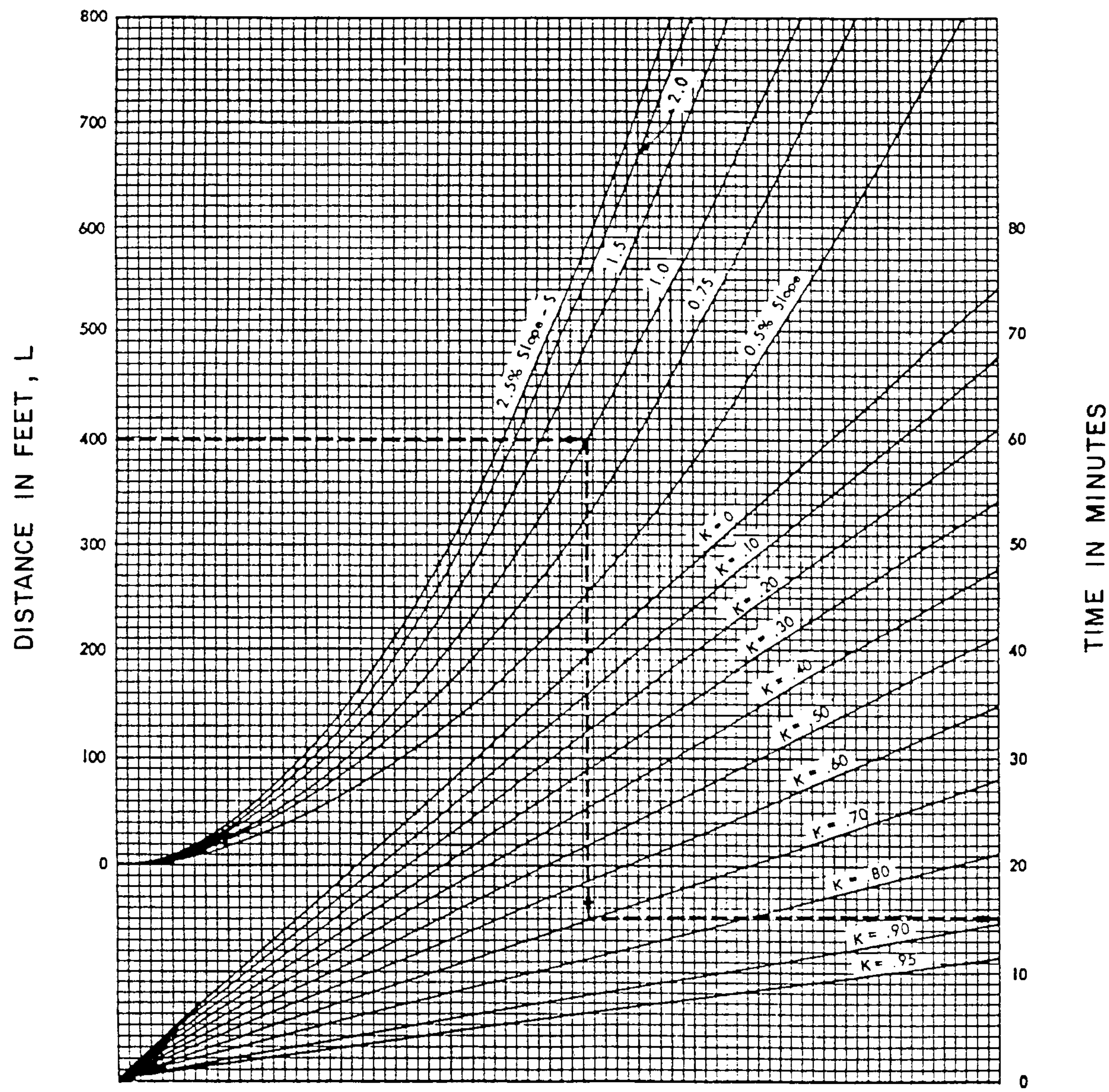
Legend: a) Harley (1975), b) Engman (1986), c) Hathaway (1945), d) Palmer (1946), e) Ragan and Duru (1972), f) Woolhiser (1975). (See Hjemfelt, 1986)

*Asphalt/Concrete n value for open channel flow 0.01 - 0.016

Note: These values are factors for using the Kinematic Routing Method, and shall not be used for Manning's "n" for open channel flow.

Revision	Date

OVERLAND TIME OF FLOW



THE ABOVE CURVES ARE A SOLUTION OF THE FOLLOWING EQUATION:

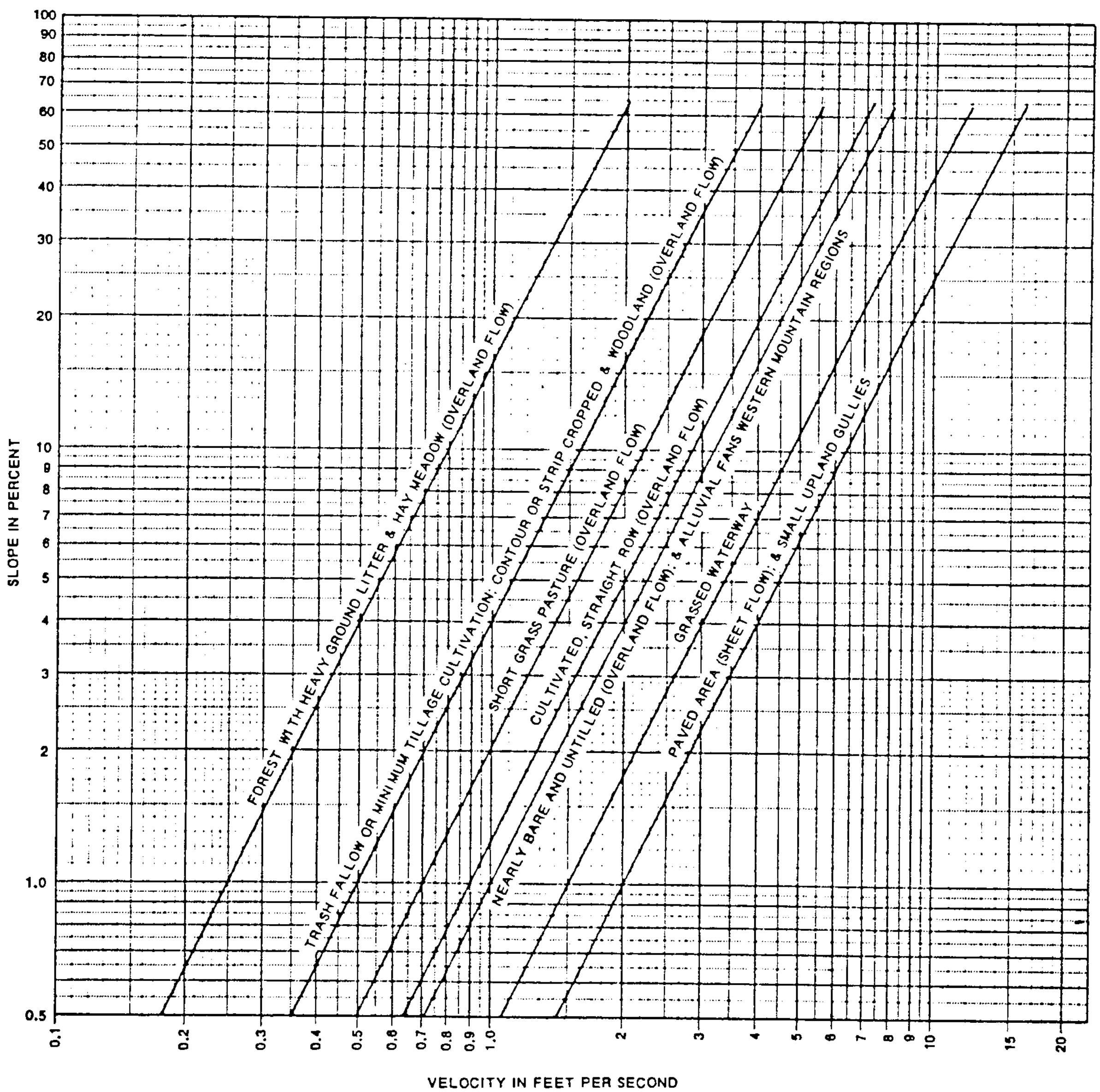
$$t_i = \frac{1.8\,(1.1 - K)\,\sqrt{L}}{\sqrt[3]{S}}$$

where: t_i = initial flow time (min.)
S = slope of basin (%)
K = flow resistance coefficient
L = length of basin (ft.)

Notes: 1. The curves shall not be used for distances in excess of 500'.

Revision	Date

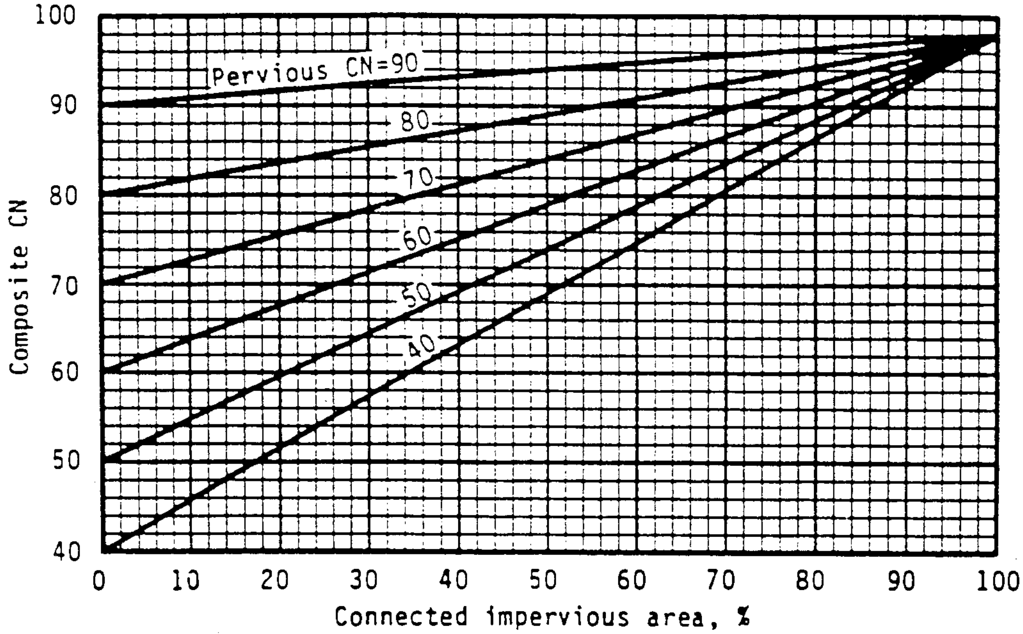
TRAVEL TIME VELOCITY



Revision	*Date*

WRC ENGINEERING	**REFERENCE:** SCS National Engineering Handbook Section 4, Hydrology, March, 1985	FIGURE 602

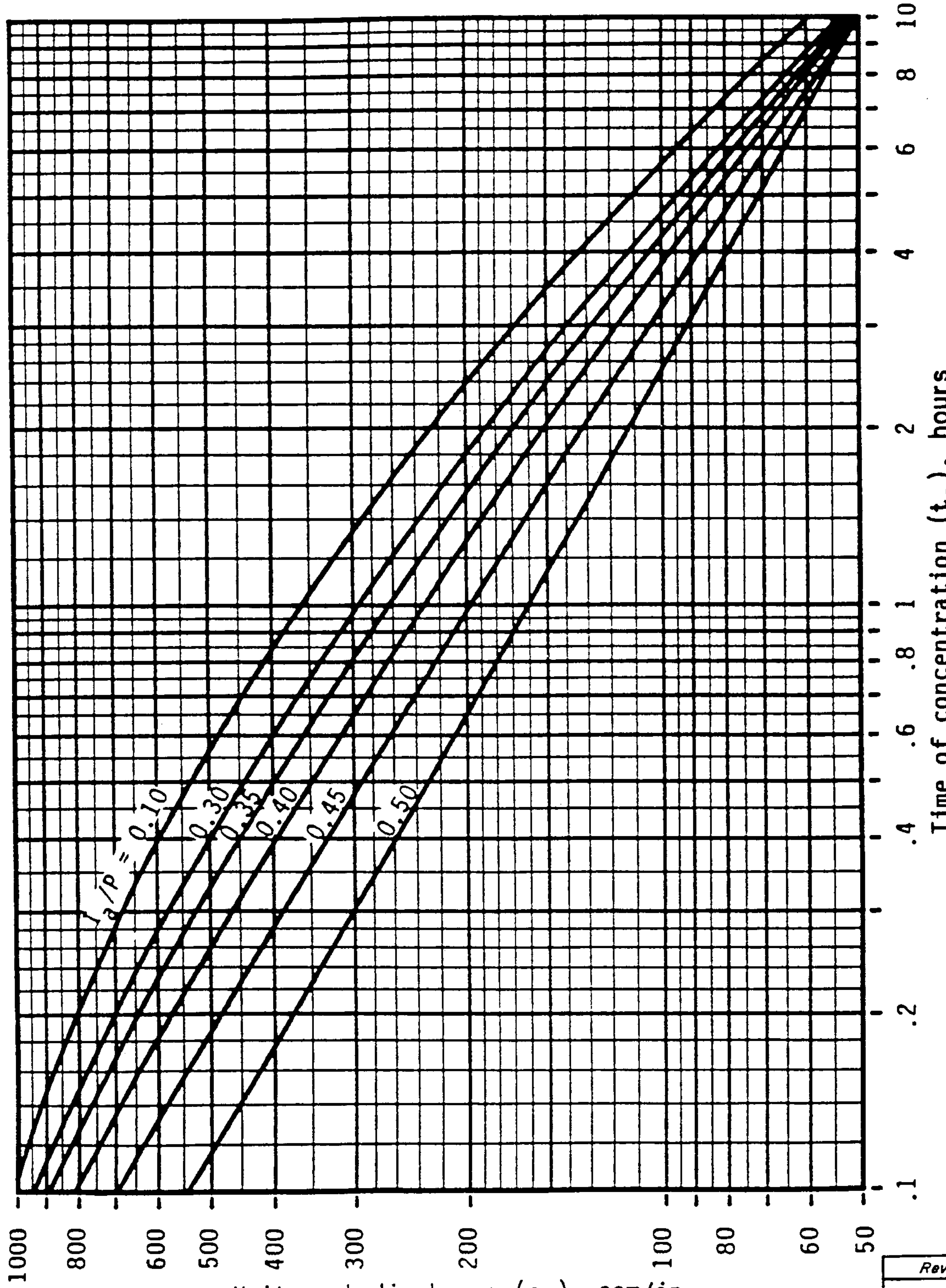
PERCENTAGE OF IMPERVIOUS AREA VS. COMPOSITE CN'S FOR GIVEN PERVIOUS AREA CN'S



NOTE: 1. Refer to Table 602 for CN values for various land use conditions.

Revision	Date

GRAPHICAL PEAK DISCHARGE METHOD
UNIT PEAK DISCHARGE

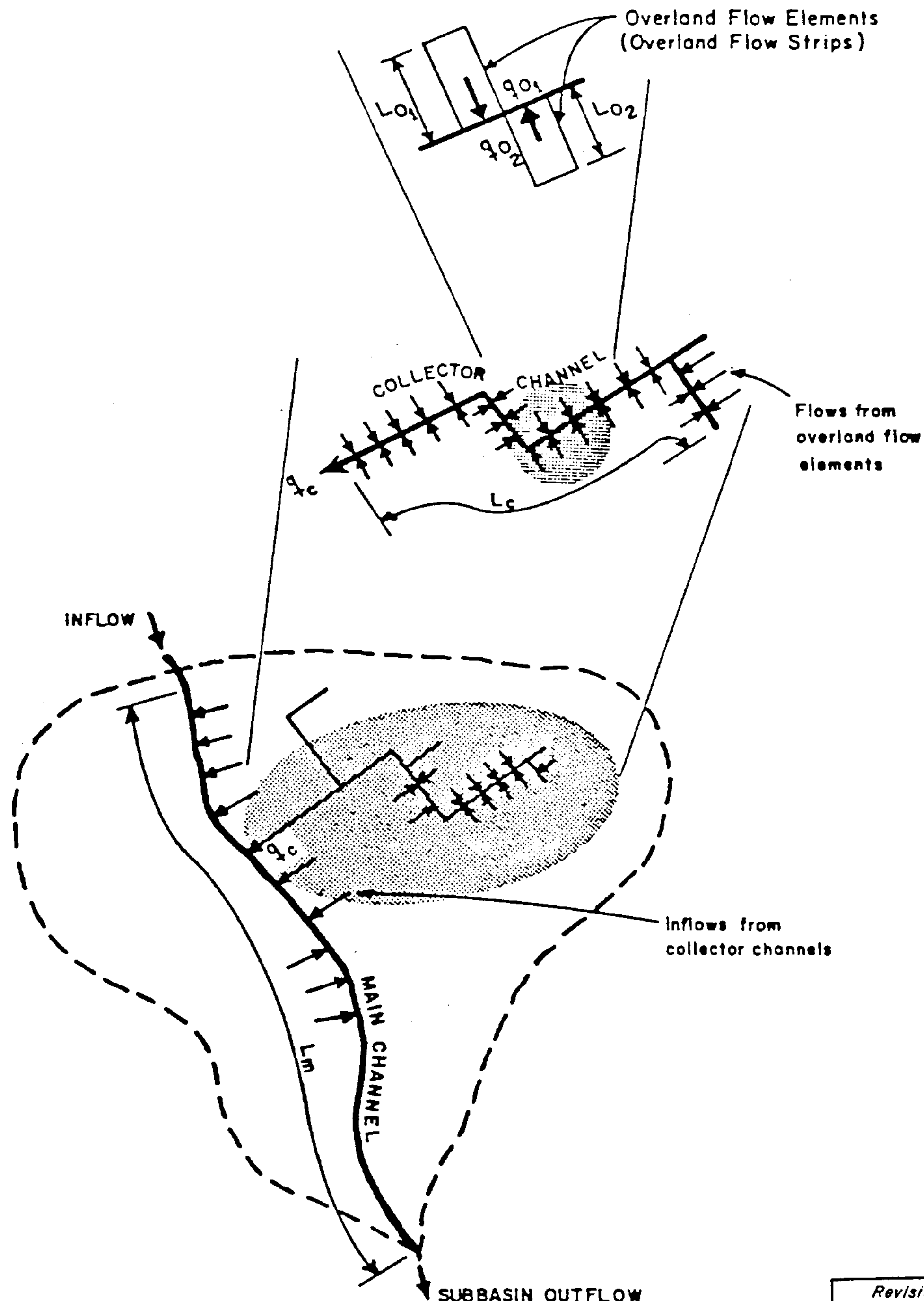


NOTE: This figure is only valid for the SCS Type II Rainfall Distribution.

Revision	Date

WRC ENGINEERING	REFERENCE: SCS TR-55, USDA, June 1986.	FIGURE 604

RELATIONSHIP BETWEEN FLOW ELEMENTS

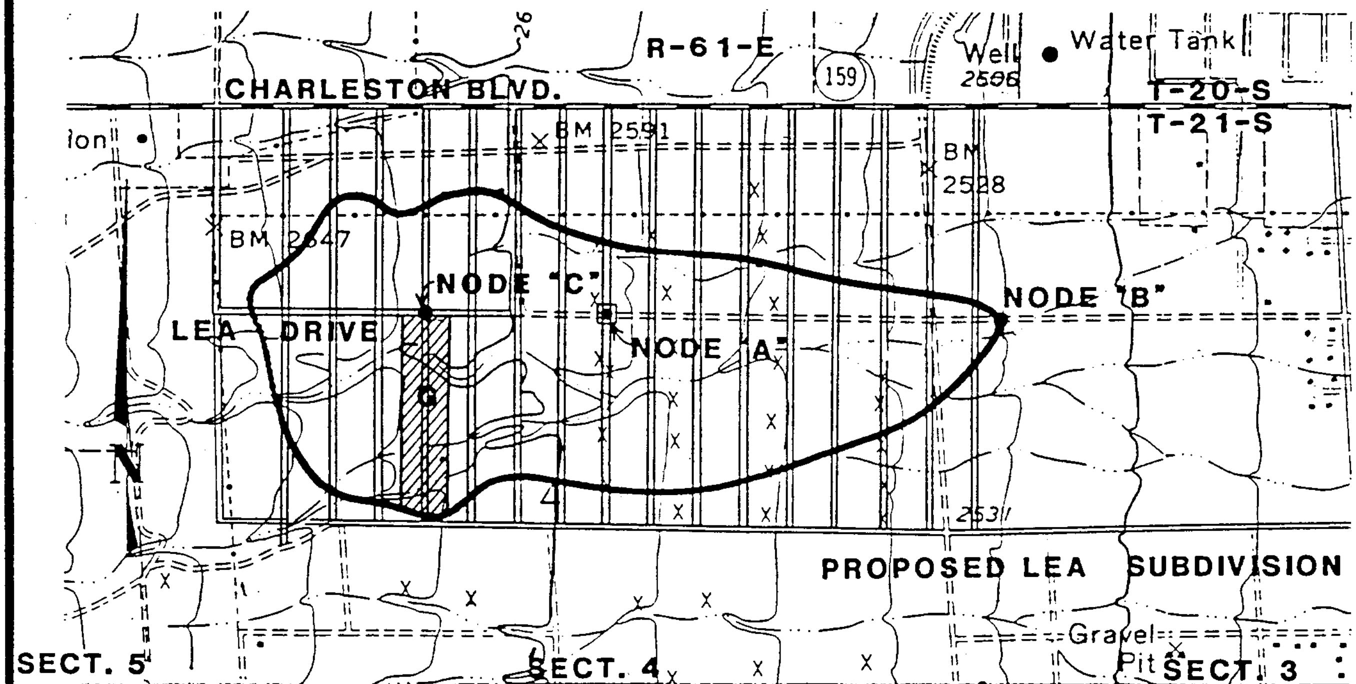


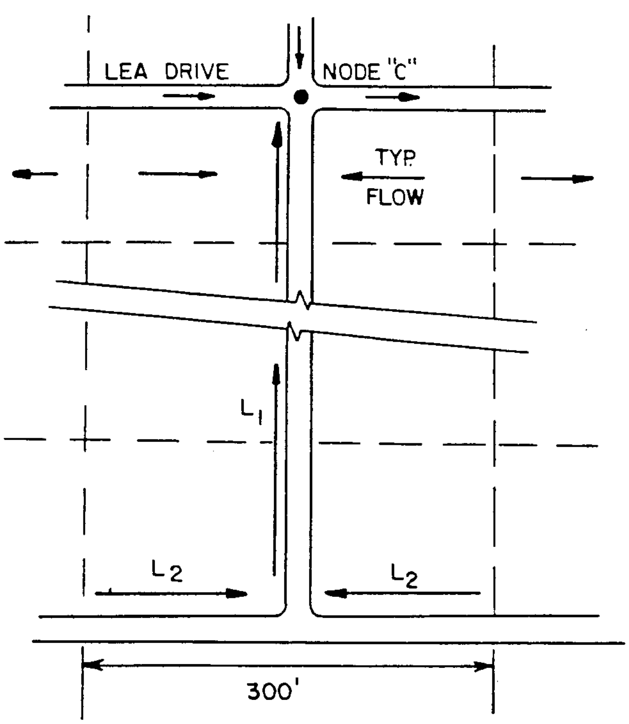
Revision	Date

REFERENCE: HEC-1 Users Manual, September 1981, (Revised March 1987)

FIGURE 605

HYPOTHETICAL URBAN BASIN FOR EXAMPLES IN SECTION 611.1 AND 611.2





SUB-BASIN G

SUB-BASIN "G"

L_1 = 1200' Flow in residential collector street.

L_2 = 150' Longest overland flow length (Typ.)

Average Slope = 2.5%

Land Use: 6.3 AC. single family, 30% impervious with lawns

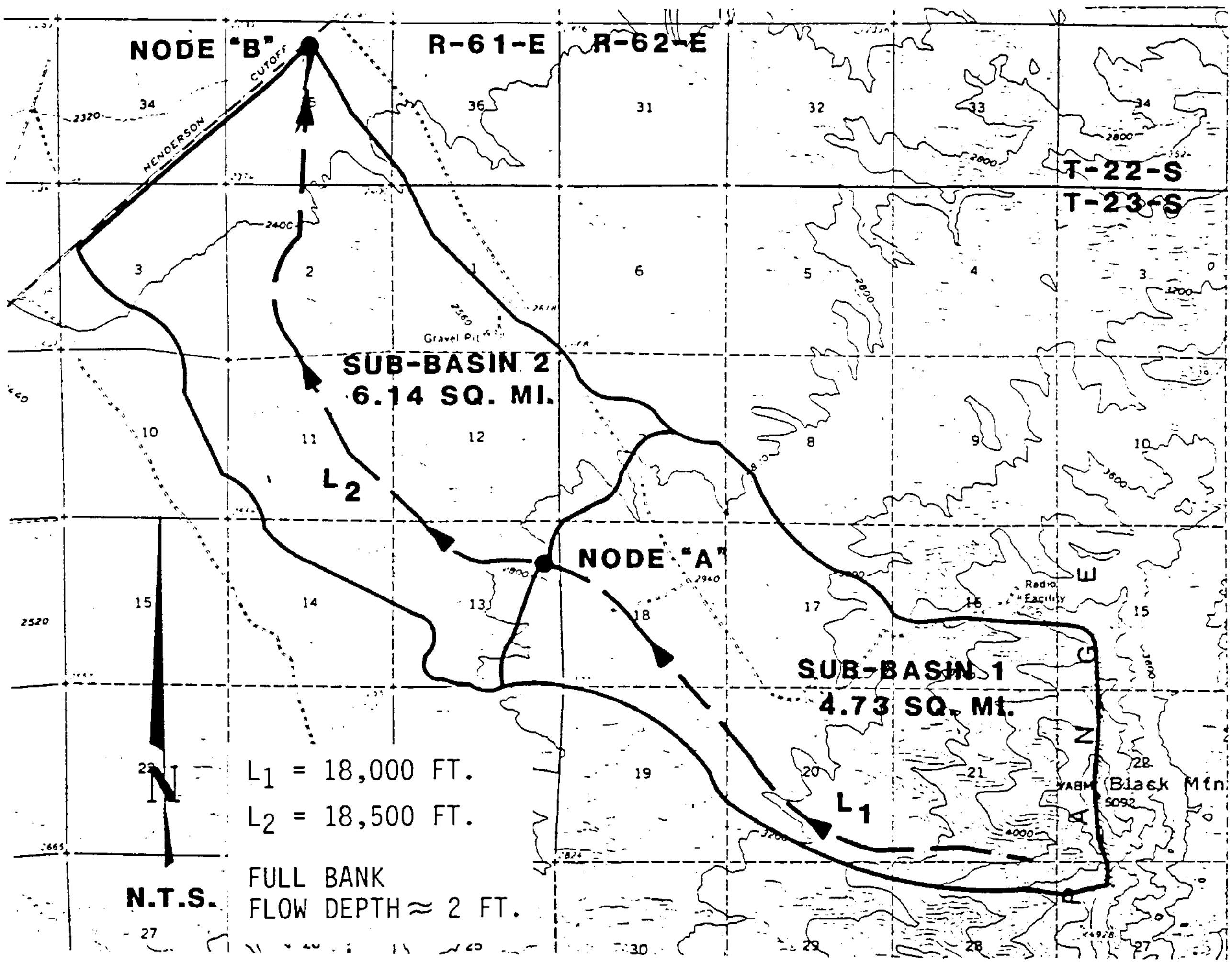
2 AC. of neighborhood commercial land

——→ Typical flow direction

— — — Lot Lines

Revision	Date

HYPOTHETICAL NON-URBAN BASIN FOR EXAMPLE IN SECTION 611.3



BASIN DESCRIPTION:

For examples in Section 611,consider the above basin as an offsite drainage area to a proposed development. The land is descibed as Rangeland in poor condition with the predominant SCS Hydrologic Soils Group of "D". During a flood event the main flow path (L_1 & L_2) would allow for some storage in the overbank area with minor obstructions in the flow path.

Revision	Date

TIME OF CONCENTRATION CALCULATION FOR EXAMPLE IN SECTION 611.1

HYDROLOGIC CRITERIA AND DRAINAGE DESIGN MANUAL

TIME OF CONCENTRATION AND HEC-1 INPUT

DEVELOPMENT Hypothetical Basin-Example 608.1

CALCULATED BY Montgomery Watson DATE January 29, 1998

Sub-Basin Data				Initial/Overland Time (t_i)			Travel Time (t_t)				tc Check (Urbanized Basins)		Final t_c			Remarks
KK			BA											UD	LS	HEC-1 Card
Desig: (1)	K (2)	Area Ac (3)	Area sq mi (4)	Length Ft (5)	Slope % (6)	t_i Min (7)	Length Ft (8)	Slope % (9)	Vel. FPS (10)	t_t Min (11)	Total Length Ft (12)	t_c = (L/180) + 10 Min (13)	Min (14)	LagTime hr (15)	CN (16)	
G	0.55	8.3	0.013	150	2.5	8.9	1200	2.5	3.2	6.3	1350	17.5	15.2	0.152		Sub-Basin G

$$t_i = 1.8\,(1.1 - K)\,L^{1.2}/S^{1/3}$$

$$t_c = t_i + t_t$$

Revision	Date

REFERENCE:

STANDARD FORM 4

Revision	Date

REFERENCE:

FIGURE 608

HEC-1 RUN FOR EXAMPLE IN SECTION 611.3

```
                                   HEC-1 INPUT                                          PAGE  1

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

               *DIAGRAM
   1           ID        CLARK COUNTY REGIONAL FLOOD CONTROL DISTRICT
   2           ID        SECTION 611.3  EXAMPLE:     SCS UNIT HYDROGRAPH METHOD
   3           ID                                    AND MUSKINGUM CHANNEL ROUTING METHOD
   4           ID        100-YR, 6-HR STORM
   5           ID        MONTGOMERY WATSON,      9/2/99
   6           IT      05       0       0     300
   7           IN       5       0       0
   8           IO       5       0       0

   9           KK    SUB1       SUB-BASIN 1 HYDROGRAPH STORM DISTRIBUTION NUMBER 4
  10           KM               SCS RUNOFF COMPUTATION SUB-1
  11           BA    4.73
  12           PB    2.51
  13           PC     0.0     2.0     5.8     7.5     9.9    12.6    13.7    14.5    14.9    15.1
  14           PC    15.5    15.6    15.9    16.2    16.9    17.2    17.9    18.9    20.1    21.1
  15           PC    22.0    22.8    23.2    24.0    24.6    25.2    26.0    26.9    27.6    28.3
  16           PC    28.6    29.2    30.2    31.2    32.1    33.2    35.2    37.6    41.5    46.2
  17           PC    53.0    61.0    71.0    73.2    75.6    78.2    79.9    81.3    82.3    83.0
  18           PC    83.5    84.4    85.1    86.4    88.5    90.8    92.4    94.4    96.8    97.3
  19           PC    97.8    98.2    98.4    98.6    98.8    99.1    99.2    99.3    99.4    99.7
  20           PC    99.8    99.9   100.0
  21           LS       0      88
  22           UD    0.63

  23           KK   NODEB         ROUTE FLOW TO NODE B
  24           KM                 USE THE MUSKINGUM METHOD TO ROUTE TO THE NEXT DESIGN POINT
  25           RM       2    0.38    0.13

  26           KK    SUB2       SUB-BASIN 2 HYDROGRAPH STORM DISTRIBUTION NUMBER 4
  27           KM               SCS RUNOFF COMPUTATION SUB-2
  28           BA    6.14
  29           PB    2.51
  30           LS       0      88
  31           UD    0.59

  32           KK   NODEB         COMBINE FLOW AT NODE B
  33           HC       2
  34           ZZ
```

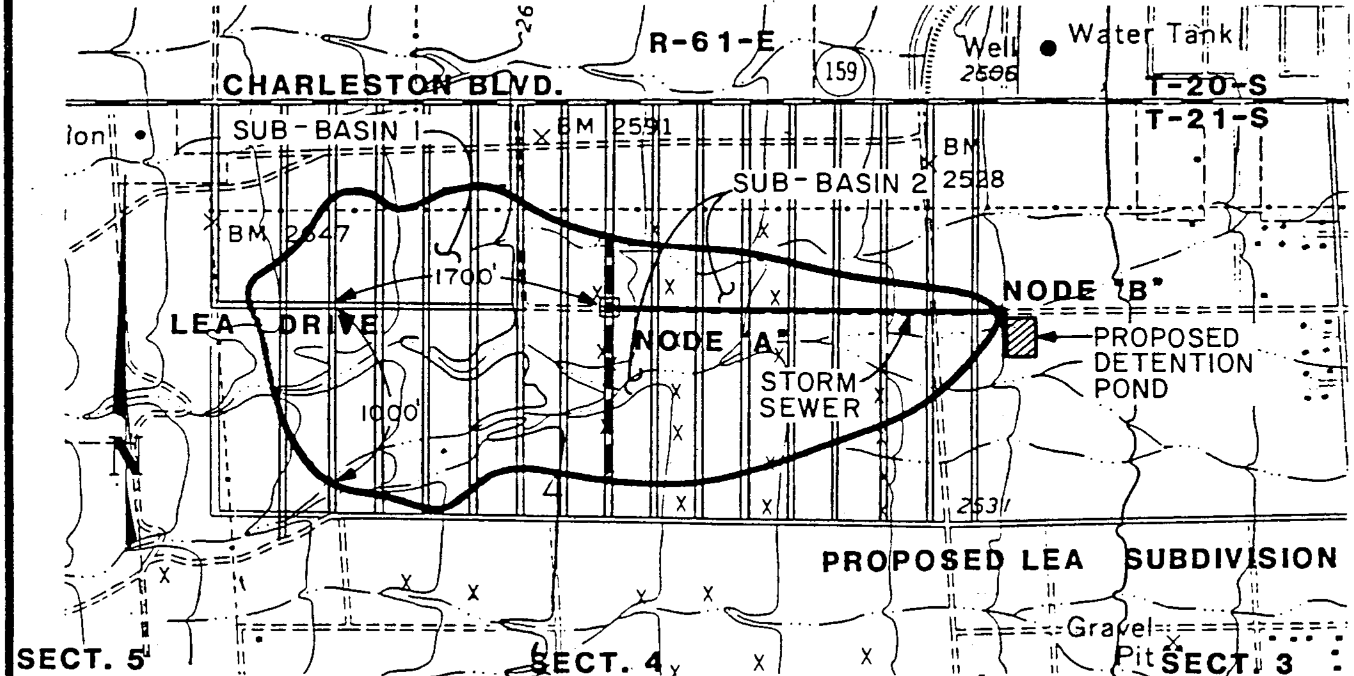
RUNOFF SUMMARY
FLOW IN CUBIC FEET PER SECOND
TIME IN HOURS, AREA IN SQUARE MILES

OPERATION	STATION	PEAK FLOW	TIME OF PEAK	AVERAGE FLOW FOR MAXIMUM PERIOD 6-HOUR	24-HOUR	72-HOUR	BASIN AREA	MAXIMUM STAGE	TIME OF MAX STAGE
HYDROGRAPH AT	SUB1	2450.	4.08	703.	177.	59.	4.73		
ROUTED TO	NODE	2182.	4.42	703.	177.	59.	4.73		
HYDROGRAPH AT	SUB2	3303.	4.00	914.	229.	76.	6.14		
2 COMBINED AT	NODE	4979.	4.17	1615.	406.	135.	10.87		

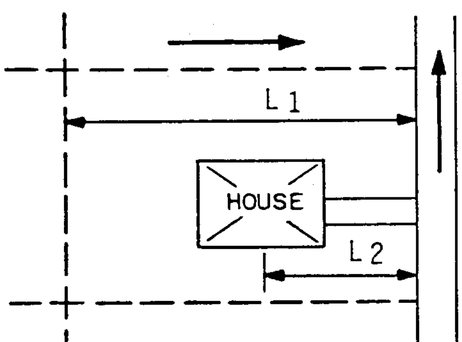
Revision	Date

REFERENCE:

HYPOTHETICAL URBAN BASIN FOR EXAMPLES IN SECTIONS 611.4 AND 611.5



A) TYPICAL RESIDENTAL LOT



L_1 = Pervious Flow Length = 150'

L_2 = Impervious Flow Length = 50'

Irrigated Lawns

Flow Direction

Lot Line

A_1 = 80 Acres

A_2 = 80 Acres

B) LAND USE FOR PROPOSED DEVELOPMENT

1. 40 Percent residential area that is 30 percent impervious
2. 12 Percent residential area that is 65 percent impervious
3. 8 Percent paved roads with open ditches
4. 10 Percent paved roads with curbs and storm sewers
5. 16 Percent open land with 50 percent fair cover and 50 percent good cover
6. 14 Percent parking lots, plaza, schools, etc. (all impervious)
7. Hydrologic soil group: 50% B, 50% C
8. Landscaping and open space (parks) is irrigated grass.

Revision	Date

HEC-1 RUN FOR EXAMPLE IN SECTION 611.5

```
                                   HEC-1 INPUT                                             PAGE  1

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

  1           ID     CLARK COUNTY REGIONAL FLOOD CONTROL DISTRICT
  2           ID     SECTION 611.5  EXAMPLE:  KINIMATIC WAVE METHOD
  3           ID     100 YEAR, 6 HOUR STORM
  4           ID     WRC ENGINEERING, INC.          1/26/90
  5           IT         5       0       0     300
  6           IO         2       0       0

  7           KK     SUB 1  SUB-BASIN 1 HYDROGRAPH
  8           KM     100-YEAR STORM  SDN=3
  9           BA      .125
 10           PB      2.74
 11           PC         0    2.00    5.70    7.00    8.70   10.80   12.40   13.00   13.00   13.00
 12           PC     13.00   13.00   13.00   13.30   14.00   14.20   14.80   15.80   17.20   18.10
 13           PC     19.00   19.70   19.70   20.10   20.10   20.40   21.40   22.90   24.10   24.90
 14           PC     25.10   25.60   27.00   27.80   28.10   28.30   29.50   32.20   35.20   40.90
 15           PC     49.90   59.00   71.00   74.40   78.10   81.20   81.90   83.50   85.10   85.60
 16           PC     86.00   86.80   87.60   88.80   91.00   92.60   93.70   95.00   97.00   97.60
 17           PC     98.20   98.50   98.70   98.90   99.00   99.30   99.30   99.40   99.50   99.80
 18           PC     99.80   99.90  100.00
 19           LS         0      79       0       0      98
 20           UK       150    .025     .25      52
 21           UK        50    .025     .10      48
 22           RK       900    .025    .015    .008    TRAP       0      50
 23           RK      2000    .025    .015            TRAP       0      50

 24           KK         B ROUT FLOW FROM NODE A TO NODE B
 25           RK      2550    .025    .013            CIRC       4

 26           KK     SUB 2  SUB-BASIN 2 HYDROGRAPH
 27           KM     100-YEAR STORM
 28           BA      .125
 29           PB      2.74
 30           LS         0      77       0       0      98
 31           UK       150    .025     .25      52
 32           UK        50    .025     .10      48
 33           RK       600    .025    .015    .008    TRAP       0      50
 34           RK      2200    .025    .015            TRAP       0      50

 35           KK         B  COMBINE FLOW AT NODE B
 36           HC         2
 37           ZZ
```

Revision	Date

HEC-1 RUN FOR EXAMPLE IN SECTION 611.5

```
                **************
                *            *
35 KK           *         B  *     COMBINE FLOW AT NODE B
                *            *
                **************

36 HC                 HYDROGRAPH COMBINATION
                          ICOMP          2   NUMBER OF HYDROGRAPHS TO COMBINE
                                                         ***
```

****

HYDROGRAPH AT STATION B
SUM OF 2 HYDROGRAPHS

****

DA	MON	HRMN	ORD	FLOW	*	DA	MON	HRMN	ORD	FLOW	*	DA	MON	HRMN	ORD	FLOW	*	DA	MON	HRMN	ORD	FLOW
1		0000	1	0.	*	1		0615	76	5.	*	1		1230	151	0.	*	1		1845	226	0.
1		0005	2	0.	*	1		0620	77	4.	*	1		1235	152	0.	*	1		1850	227	0.
1		0010	3	0.	*	1		0625	78	4.	*	1		1240	153	0.	*	1		1855	228	0.
1		0015	4	3.	*	1		0630	79	3.	*	1		1245	154	0.	*	1		1900	229	0.
1		0020	5	14.	*	1		0635	80	3.	*	1		1250	155	0.	*	1		1905	230	0.
1		0025	6	24.	*	1		0640	81	2.	*	1		1255	156	0.	*	1		1910	231	0.
1		0030	7	32.	*	1		0645	82	2.	*	1		1300	157	0.	*	1		1915	232	0.
1		0035	8	33.	*	1		0650	83	2.	*	1		1305	158	0.	*	1		1920	233	0.
1		0040	9	25.	*	1		0655	84	2.	*	1		1310	159	0.	*	1		1925	234	0.
1		0045	10	16.	*	1		0700	85	1.	*	1		1315	160	0.	*	1		1930	235	0.
1		0050	11	10.	*	1		0705	86	1.	*	1		1320	161	0.	*	1		1935	236	0.
1		0055	12	6.	*	1		0710	87	1.	*	1		1325	162	0.	*	1		1940	237	0.
1		0100	13	4.	*	1		0715	88	1.	*	1		1330	163	0.	*	1		1945	238	0.
1		0105	14	3.	*	1		0720	89	1.	*	1		1335	164	0.	*	1		1950	239	0.
1		0110	15	2.	*	1		0725	90	1.	*	1		1340	165	0.	*	1		1955	240	0.
1		0115	16	3.	*	1		0730	91	1.	*	1		1345	166	0.	*	1		2000	241	0.
1		0120	17	5.	*	1		0735	92	1.	*	1		1350	167	0.	*	1		2005	242	0.
1		0125	18	8.	*	1		0740	93	1.	*	1		1355	168	0.	*	1		2010	243	0.
1		0130	19	14.	*	1		0745	94	1.	*	1		1400	169	0.	*	1		2015	244	0.
1		0135	20	21.	*	1		0750	95	1.	*	1		1405	170	0.	*	1		2020	245	0.
1		0140	21	23.	*	1		0755	96	0.	*	1		1410	171	0.	*	1		2025	246	0.
1		0145	22	22.	*	1		0800	97	0.	*	1		1415	172	0.	*	1		2030	247	0.
1		0150	23	19.	*	1		0805	98	0.	*	1		1420	173	0.	*	1		2035	248	0.
1		0155	24	14.	*	1		0810	99	0.	*	1		1425	174	0.	*	1		2040	249	0.
1		0200	25	10.	*	1		0815	100	0.	*	1		1430	175	0.	*	1		2045	250	0.
1		0205	26	7.	*	1		0820	101	0.	*	1		1435	176	0.	*	1		2050	251	0.

RUNOFF SUMMARY
FLOW IN CUBIC FEET PER SECOND
TIME IN HOURS, AREA IN SQUARE MILES

OPERATION	STATION	PEAK FLOW	TIME OF PEAK
HYDROGRAPH AT	SUB 1	166.	3.50
ROUTED TO	B	157.	3.50
HYDROGRAPH AT	SUB 2	161.	3.50
2 COMBINED AT	B	318.	3.50

Revision	Date

CLARK COUNTY REGIONAL FLOOD CONTROL DISTRICT
HYDROLOGIC CRITERIA AND DRAINAGE DESIGN MANUAL

SECTION 700
OPEN CHANNELS

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Section 700
Open Channels

701 INTRODUCTION

Presented in this section is the technical criteria and design standards for the hydraulic evaluation and design of open channels. Discussion and standards are provided for the many various channel linings and design sections anticipated to be encountered or used in the Clark County region. Since the evaluation and design of many of the channel sections can vary greatly depending on site conditions, the ultimate responsibility for a safe channel design rests with the designer. The information presented in this section should be considered to be the minimum standards on which channel evaluation and design should be based. Additional analysis beyond the scope of this MANUAL may be necessary for unique or unusual channel conditions. In addition, the local entities or the CCRFCD may require submittal of additional design and analysis information for any of the said channel sections and linings in order to assess the adequacy of the design for the proposed application. Therefore, the designer is recommended to contact the local entity and, if necessary, the CCRFCD prior to design of an open channel to discuss the proposed channel section and lining selection and to obtain any additional requirements (if any) for the selected channel. If the designer is proposing a different channel design than presented in this section, the local entity and/or the CCRFCD must be contacted prior to designing the channel.

702 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. An example of these flow conditions is illustrated in **Figure 701.** 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.

702.1 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 one possible depth for maintaining uniform flow is the normal depth. For a prismatic channel (i.e., uniform cross section) the water surface will be parallel to the channel bottom for uniform flow.

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 Manning's formula as follows:

$$Q = (1.49 / n)R^{2/3}S^{1/2}A \qquad (701)$$

Where Q = Flow Rate (cfs)

n = Roughness Coefficient

A = Area (sq ft)

P = Wetted Perimeter (ft)

R = Hydraulic Radius, A/P

S = Slope of the Energy Grade Line (EGL in ft/ft)

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

Presented in **Table 701** are equations for calculating many of the parameters required for hydraulic analysis of different channel sections. These equations may be used to compute the input parameters to Manning's equation. These parameters and Manning's equation may also be readily computed using hand-held calculators and personal computers.

Manning's "n" roughness coefficients for various channel lining types are presented in **Table 702**. **Table 702** includes "n" values for both improved and natural channels.

For design considerations for channels that are anticipated to convey sediment as well as stormwater, a composite Manning's "n" value shall be determined and used. The composite Manning's "n" value shall take into account the anticipated bedload in the section, as well as the concrete lining above the sediment deposition.

702.2 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 (F_r) 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, because minor changes in specific energy, such as from channel debris, will cause a major change in depth.

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

$$F_r = V / (gD)^{1/2} \qquad (702)$$

Where F_r = Froude Number

V = Velocity (fps)

g = Acceleration of Gravity (ft/sec^2)

D = Hydraulic Depth (ft) = A/T

A = Channel Flow Area (sq ft)

T = Top Width of Flow Area (ft)

The F_r 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 equation:

$$Z = Q / (g)^{1/2} \qquad (703)$$

Where Z = Section Factor

Q = Flow Rate (cfs)

g = Acceleration of Gravity (ft/sec^2)

Once Z is computed and with a given channel bottom width, b, the critical depth in the channel, y, can be determined from **Figure 702.** For other prismatic channel shapes, **Equation 703** above can be used with the section factors, Z, in **Table 701** to determine the critical depth or the critical depth equation below:

$$\frac{\bar{U}^2}{2g} = \frac{D}{2}$$

Where $\bar{U}$ = Average Velocity

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	< 0.86	> 1.1 d_c
Super-Critical	> 1.13	< 0.9 d_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 check the flow state and compliance with the MANUAL.

702.3 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 program is HEC-2 and/or HEC-RAS, water-surface profiles, developed by the USACE and is the program recommended for floodwater profile computations for natural channels and floodplain analysis. For alluvial fan areas, the procedures presented in Section 1400 of this MANUAL shall be used for floodplain analysis.

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. The designer is referred to CHOW, 1959, for the details of the Direct Step method. 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 and/or HEC-RAS is recommended.

702.4 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. Whereas there are mathematical solutions to some specific cases of rapidly varying flow, 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 and orifices, hydraulic jumps, non-prismatic channel sections (transitions, culverts and bridges), and non-linear channel alignments (bends). The discussion of rapidly varying flow for these applications are located in this MANUAL as follows:

Application	Section
Weirs and Orifices	1200 - Detention
Hydraulic Jumps (Channels)	1100 - Additional Hydraulic Structures
Hydraulic Jumps (Conduits)	800 - Storm Sewer Systems
Culverts and Bridges	1000 - Culverts and Bridges
Channel Transitions and Bends	700 - Open Channels

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.

702.5 Transitions

702.51 Introduction

Channel transitions occur in open channel design whenever there is a change in channel slope, 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 minimizing surface disturbances from cross-waves and turbulence. A special case of transitions where excess energy is dissipated by design is drop structures and hydraulic jumps. Channel drop structures are discussed in Section 1100 (Additional Hydraulic Structures).

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 discussed in Section 706.1 for sub-critical flow. Conditions 3 and 4 will be considered as super-critical transitions and are discussed in Section 706.2 for super-critical flow.

703 MAXIMUM PERMISSIBLE VELOCITIES

The design of all channels in the Clark County area shall be based on maximum permissible velocities. This method of design assumes that the 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 703** are the maximum permissible velocities for natural or improved, unlined and lined channels in the Clark County area. These values shall be used for all channel designs in the Clark County area. If a higher velocity is desired, the design engineer must demonstrate to the satisfaction of the local entity and/or the CCRFCD that the higher velocity would not endanger the health or safety of the public and would not increase operation and maintenance of the channel section. For natural and improved unlined channels, a geotechnical report shall be submitted to the local entity and/or the CCRFCD which addresses the existing soil material classification upon which the maximum permissible velocity was selected. 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 or based on an equilibrium analysis of sediment transport within the channel segment.

The stated maximum permissible velocities are based on flow studies conducted by various governmental agencies and private individuals, 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 to remain stable at the final design flow rate and velocity.

704 DESIGN SECTIONS AND STANDARDS FOR NATURAL CHANNELS

704.1 Introduction

Presented in this section are the typical natural open channel sections which are encountered in the Clark County area. A graphical illustration of the typical design sections is presented in **Figure** 703. The selection of a design section for natural channels is generally dependent on the value of developable land versus the cost to remove the said 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.

The design standards presented in this section are the minimum standards by which natural channel analysis and design shall be completed within the CCRFCD. The channel designer is reminded that the ultimate responsibility for a safe 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 section. In addition, the local entity and/or the CCRFCD may require additional design analysis be performed to verify the suitability of the proposed design for the location under consideration.

For natural channel sections, the engineer shall verify through stable channel (normal depth) calculations the suitability of the floodplain to contain the major storm flows. If this analysis demonstrates erosion outside of the designated flow path (easement and/or ROW), then the analysis in Section 704.2 is required.

704.1.1 Natural Unencroached 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 wash or channel. Although the development does not alter the flow carrying capacity of the wash, the concern is to protect the development 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 which 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 headcutting and subsequent bank failures.

704.1.2 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 fill in the floodplain fringe which maintains or decreases the water surface. The designer must prepare a design which 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.

704.1.3 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 which addresses scour depths at the lining interface to assure that the lining extends below this depth to avoid undermining of the lining.

704.1.4 Partially Lined Channels

Partially lined channels are defined as channels in which half of the channel is completed and the 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 be allowed if:

a. The bottom paving is bonded or other mechanism in place to pay for the bottom paving once the channel is completed.

b. Erosion in the unlined section must be addressed.

c. Scour below the lining must be addressed.

The design must be able to state that the proposed temporary channel does not significantly adversely impact the hydraulic parameters and stability of the unlined section.

704.2 Natural Channel Systems

A natural channel system generally is continually changing its position and shape as a consequence to hydraulic forces acting on its bed and banks and related biological forces interacting with these physical forces. These changes may be slow or rapid and may result from natural environmental changes or from changes caused by man's activities. When a natural channel is modified locally, the change frequently causes alterations in channel characteristics both up and downstream. The response of a natural channel to human-induced changes often occurs in spite of attempts to control the natural channel environment.

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 in 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 immediate, delayed, and far-reaching effects of alterations man imposes 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 thereof; (7) geology, including type of sediment; and (8) works of man.

704.2.1 Natural Channel Morphology and Response

The hydraulic and geomorphic response of channels to imposed natural and human-induced changes can be evaluated utilizing different relationships, methods, and levels of analysis. In all cases, it is of value to initially analyze the behavior and response of a system utilizing qualitative geomorphic and hydraulic response relationships. These methods of analysis are simple to apply and even with the most sophisticated methods of analysis, this geomorphic and hydraulic analysis provides a valuable check on the final quantitative results.

704.2.1.1 Slope

The slope of the energy gradient plays an extremely important role in the hydraulics of natural channels. Slope is utilized in velocity equations such as Manning's equation to estimate velocity. It is also utilized in the tractive force equation to estimate the tractive force exerted on the bed and banks of open channels. A long reach of natural channel may be subjected to a general lowering or raising of the bed level over a long period oftime due to changing incoming sediment supply caused by activities such as urbanization, construction of a detention pond, etc. An equilibrium channel slope is defined as the slope at which the channel's sediment transporting capacity is equal to the incoming sediment supply.

That is, $(Q_s)_{in} = (Q_s)_{out}$

Where, $(Q_s)_{in}$ = Supply Rate of Sediment into the Channel Reach

$(Q_s)_{out}$ = Supply Rate of Sediment Out of the Channel Reach.

Under this condition, the channel neither aggrades nor degrades.

The equilibrium channel slope is used to predict the wash response to human-induced changes. The evaluation will provide an understanding of what the long-term effects of such measures as channelization or reducing sediment supply due to urbanization will have on the channel profile.

704.2.1.2 - Degradation and Aggradation

A long reach of channel may be subjected to a general degradation or aggradation of the bed level over a long period of time. Degradation and aggradation must be accurately anticipated; otherwise foundation depths may be inadequate or excessive depending on the magnitude of degradation or aggradation.

The basic principle of degradation and aggradation is to compare in a reach, the sediment supply and the sediment transport. When sediment supply is less than sediment transport, the flow will remove additional sediment from the channel bed and banks to eliminate the deficit. This results in degradation of the channel bed and possible failure of the banks. If the supply entering the reach is greater than the capacity, the excess supply will be deposited. Utilization of a sediment routing model (e.g., QUASED by Simons, Li & Associates; HEC-6 by USACE; FLUVIAL by Howard Chang; ONETWOD by Y. H. Chen; see FERC (1992) for reference) of the stream system is the best method of estimating the general degradation and aggradation on a reach by reach basis. However, less elaborate methods using rigid bed hydraulic and sediment transport calculations may be used to estimate the unbalance between sediment transport capacity and sediment supply between adjacent reaches.

The determination of sediment transport as presented in this design standard is based on easy to apply power relationships between sediment transport rate and velocity and depth as follows:

$$q_s = C_1 (Y)^{C_2} (V)^{C_3} \tag{704}$$

where q_s = Sediment Transport Rate in cfs/ft of Width

Y = Flow Depth in ft

V = Flow Velocity in fps

C_1, C_2, C_3 = Constants

Values of C_1, C_2, and C_3 for sand materials are presented in **Table 703A** with limitations noted. These power relations were developed from a computer solution of the Meyer-Peter and Muller bedload transport equation and Einstein's integration of the suspended bed-material discharge (SIMONS, LI & ASSOCIATES, 1982). For flow conditions within the ranges outlined in **Table 703B**, the regression equation should be accurate within 10 percent. This simplified equation with the recommended values for C_1, C_2 and C_3 will provide the design engineer with a reasonable first-order estimate of sediment transport as long as it is used within the specified limits of particle size and flow velocity. The final sediment transport rate should be determined for a variety of flow conditions and sediment sizes likely to occur in the study reach.

Determination of the equilibrium channel slope is a key to determining the stable channel condition. **Equation 704** can be utilized to determine the equilibrium channel slope by combining that with Manning's equation:

$$q = \frac{1.486}{n} (R)^{5/3} (S_e)^{1/2} \qquad (705)$$

where: q = Flow Discharge Per Unit Width in cfs/ft

n = Manning's Roughness Coefficient

R = Hydraulic Radius in ft, Approximated by Y for Wide Channel

S_e = Energy Slope, in ft/ft

Substituting Manning's equation in **Equation 704** and solving for depth gives:

$$q_s = C_1 (q)^{C_2} \left[\frac{qn}{1.486 S_e^{1/2}}\right]^{3/5(C_2 - C_3)} \qquad (706)$$

Rearranging and solving for slope yields:

$$S_e = \left(\frac{C_1}{q_s}\right)^{\frac{10}{3(C_2 - C_3)}} (q)^{\frac{2(3C_2 + 2C_3)}{3(C_2 - C_3)}} \left(\frac{n}{1.486}\right)^2 \qquad (707)$$

For a known upstream supply, channel roughness and sediment transport relationship, the above equation reduces to a simple function of unit discharge. This equation can be used to estimate long-term degradation and aggradation.

The equilibrium slope for areas outside of Clark County is typically determined for the more frequent flood flows such as the minor storm event because this event generally dominates the long-term degradation or aggradation process. Then, in addition to the prediction of the long-term equilibrium channel slope, degradation or aggradation from a rare flood event such as the major storm event is also evaluated because the maximum short-term degradation or aggradation usually will occur during the rare storm. However, due to the minimal number of rainstorms which occur each year in the Clark County area, the maximum long term degradation or aggradation is expected to be mainly caused by only the major type storms. Similarly, the minor type storms are expected to only produce minor changes in degradation or aggradation in the Clark County area. Therefore, the major storm event (100-year) shall be used for analysis of the equilibrium slope and provide an unobstructed flow path for events greater than the design discharge.

704.2.1.3 - Anti-Dune Trough Depth

Anti-dunes are bed forms in the shape of dunes which move in an upstream rather than a downstream direction within the channel; hence the term “antidunes.” They form as trains of waves that build up from a plane bed and a plane water surface. Anti-dunes can form either during transitional flow, between sub-critical and super-critical flow, or during super-critical flow. The wave length is proportional to the velocity of flow. The corresponding surface waves, which are in phase with the anti-dunes, tend to break like surf when the waves reach a height approximately equal to 0.14 times the wave length. A relationship between average channel velocity, V, and anti-dune trough depth, Z_a can therefore be developed (SIMONS, LI & ASSOCIATES, 1982). This relationship is:

$$Z_a = \frac{1}{2}(0.14)\frac{2\pi V^2}{g} = 0.0137V^2 \qquad (708)$$

A restriction on the above equation is that the anti-dune trough depth can never exceed one-half the depth of flow. Therefore, if the computed depth of Z_a obtained by using the above equation exceeds one-half of the depth of flow, the anti-dune trough depth should then be taken as equal to one-half the depth of flow.

704.2.1.4 - Bend Scour

Bend scour normally occurs along the outside of bends, and is caused by spiral, transverse currents which form within the flow as the water moves around the bend. Presently, there is no single procedure which will consistently and accurately predict bend scour over a wide range of hydraulic conditions. However, the following relationship has been developed by Zeller for estimating bend scour in sand-bed channels based upon the assumption of the maintenance of constant stream power within the channel bend (SIMONS, LI & ASSOCIATES, 1989):

$$Z_{bs} = \frac{0.0685\ Y_{MAX} V^{0.8}}{Y_h^{0.4}\ S_e^{0.5}} \left[2.1\left[\frac{Sin^2(\alpha/2)}{Cos\alpha}\right]^{0.2} - 1\right] \qquad (709)$$

where: Z_{bs} = Bend-Scour Component of Total Scour Depth, in ft;

= 0, When r_c / T $ 10.0, or " # 17.8 degrees

= Computed Value, When $0.5 < r_c / T < 10.0$, or 17.8 < " < 60 degrees

= Computed Value, When r_c / T # 0.5, or " > 60º

V = Average Velocity of Flow Immediately Upstream of Bend, in fps

Y_{MAX} = Maximum Depth of Flow Immediately Upstream of Bend; in ft

Y_h = Hydraulic Depth of Flow Immediately Upstream of Bend, in ft (Hydraulic Depth = $\frac{\text{Flow Area}}{\text{Flow Top Width}}$)

S_e = Energy Slope Immediately Upstream of Bend (or Bed Slope for Uniform-Flow Conditions), in ft/ft; and

" = Angle Formed by the Projection of the Channel Centerline From the Point of Curvature to a Point Which Meets a Line Tangent to the Outer Bank of the Channel, in Degrees (see **Figure 703A**).

Mathematically, it can be shown that, for a simple circular curve, the following relationship exists between " and the ratio ofthe centerline radius of curvature, r_c, to channel top width, T:

$$\frac{r_c}{T} = \frac{Cos\,\alpha}{4Sin^2(\alpha/2)} \tag{710}$$

If the bend deviates significantly from a simple circular curve, the curve should be divided into a series of circular curves, and the bend scour computed for each segment should be based upon the angle, ", applicable to that segment.

The above two equations can be applied to obtain an approximation of the scour depth that can be expected in a bend during a specific water discharge. The impact that other simultaneously occurring phenomena such as sand waves, local scour, long-term degradation, etc., might have upon bend scour is not known for certain, given the present state of the art. Therefore, in order that the maximum scour in a bend not be underestimated, it is recommended that bend scour be considered as an independent channel adjustment that should be added to those adjustments computed for long-term degradation, contraction, and sand-wave troughs.

The longitudinal extent of the bend-scour component is as difficult to quantify as the vertical extent. Rozovskii developed an expression for predicting the distance from the end of a bend at which the secondary currents will have decayed to a negligible magnitude. This relationship, in a simplified form, can be expressed as:

$$X = \frac{0.6}{n} Y^{1.17} \qquad (711)$$

where: X = Distance From the End of Channel Curvature (Point of Tangency, PT) to the Downstream Point at Which Secondary Currents Have Dissipated in ft

n = Manning*s Roughness Coefficient

Y = Depth of Flow (to be Conservative, Use Maximum Depth of Flow, Exclusive of Scour, Within the Bend) in ft

The above equation should be used for determining the distance downstream of a curve that secondary currents will continue to be effective in producing bend scour. As a conservative estimate of the longitudinal extent of bend scour, both through and downstream of the curve, it would be advisable to consider bend scour as commencing at the upstream point of curvature, PC, and extending a distance, X, (computed with the above equation) beyond the downstream point of tangency, PT.

704.2.1.5 - Contraction Scour

Contraction scour occurs when the flow area is contracted by embankments, channelization, bridges and accumulation of debris. Scour at contractions occurs because the flow area becomes smaller than the normal stream and average velocity and bed shear stress increase. Hence, there is an increase in stream power at the contraction and more bed material is transported through the contracted section than is transported into the section. As bed level is lowered, banks erode, velocity decreases, shear stress decreases, and equilibrium is restored when the transport rate of sediment through the contracted section is equal to the incoming rate.

There are two forms of contraction scour that can occur depending on how much bed material is being transported upstream of the contraction reach. The two types of contraction scour are called live-bed contraction scour and clear-water contraction scour. Live-bed contraction scour occurs when bed material is already being transported into the contracted section from upstream of the approach section (before the contractionreach). Clear-water contraction scour occurs when the bed material sediment transport in the uncontracted approach section is negligible or less than the carrying capacity of the flow.

To determine if the flow upstream is transporting bed material (i.e., live-bed contraction scour), a critical velocity for beginning of motion, V_c, (for the D_{50} size of bed material) can be calculated and compared with the mean velocity, V, of the flow in the main channel or overbank area upstream of the contraction reach. If the critical velocity of the bed material is greater than the mean velocity at the approach section, $V_c > V$, then clear-water contraction scour is assumed. If the critical velocity of the bed material is less than the mean velocity at the approach section, $V_c < V$, then live-bed contraction scour is assumed. The following equation by LAURSEN (1963) can be used to calculate the critical velocity:

$$V_c = 10.95\, Y^{\frac{1}{6}}\, d_{50}^{\frac{1}{3}} \qquad (712)$$

where V_c = Critical Velocity Above Which Material of Size d_{50} and Smaller Will Be Transported, in fps

Y = Average Depth of Flow in the Main Channel or Overbank Area at the Approach Section, in ft

d_{50} = Bed Material Particle Size in a Mixture of Which 50 percent are Smaller, in ft

(a) Live-Bed Contraction Scour

The Hydraulic Engineering Circular No. 18 (HEC No. 18, FHWA, 1995) publication recommends using a modified version of LAURSEN*s (1960) live-bed scour equation:

$$Y_2 = Y_1 \left[\frac{Q_2}{Q_1}\right]^{\frac{6}{7}} \left[\frac{W_1}{W_2}\right]^{K_1} \qquad (713)$$

$$Z_{cs} = Y_2 - Y_o \qquad (714)$$

where: Z_{cs} = Average Depth of Contraction Scour, in ft

Y_2 = Average Depth After Scour in the Contracted Section, in ft

Y_1 = Average Depth in the Main Channel or Floodplain at the Approach Section, in ft

Y_o = Average Depth in the Main Channel or Floodplain at the Contracted Section Before Scour, in ft

Q_1 = Flow in the Main Channel or Floodplain at the Approach Section, Which is Transporting Sediment, in cfs

Q_2 = Flow in the main channel or floodplain at the contracted section, which is transporting sediment, in cfs, (Q_2 is greater than Q_1 approximately by the amount of flow blocked by the structure causing channel contraction).

W_1 = Top Width of the Active Flow Area at the Approach Section

W_2 = Top Width of the Active Flow Area at the Contracted Section

K_1 = Exponent for Mode of Bed Material Transport

(1) $V_*/T < 0.5$, $K_1 = 0.59$: Mostly Contact Bed Material Discharge

(2) $V_*/T = 0.50$ to 2.0, $K_1 = 0.64$: Some Suspended Bed Material Discharge

(3) $V_*/T > 2.0$, $K_1 = 0.59$: Mostly Contact Bed Material Discharge

V_* = $(gY_1S_e)^{1/2}$, Shear Velocity in the Main Channel or Floodplain at the Approach Section, in fps

T = Fall Velocity of Bed Material Based on d_{50}, in fps (See Stokes Equation, Page 73-77, Fluvial Processes in River Engineering, H. CHANG, 1998)

g = Acceleration of Gravity, in ft/sec^2

S_e = Slope of the Energy Grade Line at the Approach Section in ft/ft

(b) Clear-Water Contraction Scour

The recommended clear-water contraction scour equation by the HEC No. 18 publication is an equation based on research from LAURSEN (1963):

$$Y_2 = \left[\frac{Q_2^2}{C d_m^{2/3} W_2^2}\right]^{3/7} \tag{715}$$

$$Z_{cs} = Y_2 - Y_o \tag{716}$$

Where: d_m = Diameter of the Smallest Non-Transportable Particle in the Bed Material (1.25 d_{50}) in the Contracted Section, in ft

d_{50} = Median Diameter of the Bed Material, in ft

C = 120 for English Units (40 for Metric)

704.2.1.6 - Local Scour

Local scour occurs in the bed at embankments due to the actions of vortex (flow jets) induced by obstruction of the flow. The basic mechanism causing local scour are flow jets which result from the backup of water on the upstream edge of the embankment and piers and subsequent acceleration of this flow around the nose of the embankment. The action of the jet is to erode bed materials away from the base region. If the transport rate of sediment away from the local region is greater than the transport rate into the region, a scour hole develops. As the depth is increased, the strength of the flow jet and the sediment transportrate is reduced, equilibrium is re-established and scouring ceases.

The depth of scour varies with time because sediment transported into the scour hole from upstream varies depending upon the upstream sediment load. The mean scour depth between the oscillation of minimum and maximum scour depths is referred to as the equilibrium scour depth.

(a) Local Scour at Abutments

Local scour occurs at abutments when the abutment obstructs the flow. The obstruction of the flow forms a horizontal vortex, starting at the upstream end of the abutment and running along the toe of the abutment, and forms a vertical wake vortex at the downstream end of the abutment.

The HEC No. 18 report recommends two equations for the computation of live-bed abutment scour. When the wetted embankment length, L*, divided by the approach flow depth, Y_1, is greater than 25, the HEC No. 18 report suggests using the HIRE equation (RICHARDSON, 1990). When the wetted embankment length divided by the approach depth is less than or equal to 25, the HEC No. 18 report suggests using an equation by Froehlich (FROEHLICH, 1989).

The HIRE equation is based on field data of scour at the end of spurs in the Mississippi River (obtained by the USACE). The HIRE equation is:

$$Z_s = 4Y_1\left(\frac{K_1}{0.55}\right) K_2 \, Fr_1^{0.33} \tag{717}$$

where: Z_s = Scour Depth, in ft

Y_1 = Depth of Flow at the Toe of the Abutment on the Overbank or in the Main Channel, in ft, Taken at the Cross Section Just Upstream of the Abutment

K_1 = Correction Factor for Abutment Shape

= 1.00, for Vertical-Wall Abutment

= 0.82, for Vertical-Wall Abutment with Wing Walls

= 0.55, for Spill-Through Abutment

K_2 = Correction Factor for Angle of Attack (2) of Flow With Abutment, 2 = 90 degrees When Abutments are Perpendicular to the Flow, 2 < 90 degrees If Embankment Points Downstream, and 2 > 90 degrees If Embankment Points Upstream:

• $(2/90)^{0.4}$

Fr_1 = Froude Number Based on Velocity and Depth Adjacent and Just Upstream of the Abutment Toe

Froehlich analyzed 170 live-bed scour measurements in laboratory flumes by regression analysis to obtain the following equation:

$$Z_s = 2.27 K_1 K_2 (L')^{0.43} Y_a^{0.57} Fr^{0.61} + Y_a \tag{718}$$

where: Z_s = Scour depth, in ft

K_1 = Correction Factor for Abutment Shape:

= 1.1, for Square Nose

= 1.0, for Round Nose, Circular Cylinder and Group of Cylinders

= 0.9, for Sharp Nose (Triangular)

K_2 = Correction Factor for Angle of Attack (2) of Flow With Abutment, 2 = 90 degrees WhenAbutments are Perpendicular to the Flow, 2 < 90 degrees If Embankment Points Downstream, and 2 > 90 degrees If Embankment Points Upstream: K_2 • $(2/90)^{0.4}$

L^1 = Length of Abutment (Embankment) Projected Normal to Flow, in ft

Y_a = Average Depth of Flow on the Floodplain at the Approach Section, in ft

Fr = Froude Number of the Floodplain Flow at the Approach Section, $Fr = V_a /(gY_a)^{½}$

V_a = Average Velocity of the Approach Flow $V_a = Q_a/A_a$, in fps

Q_a = Flow Obstructed by the Abutment and Embankment at the Approach Section, in cfs

A_a = Flow Area of the Approach Section Obstructed by the Abutment and Embankment, in ft^2

(b) Local Scour at Pier

Pier scour occurs due to the acceleration of flow around the pier and the formation of flow vortices (known as the horseshoe vortex). The horseshoe vortex removes material from the base of the pier, creating a scour hole. As the depth of scour increases, the magnitude of the horseshoe vortex decreases, thereby reducing the rate at which material is removed from the scour hole. Eventually an equilibrium between bed material inflow and outflow is reached, and the scour hole ceases to grow.

The factors that affect the depth of local scour at a pier are: velocity of the flow just upstream of the pier, depth of flow, width of the pier, length of the pier if skewed to the flow, size and gradation of bed material, angle of attack of approach flow, shape of the pier; bed configuration, and the formation of ice jams and debris.

The HEC No. 18 report recommends the use of the Colorado State University (CSU) equation (RICHARDSON, 1990) for the computation of pier scour under both live-bed and clear-water conditions. In addition to the CSU equation, an equation developed by Dr. David Froehlich (1991) has also been added as an alternative pier scour equation. The Froehlich equation has been shown to compare well with observed data.

The CSU equation predicts maximum pier scour depths for both live-bed and clear-water pier scour. The equation is:

$$Z_s = 2.0\ K_1\ K_2\ K_3\ K_4\ a^{0.65}\ Y_1^{0.35}\ Fr_1^{0.43} \qquad (719)$$

where: Z_s = Depth of Scour, in ft

a = Pier width, in ft

K_1 = Correction Factor for Pier Nose Shape:

= 1.1, for Square Nose

= 1.0, for Round Nose, Circular Cylinder and Group of Cylinders

= 0.9, for Sharp Nose (Triangular)

K_2 = Correction Factor for Angle of Attack of Flow:

= $[(L/a)\ Sin\ Cos\ 2]^{0.65}$, where L = length of the pier along the flow line in ft, and 2 = angle of attack of the flow, with respect to the pier. If L/a is larger than 12, use L/a =12 as a maximum for estimating K_2. If the angle of attack is greater than 5 degrees, K_2 dominates and K_1 should be set to 1.0.

K_3 = Correction Factor for Bed Condition:

= 1.1, for Clear-Water Scour, Plane Bed, Anti-Dune Flow,and Small Dunes (10 > H (Dune Height)$ 2 ft)

= 1.1 to 1.2, Medium Dunes (30 > H$ 10ft)

= 1.3, Large Dunes (H $ 30)

K_4 = Correction Factor for Armoring of Bed Material:

The correction factor, K_4, decreases scour depths for armoring of the scour hole for bed materials that have a d_{50} equal to or larger than 0.20 feet. The correction factor results from recent research by A. Molinas at CSU which showed that when the velocity, V_1, is less than the critical velocity, V_{c90}, of the d_{90} size of the bed material, and there is a gradation in sizes in the bed material, the d_{90} will limit the scour depth. The equation developed by J. S. Jones from analysis of the data is:

$$K_4 = [1 - 0.89\,(1 - V_R)^2]^{0.5} \tag{720}$$

where:

$$V_R = [\frac{V_1 - V_i}{V_{c90} - V_i}] \tag{721}$$

$$V_i = 0.645\,[\frac{d_{50}}{a}]^{0.053}\,V_{c50} \tag{722}$$

V_R = Velocity Ratio

V_1 = Average Velocity in the Main Channel or Overbank Area at the Cross Section Just Upstream of the Bridge, in fps

V_i = Velocity When Particles at a Pier Begin to Move, in fps.

V_{c90} = Critical Velocity for d_{90} Bed Material Size, in fps:

$$V_{c90} = 10.95\, Y^{1/6}\, d_{90}^{1/3}$$

Vc_{50} = Critical Velocity for d_{50} Bed Material Size, in fps:

$$V_{c50} = 10.95\, Y^{1/6}\, d_{50}^{1/3}$$

a = Pier Width, in ft

Y = The Depth of Water Just Upstream of the Pier, in ft

Limiting K_4 values and bed material size are:

$$K_4 \geq 0.7;\ V_R \geq 1.0;\ d_{50} \geq 0.2 \text{ ft}$$

A local pier scour equation developed by Dr. David Froehlich (FROEHLICH, 1991) has been shown to compare well against observed data (FHWA, 1996). The equation is:

where:

$$Z_s = 0.32\ \phi\ (a')^{0.62}\ Y_1^{0.47}\ Fr_1^{0.22}\ d_{50}^{-0.09} + a \qquad (723)$$

ϕ = Correction Factor for Pier Nose Shape: ϕ = 1.3 for Square Nose Piers, ϕ = 1.0 for Rounded Nose Piers, and ϕ = 0.7 for Sharp Nose (Triangular) Piers

a' = Projected Pier Width With Respect to the Direction of the Flow, in ft

This form of Froehlich's equation is use to predict maximum pier scour for design purposes. The addition of one pier width, + a, is placed in the equation as a factor of safety. If the equation is to be used in an analysis mode (i.e., for predicting the scour of a particular event), Froehlich suggests dropping the addition of the pier width, + a, pier scour from this equation. The pier scour from this equation is limited to a maximum in the same manner as the CSU equation. Maximum scour $Z_s \leq$ 2.4 times the pier width for $Fr_1 \leq 0.8$, and $Z_s \leq$ 3.0 times the pier width for $Fr_1 > 0.8$.

704.2.1.7 - Total Scour

The total scour that can occur at a structure or pertinent location is equal to the combination of long-term bed elevation changes, one-half of anti-dune trough depth, bend scour, contraction scour and local scour:

$$Z_t = Z_{lt} + \frac{1}{2} Z_a + Z_{bs} + Z_{cs} + Z_s \tag{724}$$

Bank protection and protection at structures should extend to a depth below the channel bed equal to the total scour.

704.2.1.8 - Scour Below Channel Drops

Scour below channel drops, such as grade-control structures, is a special case of local scour. Where the drop consists of a free, unsubmerged overfall, the depth of scour below the drop (U. S. BUREAU OF RECLAMATION, 1977) shall be computed from:

$$Z_{fd} = 1.32\ q^{0.54}\ H_t^{0.255} - T_w \tag{725}$$

where: Z_{fd} = Depth of Local Scour Due to a Free-Overfall Drop, in ft, Measured Below the Streambed Surface Downstream of the Drop

q = Discharge Per Unit Width of the Channel Bottom, in cfs

H_t = Total Drop in Head, Measured From the Upstream Energy Grade Line to the Downstream Energy Grade Line, in ft

T_w = Tailwater Elevation or Depth (Downstream Water-Surface Elevation or Depth), in ft

Where the drop is submerged, as will be the case for most instances involving grade-control structures placed along watercourses for the depth scour below the drop, (SIMONS, LI & ASSOCIATES, 1986) can be computed from:

$$Z_{sd} = 0.581\ q^{0.667}\ (h/Y)^{0.411}\ [1 - (h/Y)]^{-0.118} \tag{726}$$

where:

Z_{sd} = Depth of Local Scour Due to a Submerged Drop, in ft, Measured Below the Streambed Surface Downstream of the Drop

q = Discharge Per Unit Width of the Channel Bottom, in cfs

h = Drop Height, in ft, Above the Immediate Downstream Bed, in ft

Y = Downstream Depth of Flow, in ft (Note: *h/Y* # 0.99)

If $h/Y > 0.85$, the predicted scour below a channel drop should be computed using both of the above equations. The smaller of the two values thus computed should then be used for design purposes.

The longitudinal extent of a scour hole created by either a free or submerged overfall is represented by the distance from the drop to the deepest scour depth, X_s, and the distance from the drop to the end of the scour hole, L_s. These dimensions are given by the equations:

$$X_s = 6.0\,Z_{fd}, \quad \text{or} \quad 6.0\,Z_{sd}$$

$$L_s = 12.0\,Z_{fd}, \text{ or } 12.0\,Z_{sd}$$

Bank protection toe-downs downstream of a grade-control structure shall extend to the computed depth of scour for a distance equal to X_s beyond the grade-control structure. They shall then taper back to the normal toe-down depth within a total distance downstream of the grade-control structure equal to L_s.

In the absence of bridge piers and/or abutments, the depth of scour below grade-control structures is not added to the other scour components. Rather, the depth of scour caused by the grade-control structure is compared to the depth of scour computed for the long-term degradation, and the larger of the two values is then used for toe-down design.

704.2.2 Design Considerations

704.2.2.1 Stable Channel

Stable channel cross sections formed in natural soils are usually wide and shallow because the fine particles cannot withstand high velocities, turbulence, and tractive forces. Stable channel designs using maximum permissible velocity or critical shear stress criteria are frequently utilized. These methods often result in large geometric sections. In many cases, the ROW required by a wide channel is impractical and uneconomical. The design of a stable channel in the sediment laden stream usually requires bank protection. It is possible to obtain a more practical section by using a properly designed lining. Channel linings commonly used are concrete, riprap, soil-cement, or gabions.

In natural channels, stabilization of channel banks is critical because the channel banks are subjected to forces that cause lateral shifting. Channel stability may be accomplished utilizing channel bank protection only. However, the equilibrium channel slope must also be evaluated and provided so that the natural channel facilities and channel bank lining can be properly designed.

704.2.2.2 Design Discharge

The design discharge for equilibrium channel slope determination is a discharge for which the sediment supply is to be determined. The design discharge should be a discharge which will determine the long-term response of the channel. For the Clark County area, the major storm events are expected to dominate the long-term channel response as well as the maximum degradation or aggradation of the channel. Therefore, the major storm event shall be used to determine the maximum long term channel degradation or aggradation.

704.2.2.3 Sediment Supply (Upstream Reach)

A major controlling factor when assessing channel response is the upstream sediment supply. Whether a channel degrades or aggrades strongly depends on the balance between the incoming sediment supply and a reach's transporting capacity. This is especially true for channels where armoring does not occur.

It is extremely important to understand that the supply system is being subjected to conditions that can drastically alter the sediment supply. The major alterations are the increasing urbanization of the area and the construction of debris basins and detention ponds.

In many regions, the urbanization process is viewed as increasing the sediment supply because of order-of-magnitude increases in erosion during construction functions. However, if the land has protective cover, added exposure to erosion is much less significant than in many other environments. The major effect in this case, is actually a reduction in sediment supply during urbanization. Supply is also reduced if land owners take measures to prevent erosion due to flows crossing their properties. Covering the soil with pavement, rock, houses, and vegetative landscaping such as lawns, reduces erosion. All these factors contribute to bring less sediment to the river system. Thus, as urbanization continues, there should be a corresponding decrease in sediment supply to a natural channel.

Incoming sediment supply is very difficult to estimate. A practical way to estimate the incoming sediment supply is to select a supply reach. The supply reach must be close to its equilibrium condition. Usually, the supply reach is selected from the following: (1) use the sediment transport capacity at an upstream reach as sediment supply using an appropriate maximum permissible velocity and estimated flow depth if upstream urbanization is expected; (2) use a natural channel reach, upstream of the design reach, which has not and will not be disturbed by man's activities; or (3) use an upstream channelized reach which has been in existence for many years and has not experienced a recent change in profile or cross section. The sediment transport capacity of the supply reach can be calculated and used as the incoming sediment supply.

704.2.2.4 ***Bed Form Roughness***

It is known that for flow in channels composed of alluvial soils, a strong physical inter-relationship exists between the friction factor, the sediment transport rate, and the geometric configuration assumed by the bed surface. The changes in bed form result from the interaction of the flow, fluid, and bed material. Thus, the resistance to flow and sediment transport are functions of the slope and depth of the stream, the viscosity of the fluid, and the size distribution of the bed material. The analysis of flow in natural channels is extremely complex. However, with an understanding of the different type of bed forms that occur, the resistance to flow and sediment transport associated with each bed form, and how the variables of depth, slope, viscosity, etcetera, affect, bed form, the engineer can analyze, anticipate, eliminate, or alleviate problems that occur when working with natural channels.

The bed configuration is the array of bed forms, or absence thereof, generated onthe bed of a natural channel by the flow. The bed configurations thatmay form in a natural channel are plane bed without sediment movement, ripples, dunes, plane bed with sediment movement, anti-dunes, and chutes and pools.

704.2.2.5 Erodable Sediment Size

The sediment transport equations are based on the assumption that all the sediment sizes present can be moved by the flow. If this is not true, armoring will take place. The equations are not applicable when armoring occurs. The bed shear stress is given by two equations that are closely related as follows:

$$\tau_o = \gamma RS \text{ or } \tau_o = (1/8)\, Df_o V^2 \qquad (727)$$

in which,

τ_o = Sheer Stress

γ = Specific Weight of Water

R = Hydraulic Radius

S = Energy Slope

D = Density of Water

f_o = Darcy-Weisbach Friction Factor

V = Mean Flow Velocity

The equation $\tau_o = \gamma RS$ is usually the most simple one to utilize.

The diameter of the largest particle moving is then,

$$D = \tau_o / (0.047\,(S_s - 1)\,\gamma) \qquad (728)$$

in which,

D = Diameter of the Sediment

S_s = Specific Gravity of Sediment

0.047 = Recommended Value of Shields' Parameter

(All units are in feet, pounds, and seconds.) If sediment of the computed size or larger is not present, then the sediment transport equations are applicable.

Equations 727 and 728 were developed for alluvial channels and do not apply to conditions when the bed material is cohesive. The equations would over predict transport rates in a cohesive soil channel.

704.2.3 Data Requirements

a. General

The primary purpose of this section is to identify data needs for the geomorphic and hydraulic analyses of natural channel systems. Although large volumes of data relative to the morphologic and hydraulic characteristics of rivers have been collected, much of this data is not readily obtainable or applicable to natural channels in the Clark County area. Consequently, the search for data, which is a necessary preliminary step to any natural channel system analysis, can consume a significant portion of the time and money allocated to a given study. With a view toward minimizing the investment in the data gathering effort, a checklist is provided to serve as both a guide for data gathering and as an outline of basic considerations for the analysis of the impact of historical and proposed development activities on the natural channel environment.

b. Checklist

The type of data needed for qualitative and quantitative alluvial analyses and the relative importance of each data type, are listed in **Table** 705. Data with a degree of importance, “primary”, are basic data required for any geomorphic, hydraulic, and environmental study of a natural channel. Whenever possible, these data should be directly collected from the field. Other data with a degree of importance, “secondary” are also very helpful in an analysis of a natural channel, but are considered a secondary requirement. It must be noted that certain categories of data, including hydrologic, hydraulic, channel geometry, and hydrographic, are extremely dynamic in nature and strongly a function of past and present conditions. Therefore, available data should be validated against present natural channel system conditions to determine their acceptability for the analysis.

704.2.4 Design Procedure

1. Determine major storm flow rate.

2. Select upstream supply reach and obtain the following pertinent information:

    a. Channel geometry

    b. Channel slope

    c. Sediment size distribution

    d. Channel resistance (n)

    A geotechnical analysis shall be conducted to determine the sediment size distribution (Item c above).

3. Obtain the same pertinent data as in Step 2 for the channelization under consideration.

4. Calculate the hydraulic conditions based on the major storm flows

5. After it has been shown that the sediment transport equation is applicable, the sediment supply from the upstream channel is computed using **Equation 704.** The calculated sediment supply is per unit width. The total sediment transport rate is obtained by multiplying the rate per unit width by the top width of the natural channel.

6. Determine the equilibrium slope for the downstream channel with the sediment supply rate determined in Step 5. This requires a trial and error procedure by which a given slope is chosen to compute the flow conditions and from the flow conditions, the sediment transport is calculated. When the computed rate is equal to the supply rate, the equilibrium slope has been found.

7. Based on the hydraulic conditions at equilibrium slope, estimate the largest particle size moving for armoring control check. Also,compare hydraulic parameters with the range of parameters for application of the equations.

8. Check whether the channel will be degraded or aggraded during the design storm event.

704.2.5 Floodplain Management of Natural Channels

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

1. The channel and overbank areas shall have adequate capacity for the major storm runoff.

2. Natural channel segments which have a calculated flow velocity greater than the allowable flow velocity determined herein shall be analyzed for erosion potential. Additional erosion protection may be required.

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

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

5. Roughness factors (n), which are representative of unmaintained conditions, shall be used for the analysis of water surface profiles.

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

7. A general plan and profile (i.e., HEC-2 and/or HEC-RAS output) of the floodplain shall be prepared which includes in the analysis appropriate allowances for known future bridges or culverts which will increase the water surface profile and cause the floodplain to be larger.

8. 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) then an analysis of the equilibrium slope and degradation or aggregation depths is required. It may also require bank protection to prevent channel migration outside of the floodplain.

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. Developments along the channel shall be elevated in accordance with the REGULATIONS for floodplain management purposes. There are significant advantages which occur if the designer incorporates into his planning the overtopping of the channel and localized flooding of adjacent areas which are laid out to remain undeveloped for the purpose of being inundated during the major runoff peak.

If a natural channel is to be maintained or encroached upon for a development, then the applicant shall meet with the local entity and the CCRFCD (if applicable) to discuss the concept and to obtain the requirements for planning and design analysis and documentation.

705 DESIGN SECTIONS AND STANDARDS FOR IMPROVED CHANNELS

705.1 Introduction

Presented in this section are the typical improved channel design sections which may be used in the Clark County area for open channel design. A graphical illustration of the typical design sections is presented in **Figure 704.** The selection of a channel section and lining is generally dependent on physicaland economic channel restrictions (i.e., value of developable land), the slope of the proposed channel alignment, the rate of flowto be conveyed by the channel, and the comparative costs of the lining materials. The many channel sections and linings discussed herein provide a wide range of options from which an applicable channel may be selected. For channel banks which are lined or have sideslopes on the order of 2H:1V, maintainable surfaces should be used. Specific hydraulic design standards which are applicable to all improved channels (i.e., transition, freeboard, etc.) are presented in Section 706.

The design standards presented in this section are the minimum standards by which channel design shall be completed within the CCRFCD. The channel designer is reminded that the ultimate responsibility for a safe 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 section. In addition, the local entity and/or the CCRFCD may require additional design analysis be performed to verify the suitability of the proposed design for the location under consideration.

705.2 Permanent Unlined Channels

Permanent unlined channels are improved channels which are constructed to the shape of vegetation lined channels but are not revegetated. The cost of construction of these channels is relatively low for areas with shallow 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.

The stability of the channel shall be analyzed as if the channel was a natural channel using the design standards in Section 704. In addition, the layout, alignment, and cross-section of the channel shall be designed as if the channel was to be revegetated using the design standards in Section 705.3.

The hydraulic analysis of the channel (i.e., Manning's "n" value) shall be based on the channel remaining unvegetated.

705.3 Non-Reinforced Vegetation Lined Channels

Vegetal lining is not allowed in publicly-maintained channels. Vegetation lined channels are defined as channels in which a vegetation lining is maintained by a permanent sprinkler irrigation system.

Vegetation lined channels may be considered to be the most desirable artificial channels from an esthetics viewpoint. The channel storage, lower velocities, and the sociological benefits obtainable create significant advantages over other types of channels. However, a permanent irrigation system must be used to maintain the vegetation and the vegetation must be a grass species. Also, the designer must give full consideration to potential sediment deposition and scour, as well as flow hydraulics for which calculations shall be submitted for review to the local entity and/or CCRFCD.

The satisfactory performance of a vegetated channel depends on it having the proper shape, as well as the preparation of the area in a manner to provide conditions favorable to vegetative growth. Between the time of seeding the cover and the actual establishment, 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.

The entity and/or the CCRFCD may require a maintenance agreement and/or bond to cover maintenance of vegetative lined channels.

705.3.1 Design Parameters

705.3.1.1 <u>Longitudinal Channel Slopes</u>

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

705.3.1.2 <u>Roughness Coefficient</u>

The variation of Manning's "n" with the product of mean velocity and hydraulic radius is presented in **Figure 705**.

Manning's "n" from this figure shall be used to determine the channel capacity based on a mature channel (i.e., substantial vegetation with minimal maintenance).

705.3.1.3 <u>Cross Sections</u>

All vegetation lined channels shall be constructed with a trapezoidal shape.

705.3.1.4 <u>Low Flow Channel or Underground Low Flow Storm Drain</u>

All grass lined channels shall be constructed with a low flow channel or an underground low flow storm drain designed in accordance with Section 707.1.

705.3.1.5 <u>Bottom Width</u>

The minimum bottom width shall be consistent with the maximum depth and velocity criteria. The minimum bottom width shall be 5 feet or the low flow channel width, whichever is greater.

705.3.1.6 *Flow Depth*

Typically, the maximum design depth of flow (outside the lowflow channel area) for the major storm flood peak should not exceed 5 feet.

705.3.1.7 *Side Slopes*

Side slopes shall not be designed steeper than 3H:1V.

705.3.1.8 *Vegetation Lining*

The vegetation lining for channels shall be seeded or sodded with a grass species which is adapted to the hot and dry Clark County climate and will flourish under regular irrigation. Species such as Bermuda grass or similar grasses are recommended. Flowering plants, (i.e., Honeysuckle) and weeds shall not be used for vegetative lined channels.

705.3.1.9 *Establishing Vegetation*

Channel vegetation is established usually by seeding or by planting the rootsor other vegetative parts of the selected plants. In the more critical sections of some channels it may be desirable to provide immediate protection by transplanting a complete sod cover.

Jute, plastic, or paper mesh and straw or 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.

705.4 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 and limitations to minimize riprap movement in a fully lined channel as well as to minimize erosion of the channel section for channels with only bank lining.

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 frequent inspection and maintenance.

705.4.1 Types of Riprap

705.4.1.1 <u>Loose Riprap</u>

Loose riprap, or simply riprap, refers to a protective blanket of large loose stones, which 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 hydraulic 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 the slope of the riprap layer. Hydraulic factors affecting riprap include the velocity, current direction, eddy action and waves.

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.

705.4.1.2 <u>Grouted Riprap</u>

Grouted riprap is used where stone of suitable size forother types of riprap are not available. 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 the projecting rock with a wet broom. As with loose riprap, grouted riprap should be placed on an adequate bedding. The grout material shall be inaccordance with Section 610.02.01 of the STANDARD SPECIFICATIONS. Grouted riprap shall be constructed in accordance with section 610.03.04 of the STANDARD SPECIFICATIONS and shall penetrate either the full depth of the riprap layer or at least 2 feet where the riprap layer is thicker than 2 feet. Grout penetration may be accomplished by rodding, vibrating, or pumping of the grout into the riprap voids. Weep holes should be provided in the blanket to provide rapid relief of any hydrostatic pressure behind the blanket. A typical grouted riprap section is depicted in **Figure 706**.

705.4.2 Riprap Material

Riprap is classified in the Clark County area as "Riprap" and "Heavy Riprap". “Riprap" and "Heavy Riprap" shall meet all requirements and be in accordance with Section 610 of the STANDARD SPECIFICATIONS. For drainage purposes, it is desirable to have a range of sizes intermixed together to provide an even and interlocking protective layer. Therefore, the riprap gradation should be evenly spread over the range of sizes allowed in the STANDARD SPECIFICATION.

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 its length and rounded stone should be avoided. Rock having a minimum specific gravity of 2.65 is preferred.

705.4.3 Bedding Requirements

Long term stability of riprap erosion protection is strongly influenced by proper bedding conditions. A large percentage of all riprap failures are directly attributable to bedding failures.

A properly designed bedding provides a buffer of intermediate sized material between the channel bed and the riprap to prevent leaching of channel particles through the voids in the riprap. Two types of bedding are in common use, a granular bedding and filter fabric.

705.4.3.1 Granular Bedding

Two methods for establishing gradation requirements for granular bedding are described in this section. The first, a single layer of granular bedding, is adequate for most ordinary riprap, grouted riprap or wire encased riprap applications. The second is a detailed design procedure developed by Terzaghi, which is referred to as the T-V (Terzaghi-Vicksburg) design (MURPHY, 1971). The T-V filter criteria established an optimum bedding gradation for a specific channel soil. The latter requires channel soil information, including a gradation curve, while the single layer bedding may be used whether or not soil information is available.

The gradation of a single layer bedding specification was based on the T-V filter criteria and the assumption that a bedding which will protect an underlying non-cohesive soil with a mean grain size of 0.045 mm will protect anything finer. Since the T-V filter criteria provides some latitude in establishing bedding gradation, it was possible to make the single layer bedding specification generally conform with the STANDARD SPECIFICATIONS Type II Aggregate Base (704.03.04) or a 2-inch size Type I Aggregate Base (704.03.02).

A single 12-inch layer of Type II or Type I (2-inch size) bedding can be used except at drop structures. At drop structures, filter fabric must be added below the 12-inch layer of granular bedding.

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:

$$D_{15\ (filter)} \leq 5d_{85\ (base)} \tag{729}$$

$$4d_{15\ (base)} \leq D_{15\ (filter)} \leq 20d_{15\ (base)} \tag{730}$$

$$D_{50\ (filter)} < 25d_{50\ (base)} \tag{731}$$

Where the uppercase "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 6 inches. However, if a gradation analysis of the existing soils shows that more than 50 percent of the soil is smaller than the No. 40 sieve size (> 50 percent passing No. 40 sieve by weight), then a two layer 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 layers shall be 4 inches.

705.4.3.2 ***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 2.5H:1V, and that a 6-inch layer of fine aggregate (STANDARD SPECIFICATIONS 706.03.03) 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 an 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 have 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.

705.4.4 Channel Linings

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 1/4 mile). Situations for which riprap linings 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 (typically 5 feet per second); 2) where channel side slopes must be steeper than 3H:1V; 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.

705.4.5 Roughness Coefficients

The Manning's roughness coefficient "n" for hydraulic computations may be estimated for loose riprap using:

$$n = 0.0395\ (d_{50})^{1/6} \tag{732}$$

where: d_{50} = Mean Stone Size, in ft

This equation (ANDERSON, 1968) does not apply to grouted riprap (n = 0.023 to 0.030), or to very shallow flow (hydraulic radius is less than or equal to two times the maximum rock size) where the roughness coefficient will be greater than indicated by the formula.

Equation 732 was first proposed by STRICKLER (1923) for estimating Manning's "n" for streambeds. The relationship has been utilized in studies of roughness coefficients and channel design by a number of other investigations including NORMANN (1975) and MAYNORD (1978). This equation was developed using laboratory flume study data. By using data from FHWA study, BLODGETT (1986a) presented a roughness equation for a gravel bed channel:

$$n = \frac{0.0926\ Y^{0.167}}{0.794 + 1.85 \log Y/d_{50}} \tag{733}$$

for $1.5 < Y/d_{50} < 185$, where Y is the average flow depth. A comparison of **Equations 732** and **733** indicates a more hydraulically efficient channel than actually occurs. That is, for a given discharge, the water-surface elevation is lower, the cross section smaller, and the depth of flow less than actually occurs. Therefore, the above equation should be used for determining Manning's "n" for gravel bed channels or riprap protected channel, while **Equation 732** may be used for sizing riprap when velocity is used as a design parameter.

The riprap material may not cover the entire channel perimeter. Depending on the size of riprap and bed material, estimates of roughness for the entire channel that are larger than actual may result from using **Equation 733**, which gives greater depths than required for the design discharge. A method of estimating Manning's "n" for composite channels (e.g., CHOW, 1959) should be used.

705.4.6 Rock Sizing and Lining Dimensions

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) and application to design criteria (STEVENS, 1981):

$$V = 3\ (d_{50})^{0.5}\ (S_s - 1)/S^{0.17} \tag{734}$$

where: V = Mean Channel Velocity, in fps
(10 fps Maximum for Riprap Lined Channel)

S = Longitudinal Channel Slope, in ft/ft

S_s = Specific Gravity of Rock (Minimum $S_s = 2.50$)

d_{50} = Rock Size, in ft, for Which 50 percent of the Riprap by Weight is Smaller

Equation 734 was developed using laboratory data. Other procedures for design of riprap have been prepared by a number of agencies, such as Federal Highway Administration (SEARCY, 1967; NORMANN, 1975), USACE (1970), USBR (PETERKA, 1958), California Department of Transportation (1970), American Society of Civil Engineers (VANONI, 1975), (SIMONS and SENTURK, 1992). BLODGETT (1986) evaluated these procedures and presented a tentative design relationship based on field data:

$$d_{50} = 0.010\ V^{2.44} \tag{735}$$

where: V = Mean Channel Velocity, in fps

d_{50} = Rock Size, in ft, for Which 50 percent of the Riprap by Weight is Smaller

This equation is helpful for estimating the size of riprap needed and generally estimates sizes larger than those determined by using **Equation 734**. However, use of a design method based on tractive stress considering bank slope is preferred for final design.

The hydrodynamic force of water flowing in a channel is known as the tractive force. The basic premise underlying riprap design based on tractive force theory is that the flow-induced tractive force should not exceed the permissible or critical shear stress of the riprap. Assuming a specific gravity of 2.50, the following equation can be used to determine d_{50} of the riprap by the tractive stress method:

$$d_{50} = 14.2\, F_s Y_{max} \frac{S_e}{K_1} \tag{736}$$

where: F_s = Stability Factor:

= 1.0 - 1.2, For Straight or Mildly Curving Reach

= 1.2 - 1.4, For Moderate Bend Curvature With Minor Impact From Floating Debris

= 1.4 - 1.6, For Sharp Bend With Significant Impact From Floating Debris and Waves

= 1.6 - 2.0, For Rapidly Varying Flow With Significant Uncertainty in Design

Y_{max} = Maximum Channel Depth, in ft

S_e = Average Energy Slope, in ft/ft

K_1 = Bank Angle Modification Factor

= $[\,1 - (\text{Sin}^2\, M / \text{Sin}^2\, 2)\,]^{0.5}$

M = Bank Angle With Horizontal

2 = Riprap Material Angle of Repose (see **Figure 705A**)

Rock lined side slopes steeper than 2H:1V are considered unacceptable because of stability, safety, and maintenance considerations. Proper bedding is required both along the side slopes and the channel bottom for a stable lining. The riprap blanket thickness should follow the rules:

1. The thickness should be at least two times d_{50}.
2. The thickness should not be less than the diameter of the upper limit d_{100} stone.
3. The maximum allowable d_{50} shall be 24 inches.

4. The thickness determined by either (1) or (2) above should be increased by 50 percent in all sections when the riprap is placed under water in water deeper than 3 feet to provide for uncertainties associated with this type of placement.

5. An increase in thickness of 6 to 12 inches, accompanied by an appropriate increase in stone sizes, should be provided where riprap revetment will be subject to attack by floating debris or by waves from boat wakes or wind.

The blanket should extend up the side slopes to the freeboard requirements in this MANUAL. 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.

The USACE (1983) specify a riprap gradation band rather than a single gradation to consider the practical problems of quarry production. Any stone gradation within this band is acceptable. The USACE's criteria for establishing gradation limits for riprap are as follows:

- The lower limit of d_{50} stone should not be less than the size of stone required to withstand the design shear forces.

- The upper limit of d_{50} stone should not exceed five times the lower limit of d_{50} stone, the size which can be obtained economically from the quarry, or the size that satisfies layer thickness requirements.

- The lower limit of d_{100} stone should not be less than two times the lower limit of d_{50} stone.

- The upper limit of d_{100} stone should not exceed five times the lower limit of d_{50} stone, the size which can be obtained economically from the quarry, or the size that satisfies layer thickness requirements.

- The lower limit of d_{15} stone should not be less than 1/16 the upper limit of d_{100} stone.

- The upper limit of d_{15} stone should be less than the upper limit of the filter material.

- The bulk volume of stone lighter than the d_{15} stone should not exceed the volume of voids in the structure without this lighter stone.

705.4.7 Edge Protection

The edges of riprap revetments are subject to additional hydraulic forces by being adjacent to other materials. The top, toe, and flanks require special treatment to prevent undermining.

The flanks of the revetment should be designed as illustrated in **Figure 705B**. If the riprap ends at a bridge abutment or other secure point, special bank protection at the riprap perimeter is not needed. If the riprap does not terminate at a stable point, the cross-section shown as Method B in **Figure 705B** should be considered for the downstream edge as well.

Undermining of the revetment toe is one of the primary mechanisms of riprap failure. **Figure 705C** shows toe protection alternatives. It is preferable to design the toe as illustrated in **Figure 705D** (Method B from **Figure 705C**). The toe material is placed in a toe trench along the entire length of the riprap blanket. See the alternate design in **Figure 705D**. Care must be taken during the placement of the stone to ensure that the toe material does not mound and form a low dike; a low dike along the toe could result in flow concentration along the revetment face which could stress the revetment to failure. In addition, care must be exercised to ensure that the channel*s design capacity is not impaired by placement of too much riprap in a toe mound.

The size of the toe trench or alternate stone toe is controlled by the anticipated scour depth along the revetment. The depth of scour can be estimated from the scour analysis presented in Section 704.2. As scour occurs, the stone in the toe will launch into the eroded area as illustrated in **Figure 705E**. Observation of rock toe performance indicates that the riprap will launch to a final slope of approximately 2H:1V. The volume of rock required for the toe must be equal to or exceed one and one-half times the volume of rock required to extend the riprap blanket (at its design thickness and on a slope of 2H:1V) to the anticipated depth of scour.

705.4.8 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 at this location in natural or vegetation lined channels which otherwise would not need protection. Also, additional riprap thickness and sometimes larger sizes are needed at bends in riprap lined channels.

The riprap protection should be placed along the outside of the bank, and should extend from the entrance of the bend to a point downstream from the bend exit, a distance equal to the length of the bend measured at the channel centerline. Additionally, the riprap blanket should extend up the side slope at least 2 feet above the design water surface, or per the freeboard requirements in this section.

For bends in natural or vegetation lined channels, the standard straight channel riprap lining criteria (Section 705.4.6) shall be used. For bends in riprap lined channels where the bend radius is less than two times the water surface top width for major storm flows, increase the riprap thickness by 50 percent from the designed riprap thickness.

705.4.9 Transitions Protection

Scour potential is amplified by turbulent eddies in the vicinity of rapid changes in channel geometry such as transitions and bridges. For these locations, the riprap lining thickness shall be increased by 50 percent from the designed riprap thickness.

Protection should extend upstream from the transition entrance at least 5 feet and extend downstream from the transition exit at least 10 feet.

705.4.10 Concrete Cutoff Walls

Transverse concrete cutoff walls may be required by the local entity and/or the CCRFCD 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 entity and/or the CCRFCD prior to design of riprap lined channels to determine if concrete cutoff walls are required as well as their sizing and spacing, if required.

705.5 Gabion Lined Channels

Gabion lined channels are defined as channels in which gabion baskets and/or mattresses are used for lining of the channel banks and channel bottom. Gabions used for erosion protection at transitions and bends are also considered and shall be designed as a gabion lined channel.

The design parameters and standards for gabion lined channels are related to the specific product of the different gabion manufacturers. Therefore, specific design guidelines and standards are not provided herein for gabion linings. The lined channel section must, however, be designed in accordance with the hydraulic design standards for improved channels presented in Section 706. The designer must consult with the manufacturer of the gabions and prepare a design that is within the design guidelines and standards of the manufacturer. Both the manufacturer's design guidelines and standards and the designer's calculations and design shall be submitted to the local entity and/or the CCRFCD for review of the proposed design. The local entity and the CCRFCD reserve the right to reject the proposed lining system in the interests of operation, maintenance, and protecting the public safety.

705.6 Soil-Cement

705.6.1 General Parameters

Recommended design procedures and construction practices of single layer linings and plating of slopes with soil-cement are presented in this section. These design standards are found to work in similar conditions and are suggested for use in the CCRFCD area. Overland flows which can undermine or damage the channel banks shall be accounted for in the design of the channel side walls and bottom. Alternative design parameters will be considered on a case by case basis. These design parameters presented do not relieve the designer of performing the appropriate engineering analysis.

705.6.1.1 Design Parameters

The horizontal width of the section may be determined by the required construction applications and equipment. The alignment and the channel width should be determined by accurate survey and the elevation of the top of the bank protection and the toe elevation should be computed from hydraulic and geomorphic analysis. The elevation of the top of the bank should include an assessment to ensure adequate freeboard to account for the influence of vegetative debris, water and sand wave movement, and at bends, super-elevation. The toe elevation for the soil-cement bank should be based on the scour components as computed for the major storm event. An analysis of the soil-cement bank to resist sliding and overturning should be made. The soil bearing capacity and piping potential need to be evaluated based on the geotechnical recommendations or investigations of the site. The design of the soil-cement bank, therefore, should include establishing sectional dimensions, incorporating edge treatment, cross drainage details, and providing for access ramps (if required).

705.6.1.2 Code Requirements

The soil-cement lined channel sections shall be designed in accordance with *Suggested Specifications for Soil-Cement Base Course* and *Soil-Cement for Water Control: Laboratory Tests* and *Soil-Cement for Facing Slopes, and Lining Reservoirs, Channels, and Lagoons* and *Controlling Floods in the Desert with Soil-Cement* and *Soil-Cement Slope Protection for Embankments: Planning and Design* as published by *The Portland Cement Association, PCA* latest editions and other governing codes.

705.6.2 Mix Design

705.6.2.1 Proportioning

Soil may consist of any combination of gravel, stone, sand, silt, clay, caliche, scoria, slag, ash, and other materials which, as processed for construction, shall not contain material retained in a 2-inch sieve nor any material which is deleterious in its reaction with cement. The most practical and economical soil for use in producing soil-cement are soils which are easily pulverized soils containing at least 4 percent, but no more than 35 percent fines or materials passing a No. 200 sieve. For applications exposed to high velocities, superior performance with soil-cement has been shown where the soil contained at least 20 percent gravel or material retained on a No. 4 sieve. Cement contents for typical soil-cement mixtures are generally in the range of 7 to 12 percent by dry weight of soil. Fine textured soils, such as clays which are usually difficult to pulverize, or poorly graded granular soils which have no material passing the No. 200 sieve, will require more cement for satisfactory performance. In a reservoir situation where permeability may be of concern, permeabilites less than 1 foot per year can generally be achieved with the proper cement content and addition of 2 percent lime or fly ash. Cement shall be Type V.

705.6.2.2 Testing

Established test methods from ASTM and PCA should be used to determine the required amounts of cement, the optimum moisture content, and the density to which the mixture should be compacted. The optimum moisture content and maximum density should be determined by ASTM D 558. Portland cement should comply with the latest specifications for Portland cement ASTM C 150 or blended hydraulic cements, ASTM C 595. The required cement content for durability should then be established by wet-dry and freeze-thaw tests, ASTM D

559 and D 560, on representative specimens molded at optimum moisture and compaction to maximum density. Water should be free from substances deleterious to the hardening of the soil-cement. Additional details on laboratory test procedures can be found in PCA publication *Soil-Cement for Water Control. Laboratory Tests*. Data on permeability of cement-treated soils can be found in *Soil-Cement Slope Protection for Embankments: Planning and Design* as published by PCA.

705.6.3 Lining Section

705.6.3.1 General

A reliable and accurate method for controlling layer thickness to ensure uniformity should be adopted. For best results, the soil-cement should be plant mixed rather than mixed in place. In areas of known high groundwater where the liner is used for structures other than reservoirs, weep holes should be placed 15 feet on center with a collection system behind the bank approximately 1 foot above the base of the channel section.

705.6.3.2 Grade Control Structures

If analysis of the system indicates long-term channel bed degradation, grade control structures or drop structures would be required to protect the bank against the effects of undermining. The location and spacing of grade control structures is based on the analysis of the system's degradation/aggradation process as well as local scour calculations for the area directly downstream of the drop structure. Toe depths of the soil-cement bank protection should be deepened to account for increased scour directly downstream of the grade control structure.

705.6.3.3 Cross Drainage

Where constant flows are anticipated over the face of the soil-cement bank, consideration should be given to the face of the soil-cement bank to reduce the potential for abrasion/erosion of the surface. Where a tributary channel's invert elevation is comparable to that of the main channel, the soil-cement bank protection should be wrapped into the side channel and transitioned into the side channel's bank stabilization. For these instances, the portion of the main channel's soil-cement bank protection below the elevation of the invert, should be carried across the opening provided to the side drainage. A paved invert and cut-off wall of soil-cement or concrete may be required upstream of the side channel's confluence with the main channel.

705.6.3.4 Stairstep Method

For reservoirs, lagoons, channels and ditches where moderate to severe wave action is anticipated, or high flows, or for velocities greater than 8 feet per second, or where the slope will be exposed to debris carried by rapid flowing water, the stairstep method of placing slope protection where horizontal layers of soil-cement 7 to 9 feet wide and 6 to 9 inches thick adjacent to the slope.

705.6.3.5 Plating Method

For less severe applications such as small reservoirs with little wave action, ditches, or lagoons, (slope protection may consist of one or more,) the plating method of placing slope protection should be utilized by placing 6-inch to 9-inch layers of soil-cement placed parallel to the slope face. The slope should be 3H:1V, or flatter, in order to properly spread and compact the soil-cement.

705.6.4 Placement

705.6.4.1 Subgrade Preparation

Slopes to receive soil-cement should be firm and moist and shaped to the required lines and grades prior to soil-cement placement. Proper preparation of the subgrade will ensure that the soil-cement is uniform with specified thickness and adequate density being achieved with minimal compactive effort.

Proper compaction is one of the fundamental requirements of soil-cement construction. If the subgrade is soft and cannot properly support the compaction equipment, adequate density will not be obtained. Soft areas should be located and corrected before processing begins.

705.6.4.2 Mixing

For large projects, mixing of the soil, cement, and water shall be accomplished in a stationary pugmill type mixing plant equipped with a surge hopper. The plant may be either a batch type or a continuous-mixing type designed for either weight or volume proportioning. The plant shall have a rated capacity of at least 100 cubic yards per hour and shall be designed, coordinated, and operated soas to produce a uniform mixture. Facilities for efficiently storing, handling, and proportioning unmixed materials shall be provided at the plant. The plant shall be equipped with meters to obtain the proper amount of fly ash, cement, soil, and water. All measuring devices shall be sensitive to a 1 percent variation above or below the actual weight in pounds of cement required for each batch.

Proportioning may be on a volume basis, provided that the sensitivity specified for the weight basis is maintained.

Plants designed for continuous mixing shall include a means for accurately proportioning soil, fly ash, and cement and shall be equipped to ensure positive interlocking control of the flow of soil, fly ash, and cement from bins.

705.6.4.3 ***Compaction***

Soil-cement should be uniformly compacted to a minimum densityof 96 percent of maximum density as determined by field density tests. Optimum moisture and maximum density should be determined in the field during construction by moisture-density test ASTM D 558. The mixture should be in a loose uniform condition throughout its full depth at the start of compaction. Its moisture content should be within 2 percent ofthe specified optimum moisture content. Sections should not be left undisturbed for periods longer than 30 minutes during compaction operations. Compaction should be continued until uniform and adequate density is obtained to produce a dense surface free of compaction planes, cracks, ridges, or loose material.

705.6.4.4 ***Construction Joints***

When soil-cement operations are interrupted, such as at the end of each days run, a full depth vertical construction joint should be produced by cutting back into the compacted soil-cement with hand tools to produce a verticalface transverse to the direction of layer placement. A bonding agent may be placed just prior to placing the next soil-cement layer and not allowed to dry out. Dry cement may be used for bonding at the rate of 1.0 lb/10 ft^2 and sprinkled with water prior to placing the next layer or, (mixture in some cases) activated by water in the soil-cement.

705.6.4.5 ***Finishing***

The surface of the soil-cement should be shaped to the required lines, grades and cross-sections. Surfaces subjected to foot traffic should receive a light broom finish and imprints left by equipment should be removed to compaction planes. The surface should be kept moist preferably by fog-type sprays during the finishing process.

705.6.5 Curing and Cracking

705.6.5.1 Curing

Finished portions should be protected to prevent equipment from marring or otherwise damaging the completed work. The completed soil-cement must be prevented from drying out and freezing for a 7-day hydration period. Moist earth, plastic sheeting, application of water by fog spray, or bituminous or other sealing material may be used for curing. Because soil-cement has a low initial moisture content, water curing is preferred over other types of curing methods. No bituminous or other membrane curing material should be applied to surfaces that will be in contact with succeeding layers of soil-cement or potable water.

705.6.5.2 Shrinkage Cracking

Hairline shrinkage cracks often develop in soil-cement liners; however, such cracks can be considered self-healing if located in a place where sediment will accumulate and plug the crack. For applications where shrinkage cracking is a concern, keeping the soil-cement surface moist, filling a soil-cement lined reservoir or lagoon with water after completion, applying a bituminous or other type of sealing material over the soil-cement approximately one month after completion, and minimizing the initial moisture content of the soil-cement (moisture content at or slightly below optimum moisture content during compaction) all tend to minimize shrinkage in the soil-cement liner.

705.7 Concrete Lined Channels

Concrete lined channels are defined as rectangular or trapezoidal channels in which reinforced concrete is used to line the channel banks and bottom.

Concrete lined channels are the most prevalent type of improved channel in the CCRFCD. This is mainly due to their ability to accommodate super-critical flow conditions and thus can be constructed to almost any natural occurring slope. In addition, the cost of concrete channels is generally more economical than other lining types due to their greater flow carrying capacity resulting in less land area requirements.

705.7.1 Design Parameters

The following sections present the recommended design parameters for concrete lined channels. These design standards are found to work in similar conditions and are suggested for use in the CCRFCD area. Alternative design parameters will be considered on a case by case basis. The design parameters presented do not relieve the designer of performing the appropriate engineering analysis.

705.7.1.1 Code Requirements

The concrete channel sections shall be designed in accordance with *ACI Standard, Building Code Requirements for Reinforced Concrete Structures* ACI 318 and other governing codes.

705.7.1.2 Concrete Lining Section

a. All concrete linings shall have a minimum thickness of 6 inches for flow velocities less than 30 feet per second and a minimum thickness of 7 inches for flow velocities of 30 feet per second and greater.

b. In areas subjected to overland flows, longitudinal concrete cutoff walls shall be located at the top of the concrete channel lining to prevent undermining and surcharging of the channel side slopes.

c. A concrete cutoff wall shall be provided at both the upstream and downstream terminus of the channel lining.

d. The side slopes shall be a maximum of 2H:1V, or a structurally reinforced wall if steeper. All pipe or other openings at channel sides and bottom shall have additional reinforcement placed around the opening.

e. Minimum required concrete compressive strength at 28 days shall be 4,000 pounds per square inch and maximum water to cement ratio, by weight shall be 0.50.

f. The invert slab should be cross-sloped to concentrate nuisance flows and protect the longitudinal joints between the invert slab and side slopes.

705.7.1.3 Concrete Joints

Concrete linings may be designed using two approaches in the CCRFCD area. The two accepted methods are: panel type lining and continuously reinforced lining.

Panel type lining is designed with regularly spaced expansion joints and more frequently spaced contraction joints and with a relatively small amount of reinforcing steel. Regularly spaced joints accommodate movement due to material properties such as shrinkage, application of hydraulic and soil loads, and environmental conditions such as temperature changes. Panel type construction in the CCRFCD area requires a maximum expansion joint spacing with cut-off walls of 90 feet on center, with cut-off walls located at all expansion joints, with contraction joints from 15 to 30 feet on center spaced between the expansion joints.

Continuously reinforced lining is designed with few transverse joints and a relatively large amount of longitudinal reinforcing to distribute random cracking. Continuously reinforced lining incorporates expansion joint spacing of 100 to 500 feet, with cut-off walls located at all expansion joints. Transverse joint layout consists of expansion joints used sparingly and located during design at points of fixity and changes in channel cross section or alignment, and construction joints located during construction within a specified range of spacings at the end of a day's concrete placement. Expansion joints must be carefully detailed with joint filler and sealant materials specified appropriately, constructed strictly in accordance with contract documents, and inspected and maintained regularly.

a. Longitudinal joints, where required, shall be constructed on the sidewalls at least 1 foot vertically above the channel invert. They shall be protected from continuous contact with nuisance flows, raising them above the flow line for flat-bottomed channels.

b. All joints shall be designed to prevent differential movement.

c. 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 as necessary.

d. Contraction joints shall be tooled and sealed (saw cutting of contraction joints shall not be permitted).

705.7.1.4 Concrete Finish

The surface of the concrete lining shall be provided with a wood float finish, or light broom finish, unless the design requires additional finishing treatment. Excessive working or wetting of the finish shall be avoided if additional finishing is required.

705.7.1.5 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 Section 501.03.09 of the STANDARD SPECIFICATIONS.

705.7.1.6 Reinforcement Steel

a. Steel reinforcement shall be a minimum grade 60 deformed bars. Wire mesh shall not be used.

b. For continuously reinforced concrete channels, the ratio of longitudinal steel area to the concrete cross-sectional area shall be greater than 0.004 but not less than a No. 4 bar at 8-inch spacing for channels with expansion joints at 100 feet on center. For channels with expansion joints greater than 100 feet on center, the ratio of longitudinal steel area to the concrete cross-sectional area shall be greater than 0.005 but not less than a No. 5 bar at 10-inch spacing. The longitudinal steel shall be placed on top of the transverse steel.

   The ratio of transverse steel area to the concrete cross-sectional area shall be greater than 0.0020 for channel bottom widths less than 25 feet and 0.0025, but not less than a No. 4 bar at 12-inch spacing for wider channels. The maximum ratio of the transverse steel area to the concrete cross-sectional area shall be less than 50 percent of D_b.

c. For panel type construction, reinforcing ratios can be reduced if the design engineer details the contraction joints with discontinuous longitudinal reinforcing. The ratio of longitudinal steel area to the concrete cross-sectional area shall be greater than 0.0025, with contraction joints located on 30-ft centers and the longitudinal reinforcing discontinuous at the joint. If the steel at the contraction joint is not discontinuous, the reinforcing ratio shall be increased to 0.0040.

Transverse reinforcing ratio should be identical to the value selected for continuously reinforced channel linings.

d. Reinforcing steel shall be placed not farther apart than two times the slab thickness, nor 12 inches. Reinforcing steel shall be placed near the center of the section and shall have a minimum of 3 inches of cover to the subgrade and 2 inches cover to the exposed surface. Dobies to support reinforcing on grade shall have an integral wire tie. Wire ties shall be bent away from the exterior surfaces of the concrete.

705.7.1.7 Earthwork

At a minimum, the following areas shall be compacted to at least 90 percent of maximum density as determined by ASTM 1557 (Modified Proctor); the following additional requirements may be required by the geotechnical report:

a. The 12 inches of subgrade immediately beneath concrete lining (both channel bottom and side slopes)

b. Top 12 inches of maintenance road

c. Top 12 inches of earth surface within 10 feet of concrete channel lip

d. All fill material

705.7.1.8 Bedding

A geotechnical report shall be submitted to the local entity and/or the CCRFCD which addresses the need to provide bedding beneath the concrete channel section.

705.7.1.9 Underdrain and Weepholes

The necessity for longitudinal underdrains and weepholes shall be addressed in a geotechnical report submitted to the local entity and/or the CCRFCD for the specific concrete channel section.

705.7.1.10 Roughness Coefficients

Manning's "n" values for concrete lined channels are as follows:

Finish Type	"n" value
Trowel Finish	0.013
Float Finish	0.015
Light Broom Finish	0.015 to 0.016
Unfinished	0.017

705.7.1.11 Low Flow Channel

The bottom of the concrete channel shall not be constructed with a defined low flow channel but shall be adequately cross sloped to confine the lowflows to the middle or one side of the channel.

705.7.1.12 Concrete Cutoffs

For continuously reinforced concrete channels, a transverse concrete cutoff shall be installed at each expansion joint and shall extend a minimum of 3 feetbelow the bottom of the concrete slab. The cutoff shall extend across the entire width of the channel lining.

For panel type concrete construction, a transverse concrete cutoff shall be installed at any expansion joint, at a maximum spacing of 90 feet and shall extend a minimum of 3 feet below the bottom of the concrete slab. The cutoff shall extend across the entire width of the channel lining.

For either type of concrete channel lining, longitudinal cutoffs, a minimum of 2 feet in depth, at the top oflining are required to ensure integrity of the concrete lining where the channel intersects a natural wash or where sheet flow can enter the channel.

705.7.1.13 Minor Drainage Channels

Six-inch thick concrete with No. 4 reinforcing steel bars at 12 inches on center each way will normally be utilized for these channels when they are to be maintained by a public entity or constructed in a public right-of-way or easement. Alternative sections will be allowed where approved by the responsible entity.

705.7.2 Special Consideration for Super-Critical Flow

Super-critical flow in an open channel in an urbanized area creates hazards which the designer must take into consideration. Careful attention must be taken to ensure against excessive waves which may extend down the entire length of the channel from only minor obstructions. Imperfections at joints mayrapidly cause a 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 dynamicload 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 super-critical flows, minor downstream obstructions do not create any backwatereffect. Backwater computation methods are applicable for computing the water-surface profile or the energy gradient in channels having a super-critical flow; however, the computations must proceed in a downstream direction. The designer must take care to ensure against the possibility of unanticipated hydraulic jumps forming in the channel.

705.8 Hot Weather Concreting Procedures

705.8.1 General

Hot weather is defined as any combination of high air temperature, low relative humidity and wind velocity tending to impair the quality of fresh or hardened concrete or otherwise resulting in abnormal concrete properties. The effects of hot weather are most critical during periods of rising temperature, falling relative humidity, or both. Precautionary measures required on a calm, humid day should be less strict than those required on a dry, windy day even if air temperature is identical. The maximum temperature of cast-in-place concrete should not exceed ninety (90) degrees F (32 degrees C) immediately before placement. The consistency of the concrete as placed should allow the completion of initial finishing operations without the addition of water to the surface.

705.8.2 Preparation for Placing and Curing

Preparations for placing and curing in hot weather include recognition at the start of the work that certain abnormal conditions may exist. These conditions may require some items of preparation, that cannot readily be provided at the last minute, suggest that strength reduction, as well as other detrimental effects of hot weather, are directly proportional to the amount of retempering water added.

705.8.3 Construction Practices and Measures

The following list of construction practices and measures including those as described in *Hot Weather Concreting* ACI 305, may be used to reduce or avoid the potential problems of hot weather concreting:

1. Before any concrete is placed, adequate provisions should be readily available to protect the concrete from any impending weather conditions.

2. Plan the job to avoid adverse exposure of the concrete to the environment. If possible, schedule placing operations during times of the day or night when weather conditions are favorable.

3. Protect the concrete against moisture loss at all times during placing and throughout its curing period.

4. Keep the period between mixing and delivery to an absolute minimum.

705.8.4 Curing

705.8.4.1 <u>General</u>

Preparation for placing concrete in hot weather includes the special provisions necessary for its proper protection and curing, since hot weather causes rapid drying. To avoid serious damage and cracking, facilities must be ready to promptly protect all exposed surfaces from drying. Water curing is preferred for most concrete work, but it is recognized that prompt application of white-pigmented curing compound (ASTM C 309) Type 2, is more practical for curing vast areas of flatwork on sub-grade in the form of highway paving and canal lining. Other alternatives for curing are described in ACI 308 *Standard Practice* *for Curing Concrete*. Curing should commence immediately upon completion of the finish. In the event that the application or placement ofthe curing medium is delayed, curing should be done by the water method as described below.

705.8.4.2 Water Method

A fog nozzle should be used generously to cool the air, to cool the forms and steel immediately ahead of concrete placing operations, and to lessen rapid evaporation from the concrete surface before and after each finishing operation. After all finishing operations are complete, a final curing membrane should be applied.

705.9 Other Channel Linings

Other channel linings include all channel linings which are not discussed in the previous sections. These include composite lined channels which are channels where two or more different lining materials are used (i.e., riprap bottom with concrete side slope lining). They also include 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 ofthe 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 in the Clark County area

g. Ease of repair of damage section

h. Past case history (if available) of the lining system in other arid areas

These linings will be allowed on a case by case basis. Because of the potential significant unknown problems with these lining types, concurrence with the local entity and/or the CCRFCD on the design items to be addressed as well as the final design will be required. The local entity and/or the CCRFCD reserve the right to reject the proposed lining system in the interests of operation, maintenance, and protecting the public safety.

706 HYDRAULIC DESIGN STANDARDS FOR IMPROVED CHANNELS

Presented in this section are the hydraulic design standards for design of improved channels. The standards included herein are those standards which 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. For the design of channel confluences, information is included in publications such as the U. S. Army Corps of Engineers' Hydraulic Design of Flood Control Channels (EM-1110-2-1601).

706.1 Sub-Critical Flow Design Standards

The following design standards are to be used when the design runoff in the channel is flowing in a sub-critical condition ($F_r < 1.0$). Furthermore, all super-critical channels must be designed with the limits as stated in Section 702.2.

706.1.1 Transitions

For the purposes of this MANUAL, sub-critical transitions occur when transitioning one sub-critical channel section to another sub-critical channel section (expansion or contraction) or when a sub-critical channel section is steepened to create a super-critical flow condition downstream (i.e., sloping spillway entrance). Several typical sub-critical transition sections are presented in **Figure 707**. 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.

706.1.1.1 Transition Energy Loss

The energy loss created by a contracting section may be calculated using the following equation:

$$H_t = K_{tc} [(V_2^2 / 2g) - (V_1^2 / 2g)] \tag{737}$$

Where H_t = Energy Loss (ft)

K_{tc} = Transition Coefficient-Contraction

V_1 = Upstream Velocity (fps)

V_2 = Downstream Velocity (fps)

g = Acceleration of Gravity (ft/sec^2)

K_{tc} values for the typical transition sections are presented in **Figure 707**.

Similarly, the energy loss created by an expanding transition section may be calculated using the following equation:

$$H_t = K_{te} [(V_1^2 / 2g) - (V_2^2 / 2g)] \tag{738}$$

Where H_t = Energy Loss (ft)

K_{te} = Transition Coefficient-Expansion

V_1 = Upstream Velocity (fps)

V_2 = Downstream Velocity (fps)

g = Acceleration of Gravity (ft/sec^2)

K_{te} values for the typical transition sections are also presented in **Figure 707**.

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.

706.1.1.2 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 to be 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:

$$L_t = \geq 0.5\, L_c\, (T_w) \qquad (739)$$

Where L_t = Minimum Transition Length (ft)

L_c = Length Coefficient

T_w = Difference in the Top Width of the Normal Water Surface Upstream and Downstream of the Transition

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.

706.1.2 Bends

The allowed radius of curvature in sub-critical channels is based on a theoretical maximum allowed rise in the super-elevated water surface of 0.5 feet. Therefore, the minimum allowed radius of curvature of the channel centerline shall be determined from the following equation:

$$r = C\, (V^2\, T_w) / S_e\, (g) \qquad (740)$$

Where r = Radius of Curvature (ft)

C = Super-Elevation Coefficient (= 1 for Sub-Critical Flow)

S_e = Super-Elevation Water Surface Increase (ft)

T_w = Top Width of the Design Water Surface (ft)

V = Mean Design Velocity (fps)

g = Acceleration of Gravity (ft/sec^2)

In no case shall the radius of curvature be less than 50 feet.

706.1.3 Freeboard

All channels shall be constructed with a minimum freeboard determined as follows:

$$F_b = 0.5 + V^2 / 2g \qquad (741)$$

Where F_b = Freeboard Height (ft)

V = Mean Design Velocity (fps)

g = Acceleration of Gravity (ft/sec^2)

In no case shall the freeboard be less than 1 foot. All channel linings must extend to the freeboard height. At channel bends, 0.5 feet shall be added to the above determined freeboard elevation.

706.2 Super-Critical Flow Design Standards

The following design standards are to be used when the design runoff in the channel is flowing in a super-critical condition (F_r > 1.0). Furthermore, all super-critical channels must be designed within the limits as situated in Section 702.2. Based on the phenomenon that occurs with high velocity, super-critical, concrete lined channels, a rectangular cross-section shall be the preferred section shape.

706.2.1 Super-Critical Transitions

The design of a super-critical flow to super-critical flow transition is much more complicated and requires special attention than a sub-critical transition design due to the potential damaging effects of the oblique hydraulic jump which occurs in 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. The simpler design analysis is to force a hydraulic jump (super-critical flow to sub-critical flow). However, hydraulic jumps must also be carefully designed to assure the jump will remain where the jump is designed to occur. For the Clark County area, hydraulic jumps shall not be designed to occur in an erodable channel section but only in an energy dissipation or drop structure. The design of these structures is presented in Section 1100 (Additional Hydraulic Structures).

706.2.1.1 Contracting Transitions

Presented in **Figure 708** is an example of super-critical 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 2. 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 $\$_1$. Since the flow pattern is symmetrical, 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, $\$_2$. 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:

$$L_t = (b_1 - b_3) / 2 \tan 2 \qquad (742)$$

Where L_t = Transition Length (ft)

b_1, b_3 = Upstream and Downstream Topwidth of Flow (ft)

2 = Wall Angle as Related to the Channel Centerline (Degrees)

Using the continuity principle,

$$b_1 / b_3 = (Y_3^{1.5} / Y_1)(F_3 / F_1) \qquad (743)$$

Where Y_1, Y_3 = Upstream and Downstream Depths of Flow (ft)

F_1, F_3 = Upstream and Downstream Froude Numbers

Also, by the continuity and momentum principals, the following relationship between the Froude number, wave angle, and wall angle is found to be:

$$\tan\theta = \frac{\tan\beta_1\,[(1+8F_1{}^2\sin^2\beta_1)^{0.5}-3]}{2\tan^2\beta_1+(1+8F_1{}^2\sin^2\beta_1)^{0.5}-1} \qquad (744)$$

Where $\$_1$ = Initial Wave Angle (Degrees)

Equations 742, **743**, and **744** can be used by trial and error to determine the transition length and wall angle. However, **Figure 709** 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 pre-determined channel sections using **Figure 709** 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, 2

Step 5: Using 2 and F_1, read the values of F_2 and Y_2/Y_1 for Section 1 from **Figure 709**. 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 709** 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 2 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 2 and redo Steps 5 through 7.

Step 8: Repeat the trial and error procedure until the computed Y_3/Y_1 is within the 5 percent of the actual Y_3/Y_1.

Step 9: Compute transition length using **Equation 742** and the last assumed value of 2.

Figure 709 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 designed 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 super-critical contractions, refer to CHOW, 1959.

706.2.1.2 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 super-critical expansion shall be as follows:

$$\mathbf{L_t \geq 1.5\ (T_w)\ F_{r1}} \tag{745}$$

Where L_t = Minimum Transition Length (ft)

T_w = Difference in the Top Width of the Normal Water Surface Upstream and Downstream of the Transition

F_{r1} = Upstream Froude Number

706.2.2 Bends

Bends in super-critical channels create crosswaves and super-elevated flow in the bend section as well as further downstream from the bend. In order to minimize these disturbances, the minimum radius of curvature in the bend shall be based on the super-elevation of the water surface not exceeding 2 feet. **Equation 740** in Section 706.1.2 shall be utilized to determine allowable radius. In no case shall the radius of curvature be less than 50 feet.

A value of C = 1.0 shall be used for all trapezoidal channels and for rectangular channels without transition curves. For rectangular channels with transition curves, a value of C = 0.5 value may be used.

706.2.3 Spiral Transition Curves

When a designer desires to reduce the required amount of freeboard and radius of curvature in a rectangular channel, spiral transition curves may be length of the transition curve measured along the channel centerline shall be determined as follows:

$$L_c = 0.32\, T_w\, V / y^{0.5} \qquad (746)$$

Where L_c = Length of Transition Curve (ft)

T_w = Top Width of Design Water Surface (ft)

V = Mean Design Velocity (fps)

y = Depth of Design Flow (ft)

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.

706.2.4 Freeboard

Adequate channel freeboard above the designed water surface shall be provided and shall not be less than that determined by the following:

$$F_b = 1.0 + 0.025\, V\, (d)^{1/3} \qquad (747)$$

Where F_b = Freeboard Height (ft)

V = Velocity (fps)

d = Depth of Flow (ft)

Freeboard shall be in addition to super-elevation, standing waves, and/or other water surface disturbances. See Section 706.2.6 for additional information on standing waves.

The channel lining side slopes shall be extended, as a minimum, to the freeboard elevation.

706.2.5 Super-Elevation

Super-elevation of the water surface shall be determined at all channel bends and design of the channel section adjusted accordingly. Super-elevation of the water surface is limited to 2 feet in the CCRFCD area.

The design increase (super-elevation) of the water surface at channel bends shall be calculated using the following:

$$S_e = CV^2T_w / gr \tag{748}$$

Where S_e = Super-Elevation Water Surface Increase (ft)

V = Average Flow Velocity (fps)

T_w = Normal Channel Flow Top Width (ft)

r = Channel Centerline Radius (ft)

g = Acceleration of Gravity (ft/sec^2)

C = 0.5 For Rectangular Channels With Spiral Transition Curves

1.0 For All Other Channel Sections, With or Without Spiral Transition Curves

706.2.6 Slug Flow

Pulsating flow occurs in a steep channel. The uniform super-critical flow depth breaks into a train of traveling waves or pulses. A pulsating flow consists of two parts: a roughly tumbling head and a smooth tail section. In general, pulsating flow can be classified into two categories: *slug flow* and *roll waves*.

Slug flows are characterized by surges of turbulent ridges with wave crests separated by highly agitated regions. For slug flow to form, the surface velocity must be greater than the wave speed. This ensures the steepening and breaking of waves at their upstream ends. Roll waves exhibit similar characteristics to moving oblique jumps. They are the result of transition from super-critical laminar to sub-critical turbulent state of flow. Mayer's study indicated that the range of Reynolds number for sub-critical flows was 1,000 to 4,000 and Froude number was greater than two.

Roll waves are characterized by transverse ridges of high velocity. The regions between the crests are quiescent. For roll waves to form, the surface velocity of the undisturbed flow must be less than the wave velocity. This ensures that the breaking of waves is at downstream ends (similar to moving hydraulic drop or expansion waves).

Studies of roll waves and slug flow were performed primarily in connection with the mechanism of instability of uniform flow on a steep slope. Uniform flow will become unstable when the velocity of flow is very high or the channel slope is very steep. When this happens, the free surface will form a series of roll waves. In 1945, Vedernikov developed a criterion, Vedernikov number, to identify if the uniform flow is stable. Vedernikov number (N_v) is defined as:

$$Nn = x(1 - R\frac{dP}{dA})F \tag{749}$$

Where: x = 2/3 for Manning's equation

R = hydraulic radius

dP = change in wetted perimeter

dA = change in flow area

F = Froude Number

Where:

$$R\frac{dp}{dA} = \frac{by}{2y+b}\frac{d(2y+b)}{d(by)} = \frac{2Y^*}{2Y^*+1} \tag{750}$$

Where b = channel bottom width

y = flow depth

Y* = y/b

To be a stable uniform flow, Nv shall be less than or equal to unity.

$$Nv = \frac{2}{3}(1 - \frac{2Y^*}{2Y^*+1})F \tag{751}$$

$$and\ \ Nv \le 1.0$$

The limiting Froude number for having a stable uniform flow in a rectangular channel is:

$$F \leq \frac{3}{2}(2Y^* + 1) \tag{752}$$

This conclusion agrees with the straight line on Plate B-7 of EM 1110-2-1601.

Similarly, limiting Froude numbers for trapezoidal channel with various side slopes, z, can be derived as:

$$F \leq \frac{3}{2}\left[\frac{(1+2kY^*)(1+2zY^*)}{1+2zY^* + 2kzY^{*2}}\right] \textit{in which } k = \sqrt{1+z^2} \tag{753}$$

Care has to be taken when designing a steep channel. Selections of y/b ratio, channel lining roughness and slope shall satisfy the above criteria to avoid roll waves. Otherwise mitigation shall be provided, including additional freeboard or rougher linings.

The height of roll waves can be approximated using the model of positive surges which have an advancing front with the profile similar to a moving hydraulic jump. When the height of the surge is small, the surge appears undular like an undular jump. When the height is increasing, the undulation will eventually disappear and the surge will have a sharp and steep front. Such an unsteady flow can be converted to a steady pattern by adding the wave speed to the flow field. Let the subscript 2 represents the design flow condition determined by Manning's formula for the selected channel cross section and the subscript 1 represents the section without roll waves defined by the limiting Froude number. Solving the continuity and momentum principles simultaneously yields:

$$V_2 = \frac{(V_1 - V_w)A_1 + V_w A_2}{A_2} \tag{754}$$

and the wave speed in a moving jump is:

$$V_w = V_1 + \sqrt{\frac{(A_2\bar{y}_2 - A_1\bar{y}_1)g}{A_1(1 - A_1/A_2)}} \tag{755}$$

$$h = y_2 - y_1 \tag{756}$$

in which V_w = wave velocity, V = flow velocity, A = flow area, g = gravitational acceleration, h = height of roll waves, and $\bar{y}$ = distance to centroid of flow area, approximated by 0.5y. Considering that the roll waves near the center of the channel section is similar to that in a rectangular channel, the height of roll waves can be derived as

$$h = \frac{C^2}{g}\left(\frac{2y_1}{y_1 + y_2}\right)\left(\frac{V_2}{C} - \frac{V_1}{C}\right) = \frac{C^2}{g}\left(\frac{2y_1}{y_1 + y_2}\right)(F_2 - F_1) \qquad (757)$$

$$C = V_w - V_2 \qquad (758)$$

in which F_2 = Froude number for design discharge, F_1 = limiting Froude number determined by Vedernikov's number, and C = celerity of wave. When the height of roll waves is small compared with the depth of flow, i.e. $y_1 . y_2$, Eq 757 is reduced to:

$$h = \frac{C^2}{g}(F_2 - F_1) \qquad (759)$$

The above procedure predicts the height of roll waves when the design condition, F_2, deviates from the limiting condition, F_1. It is suggested that design of channel freeboard must include the considerations of roll waves. Engineers must assure that the current design criteria for freeboard provides adequate height to accommodate roll waves on top of super-elevation. Otherwise, additional freeboard shall be added to the channel depth.

Design Freeboard = Maximum (Roll Waves or Empirical Freeboard)

A design example was developed to examine how this criteria will impact the selection of the channel cross-section. For instance, channel bottom widths of 10, 15, and 20 feet are considered to design a channel to pass 5,000 cubic feet per second on a slope of 3.0 percent with Manning's "n" of 0.014. The following table summarizes the recommended channel depth under the consideration of roll waves.

Bottom Width (ft)	Side Slope	Limiting Q (cfs)	Limiting Fr	Flow Fr	Y/B Ratio	Depth Y (ft)	Waves h* (ft)	Y+h (ft)
10	**0**	**11,484.7**	**5.03**	**2.19**	**1.175**	**11.75**	**-33.32**	**11.75**
	1	4,555.9	2.81	3.66	0.665	6.65	3.32	9.97
	2	4,560.4	2.82	3.83	0.564	5.64	3.02	8.66
	3	4,730.4	2.86	3.84	0.510	5.10	2.57	7.67
15	**0**	**5,244.8**	**3.00**	**2.88**	**0.500**	**7.50**	**-1.03**	**7.50**
	1	3,799.6	2.51	3.73	0.369	5.53	3.95	9.48
	2	4,026.8	2.61	3.84	0.328	4.92	3.26	8.18
	3	4,298.1	2.71	3.83	0.304	4.57	2.72	7.29
20	**0**	**3,672.9**	**2.36**	**3.21**	**0.287**	**5.73**	**3.90**	**9.63**
	1	3,287.4	2.26	3.76	0.237	4.75	4.22	8.97
	2	3,598.5	2.42	3.83	0.219	4.37	3.42	7.79
	3	3,907.9	2.54	3.82	0.206	4.12	2.85	6.97

*Note: A negative computed roll wave height implies roll waves do not exist.

The above example is for both rectangular and trapezoidal channels with varying side slopes. When the limiting Q (cfs) is higher than the design flow in the channel, then roll waves will not occur, and freeboard per **Equation 747** is the only consideration. This occurs in two of the rectangular examples in the table above, with a bottom width of 10 feet and a bottom width of 15 feet. Roll waves occur in all of the other channel configurations, which will increase the necessary depth of the channel.

707 CHANNEL APPURTENANCES

Presented in this section are the design standards for appurtenances to improved channels. All channels in the Clark County area shall be designed to include these appurtenances.

707.1 Low Flow Channel or Storm Drain

All channels, other than concrete lined, shall be constructed with a low flow channel or an underground low flow storm drain to carry nuisance base flows. The minimum capacity shall be 1.0 percent of the major storm flow but not less than 1 cubic foot per second. Low flow channels shall be constructed of concrete or other approved materials to minimize erosion and facilitate maintenance. The minimum low flow channel width shall be 4 feet with a minimum depth of 1 foot. The minimum low flow channel slope shall be 0.4 percent. Low flow channels with a slope of less than 0.4 percent will be reviewed on a caseby case basis by the local entity and/or the CCRFCD.

707.2 Maintenance Access Road

To insure the long term reliability of flood control channels to serve their function, access must be provided for maintenance, inspection and emergency response (operation access). The level and nature of operation access required should be appropriate for the size and type of channel in consideration whether it be a small local channel either privately or publically operated or a Regional channel.

Channels identified on the Flood Control Master Plan are intended to provide a high level of dependable flood control long into the future. For channels identified on the Flood Control Master Plan, at least one operation access 12 feet in width (vehicular operation access) should be provided at the top of bank. Being that the reasonable reach of maintenance equipment is typically no more than 30 feet, channels with top widths exceeding 30 feet should have vehicular operation access at the top of each bank or at the top of one bank and at the bottom. Where vehicular operation access is not provided at the top of a bank, a 5 foot wide bench (personnel operation access) should be provided at top of bank. When circumstances exist such that the bottom cannot provide an adequate platform for vehicular operation access, it should be provided at the top of each bank. Joint use of operation access is considered desirable and should be encouraged where public safety and facility function and operation can be preserved. Every effort should be made during design ofchannels identified on the Flood Control Master Plan to provide operation access to both sides ofan open channel according to the following guideline:

> **Provided one vehicular operation access along the entire length of a channel at the top of bank with a minimum passage width of 12 feet. A personnel access path will be provided on the top of bank opposite the vehicular access with a minimum passage width of 5 feet. For channels greater than 30 feet in top width, provide vehicular operation access at the top of both banks or vehicular operation access at the bottom of the channel and at the top of one bank. Joint use of operation access will be considered on a case by case basis.**

If operation access along the channel bottom is proposed, access ramps to the channel bottom shall be provided near major street crossings. The ramps shall be parallel to the channel centerline, have a minimum width of 12 feet, with a slope no greater than 10 percent in the same direction as the channel flow. Figure 710 depicts a typical access ramp. Dimensions may vary depending upon the channel slope and configuration. In all cases, the location of the access ramp shall be coordinated with the affected local entity. A general goal is to provide channel access ramps on a 0.5 to 1.0 mile interval.

Instances may arise involving geologic, existing development, physical, existing right-of-way and utility/infrastructure considerations, which could preclude strict adherence to this criteria. These situations will be reviewed on a case by case basis to approach maximum compliance with this standard. There may also be instances involving competing federal, state and local floodplain, design and safety regulations where it becomes necessary to exceed this minimum standard and additional right-of-way may be necessary to accommodate those requirements.

Cases will also arise where it becomes necessary for private development to construct channels identified on the Flood Control Master Plan to redefine existing flow patterns and characteristics to protect their developments from flood hazards. It is considered the responsibility of the developer to provide the development with adequate, dependable protection from flood hazards. In order for a channel to remain dependable it must have adequate operation access and integrate logically into the overall system. In these cases site planning must provide for these criteria and operation access provided to the local entity for effective operation of the channel to insure the long term function of the channel to protect the development from flood hazards.

Where channels are required for private development which are not identified on the Flood Control Master Plan (local channel), the local entity can use these guidelines at their discretion based upon the needs to provide adequate operation access to suit the individual design conditions. Other considerations in determining the provision of operation access for local channels may include but not be limited to:

1) Private or public operation responsibility;

2) The design flow rate and product of depth and velocity in the channel as an indication of the hazard associated with the water

flowing in the channel. Channels conveying more than 500 cfs are eligible for inclusion in the Flood Control Master Plan. Flows with a product of depth and velocity exceeding 8 can knock a person down and keep them down.

3) The availability, location and interval of access ramps to the bottom;

4) The availability, location and interval of personnel access points;

5) Whether the bottom provides a suitable platform for operation activities;

6) If pedestrian access is limited in some fashion to reduce the incidence of personal injury requiring emergency access;

Local channels constructed of concrete having a top-width of less than 60 feet which meet the requirements for vehicular access in the bottom of the channel, provide adequate movement of maintenance equipment, have access ramps at intervals not exceeding 0.5 miles and which provide personnel access on both banks will be acceptable.

It should be noted that for a completed local channel to become eligible for inclusion in the Flood Control Master Plan for continued operation with Flood Control District Funds, the level to which this standard has been met will be a primary consideration.

707.3 Safety Requirements

The following safety requirements are required for concrete lined channels. Similar safety requirements may be required for all other channels:

a. A 6 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. Access ladder-type steps shall be provided to facilitate swift-water rescue efforts. Ladders will be placed in pairs on 600-foot centers along the channel for both rectangular and trapezoidal channels. The pairs will be on opposite sides of the channel, offset longitudinally approximately by the bottom width of the channel. If access is difficult on one side of the channel, that side should be used as the upstream of the ladder pairs. The side of the channel with the better access should be downstream of the ladder pairs to allow rescue crews to mobilize their equipment. Main-gates in the perimeter fencing should be located at the ladder locations. The bottom rung of the ladder should be placed 1 foot vertically above the channel invert with the remaining rungs at a comfortable spacing for climbing. Yellow stripes (3 feet wide) will be painted on either side of each ladder to assist rescue crews in locating the ladders. The ladders should be made of either galvanized steel or some other UV-resistant material.

707.4 Outlet Protection

Scour resulting from highly turbulent rapidly decelerating flow is a common problem at conduit outlets. Both riprap and gabions have been used for outlet protection in unlined or vegetation lined channels. The following riprap protection is suggested for outlet Froude Numbers up to 2.5 (Froude Parameters $Q/D^{2.5}$ or $Q/WH^{1.5} < 14\ ft^{0.5}/sec$) where the outlet of the conduit slope is parallel with the channel gradient and the conduit outlet invert is flush with the riprap channel protection (USDCM, 1969). Here Q is the discharge in cubic feet per second, D is the diameter of a circular conduit in feet and W and H are the width and height of a rectangular conduit in feet.

707.4.1 Configuration of Protection

Figure 711 illustrates a typical riprap basin at a conduit outlet. The additional thickness of the riprap just downstream from the outlet is to assure protection from extreme flow conditions which might cause rock movement in this region. Note that protection is required under the conduit barrel and an end slope is provided to accommodate degradation of the downstream channel.

707.4.2 Rock Size

The required rock size may be selected from **Figure 712** for circular conduits and from **Figure 713** for rectangular conduits. **Figure 712** is valid for $Q/D^{2.5}$ # 6.0 and **Figure 713** is valid for $Q/WH^{1.5}$ # 8.0. The parameters in these two figures are:

a. $Q/D^{1.5}$ or $Q/WH^{0.5}$ in which Q is the design discharge in cubic feet per second and D is a circular conduit diameter in feet and W and H are the width and height of a rectangular conduit in feet.

b. Y_t /D or Y_t /H in which Y_t is the tailwater depth in feet, D is the diameter of a circular conduit and H is the height of a rectangular conduit in feet. In cases where Y_t is unknown or a hydraulic jump is suspected downstream of the outlet. Use Y_t /D = Y_t /H = 0.40 when using **Figures 712** and **713**.

The riprap size requirements in **Figures 712** and **713** are based on the following non-dimensional parametric equations (USDCM,1969):

Circular Culvert:

$$(d_{50}/D)\ (Y_t/D)^{1.2}/(Q/D^{2.5}) = 0.023 \qquad (760)$$

Rectangular Culvert:

(a) For Q/WH1.5 < 4.0, use (USDCM, 1969):

$$(d_{50}/H)\ (Y_t/H)/(Q/WH^{1.5}) = 0.014 \qquad (761a)$$

(b) For Q/WH1.5 ≥ 4.0, use (CHEN, 1970, 1969):

$$(d_{50}/H)\ (Y_t/H)^2/(Q/WH^{1.5})^{2.5} = 0.0019 \qquad (761b)$$

The rock size requirements were determined assuming that the flow in the culvert barrel is not super-critical. **Equations 760** and **761** can be used when the flow in the culvert is less than a pipe full and is super-critical if the value of D or H is modified for use with **Figures 712** and **713**. Whenever the flow is super-critical in the culvert, substitute the average depth (D_a) for D and average height (H_a) for H, in which D_a is defined as:

$$D_a = 1/2\ (D + Y_n) \qquad (762)$$

in which maximum D_a shall not exceed D, and:

$$H_a = 1/2\ (H + Y_n) \qquad (763)$$

in which maximum H_a shall not exceed H and:

D_a = A Parameter to be Used in **Figure 712** Whenever the Culvert Flow is Super-Critical

D = Diameter of a Circular Culvert, in ft

H_a = A Parameter to be Used in **Figure 713** Whenever the Culvert Flow is Super-Critical

H = Height of a Rectangular Culvert, in ft

Y_n = Normal Depth of Super-Critical Flow in the Culvert

707.4.3 Extent of Protection

The length of the riprap protection downstream from the outlet depends on the degree of protection desired. To prevent all erosion, the riprap must be continued until the velocity has been reduced to the allowable velocity in the outlet channel. The rate at which the velocity of a jet from a conduit outlet decreases is not well known. For the procedure recommended here, the velocity decrease is assumed to be related to the angle of lateral expansions, 2, of the jet. The velocity is related to the expansion factor, (1/(2 tan 2)), which may be determined directly using **Figures 714** or **715**.

Assuming that the expanding jet has a rectangular shape:

$$L = (1/(2 \tan 2)) (A_t / Y_t - W) \qquad (764)$$

In which:

L = Length of Protection, in ft

W = Width of the Conduit, in ft (Use Diameter for Circular Conduits)

Y_t = Tailwater Depth, in ft

2 = The Expansion Angle of the Culvert Flow

A_t = Q / V (765)

Q = Design Discharge in cfs

V = The Allowable Non-Eroding Velocity in the Downstream Channel in fps

A_t = Required Area of Flow at Allowable Velocity, in sq ft

In certain circumstances, **Equation 764** may yield unreasonable results. Therefore, in no case should L be less than 3D or 3H, nor does L need to be greater than 10D or 10H whenever the Froude parameter $Q/D^{2.5} \leq 6.0$ or $Q/WH^{1.5} \leq 8.0$. Whenever the Froude parameter is greater than these maximums, increase the maximum L required by one-fourth D or H for each whole number the Froude parameter is greater than 6 or 8 for circular or rectangular pipe, respectively.

707.4.4 Multiple Conduits

The procedures outlined in the sections above can be used to design outlet erosion protection for multi-barrel culvert installations by hypothetically replacing the multiple barrels with a single hydraulically equivalent rectangular conduit. The dimensions of the equivalent conduit may be established as follows: First, distribute the total discharge, Q, among the individual conduits. Where all the conduits are hydraulically similar and identically situated, the flow can be assumed to be equally distributed, otherwise, the flow through each barrel must be computed. Next, compute the Froude parameter $Q_i/D_i^{2.5}$ or $Q_i/W_i H_i^{1.5}$, where the subscript i indicates the discharge and dimensions associated with an individual conduit. If the installation includes dissimilar conduits, select the conduit with the largest value of the Froude parameter to determine the dimensions of the equivalent conduit. Make the height of the equivalent conduit, H_e, equal to the height, or diameter, of the selected individual conduit. The width of the equivalent conduit, W_e, is determined by equating the Froude parameter from the selected individual conduit with the Froude parameter associated with the equivalent conduit, $Q_e/W_eH_e^{1.5}$.

708 EXAMPLE APPLICATIONS

708.1 Example: Degradation and Aggradation

a. Channel Design Using the Equilibrium Slope Concept (Refer to Section 704.2.1.2). The following is an example of the procedure by which the equilibrium slope of a channel can be calculated. The physical layout of the system is given in the following figure. The upstream channelized section has been in existence for many years and has not changed significantly. It has been proposed that the channelization be carried out the remainder of the distance to the main river because of a proposed development along the unchannelized portion. Since the slope for the downstream section was greater, the designer of the channel decided to make a more confined channel than the upstream channel. The rationale was that the steeper slope could result in faster velocities and thus a smaller channel could handle the same amount of water and be cheaper to construct.

PASTE FIGURE IN THIS SPACE FROM CHEN

Physical Layout of Design Example

The pertinent information for checking the channel response is:

Variables	Upstream Channel	New Downstream Channel
Dominant Discharge (Mean Annual Flood)	1,000 cfs	1,000 cfs
Channel Shape	Trapezoidal	Trapezoidal
Channel Slope	0.01	0.02
Sediment Size Distribution	d_{50} = 2 mm, G = 3	d_{50} = 2 mm, G = 3
Channel Resistance (n)	0.020	0.020
Channel Base Width	50 ft	30 ft
Side Slopes	2H:1V	2H:1V

The first step is the computation of the sediment supply from the upstream channel. The flow conditions are computed assuming normal depth (the channel is steep). The sediment transport equation is (from **Table 703A**):

$$q_s = 7.44 \times 10^{-6} Y_h^{-0.02} V^{3.86}$$

The hydraulic conditions for the upstream channel are:

Thalweg Depth:	Y	=	1.79 ft
Area:	A	=	96.1 ft
Velocity:	V	=	10.4 fps
Top Width:	T	=	57.2 ft
Hydraulic Depth:	Y_h	=	1.68 ft
Froude Number:	F	=	1.41

The variable Y_h is the hydraulic depth (area divided by top width). Since the sediment transport equations were developed for a unit width channel, the hydraulic depth is a better representation of the average channel characteristics than the thalweg depth Y. The sediment transport rate for these conditions in the upstream channel is:

q_s = 0.0621 cfs

or

Q_s = 3.55 cfs

Q_s is the total sediment transport rate from the upstream channel and is obtained by multiplying the rate per unit width by top width.

The next step is determination of the equilibrium slope for the downstream channel with sediment supply rate of 3.55 cubic feet per second. This requires a trial and error procedure by which a given slope is chosen to compute the flow conditions and from the flow conditions the sediment transport rate is calculated. When the computed rate is equal to the sediment supply rate, the equilibrium slope has been found. For an initial guess, the design slope of 0.02 was chosen. The following table presents the results of the calculations:

Equilibrium Slope Calculations: Condition 1

Slope	Thalweg Depth (ft)	Area (ft^2)	Velocity (fps)	Top Width (ft)	Hydraulic Depth (ft)	Froude Number	Qs (cfs)
0.020	1.96	66.5	15.1	37.8	1.76	2.00	9.88
0.010	2.40	83.4	12.0	39.6	2.11	1.46	4.25
0.0080	2.56	89.8	11.1	40.2	2.23	1.31	3.19
0.0085	2.51	88.1	11.4	40.1	2.20	1.35	3.53

The final equilibrium slope:
S = 0.0085
Check incipient motion:
Shear J = (RS = 1.2
Critical particle size d_c = J / [0.047 ((- ($_s$)] = 0.25 ft = 76 mm
Aggradation/degradation analysis:
) Z = (S - S_o) L_x
= (0.085 - 0.02) x 1,000 = -11.5 ft
Therefore, a headcut of 11.5 ft results.

The conditions used, including incipient motion, fall within the sediment transport equation's range. All sediment smaller than 76 millimeters will be moving. Since none of the sediment present is this large, armoring will not be a factor. The equilibrium slope is 0.0085. This is substantially less than the 0.02 design slope. Over the 1,000 feet of the channel length, using the main river as a control, a headcut of 11.5 feet would develop. As a solution, a wider channel should be chosen or a series of drop structures could be used to control the bed degradation. This conclusion is drawn from the fact that the velocity in the downstream channel, if the same size as the upstream, would be higher and, therefore, the sediment transport rates would be higher. In order for the steeper channel to have an equal transport rate, it is necessary to reduce its velocity. If the slope is to remain constant, then the channel must be widened. As an example, if the downstream channel base width was increased from 30 feet to 60 feet, then the following equilibrium slope was calculated:

Equilibrium Slope Calculations: Condition 2

Slope	Thalweg Depth (ft)	Area (ft^2)	Velocity (fps)	Top Width (ft)	Hydraulic Depth (ft)	Froude Number	Qs (cfs)
0.020	1.31	82.1	12.17	65.3	1.26	1.91	7.48
0.010	1.61	102.0	9.81	66.5	1.53	1.40	3.30
0.011	1.57	99.0	10.11	66.3	1.49	1.46	3.70
0.0106	1.58	100.1	9.99	66.3	1.51	1.43	3.53

The final equilibrium slope:
S = 0.0106
Check incipient motion:
Shear J = (RS = 0.99
Critical particle size d_c = J / [0.047 ((- ($_s$)] = 0.21 ft = 62 mm
Aggradation/degradation analysis:
) Z = (S - S_o) L_x
= (0.0106 - 0.02) x 1,000 = -9.4 ft
Therefore, a headcut of 9.4 ft results.

It is cautioned though that channels with unreasonably high width to depth ratios will develop low flow channels which are more constrictive. The low flow channel will then have a flatter equilibrium slope than predicted by the analysis.

a. <u>Anti-Dune Trough Depth (Refer to Section704.2.1.3).</u> The anti-dune trough depth could be estimated from the following equation:

$$Z_a = 0.0137\ V^2$$

$$= 0.0137 \times 9.99^2 = 1.4 \text{ ft}$$

b. <u>Contraction Scour (Refer to Section 704.2.1.5).</u> If the channel width at a distance 400 feet upstream from the downstream channel mouth was contracted by a 7 foot long vertical abutment (bottom length = 7 feet and top length = bottom length + bank slope x bank height), then a contraction scour would occur and could be estimated from either a live-bed contraction scour equation or a clear-water contraction equation. To determine which equation is to be used, a critical velocity should first be determined by using the following equation:

$$V_c = 10.95\ Y^{1/6}\ d_{50}^{\ 1/3}$$

$$= 10.95 \times 1.51^{1/6} \times (2 / 304.8)^{1/3} = 2.2 \text{ fps}$$

Because the flow velocity was 9.99 feet per second, which was greater than this value of V_c, the contraction scour was estimated from the live-bed scour equation (**Equation 713**):

$$Y_2 = Y_1\ (Q_2 / Q_1)^{6/7}\ (W_1 / W_2)^{K_1}$$

where

Q_1 • Total Discharge - Discharge Blocked By the Abutment

$= 1{,}000 \times [\ 88.1 - (7 + 10) \times 1.51/2\ ] / 88.1 = 855$

K_1 Depends on the Ratio of V_*/T, in Which Shear Velocity

$$V_* = (g\ Y_1\ S_e)^{1/2}$$

$$= (32.2 \times 1.51 \times 0.0106)^{1/2} = 0.717 \text{ fps}$$

Particle fall velocity:

T = 0.5 fps,

For V_*/T = 1.4, K_1 = 0.64.

Then Y_2 = 1.51 x (1,000 / 855)$^{6/7}$ { 66.3 / [66.3 - (7 + 1.51 x 2)] }$^{0.64}$ = 1.9 ft

The contraction scour was then:

Z_{cs} = Y_2 - Y_o • 1.9 - 1.5 = 0.4 ft

c. Local Scour at Abutment (Refer to Section 704.2.1.6a). As described in the above example, a vertical round-nose abutment with a surface length of 10 feet was constructed normal to the flow. This abutment would also cause local scour. Two equations (HIRE equation and Froehlich equation) are presented in Section 704.2.1.6 for determining local scour at abutments. If L'/Y_1 is greater than 25, the HIRE equation is used. Otherwise, the Froehlich equation is used. For this example, L'/Y_1 = 10/1.51 = 6.6, which is less than 25. Therefore, the Froehlich equation was used in the example to estimate the local scour as follows:

Z_s = 2.27 K_1 K_2 $(L')^{0.43}$ $Y_a^{0.57}$ $Fr^{0.61}$ + Y_a

where K_1 = 1.0 for Round Nose Abutment

K_2 = $(2 / 90)^{0.4}$ = $(90 / 90)^{0.4}$ = 1.0 Y_a • Y_1 • 1.51 ft

Fr = 1.43

The local scour depth was estimated to be:

Z_s = 2.27 x 1.0 x 1.0 x $(10)^{0.43}$ x $(1.51)^{0.57}$ x $1.43^{0.61}$ + 1.51

= 11.1 ft

d. Total Scour (Refer to Section 704.2.1.7). For this example, the total scour at the abutment section 400 feet from the mouth of the downstream channel would be:

$$Z_t = Z_{lt} + Z_a/2 + Z_{cs} + Z_s$$

$$= 9.4 \times 400 / 1{,}000 + 1.4 / 2 + 0.4 + 11.1$$

$$= 16.0 \text{ ft}$$

708.2 Example: Super-Critical Contracting Transition

Problem: Design a super-critical contracting transition from a 10 foot wide rectangular channel to a 5 foot wide rectangular channel for a 100-year flow rate of 300 cfs. The channel bottom slope is 0.02 feet per feet with a Mannings "n" value of 0.013.

Solution:

Step 1: Determine upstream and downstream channel flow conditions:

Upstream Channel	Downstream Channel
Y_1 = 1.62 feet	Y_3 = 3.01 feet
V_1 = 18.53 feet per second	V_3 = 19.95 feet per second
F_1 = 2.57	F_3 = 2.03

Step 2: Compute Y_3 / Y_1:

$$Y_3 / Y_1 = 3.01 / 1.62 = 1.86$$

Step 3: Assume a trial value of 2 = 10 degrees

Step 4: Determine F_2 and Y_2 / Y_1 for Section 1 from **Figure 709** using 2 = 10 degrees and F_1 = 2.57:

$$F_2 = 1.9,\ Y_2 / Y_1 = 1.5$$

Step 5: Replacing F_1 and F_2, determine F_3 and Y_3 / Y_1 for Section 2 from **Figure 709** using $F_1 = F_2 = 1.9$ and 2 = 10 degrees:

$$F_3 = 1.4,\ Y_3 / Y_2 = 1.3$$

Step 6: Compute first trial value of Y_3 / Y_1:

$$Y_3 / Y_1 = (Y_2 / Y_1)\,(Y_3 / Y_2)$$

$$= (1.5)\,(1.3) = 1.95$$

Since 1.95 > 1.86, assume a smaller 2 and redo Step 4 through Step 6 until the trial Y_3 / Y_1 is within 5 percent of the computed Y_3 / Y_1. For this example, the difference in the values is less than 5 percent. Therefore, proceed to Step 7.

Step 7: Compute transition length, L_t, from **Equation 724**.

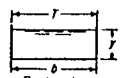
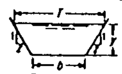
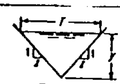
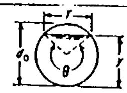
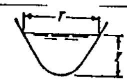
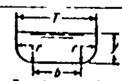
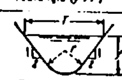
$$Lt = (b_1 - b_3) / 2 \tan 2$$

$$= (10 - 5) / 2 \tan 10 \text{ degrees}$$

$$= 14.18 \text{ ft}$$

Therefore, use a 14.2 foot long transition.

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	$2y\sqrt{1 + z^2}$	$\frac{zy}{2\sqrt{1 + z^2}}$	$2zy$	$\frac{1}{2}y$	$\frac{\sqrt{2}}{2} zy^{2.5}$
 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	$\frac{2}{3}Ty$	$T + \frac{8}{3}\frac{y^2}{T}$*	$\frac{2T^2y}{3T^2 + 8y^2}$*	$\frac{3}{2}\frac{A}{y}$	$\frac{2}{3}y$	$\frac{2}{9}\sqrt{6}\,Ty^{1.5}$
 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-bottomed 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 $P = (T/2)[\sqrt{1 + x^2} + 1/x \ln(x + \sqrt{1 + x^2})]$.

Revision	Date

WRC ENGINEERING

REFERENCE: Chow, V.T. Open-Channel Hydraulics
McGraw Hill Book Company, 1959.

TABLE 701

MANNING'S ROUGHNESS COEFFICIENTS FOR CHANNEL LINING TYPES

Channel Material	Roughness Coefficient (n)	
	Normal	Maximum
Corrugated Metal	0.025	0.030
Concrete**		
1) Trowel finish	0.013	0.015
2) Float finish	0.015	0.016
3) Unfinished	0.017	0.020
4) Shotcrete, Good section	0.019	0.023
5) Shotcrete, wavy section	0.122	0.025
Asphalt (use maximum value when cars are present)	0.016	0.020
Soil-Cement	0.020	0.025
Constructed channels with earth or sand bottom, sides of		
1) Clean earth; straight	0.022	0.025
2) Earth with grass and weeds	0.025	0.030
3) Earth with trees and shrubs	0.032	0.040
4) Shotcrete	0.022	0.025
5) Soil-cement	0.025	0.028
6) Concrete	0.020	0.024
7) Dry rubble or riprap	0.033	0.036
Natural channels with sand bottom, sides of	0.035	0.045
1) Trees and shrubs	0.032	0.040
2) Rock		
Natural channel with rock bottom	0.060	0.090
Overbank floodplains		
1) Desert brush, normal density	0.060	0.080
2) Dense vegetation	0.100	0.160

- Adapted from Chow (1959) and Aldridge and Garrett (1973).

** Manning's Coefficients for Clear Water Only

__ City of Tucson Standards Manual for Drainage Design

Revision	Date

REFERENCE:

MAXIMUM PERMISSIBLE MEAN CHANNEL VELOCITIES

Material / Lining	Maximum Permissible Mean Velocity (fps)
Natural and Improved Unlined Channels	
Fine sand, colloidal	1.50
Sandy loam, noncolloidal	1.75
Silt loam, noncolloidal	2.00
Alluvial silts, noncolloidal	2.00
Ordinary firm loam	2.50
Volcanic ash	2.50
Stiff clay, vary colloidal	3.75
Alluvial silts, colloidal	3.75
Shales and hardpans	6.00
Fine gravel	2.50
Graded loam to cobbles when noncolloidal	3.75
Graded silts to cobbles when colloidal	4.00
Coarse gravel, noncolloidal	4.00
Cobbles and shingles	5.00
Sandy silt	2.00
Silty clay	2.50
Clay	6.00
Poor sedimentary rock	10.0
Fully Lined Channels	
Unreinforced vegetation	5.0
Loose riprap	10.0
Grouted riprap	15.0
Gabions	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.

Revision	Date

POWER EQUATIONS FOR TOTAL BED MATERIAL DISCHARGE IN SAND- AND FINE-GRAVEL-BED STREAMS

$$q_s = C_1 \, (Y)^{C_2} \, (V)^{C_3}$$

d_{50}								
(mm)	0.1	0.25	0.5	1.0	2.0	3.0	4.0	5.0
(inches)	0.00394	0.00984	0.0197	0.0394	0.0787	0.118	0.157	0.197
G_r=1.0								
C_1	3.30×10^{-5}	1.42×10^{-6}	7.6×10^{-6}	5.62×10^{-6}	5.64×10^{-6}	6.32×10^{-6}	7.10×10^{-6}	7.78×10^{-6}
C_2	0.715	0.495	0.28	0.06	-0.14	-0.24	-0.30	-0.34
C_3	3.30	3.61	3.82	3.93	3.95	3.92	3.89	3.87
G_r=2.0								
C_1		1.59×10^{-5}	9.8×10^{-6}	6.94×10^{-6}	6.32×10^{-6}	6.62×10^{-6}	6.94×10^{-6}	
C_2		0.51	0.33	0.12	-0.09	-0.196	-0.27	
C_3		3.55	3.73	3.86	3.91	3.91	3.90	
G_r=3.0								
C_1			1.21×10^{-5}	9.14×10^{-6}	7.44×10^{-6}			
C_2			0.36	0.18	-0.02			
C_3			3.66	3.76	3.86			
G_r=4.0								
C_1				1.0510^{-5}				
C_2				0.21				
C_3				3.71				

Definitions: q_s=Unit sediment transport rate in ft^2/s (unbulked); V=Velocity in ft/s; Y=Depth in ft;
G_r=Gradation coefficient = $(d_{84}/d_{50} + d_{50}/d_{16})/2$.

Revision	Date

REFERENCE:

RANGE OF PARAMETERS FOR PARAMETERS FOR THE SIMONS-Li-FULLERTON METHOD

Parameters	Value Range
Froude Number	1 - 4
Velocity	6.5 - 26 ft/s
Manning Coefficient *n*	0.015 - 0.025
Bed Slope	0.005 - 0.040
Unit Discharge	10 - 200 cfs/ft
Particle Size	d_{50}

Revision	Date

REFERENCE:

CONSTANTS FOR SEDIMENT TRANSPORT EQUATION

Class	Size (in)	Geometric Mean (in)	C_1	C_2	C_3
Very fine sand	.00244 - .00492	.00346	58.50 x 10-6	1.040	3.20
Fine sand	.00492 - .00984	.00697	21.40 x 10-6	0.837	3.59
Medium sand	.00984 - .01969	.01394	6.47 x 10-6	0.535	4.05
Coarse sand	.01969 - .03937	.02783	2.90 x 10-6	0.239	4.36
Very coarse sand	.03937 - .07874	.05551	2.37 x 10-6	-0.044	4.44

Limitations: (1) Sediment sizes from .00244 to .07874 inches
(2) Channel bed slope from 0.002 to 0.010
(3) Flow velocities less than 25 fps.

Revision	Date

REFERENCE: Design Guidelines and Criteria for Channels and Hydraulic Structures on Sandy Soil Simons, Li, and Associates, 1981

CHECKLIST OF DATA NEEDS FOR NATURAL CHANNEL ANALYSIS

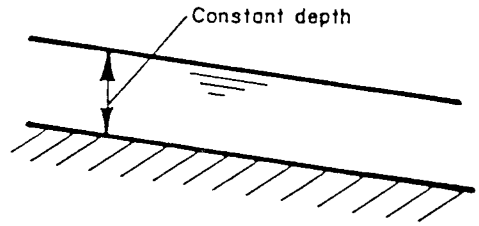
Description of Data	Degree of Data Importance
Hydrology	
Design discharges with anticipated urbanization	Primary
Design hydrographs with anticipated urbanization	Primary
Flood history (if available)	Secondary
Hydraulics	
Channel geometry	Primary
Bed slopes	Primary
Backwater calculations	Primary
Channel type (meandering, straight)	Secondary
Channel controls (drops, restrictions)	Primary
Roughness coefficients	Primary
Soils	
Bed material size distribution (geotechnical report)	Primary
Bank material size distribution (geotechnical report)	Primary
Hydraulic Structures (existing and planned structures)	
Plans and design details	Primary
Examine scour around existing hydraulic structures	Secondary
Aerial Photographs	
Recent and past photographs showing the channel and surrounding terrain	Primary
Land Use	
Existing land use	Primary
Planned land use maps	Primary
Field Surveys	
Topographic maps	Primary
On site inspection and photographs	Primary
Observe channel changes or realignment (if any) since last maps or photographs	Primary
Sample sediments	Primary
Subsurface exploration	Secondary

Notes:

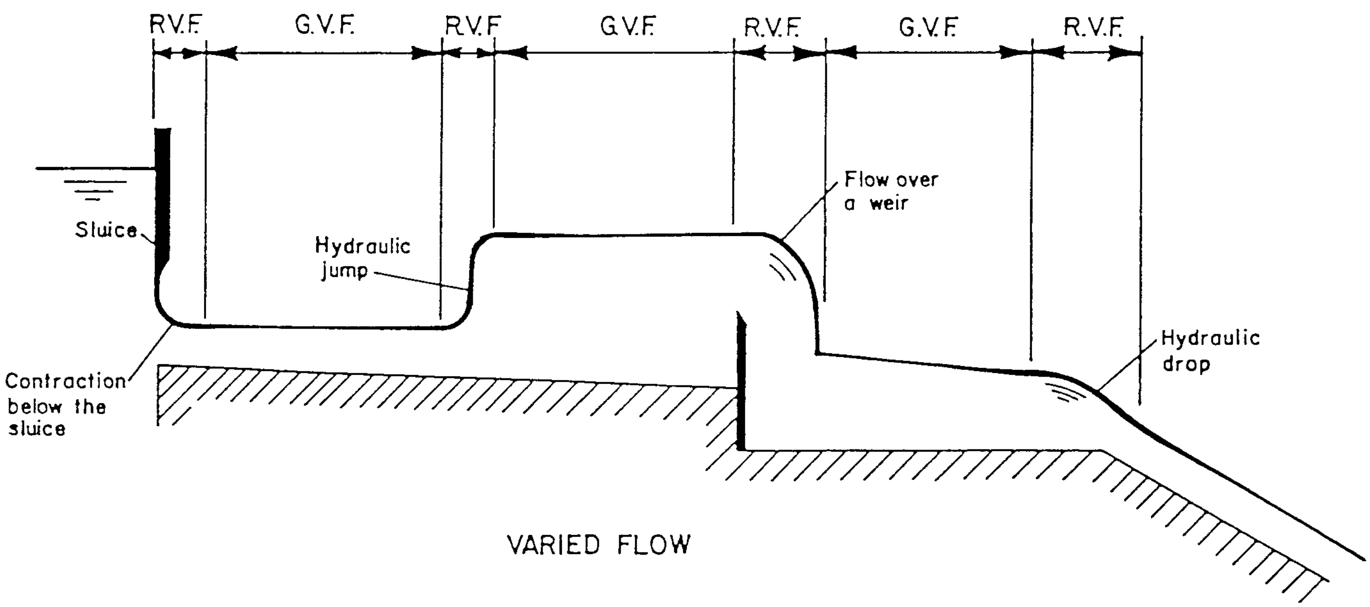
1. Primary - Required
2. Secondary - Desired but not essential

Revision	Date

OPEN CHANNEL FLOW CONDITIONS



UNIFORM FLOW
Flow in a laboratory channel



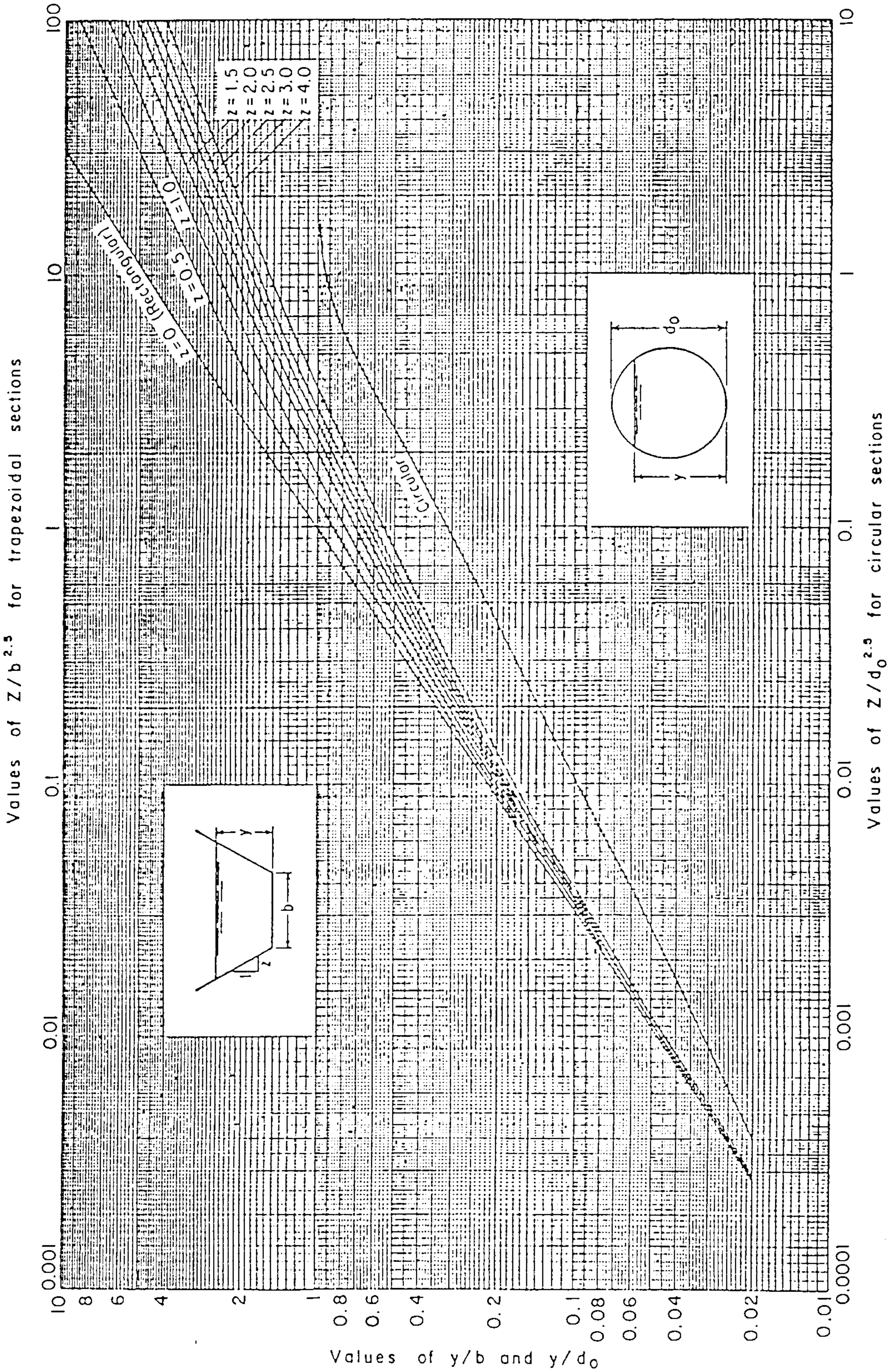
VARIED FLOW

G.V.F. - Gradually Varying Flow

R.V.F. - Rapidly Varying Flow

Revision	Date

CRITICAL DEPTH FOR TRAPEZOIDAL AND CIRCULAR SECTIONS



NOTES : Z - Section Factor = $Q / g^{0.5}$

z - Side Slope Horizontal Distance To 1 Vertical

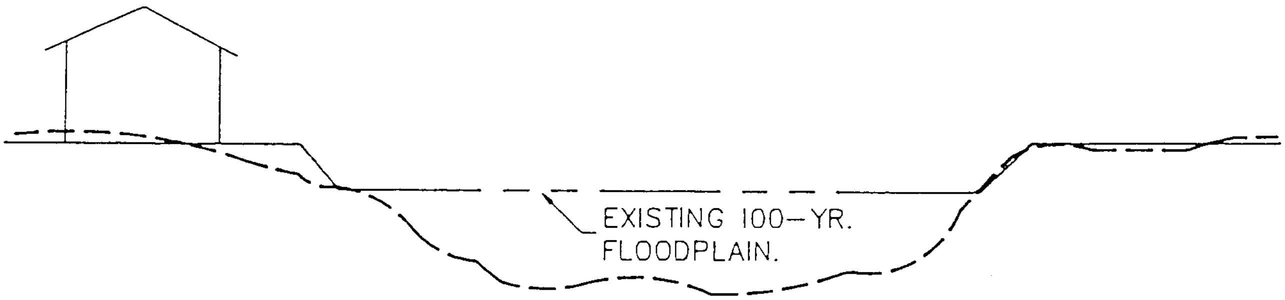
Revision	Date

WRC ENGINEERING

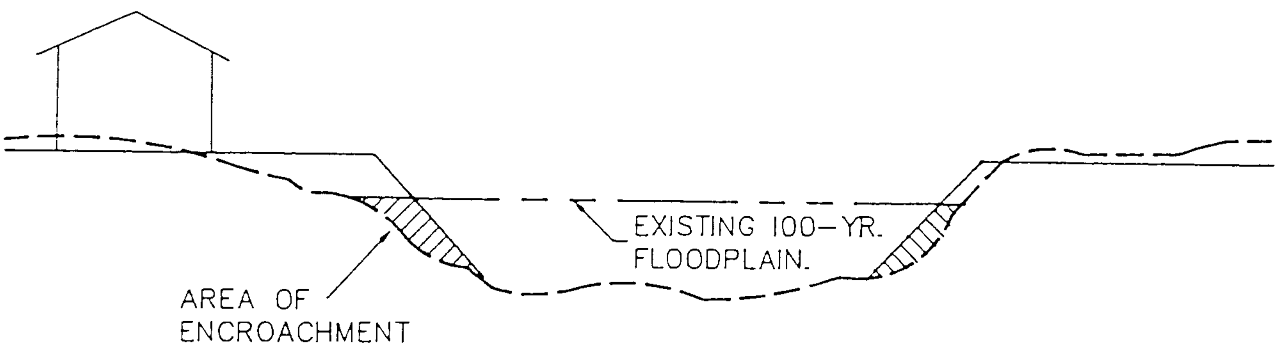
REFERENCE: Chow, V.T., Open-Channel Hydraulics McGraw Hill Book Company 1959

FIGURE 702

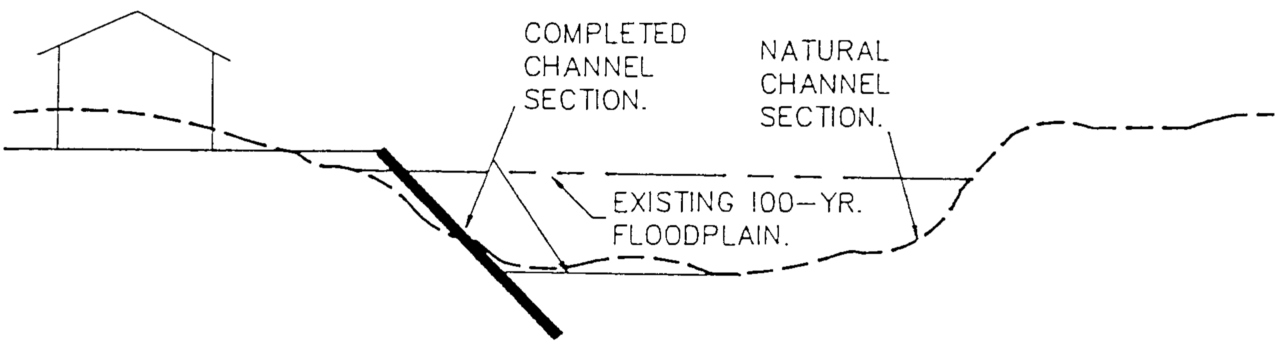
TYPICAL OPEN CHANNEL DESIGN SECTIONS
(NATURAL CHANNELS)



NATURAL UNENCROACHED CHANNEL



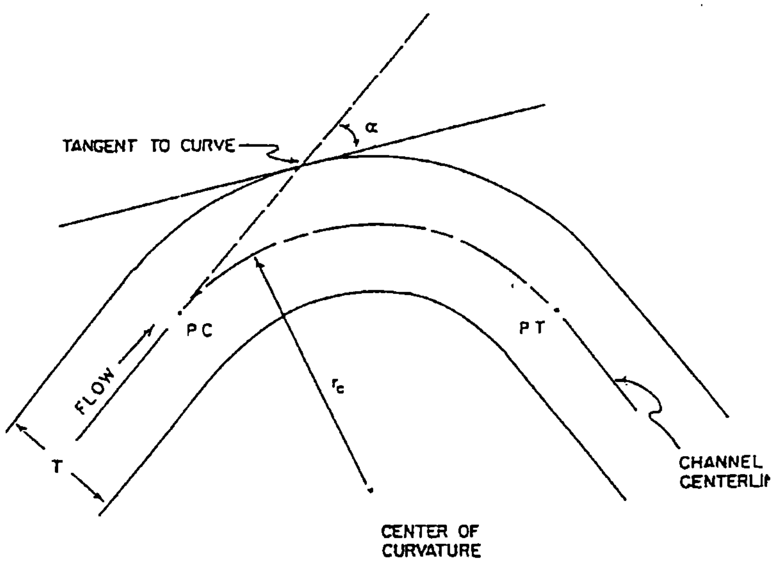
NATURAL ENCROACHED CHANNEL



BANK LINED AND TEMPORARY UNLINED CHANNEL

Revision	Date

ILLUSTRATION OF TERMINOLOGY FOR BEND-SCOUR CALCULATIONS



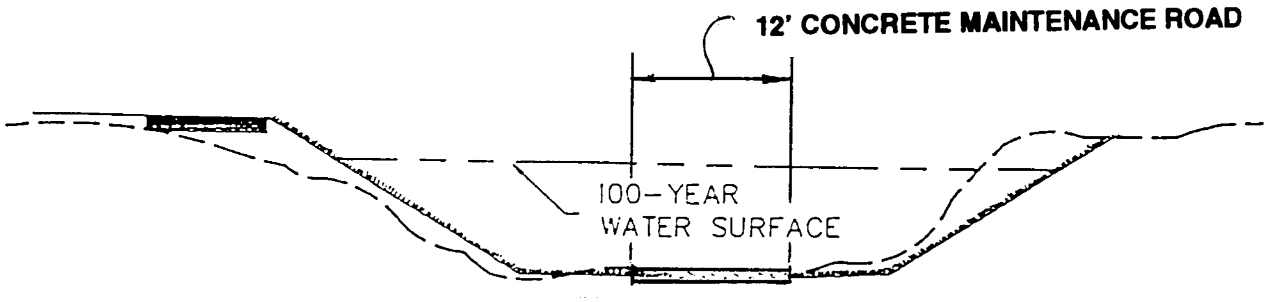
PT = Downstream point of tangency to the centerline radius of curvature
PC = Upstream point of curvature at the centerline radius of curvature

Revision	Date

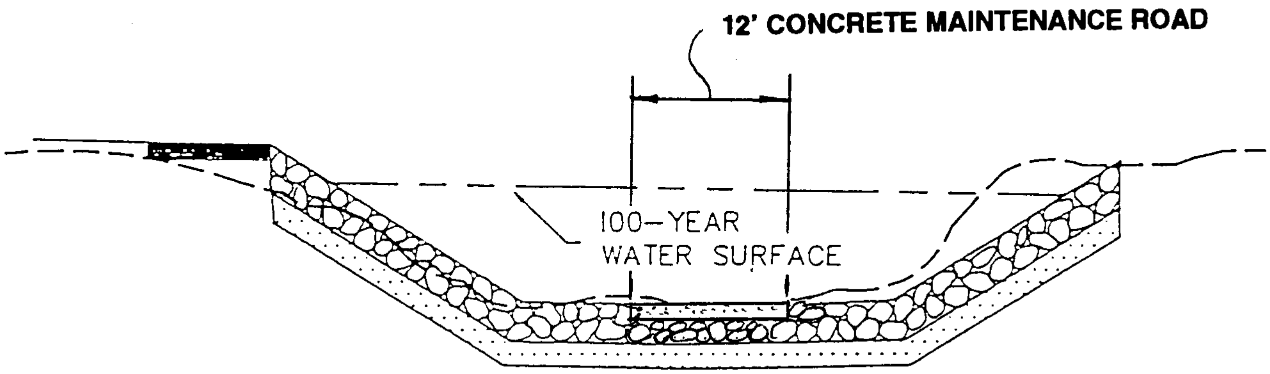
REFERENCE:

FIGURE 703A

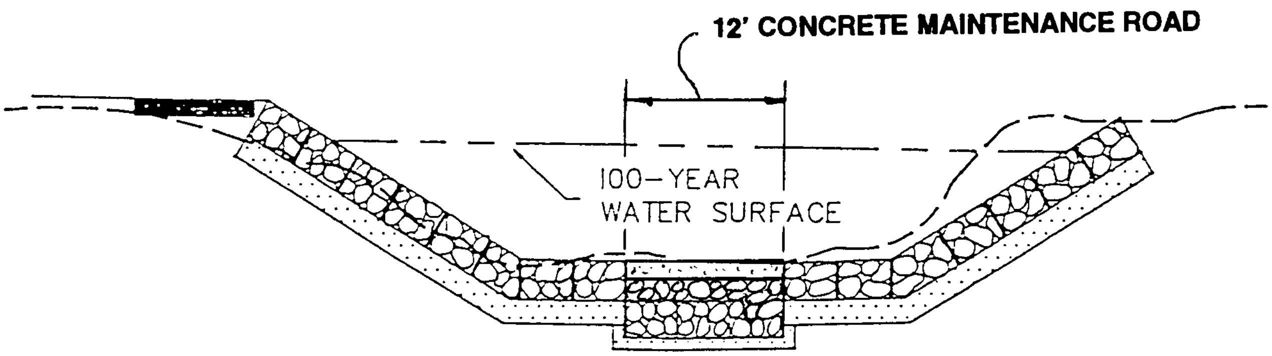
TYPICAL OPEN CHANNEL DESIGN SECTIONS (IMPROVED CHANNELS)



UNLINED AND VEGETATION LINED CHANNEL



RIPRAP LINED CHANNEL

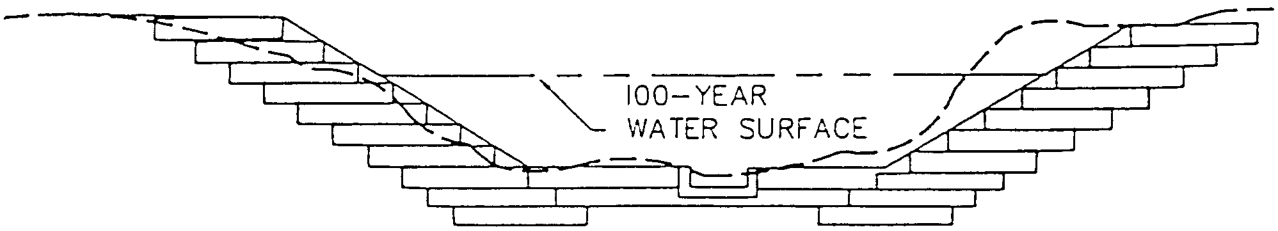


GABION LINED CHANNEL

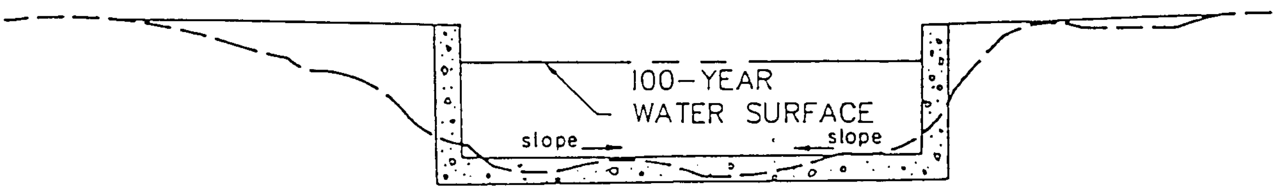
Revision	Date

REFERENCE:

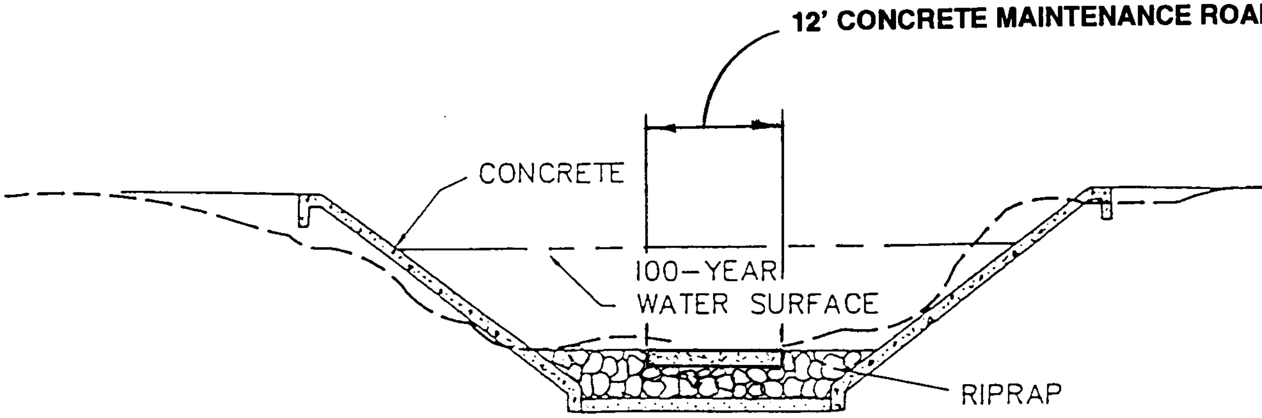
TYPICAL OPEN CHANNEL DESIGN SECTIONS (IMPROVED CHANNELS)



SOIL CEMENT LINED CHANNEL



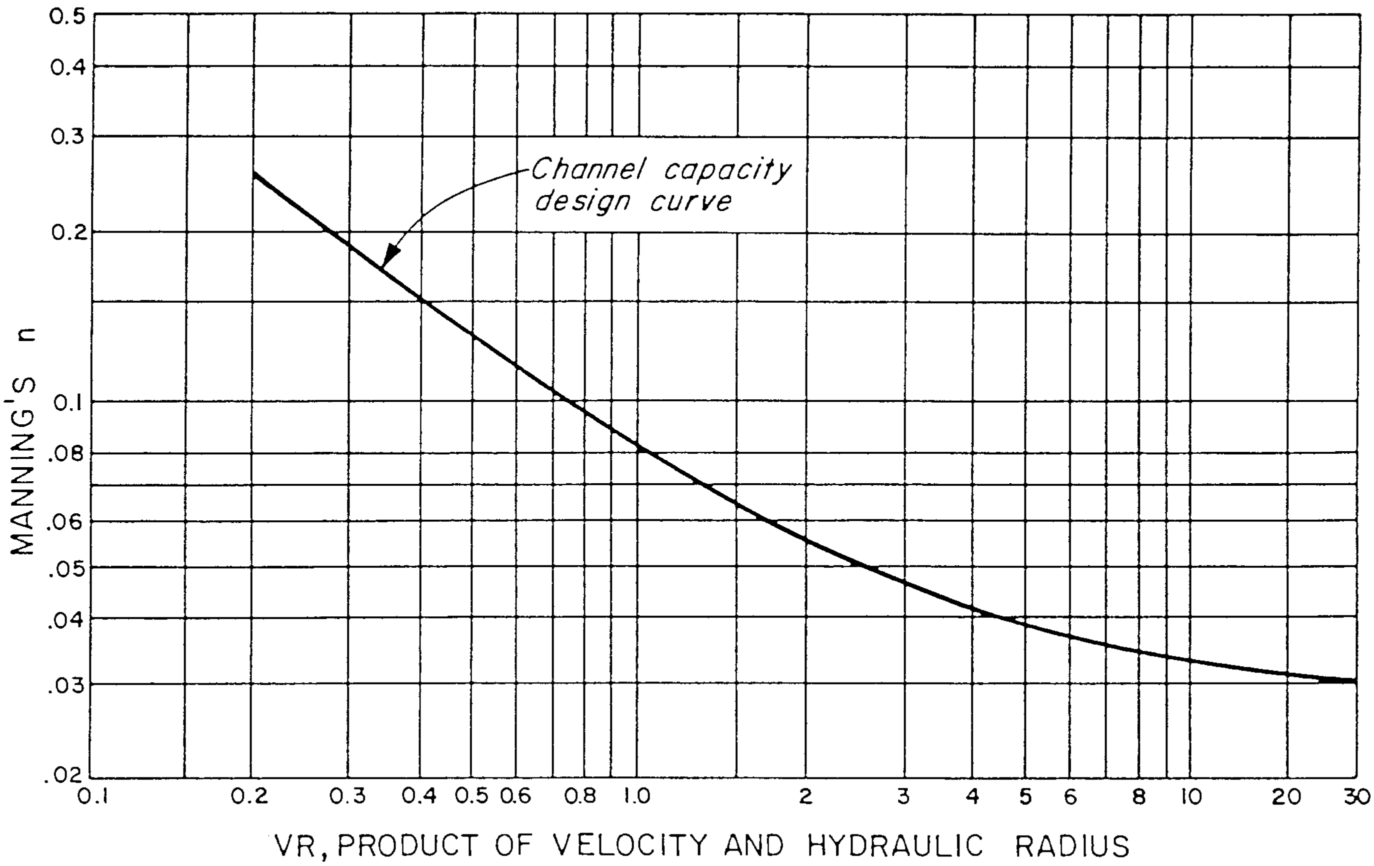
CONCRETE LINED CHANNEL



COMPOSITE LINED CHANNEL

Revision	Date

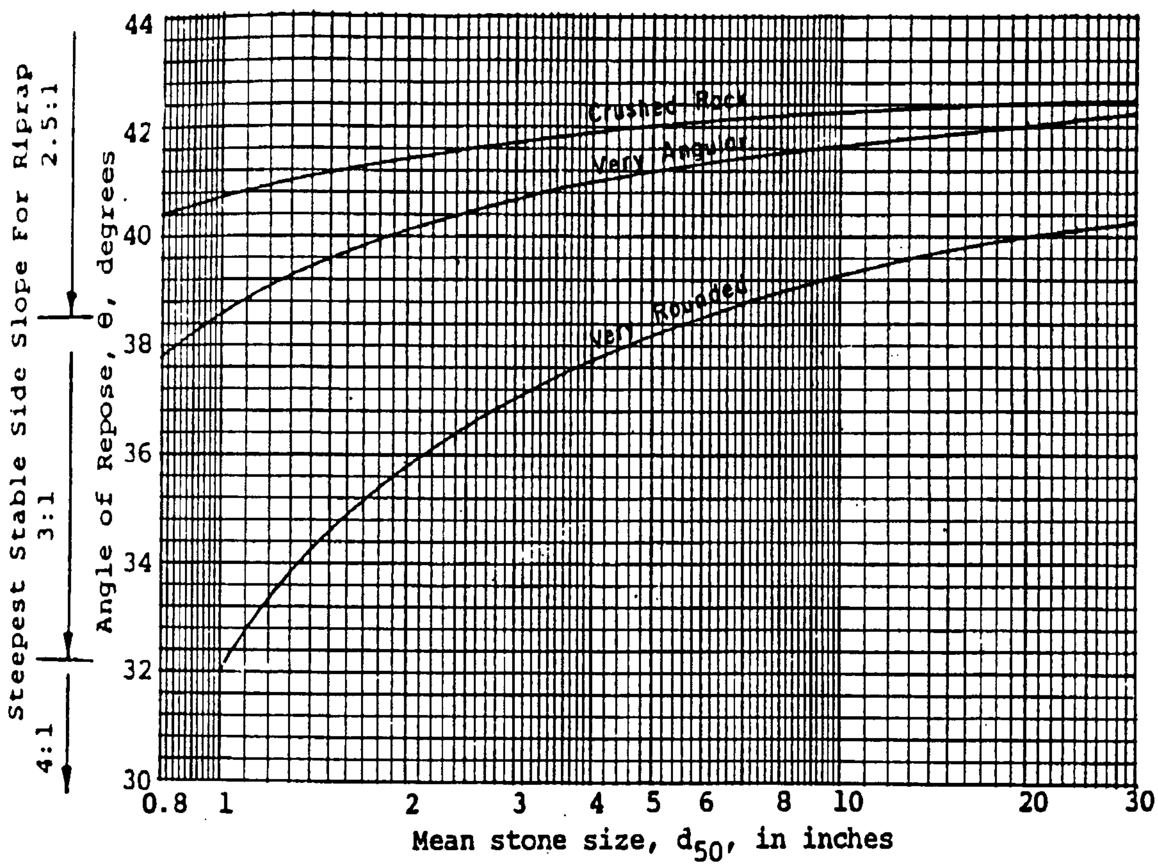
ROUGHNESS COEFFICIENT FOR VEGETATION LINED CHANNELS



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

Revision	Date

OPTIMUM RIPRAP SIDE SLOPE FOR A GIVEN SIZE RIPRAP



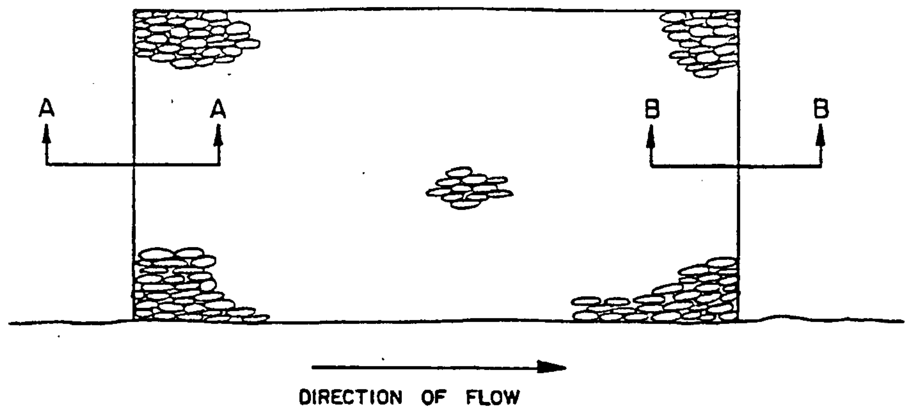
(After Soil Conservation Service, 1989)

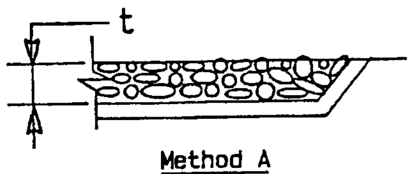
Revision	Date

REFERENCE:

FIGURE 705A

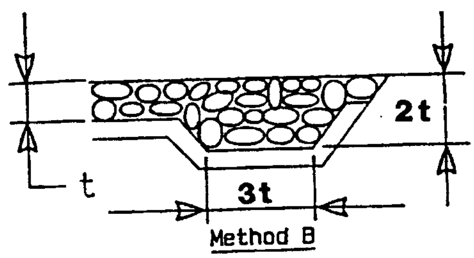
END PROTECTION

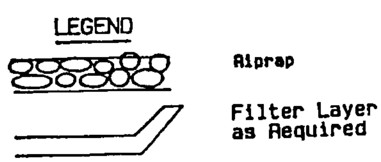




Notes:

1. Use Method B at section A-A (upstream edge).
2. Either Method A or Method B may be used at the downstre edge (section B-B).
3. t is the thickness of the riprap layer.



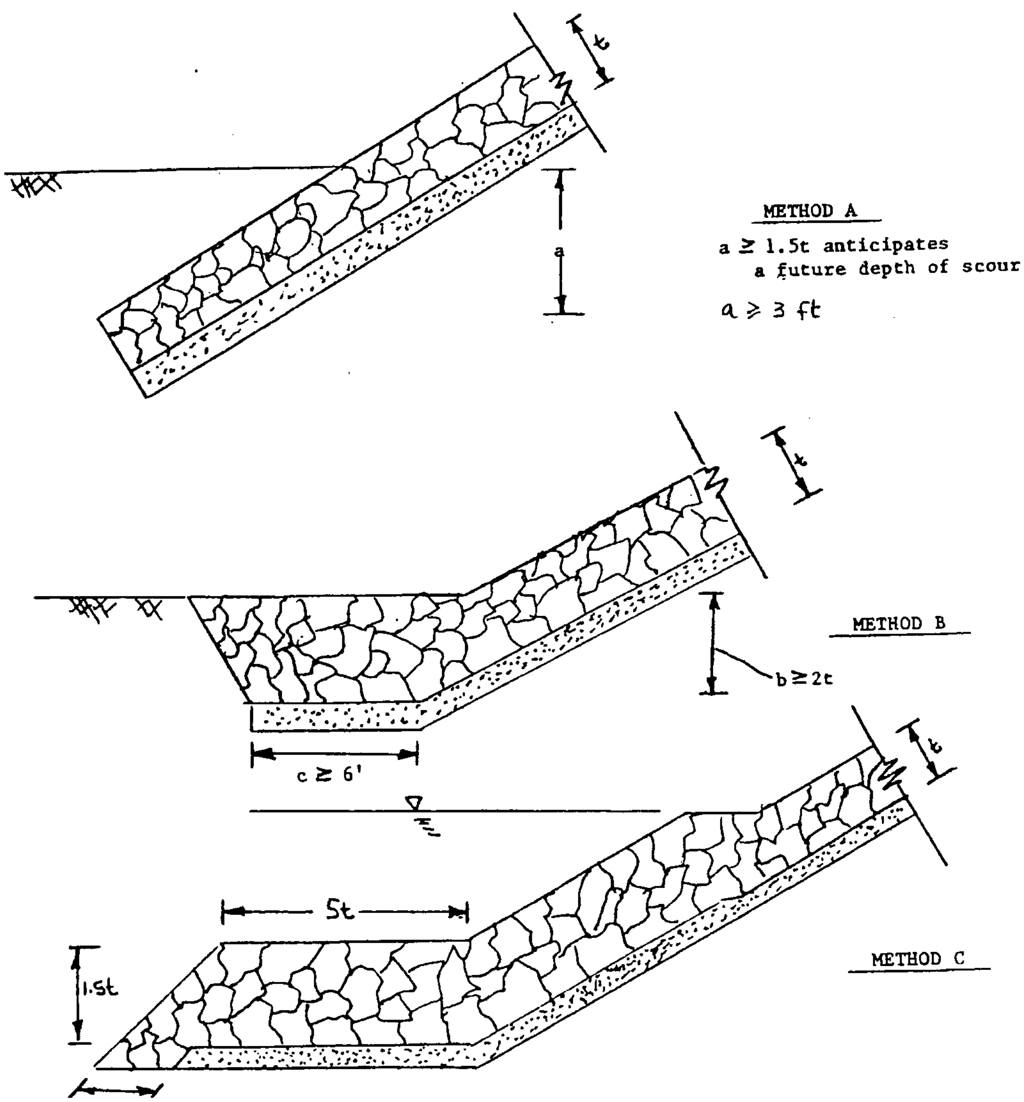


(After Blodgett, 1986)

Revision	Date

REFERENCE:

TOE PROTECTION ALTERNATIVE



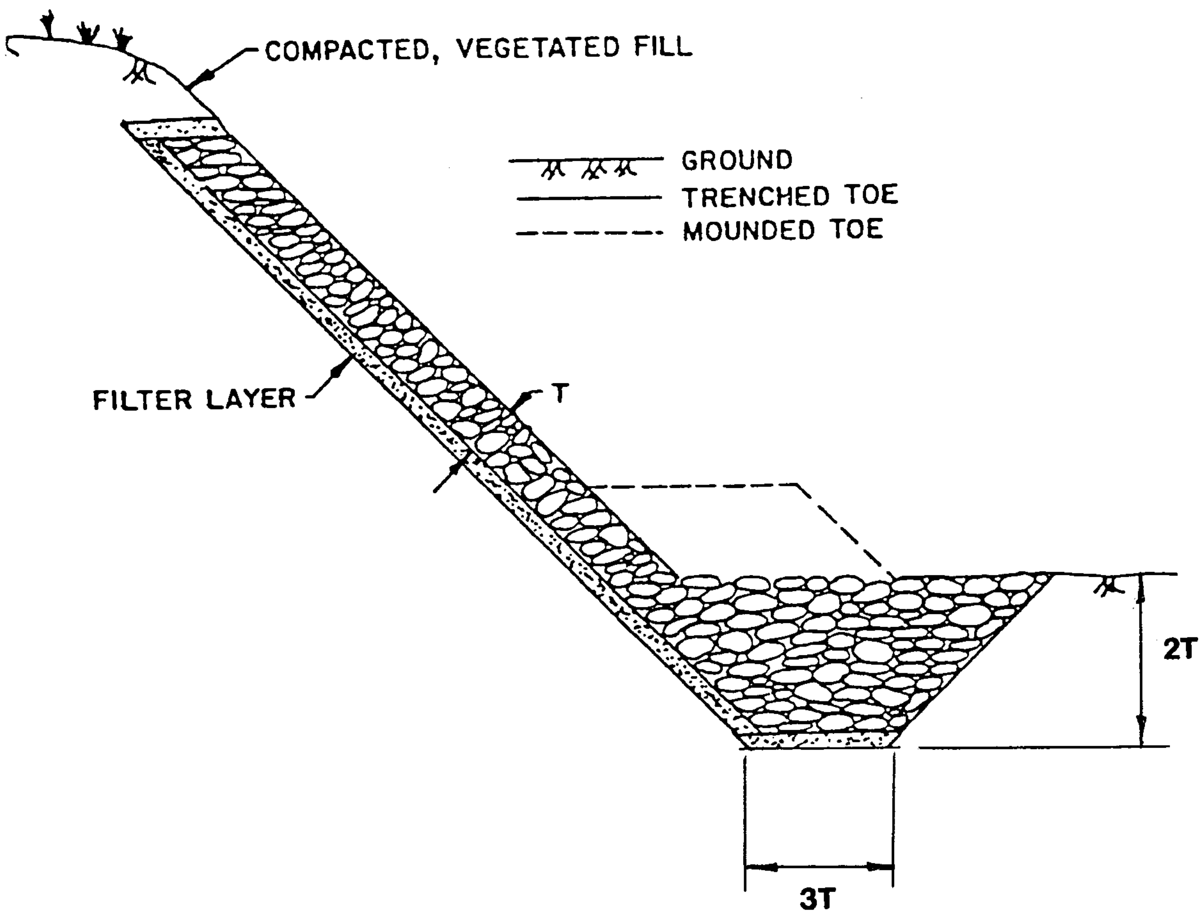
(After Blodgett, 1986)

Revision	Date

REFERENCE:

FIGURE 705C

TOE PROTECTION



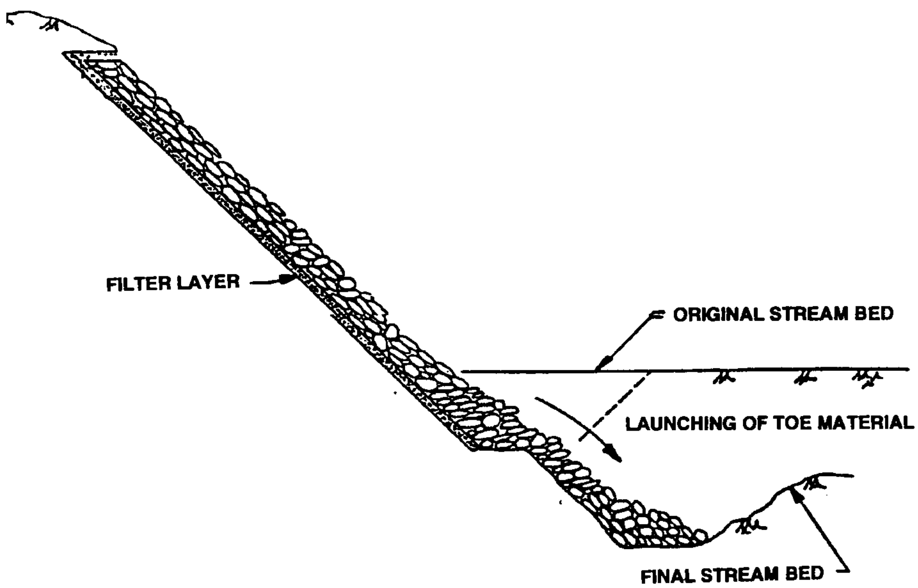
(After Blodgett, 1986)

Revision	Date

REFERENCE:

FIGURE 705D

LAUNCHING TOE MATERIAL



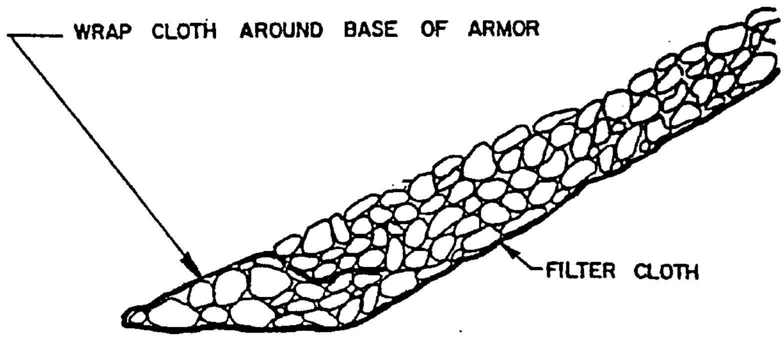
(After Blodgett, 1986)

Revision	Date

REFERENCE:

FIGURE 705E

"DUTCH TOE"

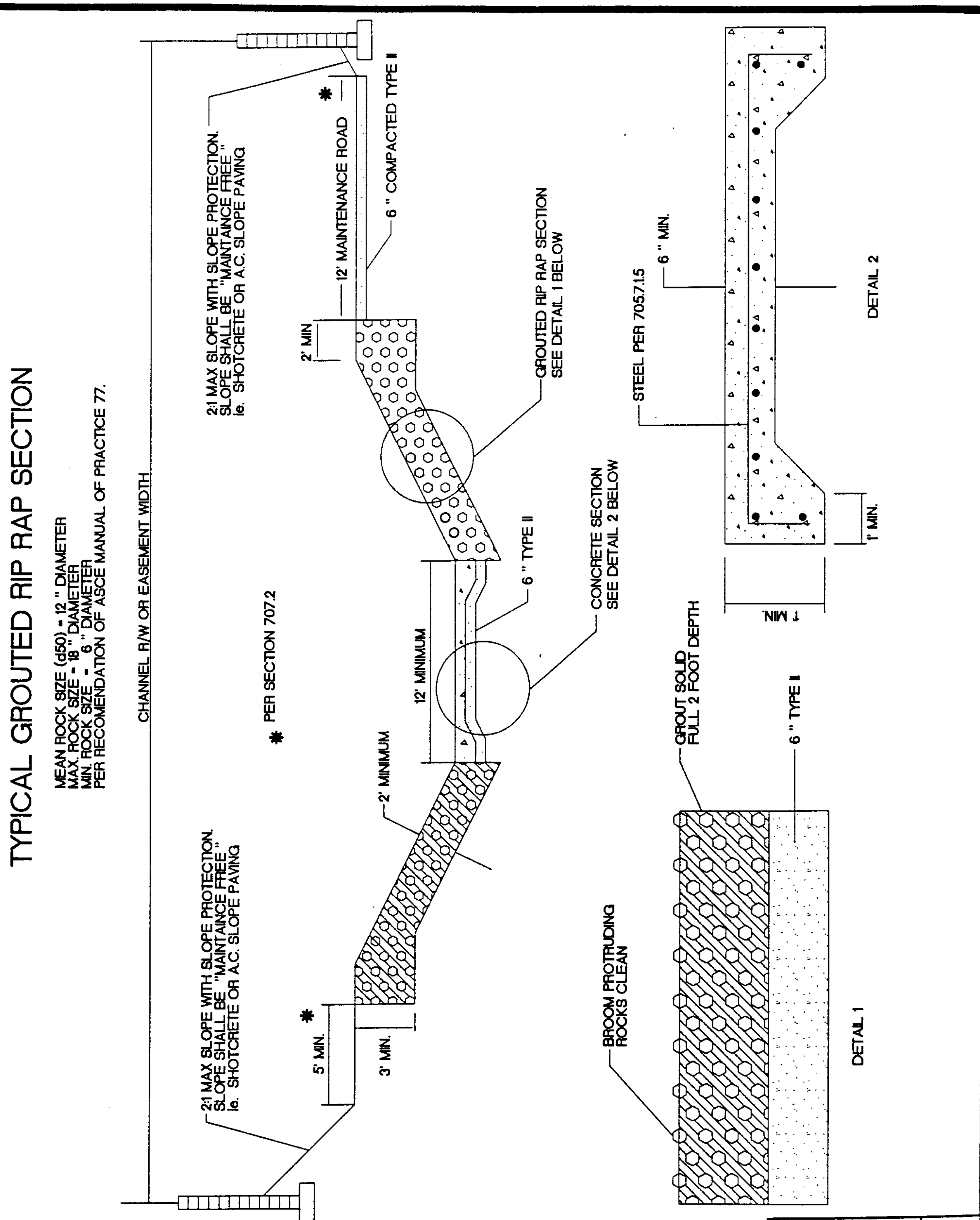


(After Blodgett, 1986)

Revision	Date

REFERENCE:

FIGURE 705F

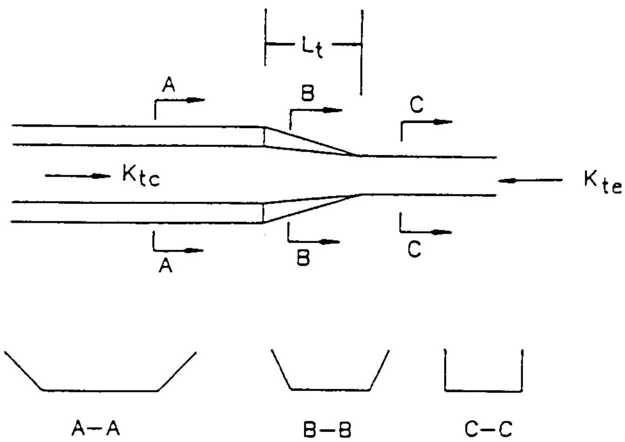


Revision	Date

REFERENCE:

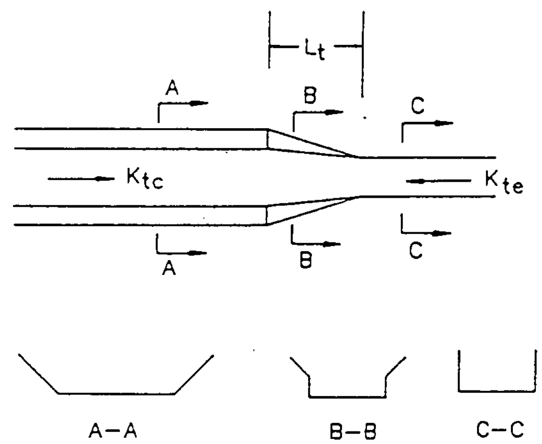
FIGURE 706

TYPICAL CHANNEL TRANSITION SECTIONS AND ENERGY LOSS COEFFICIENTS



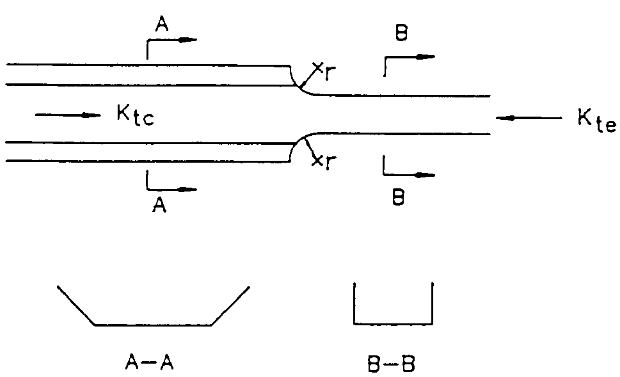
WARPED TRANSITION

K_{tc} (CONTRACTION) = 0.1
K_{te} (EXPANSION) = 0.2



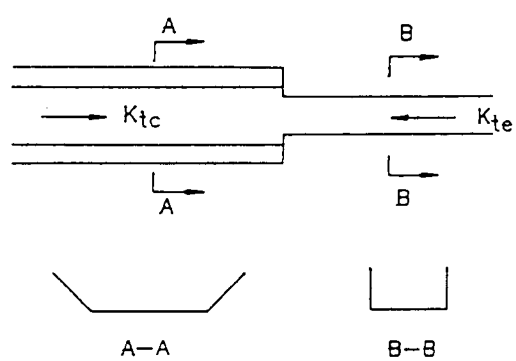
STRAIGHT–LINE TRANSITION

K_{tc} (CONTRACTION) = 0.3
K_{te} (EXPANSION) = 0.5



CYLINDER–QUADRANT

K_{tc} = .15
K_{te} = .25

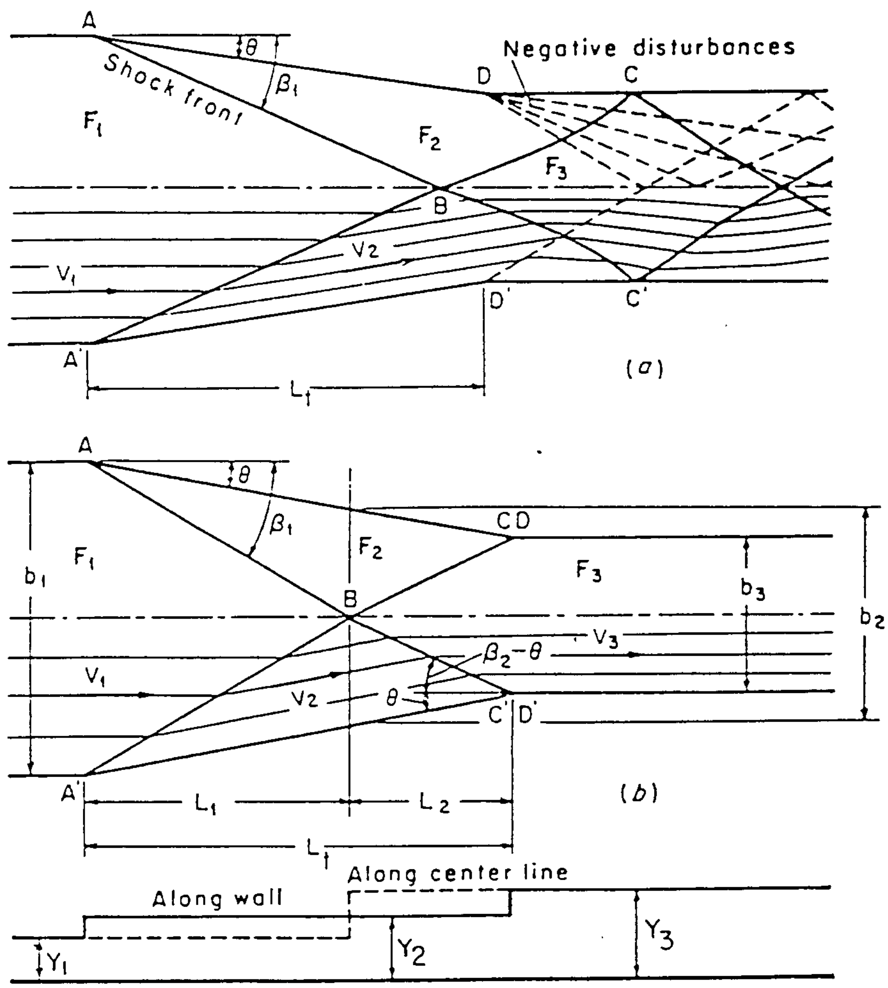


SQUARE – ENDED TRANSITION

K_{tc} = 0.30
K_{te} = 0.75

Revision	Date

TYPICAL CONTRACTING TRANSITION FOR SUPER - CRITICAL FLOW



Notes:

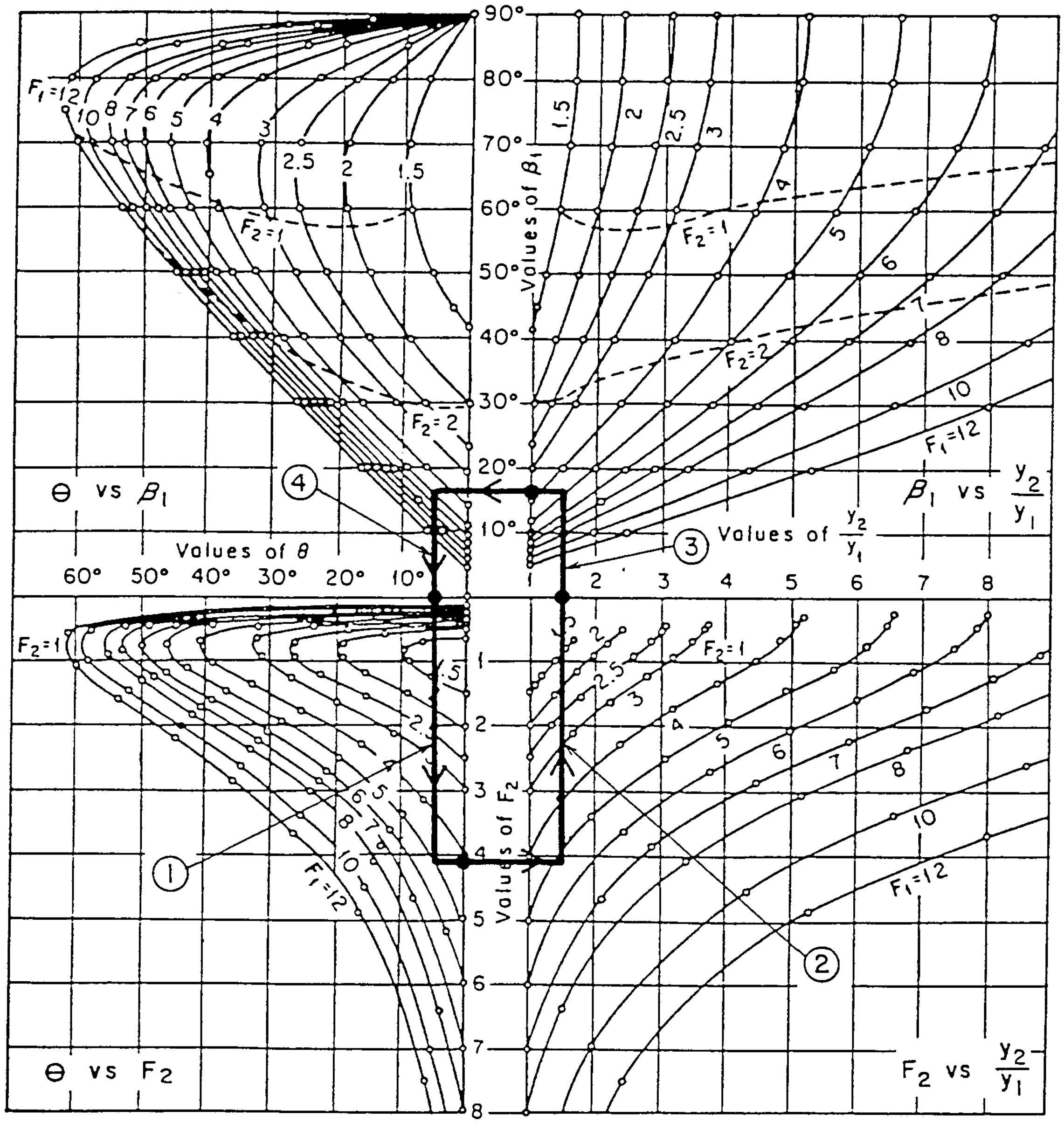
(a) General disturbance patterns

(b) Minimum downstream disturbance

(c) Schmatic profile

Revision	Date

DESIGN CHART FOR CONTRACTING TRANSITION FOR SUPER-CRITICAL FLOW



EXAMPLE : For Θ = 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 Θ = 5° (check)

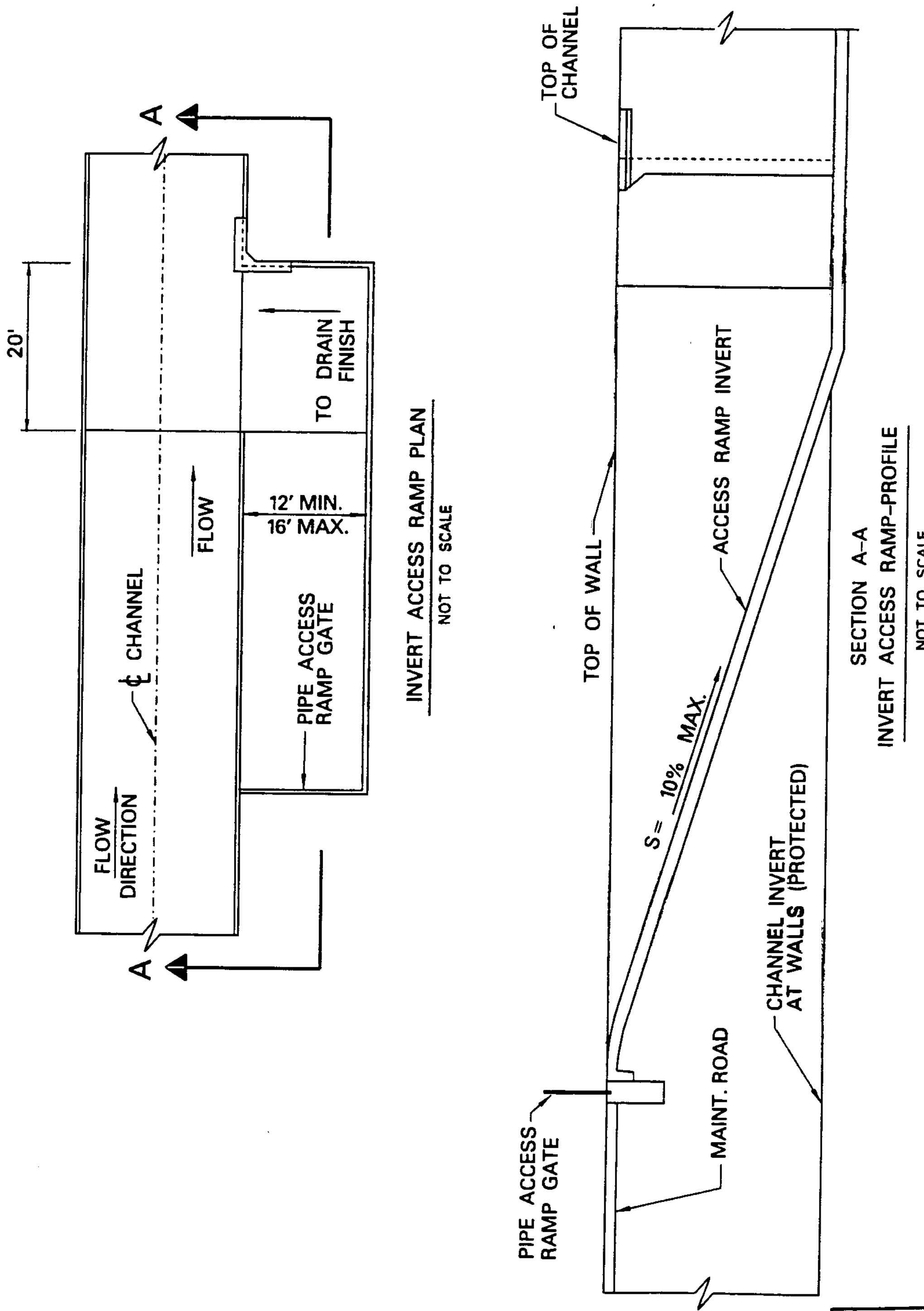
Revision	Date

WRC ENGINEERING

REFERENCE: Chow, V.T., Open-Channel Hydraulics McGraw Hill Book Company, 1959

FIGURE 709

TYPICAL CHANNEL ACCESS RAMP

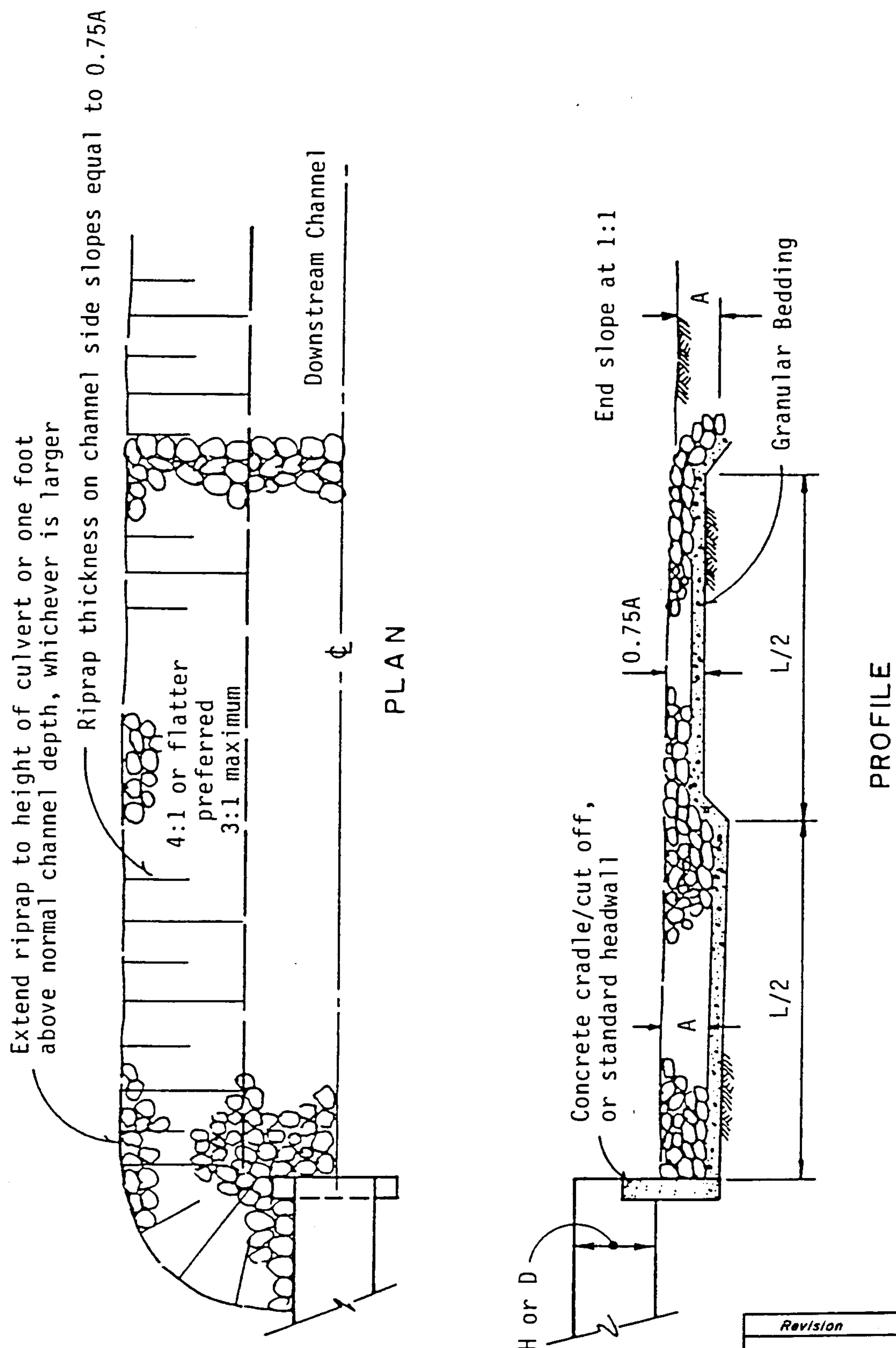


Revision	Date

REFERENCE:

FIGURE 710

CONDUIT OUTLET EROSION PROTECTION

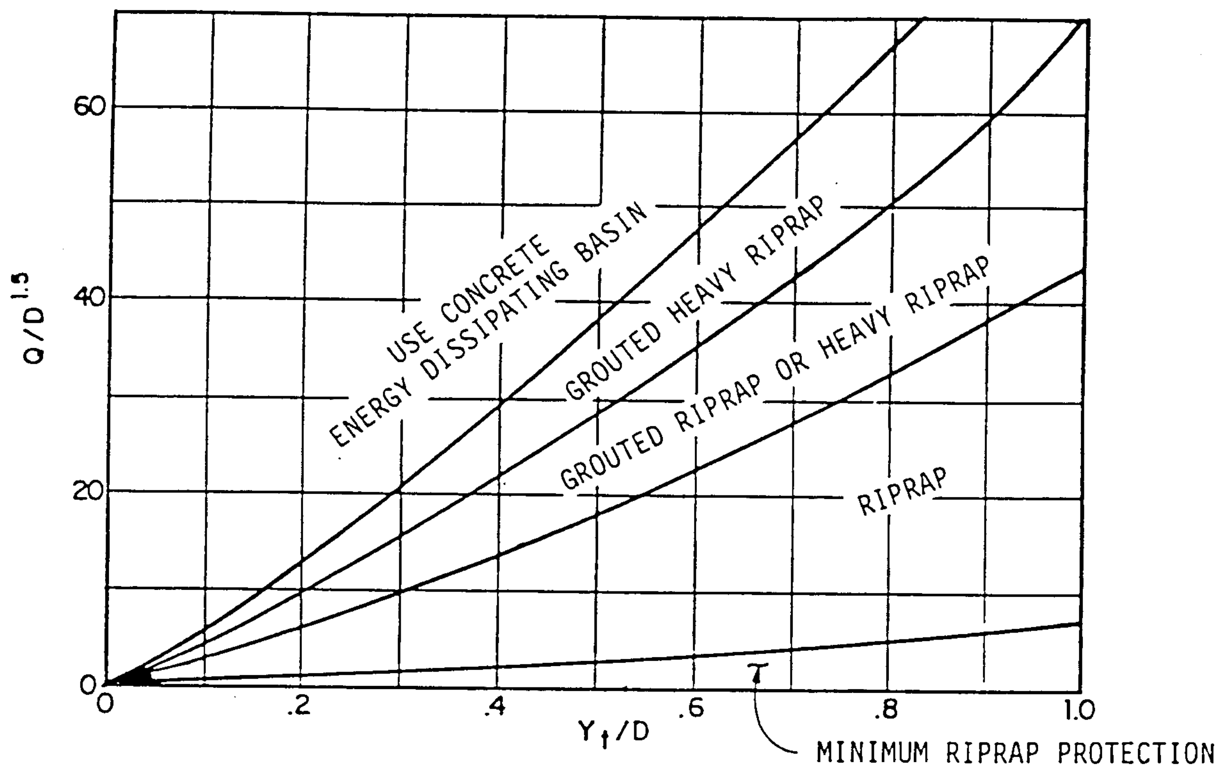


NOTE: A = 2 feet for riprap, 3 feet for heavy riprap.

Revision	Date

REFERENCE:
USDCM, DRCOG, March 1969 (with modifications)

RIPRAP EROSION PROTECTION AT CIRCULAR CONDUIT OUTLET

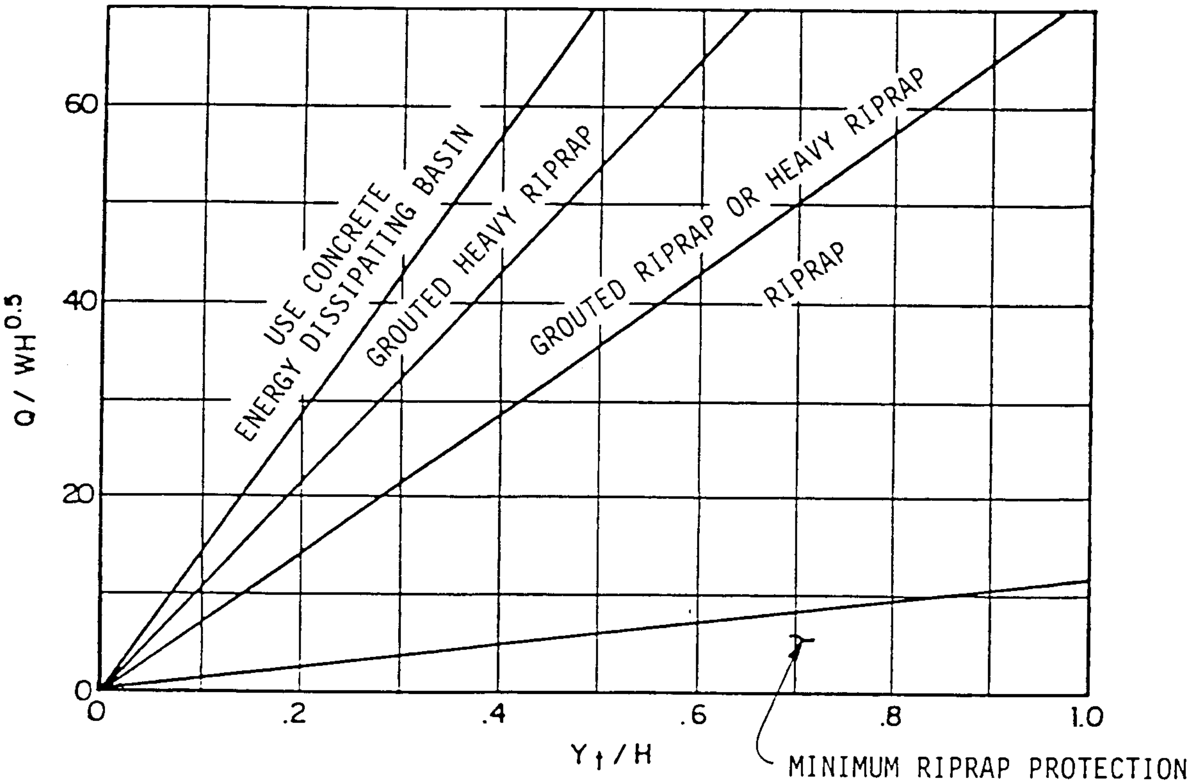


Notes:

1. Use H_a instead of H whenever culvert has supercritical flow in the barrel.
2. Minimum Riprap Protection: Use Riprap for a distance of 3H downstream for 18" to 36" equivalent round pipe size and 4D downstream for 42" and larger equivalent round pipe size.

Revision	Date

RIPRAP EROSION PROTECTION AT RECTANGULAR CONDUIT OUTLET

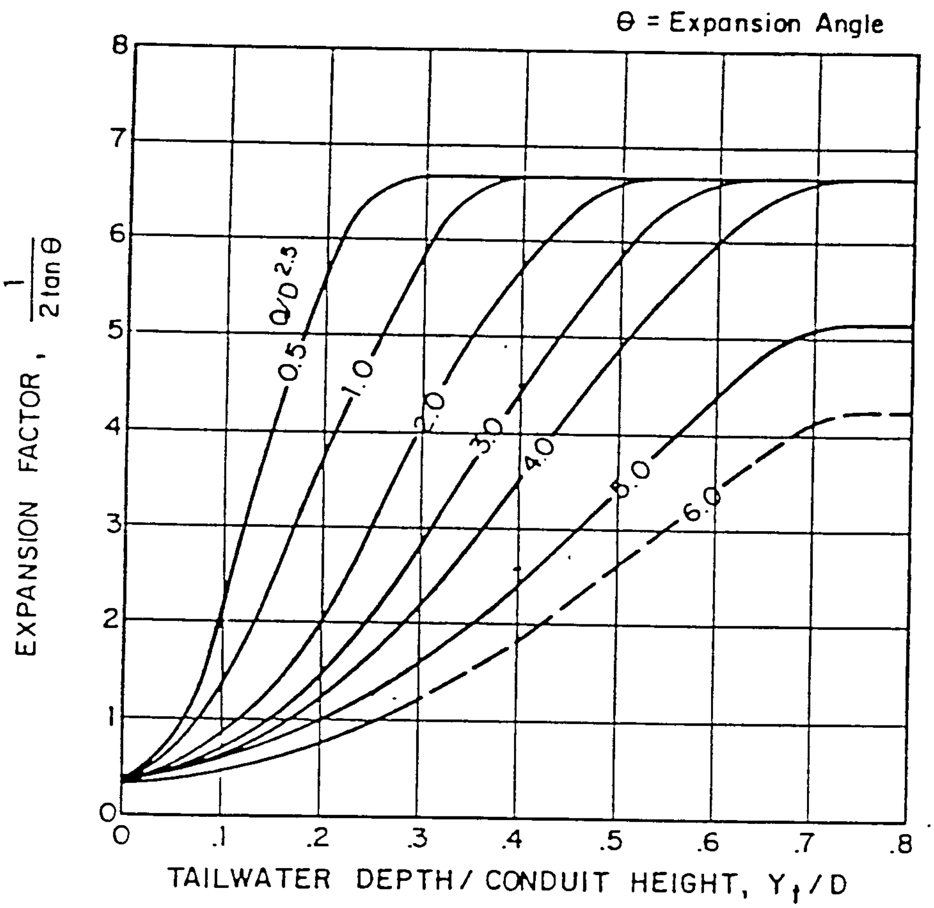


Notes:

1. Use H_a instead of H whenever culvert has supercritical flow in the barrel.
2. Minimum Riprap Protection: Use Riprap for a distance of 3H downstream for 18" to 36" equivalent round pipe size and 4D downstream for 42" and larger equivalent round pipe size.

Revision	Date

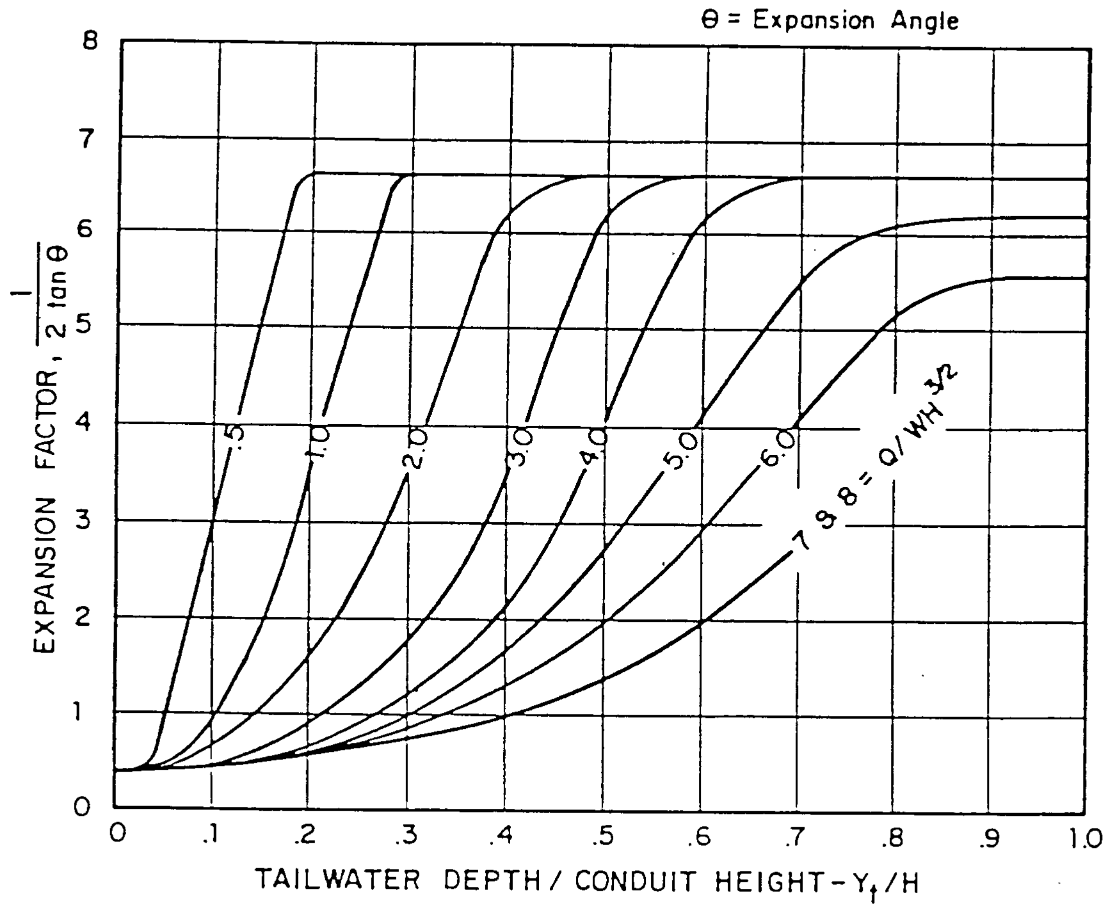
EXPANSION FACTOR FOR CIRCULAR CONDUITS



Revision	Date

REFERENCE: USDCM, DRCOG, March 1969 (with modifications)

EXPANSION FACTOR RECTANGULAR CONDUITS



Revision	Date

CLARK COUNTY REGIONAL FLOOD CONTROL DISTRICT
HYDROLOGIC CRITERIA AND DRAINAGE DESIGN MANUAL

SECTION 800
STORM SEWER SYSTEMS

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Section 800
Storm Sewer Systems

801 INTRODUCTION

A storm sewer system consists of a series of pipes, manholes, and inlets which generally convey storm runoff from streets (gutter flow) to open channels or detention facilities. Storm sewers are generally utilized when the flow carrying capacity of a street (gutter) is exceeded by the calculated storm runoff contributing to the said street (gutter). Inlets to the storm sewer are sized to reduce the amount of street (gutter) flow to a level where the downstream street (gutter) flow is not exceeded before the location ofthe next inlet. Manholes in the sewer system are provided to allow access to the storm sewer for inspection and maintenance of the storm sewer.

The size of the storm sewer system is generally governed by the minor storm flows. This is a result of the incremental flow capacity between the allowable street flow during major and minor storms being generally greater than the incremental difference in the peak runoff from major and minor storms. In addition, the storm sewer system will naturally carry some runoff in excess of the required minor storm capacity during major storms due to natural surcharging of the storm sewer system.

There are conditions, however, when the storm sewer system design will be governed by the major storm flows. A partial listing of some of the possible situations are as follows:

1. Locations where street flow is collected in a sump with no allowable overflow capacity.

2. Locations where the street cross-section is such that the allowable depth of flow in the street is limited to the curb height (i.e., elevated streets with negative slopes at the ROW line).

3. Locations where the desired major storm flow direction is not reflected by the street flow direction during a major storm (i.e., flow splits at intersections.

4. Locations where the subject storm sewer system is accepting flows from an upstream storm sewer system or branch which is designed for major storm capacity.

5. Regional storm sewers where designated in CCRFCD's adopted master plan.

802 DESIGN PARAMETERS

802.1 Allowable Storm Sewer Capacity

The storm sewer system shall be designed to convey a part or all of the minor or major storm (design storm) under surcharged or pressure flow conditions. The storm sewer shall be considered surcharged when the depth of flow (HGL) in the storm sewer is greater than eighty percent of full flow depth. The maximum level of surcharging for the capacity analysis shall be limited to maintaining the HGL to 1 foot below the final grade above the storm sewer at all locations. Special site conditions that warrant additional surcharging will require locking type manhole covers or grated covers and will be reviewed on a case by case basis by the local entity and/or the CCRFCD.

The EGL and HGL shall be calculated to include all hydraulic losses including friction, expansion, construction, bend, and junction losses. The methods for estimating these losses and for calculating the EGL and HGL are presented in the following sections.

802.2 Allowable Storm Sewer Velocity

The maximum allowable storm sewer velocity is dependent on many factors including the type of pipe, the acceptable wear level during the pipe design life, proposed flow conditions (open channel versus pressure flows), and the type and quality of construction of joints, manholes, and junctions. In consideration of the above factors, the maximum velocity in all storm sewers shall be limited to 25 fps.

The need to maintain a self-cleaning storm sewer system is recognized as a goal to minimize the costs for maintenance of storm sewer facilities. Sediment deposits, once established, are generally difficult to remove without pressure cleaning equipment. However, the infrequency of storm runoff also possesses a problem in obtaining flows large enough to maintain the self-cleaning quality of the design. Thus, a balance must be drawn between obtaining a self-cleaning system and constructing a reasonably sized and sloped storm sewer.

A generally accepted criteria is to maintain a minimum velocity of 3 fps at half or full conduit flow conditions. At half full, the storm sewer will flow under open channel flow conditions and thus the velocity in a given storm sewer is governed by the pipe slope. However, storm sewers generally cannot be constructed at slopes less than 0.25 percent and maintain a smooth even invert. At this slope a minimum velocity of 3 fps is maintained. Therefore, the minimum allowable storm sewer slope shall be 0.25 percent. Any slope less than 0.25 percent shall require the local entity and/or CCRFCD approval.

storm sewer slope shall be 0.25 percent. Any slope less than 0.25 percentshall require the local entity and/or CCRFCD approval.

802.3 Manning's Roughness Coefficient

All storm sewer system hydraulic calculations shall be performed using Manning's Formula (see Section 701). Manning's "n" value is determined based on the surface roughness of the storm sewer pipe material. In addition, for a given pipe material, Manning's ”n” value theoretically varies based on depth of flow in the pipe. For the purposes of this MANUAL Manning's ”n” value is assumed to be constant for all depths of pipe flow.

Various pipe manufacturers have determined Manning's ”n” value for use with their specific product. However, for storm sewer hydraulic design, Manning's ”n” value should also account for additional friction losses from pipe joints, potential debris and sediment in the storm runoff, and the pipe interior surface condition over the entire design life of the pipe. Therefore, presented on **Table 801** are the Manning's ”n” value to be used for all storm sewer design and analysis prepared in accordance with this MANUAL.

802.4 Storm Sewer Layout

The layout of a storm sewer system is governed by many factors including existing utility locations, street alignment, inlet placement, outfall location, and surface topography. These factors place constraints around which the storm sewer must be designed and still operate as an effective system. In addition, these constraints have inherent priorities as to which constraint takes precedence over the other constraints (i.e., relocating water lines versus designing around sanitary sewers).

The storm sewer system, however, must also take priority when other constraints would cause undesirable hydraulic conditions to occur in the storm sewer system, if the system were to be designed around the constraint. Therefore, limits are necessary in the storm sewer layout to prevent undesirable hydraulic conditions. The limits on vertical and horizontal alignments are presented in the following sections.

802.4.1 Vertical Alignment

802.4.1.1 Minimum and Maximum Cover

The required cover over a storm sewer pipe is dependent on many factors including the design pipe strength, pipe size, and cover material. For practical purposes, the storm sewer should be protected from potential surface disturbances and displacements. Therefore, the minimum allowable cover over the storm sewer pipe shall be 1 foot or greater at any point along the pipe. If there is less than 1 foot of cover the pipe shall be concrete encased. The maximum cover is contingent upon the design pipe strength.

802.4.1.2 Manhole and Junction Spacing

Manholes and junctions are used to provide a hydraulically efficient transition section at changes in the storm sewer system. Manholes and junctions are also used to provide access to the storm sewer for maintenance purposes. Therefore, to maintain hydraulic efficiency and adequate maintenance access, a manhole or junction shall be located at all changes in pipe size, direction (including bends where allowed), elevation and grade for all pipes with a diameter (or rise dimension) of less than 48 inches. Manholes or junctions will be required at inlet laterals when the lateral is not easily accessible for cleaning or maintenance from the inlet. For pipes with a diameter (or rise dimension) of 48 inches and greater, the designer shall consult with the local entity for location of manholes and junctions based on hydraulic and maintenance considerations. In addition, the maximum spacing between manholes for various pipe sizes shall be 400 feet.

802.4.2 Horizontal Alignment

The horizontal alignment of storm sewers shall generally be straight between manholes and/or junctions. However, if a curvilinear alignment is justified, then the storm sewer may be constructed with curvilinear alignment by the pulled-joint method, pipe bends, or using radius pipe. The radius of curvature for pulled-joint construction is dependent on the pipe length, diameter, and the permitted opening in the joint. The maximum allowable joint pull for pulled-joint construction shall be as presented in **Table 801**. For radius pipe, the maximum bevel angle shall not exceed 5 degrees. The maximum deflection angle for pipe bends shall not exceed 22 1/2 degrees per pipe section.

802.4.3 Utility Clearances

Storm sewers should generally be located to minimize potential contamination and disturbance of water supply and sanitary sewer mains from or to storm sewers. This should be accomplished through distancing the storm sewer from water and sanitary sewer mains where at all possible or adding additional leakage protection at joints. The requirements of CCRFCD for utility separations is presented in the following sections. Additional requirements may be imposed by the local utility companies. The storm sewer designer is responsible for adhering to the more stringent criteria.

802.4.3.1 Water Mains

Where a storm sewer or storm inlet run crosses a water main or comes within 10 horizontal feet (clear distance) of a water main, the storm sewer pipe shall be located a minimum of 18 inches clear distance vertically below the water main. If this clear distance cannot be obtained, then the storm sewer pipe section must be designed and constructed so as to protect the water main. Minimum protection shall consist of a 20 foot section of storm sewer centered over the water main being encased in concrete at least 4 inches thick. In addition, water tight joints shall be used within the 20 foot section. In no case shall the clearance between the water main and the storm sewer be less than 12 inches.

802.4.3.2 Sewer Mains

Where a storm sewer or storm inlet run crosses a sanitary sewer main or comes within 10 horizontal feet (clear distance) of each other, the storm sewer pipe shall be located a minimum of 12 inches clear distance vertically above or below the sanitary sewer main. If this clear distance cannot be obtained, then the sanitary sewer pipe section must be designed or improved to provide a structurally sound sewer main. This may be accomplished by either of the following methods:

1. Install one length of structural sanitary sewer pipe at least 18 feet long centered at the storm sewer. Joints between the sanitary sewer pipe and the structural pipe shall be encased in a concrete collar at least 4 inches thick and extending at least 6 inches either side of the joint.

2. Concrete encase the sanitary sewer with concrete at least 4 inches thick and extending a distance of 10 feet either side of the storm sewer.

Special additional backfill or structural provisions may also be required to preclude settling and/or failure of the higher pipe (storm sewer or sanitary sewer). Also all distances shall be measured from outside pipe edge to outside pipe edge.

802.5 Allowable Storm Inlet Types and Capacity Factors

Standard storm inlet types have been adopted as part of the STANDARD DRAWINGS for the Clark County Area. The allowable use of these storm inlet types is presented on **Table 802**. Also presented on **Table 802** are the allowable inlet capacity factors for each of the standard inlets. These capacity factors are applied to the theoretical capacity of the inlets to account for conditions which decrease the capacity of the standard inlets. These conditions include plugging from debris and sediment, pavement overlaying, variations in design assumptions, and the general deterioration of the inlet conditions over time.

802.6 Other Closed Conduit Criteria

802.6.1 Angle of Confluence

In general, the angle of confluence between main line and lateral shall not exceed 45 degrees and, as an additional requirement, shall not exceed 30 degrees under any of the following conditions:

- Where the flow (Q) in the proposed lateral exceeds 10 percent on the main line flow.
- Where the velocity of the flow in the proposed lateral is 20 fps or greater.
- Where the size of the proposed lateral is 60 inches or greater.
- Where hydraulic calculations indicate excessive head losses may occur in the main line due to the confluence.

Connector pipes may be joined to main line pipe at angles greater than 45 degrees up to a maximum of 90 degrees provided not of the above conditions exist. If, in any specific situation, one or more of the above conditions does apply, the angle of the confluence for connector pipes shall not exceed 30 degrees. Connections shall not be made to main line pipe which may create conditions of adverse flow in the connector pipes.

The above requirements may be waived only if calculations are submitted to the District showing that the use of a confluence angle larger than 30 degrees will not unduly increase head losses in the main line.

802.6.2 Connector Pipe and Depth Calculation

Given the available head (H), the required connector pipe size can be determined from culvert equations, such as those given in Brater and King, “Handbook of Hydraulics”, section Four, fifth edition. The minimum catch basin “V” depth shall be determined as follows:

$$V = C.F. + 0.5 + 1.2 * \frac{v^2}{2g} + \frac{d}{Cos(S)} \quad (801)$$

Where:	V	=	Depth of catch basin, measured in feet from the invert of the connector pipe to the top of the curb
	C.F.	=	Vertical dimension of the curb face at the catch basin opening, in feet
	v	=	Average velocity of flow in the connector pipe, in feet per second, assuming a full pipe section
	d	=	Diameter of connector pipe, in feet
	S	=	Slope of connector pipe

The term $1.2\ v^2/2g$ includes an entrance loss of 0.2 of the velocity head.

803 CONSTRUCTION STANDARDS

The following sections present the standards for construction of storm sewer systems. Detailed specifications for specific parts of the following standards are presented in the STANDARD SPECIFICATIONS for the Clark County Area including all future amendments. Where these detailed STANDARD SPECIFICATIONS are available, they shall be considered as an addition to the generalized standards presented in the following sections. The designer shall be responsible for referencing the most current version of the STANDARD SPECIFICATIONS.

803.1 Pipe

803.1.1 Size

The minimum allowable pipe size for storm sewers is dependent upon a practical size and length for maintenance and inspection of the storm sewer. Therefore, the minimum pipe size for storm inlet laterals to the storm sewer mains and for storm sewer mains shall be 18 inches in diameter for round pipe or shall have a minimum flow area of 2.2 square feet for other pipe shapes.

803.1.2 Material and Shape

The material and shape of the storm drain will be in accordance with the Standard Specifications.

Square or rectangular reinforced concrete box (RCB) pipe in accordance with ASTM C-789 or C-850 is acceptable for use in storm sewer construction.

Other pipe materials may be used for storm sewer construction upon approval by the local entity and/or the CCRFCD. Documentation must be submitted for review which shows that the subject pipe material has a

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803.1.3 Joint Sealants and Gaskets

Pipe joints for concrete pipe are generally sealed with either joint sealants or gaskets. Joint sealants are generally mastics which consist of bitumen and inert mineral fillers or joint mortar. The mastic is easily applied in the field but may not always provide a water tight joint. Joint gaskets are generally made of rubber and are either cemented to, recessed in, or rolled on the pipe joint. These gaskets generally provide a water tight seal and can withstand some internal pressure. Since all storm sewers within the CCRFCD will be generally designed for pressure flow conditions, rubber gasket joints shall be used for all installations where the pressure head exceeds 5 feet for the design flow. The pressure head is computed as the difference between the hydraulic grade line and the top of pipe.

803.2 Manholes

Manholes shall be constructed in accordance with the STANDARD DRAWINGS for the Clark County area. An exception is for Clark County where manholes shall be constructed in accordance with Clark County's "Improvement Standards", current revision. Precast manhole tees are not allowed where there is a change in storm sewer slope or alignment or where there are intersecting storm sewer mains or laterals. Pipes may be directly cast into the manhole base. The local entity and/or the CCRFCD may require gasketed joints, locking type manhole covers and/or grated manhole covers for pressure flow conditions.

803.3 Storm Sewer Inlets

Storm sewer inlets shall be constructed in accordance with the STANDARD DRAWINGS for the Clark County area. An exception is for Clark County where storm sewer inlets shall be constructed in accordance with Clark County's "Improvement Standards", current revision.

803.4 Storm Sewer Outlets

Storm sewer outlets shall be constructed with outlet erosion protection for discharges to channels with unlined bottoms in accordance with the following:

Outlet Velocity (fps)	Required Outlet Protection
Less than 5	Minimum Riprap Protection (Section 707.4)
Between 5 and 15	Riprap Protection (Section 707.4) or Energy Dissipater (Section 1102.2)
Greater than 15	Energy Dissipater (Section 1102.2)

For channels with lined bottoms, the outlet discharge velocity must not exceed the maximum allowable channel velocity without an energy dissipation structure.

804 STORM SEWER HYDRAULICS

Presented in this section are the general procedures for hydraulic design and evaluation of storm sewers. The user is assumed to possess a basic working knowledge of storm sewer hydraulics and is encouraged to review the text books and other technical literature available on the subject.

804.1 Hydraulic Analysis

Storm sewers in the Clark County area will typically be designed for pressure flow conditions. However, portions of the storm sewer may also act like open channels (i.e., very steep slopes, segments of storm sewers discharging to open channels). Therefore, the storm sewer capacity analysis must account for changes in flow conditions (open channel versus pressure flow) in the HGL and EGL calculations. The HGL for the design flow shall be included on all final storm sewer improvement construction plans.

804.1.1 Pressure Flow Analysis

When a storm sewer is flowing under a pressure flow condition, the energy and hydraulic grade lines may be calculated using the pressure-momentum theory. The capacity calculations generally proceed from the storm sewer outlet upstream accounting for all energy losses. These losses are added to the EGL and accumulate to the upstream end of the storm sewer. The HGL is then determined by subtracting the velocity head, H_v, from the EGL at each change in the EGL slope. To assist in accounting for and computing the energy losses and EGL, a pressure storm sewer computation form (**Standard Form 6**) is provided in this MANUAL.

Several computer software programs are available for computation of EGL's and HGL's in storm sewer systems. However, these programs are only allowed to be used for final design in the Clark County area if the user can demonstrate that the results ofthe program are consistent with the results obtained by using the energy loss equations and coefficients presented in this MANUAL.

804.1.2 Partial Full Flow Analysis

When a storm sewer is not flowing full, the sewer acts like an open channel and the hydraulic properties can be calculated using open channel techniques. For convenience, charts for various culvert shapes have been developed by the pipe manufacturers for calculating the hydraulic properties associated with partial full flow (**Figures 801**, **802**, and **803**). The data presented assumes thatthe friction coefficient, Manning's "n" value, does not vary throughout the depth.

For partial full flow analysis, the HGL and EGL are parallel when the flow reaches normal depth. The designer should check the available energy at all junctions and transitions to determine whether or not the flow in the storm sewer will be pressurized due to backwater effects even if the design flow is less than the full flow capacity of the storm sewer. In this case, a hydraulic jump will occur and the pipe should be structurally designed to accommodate the jump.

804.2 Energy Loss Calculations

Presented in this section are the energy loss equations and coefficientsfor use in the hydraulic analysis of storm sewer systems. All storm sewer analysis inthe Clark County area shall account for energy losses using the equations and coefficients in this section.

804.2.1 Pipe Friction Losses

Pipe friction losses shall be calculated using an equation for full flow conditions derived from Manning's equation as follows:

$$S_f = M\,H_v / R^{1.33} \qquad (802)$$

Where S_f = Friction Slope (ft/ft)

H_v = Velocity Head (ft)

R = Hydraulic Radius (ft)

The flow coefficient, M, is related to the Manning's "n" value for the pipe as follows:

$$M = 2gn^2 / 2.21 \qquad (803)$$

Where n = Manning's Roughness Coefficient

The total head loss due to friction in a length of pipe is then equal to the friction slope times the pipe length.

804.2.2 Pipe Form Losses

Generally, between the inlet and outlet, storm sewer flow encounters a variety of configurations in the flow passageway such as changes in pipe size, branches, bends, junctions, expansions, and contractions. These shape variations impose losses in addition to those resulting from pipe friction. Form losses are the result of fully developed turbulence and are expressed as:

$$H_L = K (V^2 / 2g) \qquad (804)$$

Where H_L = Head Loss (ft)

K = Loss Coefficient

$V^2 / 2g$ = Velocity Head (ft)

g = Gravitational Acceleration (32.2 ft/sec^2)

The following is a discussion of a few of the common types of form losses encountered in sewer system design. The reader is referred to standard hydraulic references and text books for additional form loss discussion. In the following equations, subscripts 1 and 2 denote the upstream and downstream sections respectively.

804.2.2.1 Expansion Losses

Expansion in a storm sewer conduit will result in a shearing action between the incoming high velocity jet and the surrounding sewer boundary. As a result, much of the kinetic energy is dissipated by eddy currents and turbulence. The head loss is expressed as:

$$H_L = K_e (V_2^2 / 2g) ((A_2 / A_1) -1)^2 \qquad (805)$$

in which A is the cross-sectional flow area, V is the average flow velocity, and K_e is the expansion loss coefficient. The value of K_e varies from about 1.0 for a sudden expansion to about 0.2 for a well designed expansion transition. **Table 803 (A)** presents the expansion loss coefficients for various flow conditions.

804.2.2.2 Contraction Losses

The form loss due to contraction is expressed as:

$$H_L = K_c (V_2^2 / 2g) \qquad (806)$$

where K_c is the contraction loss coefficient. K_c varies from about 0.4 for large pipe size differences (>10:1) to about 0.1 for minor pipe size differences. **Table 803 (B)** presents the contraction loss coefficients for various flow conditions.

804.2.2.3 Bend Losses

The head losses for bends, in excess of that caused by an equivalent length of straight pipe, is expressed as:

$$H_L = K_b (V_2^2 / 2g) \qquad (807)$$

in which K_b is the bend loss coefficient. The bend loss coefficient has been found to be a function of, (a) the ratio of the radius of curvature of the bend to the width of the conduit, (b) deflection angle of the conduit, (c) geometry of the cross section of flow, and (d) the Reynolds Number and relative roughness. Tables showing the recommended bend loss coefficients is presented in **Table 803 (C)**.

804.2.2.4 Junction and Manhole Losses

A junction occurs where one or more branch sewers enter a main sewer, usually at manholes. The hydraulic design of a junction is in effect the design of two or more transitions, one for each flow path. Allowances are made for head losses due to the impacts at the junctions. The head loss at a junction is expressed as:

$$H_L = (V_2^2 / 2g) - K_j (V_1^2 / 2g) \qquad (808)$$

where V_2 is the outfall flow velocity, V_1 is the inlet velocity, and K_j is the junction coefficient. Because of the difficulty in evaluating hydraulic losses at junctions due to the many complex conditions of pipe size, geometry of the junction and flow combinations, a simplified table of loss coefficients has been prepared. **Table 803 (D)** presents the recommended energy loss coefficients for typical manhole or junction conditions that will be encountered in the urban storm sewer system. This equation is valid for junctions and manholes where the incoming flow is greater than 10 percent of the main line flow. If the incoming lateral flow is less than 10 percent of the main line flow, this headloss equation is invalid.

For straight flow through manholes (single pipe with no inlet laterals), the head loss through the manhole is similar to a pipe bend. For this condition, the head loss at the manhole is expressed as:

$$H_L = K_m (V_2^{\ 2} / 2g) \qquad (809)$$

in which K_m is the manhole loss coefficient. **Figure 815** presents value of K_m for various deflection angles.

804.2.2.5 ***Inlet Losses***

When runoff enters a storm sewer system from locations other than street inlets (i.e., open channels) an energy loss occurs at the entrance in the form of a contraction loss. The head loss at storm sewer entrances is expressed as:

$$H_L = K_i (V_2^{\ 2} / 2) \qquad (810)$$

in which K_i is the inlet (entrance) loss coefficient. The coefficient K_i is the same as the K_e coefficient used for the entrance loss calculation for culverts. A list of various K_i (K_e) coefficients is presented in **Table 1001** in Section 1000.

804.2.2.6 ***Outlet Losses***

When the storm sewer system discharges into open channels, additional losses occur at the outlet in the form of expansion losses. For most storm sewer outlets, the flow velocity in the storm sewer is greater than the allowable or actual flow velocity in the downstream channel. Therefore, energy dissipating facilities are used to remove excess energy from the storm sewer flow. In addition, the alignment of the storm sewer at the outlet may not be the same as the downstream channel. Therefore, energy is lost in changing the flow direction between the storm sewer to the downstream channel. The head loss at storm sewer outlets is expressed as:

$$H_L = K_o (V_1^{\ 2} / 2g) \qquad (811)$$

where K_o is the outlet loss coefficient. For all storm sewer outlets, an outlet loss coefficient K_o of 1.0 shall be used.

805 STORM INLET HYDRAULICS

Presented in this section is discussion and criteria for sizing and locating storm sewer inlets. In the Clark County area, the allowed standard inlet types are presented in **Table 802**. For capacity calculations, the inlets are further classified as being on a “continuous grade” or in a “sump.” The term “continuous grade” refers to an inlet so located that the grade of the street has a continuous slope past the inlet and therefore ponding does not occur at the inlet. The sump condition exists whenever water is restricted to the inlet area because the inlet is located at a low point. A sump condition can occur at locations such as a change in grade of the street from positive to negative or at an intersection due to the crown slope of a cross street.

The procedure to define the capacities of standard inlets consists of defining the amount and depth of flow in the gutter and determining the theoretical flow interception by the inlet. To account for effects which decrease the capacity of the various types of inlets, such as debris plugging, pavement overlaying and variations in design assumptions, the theoretical capacity calculated in **Figures 804** through **811** for the inlet capacity should be reduced by the factors presented in **Table 802**.

Allowable inlet capacities for the standard inlets have been developed and are presented in **Figures 804** through **811** for “continuous grade" and **Figures 812** through **814** for "sump" conditions. The allowable inlet capacity is dependent on the depth of flow as determined from the street capacity calculations (for continuous grade inlets) or on the depth of ponding necessary to accept the desired flow rate (sump conditions). These depths must be kept at or below the allowable flow or ponding depths as presented in Section 304.4.

805.1 Inlets on Continuous Grade

For the “continuous grade” conditions (**Figures 804** through **811**), the capacity of an inlet is dependent upon many factors including gutter slope, depth of flow in the gutter, height and length of curb opening, street cross slope, and the amount of depression at the inlet. In addition, all of the gutter flow will not be intercepted and some flow will continue past the inlet area ("inlet carryover"). The amount of carryover must be included in the drainage facility evaluation as well as in the design of the inlet.

Flow on a street is divided into frontal flow carried by the gutter and side flow carried by the street. Street hydraulic capacity is determined by the street cross slope (S_x). The interception of the frontal flow by a grated inlet is determined by the gutter flow velocity, splash velocity, and the length of the grate. Splash velocity is the flow velocity under the grate interference. Regression analyses performed on laboratory data and resulted in an empirical relationship for determining splash-over velocity based on grate length and type. Similar relationships were developed for the interception percentage of side flow. The total flow interception relationships for grated 3 foot long inlets under various water spread widths and street cross slopes are shown in **Figures 804** through **807**. The total interception is the sum of the frontal flow interception and the side flow interception, with a clogging factor.

For curb openings on a grade, the required curb opening length (L_t) for complete interception was also determined empirically. **Figures 808** through

811 depict the flow interception for 5 foot long curb openings in a 0.5 foot high curb under various water spread widths and street cross slopes.

805.2 Inlets In A Sump Condition

The capacity of an inlet in a sump condition (**Figures 812** through **814**) is dependent on the depth of ponding above the inlet. Typically, the problem consists of determining the quantity or length of inlets required to reduce the depth of ponding to an acceptable level. The designer should be aware that several inlets or additional inlet length will generally be required when an inlet must be designed to accommodate major storm flow. Also, additional continuous grade inlets may be necessary upstream of the sump location to reduce the depth of ponding at the sump inlets to an acceptable level during major storm events.

A grated inlet in a sump condition operates like a weir under shallow ponding depths, but as an orifice when submerged by deeper ponding. **Figure 812** depicts the flow interception for 3 foot long grated inlets with a 0.5 foot high curb for water depths up to 3 feet.

Curb openings in a sump also operate like a weir under shallow ponding and as an orifice under deeper ponding. If the head on the opening is less than the curb height plus the gutter depression, the inlet operates as a weir, otherwise it operates as an orifice. **Figure 813** depicts the flow interception for 5 foot long curb openings in a 0.5 foot high curb for water depths up to 3 feet. **Figure 814** depicts inlet capacity for a beehive inlet for depths up to 2 feet.

805.3 Inlet Spacing

The optimum spacing of storm inlets is dependent upon several factors, including traffic requirements, contributing land use, street slope, and distance to the nearest outfall system. The suggested sizing and spacing of the inlets is based upon an interception rate of 70 percent to 80 percent. This spacing has been found to be more efficient than a spacing using 100 percent interception rate. Using the suggested spacing, only the most downstream inlet in a development would be designed to intercept 100 percent of the flow. Also, considerable improvement in overall inlet system efficiency can be achieved if the inlets are located in the sumps created by street intersections, if possible, without overloading of the sump inlets.

806 STORM SEWER SYSTEM DESIGN

Presented in this section is the design procedures for a storm sewer system from preliminary design consideration to final design. A typical drainage system within a development consists of flow in the storm sewer and allowable flow in the gutter, which combined would carry both the minor and major storm flows. The design flow for the storm sewer is generally governed by the amount of runoff in excess of the minor storm street capacity. In some cases, however, the amount of runoff from the major storm in excess of the major storm street capacity may be larger than the excess from the minor storm. In this case, the storm sewer and inlets would need to be designed to accommodate the excess major storm flow. To assist in this analysis, the allowable minor and major storm street capacity should be determined prior to sizing of the storm sewer system. (See Section 900).

806.1 Initial Storm Sewer Sizing

Preliminary street grades and cross sections must be available to the storm sewer designer so he can calculate the allowable carrying capacity for these streets. Beginning at the upper end of the basin in question, the designer should calculate the quantity of flow in the street until the point is reached at which the allowable carrying capacity of the street matches the design runoff. Initiation of the storm sewer system would start at this point if there is no alternate method of removing runoff from the street surface. Removal of all the street flow by the storm sewer system is not required except at sump areas. However, the sum of the flow in the sewer plus the flow in the street must be less than or equal to the allowable capacity of the street and storm sewer.

For preliminary sizing purposes, the diameter, type of pipe, and pipe slope may be determined assuming a full flow pipe capacity based on slope-area calculations. If large energy losses are anticipated (i.e., large junctions, bends), then the preliminary pipe size may need to be upsized to assure that the final pressure calculations result in an acceptable HGL and EGL. In some instances, a profile may be required to check utility conflicts or to assure compatibility with the Regional Drainage System.

At this point, the preliminary system should be reviewed to check that the system is hydraulically efficient as well as to locate segments which have potentially large energy losses. These segments should be examined carefully and options explored to minimize the energy loss. The designer should also check potential inlet locations to assure that the required inlet capacity is not larger then the allowable inlet capacities.

806.2 Final Storm Sewer Sizing

Final design consists of the preparation of plan, profiles and specifications for the storm sewer system in sufficient detail for construction. The first step consists of the review and verification of the basic data, hydrologic analysis, and storm sewer inlet sizing performed for the preliminary design. Plan and profile drawings are prepared containing the basic data. Drainage subbasins are revised as necessary, and the design flood peaks recalculated. The storm sewer and inlets are then sized taking into account actual street and storm sewer grades, locations of existing and proposed utilities, and the design of the Regional Drainage System. The calculations also include the determination of the hydraulic and energy grade lines. The manholes, junction structures, or other appurtenant structures must be evaluated for energy losses. If special transitions are required to reduce losses, the structural design of the facilities must include these requirements when detailing the structures.

807 EXAMPLE APPLICATIONS

807.1 Introduction

The following subsections include two example analyses. The first example presents the hydraulic analysis of a storm sewer system and demonstrates the use of the energy loss coefficients and the Hydraulic Calculations **Standard Form 6**.

The following procedure is based on full-flow pipe conditions. If the pipe is flowing substantially full (i.e., greater than 80 percent), the following procedures can be used with minimal loss of accuracy. However, the designer is responsible for checking the assumptions (i.e., check for full flow) to assure that the calculations are correct.

The second example presents the hydraulic analysis to compute the design capacity of a grate inlet. This analysis is based on flow in a crowned street on a grade.

807.2 Example: Storm Sewer Hydraulic Analysis

<u>Problem:</u> Compute the EGL and HGL for the storm sewer system presented in plan on **Figure 816** and profile on **Figure 817**. The starting water surface (WS) elevation in the downstream channel is 100.0. The numbers in brackets (i.e., [4]) refer to the columns on **Table 804**.

<u>Solution:</u>

Step 1: Determine the storm sewer outlet flow conditions. In this example, the normal depth is greater than critical depth, $d_n > d_c$. Therefore, calculations begin at the outfall, working upstream. Compute the following parameter:

$$M = 2gn^2 / 2.21$$

$$= (2)\ (32.2)\ (0.013)^2 / 2.21$$

$$= 0.00492$$

This equation is derived from Manning's equation by solving for velocity and converting to velocity head. This value remains constant for this design since the "n" value does not change.

Step 2: Determine starting HGL and EGL elevations:

Starting HGL = WS = 100.0

Starting EGL = $HGL + H_{LO} = HGL + K_o\ (H_v)$

Where H_{LO} = Energy Loss at Storm Sewer Outlet

K_o = 1.0

$H_v = V^2 / 2g = (Q/A)^2 / 2g$

Assuming full flow,

$$H_v = (145 / 23.76)^2 / 2g = (6.1)^2\ /2\ (32.2)$$

$$= 0.58 \text{ ft}$$

Therefore,

$$EGL = 100.0 + 0.58 = 100.58$$

Enter EGL and HGL at the top of [23] and [24], respectively.

Step 3: Fill in [1] through [11] of Row 1 for first storm sewer segment.

Step 4: Compute friction slope, S_f, and enter on [12]:

S_f = MH / $R^{1.33}$ (**Equation 802**)

= (0.00492) (0.58) $(5.5/4)^{1.33}$

= 0.0019 ft/ft

Where R = Hydraulic Radius = D/4

Step 5: Compute average friction slope, Ave. S_f, and enter in [13]:

This is the average value of S_f between the two stations being analyzed. For the first station, Ave. S_f [13] = S_f [12].

Step 6: Compute energy losses between the stations being analyzed:

Friction loss, H_f = (Ave. S_f) (L) = 0.0019 (110) = 0.21 ft

Enter H_f in [14]

Step 7: Compute EGL and HGL at upstream stations:

U/S EGL = Starting EGL + H_f

= 100.58 + 0.21

= 100.79

U/S HGL = U/S EGL - H_v

= 100.79 - 0.58

= 100.21

Enter EGL in [21] and HGL in [22].

Step 8: Check that full flow still exists (i.e., WS > 0.8D):

WS = HGL [22] - U/S Invert Elevation [5]

= 100.21 - 94.71

= 5.5 ft > 0.8 (5.5) = 4.4 ft

Since pressure flow still exists, enter “YES” in [25]. Place any additional comments in [26].

Step 9: Fill in [1] through [9] of Row 2. Compute V and H_v and enter in [10] and [11], respectively.

Step 10: Check that the downstream flow condition in this segment (EGL) does not control over the upstream flow condition in the downstream storm sewer segment. (Whichever is higher controls)

D/S EGL (this segment)	=	D/S Invert Elevation + D + H_v
	=	94.71 + 5.5 + 0.58
	=	100.79
U/S EGL (downstream segment)	=	100.79

Therefore U/S EGL (downstream segment) controls. If the D/S EGL of this segment controlled (as in Row 5), then enter the controlling EGL (and HGL) in [23] and [24] respectively and repeat Step 9 in the next row down. Continue in this row with Step 11.

Step 11: Repeat Step 4 and 5 and enter in [12] and [13], respectively.

Step 12: Compute friction and form losses in the transition section.

$$H_f = (\text{Ave. } S_f)(L) = 0.0019\ (42.4) = 0.08 \text{ ft}$$

Enter H_f in [15]

For [16] through [19], enter K factors from appropriate tables and figures as well as H Values from **Equations 804** through **811**. Separate K factor and H Value in table with a slash (/). For this row, K_b = 0.18 for a 45 degree Bend (**Table 803 (C)**).

The total energy loss is therefore:

$$H_{Total} = H_f + H_b$$

$$= H_f + K_b\ (H_v)$$

$= 0.08 + 0.18\ (0.58)$

$= 0.08 + 0.10$

$= 0.18$

Enter H_{Total} in [20].

Step 13: Compute EGL and HGL at upstream station:

U/S EGL (Transition) [23] = U/S EGL (Pipe) [21] + $_{Htotal}$

= 100.79 + 0.18

= 100.97

U/S HGL (Transition) [24] = U/S EGL (Transition) [23] - H_v

= 100.97 - 0.58

= 100.39

Enter EGL in [23] and HGL in [24]

Step 14: Check that full flow still exists (i.e., WS > 0.8D):

WS = HGL [24] - U/S Invert Elevation [5]

= 100.39 - 94.79

= 5.6 ft > 0.8 (5.5) = 4.4 ft

Since pressure flow still exists, enter "YES" in [25]. Place any additional comments in [26].

Step 15: Repeat Step 3 through Step 14 as needed to obtain the EGL and HGL for the entire storm sewer system. The EGL and HGL for this example are plotted in **Figure 817** from the results of **Table 804**.

Step 16: Special approval would be required from the local entity and/or the CCRFCD to allow the 66-inch RCP to be constructed at a slope of 0.19 percent which is less than the allowable 0.25 percent (Section 802.2).

807.3 Example: Grate Inlet Hydraulic Analysis

Problem: Compute design discharge for grate inlet presented in plan on **Figure 818**.

Waterspread, T	=	10 ft
Manning roughness, n	=	0.016
Street transverse slope, S_x	=	0.02 ft/ft
Street longitudinal slope, S_o	=	0.02 ft/ft
Gutter width, W	=	2.0 ft
Gutter length, Lg	=	3.0 ft

Solution:

Step 1: Calculate street hydraulic capacity at 10 ft waterspread:

For a given discharge, the revised Manning's equation states:

$$Q_s = Q_x + Q_w$$

$$= \frac{0.56}{n} S_x^{1.67} T^{2.67} S_0^{0.5}$$

$$= \frac{0.56}{0.016} (0.02)^{1.67} (10.0)^{2.67} (0.02)^{0.5}$$

$$= 3.37 \text{ cfs}$$

Step 2: Calculate side flow carried by the street width:

$$Q_x = \frac{0.56}{n} S_x^{1.67} T_x^{2.67} S_0^{0.5}$$

$$= \frac{0.56}{0.016} (0.02)^{1.67} (8.0)^{2.67} (0.02)^{0.5}$$

$$= 1.86 \text{ cfs}$$

Step 3: Find ratio between frontal flow (Q_w) and street capacity (Q_s), E_0:

$$E_0 = \frac{Q_w}{Q_s}$$

$$= \frac{Q_w - Q_x}{Q_s}$$

$$= \frac{3.37 - 1.86}{3.37}$$

$$= 0.45 \text{ cfs}$$

Step 4: Determine total ideal flow interception capacity of a grate inlet:

$$Q_i = R_f Q_w + R_s Q_x$$

$$= R_f Q_w + R_s (Q_s - Q_w)$$

$$= [R_f + R_s(1 + E_0)]Q_s$$

Where $R_f = 1$

When V_0, splash-over velocity, determined by empirical formula, grater than flow velocity, V_s, on the street.

$$V_s = \frac{Q_s}{A}$$

$$= \frac{3.37}{\frac{10 \times 10 \times 0.02}{2}}$$

$$R_s = \frac{1}{1 + \frac{0.15 V_s^{1.8}}{S_x L^{2.3}}}$$

$$= \frac{1}{1 + \frac{0.15 \times 3.37^{1.8}}{0.02 \times 6^{2.3}}}$$

$$= 0.48$$

Step 5: Determine the ideal capacity of a grate inlet Q:

$$Q_i = R_f Q_w + R_s Q_x$$

$$= [R_f + R_s (1 + E_0)] Q_s$$

$$= \{(1 \times 0.45) + [0.48 \times (1 - 0.45)]\} \times 3.37$$

$$= 2.41 \text{ cfs}$$

Under the idealcondition without considering a clogging factor, the carry-over flow is

$$Q_c = 3.37 - 2.41$$

$$= 0.96 \text{ cfs}$$

STORM SEWER DESIGN AND ANALYSES PARAMETERS

A. MANNING'S ROUGHNESS COEFFICIENT (N-VALUE):

Storm Sewer Type	N-Value
Concrete	.013
Corrugated Metal (Corrugated Interior)	,024
Corrugated Metal (Smooth Lined Interior)	.013
HDPE Pipe	.013

B. MAXIMUM ALLOWED DEFLECTION FOR PULLED JOINT CONSTRUCTION:

Pipe Diameter (Span) (Inches)	Allowed Deflection (Pull) (Inches)
18" - 33"	1 / 2
36" - 54"	5 / 8
60" - 78"	3 / 4
84" - 102"	7 / 8
108" - 144"	1

Revision	Date

REFERENCE:

ALLOWABLE STORM INLET TYPES AND CAPACITY FACTORS

INLET TYPE	STANDARD DRAWING NUMBER	PERMITTED USE	PERMITTED LOCATION CONDITION	CAPACITY FACTOR
DROP INLET TYPE "A" (CURB OPENING)	411	ALL STREETS WITH CURB AND GUTTER	C.G. SUMP	0.80 0.80
DROP INLET TYPE "B" (COMBINATION)	412	ALL STREETS WITH CURB AND GUTTER	C.G. SUMP	0.70 (Grate) and 0.80 (Curb Opening) 0.65
DROP INLET TYPE "C" (COMBINATION)	413	ALL STREETS WITH CURB AND GUTTER	C.G. SUMP	0.70 (Grate) and 0.80 (Curb Opening) 0.65 (Combination) and 0.80 (Extended Curb Opening)
BEEHIVE DROP INLET (GRATED)	416	ROADSIDE DITCHES	SUMP	0.50

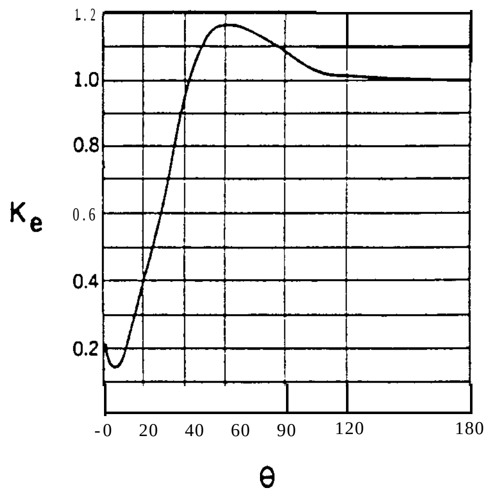
NOTES:

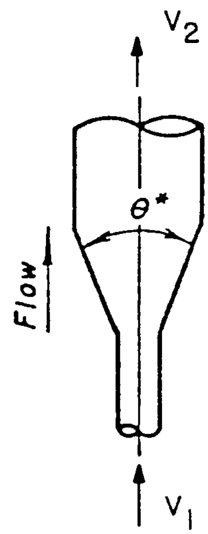
1. For the purposes of capacity calculations, Drop Inlet Type "D" can be considered to be the same as Drop Inlet Type "C" listed above.
2. C.G. = Continuous Grade.
3. Standard Drawing Numbers refer to the "Uniform Standard Drawings for Public Works Construction, Off-Site Improvements, Clark County Area, Nevada, 1988, including future amendments.
4. Capacity factor is applied to the theoretical inlet capacity to obtain the allowable inlet capacity to account for factors which reduce actual inlet capacity.

Revision	Date

STORM SEWER ENERGY LOSS COEFFICIENTS

(A) EXPANTIONS

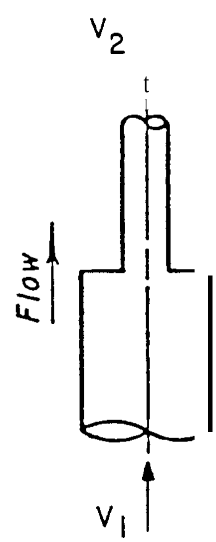




*The angle θ is the angle in degrees between the sides of the tapering section.

(B) CONTRACTIONS

A_2/A_1	K_C
0.1	0.36
0.2	0.34
0.3	0.31
0.4	0.27
0.5	0.22
0.6	0.16
0.7	0.11
0.8	0.05
0.9	0.01
1.0	0.00



Revision	Date

REFERENCE: (A) Fluid Mechanics, Daugherty and Franzini, 1977; (B) Fluid Mechanics, Streeter and Wylie, 1979

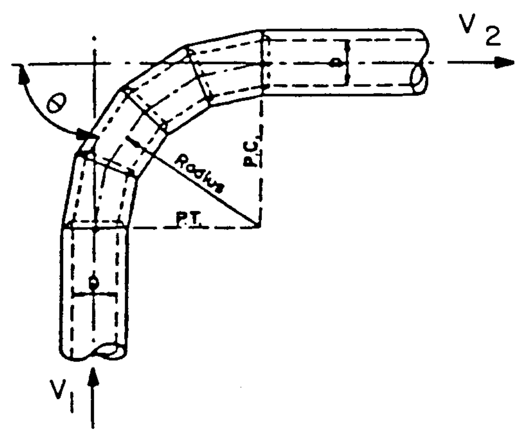
STORM SEWER ENERGY LOSS COEFFICIENTS

(C) BENDS

I. Large Radius Bends
(Pipe Diameter > Bend Radius)

$$K_b = 0.25\,(\theta/90)^{0.5}$$

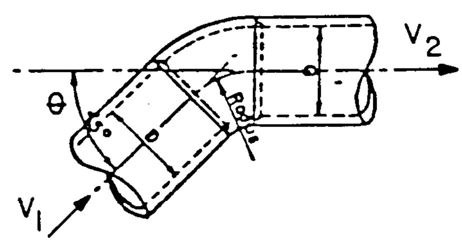
θ	K_b
90"	0.25
60"	0.20
45"	0.18
30"	0.14



Note: Head loss applied at P.C.

II. Sharp Radius Bends
(Pipe Diameter = Bend Radius)

θ	K_b
90”	0.50
60°	0.43
45°	0.35
30"	0.25

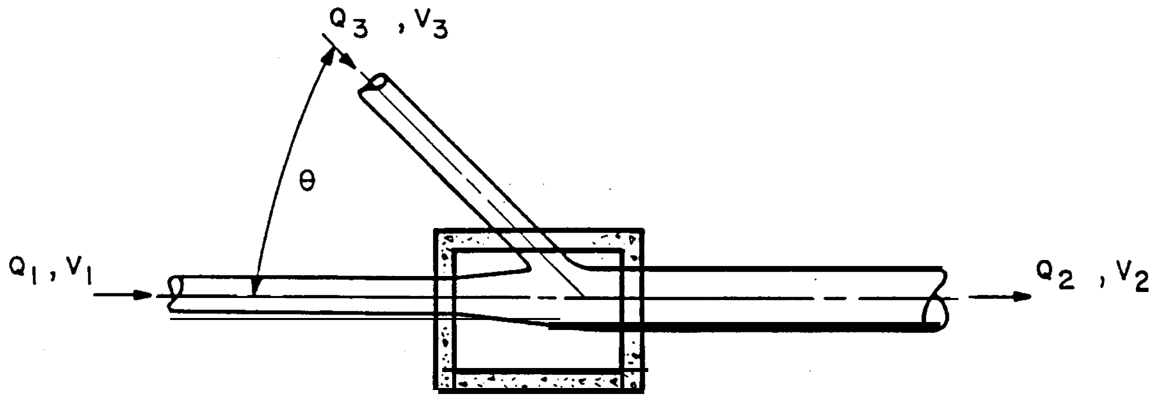


Note: Head loss applied at entrance to bend.

Revision	Date

STORM SEWER ENERGY LOSS COEFFICIENTS

(D) JUNCTIONS



θ	K_j
90"	0.25
60"	0.35
45"	0.50
30"	0.65
15"	0.85

NOTE: Head loss 'applied at exit of junction

$$H_L = (V_2{}^2 / 2g) - K_j (V_1{}^2 / 2g)$$

Revision	Date

REFERENCE: "Urban Stormwater Management"
APWA Special Report No. 49,1981

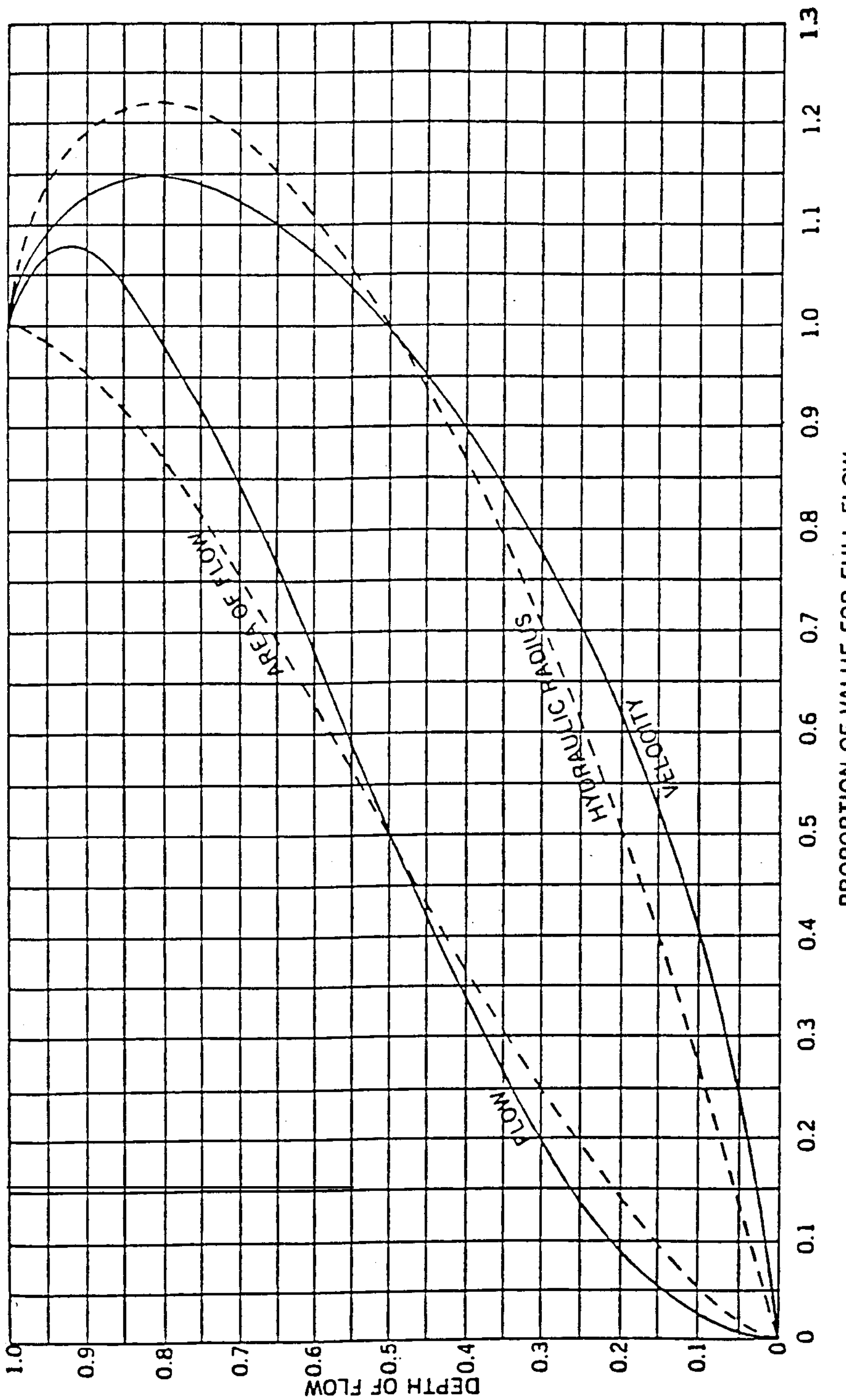
HYDRAULIC CALCULATIONS FOR EXAMPLE IN SECTION 807.2

	[1]	[2]	[3]	[4]	[5]	[6]	[7]	[8]	[9]	[10]	[11]	[12]	[13]	[14]	[15]	[16]	[17]	[18]	[19]	[20]	[21]	[22]	[23]	[24]	[25]	[26]
	CONDUIT DATA						FLOW DATA							ENERGY LOSS DATA							ENERGY AND HYDRAULIC DATA					COMMENTS
	STATION			INVERTS										PIPE	TRANSITIONS						PIPE		TRANSITION			
	FROM	TO	SIZE/ TYPE	D/S ELEV.	U/S ELEV.	LENGTH	A (FT^2)	I	Q (CFS)	V (FPS)	$H_v = V^2/2g$ (FT)	S_f (FT/FT)	AVE. S_f (FT/FT)	H_f (FT)	H_t (FT)	H_j (FT)	H_m (FT)	H_ (FT)	H_ (FT)	TOTAL (FT)	U/S EGL ELEV.	U/S HGL ELEV.	U/S EGL ELEV.	U/S HGL ELEV.	PRESS. FLOW?	OTHER
																				Enter Starting EGL and HGL Here →			100.58	100.00		
1	0+00	1+10	66"RCP	94.50	94.71	110	23.76	.00492	145	6.1	0.58	.0019	.0019	0.21							100.79	100.21			YES	
2	1+10	1+ 52.4	66"RCP	94.71	94.79	42.4	23.76	.00492	145	6.1	0.58	.0019	.0019		0.08			.18/ .10		.18			100.97	100.39	YES	HGL 0.1' above Pipe Crown
3	1+ 52.4	2+48	66"RCP	94.79	94.97	95.6	23.76	.00492	145	6.1	0.58	.0019	.0019	0.18							101.15	100.57			YES	HGL 0.1' above Pipe Crown
4	2+48	2+ 55.5	54"RCP	94.97	96.00	7.5	15.90	.00492	145	9.1	1.29	.0054	.0037		0.03				.15/ .02	.05			101.20	99.91	NO	1.0 drop + .03' slope $\theta = 2 \times 3.8° = 7.6°$
5	2+ 55.5	3+ 55.5	54"RCP	96.00	96.54	100.0	15.90	.00492	145	9.1	1.29												101.79	100.50	YES	U/S EGC Controls
6	2+ 55.5	3+ 55.5	54"RCP	96.00	96.54	100.0	15.90	.00492	145	9.1	1.29	.0054	.0054	0.54			.02/ .03			.03	102.33	101.04	102.36	101.07	YES	
7	3+ 55.5	4+ 55.5	54"RCP	96.54	97.08	100.0	15.90	.00492	145	9.1	1.29	.0054	.0054	0.54							102.90	101.61			YES	
8	4+ 55.5	4+ 65.5	48"RCP	97.08	97.58	10.0	12.57	.00492	100	8.0	0.99	.0049	.0052		.05	.65/ .65				.70			103.6	102.61	YES	0.5' Drop
9	4+ 65.5	5+ 65.5	48"RCP	97.58	98.07	100.0	12.57	.00492	100	8.0	0.99	.0049	.0049	0.49							104.09	103.10			YES	
10	5+ 65.5	5+ 75.5	36"RCP	98.07	99.07	10.0	7.07	.00492	60	8.5	1.12	.0081	.0065		.07	.32/ .80				1.67			105.76	104.64	YES	1.0' Drop; Assume $H_j = 2 \times H_j$ (2 Laterals)
11	5+ 75.5	6+ 75.5	36"RCP	99.07	99.88	100.0	7.07	.00492	60	8.5	1.12	.0081	.0081	0.81			.02/ .02			.02	106.57	105.45	106.59	105.47	YES	
12	6+ 75.5	6+ 85.5	36"RCP	99.88	99.96	10.0	7.07	.00492	60	8.5	1.12	.0081	.0081	0.08							106.67	105.55			YES	
13	6+ 85.5			99.96															.25/ .28	.28			106.95	106.95		Groove Edge Projecting Storm Sewer Entrance

$S_f = \frac{I\,H_v}{R^{1.33}}$ $\quad I = \frac{2g(n^2)}{2.21}$

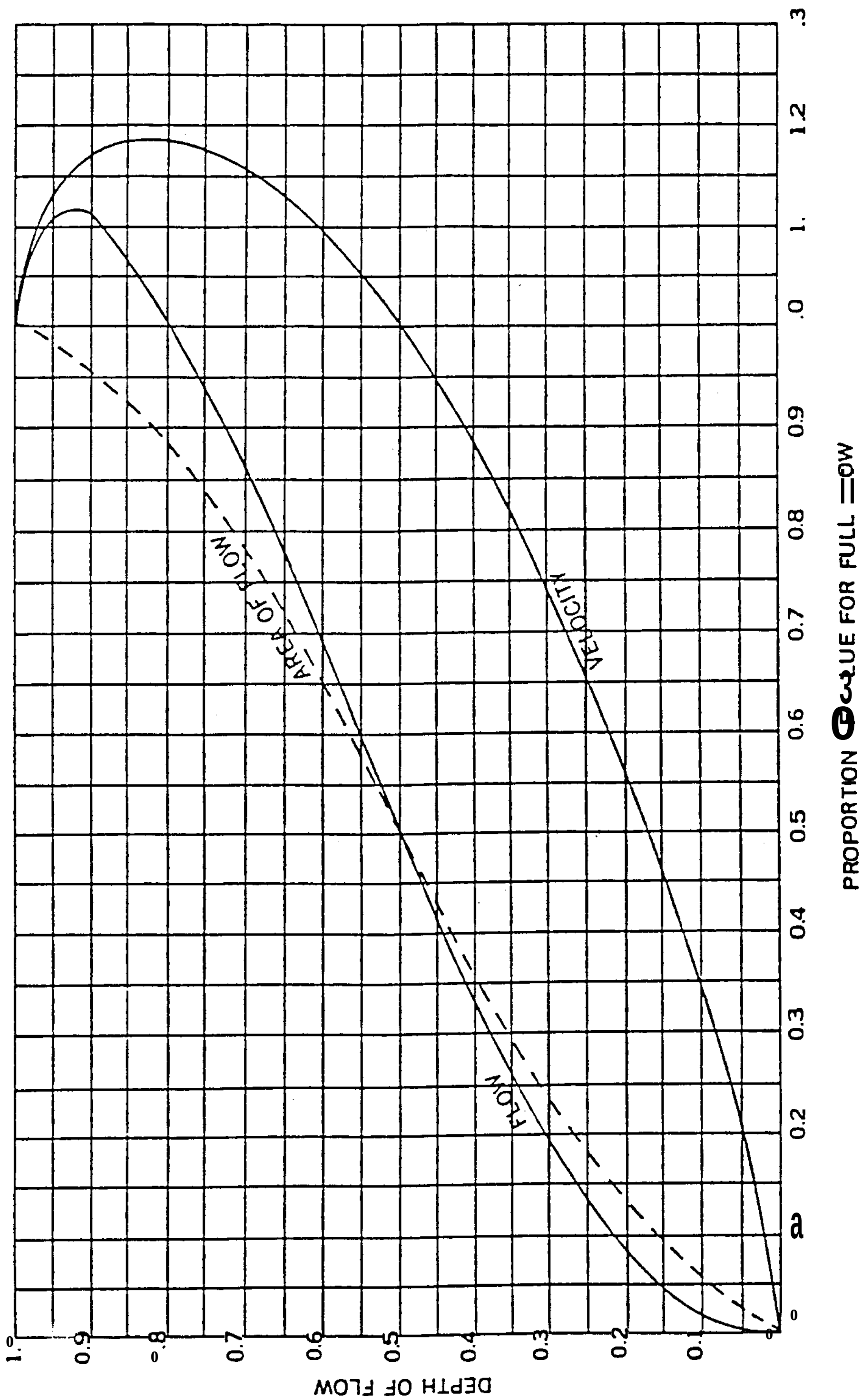
Revision	Date

HYDRAULIC PROPERTIES
CIRCULAR PIPE



Revision	Date

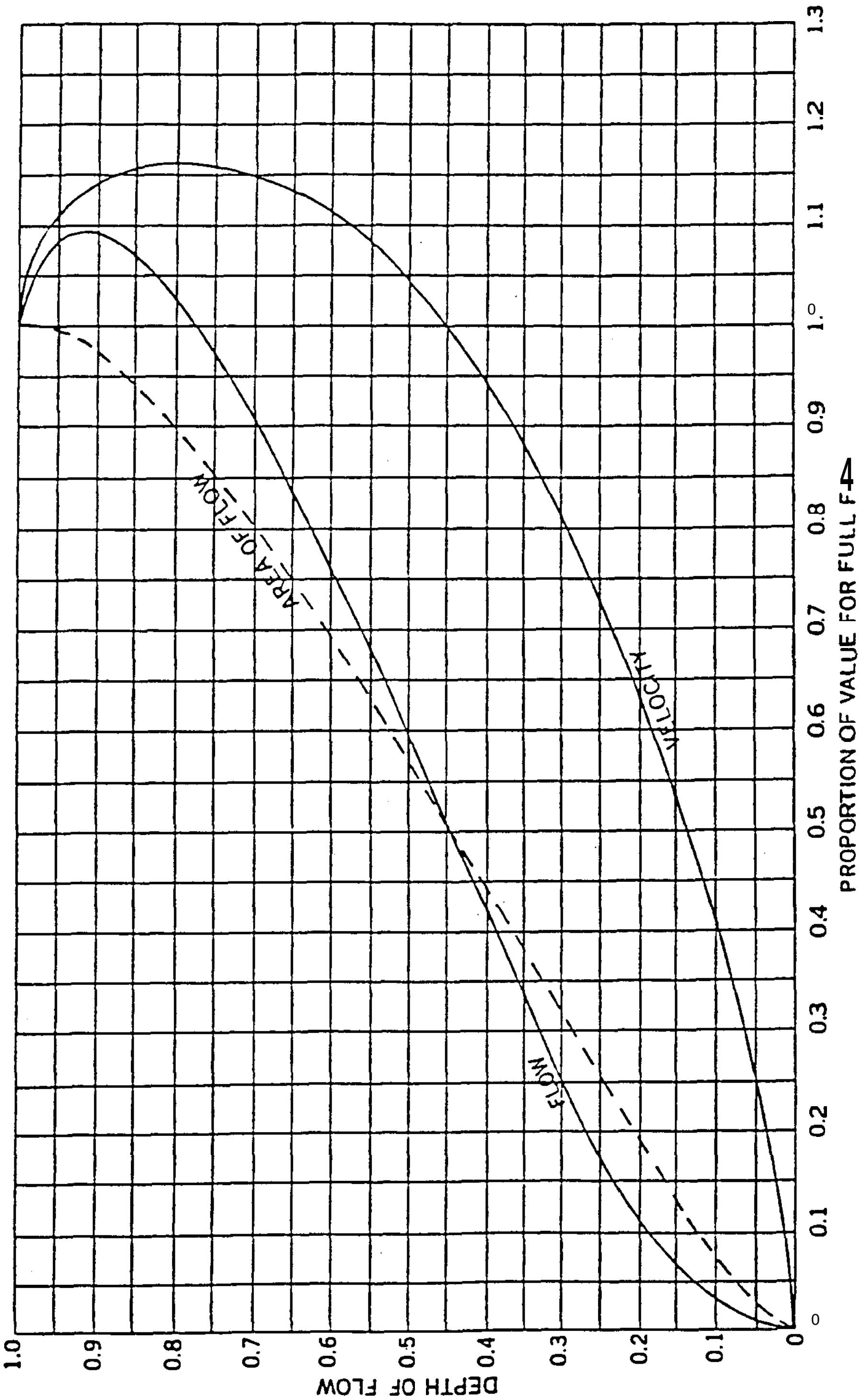
HYDRAULIC PROPERTIES
HORIZONTAL ELLIPTICAL PIPE



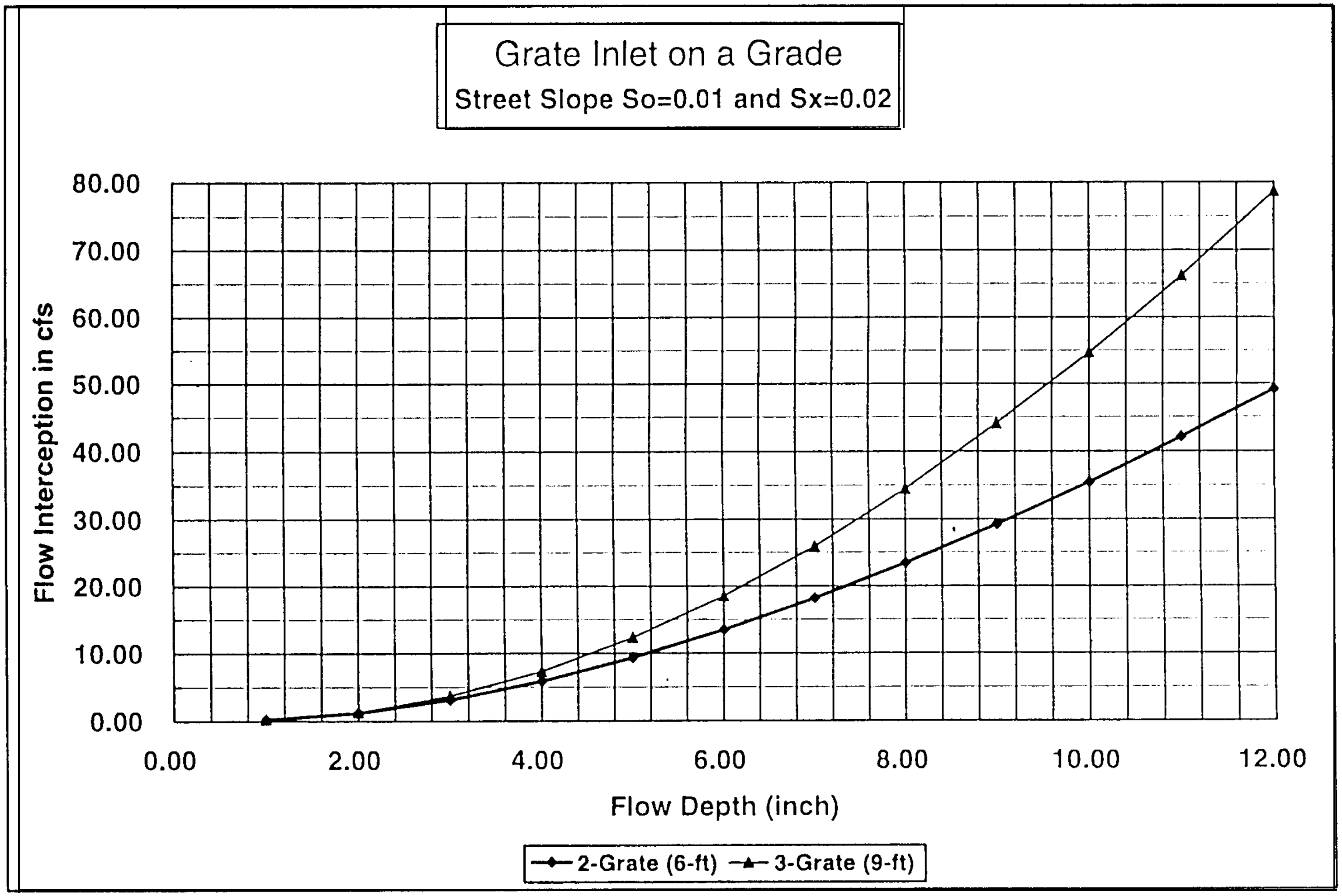
Revision	Date

REFERENCE:
"CONCRETE PIPE DESIGN MANUAL" ACPA, 1970

HYDRAULIC PROPERTIES
ARCH PIPE



Revision	Date



$Q_i = [R_f\, E_o + R_s\,(1 - E_o)]\,Q_s$

Where: $R_f = 1 - 0.09\,(V_s - V_o)$ if $V_s > V_o$, otherwise $R_f = 1$

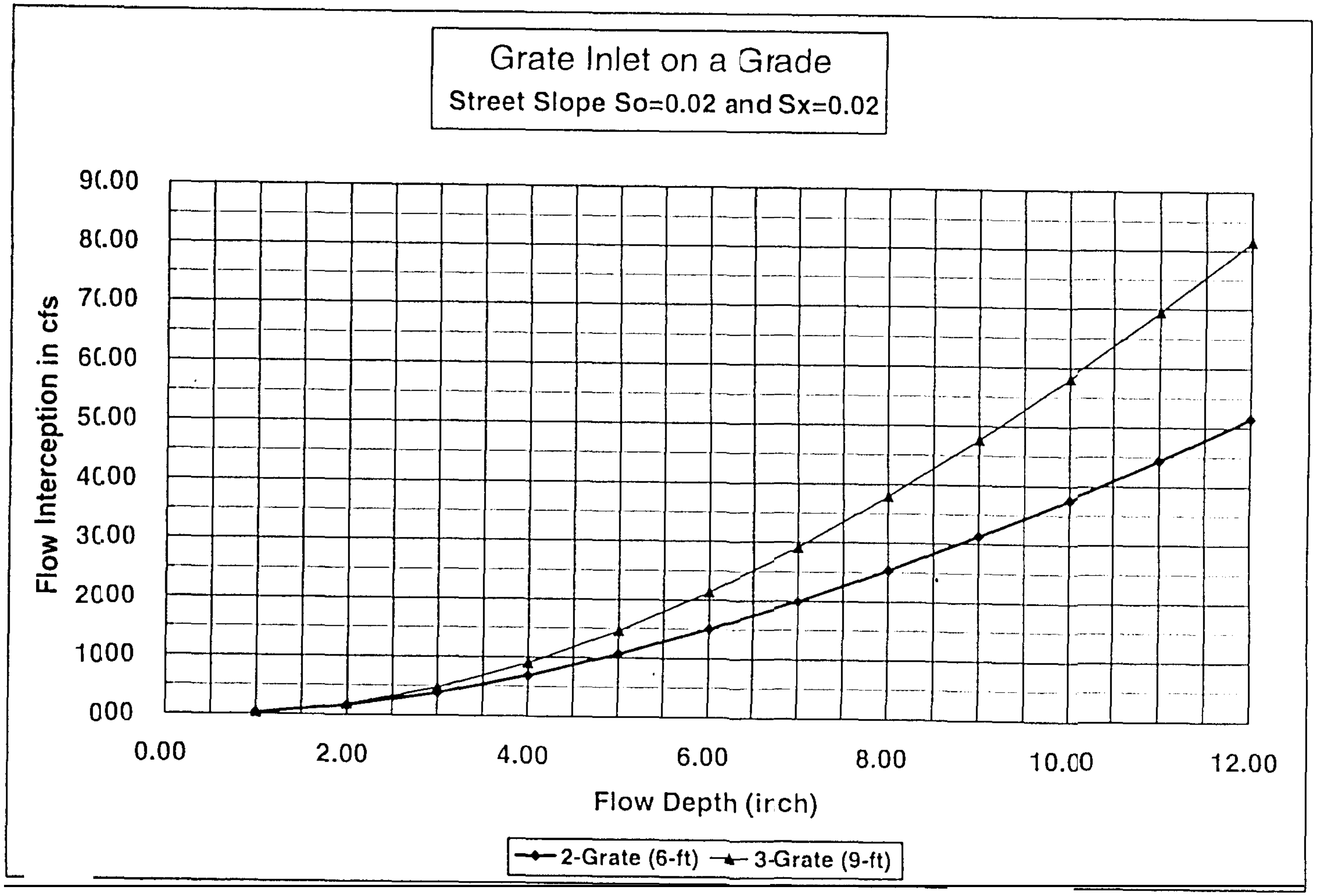
$$R_s = \frac{1}{\left[1 + \frac{0.15\,V_s^{1.8}}{S_x\,L^{2.3}}\right]} \qquad E_o = \frac{Q_w}{Q_s} \qquad Q_s = \frac{0.56}{n} S_x^{1.67} T^{2.67} S_o^{0.5}$$

Q_i	=	Ideal inlet capacity	L	=	Grate inlet length in ft
Q_w	=	Frontal flow	n	=	Manning's roughness
Q_s	=	Street capacity	S_x	=	Street transverse slope in ft/ft
V_s	=	Flow velocity	S_o	=	Street longitudinal slope in ft/ft
V_o	=	Splash-over velocity	T	=	Water spread on street

Revision	Date

REFERENCE:

FIGURE 804



$Q_i = [R_f E_o + R_s (1 - E_o)] Q_s$

Where: $R_f = 1 - 0.09 (V_s - V_o)$ if $V_s > V_o$, otherwise $R_f = 1$

$$R_s = \frac{1}{\left[1 + \frac{0.15 V_s^{1.8}}{S_x L^{2.3}}\right]} \quad E_o = \frac{Q_w}{Q_s} \quad Q_s = \frac{0.56}{n} S_x^{1.67} T^{2.67} S_o^{0.5}$$

Q_i = Ideal inlet capacity

Q_w = Frontal flow

Q_s = Street capacity

V_s = Flow velocity

V_o = Splash-over velocity

L = Grate inlet length in ft

n = Manning's roughness

S_x Street transverse slope in ft/ft

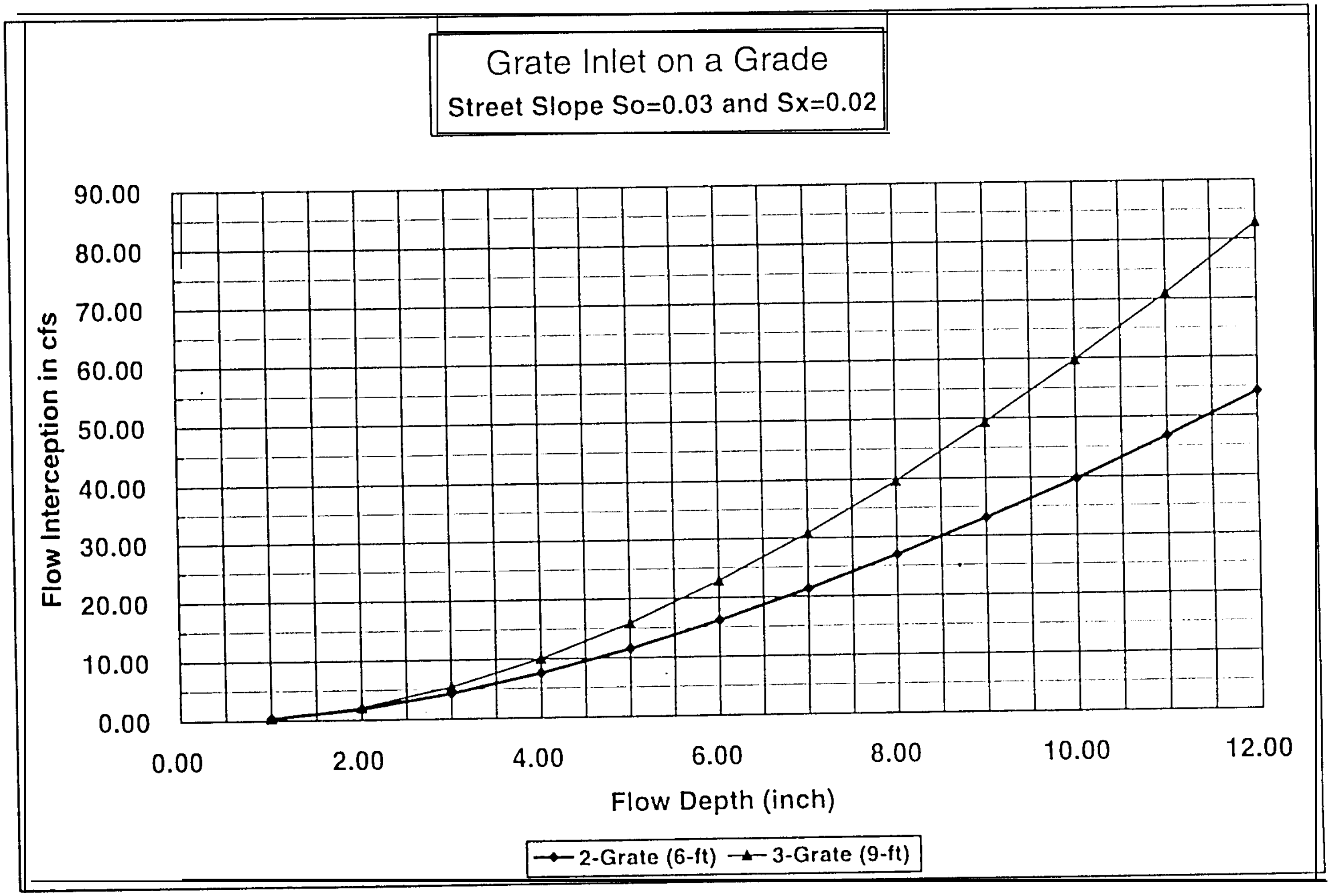
S_o = Street longitudinal slope in ft/ft

T = Water spread on street

Revision	Date

REFERENCE:

FIGURE 805



$Q_i = [R_f E_o + R_s (1 - E_o)] Q_s$

Where: $R_f = 1 - 0.09 (V_s - V_o)$ if $V_s > V_o$, otherwise $R_f = 1$

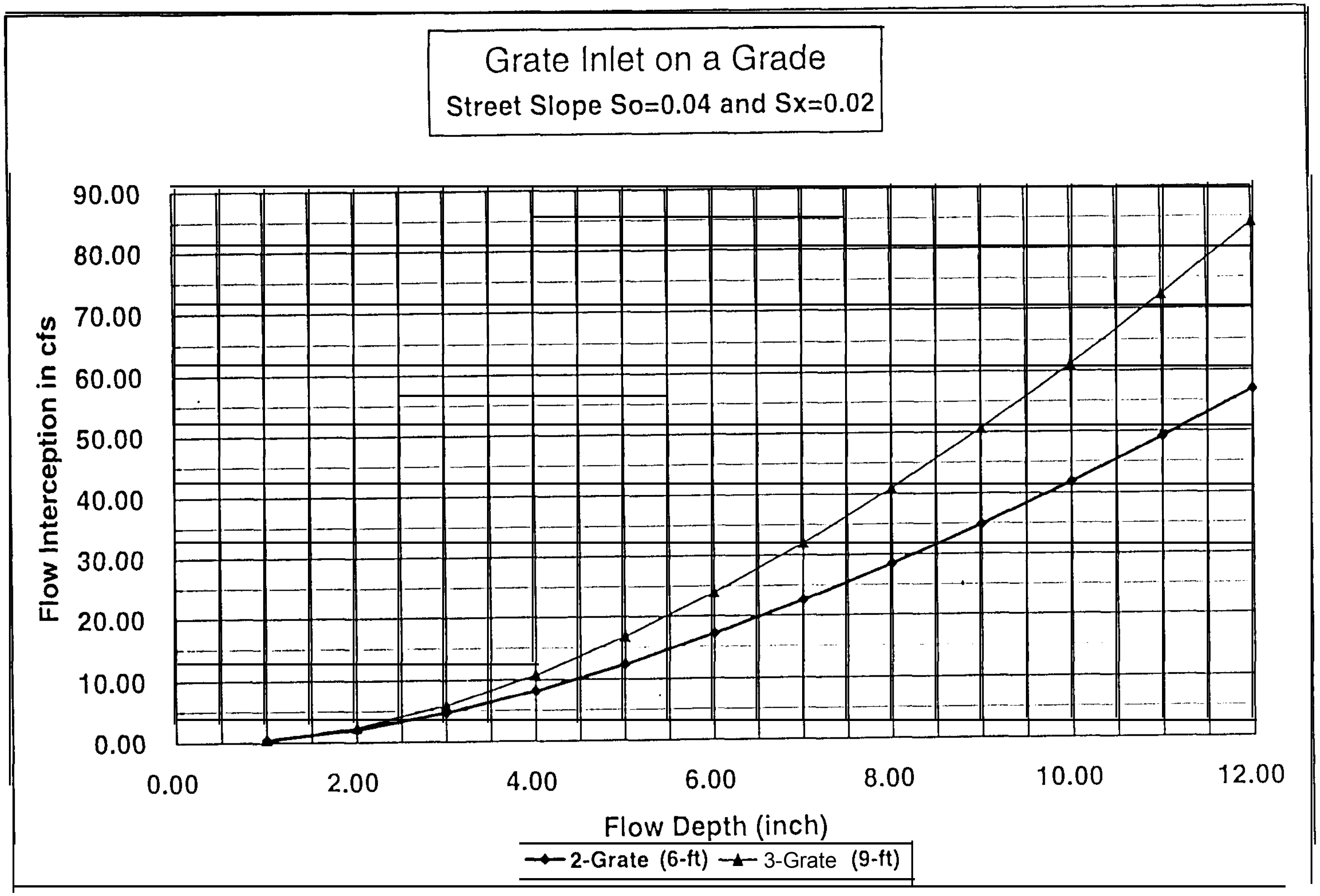
$$R_s = \frac{1}{\left[1 + \frac{0.15 V_s^{1.8}}{S_x L^{2.3}}\right]} \qquad E_o = \frac{Q_w}{Q_s} \qquad Q_s = \frac{0.56}{n} S_x^{1.67} T^{2.67} S_o^{0.5}$$

Q_i = Ideal inlet capacity
Q_w = Frontal flow
Q_s = Street capacity
V_s = Flow velocity
V_o = Splash-over velocity
L = Grate inlet length in ft
n = Manning's roughness
S_x = Street transverse slope in ft/ft
S_o = Street longitudinal slope in ft/ft
T = Water spread on street

Revision	Date

REFERENCE:

FIGURE 806



$Q_i = [R_f E_0 + R_s(1 - E_0)]Q_s$

Where: $R_f = 1 - 0.09(V_s - V_0)$ if $V_s > V_0$, otherwise $R_f = 1$

$$R_s = \frac{1}{\left[1 + \frac{0.15 V_s^{1.8}}{S_x L^{2.3}}\right]} \quad E_0 = \frac{Q_W}{Q_s} \quad Q_s = \frac{0.56}{n} S_x^{1.67} T^{2.67} S_0^{0.5}$$

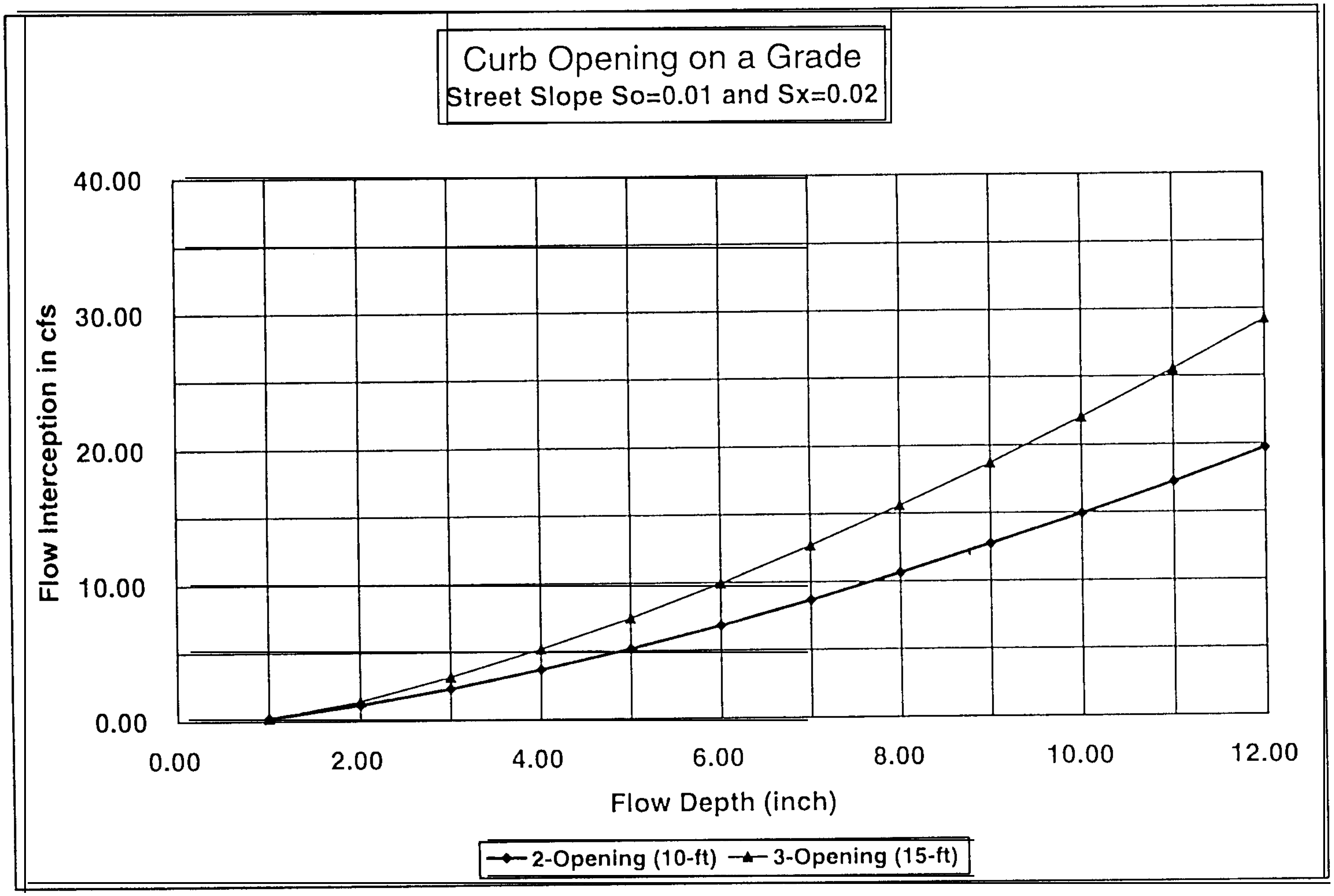
Q_i = Ideal inlet capacity
Q_W = Frontal flow
Q_s = Street capacity
V_s = Flow velocity
V_0 = Splash-over velocity

L = Grate inlet length in ft
n = Manning's roughness
S_x = Street transverse slope in ft/ft
so = Street longitudinal slope i in ft/ft
T = Water spread on street

Revision	Date

REFERENCE:

FIGURE 807



$$Q_i = \left[1 - \left(1 - \frac{L_c}{L_t}\right)^{1.8}\right] Q_s \qquad L_t = 0.60 Q_s^{0.42} S_0^{0.30} \left(\frac{1}{n S_e}\right)^{0.6}$$

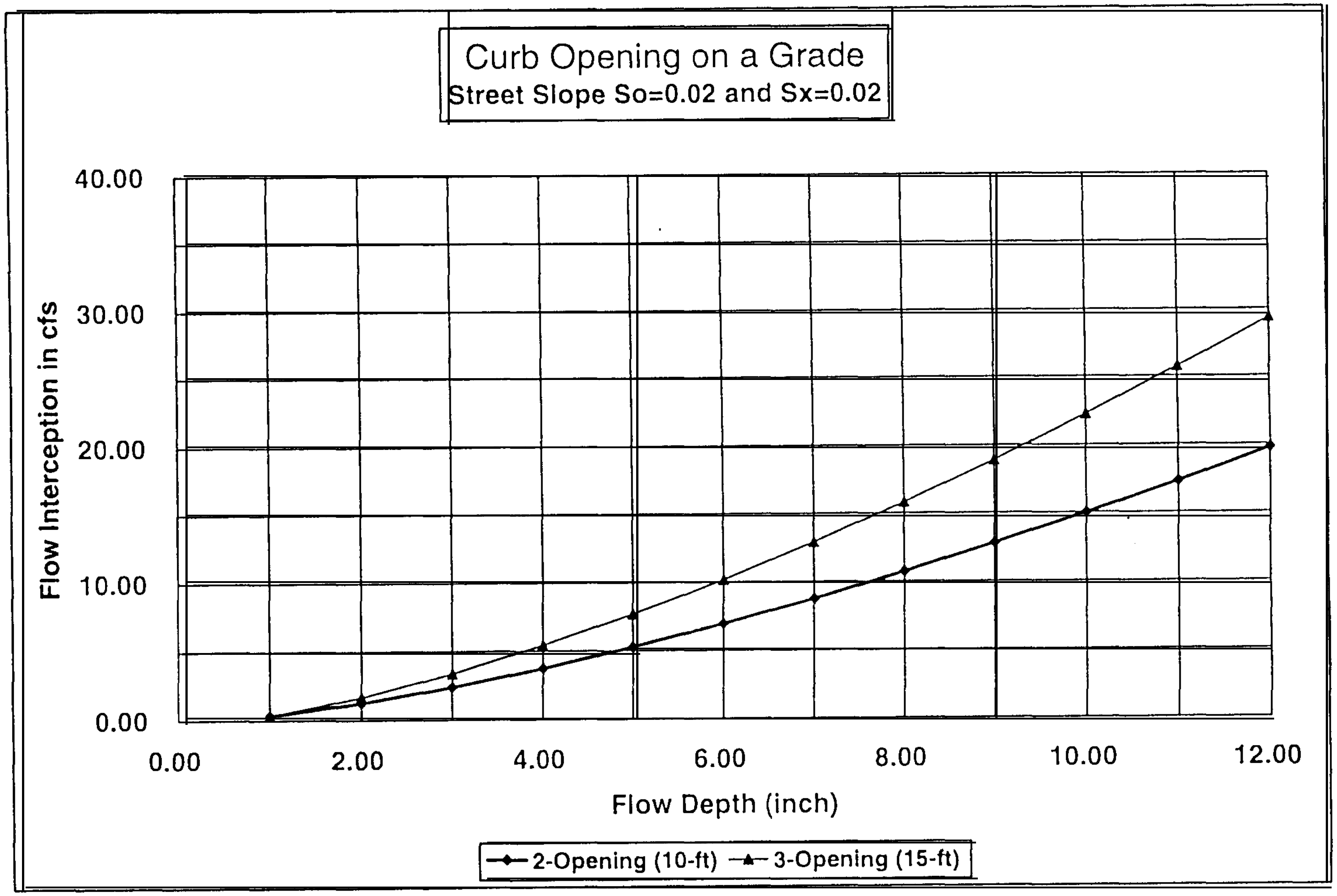
Where: $Q_s = \frac{0.56}{n} S_x^{1.67} T^{2.67} S_0^{0.5}$ $\quad S_e = S_x + (S_w \times E_o)$ $\quad E_o = \frac{Q_w}{Q_s}$

Q_i = Ideal inlet capacity
Q_s = Street capacity
Q_w = Frontal flow
L_t = Required curb opening length in ft
L_c = Designed curb opening length in ft
S_x = Street transverse slope in ft/ft
S_w = Gutter cross slope in ft/ft
S_o = Street longitudinal slope in ft/ft
T = Water spread on street

Revision	Date

REFERENCE:

FIGURE 808



$$Q_i = \left[1 - \left(1 - \frac{L_c}{L_t}\right)^{1.8}\right] Q_s \qquad L_t = 0.60 Q_s^{0.42} S_0^{0.30} \left(\frac{1}{n S_e}\right)^{0.6}$$

Where: $Q_s = \frac{0.56}{n} S_x^{1.67} T^{2.67} S_0^{0.5}$ $\quad S_e = S_x + (S_w \times E_o)$ $\quad E_o = \frac{Q_w}{Q_s}$

Q_i = Ideal inlet capacity

Q_s = Street capacity

Q_w = Frontal flow

L_t = Required curb opening length in ft

L_c = Designed curb opening length in ft

S_x = Street transverse slope in ft/ft

S_w = Gutter cross slope in ft/ft

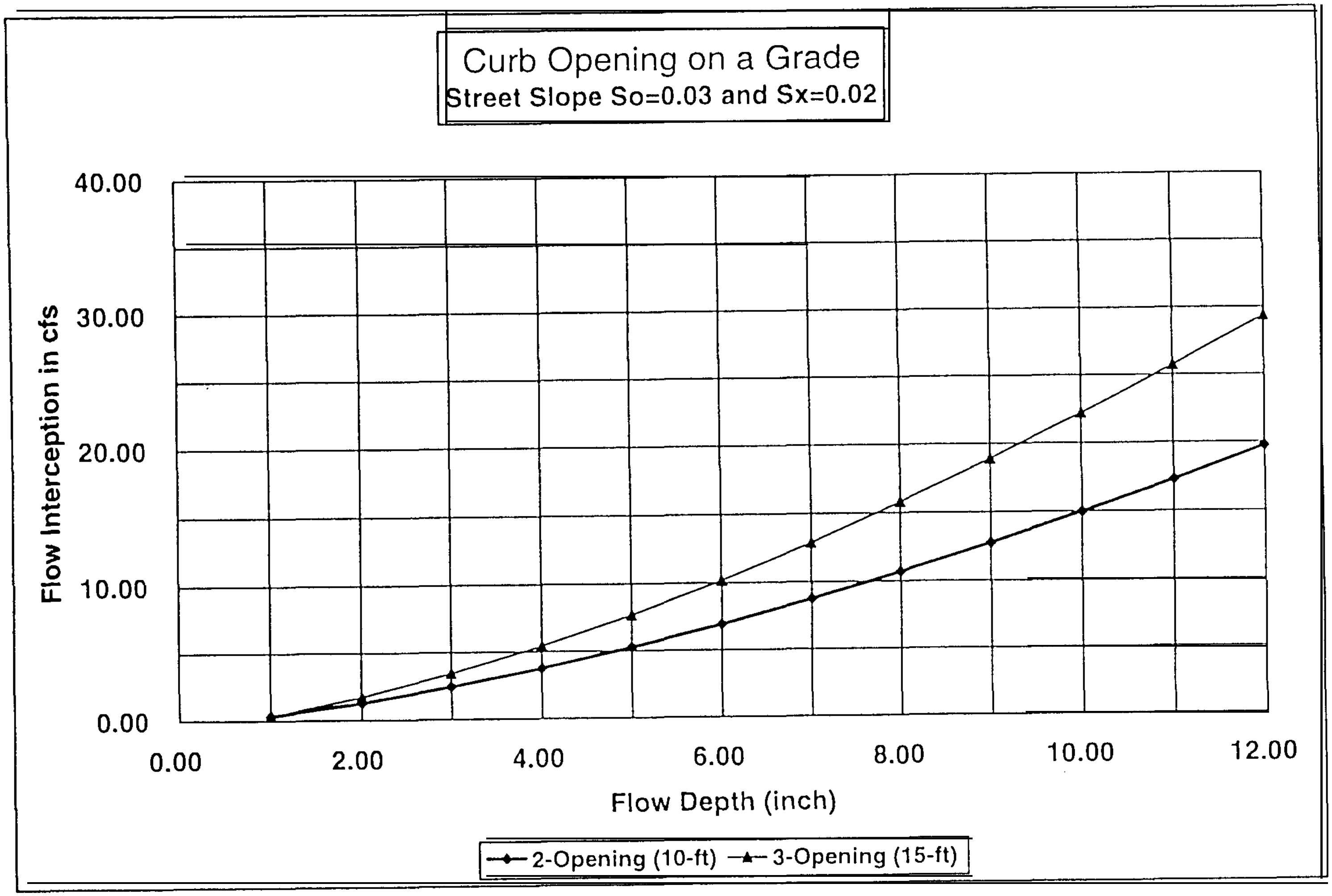
S_o = Street longitudinal slope in ft/ft

T = Water spread on street

Revision	Date

REFERENCE:

FIGURE 809



$$Q_i = \left[1 - \left(1 - \frac{L_c}{L_t}\right)^{1.8}\right] Q_s \qquad L_t = 0.60 Q_s^{0.42} S_0^{0.30} \left(\frac{1}{n S_e}\right)^{0.6}$$

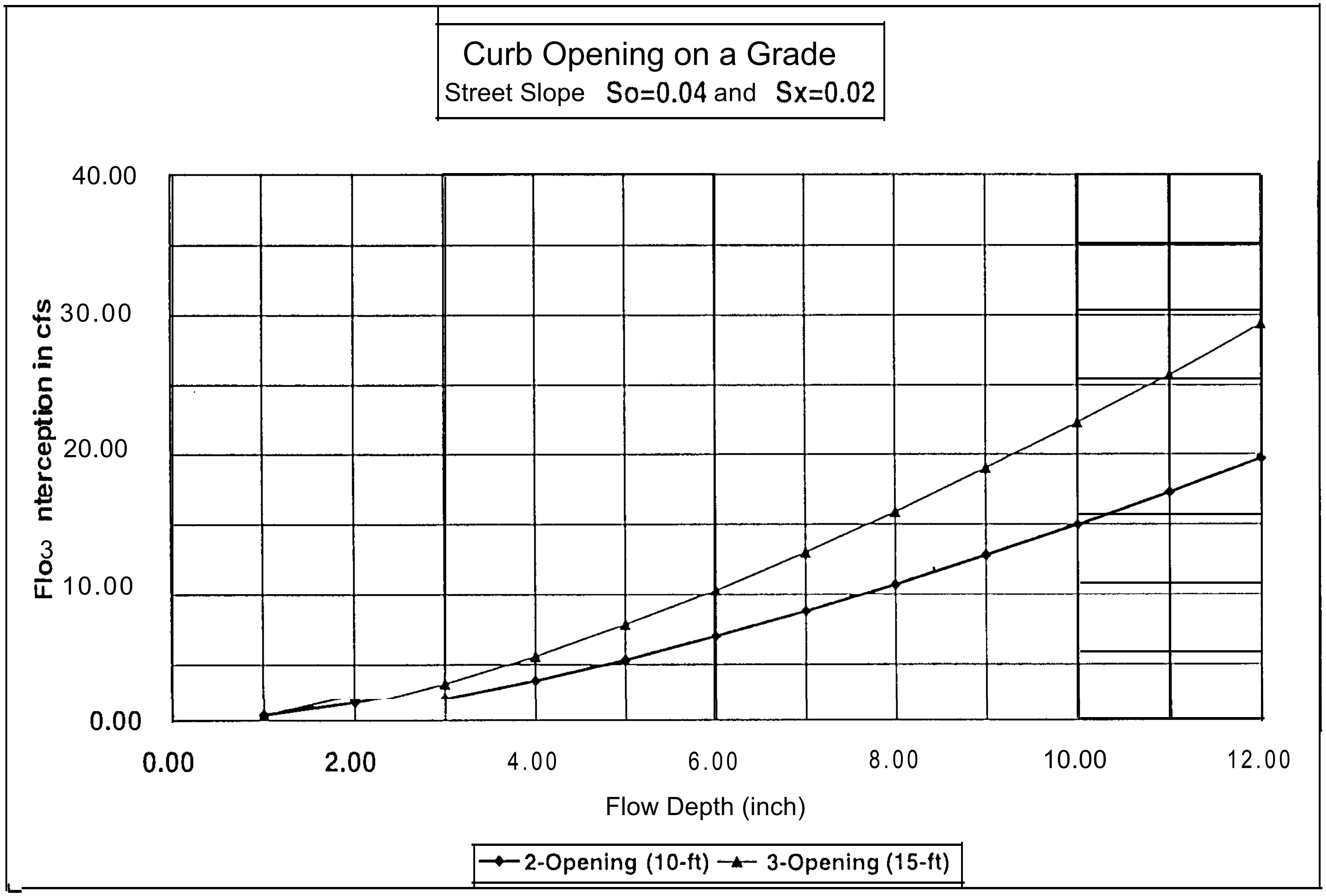
Where: $Q_s = \frac{0.56}{n} S_x^{1.67} T^{2.67} S_0^{0.5}$ $\quad S_e = S_x + (S_w \times E_o)$ $\quad E_o = \frac{Q_w}{Q_s}$

Q_i = Ideal inlet capacity
Q_s = Street capacity
Q_w = Frontal flow
S_w = Gutter cross slope in ft/ft
S_o = Street longitudinal slope in ft/ft
T = Water spread on street
L_t = Required curb opening length in ft
L_c = Designed curb opening length in ft
S_x = Street transverse slope in ft/ft

Revision	Date

REFERENCE:

FIGURE 810



$$Q_i = \left[1 - \left(1 - \frac{L_c}{L_t}\right)^{1.8}\right] Q_s \qquad L_t = 0.604,\ ^{0.42} S_0^{0.30} \left(\frac{1}{n S_e}\right)^{0.6}$$

Where: $Q_s = \frac{0.56}{n} S_x^{1.67} T^{2.67} S_0^{0.5}$ $\quad S_e = S_x + (S_w \times E_o)$ $\quad E_o = \frac{Q_w}{Q_s}$

Q_i = Ideal inlet capacity

Q_s = Street capacity

Q_w = Frontal flow

L_t = Required curb opening length in ft

L_c = Designed curb opening length in ft

S_x = Street transverse slope in ft/ft

S_w = Gutter cross slope in ft/ft

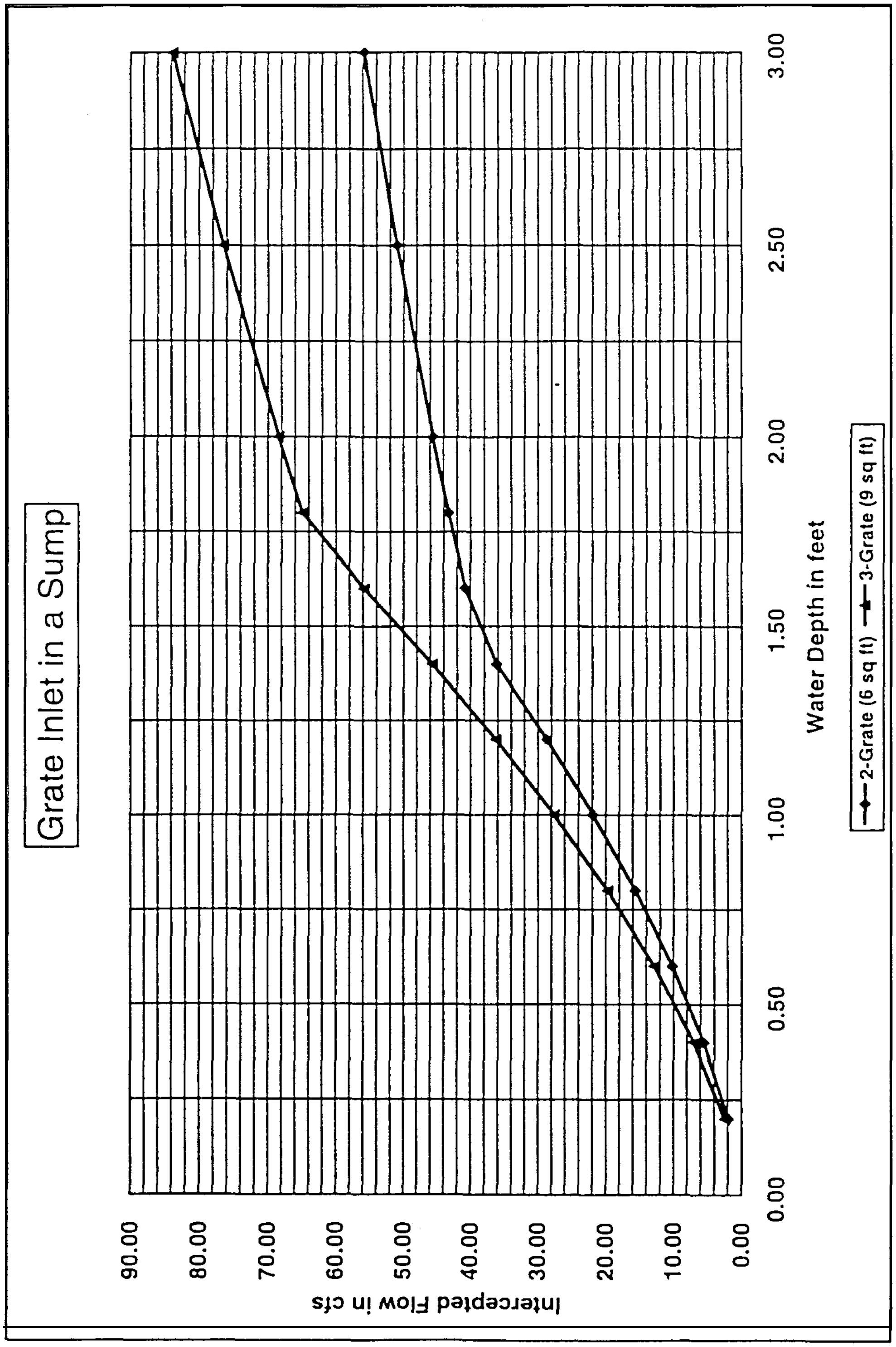
S_0 = Street longitudinal slope in ft/ft

T = Water spread on street

Revision	Date

REFERENCE:

FIGURE 811



$$Q_i = 3.0P(Y_S + Y)^{1.5}$$

Where: $P = 2(L + w)$ without curb

$= 2w + L$ with curb

Q_i = Ideal inlet capacity

P = Wetted perimeter in ft

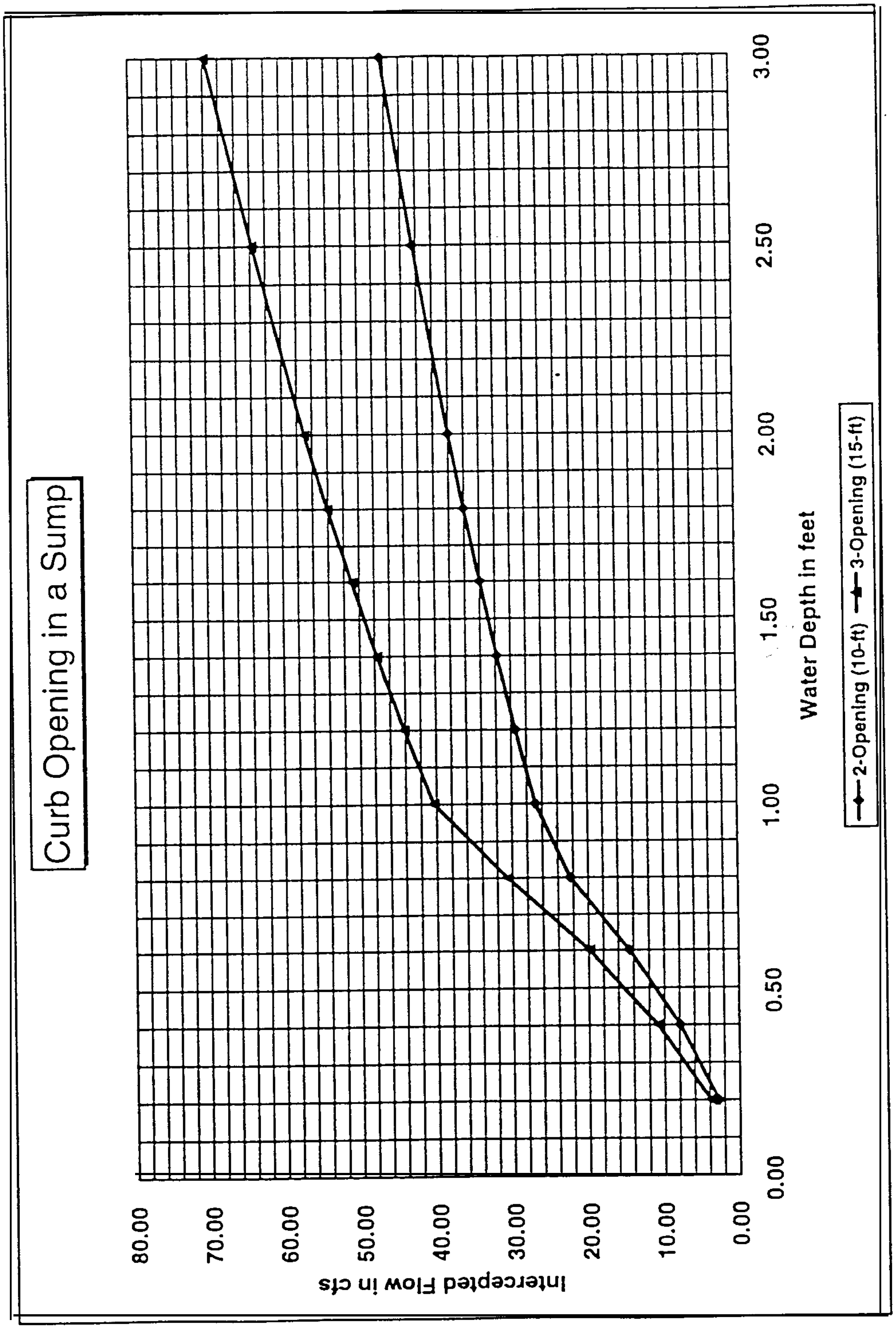
Y_S = Sump depth in addition to flow depth in ft

Y = Gutter flow depth in ft

Revision	Date

REFERENCE:

FIGURE 812



$$Q_i = 2.30\left(L_c + 1.8\, w_g\right)\left(Y_s + Y\right)^{1.5}$$

Where: Q_i = Ideal inlet capacity

L_c = Length of the curb opening inlet in ft

w_g = Lateral width of depression in ft

Y_s = Sump depth in addition to flow depth in ft

Y = Gutter flow depth in ft

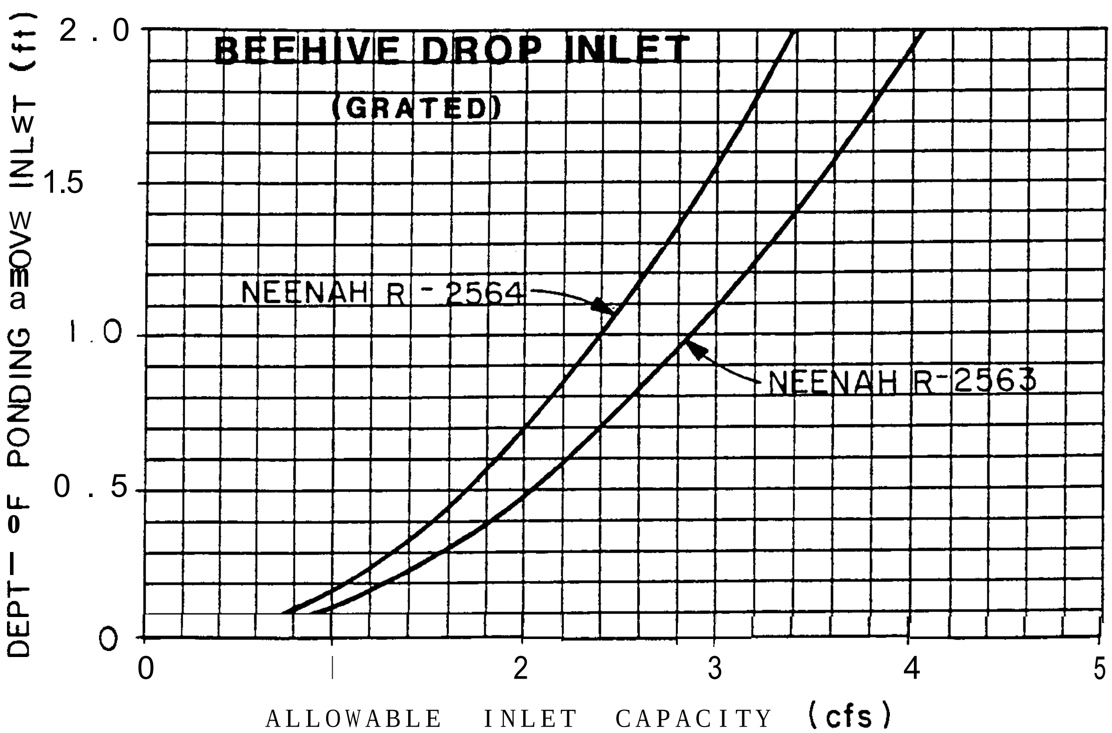
Revision	Date

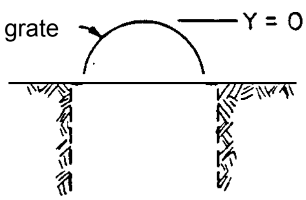
REFERENCE:

FIGURE 813

ALLOWABLE INLET CAPACITY

SUMP CONDITION



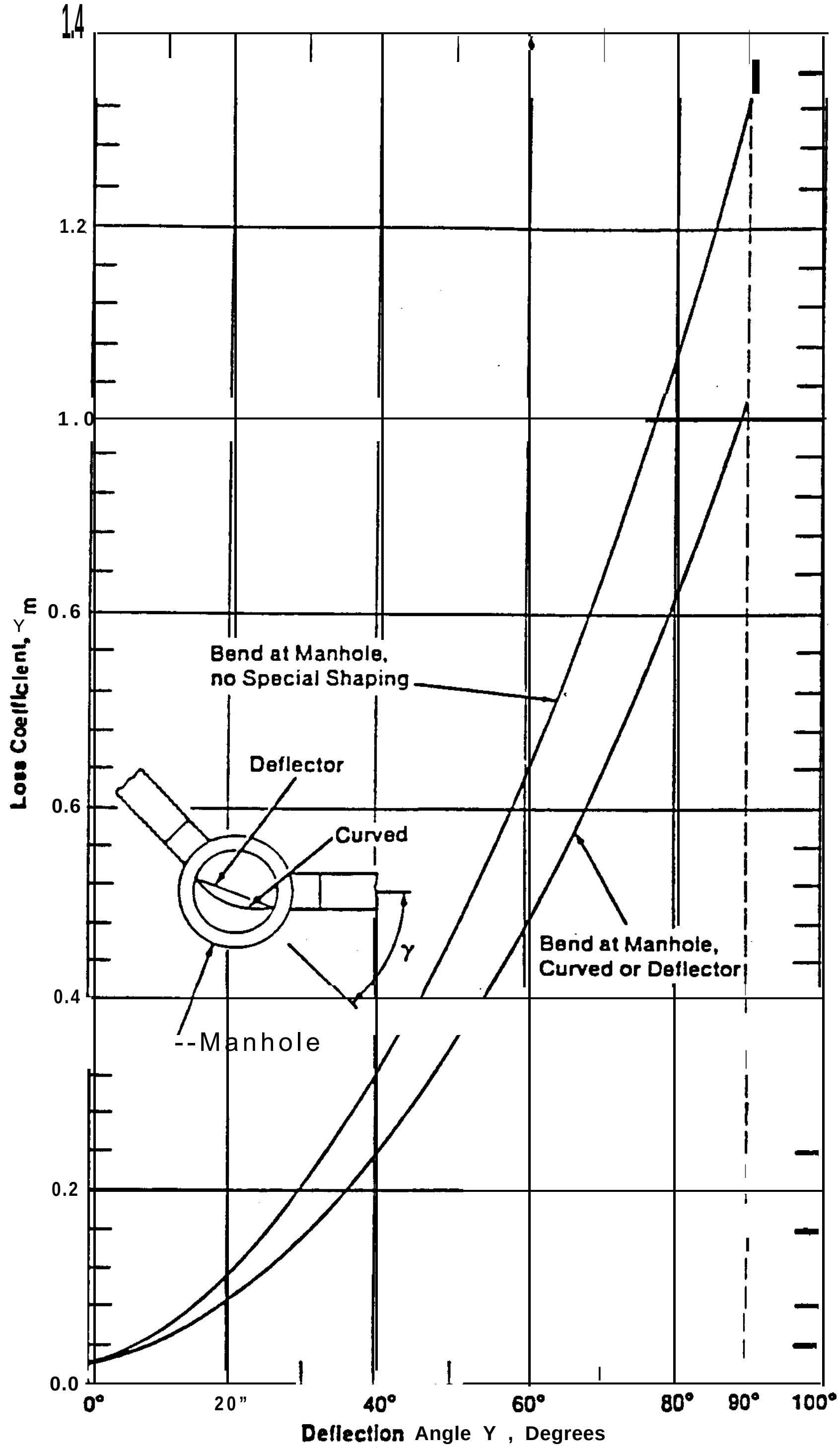


NOTES: 1. Includes capacity reduction factor of 0.50 for Beehive Drop Inlet.

2. This inlet shall only be located in a sump condition.

Revision	Date

ENERGY LOSS COEFFICIENT IN STRAIGHT THROUGH MANHOLE



NOTE: Head loss applied at outlet of manhole.

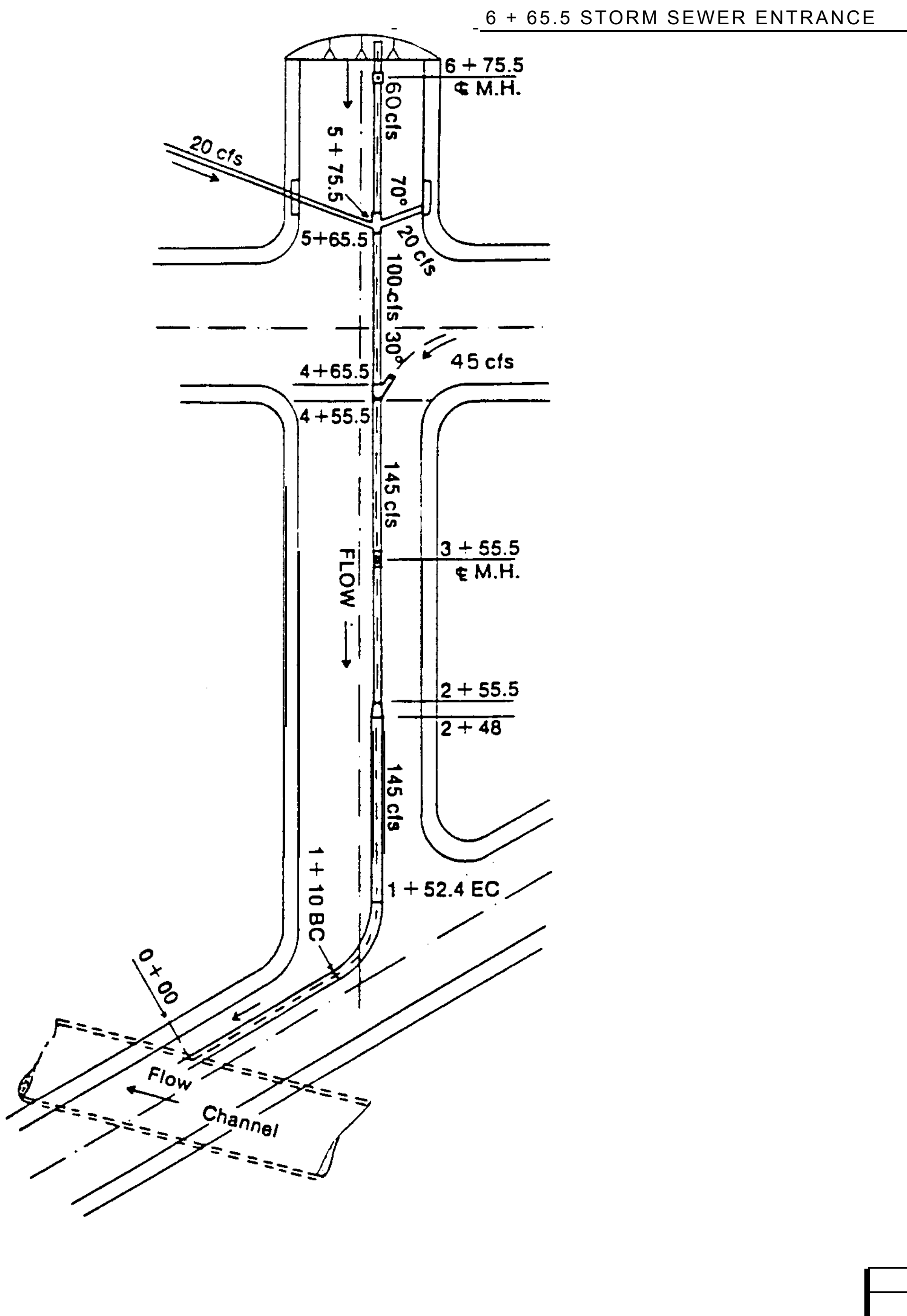
Revision	Date

WRC ENGINEERING

REFERENCE:
Modern Sewer Design, AISI, Washington D.C., 1980

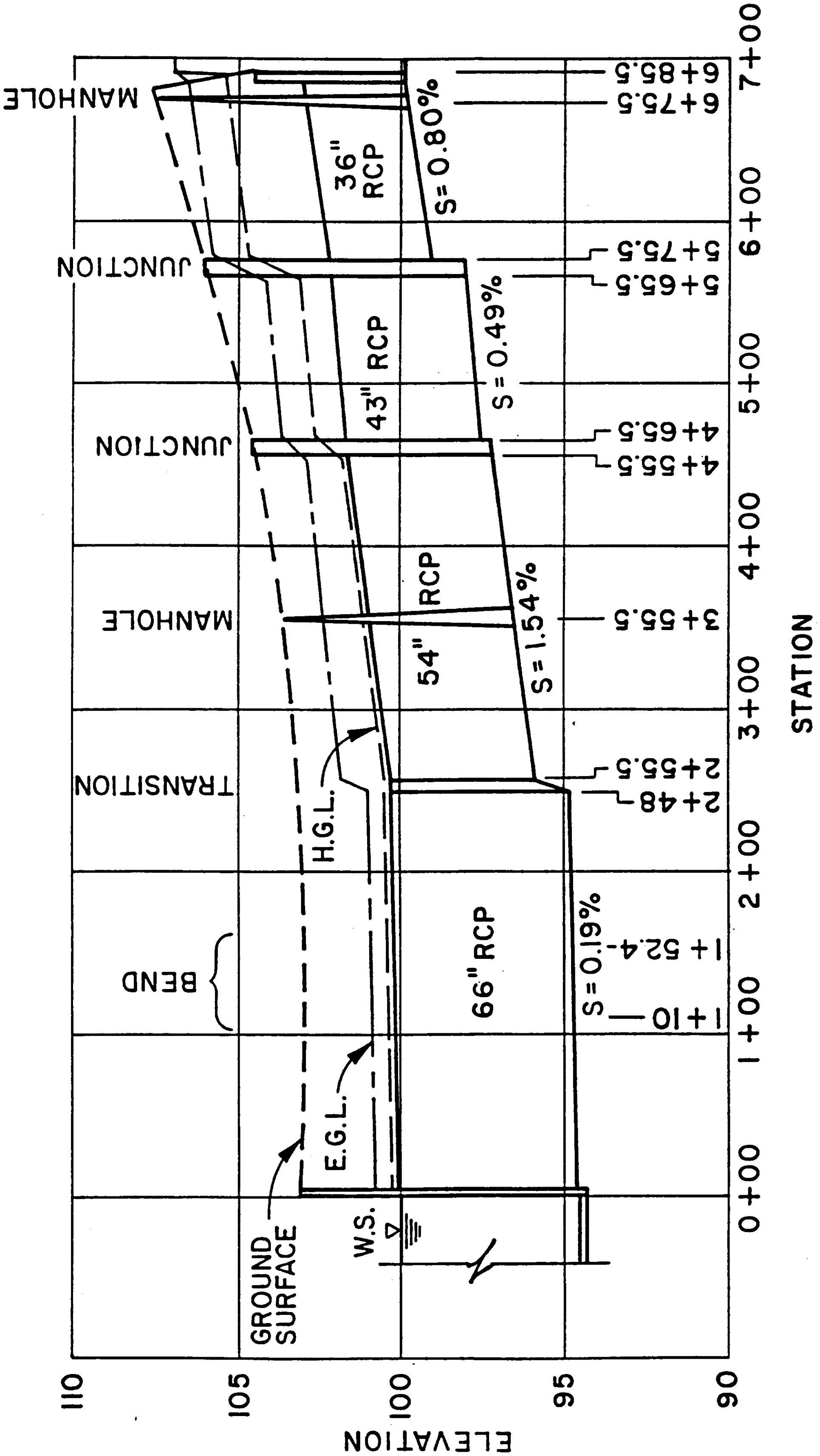
FIGURE 815

STORM SEWER PLAN FOR EXAMPLE IN SECTION 807.2



Revision	Date

STORM SEWER PROFILE FOR EXAMPLE IN SECTION 807.2



NOTE: Since the slope of the 66" RCP is less than 0.25%, special approval would be required from the local entity and/or the CCRFCD.

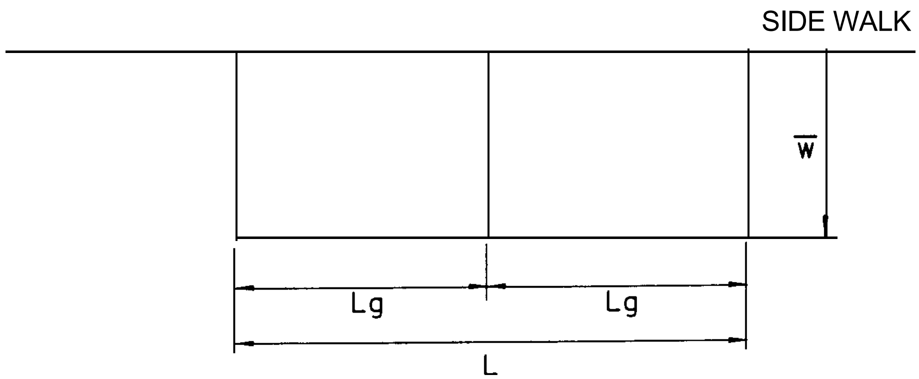
Revision	Date

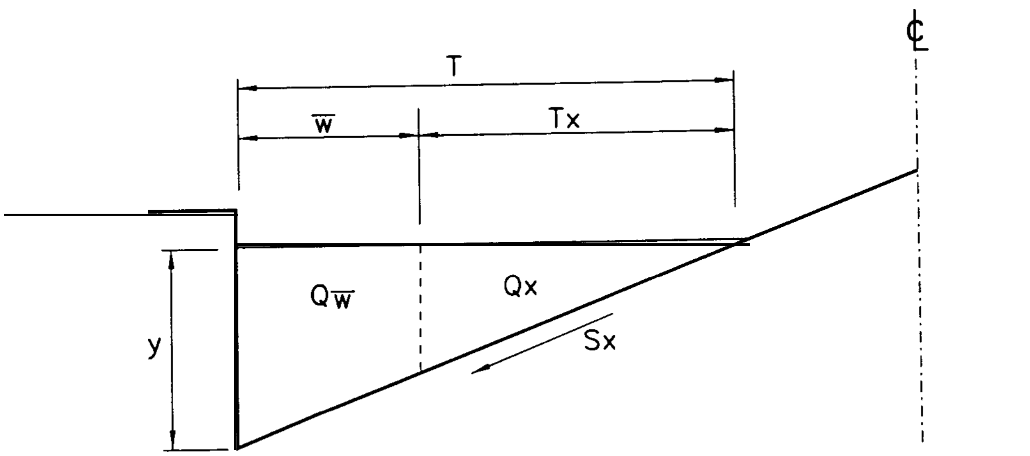
WRC ENGINEERING

REFERENCE: MODERN SEWER DESIGN, AISI, 1980 (with modifications)

FIGURE 817

GRATE INLET CALCULATION FOR EXAMPLE IN SECTION 807.3





Revision	Date

REFERENCE:

FIGURE 818

CLARK COUNTY REGIONAL FLOOD CONTROL DISTRICT
HYDROLOGIC CRITERIA AND DRAINAGE DESIGN MANUAL

SECTION 900
STREETS

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906 STREET CAPACITY CURVES - 100 FOOT ROW WITH MEDIAN

Section 900
Streets

901 INTRODUCTION

The criteria presented in this section shall be used in the evaluation of the allowable drainage encroachment within public streets. The review of all planning submittals (Section 200) which involve storm flow in streets will be based on the criteria herein.

902 FUNCTION OF STREETS IN THE DRAINAGE SYSTEM

Urban and rural streets in the Clark County area having curb and gutter facilities or roadside ditches are part of the Local Drainage System. The streets naturally carry runoff from both the minor and major storm events. For design purposes, the streets are allowed to carry runoff in excess of the minor storm (Section 304.2), subject to certain limitations (Section 304.4). When the storm flows in the street exceed allowable limits (Section 304.4), a storm sewer system (Section 800) or an open channel (Section 700) is required to convey the excess flows. The primary function of urban streets is for traffic movement and therefore the drainage functions are subservient and should not interfere significantly with the traffic function of the street.

Design criteria for the collection and conveyance of runoff water on public streets are based on a reasonable frequency and magnitude of traffic interference. That is, depending on the character of the street, certain traffic lanes can be fully inundated during the storms. During less intense storms, runoff will also inundate traffic lanes but to a lesser degree. The primary drainage function of the streets is to convey minor storm and nuisance flows quickly and efficiently to the storm sewer or open channel drainage with minimal interference of traffic movement. For the major storm event, the function of the streets is to provide an emergency passageway for the flood flows with minimal damage to the urban environment.

903 DRAINAGE IMPACTS ON STREETS

Storm runoff can influence the traffic movement function of a street in the following ways:

1. Sheet flow across the pavement resulting from precipitation runoff
2. Runoff in the gutter
3. Duration of the storm
4. Ponded water
5. Flow across traffic lanes
6. Physical damage to the street

To minimize the drainage impact on the streets, each of the above factors must be understood and controlled to within acceptable limits. The effects of the above factors is discussed in the following sections.

903.1 Sheet Flow

Rainfall on the paved surface of a street or road must flow overland in what is referred to as sheet flow until it reaches a channel. Streets, which have curbs and gutters become the channel, while on roads which have a drainage ditch, the ditch becomes the channel. The depth of sheet flow will be essentially zero at the crown of the street and will increase in the direction of the curb and gutter or drainage ditch.

Traffic interference due to sheet flow is by hydroplaning or by splash. Hydroplaning is the phenomenon of vehicle tires becoming supported by a film of water which acts as a lubricant between the pavement and the vehicle. This generally occurs at higher speeds associated with arterials and freeways and can result in loss of vehicle control. Drainage design can reduce the hydroplaning potential by increasing the street cross slope which drains the runoff more quickly.

Splashing of the sheet flows interferes with traffic movement by reducing visibility. The increase in cross slope of the street crown also reduces the splash potential. In general, a 2 percent cross slope is a desirable practical slope.

903.2 Gutter Flow

Water which enters a street as sheet flow from the pavement surface or as overland flow from adjacent land area will flow in the gutter and possibly a portion of the street section until reaching some outlet, such as a storm sewer inlet or a channel. As the flow progresses downstream and additional areas contribute to the runoff, the width of flow will increase and progressively infringe upon the traffic lane. If the roadway width allows vehicles parked adjacent to the curb, the flow width will have little influence on traffic capacity until it exceeds the width of the vehicle by several feet. However, on streets where parking is not permitted, the flow width significantly effects traffic movement after exceeding a few feet, since the flow encroaches on a moving lane rather than a normal parking lane. Field observations show that vehicles will crowd adjacent lanes to avoid curb flow. This creates a traffic hazard which contributes to the rash of small accidents that occur during rain storms.

As the flow width increases, the traffic must eventually move through the inundated lanes, progressively reducing traffic movement as the depth of flow increases. Although some reduction of traffic movement caused by runoff is acceptable, emergency vehicles (i.e., fire equipment, ambulances, police vehicles) must be able to travel the streets. Therefore, certain limitations on the depth of flow in the street are required.

903.3 Storm Duration

The storm duration also plays a role in the drainage impact on the streets. The high intensity, short duration thunderstorms typical of the Clark County area generally do not influence traffic for a long period of time (generally 30 minutes to 1 hour). Therefore, increased flow depths are tolerable for the shorter flood period.

These periods of inundation will continue after precipitation has stopped .

903.4 Temporary Ponding

Storm runoff temporary ponded on the street due to grade changes or intersection street crowns effects traffic movement by increasing flowdepths and the duration of flow at the greater depths. This temporary ponding is localized and vehicles may enter the ponded area at high speeds unaware of the ponded water until the vehicle is out of control. Ponding will often cause traffic to halt to avoid vehicle stalling, resulting in reduced traffic movement. Therefore, depths of temporary ponding must be controlled in a similar manner to gutter flow and in some cases eliminated on high traffic volume streets.

903.5 Cross Flow

Whenever storm runoff, other than sheet flow, moves across a traffic lane, traffic flow is affected. The cross flow may be caused by super-elevation of a curve, by the intersection of two streets, by exceeding the capacity of the higher gutter on a street with cross fall, or simply poor street design. The problem associated with this type of flow is the same as for ponding in that it is localized in nature and vehicles may be traveling at high speed when theyreach the location. If the speed limits are slow and the traffic volume is light, then the influence ofcross street flow may be within acceptable limits.

903.6 Parking Lots and Driveways

The maximum depth of flow through a parking lot is dependent upon a criterion that depth x velocity is less than 6. Hydraulic calculations should be limited to the area of the parking lot where cars are not parked. Parking stalls must not be included in the computation of conveyance areas.

Driveways are often graded to act as berms to protect commercial properties. In these cases, the freeboard shall be defined as one-half the velocity head, but not less than 6 inches. In other words, freeboard will be a minimum of 6 inches above the 100-year flow depth in the street.

904 DRAINAGE IMPACT ON STREET MAINTENANCE

The use of the roadway system for drainage of runoff during and immediately after storm events also has an impact on the structural integrity of the pavement system and the roadway maintenance required. If water penetrates the road surface and saturates the sub-grade material, the sub-grade may fail and cause failure of the pavement.

Additionally, runoff from rural and urban areas can carry large amounts of debris and sediment, which may reduce the performance of hydraulic structures or become a safety hazard and must be removed.

904.1 Pavement Deterioration

The efficient removal of a storm runoff from pavement surfaces has a positive effect on street maintenance and repair. Street maintenance and repair procedures can in turn affect the efficiency of a street as part of the runoff collection system. Research has indicated that pavement deterioration is accelerated by the presence of storm runoff.

Pavement surfaces are subject to numerous types of distress such as weathering, raveling, long cracks, alligator cracks, chuck holes, bleeding, depression, and edge breakup. Water is probably the greatest cause of distress in a pavement structure. Flow of water across a bituminous pavement surface has little effect on the pavement so long as the pavement retains its watertight condition. A number of types of pavement distress may cause the pavement to become permeable, allowing water to reach the sub-grade. Once the water reaches the sub-grade, the problems multiply as the sub-base and sub-grade weakens and increases the cracks through the surface.

A common practice to reduce the problem of bituminous surface deterioration is to seal-coat or overlay the surface. This reduces the problem of pavement deterioration, but indirectly creates a problem with the carrying capacity of the adjacent gutter. As the street section is resurfaced, the flow area of the section is decreased. Over a period of 20 to 30 years, a considerable portion ofthe runoff carrying capacity of the street may be lost. Scarifying the surface to remove the upper layer of asphalt prior to resurfacing reduces the problem, but is expensive. In any case, the street section flow capacity must be maintained.

904.2 Sedimentation and Debris

A common problem in Clark County is the deposition of sediment on the street surface during and after a storm event. During the flow event, this sedimentation can cause problems by reducing the flow carrying capacity of the street section and causing increased encroachment into the traffic lanes. This problem is most prevalent at major grade changes where the flow velocity in the street section is reduced. Reducing the flow velocity decreases its sediment transport ability and sediment is deposited.

Additionally, sediment and other debris carried by runoff can impair the operation of hydraulic structures such as curb inlets and grated drop inlet structures. The sediment and debris can block a portion of the flow area into these facilities and cause artificially increased water surface elevations.

Immediately after a storm event, identified problem areas should be reviewed and street sweeping initiated to remove accumulated sediment and debris. By regularly scheduled sweeping of upstream areas the source of some of the sediment can be eliminated. Also, runoff from construction sites may cause site-specific sedimentation problems and should be controlled as recommended in Section 1300.

904.3 Landscaped Areas

If flow is expected in landscaped areas behind the back-of-sidewalk (i.e., when flows are over the top-of-curb), such areas need to be protected from erosion to stabilize them. This is of particular concern where building pad elevations are below the top-of-curb elevations.

905 STREET CLASSIFICATION AND ALLOWABLE FLOW DEPTH

The streets in Clark County are classified according to traffic volume and ROW width. The standard street sections are provided in Drawings 202-210 of the STANDARD DRAWINGS. The street classifications, ROW requirements, and allowable storm flow depth criteria are provided in Policy Section 304.4.

A minimum street slope of 0.4 percent (0.004 ft/ft) or as identified in Section 1600 shall be used. The outside slope shall be used for “knuckles” in roadways. Where this slope cannot be achieved, mitigation shall be considered through underground storm drains at flatter slopes.

The calculation of the water surface elevation and velocity must be based on limiting the flow to the width of the ROW. This implies that for calculation purposes only, an infinitely high vertical wall exists at the right of way boundary and any flow area outside of the ROW is not considered in the analysis.

906 HYDRAULIC EVALUATION

The hydraulic analysis of flow in street sections is similar to open channel flow analysis for larger flood control channels (Section 700). The basic governing equation, Manning’s equation, is as follows:

$$Q = (1.49 / n) AR^{2/3} S^{1/2} \tag{901}$$

where Q = Discharge in cfs
n = Roughness Coefficient (0.016)
A = Flow Area in sq ft
R = Hydraulic Radius, A/P
P = Wetted Perimeter, ft
S = Slope of the EGL, Generally Assumed Equal to the Street Slope, ft/ft

Based upon the policy of Section 304.4, the allowable storm capacity of the minor storm of each street section is calculated using **Equation 901**.

The calculation of depth of flow for the major storm event is also based on **Equation 901.** The major difference is in the assumed flow area. For the calculation of flow depth and velocity, the area outside the limits of the right of way is not considered in the calculation of conveyance. Even though water will flow in the area outside of the ROW, the depth of flow allowed is based on containment of the flow within the ROW.

The maximum allowable capacity for standard Clark County area street cross-sections has been calculated and is presented in **Figures 901** through **906**. The calculations were performed for various allowable flow depths and street slopes. A Manning's "n" value of 0.016 was assumed for the gutter and street flow areas and a cross slope of 2 percent was used. If standard street sections are used, the maximum allowable street capacity shall be obtained from **Figures 901** through **906**. If non-standard sections are used, the standard Manning's equation with a Manning's "n" value of 0.016 shall be used to calculate allowable flows.

Streets with grades flatter than 0.4 percent must be given special consideration when calculating allowable flow depth. These streets are subject to ponding and are candidates for storm sewers. Storm sewers and their inlets are described in Section 800.

907 EXAMPLE APPLICATION

907.1 Introduction

The criteria and methods developed in Section 900 will be used to calculate the allowable flow in a standard street section.

907.2 Example: Allowable Flow In 100 Foot ROW Street

<u>Problem:</u> A major arterial roadway (100 foot ROW without median) is to be constructed with a longitudinal slope of 1.5 percent with the standard 6-inch curb height. A determination of the allowable street capacity is required to determine the need for storm inlets and storm sewers.

<u>Solution:</u>

Step 1: Enter **Figure 905** with a longitudinal street slope of 1.5 percent and record flows for vd = 6 and vd = 8 (See Policy Section 304.4)

For Minor Storm:

vd = 6; Q = 120 cfs; d = 0.95 ft for 1/2 street)

For Major Storm:

vd = 8; Q = 190 cfs; d = 1.1 ft (for 1/2 street)

Step 2: Calculate allowable flow depth to provide 12 foot dry lane in each direction (center turning lane cannot be used for dry lane).

Allowable Flow Width for Minor Storm =

45 ft - 0.5 ft - 6.0 ft - 12.0 ft = 26.5 ft

Step 3: Check for adequate dry lane width by calculating flow width for minor storm based on allowable vd = 6.

W = Width of Gutter + (Flow Depth - 2 in) / 0.02
= 1.5 ft + (0.95 - 2 / 12) /0.02
= 1.5 ft + 39.2 ft
= 40.7 ft

Therefore, the allowable depth of flow in street must be reduced.
Allowable Flow Depth for Minor Storm =

(26.5 ft - 1.5) x 0.02) + 2 in / 12 = 0.67 ft

Step 4: From **Figure 905** find flow for depth of 0.67 feet and a longitudinal slope of 1.5 percent.

Allowable Flow for Minor Storm =

35 cfs / gutter

Step 5: Determine minimum required structures elevation above gutter flowline (H).

The maximum major storm flow depth of 1.1 feet must be checked against the allowable water surface criteria provided in Policy Section 304.4.

1. Special Flood Hazard Area and Areas of Interim Delineation:

   Residential finished floor elevations

   H = 1.1 ft + 1.5 ft = 2.6 ft above gutter flowline.

2. Non-Special Flood Hazard Areas:

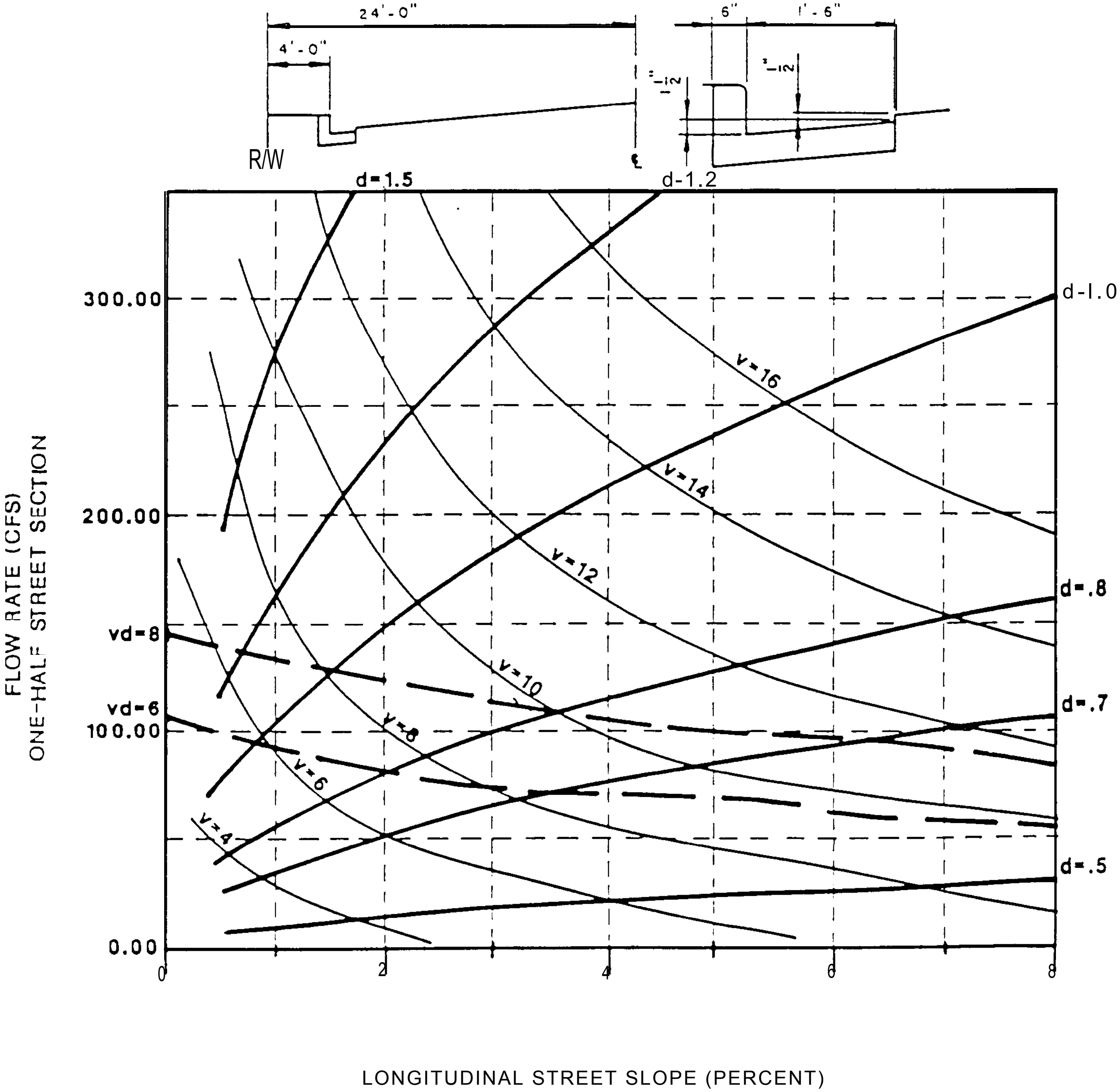
   Finished floor elevation = 2 (1.1) or 1.5 feet above gutter flowline, whichever is greater.

3. Therefore, if the above criteria can be met, the allowable flows for the total street section are as follows:

   Minor Storm = 70 cfs
   Major Storm = 380 cfs

Step 6: Compare allowable street capacity to design runoff rates. If runoff rates exceed street capacity, then a storm sewer system or channel system will be required.

STREET CAPACITY CURVES
48 FOOT ROW



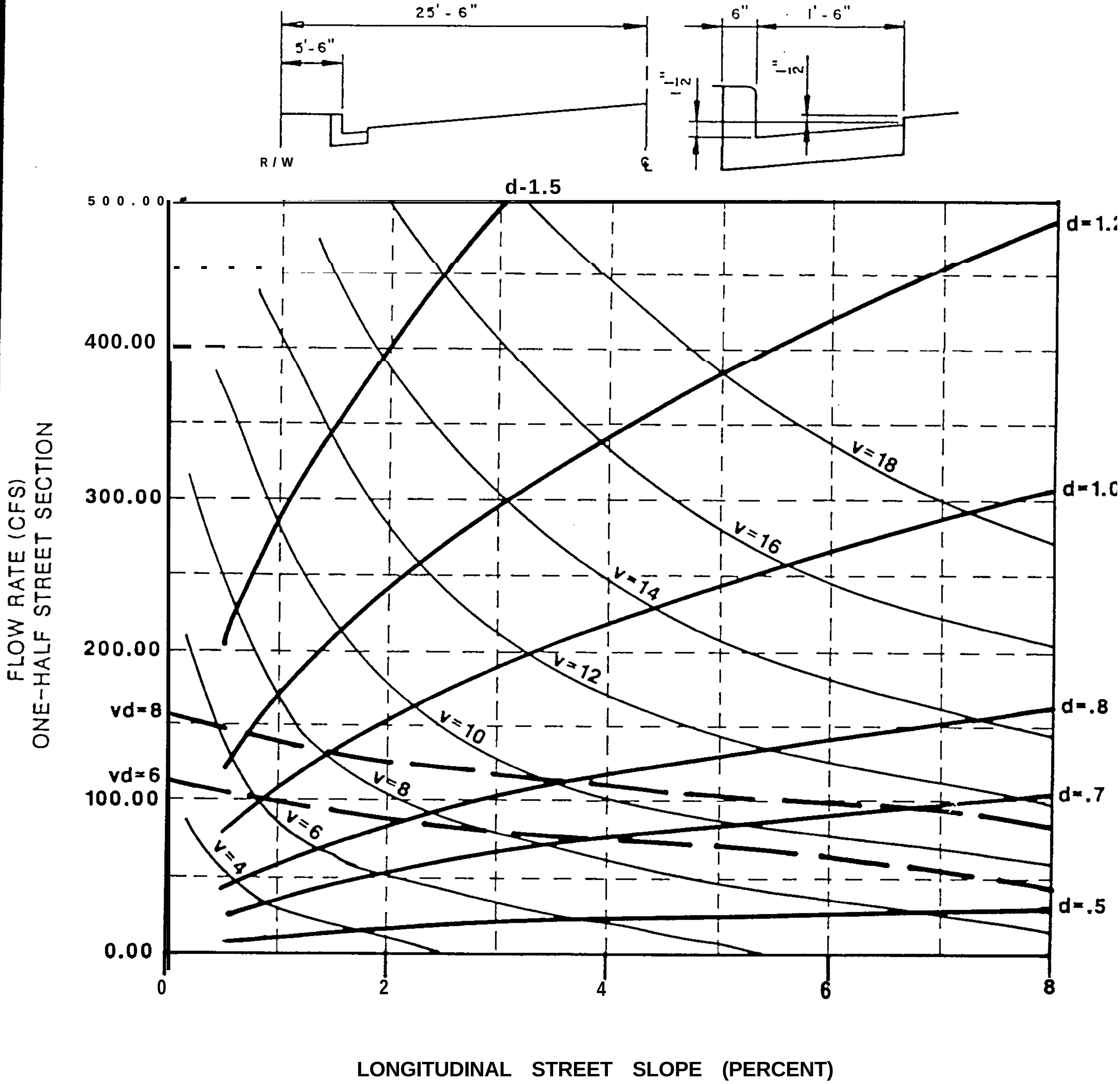
EXPLANATION

v velocity (feet per second)

d depth (feet)

Revision	Date

STREET CAPACITY CURVES
51FOOT ROW



EXPLANATION

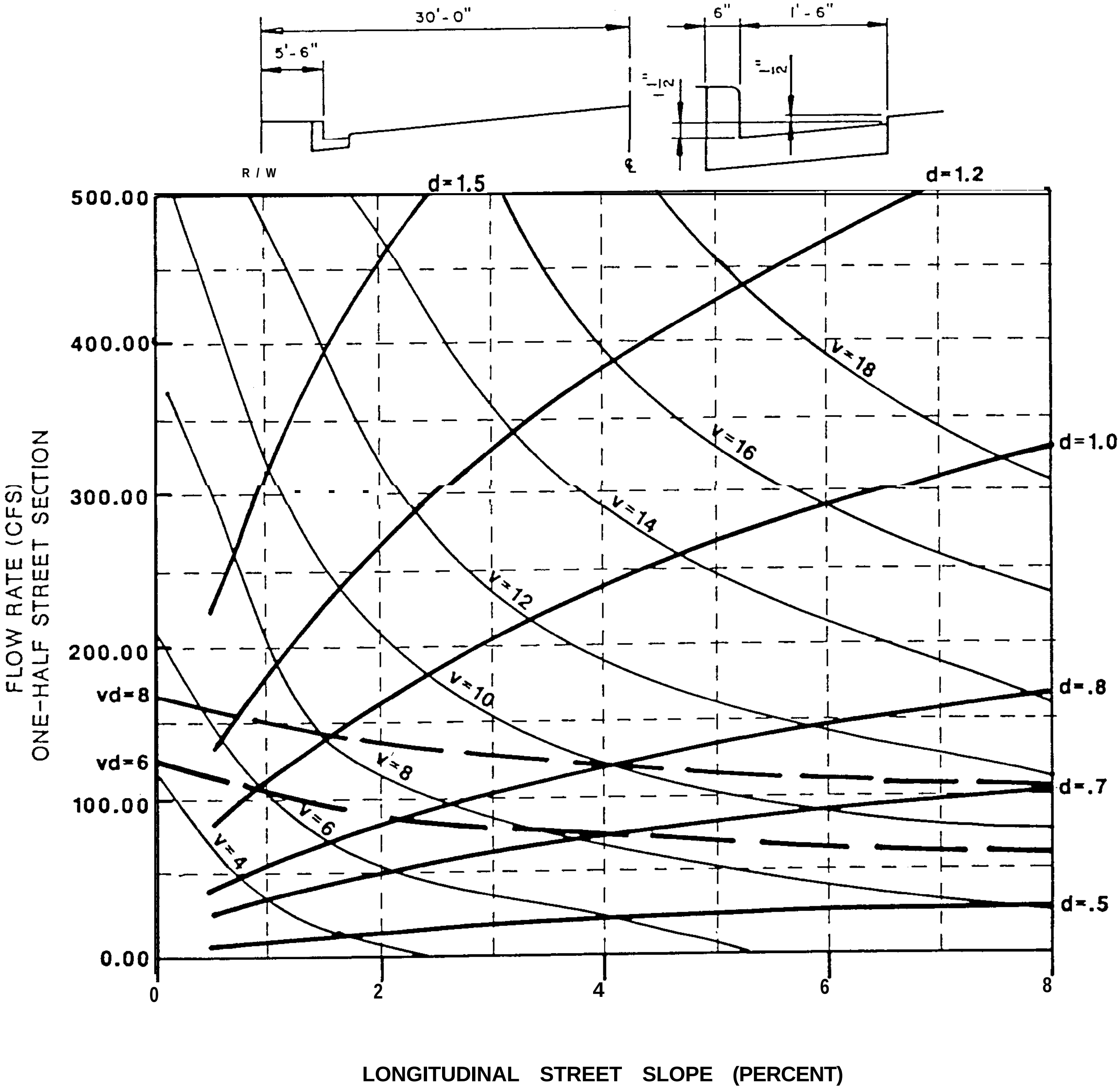
v velocity (feet per second)

d depth (feet)

Revision	Date

WRC ENGINEERING | REFERENCE: | FIGURE 902

STREET CAPACITY CURVES
60 FOOT ROW



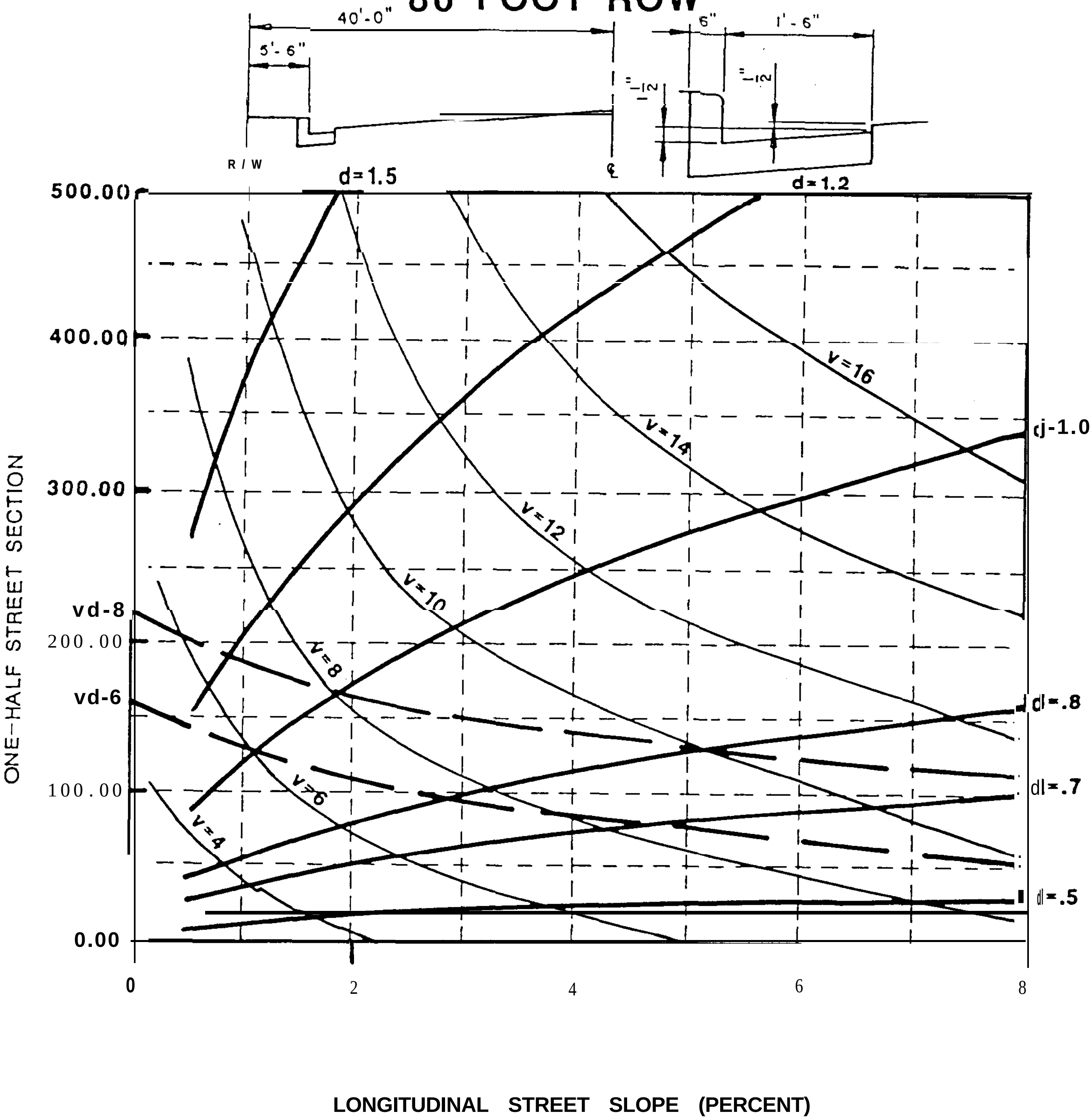
EXPLANATION

v vdocity (feet per second)

d depth (feet)

Revision	Date

STREET CAPACITY CURVES
80 FOOT ROW



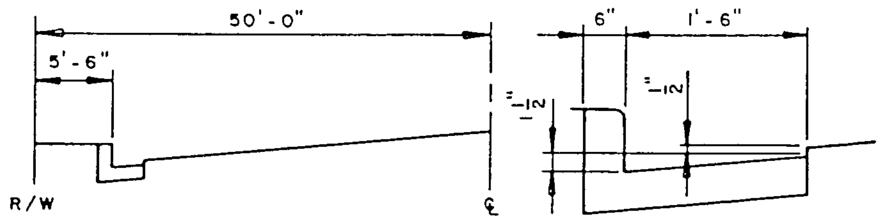
EXPLANATION

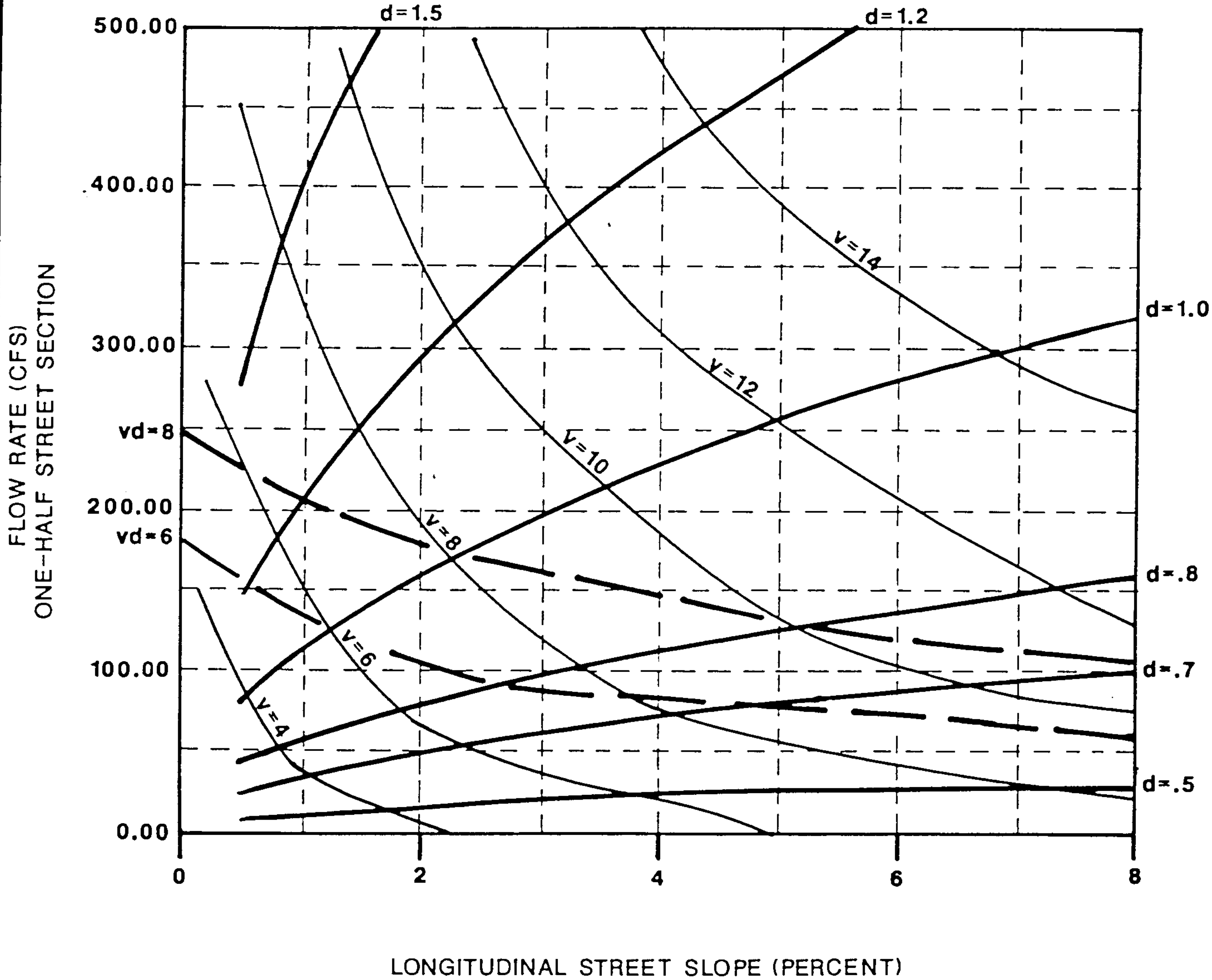
v velocity (feet per second)

d depth (feet)

Revision	Date

STREET CAPACITY CURVES
100 FOOT ROW





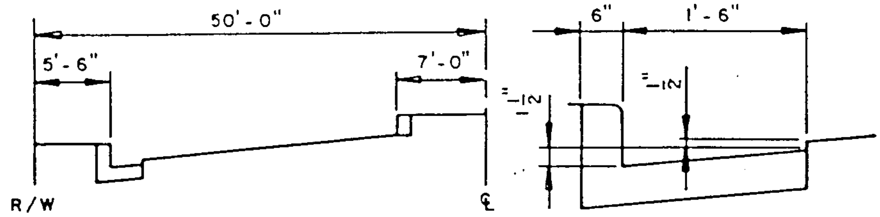
EXPLANATION

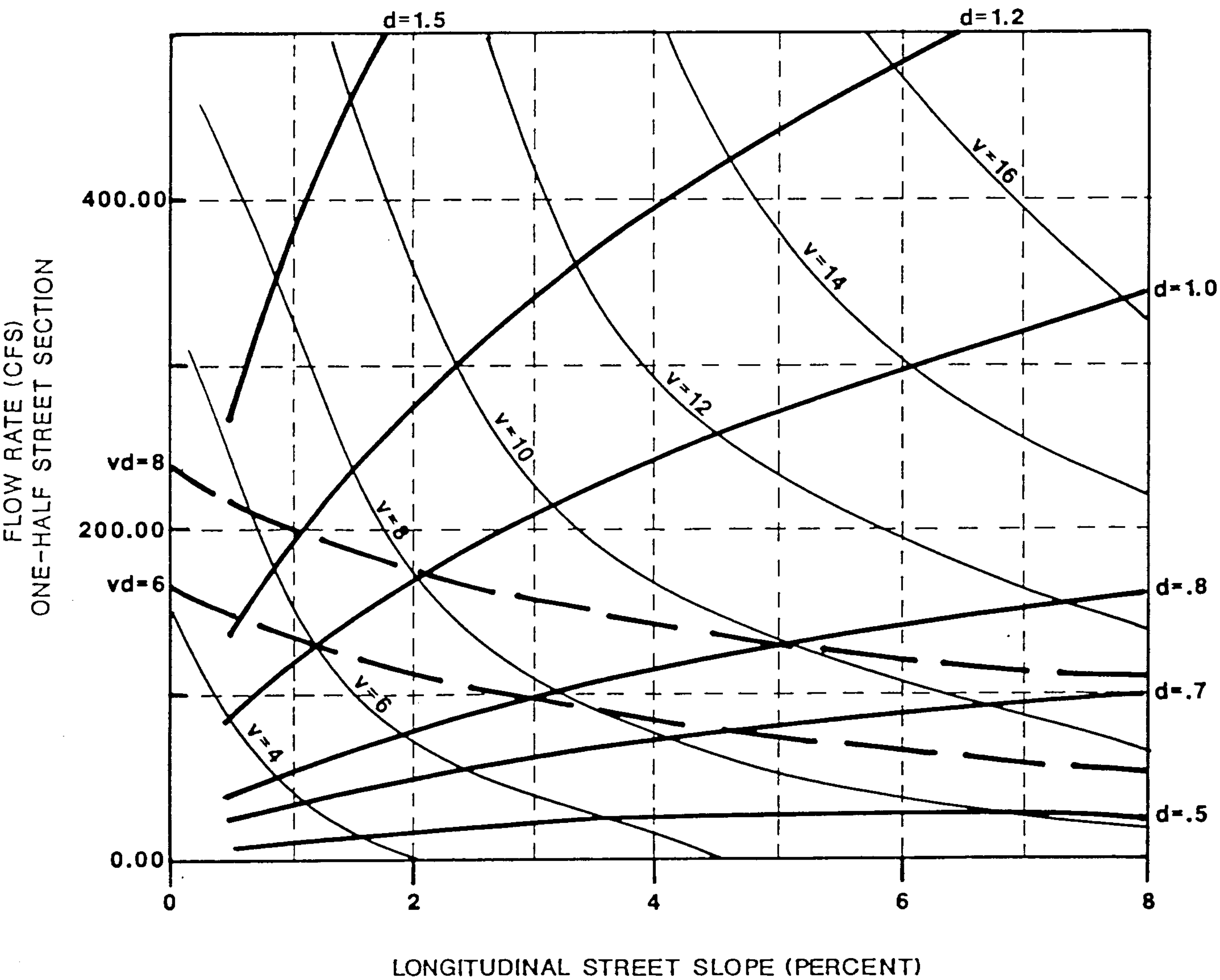
v velocity (feet per second)

d depth (feet)

Revision	Date

STREET CAPACITY CURVES
100 FOOT ROW WITH MEDIAN





EXPLANATION

v velocity (feet per second)

d depth (feet)

Revision	Date

CLARK COUNTY REGIONAL FLOOD CONTROL DISTRICT
HYDROLOGIC CRITERIA AND DRAINAGE DESIGN MANUAL

SECTION 1000
CULVERTS AND BRIDGES

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1011 NOMOGRAPH - OUTLET CONTROL CMP

Section 1000
Culverts and Bridges

1001 INTRODUCTION

Culverts and bridges are used to convey water through or beneath engineered structures. The size, alignment, and support structures of a bridge or culvert will directly affect the carrying capacity of the drainage system. Inadequate culvert or bridge capacity can force water out of the conveyance system and the flood water may take an alternate path and cause damage away from the channel.

The primary distinction between a culvert and a bridge is the change in flow area from the upstream channel cross-section. A culvert is usually designed to allow the design upstream water surface elevation to be greater than the top of the culvert, while bridge design generally allow freeboard between the water surface elevation and the low chord of the bridge.

For the purposes of this MANUAL, any facility passing flow transverse to a roadway will be designed under bridge criteria ifit is on an alignment shown on the CCRFCD's Master Plan.

1002 DESIGN STANDARDS FOR CULVERTS

All culverts within the CCRFCD shall be designed and constructed using the following standards. The analysis and design shall consider design flow, culvert size and material, entrance structure layout, outlet structure layout, and erosion protection.

1002.1 Culvert Sizing Criteria

For hydraulic analysis, sizing of culverts is important because of potential effects on water surface elevations in a channel. 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.

1002.1.1 Design Frequency

As indicated in Policy Section 304.5, all culverts in the CCRFCD will be designed to pass the flow from the major storm including an overflow section where permitted.

1002.1.2 Allowable Cross Street Flow

Cross street flow of the design storm flow will not be allowed except on streets with ROW less than 80 feet. In addition, the overflow will only be allowed on these roadways if the product of the velocityand depth of the overflow is less than six. If the product is greater than six, the culvert size must be increased.

The maximum allowable depth at the road crown of any overflow section is 2.0 feet. Additionally, all overflow sections must be posted and depth indication markers placed at the location of greatest depth.

1002.1.3 Minimum Size

The minimum culvert size shall be 18-inch diameter for round pipe or shall have a minimum flow area of 2.2 square feet for other pipe shapes.

1002.2 Construction Materials

The material and shape of culverts shall be in accordance with the STANDARD SPECIFICATIONS.

Soil tests are required for all placements of corrugated steel pipe. If tests indicate corrosive soil conditions, coatings may be required.

The required thickness of corrugated steel pipe depends on many factors including depth of cover, weight of backfill, diameter of culvert, design load, and corrugated dimensions. Designers are directed to Handbook of Steel Drainage and Highway Construction Products by The American Iron and Steel Institute for design standards (AISI, 1983).

Other pipe materials may be used for culvert construction upon approval by the local entity and/or the CCRFCD. Documentation must be submitted for review which shows that the subject pipe material has a design life similar to the above materials and that the interior lining, if any, will maintain the design Manning's "n" value for the life of the pipe material.

1002.3 Velocity Limitations and Outlet Protection

In the proper design of culverts, the velocity of the flow through the culvert is very important. If the velocity is too low, suspended sediment in the flow may settle. This decreases the effective area of the culvert and increases the frequency of required maintenance. If the velocity of the flow exiting the culvert is too high, erosion may take place, possibly jeopardizing the integrity of the roadway.

The criteria for outlet erosion protection for discharges to channels with unlined bottoms are as follows:

Outlet Velocity (fps)	Required Outlet Protection
Less than 5	Minimum riprap protection (Section 707.4)
Between 5 and 15	Riprap protection (Section 707.4) or Energy dissipator (Section 1102.2)
Greater than 15	Energy dissipator (Section 1102.2)

1002.4 Headwater Criteria

The maximum headwater for the design storm flow for culverts greater than 36-inch diameter or a culvert rise of 36-inch shall be 1.5 times the culvert height. The maximum headwater for culverts with a height of 36-inchor less shall be 5 feet if adjacent properties are not adversely affected. If the design flow exceeds 500 cfs in an urban area, the maximum headwater shall not exceed the heightof the culvert for an ultimate condition.

1002.5 Alignment

The 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 transition problems.

Where the natural channel alignment would result in an exceptionally long culvert, modification to the natural alignment may be necessary. Since such modifications will change the natural stability of the channel, such modifications should be thoroughly investigated. Although the economic factors are important, the hydraulic effectiveness of the culvert must be given major consideration. Improper culvert alignment may cause erosion to adjacent properties or siltation of the culvert. Culvert alignment considerations are shown in **Figure 1003**.

Roadway alignment also impacts culvert design. The vertical alignment of roadways will fix the maximum culvert diameter that can be used. This may force the use of elliptical or arched culverts or the use of a multiple barrel culvert system.

1002.6 Temporary Crossing

Temporary crossings are defined as dip road sections with a culvert sized to pass nuisance flow, or a culvert system that does notmeet criteria presented in Section 1000.

Temporary crossings will be reviewed on a case by case basis. Major consideration will be given to the following items:

1. Drainage area contributing to crossing
2. Level of roadway traffic
3. Vertical and horizontal roadway alignment (sight distance)
4. Alternate access routes
5. Time frame for temporary crossing
6. Current and projected development density
7. 10-year and 100-year storm flows

1002.7 Multiple Barrel Culverts

If the available fill height limits the size of culvert necessary to convey the flood flow, multiple culverts can be placed. If a multiple culvert consisting ofthe same type and size of barrel is placed so that all the elements are equal, the total flow is assumed to be equally divided to each of the barrels.

1003 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 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 on the culvert and the inlet geometry. Under outlet control, the flow through the culvert is controlled additionally by culvert slope, roughness, and tailwater elevation.

1003.1 Inlet Control Condition

Inlet control for culverts may occur in two ways (see **Figure 1001**):

1. Unsubmerged – The headwater is not sufficient to submerge the top of the culvert and the culvert invert slope is super-critical. The culvert acts like a weir (Condition A, **Figure 1001**).

1003.1 Inlet Control Condition

Inlet control for culverts may occur in two ways **(see Figure 1001):**

1. Unsubmerqed - The headwater is not sufficient to submerge the top of the culvert and the culvert invert slope is super-critical. The culvert acts like a weir (Condition A, **Figure 1001).**

2. Submerqed - The headwater submerges the top of the culvert but the pipe does not flow full. The culvert inlet acts like an orifice (Condition B, **Figure 1001).**

The inlet control rating for several culvert materials, shapes and inlet configurations are presented in **Figures 1004** to 1007. Additional nomographs are available in HDS No. 5. These nomographs were developed empirically by the pipe manufacturers, Bureau of Public Roads, and the Federal Highway Administration (USDOT, 1985). The nomographs shall be used in the CCRFCD area, rather than the orifice equation, due to the uncertainty in estimating the orifice coefficient.

1003.2 Outlet Control Condition

Outlet control will govern if the headwater and/or tailwater is deep enough, the culvert slope 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 1001).**

2. The headwater submerges the top of the culvert and the culvert is unsubmerged by the tailwater (Condition B or **C, Figure 1001).**

3. The headwater is insufficient to submerge the top of the culvert. The culvert slope is sub-critical and the tailwater depth is lower than the pipe critical depth (Condition D, **Figure 1001).**

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 1002).** The equation is then expressed as:

$$H = h_e + h_f + h_v \tag{1001}$$

where H = Total Energy Head (ft)
h_e = Entrance Head Losses (ft)
h_f = Friction Losses (ft)
h_v = Velocity Head (ft) = $V^2 / 2g$ (1002)

For inlet losses, the governing equation is:

$$h_e = k_e (V^2 / 2g) \tag{1003}$$

where k_e is the entrance loss coefficient. Typical entrance loss coefficients recommended for use are given in **Table 1001 (D)**.

Friction loss is the energy required to overcome the roughness of the culvert and is expressed as follows:

$$h_f = (29n^2 L / R^{1.33}) (V^2 / 2g) \tag{1004}$$

where n = Manning's Coefficient (see **Table 1001**)
L = Length of Culvert (ft)
R = Hydraulic Radius (ft)
V = Velocity of Flow (fps)

Combining the **Equations 1001, 1002, 1003**, and **1004** and simplifying the terms results in the following equation:

$$H = [K_e + (29n^2 L / R^{1.33}) + 1] V^2 / 2g$$

Equation 1005 can be used to calculate the culvert capacity directly when the culvert is flowing under outlet Conditions A or B as shown on **Figure 1001**. The actual headwater (Hw) is calculated by adding H to the tailwater elevation (see **Figure 1002**). For Conditions C or D, the HGL at the outlet is approximated by averaging the critical depth and the culvert diameter, which is used if the value is greater than the tailwater depth (Tw) to compute headwater depth (Hw) this is an approximate method and is more fully described in <u>Hydraulic Design Series No. 5</u>, Bureau of Public Roads.

A series of outlet control nomographs for various culvert materials and shapes have been developed by the pipe manufacturers, Bureau of Public Roads, and the Federal Highway Administration. The nomographs are presented in **Figures 1008** to **1011**. Additional nomographs are available in HDS No. 5. When rating a culvert, either the outlet control nomographs or **Equation 1005** can be used to calculate the headwater requirements.

When using the outlet nomographs for corrugated steel 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	=	$L (n^1 / n)^2$
L^1	=	Equivalent Length
L	=	Actual Length
n	=	Value of Manning's "n" Value Shown on **Figures 1008** to **1011**
n^1	=	Actual "n" Value of Culvert

The actual n-value of the culvert can be obtained from **Table 1001.**

1003.3 Hydraulic Data

The hydraulic data provided in **Table 1001** shall be used in the hydraulic design of all culverts within the District. The design capacity of culverts shall be calculated using the computation sheet provided as **Standard Form 7**.

1003.4 Inlet and Outlet Configuration

Culverts are to be designed with protection at the inlet and outlet areas. The culvert inlet shall typically include a headwall with wingwalls or a flared end-section.

The outlet area shall also typically include a headwall with wingwalls or a flared end-section in addition to the riprap protection as defined in Section 707.4. Where outlet velocities exceed the limitation set forth in Section 1002.3, the energy dissipator shall be required .

1003.5 Structural Design

All culverts shall be designed as a minimum to withstand an H-20 loading in accordance with the design procedures of AASHTO "Standard Specifications for Highway Bridges" and with the pipe manufacturer's recommendations. At least 12 inches of cover is recommended.

1004 DESIGN STANDARDS FOR BRIDGES

All bridges shall be in accordance with "Standard Specifications for Highway Bridges" by AASHTO and "Standard Plans for Road and Bridge Construction" by the State of Nevada Department of Transportation. Hydraulic design and analysis shall be in accordance with the following criteria.

1004.1 Bridge Sizing Criteria

All bridges within the CCRFCD shall be designed to pass the 100-year design flow. Additionally, the design water surface elevation within the bridge shall be a minimum of 2 feet below the bridge low chord. Additional freeboard may be required for special hydraulic conditions. In special flood hazard areas, the bridge shall not back up the 100-year storm flow greater than 1 foot above the natural water surface elevation without mitigation measures. The designer must also ensure that no adjacent properties are adversely affected.

1004.2 Velocity Limitations

The velocity limitations through the bridge opening are controlled by the potential abutment scour and subsequent erosion protection provided. Using the regular riprap (defined in the STANDARD SPECIFICATIONS) for the channel lining and/or protection of the abutments and wingwalls (see Section 707.4), the maximum channel velocity is between 15 to 20 fps depending on channel slope. For consistency with culvert design and as a practical limit on the flow energy, a maximum velocity of 15 fps shall be allowed through a bridge, unless the bridge is designed and constructed in conjunction with the channels.

1005 BRIDGE HYDRAULICS

1005.1 Hydraulic Analysis

The procedures for analysis and design as outlined in the publication "Hydraulics of Bridge Waterways" (USDOT, 1978) shall be used for the hydraulic design of all bridges in the CCRFCD. This analysis shall be supplemented by an appropriate backwater analysis (see Section 702) to verify the resulting hydraulic performance.

1005.2 Inlet and Outlet Configuration

The design of all bridges shall include adequate wingwalls of sufficient length to prevent 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.

1006 EXAMPLE APPLICATION

The procedure to evaluate existing and proposed culverts within the CCRFCD is based on the procedures presented above. The methodology consists of evaluating the culvert headwater requirements assuming both inlet control

(**Figures 1004** to **1007**) and outlet control (**Figures 1008** to **1011**). The rating which results in the larger headwater requirements is the governing flow condition.

1006.1 Example: Culvert Sizing

Problem: A sample calculation for rating an existing culvert is presented in **Table 1002.** The required data are as follows:

Culvert size, length, and type (48 in CMP, L = 150 ft)

Inlet and outlet elevation, and slope (5540.0, 5535.5, S_o = 0.030)

Inlet treatment (flared end-section)

Low point elevation of embankment (EL = 5551.9)
Tailwater rating curve (see **Table 1102**, Column 6)

Solution: From the above data, the entrance loss coefficient, K_e, and the "n" value are determined. The full flow Q and the velocity are calculated for comparison. The rating then proceeds in the following sequence:

Step 1: Headwater values are selected and entered in Column 4. The headwater to pipe diameter ratio (HW/D) is calculated and entered in Column 3. If the culvert is other than circular, the height of the culvert is used.

Step 2: For the HW/D ratios, the culvert capacity is read from the rating curves (Section 1003.1) and entered into Column 1. This completes the inlet condition rating.

Step 3: For outlet condition, the Q values in Column 1 are used to determine the head values (H) in Column 5 from the appropriate outlet rating curves (Section 1003.2).

Step 4: The tailwater depths (T_w) are entered into Column 6 for the corresponding Q values in Column 1 according to the tailwater rating curve (i.e., downstream channel rating computations). If the tailwater depth (T_w) is less than the diameter of the cu(D), Columns 7 and 8 are to be calculated (go to Step 5). If T_w is more than D, the tailwater values in Column 6 are entered into Column 9 for the h_o values, and proceed to Step 6.

Step 5: The critical depth (d_c) for the corresponding Q values in Column 1 are entered in Column 7. The average of the critical depth and the culvert diameter is calculated and entered in Column 8 as the h_o values.

Step 6: The headwater values (H_w) are calculated according to the equation:

$$H_w = H + h_o - LS_o$$

where H is from Column 5, and h is from Column 9 (for T > D) or the larger value between Column 6 and Column 8 (for T_w < D). The values are entered into Column 10.

Step 7: The final step is to compare the headwater requirements (columns 10 and 4) and to record the type of control in Column 11, depending upon which case gives the higher headwater requirements. The headwater elevation is calculated by adding the controlling H_w to the upstream invert elevation. A culvert rating curve can then be plotted from the values in Columns 12 and 1.

Step 8: Compute the outlet velocity of the culvert for flow rate in Column 1 and record in Column 13. This velocity is used for sizing of outlet protection. Please note that for submerged outlets, the computed velocity and corresponding flow rate may not be the controlling velocity and flow rate for outlet protection design. A range of flow rates and corresponding outlet velocities should be checked to determine the controlling design condition.

To size a culvert crossing, the same form can be used with some variations in the basic procedures. First, a design capacity is selected and the maximum allowable headwater is determined. An inlet type (i.e., headwall) is selected, and the invert elevations and culvert slope are estimated based upon site constraints. A culvert type is then selected and first rated for inlet control and then for outlet control. If the controlling headwater exceeds the maximum allowable headwater, a different culvert configuration is selected and the procedure repeated until the desired results are achieved.

The criteria are considered a minimum design standard and must be modified where other factors are considered more important. For instance, if the procedure still results in certain structures remaining in the 100-year floodplain, the culvert may be increased to lower the water surface elevation. Also, if only a small increase in culvert size is required to prevent overtopping, then the larger culvert is recommended.

HYDRAULIC DATA FOR CULVERTS

(A) Manning's n-values for Corrugated Steel Pipe

Corrugations	Annular 2⅔" x ½"	Helical						
		1½" x ¼"[11, 12]		2⅔" x ½"				
	All Diam.	8"	10"	12"	18"	24"	36"	48"
Unpaved	.024	.012	.014	.011	.014	.016	.019	.020
25% Paved	.021					.015	.017	.020
Fully Paved	.012					.012	.012	.012

Corrugations	Annular 3" x 1"	Helical—3" x 1"					
	All Diam.	36"	48"	54"	60"	66"	72"
Unpaved	.027	.021	.023	.023	.024	.025	.026
25% Paved	.023	.019	.020	.020	.021	.022	.022
Fully Paved	.012	.012	.012	.012	.012	.012	.012

(B) Manning's n-values for Structural Plate Metal Pipe

Corrugations 6" x 2"	Diameters			
	5 ft	7 ft	10 ft	15 ft
Plain—unpaved	.033	.032	.030	.028
25% Paved	.028	.027	.026	.024

(C) Manning's n-values for Concrete Pipe/Culvert

TYPE	n-VALUE
Pre-Cast	0.012
Cast-in-Place	—
With Steel Forms	0.013
With Wood Forms	0.015

Revision	Date

REFERENCE: Handbook of Steel Drainage and Highway Construction Products, AISI, 1971 (with modifications)

HYDRAULIC DATA FOR CULVERTS

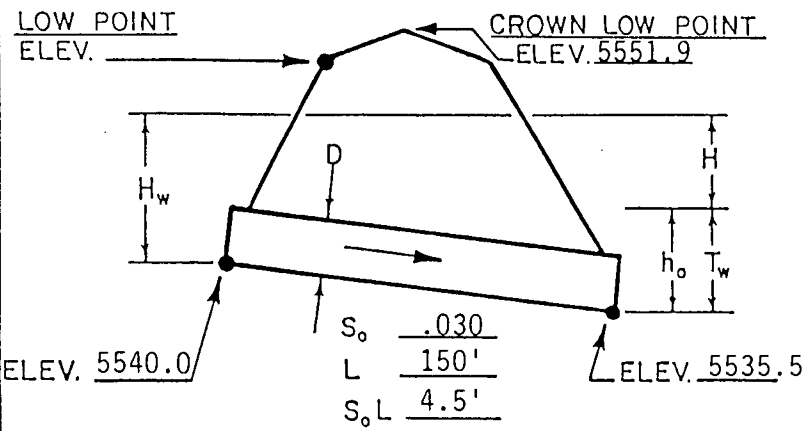
(D) CULVERT ENTRANCE LOSSES

Type of Entrance	Entrance Coefficient, Ke
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
Square 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 edged at crown	0.40
Crown edge rounded to radius of 1/12 barrel dimension	0.20
Wingwalls at 10° to 30° to barrel	
Square edged at crown	0.50
Wingwalls parallel (extension of sides)	
Square edged at crown	0.70

NOTE: The entrance loss coefficients are used to evaluate the culvert or sewer capacity operating under outlet control.

CULVERT RATING FOR EXAMPLE IN SECTION 1006.1

PROJECT: EXAMPLE LOCATION: CLARK COUNTY STATION: 1 + 00



CULVERT DATA

TYPE: 48" CMP n: .024
INLET: F.E.S. Q_{FULL}: 135
K_e: 0.5 V_{FULL}: 10.7

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)
$T_w \geq D$; $h_o = T_w$

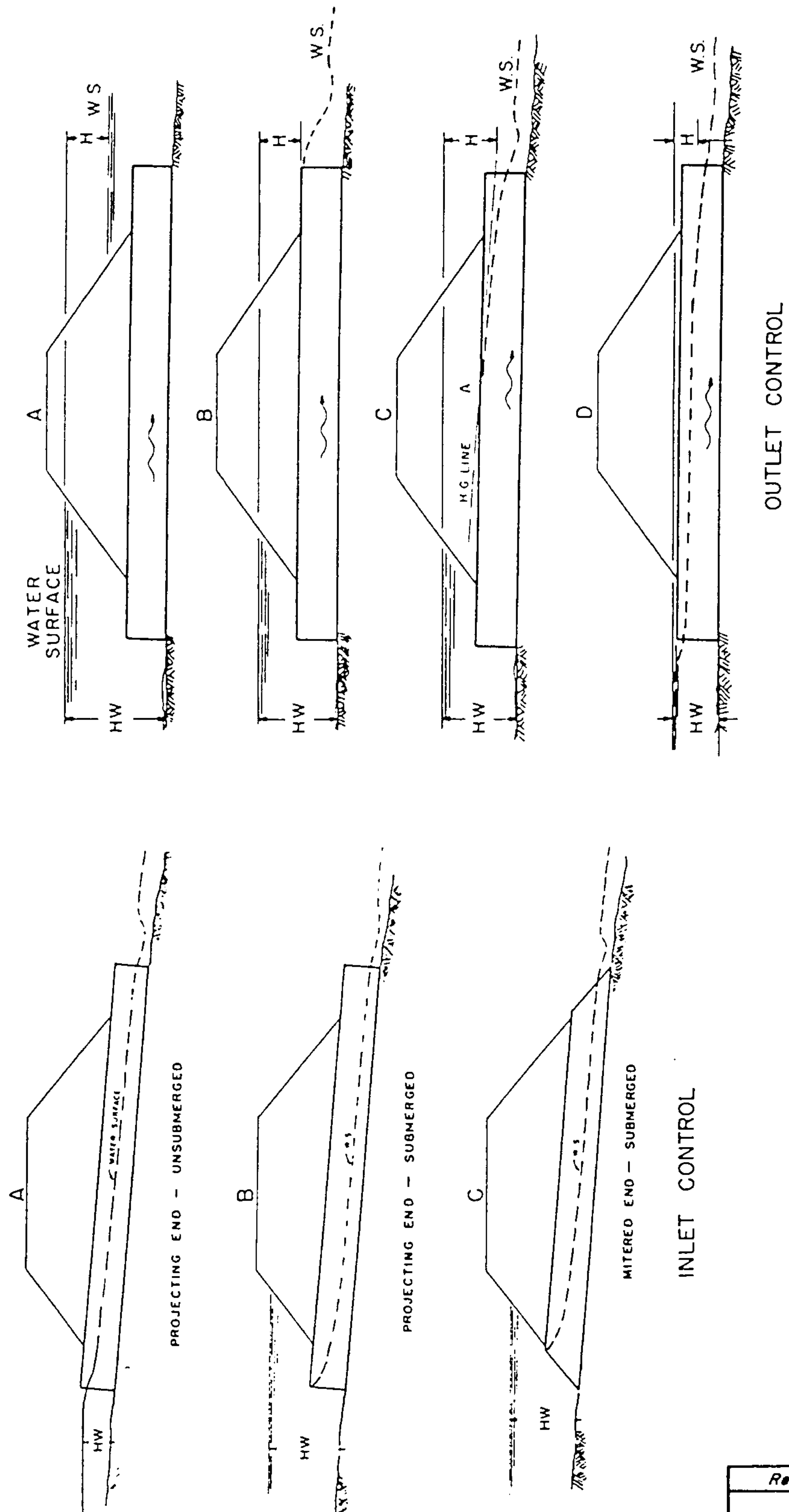
(3) For Box Culvert: $d_c = 0.315(Q/B)^{2/3} \leq D$

	Q	STORM EVENT	INLET CONTROL $\frac{H_w}{D}$	INLET CONTROL H_w	OUTLET CONTROL H	OUTLET CONTROL T_w	OUTLET CONTROL $T_w < D$: d_c	OUTLET CONTROL $T_w < D$: $\frac{d_c + D}{2} = h$	OUTLET CONTROL $T_w \geq D$: h_o	OUTLET CONTROL H_w	TYPE OF CONTROL	CONTROL HEADWATER ELEVATION	OUTLET VELOCITY
	1	2	3	4	5	6	7	8	9	10	11	12	13
	70		1.0	4	1.9	1.5	2.5	3.3		0.7	Inlet	5544.0	10.8
	115		1.5	6	5.5	2.0	3.0	3.5		4.5	Inlet	5546.0	12.0
	145		2.0	8	8.9	2.5	3.4	3.7		8.1	Outlet	5548.0	11.5
(1)	170		2.5	10	12.5	3.0	3.7	3.9		11.9	Outlet	5551.9	13.5
(2)	195		3.0	12	16.0	3.5	4.0	4.0		15.5	Outlet	5555.5	15.5

NOTES: (1) Culvert capacity without road overtopping.
(2) Excessive road overtopping depth (3.6').

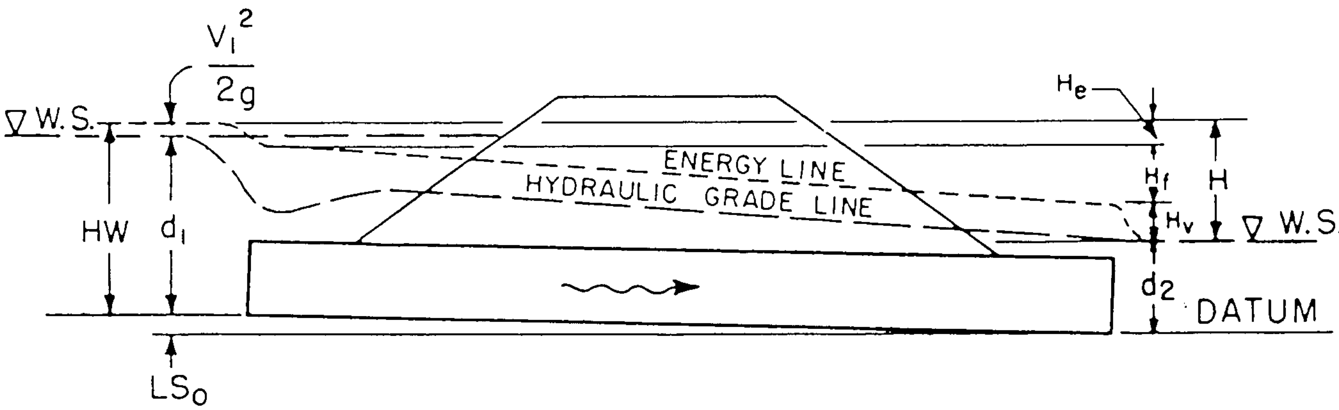
Revision	Date

CULVERT FLOW TYPES



Revision	Date

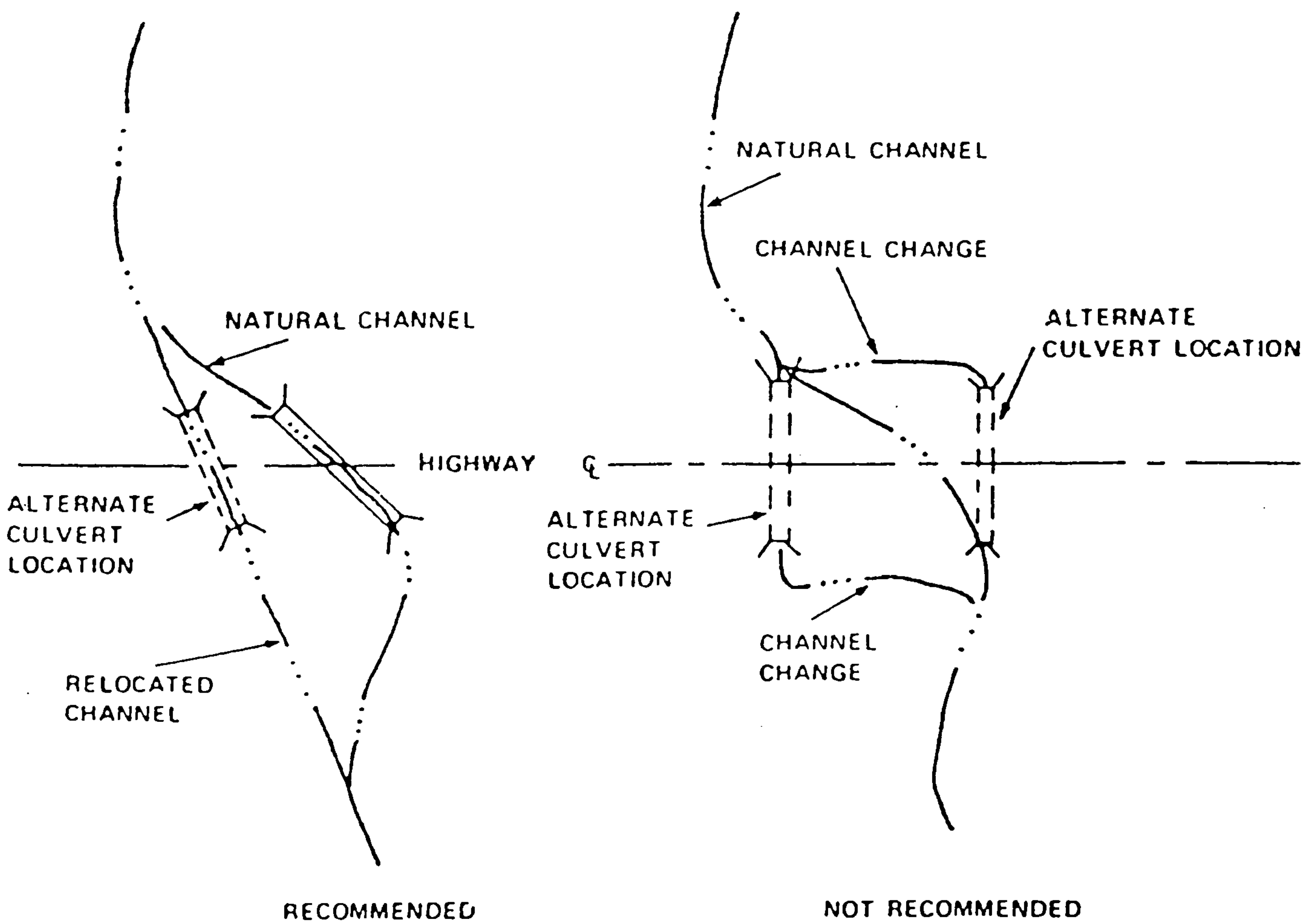
OUTLET CONTROL CULVERT HYDRAULICS



Revision	Date

REFERENCE:
USDOT, FHWA, HDS No. 5, 1985

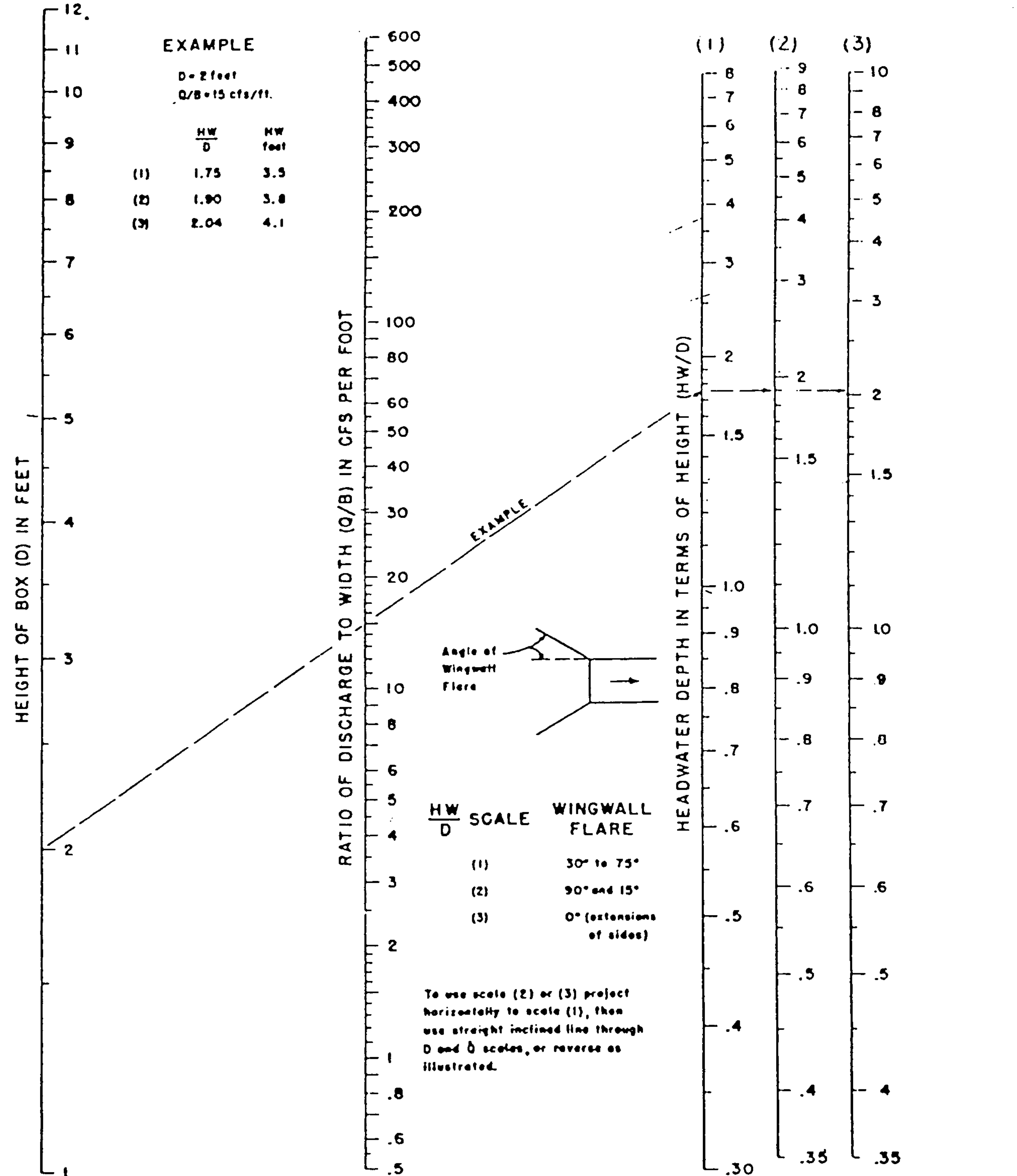
CULVERT ALIGNMENT



Revision	Date

REFERENCE: Nevada Department of Transportation, Design Manual, Road Design Division, 1975

NOMOGRAPH - INLET CONTROL BOX CULVERT



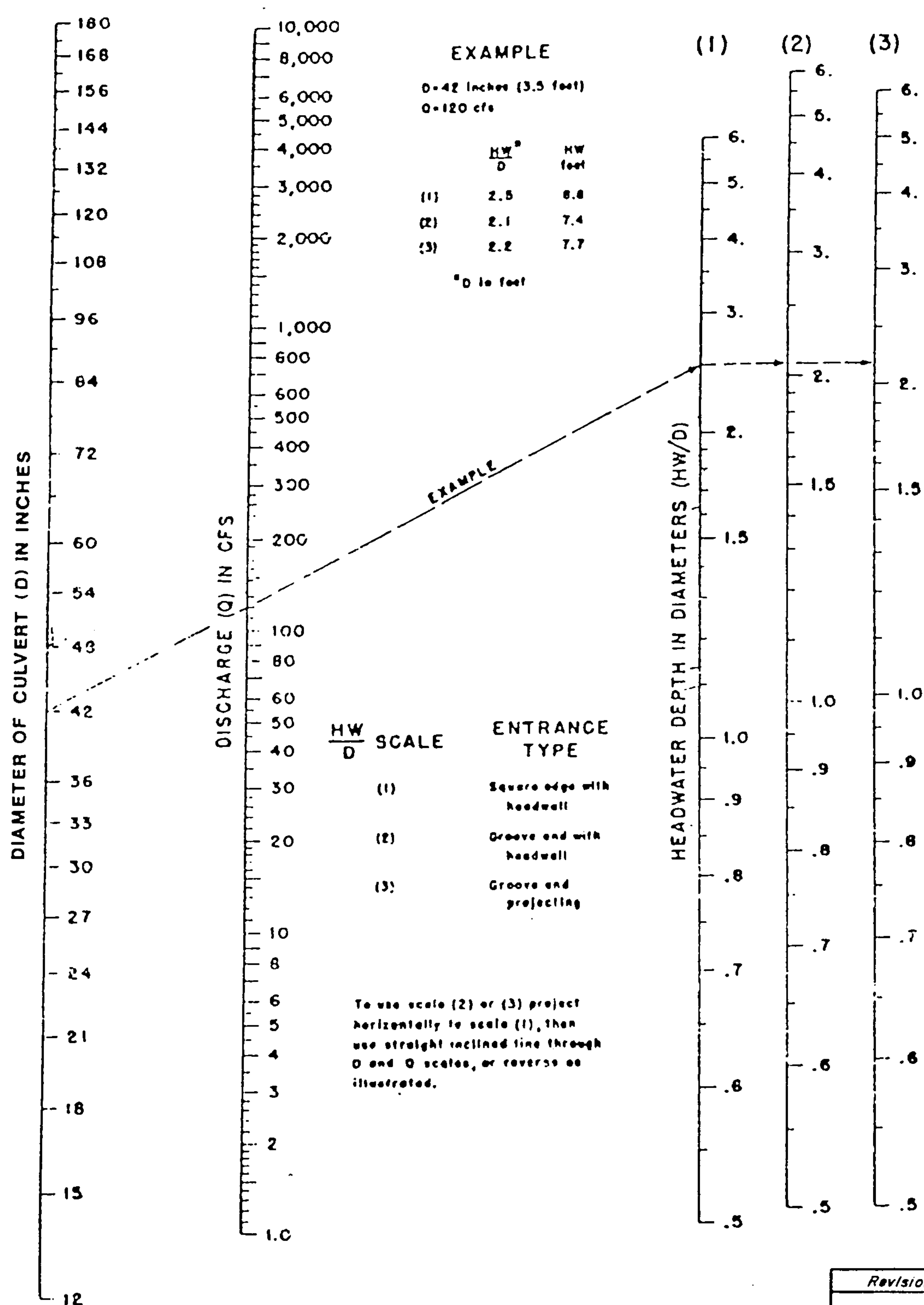
Revision	Date

WRC ENGINEERING

REFERENCE:
USDOT, FHWA, HDS No.5, 1985

FIGURE 1004

NOMOGRAPH - INLET CONTROL RCP



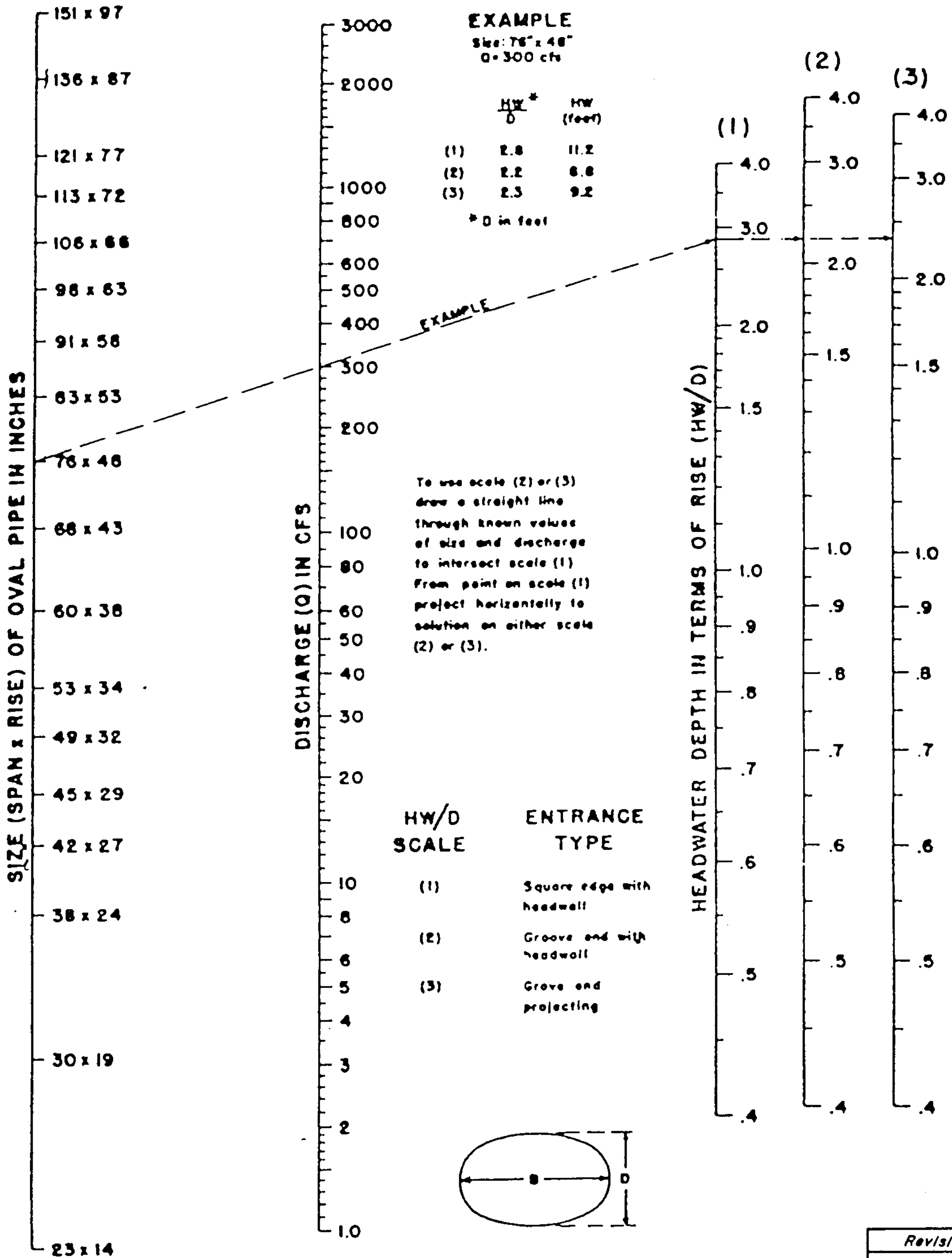
Revision	Date

WRC ENGINEERING

REFERENCE:
USDOT, FHWA, HDS, No. 5, 1985

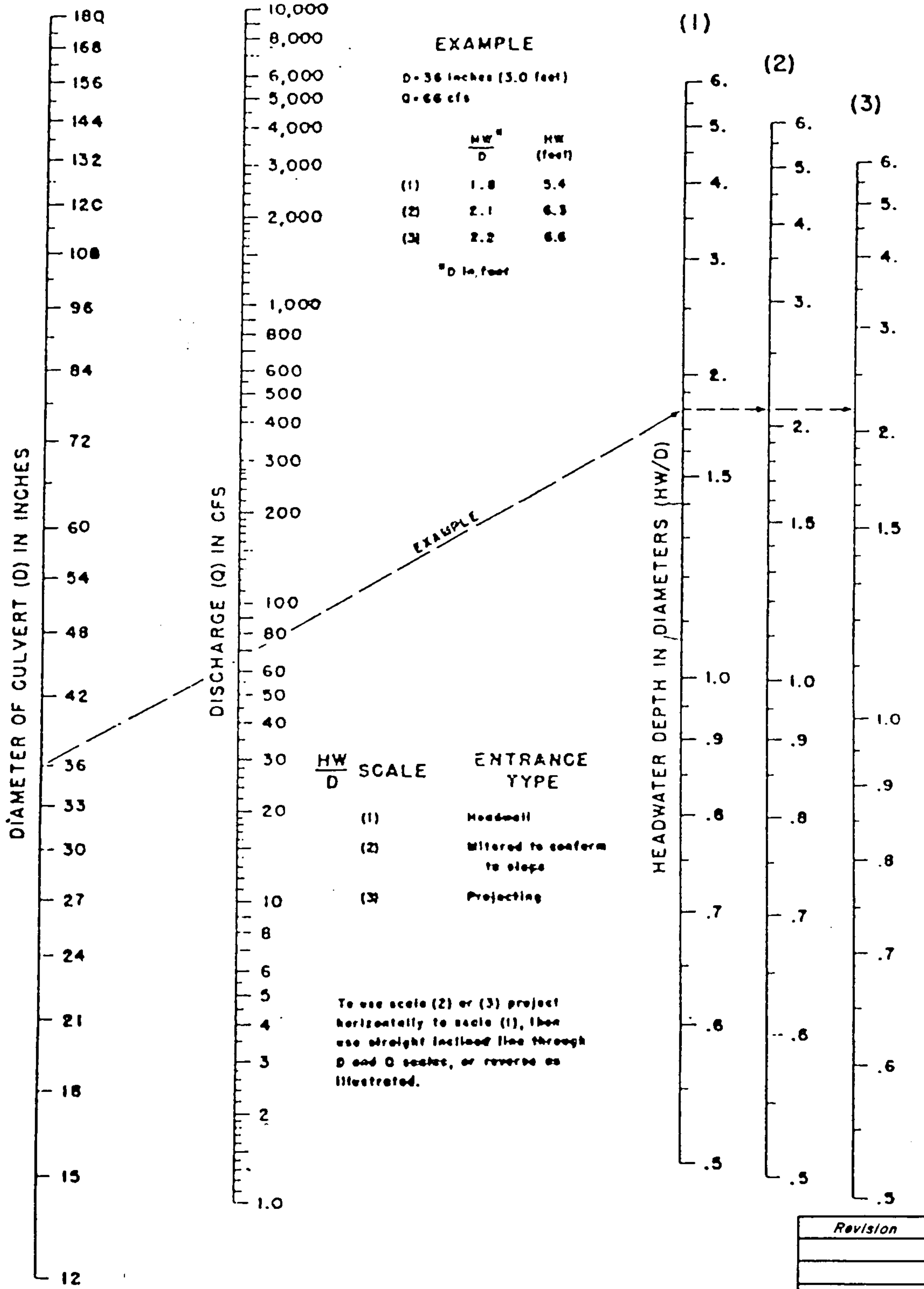
FIGURE 1005

NOMOGRAPH – INLET CONTROL ELLIPTICAL PIPE



Revision	Date

NOMOGRAPH - INLET CONTROL CMP



Revision	Date

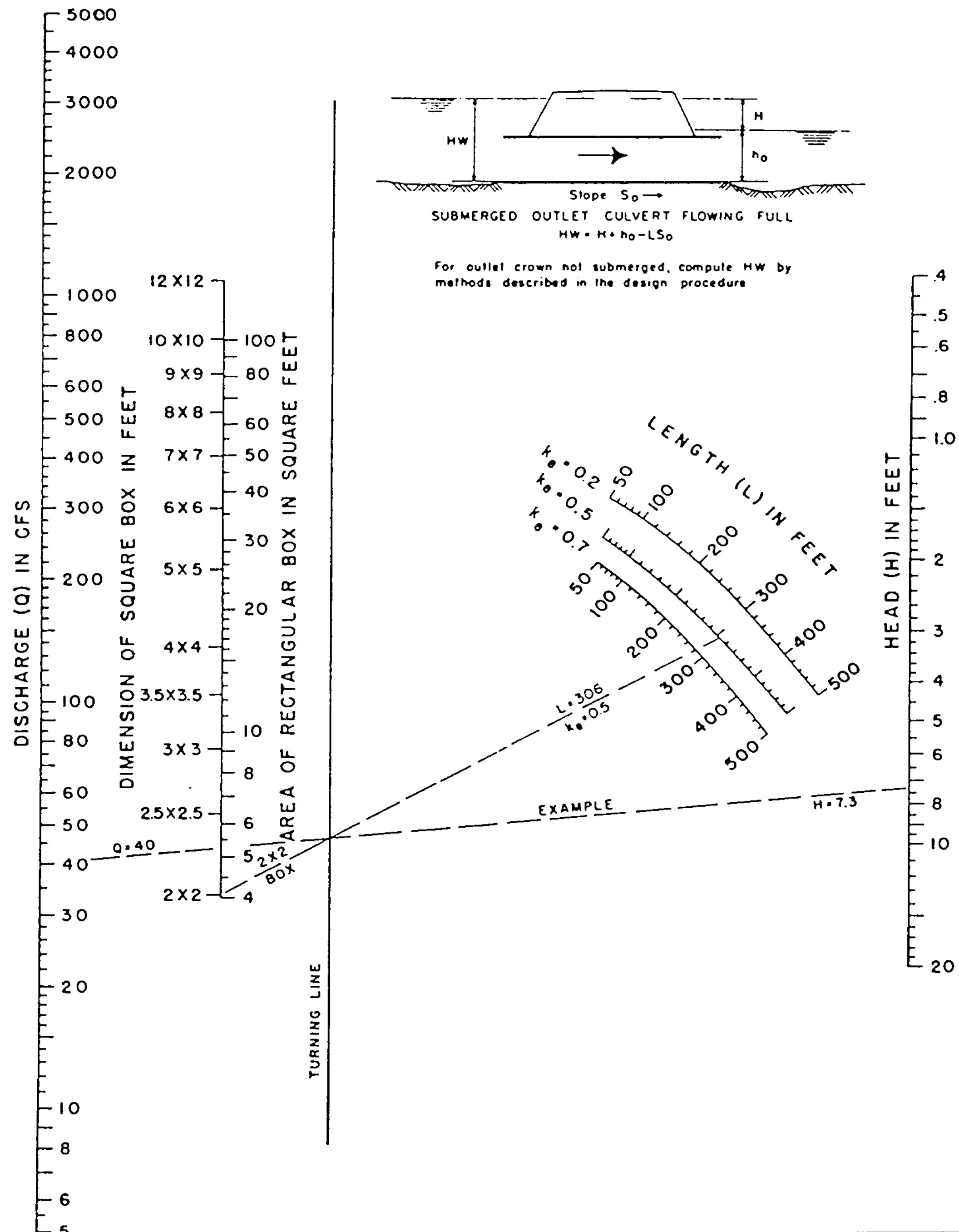
WRC ENGINEERING

REFERENCE:
USDOT, FHWA, HDS No. 5, 1985

FIGURE 1007

NOMOGRAPH - OUTLET CONTROL BOX CULVERT

(n=0.012)

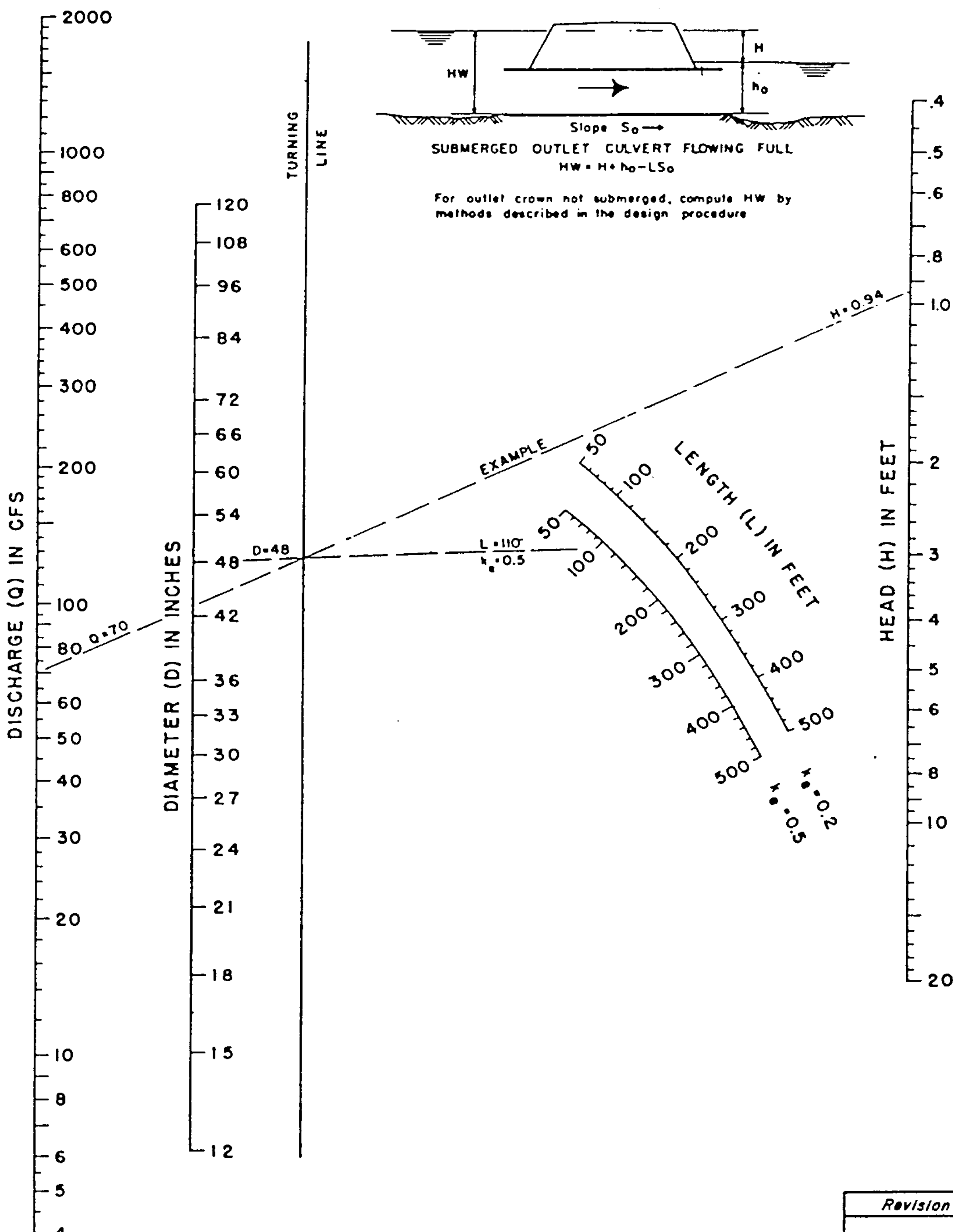


Revision	Date

REFERENCE:
USDOT, FHWA, HDS No. 5, 1985

NOMOGRAPH - OUTLET CONTROL RCP

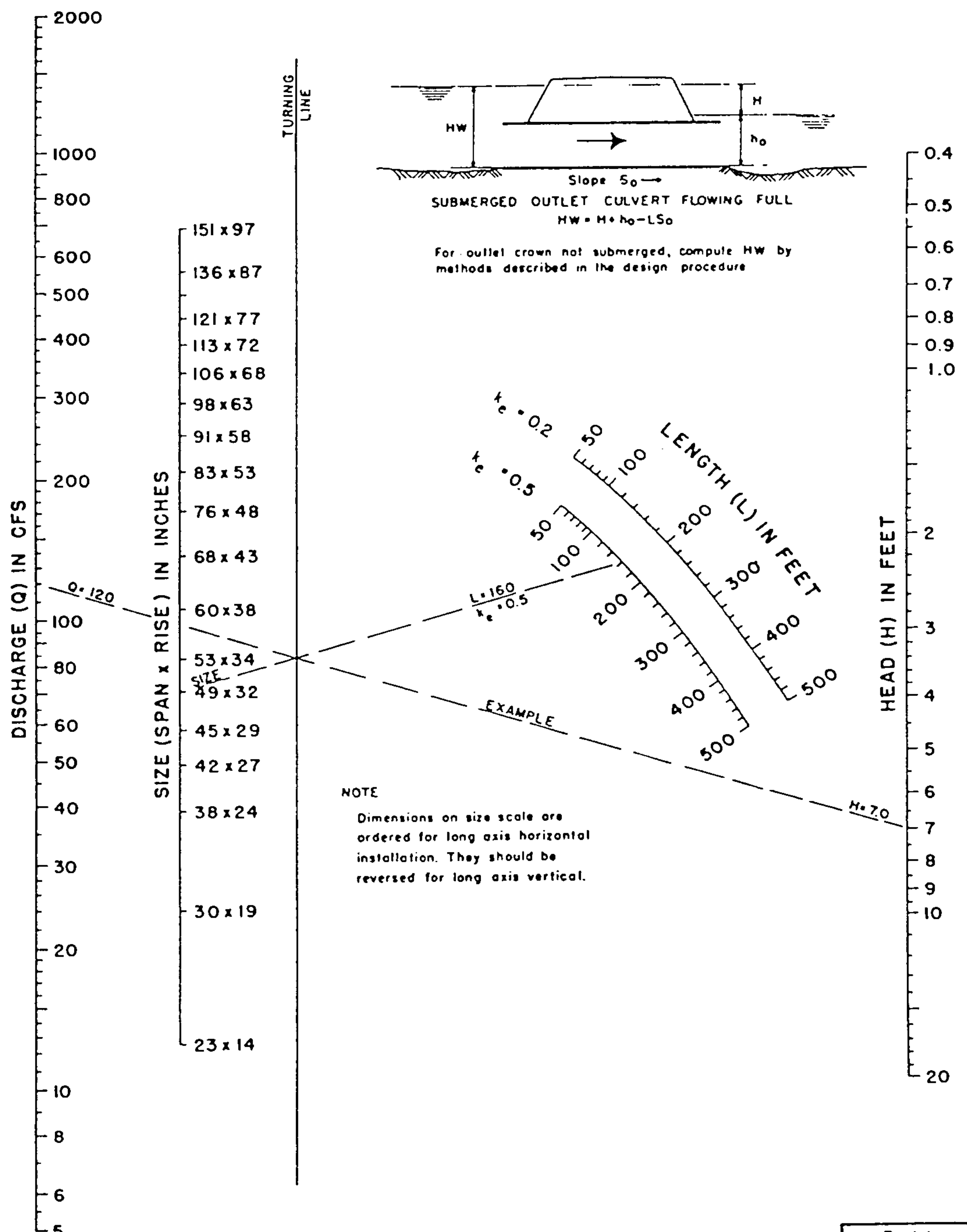
(n=0.012)



Revision	Date

NOMOGRAPH - OUTLET CONTROL ELLIPTICAL PIPE

(n=0.012)



Revision	Date

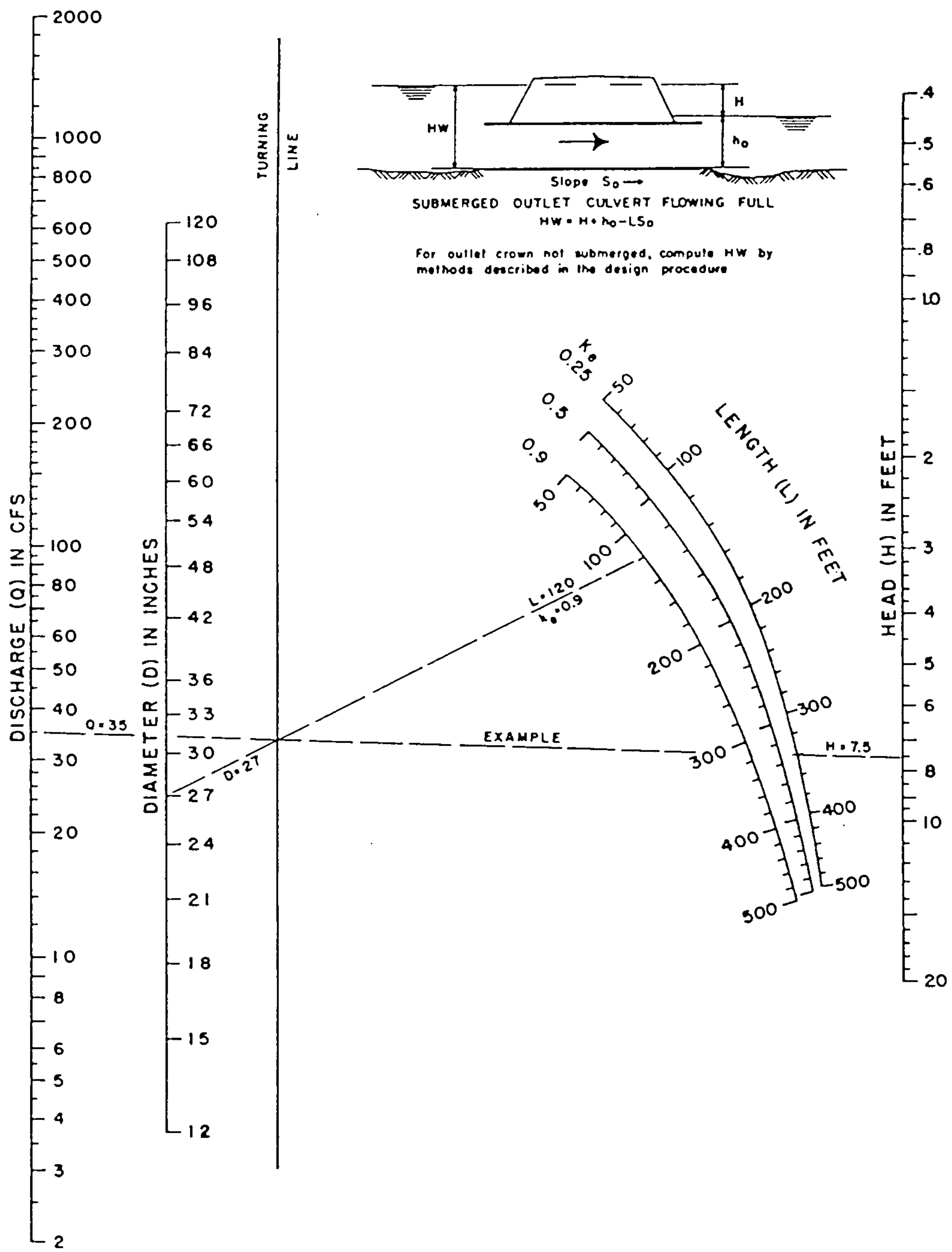
WRC ENGINEERING

REFERENCE:
USDOT, FHWA, HDS No. 5, 1985

FIGURE 1010

NOMOGRAPH - OUTLET CONTROL CMP

(n=0.024)



Revision	Date

WRC ENGINEERING

REFERENCE:
USDOT, FHWA, HDS No. 5, 1985

FIGURE 1011

CLARK COUNTY REGIONAL FLOOD CONTROL DISTRICT
HYDROLOGIC CRITERIA AND DRAINAGE DESIGN MANUAL

SECTION 1100
ADDITIONAL HYDRAULIC STRUCTURES

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Section 1100
Additional Hydraulic Structures

1101 INTRODUCTION

Presented in this section are design guidelines and standards for hydraulic structures which are appurtenant to both storm sewer outlet and open channel design. These guidelines and standards are generalized since each structure is unique, with the possible exception of channel drops. The user is encouraged to coordinate with the local entity and/or the CCRFCD when planning and designing these types of hydraulic structures.

1102 CHANNEL DROPS AND ENERGY DISSIPATION STRUCTURES

The design of open channels often require the use of channel drop and/or energy dissipation structures to dissipate excess energy created by gravity acting on the storm water flow. The most common use of these structures is to control the longitudinal slope of channels to keep design velocities within acceptable limits (Section 700). These structures are also used to dissipate excess energy at storm sewer outlets and to safely lower flood flow elevations at abrupt drops in existing topography.

For the purposes of this MANUAL, channel drop and energy dissipation structures are classified into two groups. Channel drops are classified as structures which shall only be used when the inflow channel flow is sub-critical (Froude Number, $F_r < 0.86$). Energy dissipators and stilling basins, are classified as structures which may be used for either sub-critical ($F_r < 0.86$) or super-critical ($F_r > 1.13$) inflow conditions.

Presented in **Table 1101** is a listing of the structures discussed in this section along with the hydraulic limitations under which these structures are allowed to be used within the Clark County area. The designer must obtain prior approval from the local entity to use any of the listed structures outside of the stated limits. Also, if the designer desires to use a structure not discussed in the section, pertinent detailed information on said structure must be submitted to the local entity for review and approval prior to designing the facility.

Criteria and charts to aid in the design of these types of structures have been developed based on many various hydraulic studies. Generalized standards for each type of channel drop based on these hydraulic studies are provided in the following sections.

The reader should refer to the standard channel drop and energy dissipation design references to become familiar with the detailed information available on each structure prior to design. Suggested references include Peterka, 1978; USBR, 1987; and USACE, 1970.

1102.1 Channel Drop Structures

Presented in **Table 1101** are the types of channel drop structures allowed in the Clark County area. By definition, channel drop structures are to be used only when the upstream channel flow is sub-critical. Presented in **Figure 1101** are the generalized profiles and nomenclature for riprap drop structures. This nomenclature is used throughout this section for discussion of specific standards for each part of the structure. The nomenclature is also applicable to gabion drop structures.

1102.1.1 Sloping Riprap Drop Structures

Presented in **Table 1102** and **Figure 1102** are the design standards and details for sloping riprap drop structures. The design chart for sloping riprap drop structures is based upon the unit discharge (q) of the approach channel, the riprap classification and the slope of the drop structure, and is valid only for sub-critical flow in the approach channel (i.e., Froude Number (F,) < 0.86). The unit discharge is found by taking the average or normal channel velocity (V,) for the 100-year discharge times the normal depth of the channel (Y_n).

The design chart is also based upon a prismatic channel section throughout, from the upstream channel through the drop to the downstream channel. The maximum (steepest) allowable side slope for the riprap lined channel within the drop structure is 4:1. Flatter side slopes are allowable and encouraged when available ROW permits.

The classification of riprap chosen for the sloping portion of the structure should be used throughout the structure, including the upstream and downstream aprons, the channel bottom and side slopes. See Section 700 for riprap classification. The riprap should extend up the side slopes to a depth equal to 1 foot above the normal major storm flow depth projected upstream from the downstream channel, or 1 foot above the critical depth in the sloping section, whichever is greater **(see Figure 1102).** The maximum fall allowed at any one drop structure is 3 feet from the upper channel bottom to the lower channel bottom, excluding the trickle channel.

A detailed description of the drop structure and the design procedure proceeding from upstream to downstream is given below based on **Figure 1102.**

1102.1.1.1 <u>Criteria</u>

a. <u>Approach Depth:</u> The upstream and downstream channels will normally be trapezoidal sections with low flow channels to convey normal low water flows. The maximum normal depth, Y_n, is 5 feet and the maximum normal velocity, V,, is 7 fps.

b. Low Flow Channel: The low flow channel shown in this case is a rectangular concrete channel. The concrete channel ends at the upstream end of the upstream riprap apron. A combination cut-off wall and foundation wall is provided to give the end of the low flow channel additional support. The water is allowed to “trickle” through the upstream apron and through the crest wall (discussed below). Riprap low flow channels would simply feather into the upstream apron.

c. Approach Apron: A IO-foot long riprap apron is provided upstream of the cutoff wall to protect against the increasing velocities and turbulence which result as the water approaches the sloping portion of the drop structure. The same riprap design and bedding should be used as specified for the portion of the drop structure downstream of the cut-off wall.

d. Crest Wall: The crest wall is a very important part of the drop structure, and has several purposes, one of which is to provide a level rigid boundary section and distribute the flow evenly over the entire width of the structure. This is extremely important since the selection of the riprap is based upon the unit discharge, and without the wall, flow concentrations could result which would greatly exceed the design discharge. The crest wall is also used to reduce or eliminate seepage and piping along with the failures which can result from these problems.

The low flow channel is ended at the upstream end of the upstream apron to prevent the low flow channel from concentrating additional water at a point during high flows, thus exceeding the design unit discharge. The apron and the crest wall combine to disperse the concentrated flow. The low flows must be allowed through the crest wall to prevent ponding. A series of notches in the wall will allow the low flows to do this. The size and number of notches will depend on the design discharge of the low flow channel. Note that they are offset from the trickle channel to permit flow of water through the upstream apron. The voids in the riprap below the notch inverts are expected to silt in rapidly or they can be filled at the time of construction.

The two most common types of walls used will be reinforced concrete or sheet pile. The design of the wall is a structural problem which will not be addressed here. The depth of the wall should be at least to the bottom of the bedding material and could be deeper if necessary for the control of piping.

The top of the crest wall should be placed a distance P above the upstream channel bottom. This is done to create a higher water surface elevation upstream, thus reducing the drawdown effects normally caused by a drop structure. P can be determined from **Table 1104** and is not considered in the total allowable vertical drop.

e. Chute Apron: The allowable size of riprap and chute slope for the chute portion of the drop structure and the downstream apron are obtained from **Table 1102.** The riprap size and chute slope are determined from the table by first computing $q = V_n Y_n$. Next, enter the table at the proper value of q in the left-hand column. Then, determine the allowable slopes in the row for that q and select the best combination of riprap classification and slope using site and cost considerations.

The length of the downstream apron L_B and the depth of the riprap D_R can also be obtained from **Table 1102.** The riprap must be placed on bedding as shown in **Figure 1102.** The term "bedding" used in this section (Section 1100) refers to 6-inch Type II Aggregate Base as specified in the STANDARD SPECIFICATIONS. The 2 foot long filter fabric cutoffs help prevent piping failures. The riprap should extend up the side slopes a distance of Y_n + 1 foot as projected from the downstream channel or the critical depth plus 1 foot, whichever is greater. The side slopes for the chute and downstream apron should be the same as the crest wall and upstream channel with the exception that a riprap slope as steep as 2:1 can be used starting above the height of the riprap lining required above. The thickness of the riprap immediately downstream of the crest wall should be increased to D_{RW} as shown in **Table 1102.** This extra thickness is necessary to protect the most critical area of the structure. The voids in the apron can be filled during construction to reduce ponding of low flows in the apron area.

f. Exit Depth: The downstream channel should be the same as the upstream channel, including a low flow channel. The low flow channel invert must be below the top of the adjacent riprap section to ensure that low flows will drain into the low flow channel. For concrete low flow channels a foundation wall similar to the one used for the upstream low flow channel should be used. In some instances the wall may also be used to control seepage and piping.

A design example for a sloping riprap drop structure is presented in Section 1102.3.1.

1102.1.2 Vertical Riprap Drop Structures

Presented in **Table 1103** and on **Figure 1103** are the design standards and details for vertical riprap drop structures.

The design chart for the vertical channel drop structures is based upon the height of the drop and the normal depth and velocity of the approach and exit channels. The channel must be prismatic throughout, from the upstream channel through the drop to the downstream channel.

The maximum (steepest) allowable side slope for the riprap stilling basin is 4:1. Flatter side slopes are allowable and encouraged when available ROW permits. The riprap should extend up the side slopes to a depth equal to 1 foot above the normal depth projected upstream from the downstream channel. **(See Figure 1103).** The maximum fall allowed at any one drop structure is 3 feet from the upper channel bottom to the lower channel bottom, excluding the low flow channel.

A detailed description of the drop structure and the design procedure from upstream to downstream is given below and is presented on **Figure 1103.**

a. Approach Depth: The upstream and downstream channels will normally be trapezoidal channels with trickle channels to convey normal low water flows. The maximum normal depth, Y,, is 5 feet and the maximum normal velocity, V,, is 7 fps.

b. Low Flow Channel: The low flow channel shown in this case is a rectangular concrete channel. The concrete channel ends at the upstream end of the upstream riprap apron. A combination cut-off wall and foundation wall, to give the end of the low flow channel additional support, is provided. The water is allowed to “trickle” through the upstream apron and through the vertical wall. Riprap low flow channels would simply feather into the upstream apron.

c. Approach Apron: A lo-foot long apron is provided upstream of the cutoff wall to protect against the increasing velocities and turbulence which result as the water approaches the vertical drop. Heavy riprap should be used for this apron.

d. Crest Wall: The vertical wall should have the same trapezoidal shape as the approach channel. The wall distributes the flow evenly over the entire width of the drop structure. This is important to prevent flow concentrations which would adversely affect the riprap basin.

   The low flow channel is ended at the upstream end of the upstream apron to prevent the low flows from concentrating additional water at a point during high flows, thus exceeding the design assumptions. The apron and the vertical wall combine to disperse the flow concentrated in the low flow channel. The low flows are allowed to trickle through the , wall through a series of notches in order to prevent ponding. The voids in the riprap below the notch inverts are expected to silt in rapidly, or they can be filled at the time of construction.

The wall must be designed as a structural retaining wall. The top of the wall should be placed a distance P above the upstream channel bottom. This is done to create a higher water surface elevation upstream, thus reducing the drawdown effects normally caused by a sudden drop. P can be determined from **Table 1104.**

e. <u>Chute Apron:</u> The riprap stilling basin is designed to force the hydraulic jump to occur within the basin, and is designed for essentially zero scour. The floor of the basin is depressed an amount B below the downstream channel bottom, excluding the trickle channel. This is done to create a deeper downstream sequent depth which helps keep the hydraulic jump in the basin. This arrangement will cause ponding in the basin. The trickle channel can, depending on the depth, relieve all or some of the ponding. The riprap can also be buried and vegetated to reduce the ponded area to a smaller size.

The riprap basin is sized using **Table** 1103. To use the table, the designer must first determine the necessary height of the drop C, the normal velocity of the approach channel V,, and the upstream and downstream normal depths Y_n and Y,. Both channels must have the same geometry and Y_2 must be equal to Y,. Designs for drops when $Y_n \neq Y_2$ shall be discussed with the local entity and/or the CCRFCD prior to design. Enter the row which contains the correct C, V,, Y_n, and Y_2 and select the riprap classification and necessary channel drop dimensions from that row.

The riprap must be placed on bedding and filter fabric as shown in **Figure 1103.** The riprap should extend up the channel side slopes a distance of Y_2 + 1 foot as projected from the downstream channel. The basin side slopes should be the same as those in the downstream channel (4: 1 or flatter) up to the Y_2 + 1 foot location, above which riprap slopes as steep as 2:1 are allowed.

f. <u>Exit Depth:</u> The downstream channel should be the same as the upstream channel, including a low flow channel. For concrete low flow channels a foundation wall similar to the one used for the upstream trickle channel should be used. In some instances the wall may also be used to control seepage and piping.

A design example for a vertical riprap drop structure is presented in Section 1102.3.2.

1102.1.3 Gabion Drops

Gabion drops are classified into three principal drop types according to the slope of their downstream face:

a. Vertical type
b. Stepped type
c. Sloped type

Presented in **Figures 1104** through **1106** are examples of these three types of gabion drops.

Gabions are generally made in standard sizes ranging from 1 to 3 feet in thickness, 3 feet in width, and 6 to 12 feet in length. The number of cells in each standard gabion varies according to its dimensions, however, a cell should not be greater than 3 linear feet. Manufacturers of gabions should be consulted and their design information should be reviewed before planning and designing a gabion structure.

In constructing gabion drops, either PVC coated wire mesh gabion baskets or galvanized steel wire mesh gabion baskets are used. The designer shall obtain soil corrosion data for the specific site to determine which type of coating may be used or if the soil is too corrosive for either coating. The data and coating recommendations shall be submitted to the local entity for review and approval.

1102.1.3.1 Design Criteria:

The manufacturer suppling the gabions should supply design guidelines and criteria used in designing gabion drops. Specific reference is made in this MANUAL to MACCAFERRI, 1987. This reference outlines typical step by step procedures for design of gabion drops and thus typical procedures are not repeated in this MANUAL. The naming of MACCAFERRI should not be construed as an endorsement or acceptance of their products.

A few highlights of the design criteria are as follows:

a) Vertical Drops

1. Vertical drops are used for small drops.

2. The maximum design discharge appears to be 100 cfs per foot. The recommended unit discharge is 35 cfs per foot.

3. The drop structure should be designed according to the procedure specified in MACCAFERRI, 1987 or similar manufacturers guidelines.

b) Stepped Drops

1. Stepped drops shall only be used with light bed loads and with a maximum unit discharge of 35 cfs per foot.

2. Technical reasons prevent in most instances a rational design of stepped drops ensuring the formation of a hydraulic jump at the toe of each fall. It is advisable not to rely upon energy dissipation in each step.

3. The drop structure should be designed according to the procedure specified in MACCAFERRI, 1987 or similar manufacturers guidelines.

c) Sloped Drop

1. Sloped drops are used where, due to the poor quality of the foundation soil, a large foundation area and a fairly uniform pressure distribution are required.

2. The maximum design discharge recommended is 35 cfs per foot.

3. Design of the sloped drop should be based upon the procedures specified in MACCAFERRI, 1987 or similar manufacturers guidelines.

In addition. the following criteria should be considered in design of gabion drops:

a. Where possible small drops in series would be more desired than one large drop.

b. Counter weirs and stilling basins should be considered when scouring problems are present. When they are not used the foundation of the drop should be below the scouring ability of channel.

c. Structural stability should be checked for over turning and/or sliding.

d. In heavy debris areas the gabion drop crest should be capped with concrete to avoid debris blockage.

e. Design procedures may generally be obtained from gabion manufacturers free of charge or obligation.

1102.1.4 Straight Drop Spillways

Presented in **Figure 1107** are the design details for a straight drop spillway. The spillway produces a controlled overflow jet which is dissipated through impact on the structure floor and baffle blocks. The jet energy is also dissipated in the plunge pool created when impact blocks are used or through an hydraulic jump using the typical baffle block arrangements from the USBR stilling basin designs.

The basin design is based on the drop distance, Y, and the unit discharge, q, as related through the drop number, D, computed as follows:

$$D = q^2 / gY^3 \tag{1101}$$

where D = Drop Number
q = Unit Discharge (cfs / ft of width)
Y = Drop Distance (ft)

The remaining design parameters can be obtained from **Figure 1107**.

The impact block basin is applicable for low heads with a wide range of tailwater depths. The hydraulic jump basin may be used as long as the design parameters for the selected basin type are meet. The designer is referred to USBR, 1987, for detailed design information, guidelines, and examples.

1102.1.5 Baffled Aprons (USBR Type IX)

Presented on **Figure 1108** is the baffled apron stilling basin. This structure requires no initial tailwater to be effective, although when the tailwater forms a pool into which the flow discharges, the channel bed scour is not as deep and is less extensive. The chutes are constructed on an excavated slope, 2:1 or flatter, extending to below the channel bottom. Backfill is placed over one or more rows of baffles to restore the original streambed elevation. When scour or downstream channel degradation occurs, successive rows of baffle piers are exposed to prevent excessive acceleration of the flow entering the channel. If degradation does not occur the scour creates a stilling pool at the downstream end of the chute, stabilizing the scour pattern.

Generalized design information is presented in **Figure 1109**. The designer is referred to PETERKA, 1978 for detailed design information, guidelines, and examples.

1102.2 Energy Dissipation Structures

Presented in **Table 1101** are the types of energy dissipation structures allowed in the Clark County area. By definition, energy dissipation structures may be used for both sub-critical and super-critical upstream channel (or pipe) flow conditions. For sub-critical flow conditions, these structures are designed similar to the channel drop structures discussed in the previous section. For super-critical flow conditions, the upstream channel is tied directly into the stilling basin floor (hydraulic rise) or the upstream channel is transitioned into the structure through the use of a trajectory transition section. The hydraulic design of trajectory transition sections is discussed in Section 1102.2.9.

1102.2.1 Types of Energy Dissipation Structures

Many stilling basins and energy-dissipating devices have been designed in conjunction with spillways, outlet works, and canal structures, utilizing blocks, sills, or other roughness elements to impose exaggerated resistance to the flow. The type of stilling basin selected is based upon hydraulic requirements, available space and cost. The hydraulic jump which occurs in a stilling basin has distinctive characteristics depending on the energy of flow which must be dissipated in relation to the depth of the flow. A comprehensive series of tests have been performed by the USBR for determining the most efficient energy dissipators (PETERKA, 1978).

The energy dissipation structures discussed herein provide a wide range of structures from which to choose the most hydraulically and cost efficient structure. The reader is encouraged to review the analysis, results, and recommendation in PETERKA, 1978, prior to final selection of energy dissipation structure.

1102.2.2 Stilling Basins With Horizontal Sloping Aprons

The basis for design of all of the USBR stilling basins is analysis of the hydraulic jump characteristics on horizontal and sloping aprons. The governing equation for hydraulic jumps is based on pressure-momentum theory and may be written as follows:

$$D_2 / D_1 = 0.5\left(\left(1 + 8F^2_{r1}\right)^{0.5} - 1\right) \qquad (1102)$$

where D_1 = Depth of Flow at Jump Entrance (ft)
D_2 = Depth of Flow at Jump Exit ft)
F_{r1} = Froude Number at Jump Entrance

The results of the USBR analysis are presented in **Figure 1109**. In this figure T_w is the tailwater depth necessary to create or assist in forming the hydraulic jump. Generally, T_w is greater than D_2.

The above equation is generally used to determine the approximate location of a hydraulic jump in a channel. In practical application, the actual flow depths and location of the jump will vary due to inaccuracies in estimating actual flow parameters (i.e., channel roughness, flow characteristics). The location of the jump will also vary depending on the flow rate in the channel. Therefore, from a structural and safety standpoint, horizontal and sloping apron stilling basins should not be used as energy dissipation structures without the addition of appurtenances (i.e., baffle blocks, end sills) to control the location of the hydraulic jump. Standard designs for these types of structures are discussed in the following sections.

1102.2.3 Short Stilling Basin (USBR Type III)

Presented in **Figure 1110** is the standard design for a Type III stilling basin. The chute blocks at the upstream end of a basin tend to corrugate the jet, lifting a portion of it from the floor to create a greater number of energy dissipating eddies. These eddies result in a shorter length of jump than would be possible without them, and tend to stabilize the jump. The baffle piers act as an impact dissipation device and the end sill is for scour control. The end sill has little or no effect on the jump. The only purpose of the end sill in a stilling basin is to direct the remaining bottom currents upward and away from the channel bed.

This type of basin is recommended at the outlet of a sloping channel drop when there is adequate tailwater. For insufficient tailwater, a USBR Type VI basin is recommended.

1102.2.4 Low Froude Number Basins (USBR Type IV)

Presented in **Figure 1111** is the standard design of a low Froude number basin. The basin is used instead of the USBR Type II and Type III basins in order to achieve better jump characteristic at low Froude numbers ($2.5 < F_r < 4.5$). At these low Froude numbers, excess waves are created because the jump is not fully developed.

This basin minimizes the waves by directing jets from the tops of the baffle blocks into the roller to strengthen and intensify it. In addition, the tail water depth (T_w) should be at least 1.1 x D_2 (conjugate depth) to minimize the chance of the jump sweeping out of the basin (See **Figure 1109**). The end sill has little or no effect on the jump but rather directs the bottom currents upward and away from the channel bed.

1102.2.5 Impact Stilling Basin (USBR Type VI)

This stilling basin is an impact-type energy dissipator, contained in a relatively small box-like structure, and requiring little or no tailwater for successful performance. The general arrangement of the basin is shown on **Figure 1112**. This type of basin is subjected to large dynamic forces and turbulences which must be considered in the structural design. The structure should be made sufficiently stable to resist sliding against the impact load on the baffle wall and must resist the severe vibrations. Riprap should also be provided along the bottom and sides adjacent to the structure to avoid the tendency for scour of the outlet channel downstream from the end sill when shallow tailwater exists. This type of stilling basin is very effective at the outlet of storm drains or culverts where there is little or no tailwater.

1102.2.6 Hydraulic Design

The three different stilling basin configurations can be divided into two categories, basins for spillways or channels (Type III or IV) and basins for pipe outlets (Type VI). A summary of the design data for all three basin types is presented in **Figure 1109**. The reader is referred to PETERKA, 1978, for a detailed discussion of the structural design requirements.

1102.2.7 Riprap Protection

Riprap protection shall be provided downstream of the Type III, IV, and VI stilling basins (except in fully concrete lined channels). This protection is necessary to protect the downstream channel from erosion due to eddy currents and excess velocities in the transition zone between the structure and the design channel section.

For the Type III and Type IV basin, a 2-foot layer of regular riprap shall be installed from the end sill a distance of 4 times the design depth of flow in the downstream channel.

For the Type VI stilling basin, riprap protection shall extend downstream a distance equal to the outlet width, W, of the basin. The minimum downstream distance shall be 5 feet. A 2-foot layer of regular riprap shall be used for all basin widths of 12 feet or less. For basin widths between 12 feet and 20 feet, a 3-foot layer of heavy riprap shall be used. For basin widths greater than 20 feet, a 2-foot layer of grouted riprap shall be used.

1102.2.8 Design Flow Rates

The effectiveness of energy dissipation structures is dependent on many factors including flow rates, tail water depths, and type of dissipation structure. The structures also must function over a wide range of flow rates typical of stormwater runoff. Therefore, a minimum of the minor and major storm flow rates should be analyzed to assist in protecting the structure against drowning of the hydraulic jump or sweepout of the jump into the downstream channel. The design of the impact stilling basin shall be based on the design flow rate for the upstream pipe or channel.

1102.2.9 Trajectory Transition Section

Energy dissipation structures may be designed for either sub-critical or super-critical upstream flow conditions. For sub-critical flow, an abrupt change in grade at the structure entrance performs satisfactorily. However, for super-critical flow, the flow tends to separate and spring away at any abrupt change in grade. Therefore, to avoid the possibility of flow separation from the channel floor, the floor shape should be flatter than the trajectory of a free discharging flow jet.

Presented in **Figure 1113** is a typical design of a trajectory transition section. The curvature of the trajectory section can be determined by the following equation.

$$y = x \tan 2 + x^2 / K\,(4(d + h_v) \cos^2 2) \qquad (1103)$$

where Y = Change in Vertical Elevation (ft)
X = Change in Horizontal Location (ft)
K = Safety Factor
d = Depth of Flow at Trajectory Entrance (ft)
h_v = Velocity Head at Trajectory Entrance (ft)
2 = Slope Angle From Horizontal of the Upstream Channel (Degrees)

The safety factory, K, should be equal to or greater than 1.5 to assure positive contact pressure.

The trajectory section should be connected to the stilling basin apron by a short, steep chute section. This section should be at a slope between 1.5 horizontal to 1 vertical and 3 horizontal to 1 vertical with 2 horizontalto 1 vertical preferred. In no case should the slope be flatter than 6 horizontal to 1 vertical.

1102.3 Example Applications

The following example applications present typical design calculations for various channel drop and energy dissipation structures. The reader is referred to PETERKA, 1978, for additional design examples.

1102.3.1 Example: Sloping Riprap Drop Structure

Problem: Design a sloping riprap drop for a channel with the following characteristics:

Q = 1,600 cfs

Upstream and Downstream Channel Parameters

Bottom Width	=	50 ft	S	=	0.0043 ft / ft
Side Slopes	=	4:1	Y_c	=	2.9 ft
Y_n	=	4.0 ft	V_n	=	6.0 fps

Concrete Low Flow Channel

Drop Required = 3.0 ft

Solution:

Step 1: Determine Maximum Unit Discharge.

$q = V_n Y_n$ = (6.0 fps) (4 ft) = 24 cfs / ft

Step 2: Select the chute slope from **Table 1102** for q = 25 cfs / ft

The following options are available:

1) Heavy Riprap 10:1 or Flatter; DR = 1.75 ft, DRW = 2.6 ft

2) Grouted Riprap at 6:1 or Flatter; DR = 2.6 ft; DRW = 3.25 ft

The best slope will depend on factors such as availability and cost of the riprap bedding and filter cloth and ROW limitations. For this example, a 7:1 slope was selected.

Step 3: Select Length of Downstream Apron L_B = 20 ft

Step 4: Determine Crest Wall Elevation. (**Table 1104**)
Bottom width = 50 ft, Y_n = 4.0 ft
Use P = 0.1 ft

1102.3.2 Example: Vertical Riprap Drop Structure

Problem: Design a vertical riprap drop for a channel with the following characteristics:

Q = 1,600 cfs

Upstream and Downstream Channel Parameters:

Bottom Width	=	50 ft	S	=	0.0043 ft / ft
Side Slopes	=	4: 1	Y_c	=	2.9 ft
Y_n	=	4.0 ft	V_n	=	6.0 fps

Concrete Low Flow Channel
Drop Required = 3.0 ft

Solution:

Step 1: From **Table 1103**, for C = 3.0 ft, V_n = 6.0 fps and Y_n and Y_2 = 4.0 ft

Select the riprap designation and the riprap basin dimensions.

Riprap - Heavy

B = 1.0 ft
A = 2.5 ft
L_B = 20 ft
D = 5.0 ft
E = 4.0 ft

Step 2: Determine P = 0.1 from **Table 1104**

Step 3: Design retaining wall and finalize dimension

1102.3.3 Example: Vertical Gabion Drop Structure

Problem: Design a vertical gabion drop structure for a channel with the following characteristics:

Q = 1,600 cfs

Upstream and downstream channel parameters are:

Bottom Width = 60 ft
Side Slope 4:1,
Y_n = 3.3 ft
Drop Required = 4 ft

S = 0.0043 ft / ft
Y_c = 2.63 ft

Assume the drop is to be built with a lined stilling pool floor and counter drop (weir).

Refer to **Figure 1114** for definition of variables.

Solution:

Step 1: Design of Crest:

Assume the width of the rectangular weir, Lg = 48 feet and length along stream = 12 feet minimum, (use more if needed for stability). Assume C = 3.1 in the weir formula:

$$Q = CL_g (Z_o - f_g)^{3/2}$$

$$Z_o - f_g = Q^{2/3} / CL_g = 1,600^{2/3} / 3.1 \times 48$$

Use 6 feet as the height of the crest.

Step 2: Design of Stilling Pool:

Assume the width of the counter drop = 54 feet

q = 1,600 / 54 = 19.6 cfs / ft

With q = 29.6 cfs and Z_o -f_b = (4 + 4.9) = 8.9 ft using Figure 53 in MACCAFERRI,1987

Z_1 -f_b = 1.25 ft

With Z_1-f_b = 1.25 ft and q = 29.6 and using dashed line in Figure 53 in MACCAFERRI, 1987

Z_2 -f_b = 5.8 ft

Determine Z_2 -f_c by using weir equation

$Z_2 - f_c = Q^{2/3} / CL_b = (1,600)^{2/3} / 3.1 \text{ x}54$

The height of the counter drop is

$f_c - f_b \;\; (Z_2 - f_b) - (Z_2 - f_c) = 5.8 - 4.5 = 1.3$ ft

Use 6 feet (minimum) as the length of the counter drop along stream. Determine the length of the stilling pool, L_b

$L_b = L_{g1} + L_{12}$

Since the drop is backed by streambed material

$L_{g1} / (f_g - f_b) = 4.30D^{0.27}$ (Eq.16)

(Eq. 15) $D = q^2 / g\ (f_g - f_b)^3$

$D = (1,600 / 48)^2 / 32.2\ (4)^3$

$L_{g1} = 4.30\ (0.5392)^{0.27} \text{ x } 4 = 14.6$ ft
$L_{12} = 6.9\ (Z_2 - Z_1)$ (Eq. 20)
$= 6.9 \text{ x } [\ (Z_2 - f_b) - (Z_1 - f_b)\ ]$
$= 6.9 \text{ x } (5.8 - 1.25)$
= 31.4 ft
$L_b = L_{g1} + L_{12} = 14.6 + 31.4 = 46$ ft

Use L_b = 48 ft

Note: The above Equations 15, 16, and 20 are found in MACCAFERRI, 1987.

Step 3: Check for seepage:

The total path L of seepage under and the structure must be L > C) H (Equation 22, MACCAFERRI,1987)

Where C is a coefficient depending on the type of soil, and) H is the difference between the upstream and downstream water surfaces.

Assume C = 6 (riverbed sediment)
Therefore L > 6 x 5.5 = 33 ft

The length of stilling pool itself is 48 ft > 33 ft. Thus seepage consideration is satisfied.

To prevent undermining of the counter drop (weir), an apron is constructed downstream; the length of this apron will be approximately 9 feet.

Step 4: To complete the design of the drop structure, a stability analysis will be required to determine the required thickness of the gabion mattresses and depth to foundation. This stability analysis is beyond the scope of this manual. The user should refer to MACCAFERRI, 1987 for guidance for stability analysis.

The hydraulic design dimensions of the structure are given in **Figure 1114**.

1102.3.4 Example: Impact Stilling Basin

Problem: Design an impact stilling basin (USBR Type VI) for a 48-inch RCP outlet with the following parameters:

Pipe Dia = 48 in RCP
Q = 214 cfs
V = 17 fps < 30 fps (upper limit)
Tail water depth = 2.5 ft
Channel slope = 1.0 percent

Solution:

Step 1: Using the discharge (Q = 214 cfs) enter the discharge limits portion of **Figure 1109** and read the maximum and minimum basin width.

W_{min} = 12.5 ft
W_{min} = 15.0 ft

Step 2: From the basic dimension portion of **Figure 1109** and using the discharge Q = 214 cfs, interpolate for the basin dimensions. Note that the corresponding pipe size in the table is between a 54-inch and 60-inch diameter, which is larger than the example pipe size of 48 inches. The basin will therefore provide ample room for the example pipe.

The basic basin dimensions are as follows:

W = 12 ft - 4 in	b = 10 ft - 6 in
H = 10 ft - 3 in	c = 5 ft - 8 in
L = 18 ft - 2 in	d = 2 ft - 4 in
A = 7 ft - 8 in	g = 5 ft -1 in

Step 3: Determine length of downstream riprap.

Downstream length = W = 12 ft - 4 in
Use a downstream length of 13 in

Step 4: Determine size and thickness of downstream riprap.

For a basin width of 12 ft - 4 in, use a 3 ft layer of heavy riprapper Section 1102.2.7.

CHANNEL DROP AND ENERGY DISSIPATION STRUCTURES

STRUCTURE	1 UPSTREAM FLOW CLASS	2 MAXIMUM ALLOWED DROP HEIGHT (FEET)	3 MAXIMUM ALLOWED FLOW RATE (CFS/FT)	4 MAXIMUM ALLOWED INFLOW VELOCITY (FPS)	5 APRON ENTRANCE FROUDE NO. (FR)	6 MAXIMUM APRON ENTRANCE VELOCITY (FPS)	7 REQUIRED CROSS-SECTION GEOMETRY	8 REFERENCE FIGURE NUMBER
SLOPING RIPRAP DROP	SUB.	3	35	7	-	-	TRAP.	1102
VERTICAL RIPRAP DROP	SUB.	3	35	7	-	-	TRAP.	1103
GABION DROP	SUB.	10	35	-	-	-	TRAP. OR RECT.	1104-1106
STRAIGHT DROP	SUB.	8	-	-	-	-	RECT.	1107
BAFFLED APRON (USBR TYPE IX)	SUB.	-	60	12	-	-	RECT.	1108
SHORT STILLING BASIN (USBR TYPE III)	SUB. OR SUPER	-	-	-	>4.5	60	RECT.	1110
LOW FROUDE NUMBER BASIN (USBR TYPE IV)	SUB. OR SUPER	-	-	-	2.5 TO 4.5	-	RECT.	1111
BAFFLED OUTLET (USBR TYPE VI)	SUB. OR SUPER	NA	-	30	NA	NA	NA	1112

COLUMN DESCRIPTIONS:

1. SUB. = SUBCRITICAL ($Fr < 0.86$)
   SUPER = SUPERCRITICAL ($Fr > 1.13$)
2. DROP HEIGHT MEASURED FROM BOTTOM OF UPSTREAM CHANNEL TO BOTTOM OF DOWNSTREAM CHANNEL.
3. FLOW RATE = NORMAL DEPTH (Y_n) MULTIPLIED BY NORMAL VELOCITY (V_n).
4. INFLOW VELOCITY = UPSTREAM NORMAL CHANNEL VELOCITY.
5. FROUDE NUMBER FOR FLOW CONDITIONS AT ENTRANCE TO STRUCTURE APRON.
6. VELOCITY FOR FLOW CONDITIONS AT ENTRANCE TO STRUCTURE APRON.
7. CROSS-SECTION OF CHUTE AND STILLING BASIN.
8. REFERENCE TO FIGURES IN THIS MANUAL.

NOTE: Tail water (TW) is measured from invert of basin to water surface in channel immediately downstream of end sill.

Revision	Date

WRC ENGINEERING

REFERENCE:

SLOPING RIPRAP CHANNEL DROP DESIGN CHART

MAXIMUM UNIT DISCHARGE q (cfs/ft)	ALLOWABLE CHUTE SLOPE - So FOR EACH RIPRAP CLASSIFICATION			LENGTH OF DOWNSTREAM APRON L_b (ft)
	Regular Riprap	Heavy Riprap	Grouted Riprap	
0 - 15	Not Allowed	0 to 7:1	7:1 to 4:1	15
15.1 - 20	Not Allowed	0 to 8:1	8:1 to 5:1	20
20.1 - 25	Not Allowed	0 to 10:1	10:1 to 6:1	20
25.1 - 30	Not Allowed	0 to 12:1	12:1 to 7:1	25
31.1 - 35	Not Allowed	0 to 13:1	13:1 to 8:1	25
> 35	Not Allowed	Not Allowed	Not Allowed	Not Allowed
Dr ($V \leq 5$fps)	Not Allowed	1.75'	2.6'	
Dr ($V > 5$fps)	Not Allowed	2.0'	3.0'	
Drw	Not Allowed	1.5 x Dr	1.25 x Dr	

NOTES:

1. See Figure 1102 for definition of symbols.
2. q = Unit discharge = VnYn, where Vn = average channel velocity and Yn = normal depth of the upstream channel.
3. So = Longitudinal channel slope expressed in feet horizontal per foot vertical.
4. Dr = Depth of riprap blanket in feet.
5. Drw = Depth of riprap blanket at the downstream face of the crest wall and in upstream apron.
6. Rock size, Dr, and Drw shall be the same throughout the drop structure.
7. Chute and channel side slopes shall not be steeper than 4:1.
8. Maximum allowable drop = 3.0'
9. See Section 700 for riprap gradation, classification and bedding requirements.
10. This chart is for ordinary riprap structures only. Other types of drop structures require their own hydraulic analysis.
11. See Table 1104 to calculate P.

Revision	Date

VERTICAL RIPRAP CHANNEL DROP DESIGN CHART

C (ft)	Vn (fps)	Y_n&Y_2 (ft)	P (ft)	B (ft)	A (ft)	L_b (ft)	D (ft)	E (ft)	RIPRAP CLASS.
2	≤5	4	Table 1104	0.6	2.0	20	4	3	Heavy Riprap
2	≤5	5	Table 1104	0.8	2.5	25	5	4	Heavy Riprap
2	≤7	4	Table 1104	0.8	2.5	20	5	4	Heavy Riprap
2	≤7	5	Table 1104	0.8	2.5	25	5	4	Heavy Riprap
3	≤5	4	Table 1104	1.0	2.5	20	5	4	Heavy Riprap
3	≤5	5	Table 1104	1.0	2.5	25	5	4	Heavy Riprap
3	≤7	4	Table 1104	1.0	2.5	20	5	4	Heavy Riprap
3	≤7	5	Table 1104	1.0	2.5	25	5	4	Heavy Riprap

NOTES:
1. See Figure 1103 for definition of symbols
2. See Section 700 for riprap gradation, classification, and bedding requirements.
3. Maximum Allowable C = 3.0'
4. This chart is for ordinary riprap structures only. Other types of drop structures require their own hydraulic analysis.
5. See Table 1104 to calculate P.

Revision	Date

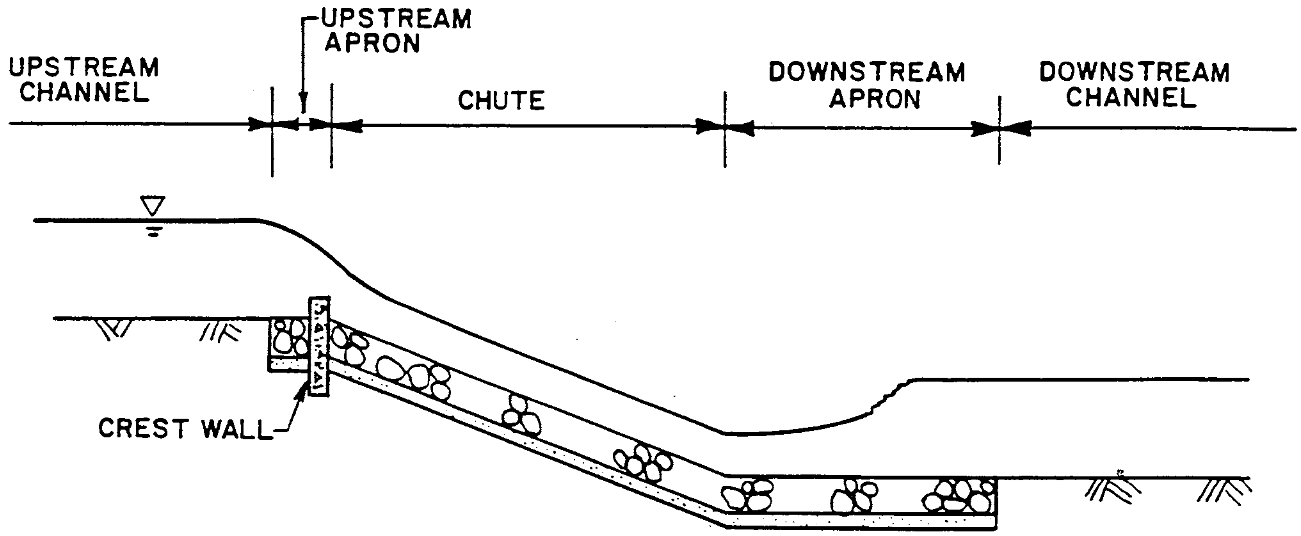
CREST WALL ELEVATION DESIGN CHART
(P in feet)

CHANNEL BOTTOM WIDTH	Yn ≤4.0'	Yn >4.0'	
		Vn ≤5fps	Vn >5fps
0'- 20'	0.1	0.2	0.2
20'- 60'	0.1	0.4	0.2
60'- 100'	0.1	0.5	0.3
> 100'	0.2	0.5	0.3

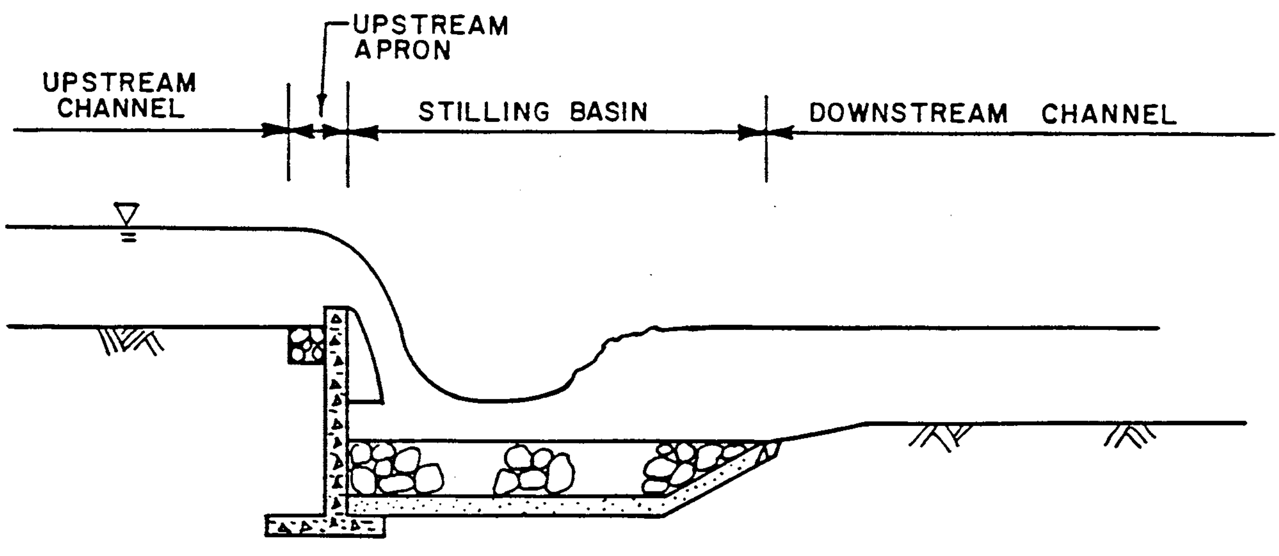
NOTE:
1. Refer to Figure 1102 and 1103 for definition of crest wall elevation "P".

Revision	Date

GENERALIZED PROFILES OF RIPRAP DROP STRUCTURES

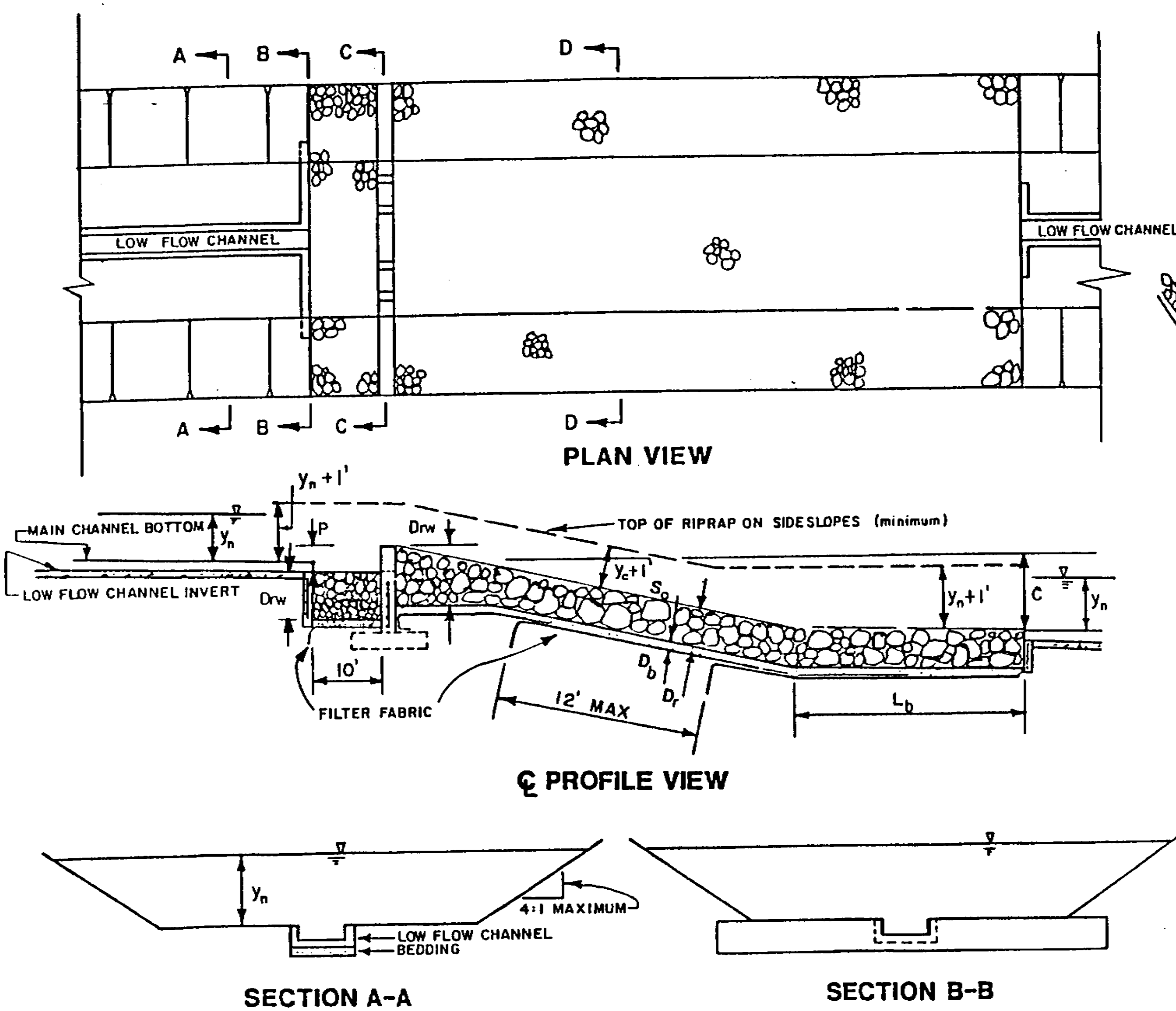


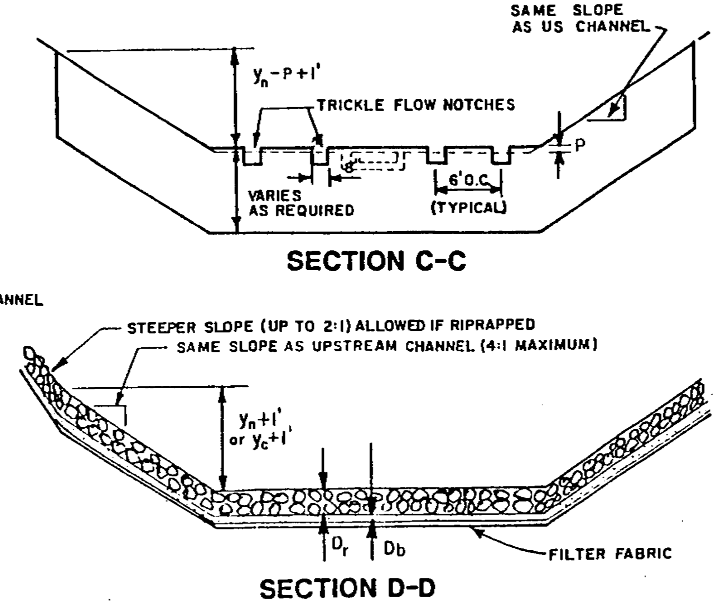
A. SLOPING CHANNEL DROP



B. VERTICAL CHANNEL DROP

Revision	Date





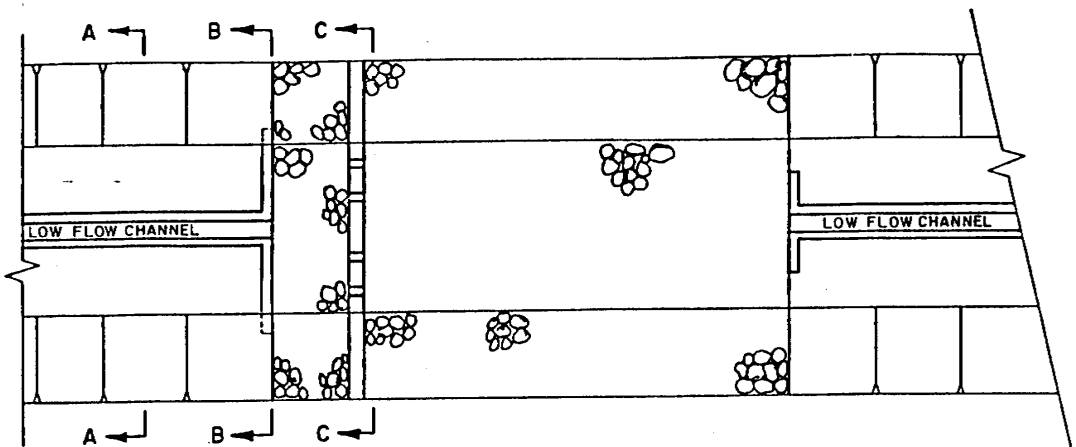
LEGEND

D_r	=	DEPTH OF RIPRAP
D_b	=	DEPTH OF BEDDING (Type II Aggregate Base)
D_{rw}	=	DEPTH OF RIPRAP AT CRESTWALL
y_n	=	NORMAL DEPTH OF APPROACH CHANNEL
y_c	=	CRITICAL DEPTH OF SLOPING CHANNEL

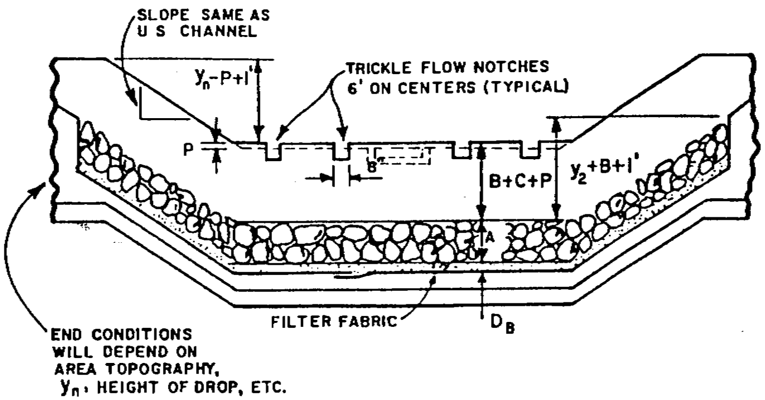
NOTES:
1. Total number of trickle flow notches = 2x low flow channel bottom width/8 inches.
2. For channels without low flow channels, eliminate trickle flow notches and locate top of upstream apron riprap even with upstream channel bottom.
3. See Table 1101 and 1102 for design limitations.

Revision	Date

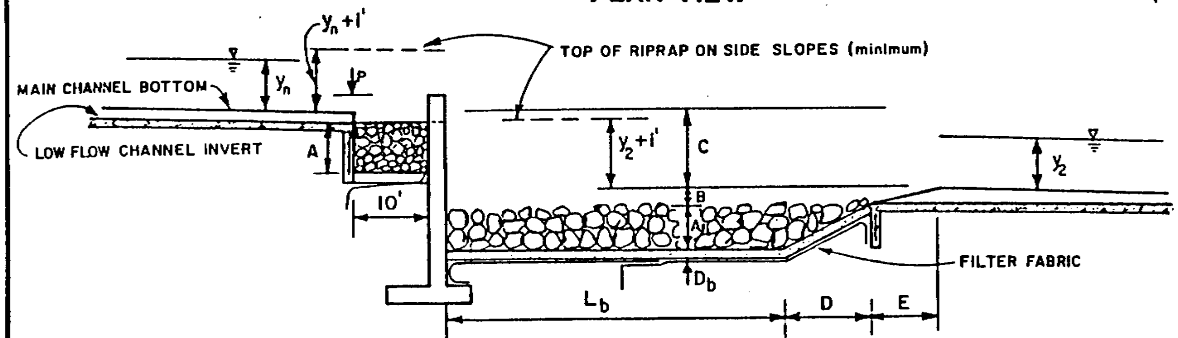
WRC ENGINEERING, INC.	REFERENCE: USDCM, DRCOG, 1982 DRAFT REVISIONS FOR CHANNEL DROPS	HYDROLOGIC CRITERIA AND DRAINAGE DESIGN MANUAL	SLOPING RIPRAP CHANNEL DROP	FIGURE 1102



PLAN VIEW



SECTION C-C

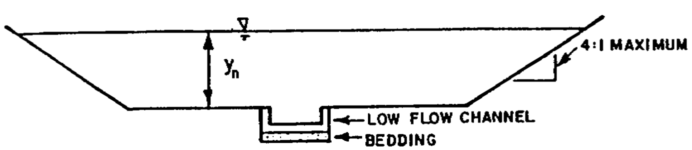


℄ PROFILE VIEW

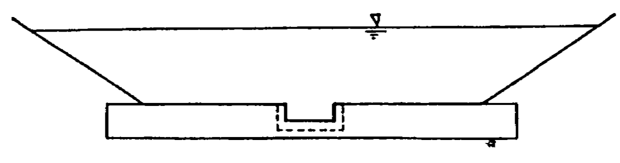
LEGEND

A	=	THICKNESS OF RIPRAP
B	=	DISTANCE TOP OF RIPRAP IS BELOW D.S. CHANNEL
C	=	TOTAL DROP BETWEEN UPPER AND LOWER CHANNELS
D_b	=	DEPTH OF BEDDING (Type II Aggregate Base)
$y_n = y_2$	=	NORMAL DEPTH OF CHANNEL

NOTES:
1. Total number of trickle flow notches = 2x low flow channel bottom width/8 inches.
2. For channels without low flow channels, eliminate trickle flow notches and locate top of upstream apron riprap even with upstream channel bottom.
3. See Table 1101 and 1103 for design limitations.



SECTION A-A

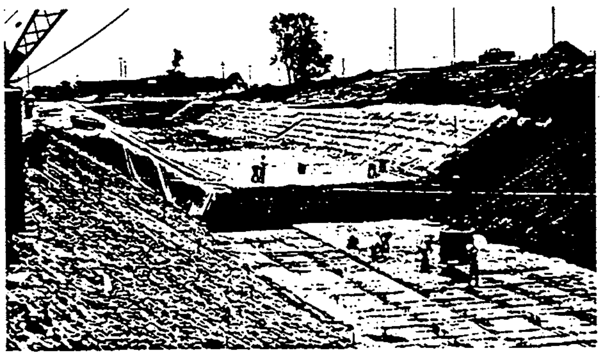


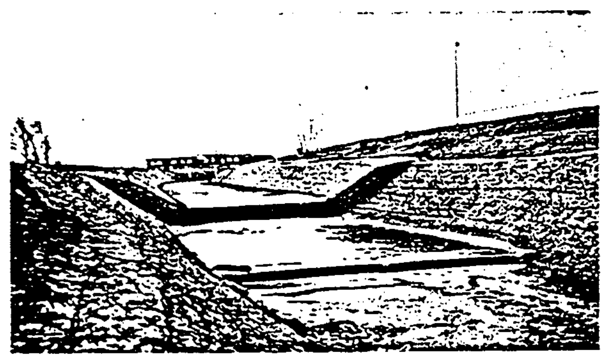
SECTION B-B

Revision	Date

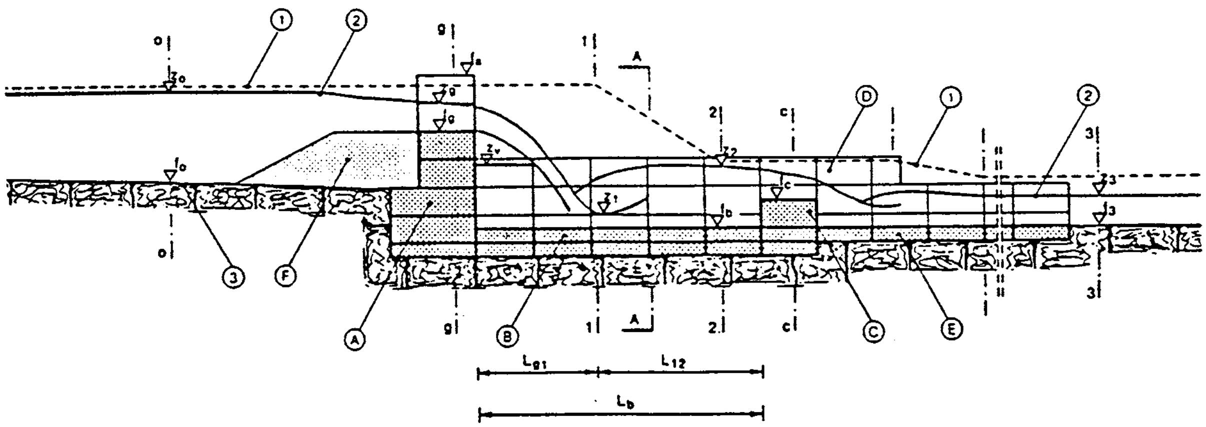
WRC ENGINEERING, INC.	REFERENCE: USDCM, DRCOG, 1982 DRAFT REVISIONS FOR CHANNEL DROPS	HYDROLOGIC CRITERIA AND DRAINAGE DESIGN MANUAL	VERTICAL RIPRAP CHANNEL DROP — FIGURE 1103

TYPICAL VERTICAL GABION DROP





Section B-B



GLOSSARY:
- A : Gabion weir
- B : Stilling pool lining
- C : Counterweir
- D : Downstream wing walls
- E : Downstream apron
- F : Artificial backfill
- Lb : Length of stilling pool
- Lg1 : Distance of section 1 from the downstream face of the weir
- L12 : Minimum length necessary for the hydraulic jump
- lb : Width of stilling pool
- lg : Width of crest
- 1 : Total head line
- 2 : Free surface profile
- 3 : Original soil profile

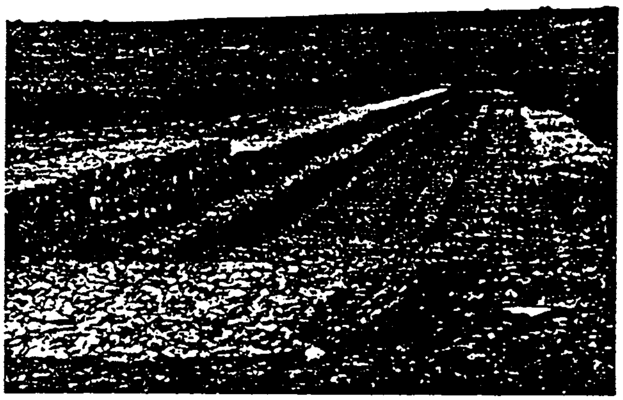
ELEVATIONS:
- z : Water levels
- f : River bed and structure elevations
- a : Elevation of weir's wings

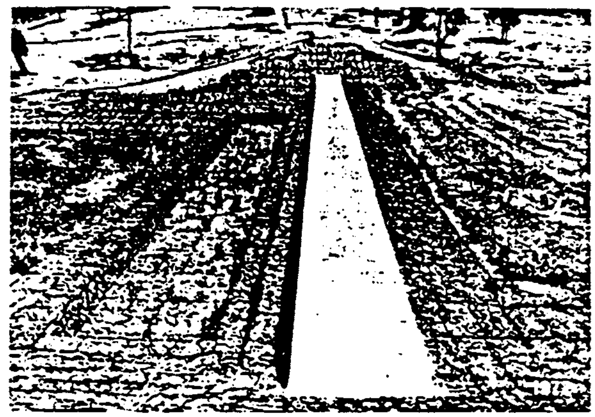
SECTIONS:
- 0 : Section upstream of the weir
- g : Section at crest
- 1 : Section at initial depth
- 2 : Section at sequent depth
- c : Section at counterweir
- 3 : Section downstream of the weir

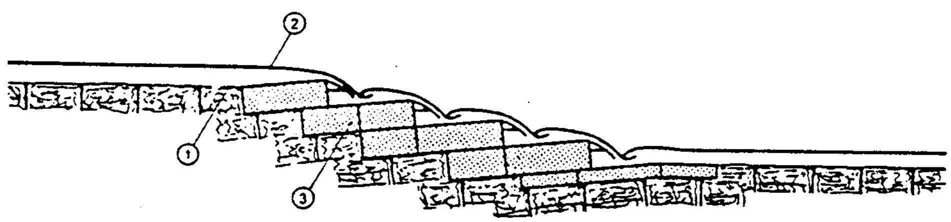
Revision	Date

REFERENCE: Flexable Gabion and Reno Mattress Structures, Maccaferri Gabions, 1986

TYPICAL STEPPED GABION DROP

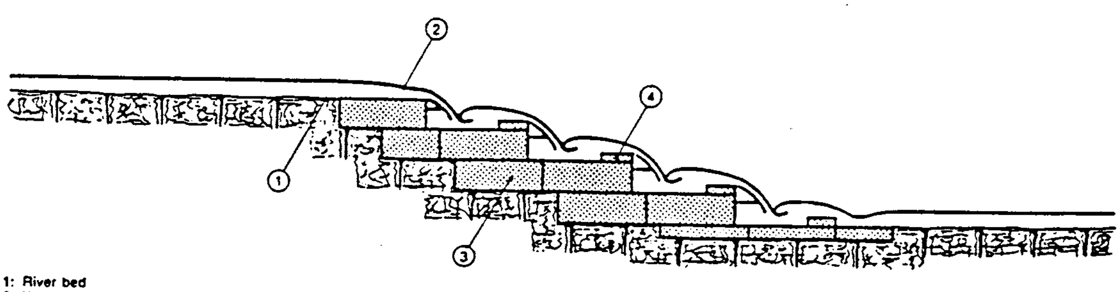






Inclined Steps

1: River bed
2: Water surface
3: Stepped weir with inclined steps



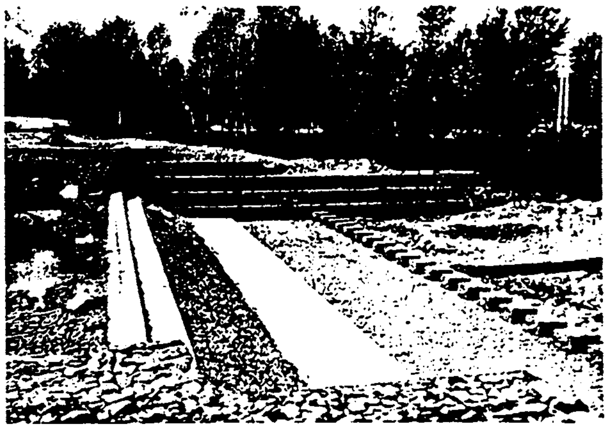
1: River bed
2: Water surface
3: Stepped weir with pooled steps
4: Sill

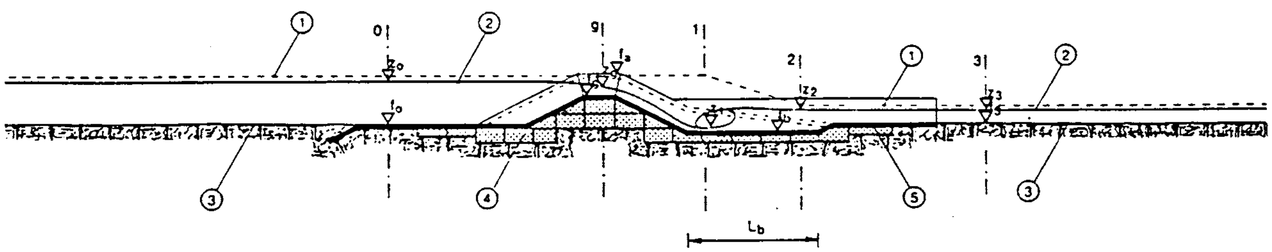
Pooled Steps

Revision	Date

REFERENCE: Flexable Gabion and Reno Mattress Structures, Maccaferri Gabions, 1986

TYPICAL SLOPED GABION DROP





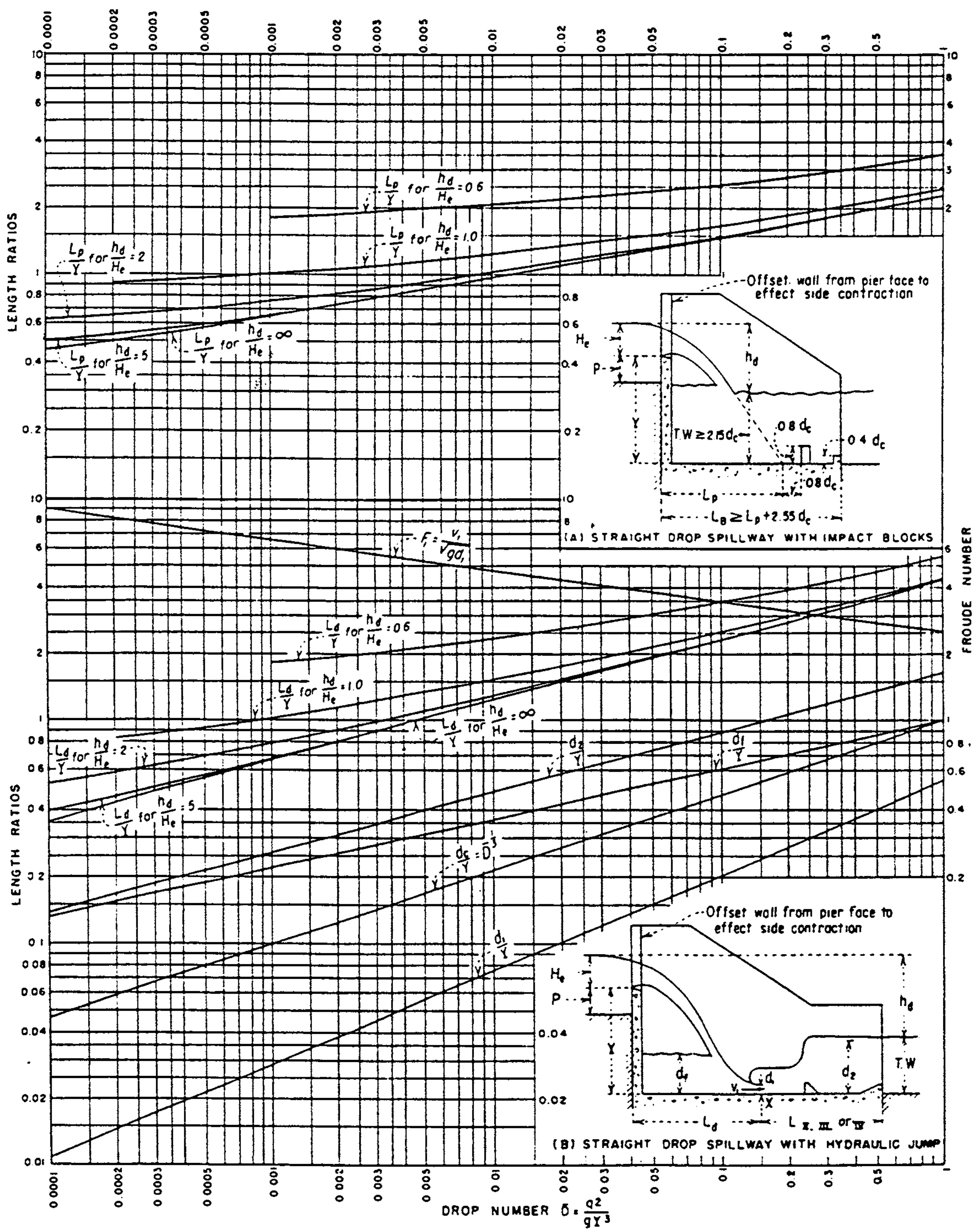
GLOSSARY:
1 : Total head line
2 : Water surface
3 : Original soil profile
4 : Gabion weir
5 : Mastic grouted Reno mattresses or gabions

ELEVATIONS:
z : Water surface
f : River bed and structure elevations
a : Elevation of weir's wings
Lb: Length of stilling pool

SECTIONS:
0 : Section upstream of the weir
g : Section at the crest
1 : Section at initial depth
2 : Section at sequent depth
3 : Section downstream of the counterweir

Revision	Date

STRAIGHT DROP SPILLWAY



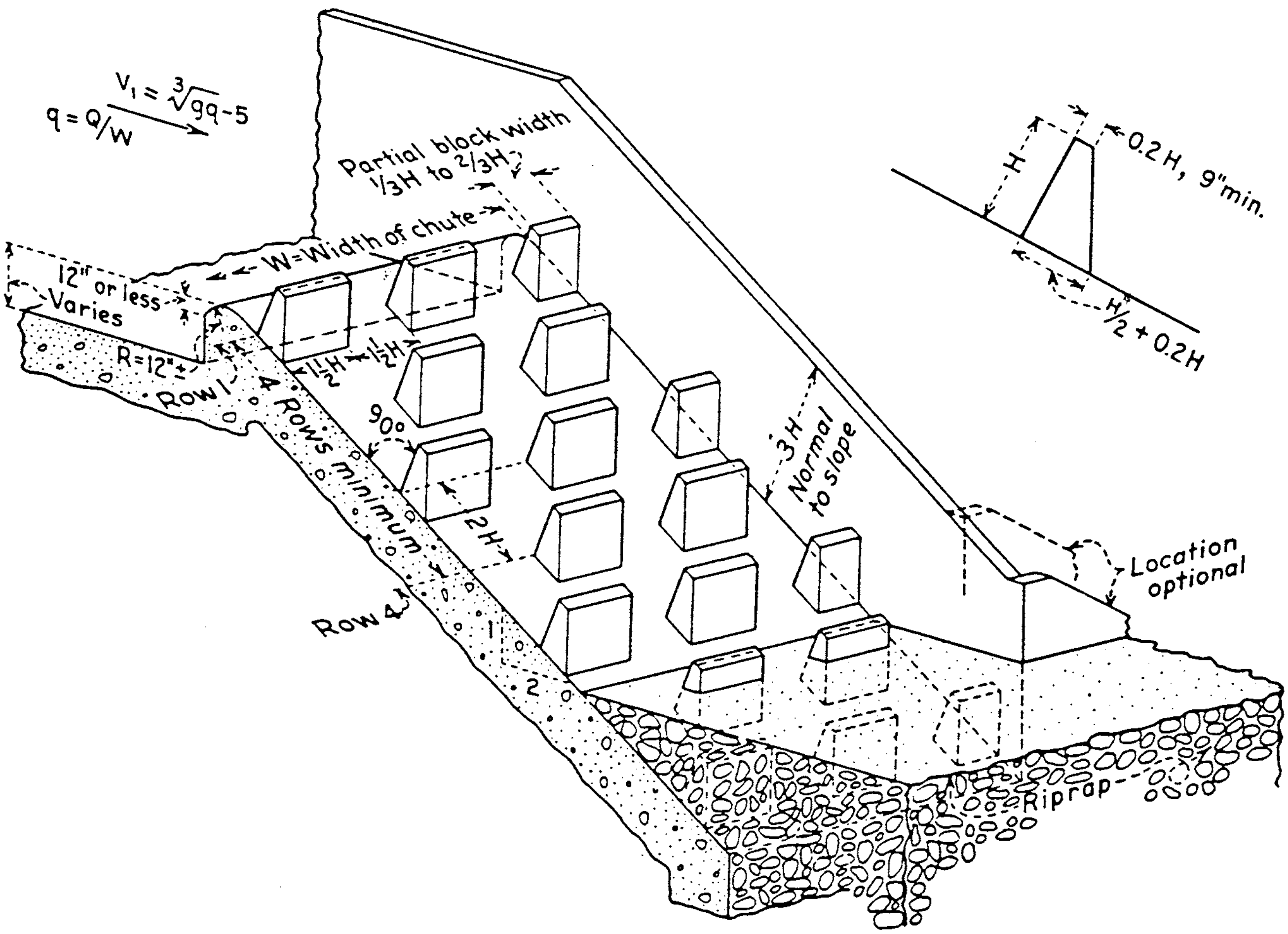
Revision	Date

WRC ENGINEERING

REFERENCE:
"Design of Small Dams", USBR, 1987

FIGURE 1107

BAFFLED APRON STILLING BASIN
(USBR TYPE IX)



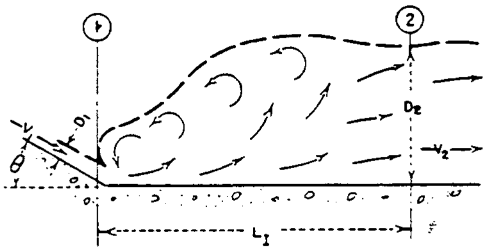
NOTE: See Figure 1109 for design data

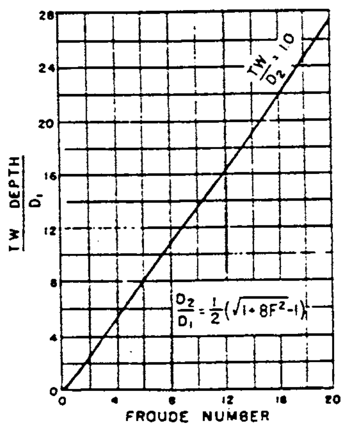
Revision	Date

DESIGN DATA-USBR TYPE STILLING BASINS

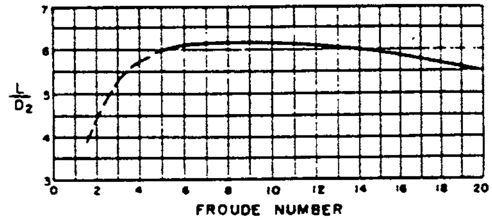
GENERAL INVESTIGATION OF THE HYDRAULIC JUMP ON HORIZONTAL APRONS (BASIN I)

Jump occurs on flat floor with no chute blocks, baffle piers or end sill in basin. Usually not a practical basin because of excessive length. Elements and characteristics of jumps for complete range of Froude numbers is determined to aid designers in selecting more practical basins II, III, IV, V, and VI.

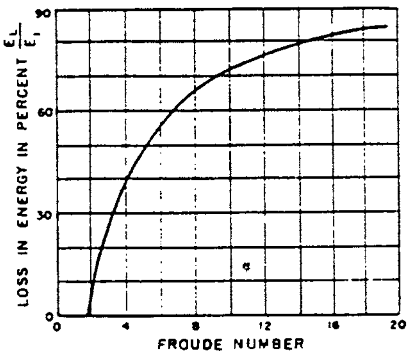




RATIO OF TW DEPTH TO D_1



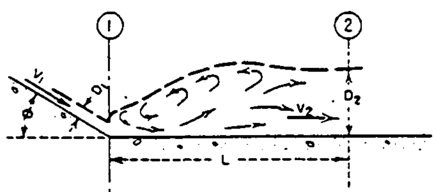
LENGTH OF JUMP



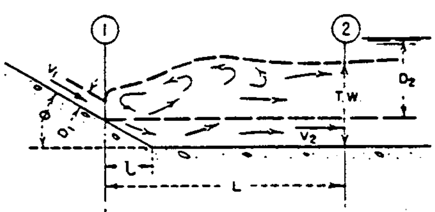
LOSS OF ENERGY IN JUMP

STILLING BASIN WITH SLOPING APRON (BASIN V)

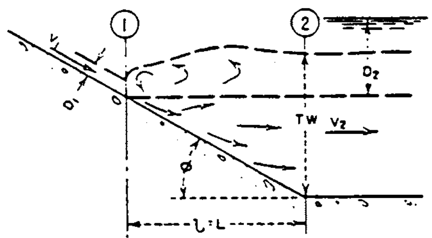
For use where structural economies dictate desirability of sloping apron, usually on high dam spillways. Needs greater tailwater depth than horizontal apron.



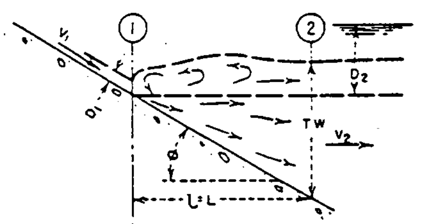
(CASE A)



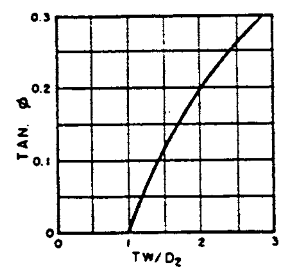
(CASE B)



(CASE C)

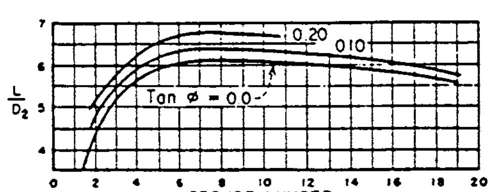


(CASE D)



TAILWATER DEPTH RELATED TO CONJUGATE DEPTH

(CASE D)



LENGTH OF JUMP

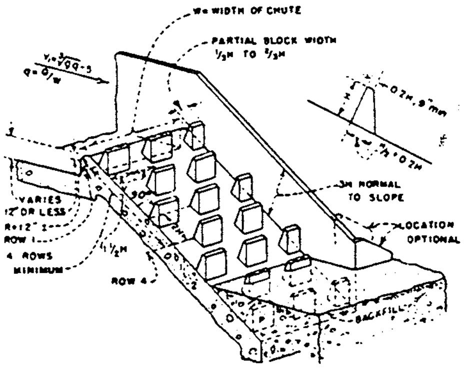
(CASE D)

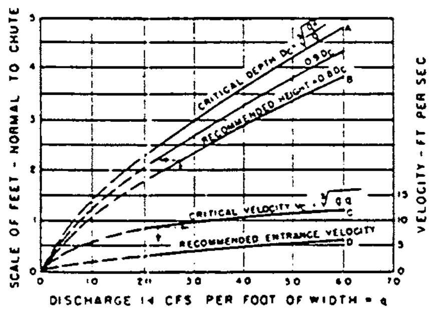
Revision	Date

DESIGN DATA-USBR TYPE STILLING BASINS

BAFFLED APRON FOR CANAL OR SPILLWAY DROPS (BASIN IX)

For use in flow ways where water is to be lowered from one level to another. The baffle piers prevent undue acceleration of the flow as it passes down the chute. Since the flow velocities entering the downstream channel are relatively low, no stilling basin is required. The chute may be designed to discharge up to 60 cubic feet per second per foot of width, and the drop may be as high as structurally feasible.





BAFFLE PIER HEIGHTS AND ALLOWABLE VELOCITIES

DESIGN PROCEDURE

The baffled apron should be designed for the maximum expected discharge, Q, up to 60 c.f.s. per foot of width.

Entrance velocity V_1, should be as low as practical or $V = \sqrt[3]{gq}$

See Figures 103, 105, 107 and 109 for sample approach pools.

Baffle pier height, H, should be about $0.8D_c$ to $0.9D_c$, Curve B above.

Baffle pier widths and spaces should be equal, up to $3/2$ H, but not less than H.

The slope distance between rows of baffle piers should be 2H, twice the baffle height H. See text for increase in row spacing on flat chutes.

Four rows of baffle piers are required to establish full control of the flow, although fewer rows have operated successfully. At least one row of baffles should be buried in the backfill.

The chute training walls should be three times as high as the baffle piers.

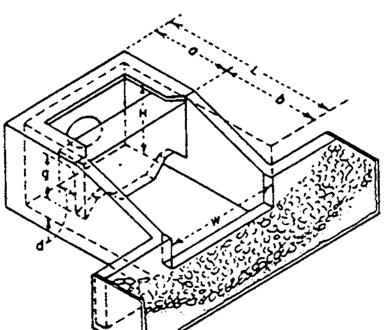
Riprap consisting of 6-to 12-inch stones should be placed at the downstream ends of training walls to prevent eddies from undermining the walls.

STILLING BASIN FOR PIPE ON OPEN CHANNEL OUTLET (BASIN VI)

For use on pipe or open channel outlets. Sizes and discharges from table V_1 should not exceed 30 feet per second. No tailwater required. Froude number usually 1.5 to 7 but not important.

May substitute for Basin IV.

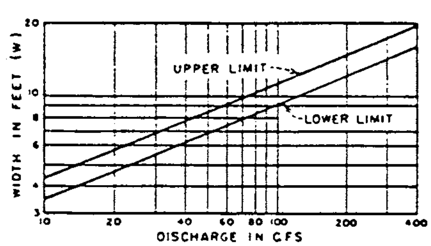
Energy loss greater than in comparable jump



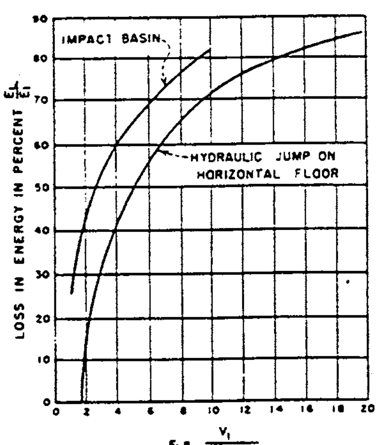
PIPE DIA IN	PIPE AREA SQ FT	Q	W	H	L	a	b	c	d	g
18	1.77	21	5-6	4-3	7-4	3-3	4-1	2-4	0-11	2-1
24	3.14	38	6-9	5-3	9-0	3-11	5-1	2-10	1-2	2-6
30	4.91	59	8-0	6-3	10-8	4-7	6-1	3-4	1-4	3-0
36	7.07	85	9-3	7-3	12-4	5-3	7-1	3-10	1-7	3-6
42	9.62	115	10-6	8-0	14-0	6-0	8-0	4-5	1-9	3-11
48	12.57	151	11-9	9-0	15-8	6-9	8-11	4-11	2-0	4-5
54	15.90	191	13-0	9-9	17-4	7-4	10-0	5-5	2-2	4-11
60	19.63	236	14-3	10-9	19-0	8-0	11-0	5-11	2-5	5-4
72	28.27	339	16-6	12-3	22-0	9-3	12-9	6-11	2-9	6-2

(W through g in FEET AND INCHES)

BASIC DIMENSIONS



DISCHARGE LIMITS



$F_1 = \frac{V_1}{\sqrt{gD}}$

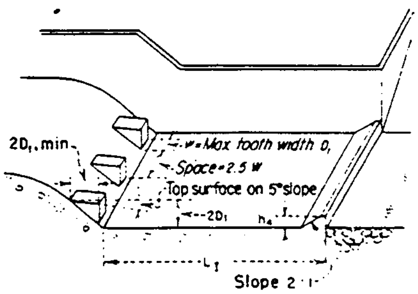
COMPARISON OF ENERGY LOSSES

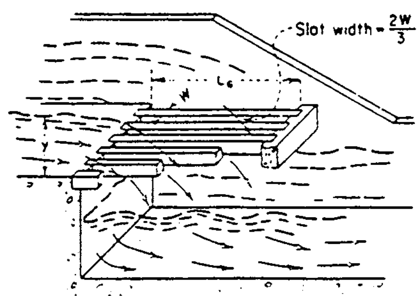
Revision	Date

DESIGN DATA-USBR TYPE STILLING BASINS

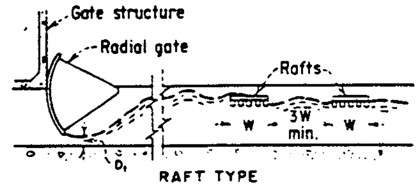
STILLING BASIN DESIGN AND WAVE SUPRESSORS FOR CANAL STRUCTURES, OUTLET WORKS, AND DIVERSION DAMS (BASIN IV)

For use with jumps of Froude number 2.5 to 4.5 which usually occur on canal structures and diversion dams. This basin reduces excessive waves created in imperfect jumps.
May also use alternate design and/or wave suppressors shown below, or Basin VI

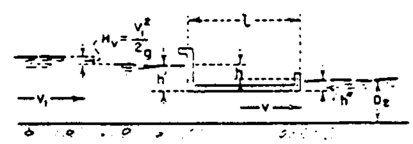




ALTERNATIVE DESIGN



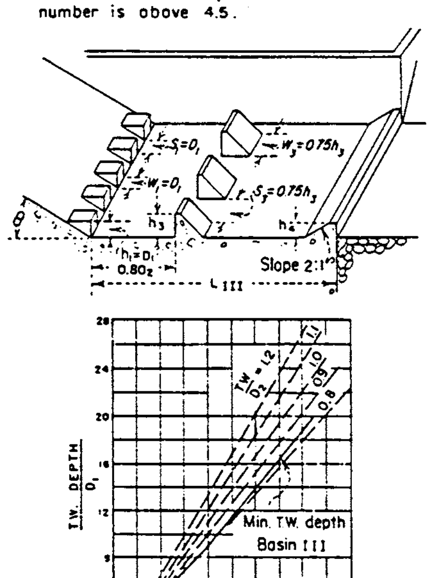
RAFT TYPE

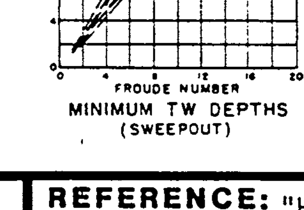


UNDERPASS TYPE
WAVE SUPPRESSORS

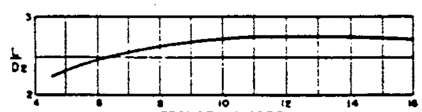
SHORT STILLING BASINS FOR CANAL STRUCTURES, SMALL OUTLET WORKS AND SMALL SPILLWAYS (BASIN III)

Jump and basin length reduced about 60 percent with chute blocks, baffle piers, and solid end sill.
For use on small spillways, outlet works, small canal structures where V_1 does not exceed 50-60 feet per second and Froude number is above 4.5.

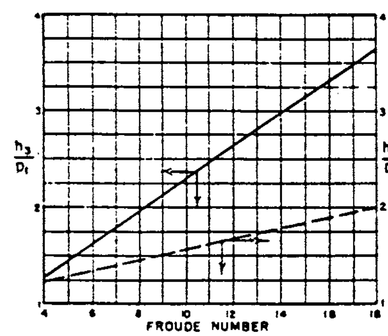




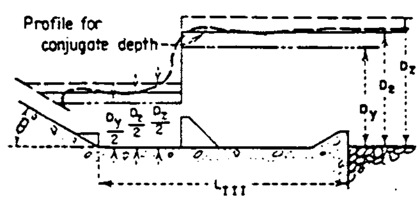
MINIMUM TW DEPTHS
(SWEEPOUT)



LENGTH OF JUMP



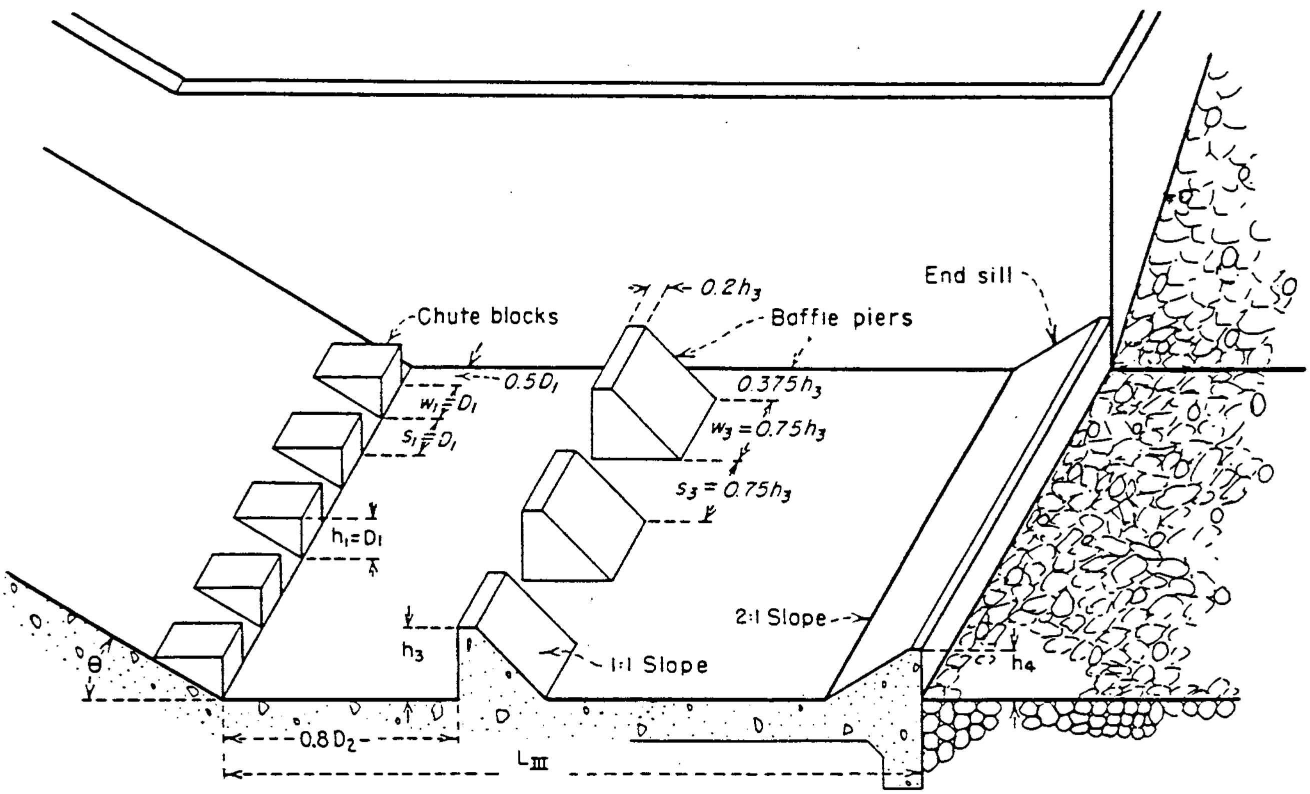
HEIGHT OF BAFFLE BLOCKS
AND END SILLS



WATER SURFACE AND
PRESSURE PROFILES

Revision	Date

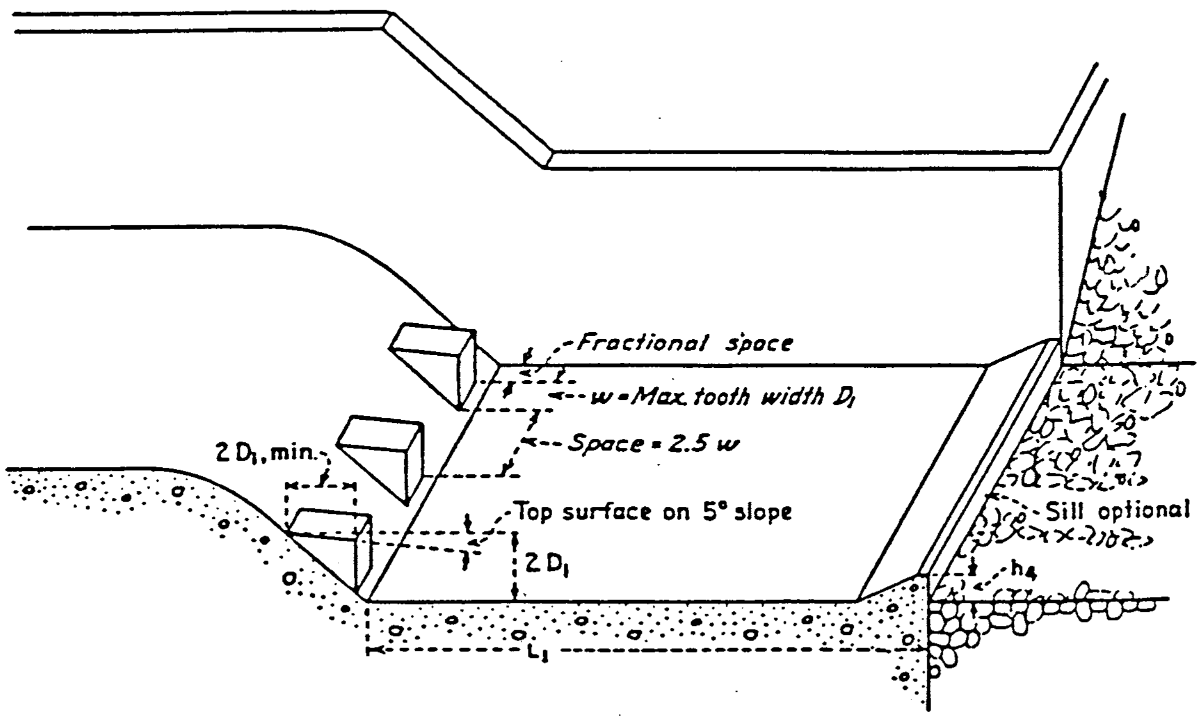
SHORT STILLING BASIN
(USBR TYPE III)



NOTE: See Figure 1109 for design data

Revision	Date

LOW FROUDE NUMBER STILLING BASIN (USBR TYPE IV)



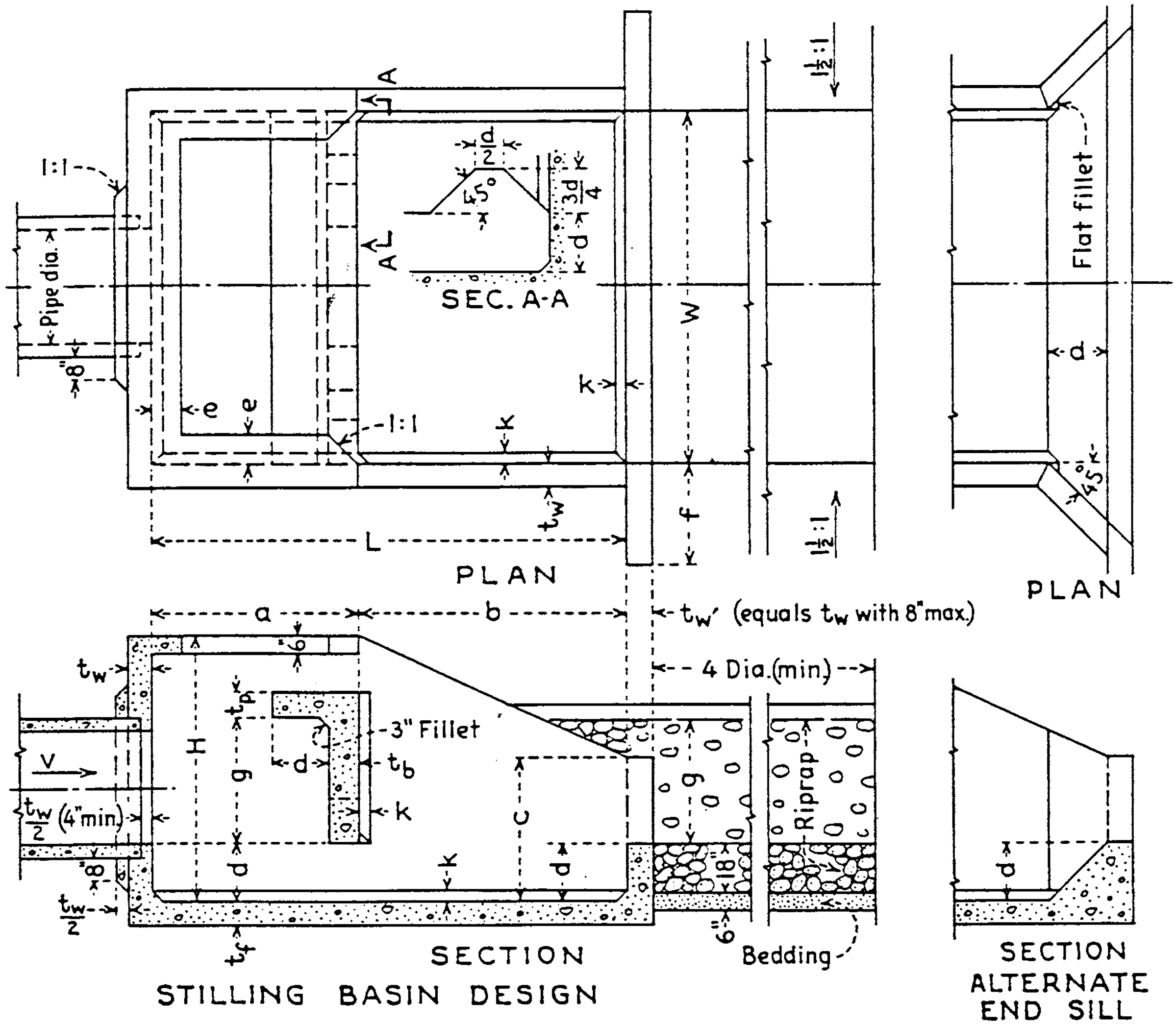
NOTE: See Figure 1109 for design data

Revision	Date

REFERENCE: "Hydraulic Design of Stilling Basins and Energy Dissipators", EM25 BR, January 1978

FIGURE 1111

IMPACT STILLING BASIN
(USBR TYPE VI)



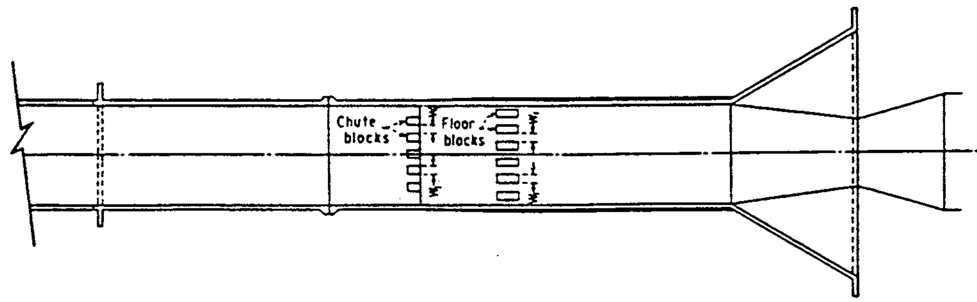
NOTE: 1. See Figure 1109 for design data

2. Refer to reference for structural details

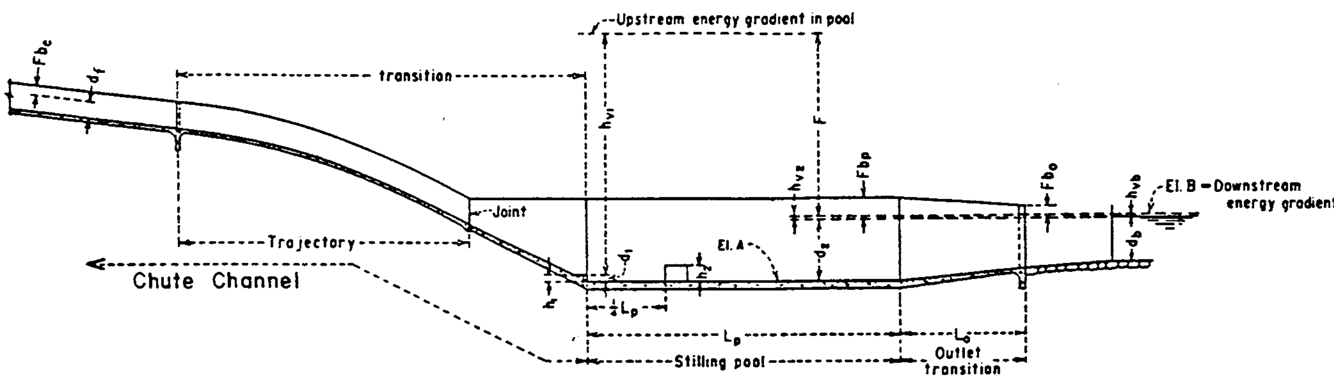
Revision	Date

WRC ENGINEERING	**REFERENCE:** "Design of Small Canal Structures", USDI, BR, Denver 1974	**FIGURE 1112**

TYPICAL TRAJECTORY TRANSITION



PLAN



PROFILE

EXPLANATION

d_1 ---- Computed depth at beginning of hydraulic jump.
h_{v1} --- Velocity head at beginning of hydraulic jump.
d_2 ---- Depth at end of hydraulic jump.
h_{v2} --- Velocity head at end of hydraulic jump.
d_b ---- Normal water depth in canal or channel downstream.
h_{vb} --- Velocity head for canal or channel downstream.
d_f ---- Maximum computed water depths in chute channel.
h_1 ---- Height of chute blocks.
h_2 ---- Height of floor blocks in pool.
W_1 ---- Width and spacing of chute and pool floor blocks.

L_i & L_o -- Inlet and outlet lengths as determined by local conditions and hydraulic requirements.
L_p ---- Length of stilling pool as determined by proposed use and hydraulic requirements.

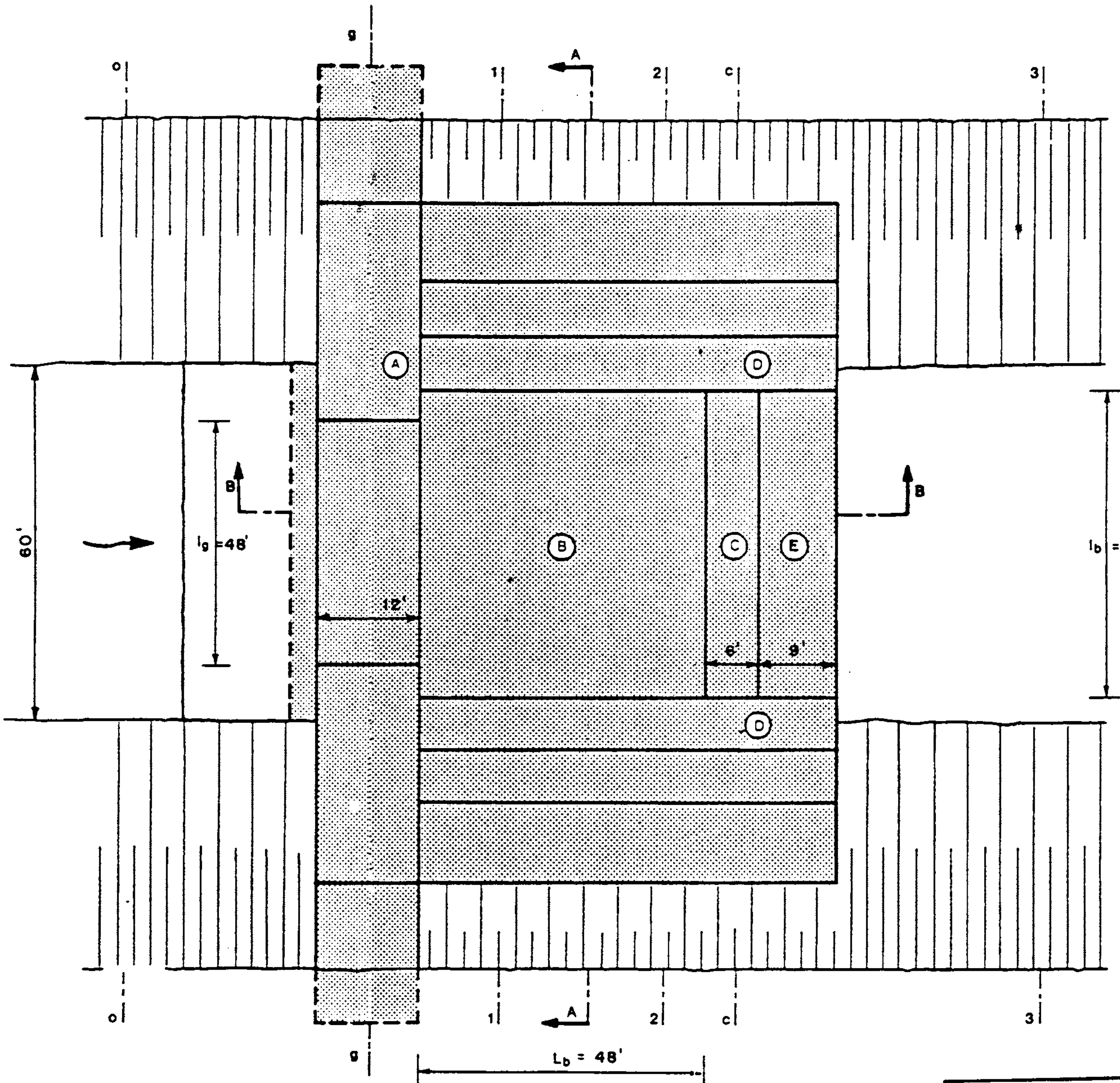
Fb_p --- Freeboard in stilling pool.
Fb_c --- Freeboard in chute.
Fb_o --- Freeboard at outlet transition.
F ----- Energy loss in hydraulic jump.
El. A --- Elevation of stilling pool invert.

Revision	Date

WRC ENGINEERING	**REFERENCE:** "Canals and Related Structures", Design Standards No.3, USBR, 1967	FIGURE 1113

GABION DROP FOR EXAMPLE IN SECTION 1102.3.3

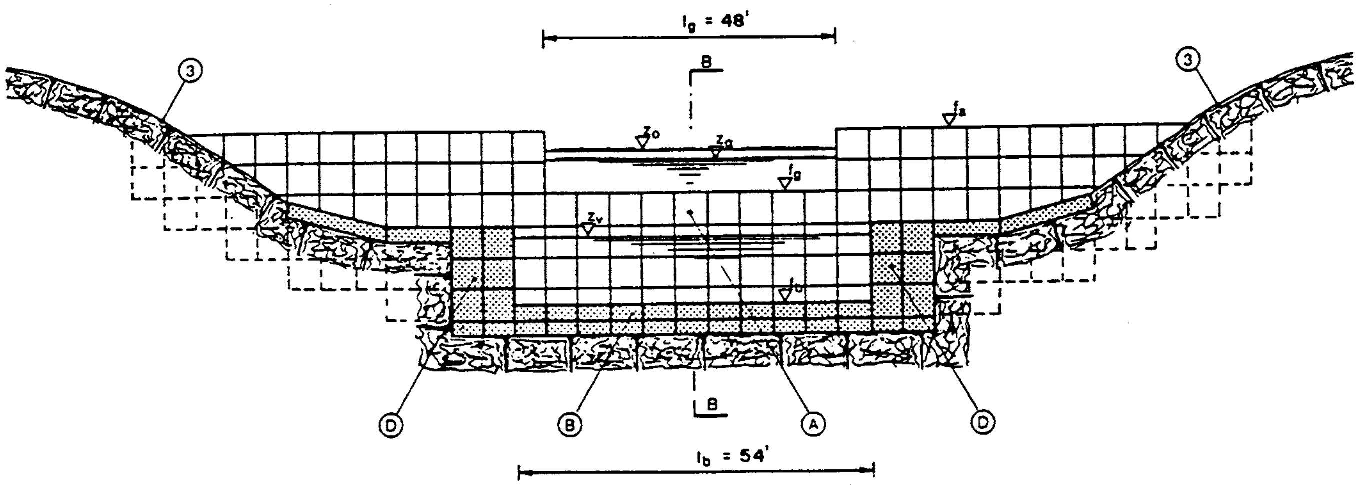
Plan (Schematic, no Scale)



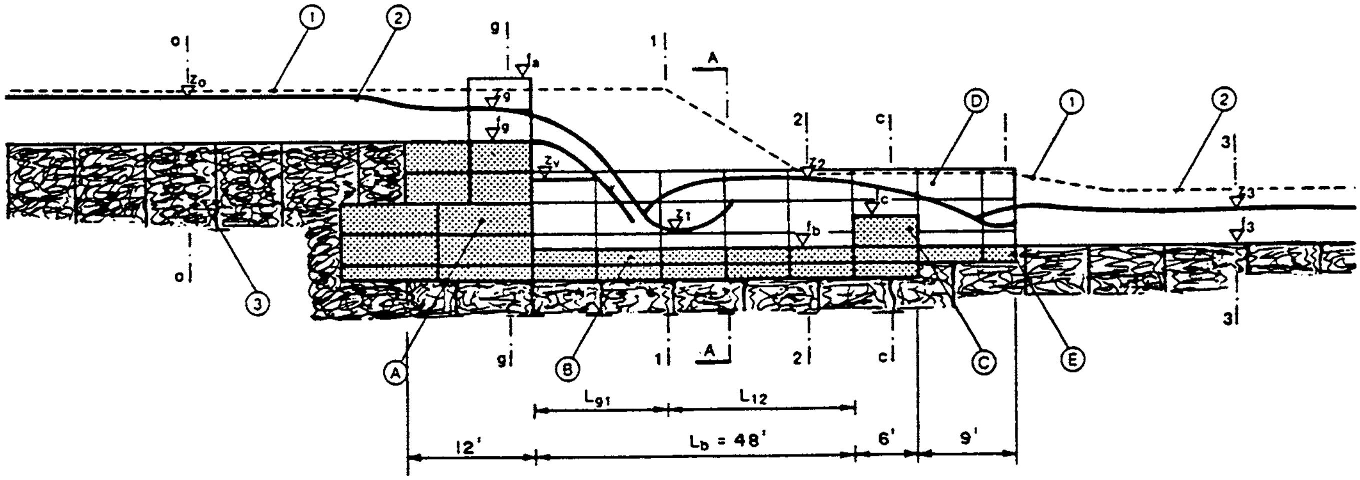
Revision	Date

GABION DROP FOR EXAMPLE IN SECTION 1102.3.3

Section A-A (Schematic, no Scale)



Section B-B (Schematic, no Scale)



GLOSSARY:

A : Gabion drop
B : Stilling pool lining
C : Counter drop
D : Downstream wing walls
E : Downstream apron
Lb : Length of stilling pool
Lg1 : Distance of section 1 from the downstream face of the weir
L12 : Minimum length necessary for the hydraulic jump
lb : Width of stilling pool
lg : Width of crest
1 : Total head line
2 : Free surface profile
3 : Original soil profile

ELEVATION:

Z : Water levels
f : River bed and structure elevations
a : Elevation of weir's wings

SECTIONS:

0 : Section upstream of the weir
g : Section of crest
1 : Section at initial depth
2 : Section at sequent depth
c : Section at counter drop
3 : Section downstream of the drop

Revision	Date

CLARK COUNTY REGIONAL FLOOD CONTROL DISTRICT
HYDROLOGIC CRITERIA AND DRAINAGE DESIGN MANUAL

SECTION 1200
DETENTION

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1203 OGEE-CRESTED WEIR COEFFICIENTS

1204 EXAMPLE TRASH RACK

1205 HYDROGRAPH FOR EXAMPLE IN SECTION 1208.2

1206 U.S. MEAN ANNUAL RUNOFF-PRODUCING RAINFALL DEPTHS (WEF MANUAL OF PRACTICE NO. 23)

1207A/B CALCULATIONS FOR SIZING WATER QUALITY CAPTURE VOLUME (WQCV) IN REGIONAL DETENTION BASINS

Section 1200 – Detention

1201 INTRODUCTION

The main purpose of a detention basin is to temporarily store runoff and reduce peak discharge by allowing flow to be discharged at a controlled rate. This controlled discharge rate is based on either limited downstream capacity (regional and local facilities) or on a limit on the increase in flows over pre-development conditions (local facilities only). Regional and local detention facilities are more fully discussed below. CCRFCD Policy regarding detention basin design is presented in the "Policy" Section 303.7.

1201.1 Definition of Regional Facilities

Regional detention facilities are those identified in the current flood control master plan of the CCRFCD. Generally, these facilities control flow on major washes, are of major proportion, and are funded in large part by the CCRFCD. The purpose of these facilities is to significantly reduce downstream flows, not to return the flows to pre-development levels.

1201.2 Definition of Local Facilities

Local detention facilities are usually designed by and financed by developers or local property owners or local entities. The facilities are intended to allow development by protecting a site from existing flooding conditions or to protect downstream property from increased runoff caused by development. Two classes of local facilities are defined below.

1201.2.1 Local Minor Facilities

Local minor detention facilities are defined as serving hydrologic basins smaller than or equal to 20 acres, and are designed to mitigate the impact of increased runoff due to development. The outlet capacity is based on pre-development hydrology and downstream conveyance system capacity and the structures are generally small (0.01 to 1 acre-feet). Detention storage volume may be provided as small landscaped or turfed basins, parking lot storage, roof top storage, or a suitable combination of all three.

1201.2.2 Local Major Facilities

Local major detention facilities are defined as serving hydrologic basins greater than 20 acres. These facilities may serve a double function. They are required to reduce existing flooding to allow development and/or control increased runoff caused by the development. These facilities may store significant flood volumes and will generally be funded by the developer. They may handle both off-site and on-site flows. Due to their considerable size, these basins are designed much the same as regional facilities.

1202 DETENTION/RETENTION DESIGN GUIDELINES AND STANDARDS

Certain guidelines for detention basin design need to be identified in order to properly design facilities. These guidelines cover items such as outlet flows, spillway sizing, and sedimentation. The following sections describe major guidelines governing detention basin designs.

1202.1 Regional Detention

The design of regional detention facilities will be coordinated with the CCRFCD. Also, as mentioned in Section 1206.1, the Nevada State Engineer must review detention basins which require dams having embankments greater than 20 feet in height or impounding over 20 acre-feet of water. Regional detention guidelines include:

1. Regional detention basins are preferred to smaller local detention basins.

2. Off-channel detention basins are preferred.

3. Multi-use (e.g., recreation) can be considered in the design of detention basins.

4. Below-grade detention basins are preferred to above-grade facilities.

5. Basins should be sited on publicly-owned lands whenever possible.

Regional Detention Standards include:

1. Detention basin outlet capacity shall be based on the downstream channel capacities (existing or Master Planned) with consideration given to inflows occurring below the detention basin.

2. All detention basins are required to properly function under all debris and sedimentation conditions.

3. In-channel detention basins typically will be required to safely pass the PMF discharge as a minimum. HMR 49 (1977) shall be used to calculate PMF flows.

4. Detention ponds shall be designed to include provisions for security/public safety.

5. Basins should be drained in not more than 7 days with the preferred standard drain time set at 24 hours. (Drain time is defined as the time

from the end of precipitation until the basin is drained of 90 percent of design capacity.)

6. A minimum of 1 foot of freeboard is required above the emergency spillway design water surface elevation. (See **Figure 1201**.)

7. Basins shall be self-regulating (passive).

8. Dams greater than 20 feet in height or impounding more than 20 acre-feet of water must be approved by the State Engineer.

9. Inflows shall be based on ultimate development conditions and Master Planned tributary area.

10. Design of all detention basins shall include emergency spillways.

11. Embankment protection will be considered for each basin.

1202.2 Local Detention

Since the functions of local minor and local major detention facilities are different, the development guidelines for each are described separately below:

1202.2.1 Local Minor Detention

Local minor detention may be required for developments in hydrologic basins of less than 20 acres in size. The need for local minor detention is based on analysis of downstream conveyance (e.g., street or storm sewer system capacity) and/or pre- and post-development hydrology.

Local Minor Detention Guidelines include:

1. Public safety should be paramount in all designs.

2. Accommodation of debris and sedimentation should be considered in all designs.

Local Minor Detention Standards include:

1. Post-development peak discharges must not exceed pre-development discharges if downstream facilities lack adequate capacity to handle the increased flow rates.

2. Basins must drain completely in less than 24 hours.

3. A minimum 1 foot of freeboard is required above the major design storm water surface elevation.

1202.2.2 Local Major Detention

Local major detention (typical storage > 1.0 acre-feet) may be required in accordance with Section 303.7 or where upstream off-site flows must be intercepted and controlled to protect the development. Design of such basins should be coordinated with the local entity.

Local Major Detention Guidelines include:

1. Off-channel detention basins are preferred.
2. All basins are required to properly function under debris and sedimentation conditions. Adequate access must be provided for the necessary equipment to periodically remove accumulated sediment and debris.
3. Multi-use (e.g., recreation) can be considered for all detention basins.
4. Below-grade detention basins are preferred to above-grade detention basins.

Local Major Detention Standards include:

1. Detention basin outlet capacity will be based on either (a) downstream conveyance system capacities with consideration given to inflows below the detention basin or (b) pre- and post-development hydrology.
2. Detention basins shall be drained in not more than 3 days with the preferred drain time set at 24 hours.
3. A minimum of 1 foot of freeboard will be required above emergency spillway design water surface elevation or as required by the State Engineer.
4. Detention basins will be passive.
5. Emergency outlets will be incorporated on all detention basins.

1202.2.3 Local Minor Retention

Local minor retention may be required for containing stormwater in the event downstream conveyance is unavailable or detention is infeasible. The purpose of a retention basin is to temporarily store runoff and allow for infiltration into the underlying soils. Local minor retention basins are defined as serving hydrologic basins smaller than or equal to 20 acres. Local major retention basins are not recommended.

Local Minor Retention Guidelines include:

1. Public Safety should be paramount in all designs.

2. Flat terrain is the preferred location for a retention basin.

3. The basin shall be below-ground and have dimensions that maximize infiltration.

4. Soil permeability shall be determined in the soil layer with the minimum permeability.

5. Soil shall have a permeability equal to **or** greater than 1-inch per hour.

6. Soil permeability should be determined using percolation or “Perk” tests used to design septic systems or equivalent.

7. The depth of bedrock and/or groundwater shall be a minimum of 5 feet below the design bottom elevation of the basin at all times.

8. The basin shall be designed to allow bypassing of the peak runoff in the event the facility clogs. This bypass can be provided by overland relief.
9. Accommodation of debris and sedimentation should be considered in the design.

10. The basin shall be designed to contain the volume of runoff generated by the peak discharge and the volume of sediment accumulated in a three year period.

11. Designs should be based on ultimate development conditions.

12. Erosion protection shall be considered for side slopes and inlet works.

13. Adequate access must be provided for the necessary equipment to periodically remove accumulated sediment and debris.

14. Designs shall include an analysis of groundwater effects of the completed and operating basin on the surrounding groundwater levels, since change in the groundwater could adversely impact neighboring facilities including basements, septic systems and existing wells.

15. Permanent structures such as buildings and roads and other surcharge loads shall be located a safe distance away from the basin.

1203 HYDROLOGIC DESIGN METHODS AND CRITERIA

The hydrologic design of detention facilities is based on the type of facility (regional versus local) and the method used to estimate the runoff (HEC-1 and SCS TR-55 versus Rational Method). If HEC-1 or SCS TR-55 is used, a full hydrograph is available for traditional storage routing (Section 1203.3.1). If the Rational Method is used, a simplified triangular procedure has been developed as described below.

1203.1 Inflow Hydrograph

The determination of required detention storage is based on volume calculations derived from the inflow hydrograph, along with the maximum outlet flow. The inflow hydrograph shall be based on ultimate development conditions.

1203.1.1 HEC-1 Method

The hydrograph for local and regional facilities may be calculated using HEC-1 or SCS TR-55 (Section 600). HEC-1 or SCS TR-55 can calculate a hydrograph for any location in the hydrologic basin. The HEC-1 data input file must be structured so that the proposed detention basin site is a hydrograph routing or hydrograph combining point. For specific model input format, see the HEC-1 User's Manual.

1203.1.2 Modified Rational Method

For the design of local minor detention facilities in hydrographic areas of less than 150 acres, a simple, "triangular" hydrograph will be developed using the Modified Rational Formula Method. The application of the Modified Rational Formula Method is described in Section 604.

The Rational Method is traditionally used solely for peak runoff estimation, but a hydrograph can be constructed using the following assumptions:

a) Peak Flow Occurs at the t_c;

b) Flow Increases Linearly from $Q = 0$ to $Q = Q_{peak}$ for
$T = 0$ to $t = t_c$;

c) Flow Decreases Linearly from $q = Q_{peak}$ to $Q = 0$ for
$t = t_c$ to $t = 2t_c$.

The resulting hydrograph is triangular in shape and has a volume given by

$$V = 60 (t_c * Q_p) \quad (1201)$$

Where V = Volume in ft^3

t_c = Time of Concentration in Minutes

Q_p = Peak Flow Rate in cfs

1203.2 Detention Basin Design Outflow Limitations

The controlled outlet capacity has direct influence on size of the basin. The outflow limitation can be based on existing undeveloped peak flow from the hydrologic limitations in the capacity of the downstream conveyance on a hydrologic analysis of local conditions.

1203.2.1 Regional Facilities

The allowable release rate for regional facilities in the Master Plan is based on the non-damaging capacity of the downstream conveyance system or on the conveyance capacity of the system as improved by the detention project. The design maximum outlet capacity of a regional facility must be coordinated with the CCRFCD.

1203.2.2 Local Facilities

The outflow limitation for local facilities is stated in Section 303.7. Existing flow conditions will be calculated based on development conditions that exist prior to construction of project. The allowable outlet rate is equal to the existing peak runoff rate.

1203.3 Hydrologic Calculation Methods

After the inflow hydrograph has been calculated (1203.1) and the outflow limits (1203.2) have been established, the storage volume requirement can be estimated. Separate methods for calculating required storage are used depending on the method used to estimate the inflow hydrograph.

1203.3.1 HEC-1 Method

In order to calculate the required storage volume at a particular detention basin site, the following information must be available or prepared:

a) Inflow hydrograph

b) Outlet capacity limitation

c) Proposed outlet discharge versus elevation data for the proposed basin site

d) Proposal storage versus elevation data for the proposed basin site

e) Proposed drain time for the proposed basin site

The HEC-1 computer program can be used to determine the required storage volume and outflow limitation based on a reservoir routing procedure. The data described above is added to the existing HEC-1 data set as described in HEC-1 Users Manual. Initial estimates of outlet size are made and the program is run. The output is reviewed and changes are made to the outlet configuration as needed until the desired degree of flood peak attenuation and acceptable drain time is achieved. This method is shown in the example in Section 1208.1.

The storage-routing determination can also be performed manually by the modified Puls method described in Section 609. Using data for the inflow hydrograph, the storage versus elevation data for the proposed site and the outlet limits, the outflow hydrograph from the proposed detention facility can be predicted.

1203.3.2 Rational Method

After the inflow hydrograph (1203.1) and the outflow limitation (1203.2) have been determined, the required storage volume can be calculated. The estimated hydrograph is plotted at a suitable scale. The maximum outflow rate is plotted on the receding limb of the hydrograph. A straight line is constructed from the origin to the outlet limit on the receding limb. The area above this line is the required storage volume. The estimation of required storage volume is shown in the example in Section 1208.2.

1204 HYDRAULIC CALCULATIONS

This section describes the methods to be used to size outlet structures for detention facilities. Although the methods presented are recommended for the hydraulic structures described, alternative hydraulic techniques may be more appropriate depending upon the configuration of the outlet structure.

1204.1 Low Flow Outlets

The low flow outlet (principal spillway) is sized to control discharge from a basin as set forth in Section 1203.2.

In traditional detention basins, outlet control is usually provided by a culvert or large (> 18-inch diameter) pipe conduit. The types of low flow control typically used for parking lot detention are small under-sidewalk weirs or pipes.

1204.1.1 Minimum Conduit Size

To reduce the potential for outlet clogging by debris, minimum conduit sizes have been set for the CCRFCD area. The minimum conduit size for use in detention facilities is 18-inch diameter or equivalent. Orifice plates may be utilized to reduce flows from these minimum pipe sizes.

1204.1.2 Flow Calculations

The capacity of outlets shall be calculated using nomographs in Section 1000.

The capacity of a small closed conduit (Section 1000 nomographs are not applicable) is estimated assuming inlet control using the orifice equation shown below:

$$Q = CA\,(2gh)^{1/2} \quad (1202)$$

Where Q = Discharge in cfs

A = Cross-sectional Area of Conduit in ft^2

g = Gravitational Constant (32.2 ft/sec^2)

h = Head, in ft, Above Centerline of Orifice Opening

C = Orifice Coefficient (0.65)

The orifice coefficient to be used in all calculations is 0.65, unless deviation is approved by local entity. An example of this calculation is provided in Example 1208.1.

The capacity of a weir can be estimated using the following equations:

1. For Horizontal crested weirs:

$$Q = CLH^{3/2} \quad (1203)$$

Where Q = Flow in cfs

C = Weir Coefficient

L = Horizontal Length of Weir in ft

H = Head, in ft, Above Weir Crest

2. For V-notched weirs:

$$Q = C\,(8/15)\tan(\theta/2)\,H^{5/2} \quad (1204)$$

Where Q = Flow in cfs

C = Weir Coefficient

θ = Angle of V-notch in Degrees

H = Head, in ft, Above Weir Crest

For the horizontal sharp crested weirs, the weir coefficient can be taken to be 3.1 while the coefficient for broad crested weirs can be taken to be 3.0. The V-notch weir coefficients are provided in **Figure 1202**.

1204.2 Spillways

Since storm flows may enter a detention facility in excess of the maximum design flow of the outlet works, a safe method of passing these flows must be provided. All detention facilities must have the ability to pass flows in excess of the major design storm without endangering the structural integrity of the facility or diverting flows from their historic drainage pattern.

A detention basin may have more than one spillway, or in the case of local facilities, the complete structure may be designed to act as an overflow section. If a basin has only one spillway, it must be able to pass both the design flow and a larger flow to provide a margin of safety. These larger flows are discussed in Section 1202. If the geometry of the basin site does not allow for a single spillway to serve these two flows, two spillways may be provided. The principal spillway will be designed to handle the major design storm flow. It flows greater than the major design storm flow, the emergency spillway would allow these greater flows to be passed safely. For minor local detention structures, the structure may be designed to be safely overtopped and the structure itself is the emergency spillway.

1204.2.1 Sizing Requirements

All detention basins in the CCRFCD region shall have emergency spillways which safely pass the following peak flow rates:

1. Regional Facilities: The spillway will be required to pass, as a minimum, a hydrograph developed by using twice the adjusted point precipitation of the major storm if approval of the State Engineer's Office is not required (1206.1).

2. Local Major Facilities: The spillway will be required to pass, as a minimum, a hydrograph developed by using twice the adjusted point precipitation of the major storm if approval of the State Engineer's Office is not required (1206.1).

3. Local Minor Facilities: Emergency spillways for local minor facilities shall be designed to pass the major storm.

1204.2.2 Flow Calculations

The equation for flow over a spillway is the same as that for flow over a sharp crested weir given in Section 1204.1.2, although the discharge coefficient, C, for broad or ogee-crested weirs is normally used in design. A graph for coefficient estimation for ogee-crested weirs is provided as **Figure 1203**.

1205 DEBRIS AND SEDIMENTATION

The performance and reliability of detention facilities can be reduced by natural and man-made debris. Naturally occurring sedimentation can over a period of time reduce the storage capacity of a detention basin and thereby reduce the degree of flood protection provided. The obstruction of low flow conduits by debris can reduce outlet capacity and cause the premature filling of the detention basin with storm water, again reducing the flood protection provided by the structure. Consequently, adequate care must be exercised in design to provide for protection of the outlet works from debris and for the control and removal of sedimentation in the basin.

1205.1 Trash Racks

All outlet works and low flow conduits shall be provided with a trash rack for debris control. The trash rack shall provide a maximum bar spacing not to exceed two-thirds of the outlet opening or diameter. The total area of the trash rack shall allow for passage of the design flow with 50 percent of the trash rack blocked. Examples of common trash rack designs are provided in **Figure 1204**. Calculations for head losses through a trash rack shall be included in the outlets hydraulic evaluation.

1205.2 Sedimentation

The storage volume of a detention basin can be reduced and/or eliminated by sediment deposition. Depending on the cover and soil conditions in a watershed, detention basin filling may happen slowly over a period of many years or, in extreme cases, during one storm event.

Sedimentation effects may be reduced by the construction of debris basins (Section 1300) upstream of the detention facility or by providing additional storage capacity in the detention facility for storage of sediment. Section 1300 presents some basic information regarding debris sedimentation, control, facilities.

1206 DESIGN STANDARDS AND CONSIDERATIONS

The following section describes current standards and special considerations for detention design.

1206.1 Dam Safety

All dams which store more than 20 acre-feet of water or have an embankment 20 feet or greater in height must be approved by the State Engineer.

1206.2 Grading Requirements

All detention facilities will be graded to allow for complete drainage by the low flow outlet of the principal spillway. No permanent standing water will be allowed. Minimum grade is 0.5 percent.

1206.3 Depth Limits

The maximum ponding depth for parking lot detention facilities is 18 inches.

1206.4 Trickle Flow and Basin Dewatering

All detention basins shall include provisions for a concrete low flow channel and/or a storm drain to ensure positive dewatering of the basin. Low flow criteria are presented in Section 705.

1206.5 Embankment Protection

Embankments shall be protected from structural failure from overtopping. Overtopping can be caused by a larger than design inflow or from obstruction of the low flow outlet. Embankment protection may be provided by embankment armoring (i.e., riprap) or by a design overflow section (i.e., emergency spillway). The invert of the emergency spillway shall be set equal to or above the major design storm water surface elevation.

1206.6 Maintenance Requirements

All detention facilities will be designed to minimize required maintenance and to allow access by equipment and workers to perform maintenance. Maintenance for facilities on public lands or within dedicated easements will be maintained by the local entity. Regional facilities will be maintained by the local entities and may be eligible for funding by CCRFCD. Facilities on private land will be the responsibility of the owner. The local entity reserves the right to perform required maintenance on facilities located on private land and charge the owner for the cost of such maintenance. Sediment must be removed from detention facilities with water quality features before sediment accumulation is 50% of the height of the water quality feature.

1206.7 Local Detention Basin Siting Guidelines

Local detention basins should be located as to minimize the impact on the site and to ensure public safety. Basins should not be located adjacent to buildings due to the potential of saturating foundation materials. To ensure public safety, basins should not be located adjacent to pedestrian walkways. Basins should also be placed to minimize detrimental impact on public facilities (e.g., roadway and sidewalk deterioration).

1207 WATER QUALITY

One of the requirements of the Las Vegas Valley Municipal Separate Storm Sewer System (MS4) permit is to address water quality impacts from new development and significant redevelopment (NDSR). NDSR properties are those that disturb areas ≥ 1 acre, including projects with areas <1 acre that are part of a larger common plan of development or sale, that discharge to the MS4. The Las Vegas Valley Storm Water Management Plan (SWMP) Technical Memorandum IV.11, *Potential Water Quality Retrofits of Regional Detention Basins – Strategic Plan*, (MWH 2011) (TM IV.11), suggests using existing and proposed detention basins for water quality management in each major watershed in the Las Vegas Valley. Technical Memorandum IV.16, *Strategic Plan for Use of Regional Detention Basin for Water Quality Management*, (MWH 2012) (TM IV.16) further expands on water management for regional detention basins. This section describes design measures for incorporating water quality features into existing and proposed detention basins. The two TMs from the SWMP should be used to supplement to this section.

Water quality for existing and proposed detention basins will be managed primarily by incorporating extended detention basins (EDBs) within the regional detention basins. An EDB is similar to a flood control detention basin with a water quality outlet to extend draining time. Recommendations for water quality outlet sizing and other design details are available from Denver Urban Drainage and Flood Control District (2008). The long drain time provides quiescent conditions for fine sediment and pollutants associated with that sediment to settle to the basin floor. EDBs are commonly combined with flood control detention basins as the lowest level of water storage. The water treated by these EDBs is referred to as water quality capture volume (WQCV). EDBs should detain a WQCV for a minimum of 24 hours and up to a maximum of 48 hours.

1207.1 Water Quality Capture Volumes

Calculations for sizing water quality capture volumes for detention basins should use the maximized detention volume calculation (per Water Environment Federation [WEF] Manual of Practice [MOP] No. 23), a 24-hr drain time and the appropriate adjustment factor as described below. The MOP uses 0.37 inches for the Las Vegas Valley for the average rain fall depth (**Figure 1206)**. This average was used in the equation below.

The following empirical equations are used to calculate the water quality capture volume, is computed by multiplying P_o by the area of new development.

Equation 1. Calculation of Runoff Coefficient

$$C = 0.858i^3 - 0.78\, i^2 + 0.774i + 0.04$$

i = watershed percent imperviousness

Equation 2. Calculation of Maximized Detention Volume

$$P_o = (a \times C) * P_6$$

P_o = maximized detention volume in watershed inches
a = constant, for event maximization, drain times 24 hours (1.299)
C = runoff coefficient
P_6 = event average rainfall depth, for Las Vegas = 0.37 inches

The volumes determined using the empirical methods are reasonable for smaller catchments but overestimate volumes from larger catchment areas. For drainage areas over 1.0 square mile an adjustment factor is needed to replicate results from modeling of larger drainage areas. Adjustment factors based on depth-area reduction factors shown below should be used to reduce the WQCV results from Equation 2 for larger drainage areas.

Tributary Area (sq miles)	Adjustment Factor
0 - <1	1.00
1 - <10	0.85
10 - <40	0.75
40 and greater	0.65

The estimated total WQCV needed to treat all potential new development in Las Vegas Valley is 1,439 acre-feet. TM.IV.16 and any corresponding updates should be used to obtain the WQCV needed at each proposed and existing detention basin based on potential new development in the upstream watershed. The WQCV developed in one watershed beyond that watershed's required WQCV based on potential new development can be counted toward the WQCV deficit in another watershed through a pollutant trading approach. The pollutant trading approach allows runoff treated from currently developed areas to be counted as a credit towards the required total WQCV to address future developed areas. In practice, the WQCV for all new and modified regional detention basins should be maximized based on site and hydrologic constraints.

1207.2 Water Quality Feature Options

The primary method of installing water quality treatment features in regional detention basins is to create an EDB within the basin with the necessary WQCV for the upstream developable area. Another likely retrofit option is to modify outlets (e.g., with perforated riser pipe) to allow it to drain slowly, over approximately 12 – 48 hours, for minor storms.

Recommendations for water quality outlet perforation sizing and other design details are available from Denver Urban Drainage and Flood Control District (2008). The amount of water detained upstream of a water quality outlet is the WQCV. This amount of water would be stored in addition to the flood control storage of a regional detention basin.

The addition of sediment forebays near the detention basin inlets is another design option. This would provide an opportunity for larger particles to settle from the flow. Sediment forebays are designed based on sediment capture requirements described in Section 1205 and Section 1300 of the HCDDM, and are typically hard surface-lined so that material can be removed by heavy equipment.

1207.3 Existing Detention Basin Retrofit

Retrofits for existing detention basins to incorporate stormwater quality treatment features should be included in any planned construction and upgrades when possible to keep total design and construction costs to a minimum.

Retrofits should use existing capacity that does not take away from 100-year flood storage where possible to avoid the cost of additional excavation. Existing capacity opportunities include sediment storage and existing capacity that is not used for 100-year flood storage. A list of existing detention basins in 2012 with freeboard of more than 3 feet between the 100-year storage level and the emergency spillway is included in TM.11. Sediment capacity included in detention basin design must be determined through review of design documents for each facility.

1208 EXAMPLE APPLICATIONS

1208.1 Example: Detention Pond Outlet Sizing

<u>Problem:</u> Size the principal and emergency spillway for a detention pond given the following information:

Inflow hydrograph in **Table 1201 (A)**

Basin Site characteristics in **Table 1201 (B)**
Outflow limitation of 300 cfs (Major Storm)
Emergency spillway design flow = 1,000 cfs

Solution:

Step 1: Size Low Flow Conduit:

$$Q = C_d A (2gh)^{1/2}$$

$$300 \text{ cfs} = 0.65 A (2gh)^{1/2}$$

$$A = 21.8 \text{ ft}^2$$

Diameter = 5.3 ft, Use 72 in RCP

Step 2: Develop depth-outflow data for low flow conduit as presented in **Table 1201 (C)**.

Step 3: Perform storage routing using HEC-1. The input data listing and resulting outflow summary is presented in **Table 1202**.

The results show that a storage volume of 31.4 acre-feet is sufficient to limit the pond outflow to less than 300 cfs (actual outflow = 302 cfs).

Step 4: Size Emergency Spillway

Assume H = 2.0 ft

For a Broad Crested Weir, $C_d = 3.0$

$1{,}000 \text{ cfs} = 3.0 L (2.0)^{1.5}$

L = 117.9

Use 120 ft

Step 5: The actual water surface elevation for the emergency spillway design flow is then found by repeating the storage routing procedure for the required emergency spillway design hydrograph.

1208.2 Example: Rational Formula Detention Method

Problem: Determine the required detention volume given the following parameters:

Peak flow from Modified Rational Formula Method is 29 cfs

Time of Concentration is 15.2 min

Outflow is Limited to an Existing Flow Rate of 13 cfs

Solution:

Step 1: Plot triangular hydrograph as described in Section 1203.1.2 (see **Figure 1205**).

Step 2: Plot outflow limitation of 13 cfs on falling limb of hydrograph (Point D on **Figure 1205**).

Step 3: Calculate area under triangle above line A-D (**Figure 1205**)

$$V = 14,592\ ft^3$$

1208.3 Example: WQCV in Regional Detention Basins

Problem: Determine the required WQCV for Sample Detention Basin in the Pittman Watershed. Facility size is 377 acre-feet.

Solution:

Step 1: Determine the WQVC needed for all the developable land in the Pittman Watershed.

a. The Watershed area is determined from the MPU.
b. The Percent Impervious for planned development conditions is determined from the MPU.
c. Use the Maximized Detention Volume table (**Figure 1207A**) to determine $P_{o.}$
d. The maximized WQCV values is the result of the developable vacant land watershed area times the P_o, reported in acre-feet.
e. The adjustment factor is based on the Tributary Area Adjustment Factor table (**Figure 1207A**).
f. The WQCV for the watershed is calculated by the maximized WQCV times the Adjustment Factor.

Determine the WQCV for the tributary area (area upstream of facility).

a. The tributary area is determined from the MPU.
b. The Percent Impervious for planned development conditions is determined from the MPU.
c. Repeat steps C – E above.
d. The WQCV for the tributary area is calculated by the maximized WQCV times the Adjustment Factor.

Determine if developing the greater of the two WQCV values (WQCVWatershedA or WQCVTribA) is feasible for Sample Detention Basin. If not, move to Step 2.

Watershed Developable Land WQCV

Watershed Area	46	sq. miles
Planned % Impervious	48%	

Maximized Detention Volume (inches)

P_o =	0.16	inches
Maximized WQVC =	393	acre-feet

Adjusted Detention Volume

Adjustment Factor	0.65	
WQCV $_{WatershedA}$ =	**255**	**acre-feet**

Detention Basin Tributary Area WQCV

Tributary Area	7.7	sq. miles
Planned % Impervious	49%	

Maximized Detention Volume (inches)

P_o =	0.16	inches
Maximized WQCV =	66	acre-feet

Adjusted Detention Volume

Adjustment Factor	0.85	
WQCV $_{TribA}$ =	**56**	**acre-feet**

Step 2: Determine the WQCV for developable vacant land in tributary area. Developable vacant land excludes area outside the ultimate development boundary.

a. Developable vacant land area is determined from the MPU or estimated based on aerial photography.
b. The Percent Impervious for planned development conditions is determined from the MPU.
c. Repeat steps C – E.
d. The WQCV for the developable vacant land is calculated by the maximized WQCV times the Adjustment Factor.

Tributary Area Developable Land WQCV

Vacant Land Area	3.0	sq. miles
Planned % Impervious	40%	

Maximized Detention Volume (inches)

P_o =	0.13	Inches
Maximized WQVC =	21	acre-feet

Adjusted Detention Volume

Adjustment Factor	0.85	
WQCV $_{VacantA}$ =	**18**	**acre-feet**

INFLOW HYDROGRAPH AND BASIN CHARACTERISTICS FOR EXAMPLE IN SECTION 1208.1

(A) INFLOW HYDROGRAPH

TIME (MINUTES)	RUNOFF (CFS)	TIME (MINUTES)	RUNOFF (CFS)
0	0	130	442
10	21	140	394
20	38	150	346
30	72	160	300
40	105	170	233
50	171	180	163
60	247	190	102
70	336	200	73
80	411	210	57
90	480	220	43
100	499	230	30
110	504	240	24
120	480	250	0

(B) BASIN CHARACTERISTICS

ELEVATION (feet)	SURFACE AREA (acres)
103	0
104	0.34
105	1.54
106	3.38
107	5.26
108	5.86
110	6.46
112	8.26

(C) 72" RCP DISCHARGE RATING

DEPTH	DISCHARGE
0	0
1	20
2	40
3	70
4	110
5	160
7	270
9	350

Revision	Date

HEC - 1 RUN FOR EXAMPLE IN SECTION 1208.2

HEC-1 INPUT

PAGE 1

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

  1         ID      CLARK COUNTY REGIONAL FLOOD CONTROL DISTRICT
  2         ID      SECTION 1208.1   EXAMPLE:  DETENTION POND OUTLET SIZING
  3         ID      WRC ENGINEERING, INC.   1/26/90
  4         IT      10       0       0     300
  5         IO       2       0       0
  6         QI       0      21      38      72     105     171     247     336     411     480
  7         QI     499     504     480     442     394     346     300     233     163     102
  8         QI      73      57      43      30      24       0

  9         KK       A  ROUTED DISCHARGE FROM DETENTION POND
 10         RS       1     STO       0
 11         SA       0     .34    1.54    3.38    5.26    5.86    6.46    8.26
 12         SE     103     104     105     106     107     108     110     112
 13         SQ       0      20      40      70     110     160     270     350
 14         ZZ
```

RUNOFF SUMMARY
FLOW IN CUBIC FEET PER SECOND
TIME IN HOURS, AREA IN SQUARE MILES

OPERATION	STATION	PEAK FLOW	TIME OF PEAK		PEAK STORAGE	TIME
				+	(AC-FT)	(HR)
					31.	2.67
HYDROGRAPH AT					PEAK STAGE	TIME
	A	504.	1.83			
ROUTED TO				+	(FEET)	(HR)
	A	302.	2.67		110.79	2.67

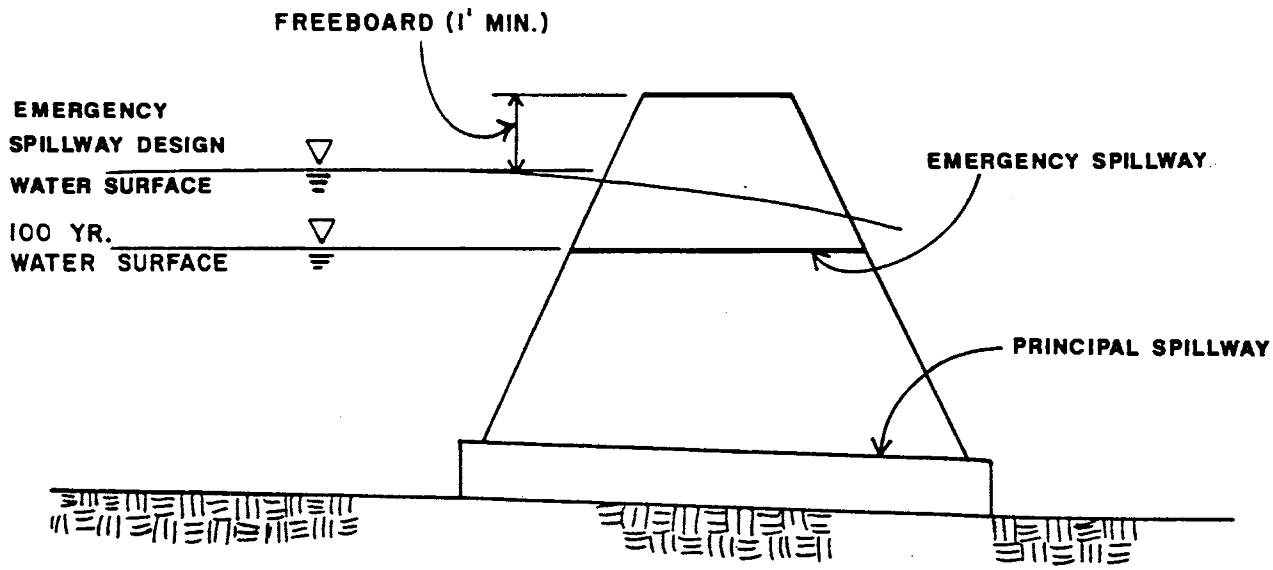
Revision	Date

WRC ENGINEERING

REFERENCE:

TABLE 1202

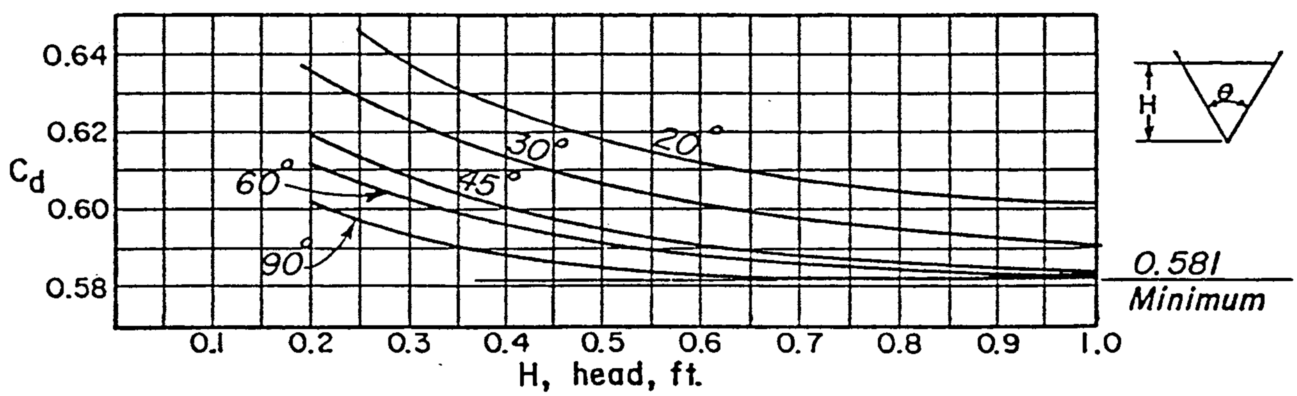
TYPICAL
BASIN GEOMETRY



NOTES: 1. For local minor detention facilities, the required one-foot freeboard shall be above the 100-year water surface elevation.

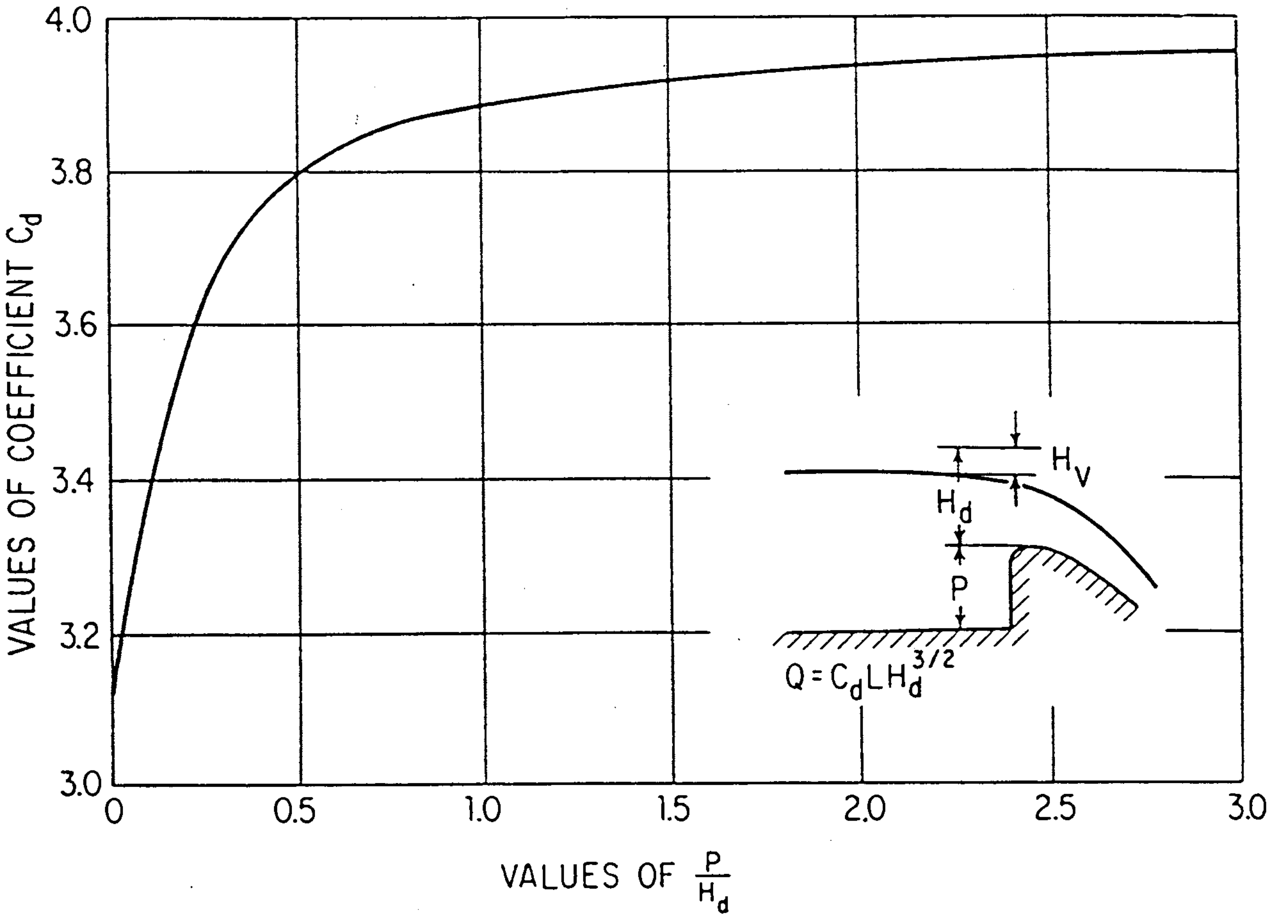
Revision	Date

V-NOTCH WEIR COEFFICIENTS

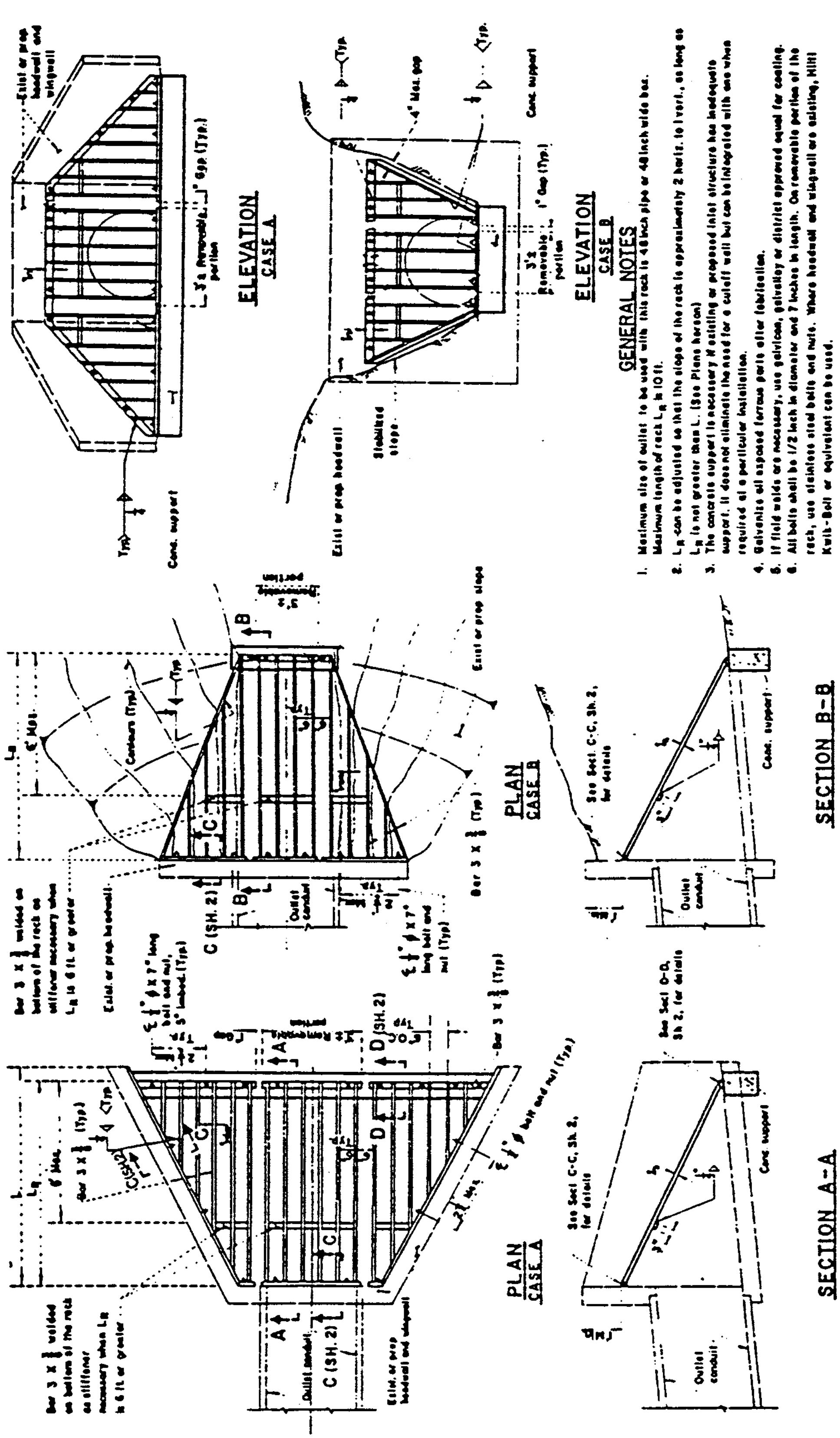


Revision	Date

OGEE-CRESTED WEIR COEFFICIENTS



Revision	Date

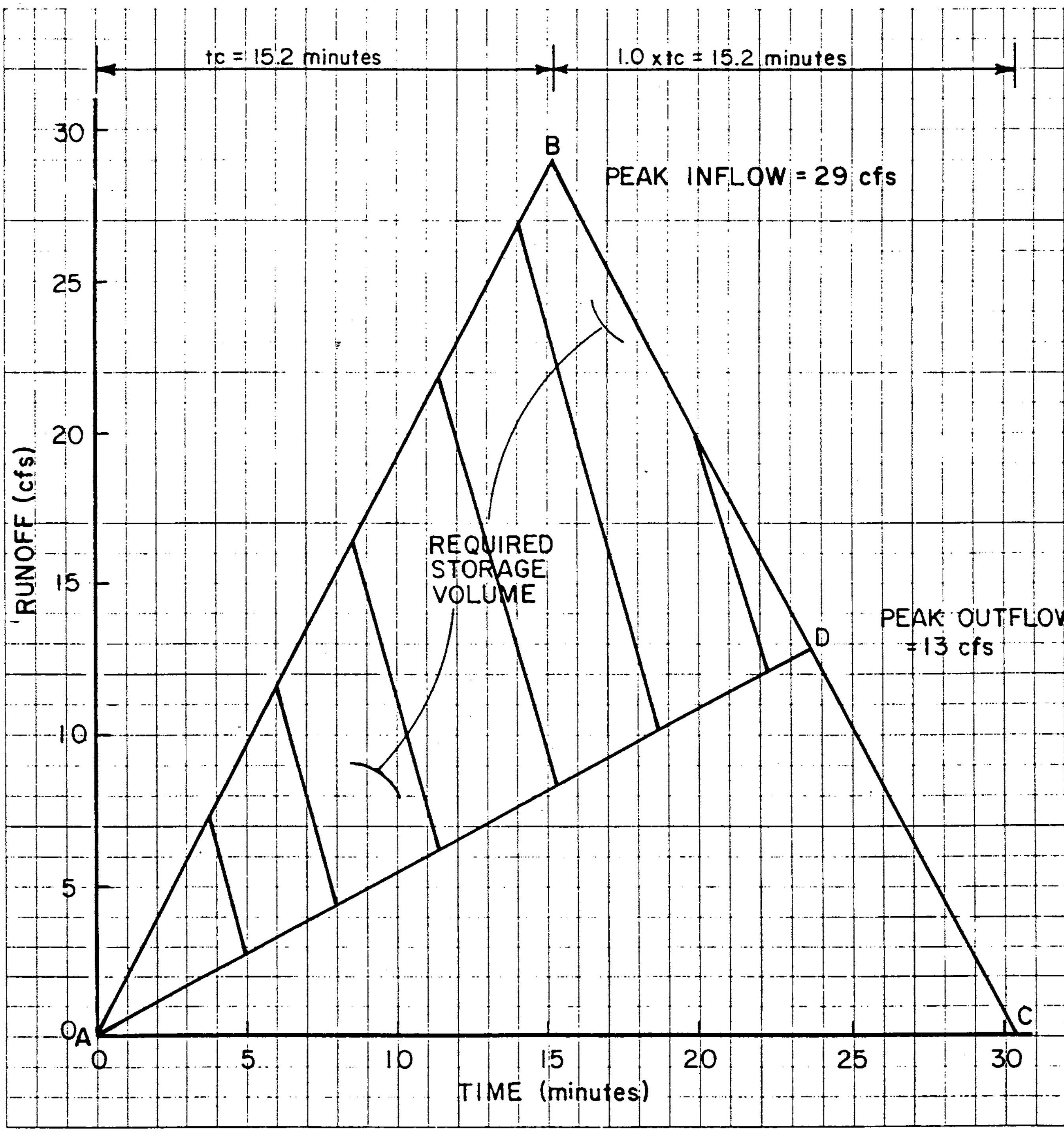


EXAMPLE TRASH RACK

Revision	Date

WRC ENGINEERING	**REFERENCE:** L.A. County Flood Control District, Design Manual - Hydraulic, 1982.	FIGURE 1204

HYDROGRAPH FOR EXAMPLE IN SECTION 1208.2



Revision	Date

U.S. Mean Annual Runoff-producing Rainfall Depths

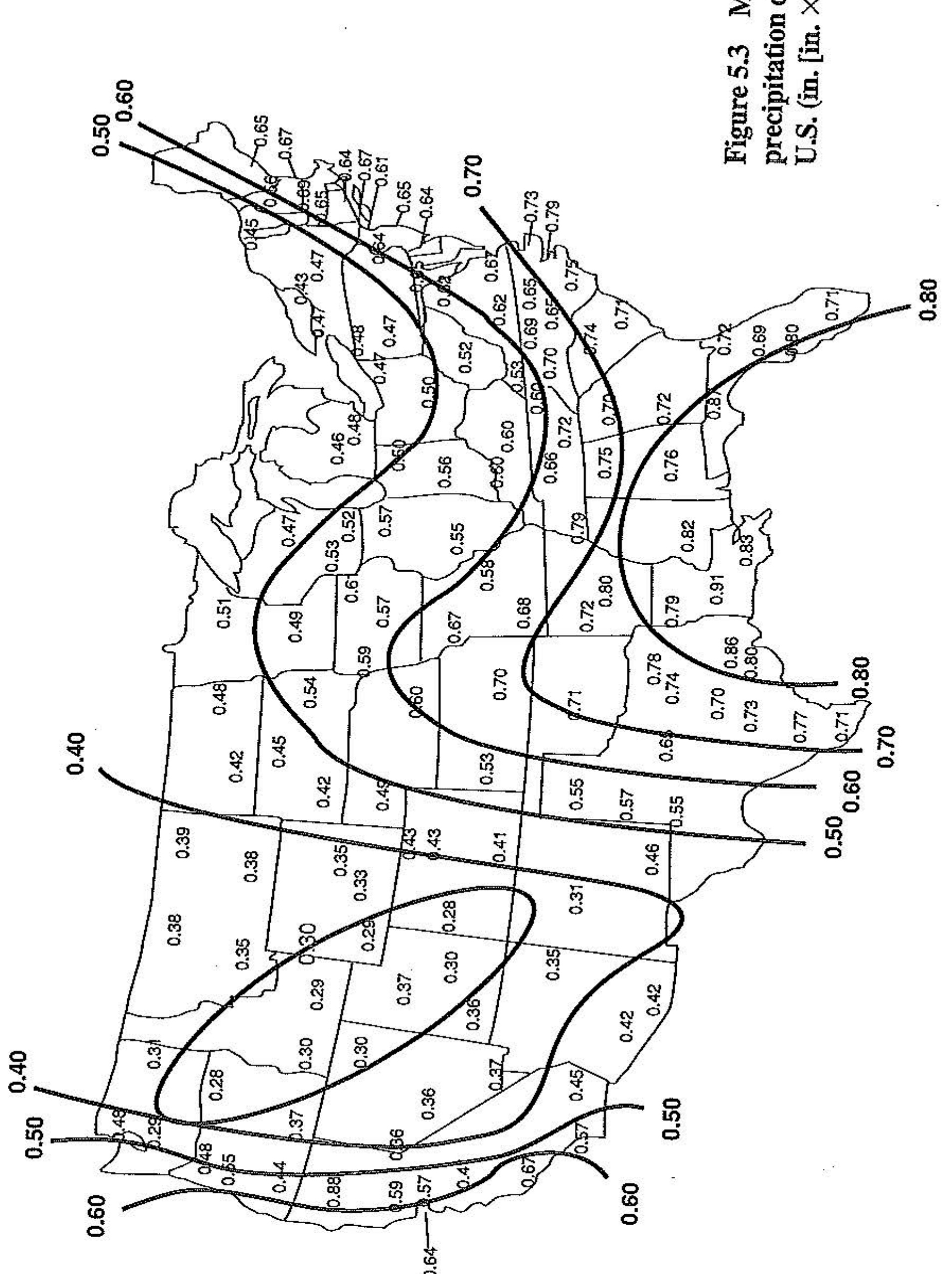


Figure 5.3 Mean storm precipitation depth in the U.S. (in. [in. × 25.4 = mm]).

Revision	Date

REFERENCE: Urban Runoff Quality Management
WEF Manual of Practice No. 23,
ASCEM Anual and Report on Engineering
Practice No. 87 June 1998, Page 176

FIGURE 1206

CALCULATIONS FOR SIZING WATER QUALITY CAPTURE VOLUME (WQCV) IN REGIONAL DETENTION BASINS

Determine WQCV Using Equation I and Equation II from ***Urban Runoff Quality Management***
(ASCE Manual No. 87)

Equation I: Determine Runoff Coefficient

$C = 0.858i^3 - 0.78i^2 + 0.774i + 0.04$

C = Runoff Coefficient

i = watershed percent impervious

Equation II: Determine Maximized Detention Volume (in inches)

$P_o = (a*C)*P_6$

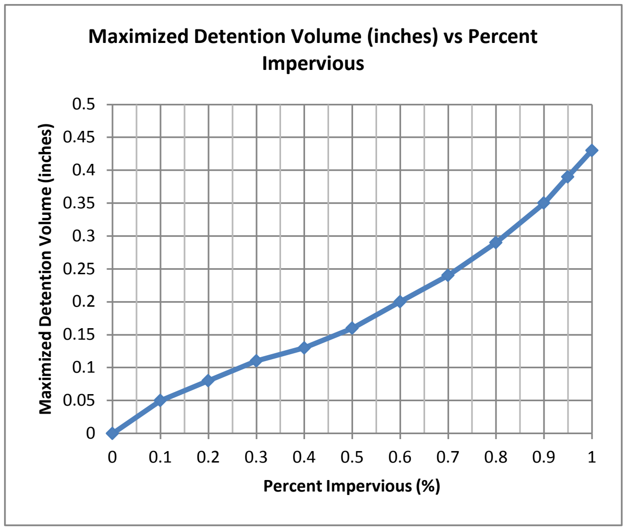
P_o = maximized detention volume in watershed inches

a = constant, for event maximization, drain time 24 hours (1.299)

P_6 = event average rainfall depth, for Las Vegas (0.37 inches)

Maximized Detention Volume Table

Impervious Ratio	C	P_o
0.00	0.00	0.00
0.10	0.11	0.05
0.20	0.17	0.08
0.30	0.23	0.11
0.40	0.28	0.13
0.50	0.34	0.16
0.60	0.41	0.20
0.70	0.49	0.24
0.80	0.60	0.29
0.90	0.73	0.35
0.95	0.81	0.39
1.00	0.89	0.43



Tributary Area Adjustment Factor Table

Area Range (sq. mi.)	Adjustment Factor
0 - <1	1.00
1 - <10	0.85
10 - <40	0.75
40 +	0.65

Revision	*Date*

REFERENCE: MWH

FIGURE 1207A

CALCULATIONS FOR SIZING WATER QUALITY CAPTURE VOLUME (WQCV) IN REGIONAL DETENTION BASINS

METHOD TO CALCULATE WATER QUALITY CAPTURE VOLUME

Step I. **Determine the following:**

a) WQCV for Watershed Developable Land

b) WQCV for Tributary Area (area upstream of facility)

Determine if the greater of the two values is feasible. If not, determine if the lesser value is feasible. If "no" to both, move to Step II.

Step II. **Determine the following:**

a) WQCV for Tributary Area Developable Land (vacant area upstream of facility)

Use the maximum volume feasible in the range of values determined in Step I and Step II.

CALCULATIONS

Watershed:

Facility:

Facility Size:

Step I

Watershed Developable Land WQCV

Watershed Area sq. miles

Planned % Impervious

Maximized Detention Volume (inches)

P_o = inches

Maximized WQCV = acre-feet

Adjusted Detention Volume

Adjustment Factor

$WQCV_{WatershedA}$ = **acre-feet**

Detention Basin Tributary Area WQCV

Tributary Area sq. miles

Planned % Impervious

Maximized Detention Volume (inches)

P_o = inches

Maximized WQCV = acre-feet

Adjusted Detention Volume

Adjustment Factor

$WQCV_{TribA}$ = **acre-feet**

Step II

Tributary Area Developable Land WQCV

Developable Vacant[a] Land Area sq. miles

Planned % Impervious

Maximized Detention Volume

P_o = inches

Maximized WQCV = acre-feet

Adjusted Detention Volume

Adjustment Factor

$WQCV_{VacantA}$ = **acre-feet**

[a] Developable vacant land excludes area outside the Ultimate Development Boundary

Revision	*Date*

REFERENCE: **MWH**

CLARK COUNTY REGIONAL FLOOD CONTROL DISTRICT
HYDROLOGIC CRITERIA AND DRAINAGE DESIGN MANUAL

SECTION 1300
EROSION AND SEDIMENTATION

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1301 DEBRIS CONTROL STRUCTURES AND BASINS

1301.1 Introduction

Debris transported by storm water can cause severe problems with flood control structures and other public facilities. Debris-related problems include: clogging of channels and culverts, filling of detention ponds, and burial of or physical damage to roadways and other property. Consequently, the need for debris control is an essential consideration in the design of hydraulic structures, particularly culverts and detention basin outlets.

In order to select an appropriate debris-control measure, the debris within a particular basin should be classified. A classification used by the U.S. Department of Transportation (USDOT, 1971) follows:

1. Light floating debris -- small limbs or sticks, orchard prunings, tules and refuse

2. Medium floating debris -- limbs or large sticks

3. Heavy floating debris -- logs or trees

4. Flowing debris -- heterogeneous fluid mass or clay, silt, sand, gravel, rock, refuse, or sticks

5. Fine detritus -- fairly uniform bedload of silt, sand, gravel more or less devoid of floating debris, tending to deposit upon diminution of velocity

6. Coarse detritus -- coarse gravel or rock fragments carried as channel bedload at flood stage

Debris can be controlled by three methods: (a) interception near the debris source or above a critical hydraulic structure downstream of the source: (b) deflecting the debris for detention near (usually above) a culvert or inlet; or (c) passing the debris through the channel or inlet structure. Commonly used structures for controlling various types of debris are listed in **Table 1301** and described in the following sections.

1301.2 Debris Deflectors

Debris deflectors are used to divert medium and heavy floating debris and large rocks from the culverts (or other inlets) for accumulation in a storage area and subsequent removal after the flood subsides. The storage area must be adequate to retain the anticipated type and quantity of debris during any one storm or between clean-outs. Typical debris deflectors for culvert protection are shown in **Figure 1301**.

1301.3 Debris Racks

Debris racks provide barriers across stream channels to stop debris that is too large to pass through downstream channels or culverts. Debris racks vary greatly in size and in construction material. Height of racks should allow some freeboard above the expected depth of flow in the upstream channel for the design flood. Racks should not be placed in the plane of the culvert entrance, since they induce plugging when thus positioned. Access to the rack is necessary for maintenance.

The rack should be placed well upstream from the culvert or improved channel inlet in those situations where a well-defined upstream channel exists. However, they should not be placed so far upstream that debris enters the channel between the rack and the inlet. Typical debris racks for use with small to medium-sized culverts and on improved channels are shown in **Figure 1302**.

1301.4 Debris Risers

Debris risers generally consist of a vertical culvert pipe and are usually suitable for installations of less than 54-inch diameter. Risers are normally used with detention ponds or debris basins or where a considerable height of embankment is available above a culvert crossing. The riser is particularly effective where debris consists of flowing masses of clay, silt, sand, sticks, or medium floating debris without boulders. Risers are seldom structurally stable under high-velocity flow conditions because of their vulnerability to damage by impact. A typical debris riser is shown in **Figure 1303**.

1301.5 Debris Cribs

Debris cribs are particularly adapted to small-size culverts where a sharp change in stream grade or constriction of the channel causes deposition of detritus at the culvert inlet. The crib is usually placed directly over the culvert inlet and in "log cabin" fashion.

Figure 1304 shows the general dimensional details of a typical debris crib. Spacing between bars should be about 6 inches. A crib may be open or covered with horizontal top members spaced equal to the crib members. Debris can almost envelop a crib without completely blocking the flow and plugging the culvert. When an open crib is used as a riser and an accumulation of detritus is expected, provision can be made for increasing the height.

1301.6 Debris Dams and Basins

On channels carrying heavy sediment and debris loads, it is often economically impracticable to provide culverts large enough to carry surges of debris. If the height of an embankment and storage area are not sufficient for a riser or crib, a debris dam and/or basin placed some distance upstream from the culvert may be feasible. These are sometimes used to trap heavy boulders or coarse gravel that would clog culverts.

A number of detention and/or debris basins have been identified in the CCRFCD Master Plan. The larger basins are generally located at or just below the mouths of mountain canyons at points just above the alluvial fans on the periphery of the valley areas. These canyon areas and the immediately down gradient fans are the source areas for large quantities of suspended sediment and bedload, which are carried in the washes during floods.

Detention basins located in the mountain canyon areas can accumulate large deposits of rocky debris, either over the course of several years or after each extremely large load event. Design of detention ponds (Section 1200) in these areas must include provisions for debris (and suspended sediment) deposits and control of floating debris using debris racks and/or risers.

Much of the rock debris will deposit in the upper reaches of detention ponds where high-velocity flood waters first encounter slack, ponded water. If regularly maintained and cleaned of these deposits, detention ponds can effectively serve multiple purposes of attenuation of flood peaks and entrapment of sediment and debris (see Section 1200 for further discussion of detention pond design).

1301.7 Sizing of Control Structures and Basins

The spacing of bars on trash racks, debris racks, debris deflectors, debris risers and debris cribs is based on the size of the structure to be protected and the anticipated size and gradation of the debris. To minimize the potential for clogging, in no case shall the barrier members be spaced more than two-thirds of the conduit diameter.

The size of debris basins is most dependent on the physical properties of the watershed and the intensity of flood events. Specific sedimentation data have not been developed for the Clark County area, and designs must be based on site specific data from other areas. The U.S. Department of Agriculture reports sedimentation rates for reservoirs nation-wide in a report "Sedimentation Deposition in U. S. Reservoirs: Summary of Data Reported Through 1975" (USDA, 1976). The average annual sedimentation rates reported vary over five orders of magnitude. For this reason, the use of data from other areas is limited.

The major threat to debris basins is from a single rare flood event. The Los Angeles Department of Public Works has published curves for debris production per storm event for the Los Angeles area (LADPW, 1989). These rates vary from approximately 250,000 yd^3/square mile to 4,200 yd^3 /square mile. Again, the soil types and storm patterns vary considerably between Los Angeles and Clark County, but the data developed for Los Angeles does illustrate the problem.

1301.7.1 Sediment Sources

To size detention/debris basins, amounts of sediment/debris carried by flood events should be estimated. These amounts of sediment are derived from sediment eroded from watersheds. The gross erosion depends on the source of sediments in terms of upland erosion, gully erosion, and local stream bank and bed erosion. Upland erosion generally constitutes the primary source of sediment; other sources of gross erosion, such as mass wasting or bank erosion and gully erosion should be estimated separately by calculating the volume of sediment scoured through lateral migration of the stream and the upstream migration of headcuts. In relatively stable fluvial systems, the analysis of sediment sources and yield focus on upland erosion from rainfall and snowmelt (JULIEN, 1995). For watershed basins having defined channels, potential sediment supply from stream bank and bed erosion can be estimated using a sediment transport equation. The total sediment yield is then the sum of the sediment supply from upland erosion and the sediment supply from stream bank and bed erosion.

1301.7.2 Types of Methods for Predicting Sediment Yield

Numerous mathematical approaches can be used to determine sediment yield from natural or disturbed land surfaces. One category of mathematical models is the “black box,” or lumped parameter model. Another category is based on regression equations as typified by the Universal Soil Loss Equation (USLE), to be discussed later. Both types interpret input-output relations using simplified forms that may or may not have physical significance. Processes related to the movement of water and sediment through the watershed are grouped into coefficients, such as in the rational formula for estimating peak discharge, i.e., $Q = CIA$, where Q is peak discharge, I is rainfall input, A is the drainage area, and C is the runoff coefficient that represents all hydrologic processes. Although lumped parameter and regression methods are often used, the parameters may not accurately represent observable physical characteristics. Another disadvantage is that some methods do not consider the physical environment as dynamic with respect to time and location.

Another approach is through the use of stochastic models. If rainfall events, watershed response, and runoff events are stochastic, i.e., probabilistic in nature, the processes of sediment yield are also stochastic. However, stochastic models are difficult to apply (SHEN and LI, 1976) and do not readily show the response of a watershed undergoing changes as a result of various land use activities. Most hypotheses used in stochastic models have not been tested by field data. Knowledge in applying stochastic models to sediment yield from watersheds is still primitive.

The physical process simulation model is another type of method in which the governing processes controlling sediment yield are formulated and analyzed separately to provide model sensitivity to land management alternatives. These models are used to estimate or predict sediment yields resulting from natural or disturbed watershed lands, taking into account important physical processes such as raindrop splash, overland flow erosion, channel erosion, and movement of different sediment size fractions. However, these models are quite complex and are beyond the scope of this MANUAL.

One important aspect of model development and operation is data. Without adequate data, the testing and verification of models for application to field situations may produce erroneous results unrepresentative of actual conditions. An understanding of model operations and the controlling physical processes aids in the detection of erroneous data. Development or prediction methods, keeping physical processes and data needs in the forefront, can produce realistic, accurate methods for estimating sediment yield from watersheds.

1301.7.3 The Universal Soil Loss Equation

The USLE is the most widely used equation for empirical estimation of gross erosion from upland areas (SMITH and WISCHMEIER, 1957). This equation has been used on cropland and rangeland to estimate long-term (10 years or more) average annual soil losses from sheet and rill erosion with varying degrees of success, depending on the amount of quantitative data available to estimate factor values (WISCHMEIER, 1973). The USLE equation is:

$$A = RKLSCP \qquad (1301)$$

where A is the estimated annual soil loss in tons per acre, R is the rainfall-erosivity factor, K is the soil-erodibility factor, LS is the topographic factor, C is the cropping factor, and P is a supporting conservative practices factor. SMITH and WISCHMEIER (1957), MEYER and MONKE (1965), and WISCHMEIER (1973) provide detailed descriptions of this equation.

The rainfall-erosivity factor R can be calculated for each storm from:

$$R = 0.01\ \Sigma(916 + 331 \log \mathrm{I})\ \mathrm{I} \qquad (1302)$$

where I is the rainfall intensity in inches per hour. The annual rainfall erosion factor in the United States decreases from a value exceeding 500 near the Gulf of Mexico to values under 100 in the northern states and in the Rockies.

Soil erodibility factor K was found by WISCHMEIER, et al. (1971) to be a function of percent of silt, percent of coarse sand, soil structure, permeability of soil, and percent of organic matter. The soil erodibility nomograph is shown in **Figure 1303a**.

The topographic factor LS was defined as the ratio of soil loss from any slope and length to soil loss from a 72.6 foot plot length at a 9 percent slope, with all other conditions the same. This factor can be approximated from the field runoff length X_r in feet and surface slope S_o in feet per feet by:

$$LS = \sqrt{X_r}\,(0.0076 + 0.53\,S_o + 7.6\,S_o^2) \quad (1303)$$

where the runoff length was defined as the distance from the point of overland flow origin to the point where either slope decreases to the extent that deposition begins or runoff water enters a well-defined channel (SMITH and WISCHMEIER, 1957). The effect of the runoff length on soil loss is primarily a result of increased potential due to greater accumulation of runoff on the longer slopes.

The cropping-management factor C was defined as the ratio of soil loss from land cropped under specific conditions to corresponding loss from tilled, continuously fallow ground. WISCHMEIER (1972) presented a method including graphical aids for determining the cropping-management factor. This factor, ranging from approximately 0 to 1.0, is the product of the effect of canopy cover (C_I), effect of mulch or close-growing vegetation in direct contact with the soil surface (C_{II}), and tillage and residual effect of the land use (C_{III}). That is,

$$C = C_I C_{II} C_{III} \quad (1304)$$

Figures 1303b, 1303c, and **1303d** show graphical relations to estimate these factors.

The conservation practice factor P accounts for the effect of conservation practices such as contouring, strip cropping and terracing on erosion. Its values can be obtained from **Table 1301A**. This factor has no significance for wildland areas and can be set at 1.0.

The USLE is used with a sediment delivery ratio, S_{DR} to estimate the amount of sediment delivered by channels at a point downstream. This ratio takes into account the storage and deposition of sediment within a watershed, and is found to be highly dependent on the drainage area of the upstream watershed, A_t:

$$S_{DR} = 0.31\ A_t^{-0.3} \tag{1305}$$

The sediment yield can, therefore, be written as:

$$Y_s = A\ S_{DR} \tag{1306}$$

This method was used by the U. S. FOREST SERVICE (1980) and many others, and was compared with other predictive methods by ALLEN (1981). ALLEN indicated that the sediment delivery ratio is oversimplified and unreliable. WISCHMEIER (1971) cautioned that large errors can occur if the R factor is used to predict soil loss on a storm basis.

1301.7.4 The Modified Universal Soil Loss Equation

Because sediment yield in many watersheds is limited by hydraulic conditions, the amount of sediment leaving the watershed is strongly related to flow characteristics and less to rainfall characteristics. Consequently, WILLIAMS and BERNDT (1972) modified the USLE by replacing the rainfall factor R with a runoff factor which is more applicable to short-term, high-intensity storm events. Sediment yield is computed as:

$$Y_s = \alpha\ (VQ_p)^{\beta}\ KLSCP \tag{1307}$$

where Y_s is the storm-event sediment yield in tons, Q_p is the storm-event peak flow in cubic feet per second, V is the storm-event runoff volume in acre-feet, α and β are coefficients, and the other terms are defined above as for the USLE. The coefficients were calibrated as 95 for α and 0.56 for β in watersheds in Texas and Nebraska. These coefficients vary and should be determined in other locations by calibration with watershed data. Application of these coefficients to watersheds in southern California, Arizona, and Nevada also yielded reasonable results.

If the sediment yield from the land surface on an annual basis rather than a single storm event is desired, the MUSLE can also be used. This application is accomplished by determining the soil loss for events of varying return periods. Recommended return periods are 2, 10, 25, 50 and 100 years. The sediment yields are then weighted according to their incremental probability, resulting in a weighted storm average.

To compute the annual water yield, the weighted storm yield is multiplied by the ratio of annual water yield to an incremental probability-weighted water yield. For the return periods recommended, the computation is:

$$A_s = \frac{V_A\,(0.01Y_{s100} + 0.01Y_{s50} + 0.02Y_{s25} + 0.06Y_{s10} + 0.4Y_{s2})}{0.01V_{100} + 0.01V_{50} + 0.02V_{25} + 0.06V_{10} + 0.4V_2} \qquad (1308)$$

where A_s is the annual sediment yield, V_A is the average annual water yield, and the numerical subscripts in the single storm event (Y_s) and water yield (V) refer to the return period of the storm.

When estimating sediment yield using either the MUSLE or other methods, a useful computation is to express the sediment yield in terms of an average concentration (ppm) based on the total water and sediment yields. This value can be compared with measured stream data in the area and results of sediment routing analysis.

1301.7.5 Total Sediment Yield

Total sediment yield is the sum of wash load and bed-material load. Wash load is defined as “that part of the sediment load which is composed of particles smaller than those found in appreciable quantities in the shifting portion of the stream bed” (EINSTEIN, 1950). Quantifying, EINSTEIN suggested the limiting sizes of wash load and bed-material load may be chosen as the grain diameter (D_{10}) of which 10 percent of the bed mixture is finer. SIMONS and SENTURK (1992) give a similar definition. The wash load is usually carried away by the stream without much deposition. In contrast, the transport of bed-material load is controlled by the transport capacity of the channel. The above wash load and bed-material load definitions were applied for this MANUAL.

The USLE and MUSLE methods are generally applicable as predictors of wash load. This section presents an example of applying a sediment transport equation with the MUSLE to determine annual sediment yield. The information required to determine the sediment transport capacity in a channel are the hydraulic characteristics of the channel and the sediment sizes present in the bed. It is assumed that the transporting capacity of material larger than D_{10} controls the transport rate, while the supply controls transport for the smaller sizes. This concept is illustrated in **Figure 1303E.** The transport capacity is determined using a combination of the MEYER-PETER, MULLER bed-load equation and the EINSTEIN integration for suspended load (i.e., **Equation 704**), which is found to provide reasonable estimation of sediment discharges for sandy gravel channels. Individual size fractions are considered. The supply of smaller sediment is determined using the MUSLE.

Table 1301B shows the results of the calculations for individual storms of specified return periods. Assuming the average annual runoff volume for the area is 202 acre-feet and applying **Equation 1308**, the average annual sediment yield is 32,000 tons. This total sediment yield is 50 percent larger than that computed from the MUSLE alone. This illustrates the inaccuracy that can result when the bed-material load in the channels is not considered.

In this example, it was assumed that sizes smaller than 1 millimeter were wash load. The actual division between wash load and bed-material load is difficult to determine and varies depending on the characteristics of the watershed and river system. Often, the division is set at 0.062 millimeter (the largest silt size) for rivers with mild slopes. For steeper watershed streams, the wash load is set at all sizes smaller than a given percent finer, such as D_{10}.

Since for the bed material load the transport rate was assumed equal to the transport capacity, the effect of gullying or bank erosion was indirectly incorporated. The supply of sediment, whether from the channel bed, channel banks, or from gullying, was assumed to be sufficient to allow the channel to transport sediment at its capacity. Therefore, the method used gives a maximum estimate of the combined erosion processes, since a lesser amount of sediment might be supplied to the channel; however, if a greater amount were supplied, the excess would be deposited.

The methodology just described is recommended for the western United States, in most circumstances, rather than a straightforward application of the USLE. The effect of the infrequent, high runoff-producing events is incorporated directly. The substitution of the MUSLE for the USLE provides a methodology that is more applicable to western conditions, especially in arid regions. The inclusion of channel transporting capacity is also important. It is most significant in steep sand-bed channels where the transporting capacity of the bed material sizes can be high.

1301.8 Siting of Control Structures and Basins

Debris control structures which protect other hydraulic structures (e.g., culverts, bridges, channel) are placed based on structure cost, debris production potential and the importance of the structure. Minor culverts whose failure would have a limited impact on downstream structures would require less debris protection than a major lined channel. Generally speaking, debris control structures should be placed at the source of debris.

1302 CONTROL OF EROSION FROM CONSTRUCTION ACTIVITIES

1302.1 Introduction

The cleaning, stripping, and grading of land may cause severe localized erosion with subsequent sediment deposition and damage to downstream streets, channels, culverts, and other property. Erosion during and immediately following construction may be particularly sever in the following cases:

1. Sites having slopes greater than 4-5 percent

2. Sites where large areas of loose dirt and/or graded earth are left unprotected during the potential rainy season (summer in the Clark County area)

3. Areas of concentrated storm flows in unprotected channels or outlets from storm drains

Studies have shown the construction activities can easily increase the annual soil erosion rates from a parcel by 10 to 100 times those experienced under undisturbed conditions. High erosion from constructed areas normally subside after 2 to 3 years, but can persist indefinitely if gullies, slope failures, and other erosion features are not repaired and controlled.

The purpose of an erosion and sediment control plan is to reduce erosion to an acceptable level, but without undue economic burden to the property, owner, developer, or contractor. To this extent, the local entities rely on the property owner to control erosion from their sites and generally do not require an erosion and sediment control plan. However, when the local entity believes there is a potential erosion and/or sedimentation problem which would affect public or private facilities, the local entity may require the submittal of an erosion and sediment control plan in accordance with Section 1302.

1302.2 References

The following references are suggested for use in preparing an erosion and sediment control plan:

1. GOLDMAN, 1986 - Erosion and Sediment Control Handbook by S. J. Goldman, K. Jackson, T. A. Bursztyusky, McGraw-Hill, 1986 (particularly see Chapter 9, " Preparing and Evaluating an Erosion and Sediment Control Plan").

2. USEPA, 1972 - Guidelines for Erosion and Sediment Control Planning and Implementations, U. S. Environmental Protection Agency Pub. EPA-R2-72-015.

3. DRCOG, 1980 - "Managing Erosion and Sedimentation from Construction Activities", Denver Regional Council of Governments, 2480 East 26th Avenue, Suite 200-B, Denver, Colorado 80211, April 1980.

1302.3 Erosion, Sediment, and Debris Control Plans

All subdivision plats, commercial developments, or other major construction projects requiring a drainage plan shall address erosion and sedimentation control. In cases where there is significant potential for erosion-related problems, the local entity may require an erosion and sediment control plan be prepared which identifies specific measures and structures to mitigate potential damage.

In general, the development of an erosion and sediment control plan involves five steps:

1. Collect and review information on site topography, soils, vegetation, and important adjacent (off-site) features such as open channels and storm drains.

2. Evaluate the information relative to the potential for erosion/sedimentation problems caused by construction.

3. Devise and/or modify construction activities (e.g., schedule) to minimize erosion problems.

4. Develop an erosion and sediment control plan with specific measures tailored to the construction and terrain conditions.

5. Follow and monitor the plan and revise as necessary.

Typically, if the local entity requires an erosion and sediment control plan, only Item I below will need to be addressed. The decision as to the need for a special construction schedule and/or a formal erosion and sediment control plan (Items II and III) will be based on project size, amount of earth disturbance, soil type, slope, and proximity to adjacent property or facilities likely to be damaged by erosion or sediment deposits.

Items to be discussed and/or prepared as part of an erosion and sediment control plan (when required) include the following:

I. <u>Erosion and Sediment Control Narrative</u>

   a. Project description: Nature and purpose of the land-disturbing activity, and the estimated amount of grading involved.

   b. Existing site conditions: Existing topography, vegetation, drainage, and soils (including erodibility and particle size).

c. Adjacent areas: Neighboring areas, such as channels, residential areas, and roads that might be affected by the land disturbance.

d. Critical areas: Areas within the developed site that have potential for serious erosion or sediment problems.

e. Erosion and sediment control measures: Construction schedule and methods that will be used to control erosion and sediment on the site.

f. Permanent stabilization: Description of how the site will be stabilized after construction is completed.

g. Schedule: Schedule of construction activities and regular inspections and repairs of erosion and sediment control structures.

II. <u>Formal Erosion and Sediment Control Plan</u> (When Required)

a. Existing contours: Existing elevation contours of the site at an interval sufficient to determine drainage pattern.

b. Preliminary and final grades: Proposed changes in the existing elevation grades for each stage of grading (e.g. rough grading and final grading).

c. Soils: Boundaries of the different soil types within the proposed development.

d. Existing and final drainage patterns: Map showing the dividing lines and the direction of flow for the different drainage areas before and after development.

e. Limits of clearing and grading: Finished contours and/or boundaries showing the area to be disturbed.

f. Erosion and sediment control facilities: Locations, names, and dimensions of the proposed temporary and permanent erosion and sediment control facilities.

g. Storm water management system: Location and size of permanent storm drain inlets, pipes, outlets, and other permanent drainage facilities (swales, waterways, etc.).

h. Detailed drawings: Dimensioned drawings of key features such as sediment basin risers, energy dissipators, and waterway cross-sections.

i. General seeding and mulching information: Seeding dates, seeding, fertilizing, and mulching rates in pounds per acre and application procedures.

j. Maintenance program: Inspection schedule, spare materials needs, stockpile locations, and instructions for sediment removal and disposal and for repair of damaged structures.

III. Detailed Calculations (When Required)

a. Calculations and Assumptions: Data for design storm, including frequency and intensity, used to size pipes, channels, sediment basins, and traps; design particle size for sediment traps and basins; estimated trap efficiencies, basin discharge rates; size and strength characteristics for filter fabric, wire mesh, fence posts, etc. and other calculations necessary to support drainage, erosion, and sediment control systems.

Drainage studies and plans for rough grading to be done early and long-term construction projects shall address the narrative items (Item I) if required by the local entity. If a formal erosion and sediment control plan is deemed necessary, all of the listed drawing features, details, and calculations will be required and shall be submitted either with the drainage plan or as a separate document.

1302.4 Performance Standards

The general standards and criteria for developing an erosion and sediment control plan include the following:

1. Fit the development to the terrain-minimize radical changes in terrain features.

2. Time grading and construction to minimize soil exposure during rainy season (summer).

3. Retain existing vegetation whenever possible.

4. Vegetate, mulch, and/or otherwise protect denuded areas.

5. Direct runoff away from denuded or erosion-prone areas.

6. Minimize length and steepness of slopes.

7. Keep runoff velocities low.

8. Design drainage ways and outlets to withstand concentrated or increased runoff.

9. Trap sediment on-site using such facilities as sediment basins, berms, straw bale dikes, and silt fences during construction.

10. Inspect and maintain erosion control measures and facilities.

DEBRIS STRUCTURE PERFORMANCE

Debris Classification / Type of Structure	Debris Deflector	Debris Rack	Debris Riser	Debris Crib	Debris Basin and Dam
Light Floating Debris		X		X	
Medium Floating Debris	X	X			
Heavy Floating Debris	X				
Flowing Debris			X		X
Fine Detritus			X		X
Coarse Detritus			X	X	X
Boulders	X				

Revision	Date

CONSERVATION PRACTICE FACTOR P FOR CONTOURING, STRIP CROPPING AND TERRACING

Land Slope (%)	Farming on Contour	Contour Strip Crop	Terracing	
			(a)	(b)
2-7	0.50	0.25	0.50	0.10
8-12	0.60	0.30	0.60	0.12
13-18	0.80	0.40	0.80	0.16
19-24	0.90	0.45	0.90	0.18

- For erosion-control planning on farmland.
- For prediction of contribution to off-field sediment load.

Source: After WISCHMEIER (1972).

Revision	Date

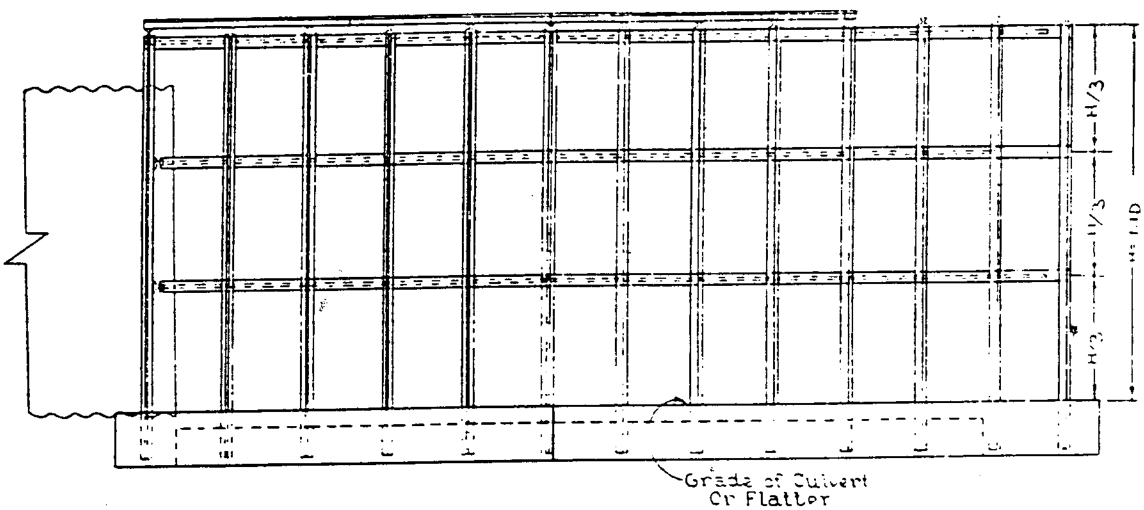
AN EXAMPLE OF TOTAL SEDIMENT YIELD COMPUTATION

Return Period (yrs)	Runoff Volume (ac-ft)	Runoff Peak (cfs)	Wash Load* (tons)	Bed-Material Load** (tons)	Total Sediment Yield (tons)
2	121	336	11,000	9,000	20,000
10	593	1,646	64,000	27,000	91,000
25	917	2,544	100,000	35,000	135,000
50	1,233	3,329	140,000	49,000	189,000
100	1,510	4,190	180,000	62,000	242,000
Incremental Probability Weighted	130	—	13,000	7,030	20,470
Annual Yield	202	—	21,000	11,000	32,000

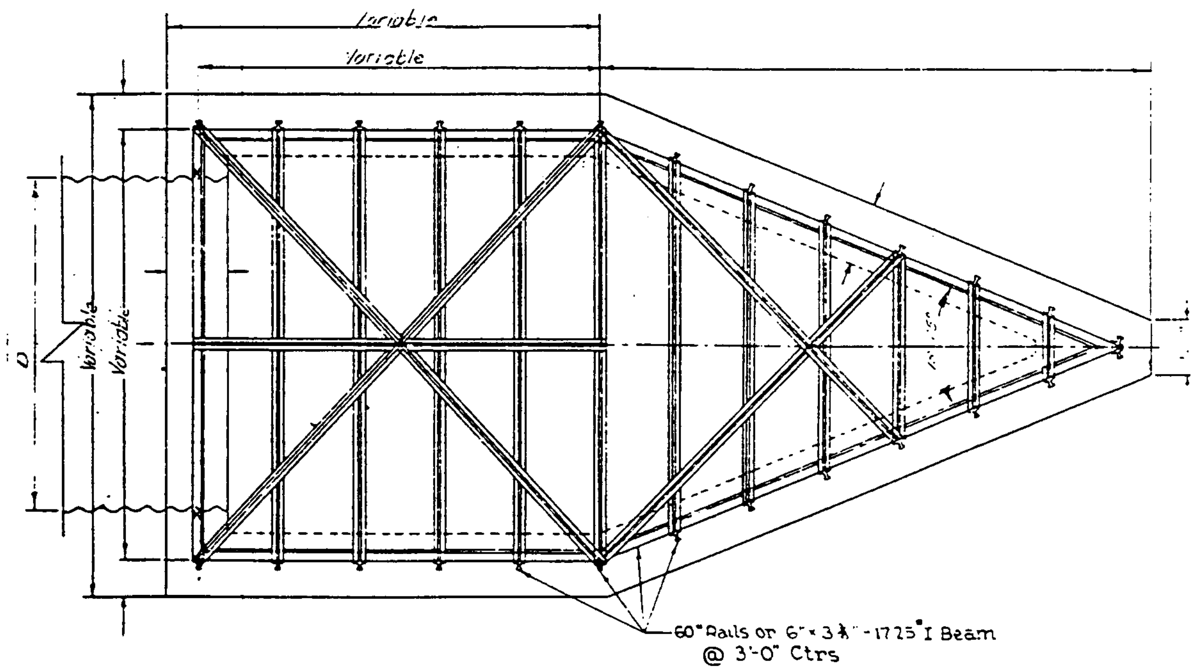
Revision	Date

REFERENCE:

DEBRIS DEFLECTOR (TYPICAL)

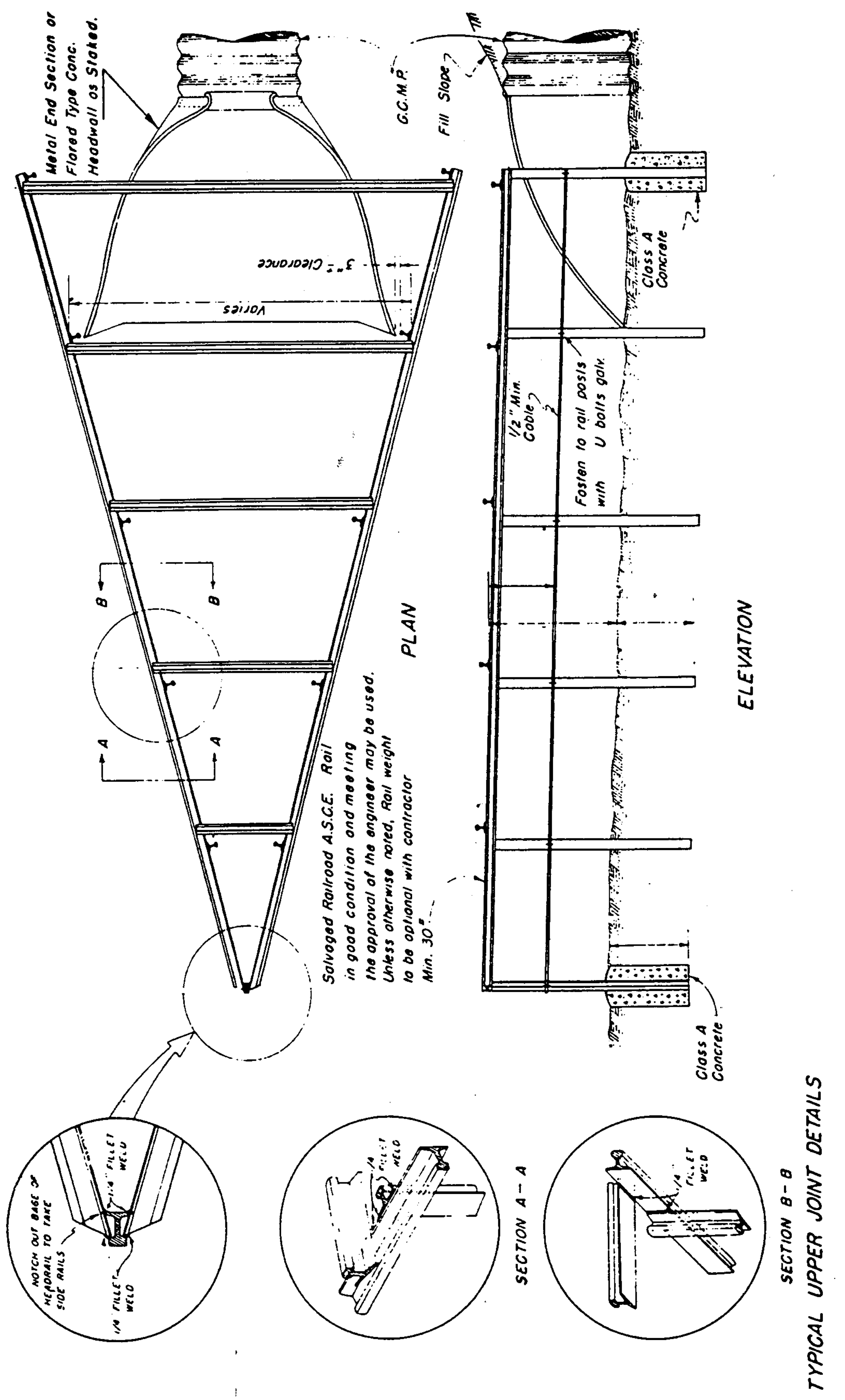


SIDE ELEVATION



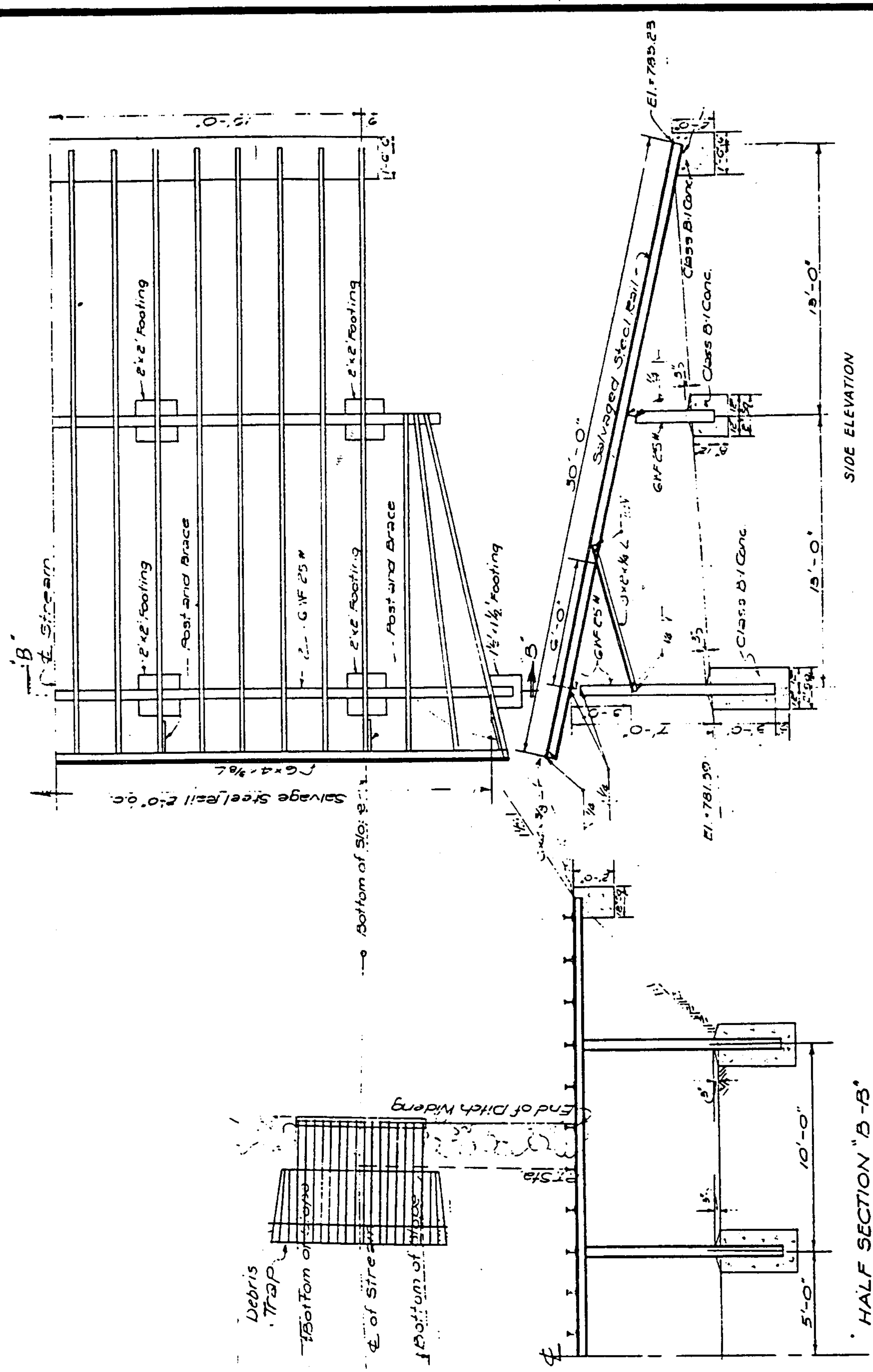
PLAN

Revision	Date



DEBRIS DEFLECTOR (TYPICAL)

Revision	Date



DEBRIS RACK
(TYPICAL)

Revision	Date

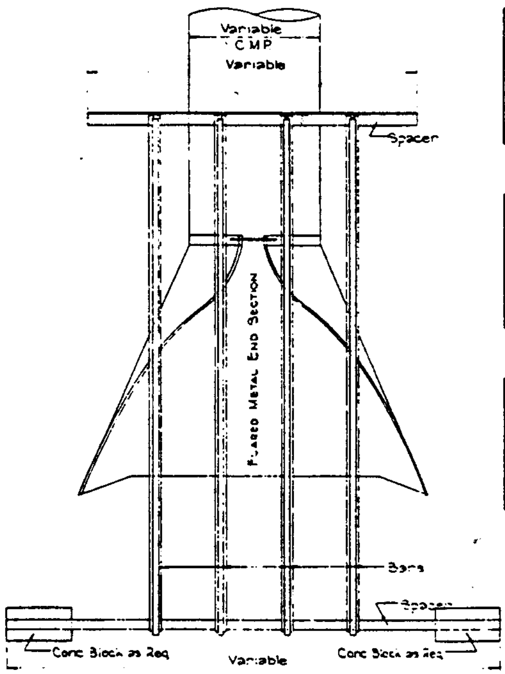
WRC ENGINEERING	REFERENCE: USDOT, FHWA, HEC NO. 9, 1971	FIGURE 1302 1 OF 2

DEBRIS RACK (TYPICAL)

BAR SPACING FOR VARIOUS CULVERTS		
CMP	BARS REQUIRED	BAR SPACING
18"	3	1'-0"
24"	4	1'-2"
30"	4	1'-4"
36"	4	1'-8"
42"	5	1'-8"
48"	5	1'-9"

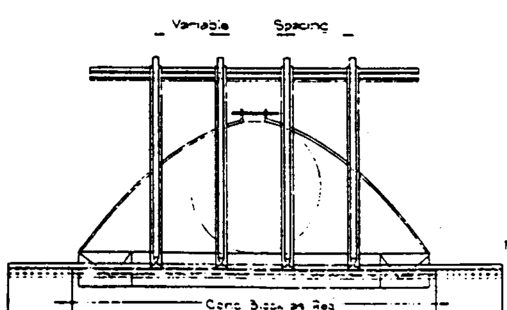
REQUIRED LENGTH OF BARS			
CMP	SLOPE OF BAR	BAR LENGTH	MATERIAL
18"	3:1	8'-3"	Std. 3" Pipe or [illegible] Lb. Rail
24"	3:1	11'-0"	" " " " "
30"	3:1	12'-0"	" " " " "
36"	3:1	13'-6"	40 to 60 Rail or Steel I's
42"	3:1	15'-0"	" " " " " " "
48"	3:1	16'-6"	" " " " " " "

LENGTH OF SPACERS			
CMP	TOP SPACER	BOTTOM SPACER	MATERIAL
18"	6'-0"	8'-0"	4"x4"x3/8" ∠s
24"	7'-0"	10'-0"	40 to 60 Lb
30"	7'-6"	11'-0"	Rail or 3"
36"	8'-0"	12'-6"	Pipe
42"	9'-0"	13'-9"	
48"	10'-0"	15'-0"	

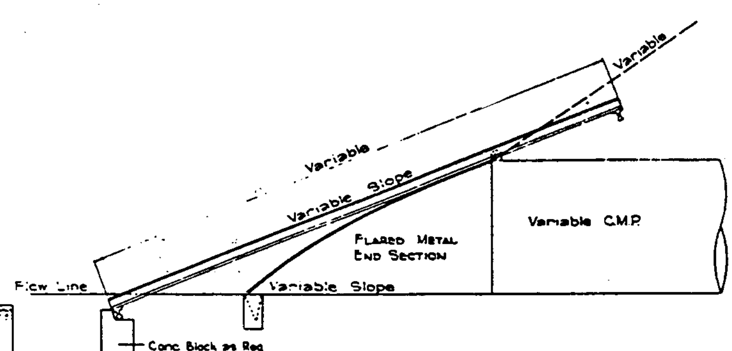


PLAN

NOTE
- Special Treatment Required For Pipes Larger Than 48"
- Minimum Bar Spacing 0'-10"
- Maximum Bar Spacing 2'-0"
- Gradients Steeper Than 15% May Require Special Treatment
- Sizes Shown Are Minimums To Be Used.
- Heavier Sections Permitted In All Cases



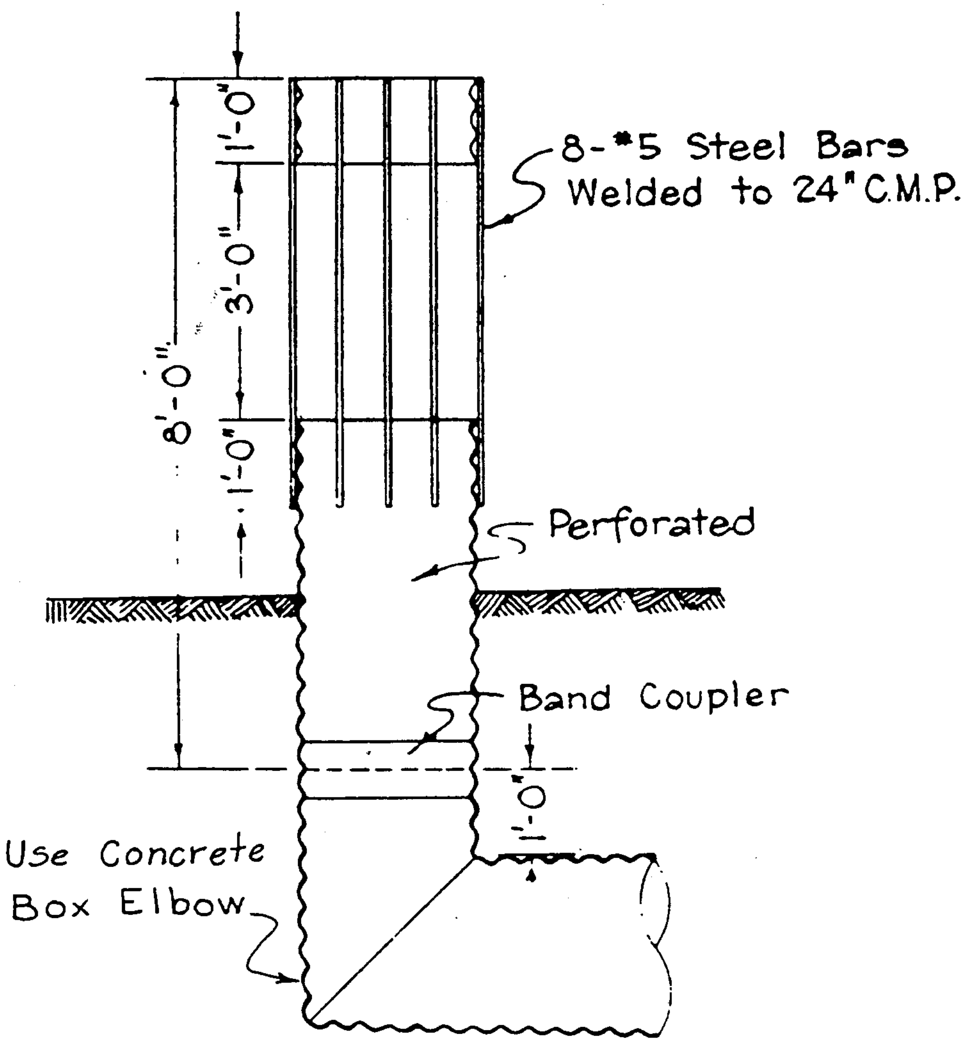
FRONT ELEVATION



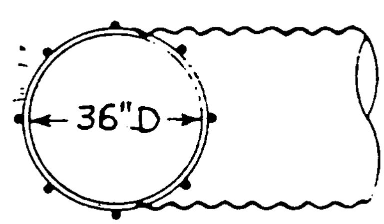
RIGHT SIDE ELEVATION

Revision	Date

DEBRIS RISER (TYPICAL)



ELEVATION

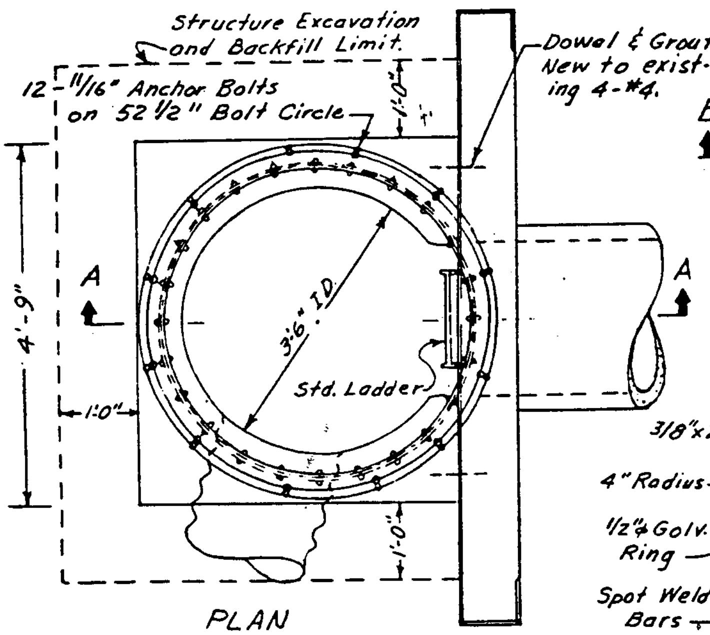


PLAN

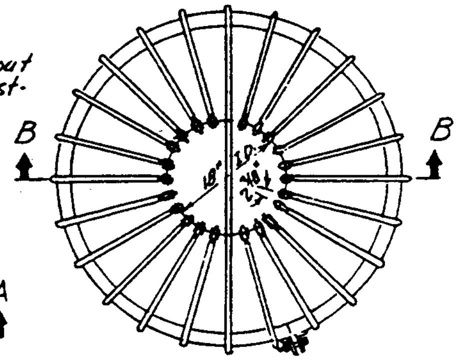
Revision	Date

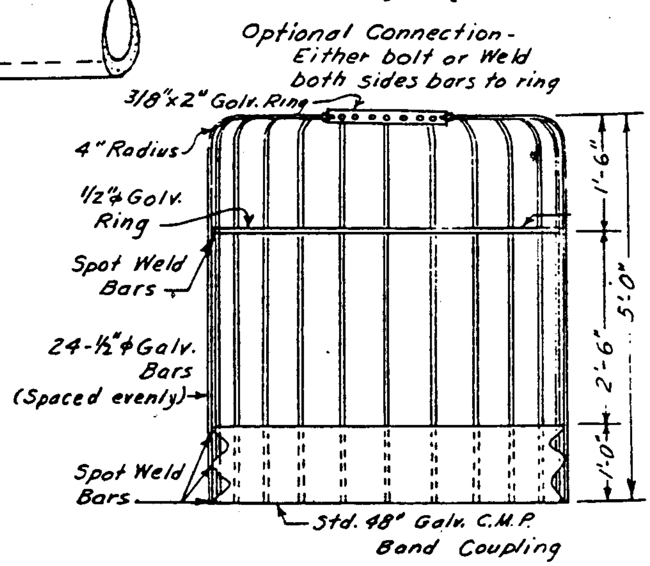
REFERENCE:
USDOT, FHWA, HEC NO. 9, 1971

DEBRIS RISER (TYPICAL)

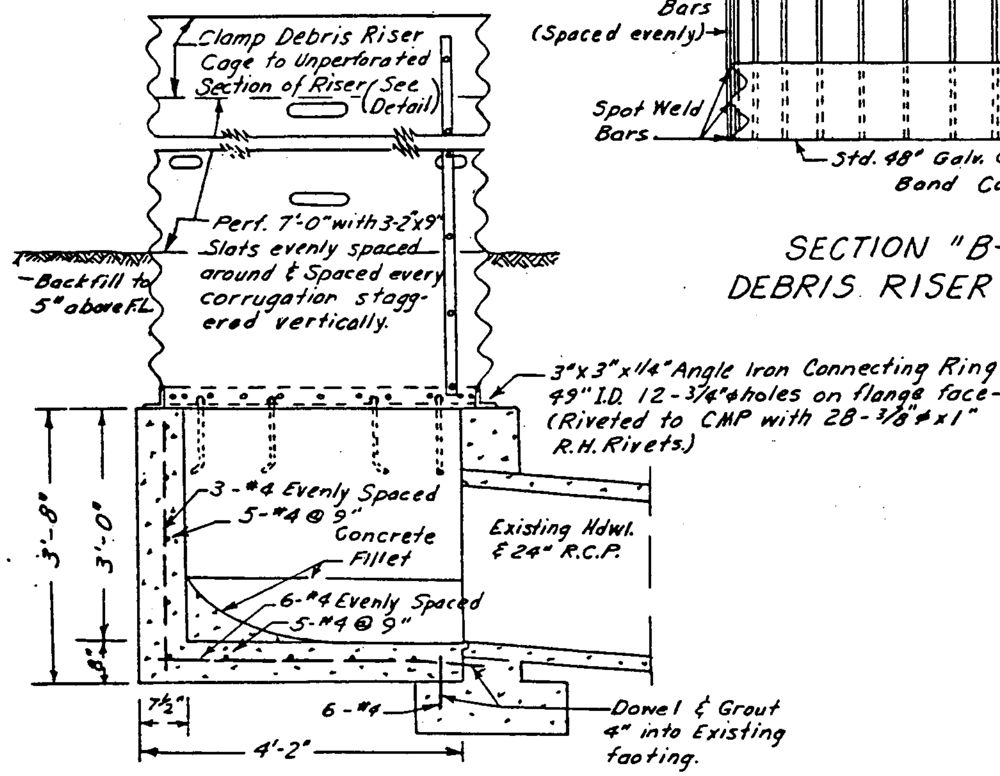


PLAN





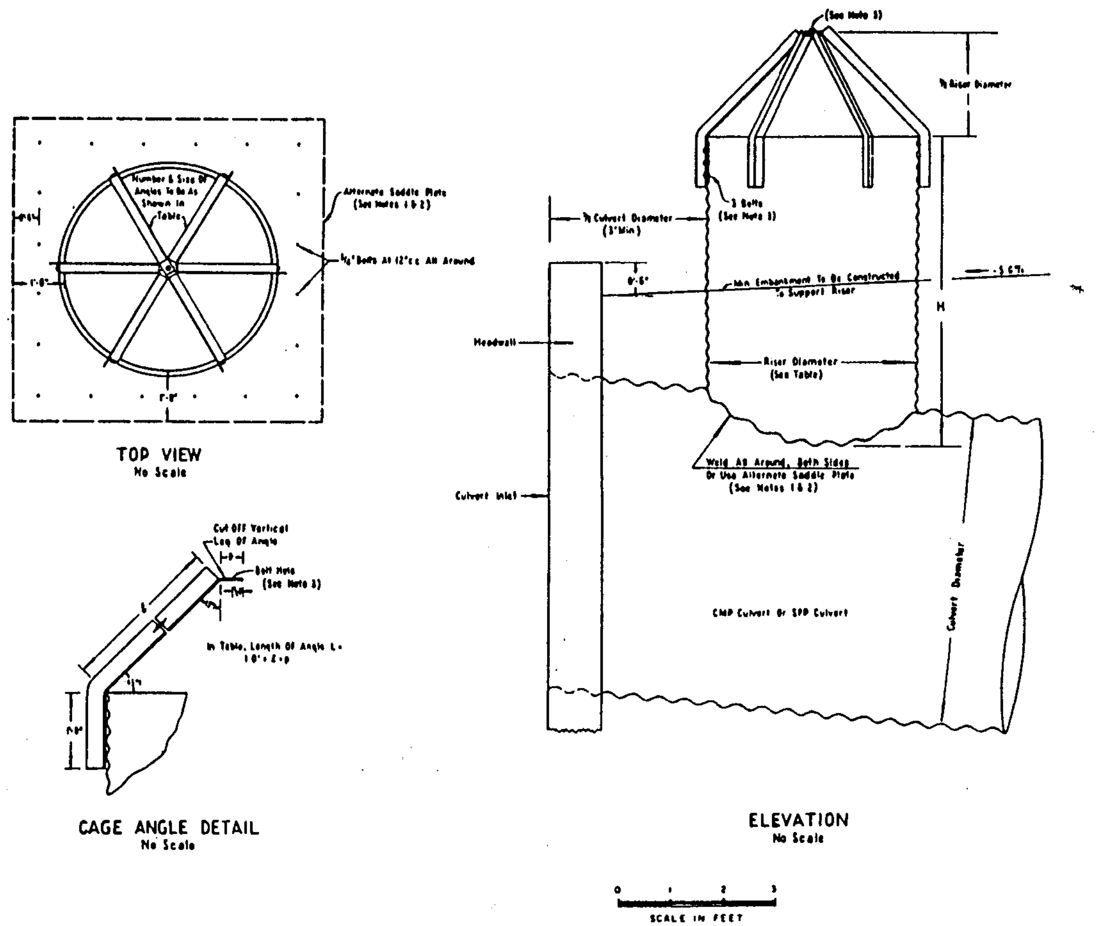
SECTION "B-B"
DEBRIS RISER CAGE



SECTION "A-A"
CMP RISER & JUNCTION BOX

Revision	Date

WRC ENGINEERING	REFERENCE: USDOT, FHWA, HEC NO. 9, 1971	FIGURE 1303 2 OF 3

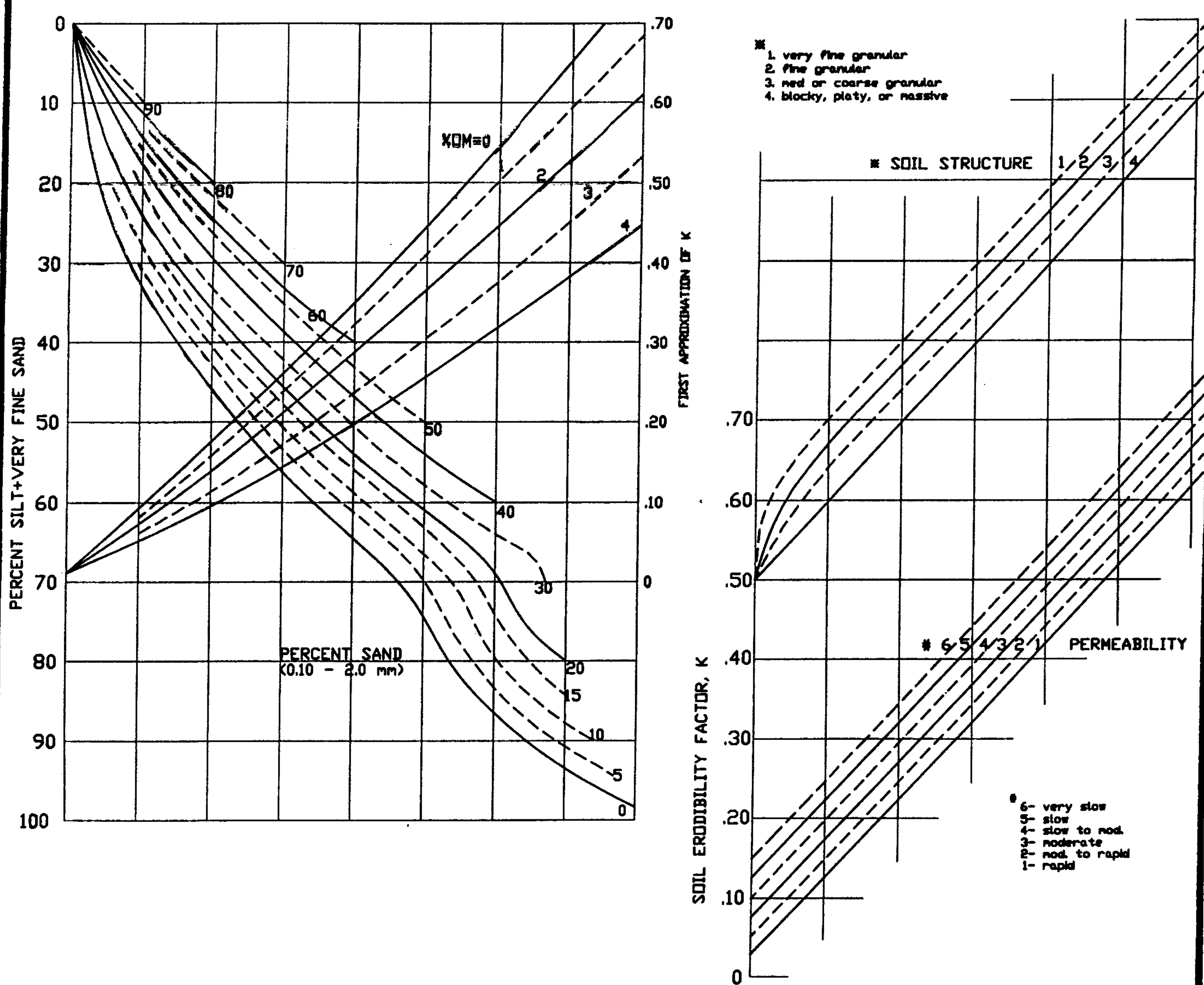


CULVERT DIAM.	RISER			RISER CAGE				
	C.M.P		H*	STEEL				
	DIAM	GAGE		ANGLE SIZE	NO. OF PIECES	LENGTH, FT.		
INCHES	INCHES		FEET			[illegible]	[illegible]	L
36	24	14	4	2" x 2" x 1/4"	4	1'-5"	2"	2'-7"
42	24	14	4	2" x 2" x 1/4"		1'-5"	2"	2'-7"
48	30	14	4	2 1/2" x 2 1/2" x 1/4"		1'-10"	3"	3'-1"
54	36	12	4			2'-1"	3"	3'-4"
60	42	12	6			2'-6"	3"	3'-9"
66	42	12	6			2'-6"	3"	3'-9"
72	48	12	6			2'-10"	3"	4'-1"
78	48	12	6			2'-10"	3"	4'-1"
84	54	12	6	3" x 3" x 1/4"		3'-3"	3"	4'-6"
90	60	10	8			3'-6"	3"	4'-9"
96	60	10	8			3'-6"	3"	4'-9"
102	66	10	8			3'-11"	3"	5'-2"
108	72	10	8	3 1/2" x 3 1/2" x 5/16"		4'-3"	4"	5'-7"
114	72	10	8			4'-3"	4"	5'-7"
120	78	8	8			4'-8"	4"	6'-0"
126	84	8	10			5'-0"	4"	6'-4"
132	84	8	10			5'-0"	4"	6'-4"
138	90	8	10		6	5'-4"	7"	6'-11"
144	96	8	10			5'-8"	7"	7'-3"
150	96	8	10			5'-8"	7"	7'-3"
156	102†	12	12	4" x 4" x 3/8"	8	6'-0"	11"	7'-11"
162	108†	12	12			6'-5"	11"	8'-4"
168	108†	12	12			6'-5"	11"	8'-4"
174	114†	10	12			6'-9"	11"	8'-8"
180	120†	10	12			7'-1"	11"	9'-0"

DEBRIS RISER (TYPICAL)

Revision	Date

SOIL ERODIBILITY NOMOGRAPH USED TO DETERMINE FACTOR K (TONS/ACRE) FOR SPECIFIC TOPSOILS OR SUBSOIL HORIZONS

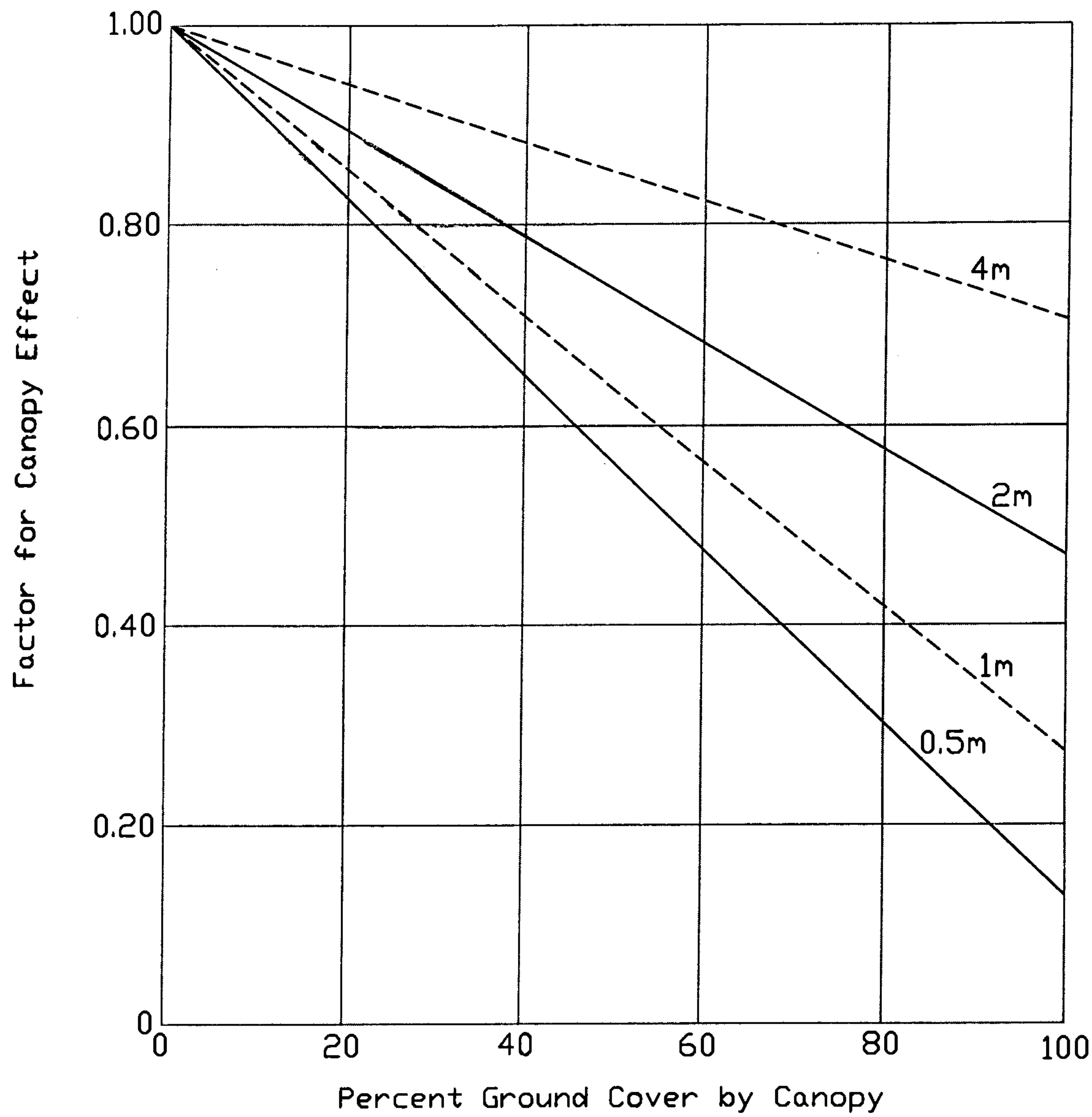


Revision	Date

REFERENCE: A Soil-Erodibility Nomograph for Farmland and Construction Sites, 1971

FIGURE 1303a

INFLUENCE OF VEGETAL CANOPY

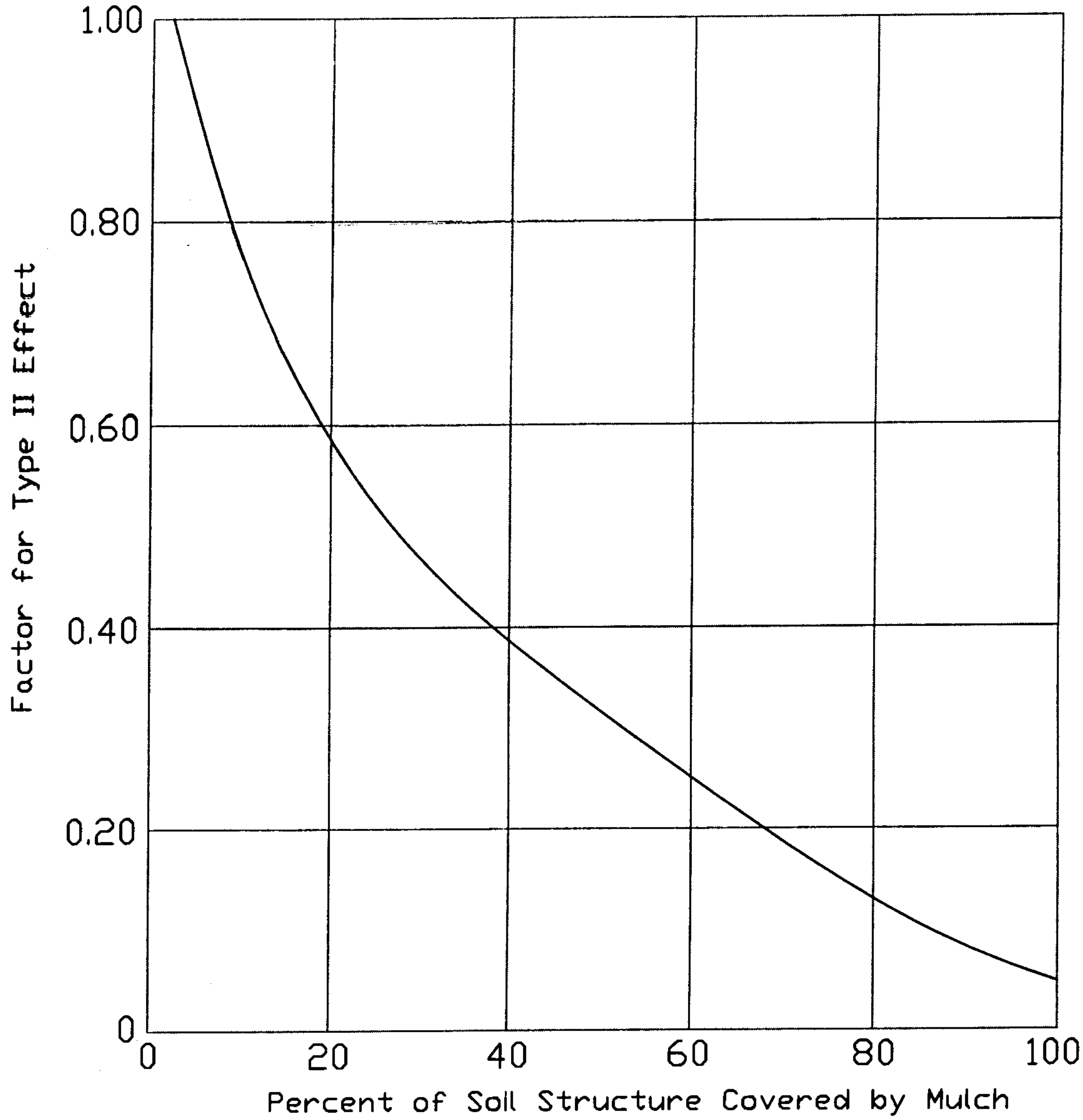


Revision	Date

REFERENCE: A Soil-Erodibility Nomograph for Farmland and Construction Sites, 1971

FIGURE 1303b

EFFECT OF PLANT RESIDUES OR CLOSE-GROWING STEMS AT THE SOIL SURFACE

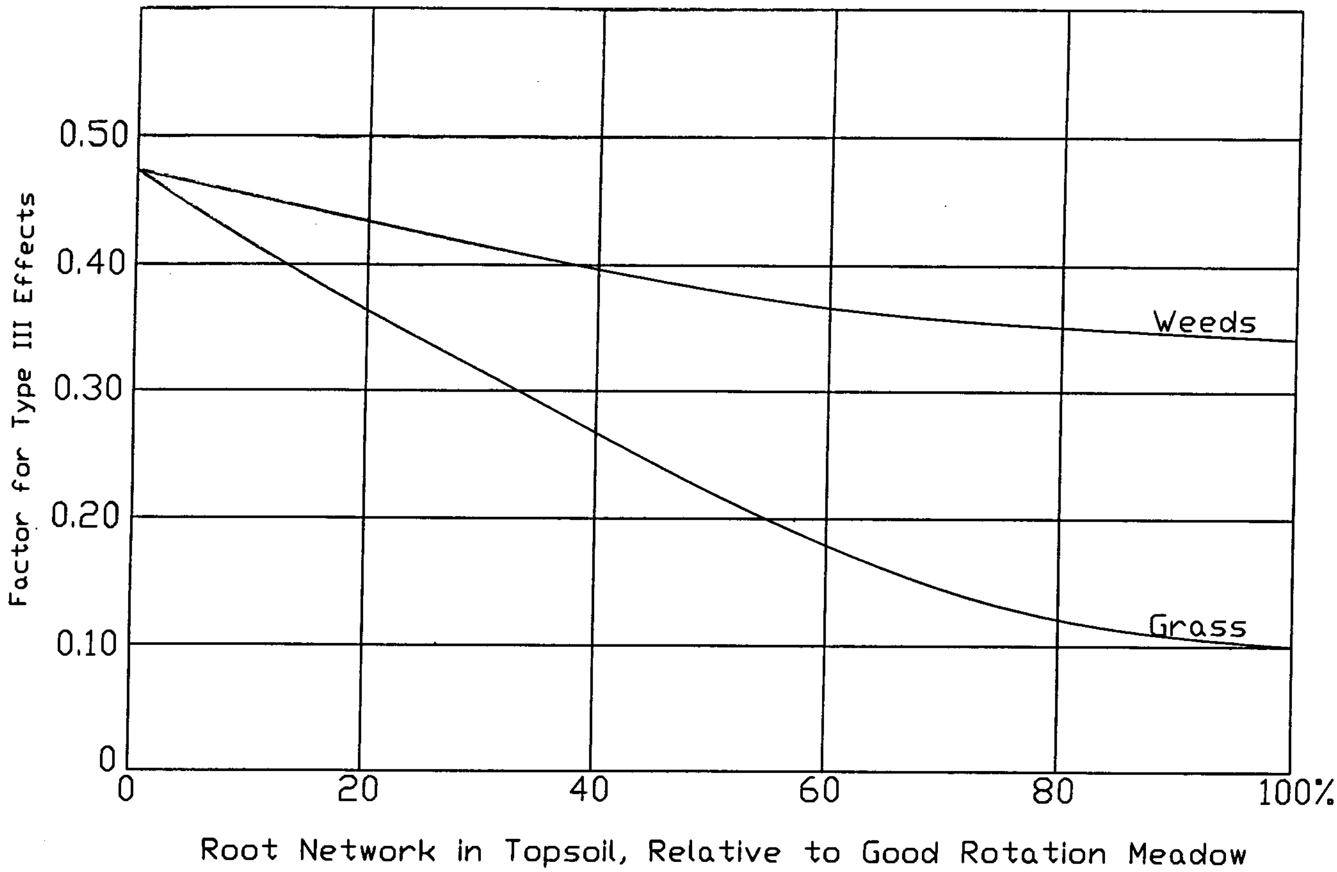


Revision	Date

REFERENCE: Estimating the Cover and Management Factor for Undisturbed Areas, 1972

FIGURE 1303c

TYPE III EFFECTS ON UNDISTURBED LAND AREAS

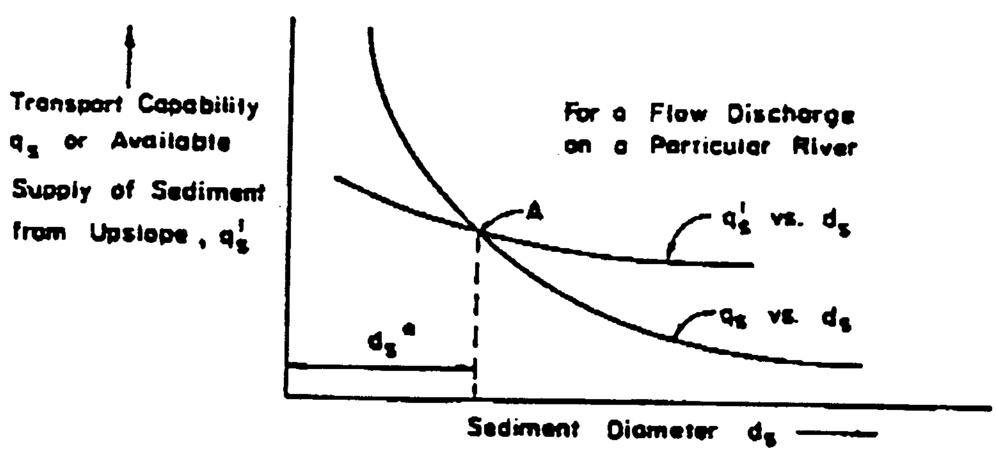


Revision	Date

REFERENCE: Estimating the Cover and Management Factor for Undisturbed Areas, 1972

FIGURE 1303d

CONCEPT OF CONTROLLING SEDIMENT TRANSPORT RATES

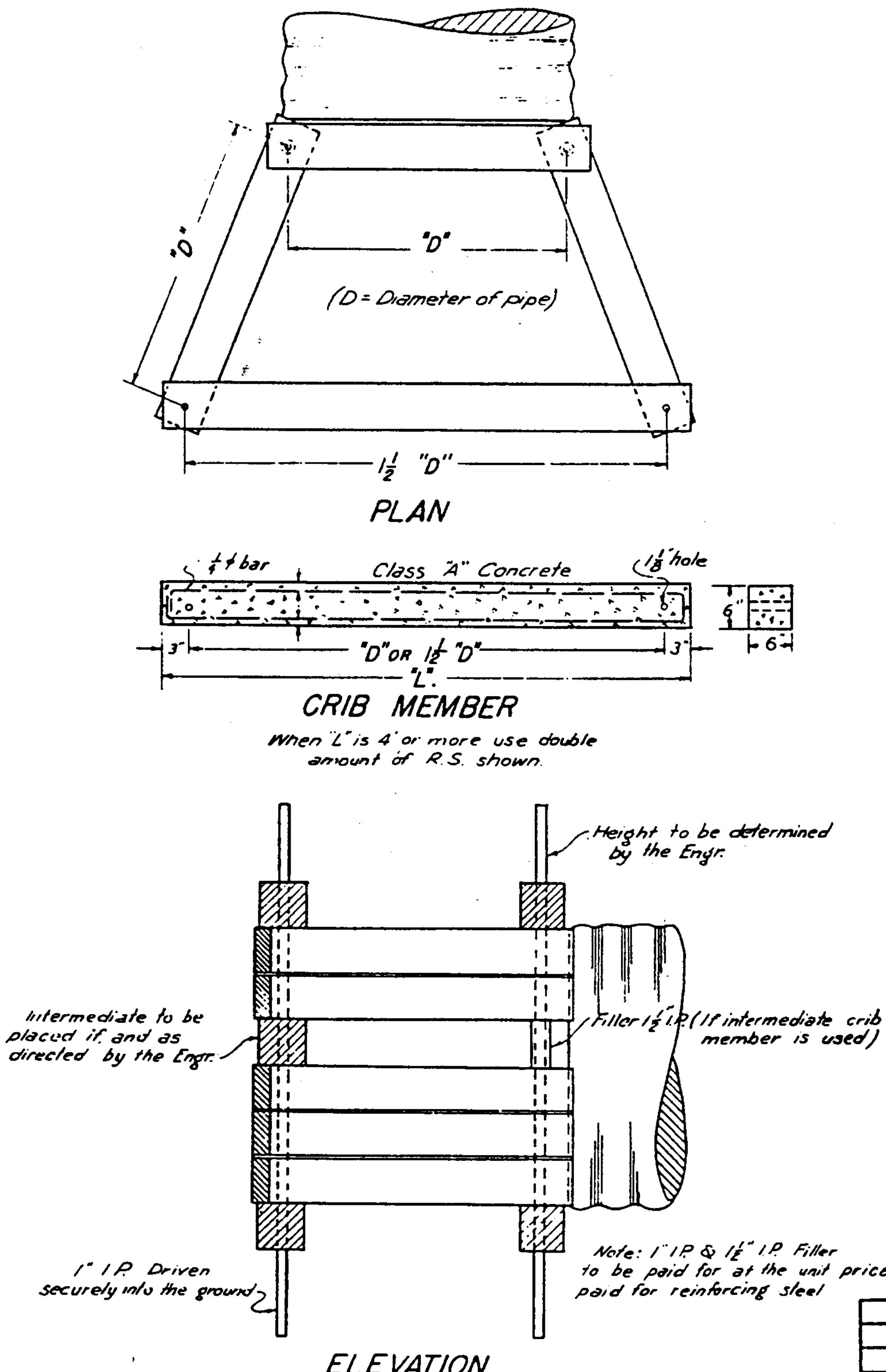


Revision	Date

REFERENCE: River Mechanics, 1971

FIGURE 1303e

DEBRIS CRIB (TYPICAL)



Revision	Date

WRC ENGINEERING

REFERENCE:
USDOT, FHWA, HEC NO. 9, 1971

FIGURE 1304

CLARK COUNTY REGIONAL FLOOD CONTROL DISTRICT
HYDROLOGIC CRITERIA AND DRAINAGE DESIGN MANUAL

SECTION 1400
DEVELOPMENT ON ALLUVIAL FANS

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1403 EXAMPLE VIRGIN FAN WITH FLOOD HAZARD ZONES DEFINED

1404 EXAMPLE IMPACT OF DEVELOPMENT ON FLOOD HAZARD ZONES

1405 IMPACT OF DEVELOPMENT POSITIONING ON FLOOD HAZARD ZONES

Section 1400
Development of Alluvial Fans

1401 INTRODUCTION

By commonly accepted definition, an alluvial fan is a triangular or fan-shaped deposit of boulders, sand and fine sediment at the base of desert mountain slopes deposited by ephemeral (intermittent) streams as theydebauch onto the valley floor (STONE, 1967). Alluvial fans are a common and dominant landscape feature in the Clark County area. The rather symmetric shape of the alluvial fan is attained through geologic time by the active flow channel migrating back and forth over the alluvial surface. All engineers designing facilities on alluvial fans for drainage and flood control should become familiar with the geologic and hydrologic processes by consulting one or more of the standard references. These references include, for example, FRENCH (1987), COOKE and WARREN (1973), or RACHOCKI (1981). It must be noted that most of the alluvial fans observed in the Clark County area will not have the idealized shape because over geologic time the fans have coalesced creating complex and poorly defined shapes.

FEMA and others have recognized that definition of a floodplain ori an alluvial fan cannot be accurately accomplished by using traditional methods of floodplain analysis (i.e., HEC-2 (FRENCH, 1985 orHOGGAN, 1989)). Given the fact that hydraulic processes on active alluvial fans are quite different than those in humid regions, a probabilistic methodology for defining floodplains on active virgin (undeveloped) alluvial fans that recognizes the potential for the flow channel to change location during a single flood event has been developed. The original methodology is described in DAWDY (1979), FEMA (1983), and FRENCH (1987) . As development in the Southwest proceeded, and the problem of flooding on active alluvial fans became a primary concern, additional data has become available; and the original methodology was modified to take these new data into account; (see for example FEMA (1985) or FRENCH (1987).

The engineer is cautioned that the study of hydraulic processes onactive alluvial fans is an area of current research interest. The methods available for addressing drainage problems on active alluvial fans at the time this manual was prepared should be considered initial or preliminary results, and rapid change in these methods must be anticipated. It is recommended that the engineer should examine the literature to determine the current state-of-the-art at the time of analysis.

The engineer is further cautioned that while the methodology described in DAWDY (1979), FEMA (1983 and 1985) and FRENCH (1987) appears straightforward, there are inherent subleties in these techniques that may not be initially recognized. The accurate application of these methods required --

experience in arid region hydrology, geology, and sound engineering judgment. A crucial consideration is the determination that the area of interest is an "active alluvial fan." The definition of an alluvial fan provided in the initial paragraphs of this section is a geomorphological rather than an engineering definition. The methodology discussed in this section is appropriate to all alluvial surfaces that exhibit hydraulic behavior similar to that on an active alluvial fan. In identifying areas where the alluvial fan approach discussed in the manual is appropriate, the engineer should examine the following criteria:

1. <u>Lack of Defined, Stable Channels:</u> On an alluvial fan where the methodology discussed in this manual is appropriate, flow channels are neither well-defined nor stable. Both the area of interest and surrounding area should be examined to determine if (1) there are well-defined natural channels capable of conveying the 100-year flood with only minor modification in depth and width and (2) the channels identified are sufficiently incised to be stable during the 100 year flow event. FRENCH (1987) provides equations to estimate natural channel capacity.

2. <u>Surface Slope:</u> In general, the longitudinal slope of an alluvial fan should lie between 0.0087 and 0.1405 ft/ft. Lesser slopes may preclude alluvial fan behavior by flow events.

3. <u>Canyon/Fan Slope Ratio:</u> The ratio of the slope of the canyon above the fan to the slope of the fan has been found to be a key parameter in determining the number of channels that will be formed by an extreme event. Use of this ratio with the figures in FEMA (1985) and FRENCH (1987) allow the alluvial surface to be divided into single channel and multiple channel (not sheet flow) regions.

4. <u>Upstream Sediment Production:</u> It is generally believed that channels on alluvial fans change location either in response to massive deposition (channel blockage) or erosion that causes a breakthrough to topographically low areas on the alluvial surface. Thus, upstream sediment production is a parameter that should be examined. If the sediment available upstream is capable of satisfying the equilibrium sediment transport requirements and the channels are stable, then a probabilistic method of floodplain analysis may not be appropriate.

5. <u>Surficial Geology:</u> The geology of the area of interest plays a crucial role in determining hydraulic behavior. For example, is the flow constrained by the geology such as outcrops of bedrock in the transverse direction or by caliche in the vertical dimension?

6. <u>Surface Stability:</u> The methods discussed here are applicable to active alluvial surfaces and not all alluvial surfaces are active. If a surface is not active, then flood hazard is reduced. For example, within Clark County there

are a number of alluvial surfaces that have been abandoned because of nearby channel incision; and these surfaces should not be considered active alluvial surfaces.

If the site being investigated exhibits the characteristics noted above, then it may be an alluvial surface which should be analyzed with the techniques discussed in this section of the manual. Of the above, the problems of channel stability and surface stability are the most important in making a decision regarding the method of analysis.

1402 ANALYSIS REQUIREMENTS

In preparation of the analysis for development on an alluvial fan, the following items must be addressed:

1. Analysis to quantify the design discharges and the volumes of water, debris, and sediment associated with the major storm at the apex of the fan under current watershed conditions and under potential adverse conditions (e.g., deforestation of the watershed by fire). The potential for debris flow and sediment movement must be assessed considering the characteristics and availability of sediment in the drainage basin above the apex and on the alluvial fan.

2. Analysis which demonstrates that the proposed facilities will accommodate the major storm peak discharge, consisting of the total volume of water, debris, and sediment previously determined as well as the associated hydrodynamic and hydrostatic forces.

3. Analysis which demonstrates that the proposed facilities have been designed to withstand the potential erosion and scour forces .

4. Analysis or evidence which demonstrates that the proposed facilities will provide protection against flows that migrate or suddenly move to the project site from other portions of the fan.

5. Analysis which assesses the methods by which concentrated floodwater and the associated sediment load will be disposed of and the effect of those methods on adjacent properties.

6. Analysis which demonstrates that flooding from local runoff, or sources other than the fan apex, will be insignificant or will otherwise be accommodated by appropriate flood control or drainage measures.

Recently, FRENCH (1992) described a method to provide discharge estimate as a function of return period for drainage protection for developments crossing alluvial fans. The methodology is a modification of that used by FEMA to define floodplains on alluvial fans, and has been accepted by FEMA for such analyses in Clark County.

1403 PENINSULA DEVELOPMENT

A common occurrence in the Clark County area is peninsula development up an alluvial fan (see **Figure 1401**). A typical and appropriate question that the developer of the peninsula is asked is the effect of the development on downstream property owners. If the developer passes the flood flow through the development in a manner that simulates undeveloped conditions, then flow is neither concentrated nor diverted. As with all other design alternatives, there would be an increase in the quantity of flow due to the development. Routing of flows along streets with junctions can be handled with traditional hydraulics. If the developer chooses to build a hydraulic structure that does not pass the flow through the development, then he has the obligation to analyze the effect of his development on downstream property owners. (See **Figure 1402**).

It is recommended that peninsula development that does not pass flood flows through the development such as that shown in **Figure 1402** treat the development as a reduction in fan arc width. An example of an analysis appropriate to this problem is presented in Section 1406.

Finally, the engineer is reminded that even though down fan developments may be outside the currently defined alluvial fan flood hazard zone, large developments can modify the flood plain boundaries. That is, size of the development may become a factor. For examples, see Mifflin (1988), French (1987) and the example given in Section 1406.

1404 ADDITIONAL CONSIDERATIONS

The existing FIRM's, in general, estimate the extent of floodplains under conditions existing at the time of analysis.

The engineer must recognize and take into consideration that the development of areas on alluvial fans - even minor development such as streets and culverts - can have a very significant and crucial impact on drainage patterns. The engineer must ensure that all drainage systems match.

Sediment transport on alluvial fans is a crucial concern to both CCRFCD and FEMA. The analysis of the effects of sediment transport is to a large degree more of an art than a science. The engineer must consider in a reasonable fashion sediment transport. The engineer must realize that in unlined channels there is an equilibrium sediment load. If the actual sediment load transported exceeds the equilibrium load, then deposition occurs. However, if the sediment load is less than the equilibrium load, erosion will occur.

1405 ALLUVIAL FAN FLOOD PROTECTION MEASURES

Three general approaches may be taken to flood management on alluvial fans. They are based on size and density of the planned development. The approaches are:

1. Whole Fan Protection
2. Subdivision or Localized Protection
3. Single Lot/Structure Protection

1405.1 Whole Fan Protection

Whole fan protection can be achieved by utilizing the following measures:

1. Levees
2. Channels
3. Detention basins
4. Debris basins/fences/deflectors/dams

Whole-fan protection includes large scale structural measures appropriate to use on extensively developed fans, and which are most cost effective in high density situations. Structures must be designed to intercept upstream watershed flow and debris at the apex and to transport water and sediment around the entire urbanized fan. Structures must be designed to withstand scour, erosion, sediment deposition, hydrostatic forces, impact and hydrodynamic forces, and high velocity flows. Continual maintenance is essential for optimal operation and can be costly. These structures are most often funded through federal and state sources, but can also be financed through special regional districts, local governments or developers.

1405.2 Subdivision or Localized Protection

Individual subdivision or a localized development can be protected from flood hazards by utilizing the following measures:

1. Drop structures
2. Debris fences
3. Local dikes, channels
4. Site plans to convey flow
5. Street design to convey flow
6. Elevation on armored fill

These are smaller scale measures that can be used throughout moderate density fans to safely trap debris and to route water and sediment around or through individual residential developments.

1405.3 Single Lot or Structure Protection

A single lot or a structure can be protected from flood hazard by using the following protection measures:

1. Elevate and properly design foundations
2. Floodwalls and berms
3. Reinforcement of uphill walls, windows and doors against debris impact

These measures are most cost effective when implemented at low development densities.

1406 EXAMPLE APPLICATION

1406.1 Introduction

The following example is provided to demonstrate basic problems and analysis for developments on alluvial fans and may not necessarily represent the best method of alluvial fan analysis for all situations. For all submittals to FEMA for conditional or final Letters of Map Amendment or Revision, the engineer must analyze alluvial fans with a method acceptable to FEMA.The CCRFCD and the local entities do not guarantee that the analysis and information presented in this example is acceptable to FEMA.

1406.2 Example Development

In **Figure 1403**, a typical virgin (undeveloped) alluvial fan with FEMA flood hazard zones is delineated. In **Figure 1404**, an example proposed development on this typical virgin fan is shown. With regard to the proposed development on the alluvial fan (**Figure 1404**), the following should be noted:

1. The proposed development is within the 100-year floodplain defined by FEMA. It has been previously decided that potential flood flows will not be passed through the development.

2. The northern boundary of the proposed development, line M', will consist of a street and floodwall. The street/floodwall system will be designed such that all flows impinging on M' will be discharged at point A. Given the size of this development relative to the width of the alluvial fan, the method of Mifflin (1988) should be considered in designing the floodwall.

3. The line AB is a street/floodwall system. However, the intersection at point A of Streets M'and AB is designed so that there is no preferential flow direction.

4. The line CBD is an existing street. and the down-fan point beyond which FEMA alluvial fan methods of analysis are no longer appropriate since there are preferential directions of flow.

Given the situation shown in **Figures 1403** and **1404**, the question is what effect will the proposed development have on the downstream undeveloped property.

1406.3 Example Analysis

It must be realized that from a technical viewpoint it is virtually impossible to develop rectilinear street systems on an alluvial fan without concentrating and diverting flow since alluvial fans are best described by curvilinear coordinate systems. In the following steps, a method of analyzing the hypothetical situation is suggested. This is not the only procedure available, and it may not be the best procedure in other situations. The engineer evaluating the hypothesized situation must be experienced and willing to exercise sound engineering judgment.

Step 1: The procedures used by FEMA contractors to define flood hazard zones on alluvial fans should be carefully reviewed. First, review FEMA (1983) which is summarized in French (1987). Second, review FEMA (1985) that presents a modified and improved methodology. It is-important to determine which methodology was used to determine the flood hazard zones. In **Table 1401** (A and B) the difference in the flood hazard zone boundaries between FEMA (1983) and FEMA (1985) for depth and velocity are summarized based on the example in Section 1406. These results indicate that the new methodology is significantly more conservative than the former methodology.

The flood hazard zones in **Figures 1403** and **1404** were determined using FEMA (1985). If the previous FEMA methodology had been used, it is recommended that the analysis be redone using the FEMA (1985) methodology.

Step 2: Obtain values of the FEMA alluvial hazard zone parameters Z, S_z and C used in delineating the flood hazard zones shown in **Figures 1403** and **1404**. Also, obtain any additional information or data that is available regarding the analysis. For the alluvial fan in **Figure 1403**:

Z = 2.29 (transformation mean)
S_z = 0.4965 (transformation standard deviation)
C = 7.4 (transformation coefficient)

Note: These values are those used in examples by FEMA (1983, 1985) and FRENCH (1987).

Step 3: The proposed development lies below the bifurcation point on the alluvial fan (see **Figure 1404**) and is therefore in the FEMA (1985) multiple channel area. Within the multiple channel region, the various FEMA depth zone boundaries are estimated by the trial and error solution of

$$Y = [(0.0917\,(n)^{0.6}\,(S)^{-0.3}\,(Q)^{0.36}] + [(0.001426\,(n)^{-1.2}\,(S)^{0.6}\,(Q)^{0.48}] \qquad (1401)$$

where y = depth of flow (ft), n = Manning's “n” value for the fan (n= 0.02 is a reasonable assumption), S = fan slope (ft / ft), and Q = flow rate (cfs) corresponding to y.

Within the multiple channel region, the FEMA velocity zone boundaries are calculated by

$$Q = 99314\,(n)^{4.17}\,(S)^{-1.25}\,(U)^{4.17} \qquad (1402)$$

where u = velocity (ft / s) and Q = flow rate (cfs).

Step 4: The positioning of the proposed development on the alluvial fan suggests that its effect is equivalent to a transverse reduction in alluvial fan width. That is, the new alluvial fan boundary is on the west side TEC and on the east side TA'AB. Given these fan boundaries, the FEMA analysis for delineating flood hazard zones must be repeated.

The log-Pearson Type III standard deviates (K) are computed for the discharges corresponding to each depth and velocity zone boundary by

$$K = (\log Q - Z) / S_Z \qquad (1403)$$

The probability of occurrence (P) of the discharges for the required depth and velocity boundaries are determined by interpolation of the deviate values (K) in IAC, 1982. Given that the proposed development is in the multiple channel region the fan arc width is estimated as

$$W = 3610\,(A)\,(C)\,(P) \qquad (1404)$$

where A = avulsion coefficient and W = fan arc width (ft). Without additional information, a reasonable estimate of A is 1.5.

For example, to determine Q at the FEMA 0.5 ft depth boundary, solve **Equation 1401** with y = 0.5 ft, n = 0.02, and S = 0.03:

$$Q = 310 \text{ cfs.}$$

The log-Pearson Type III standard deviate from **Equation 1403** is:

$$K = [\ \log (310) - 2.29\ ] / 0.4965 = 0.4055$$

Then, following FEMA and interpolating among the log-Pearson deviate values in IAC, 1982:

$$P\ (Q \ \$310) = P\ (K \ \$\ 0.4055) = 0.3438$$

In performing the interpolation, it was assumed that the skew coefficient is zero which is a reasonable assumption for the Clark County area unless other data and information are available.

The fan width corresponding to this depth boundary is determined by **Equation 1404**:

$$W = 3{,}610 = 3{,}610\ (1.5)\ (7.4)\ (0.3438)$$
$$W = 13{,}800 \text{ ft}$$

The fan widths corresponding to velocity boundaries are summarized in **Table 1401 (c)**.

As indicated in **Figure 1404**, the impact of the development on downstream property owners is to incorporate the whole undeveloped area (ECBA) into the 1 ft depth 6.0 fps flood hazard zone whereas previous to development part of the area was in the 6.0 fps velocity zone and part in the 5.0 fps zone. While this is a rather minor change, it should be recognized that this change may result in some increased erosion on the adjoining and downstream property.

The situation in **Figure 1405** is the same as that shown in **Figure 1404** with the exception that the development has beenmoved to the center of the fan. The question is whether or not this rearrangement changes the answer previously obtained.

The answer is no because the FEMA methodology is a probabilistic methodology. The proposed development again limits the fan transverse width. Thus, the answer previously obtained is valid.

EXAMPLE DEPTH AND VELOCITY ZONE BOUNDARY DETERMINATIONS

A. COMPARISON OF FEMA (1983) AND FEMA (1985) RESULTS REGARDING DEPTH ZONE BOUNDARIES

DEPTH ZONE BOUNDARY FT	FEMA (1983) FT	FEMA (1985) FT
2.5	110	110
1.5	1,240	1,240
0.5	9,290	13,780

B. COMPARISON OF FEMA (1983) AND FEMA (1985) RESULTS REGARDING VELOCITY ZONE BOUNDARIES

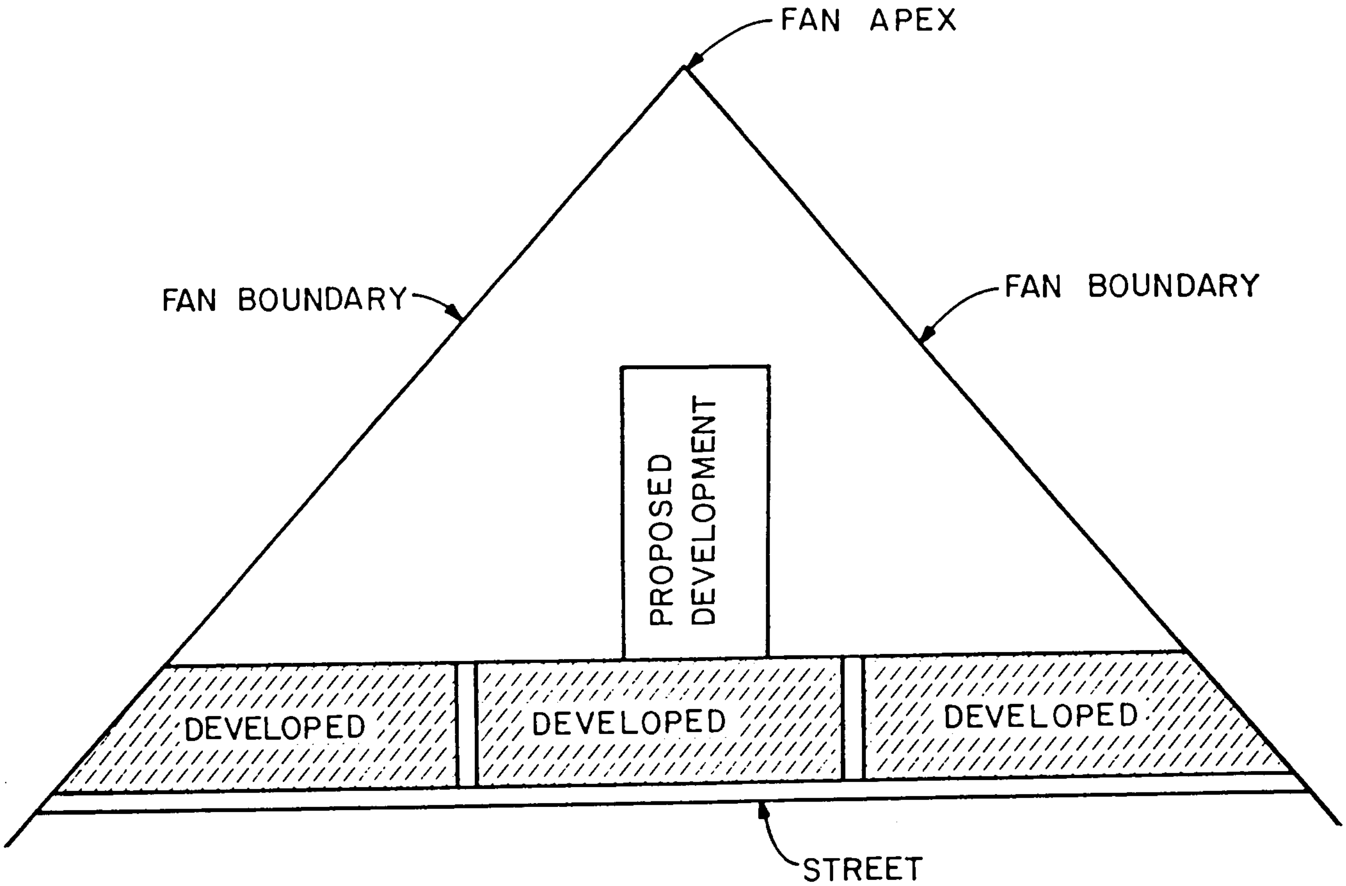
VELOCITY BOUNDARY FT	FEMA (1983) FT	FEMA (1985) FT
6.5	390	390
5.5	1,580	4,400
4.5	4,430	12,280
3.5	8,640	26,360

C. SUMMARY OF VELOCITY ZONE BOUNDARIES FOR PROPOSED DEVELOPMENT IN FIGURE 1404.

VELOCITY ZONE BOUNDARY VALUE FT/S	Q FT^3/S	K	P	W FT
6.5	1611	1.8471	0.0331	1330
5.5	803	1.2381	0.1099	4400
4.5	348	0.5067	0.3065	12,280
3.5	122	-0.4102	0.6578	26,360

Revision	Date

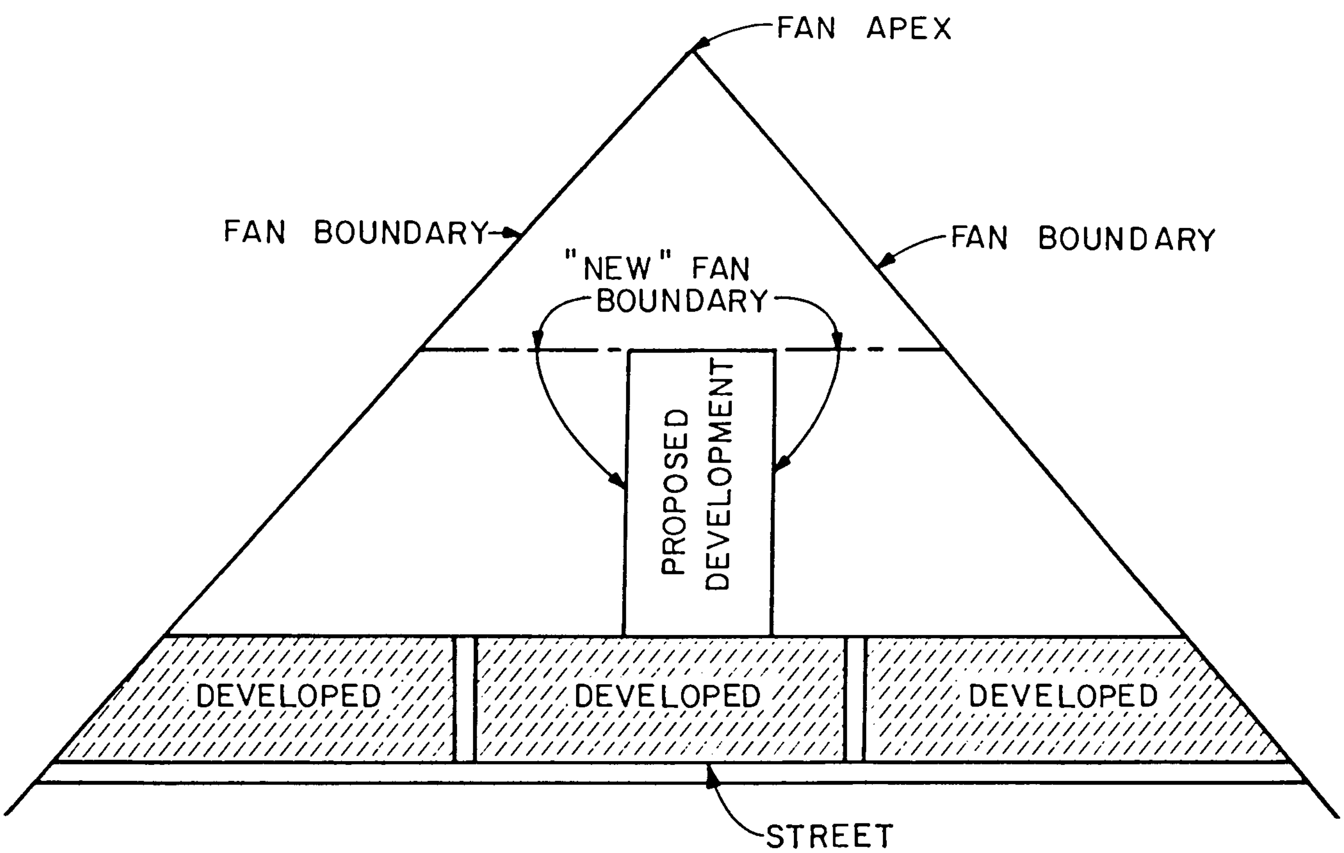
TYPICAL PENINSULA DEVELOPMENT ON AN ALLUVIAL FAN



Revision	Date

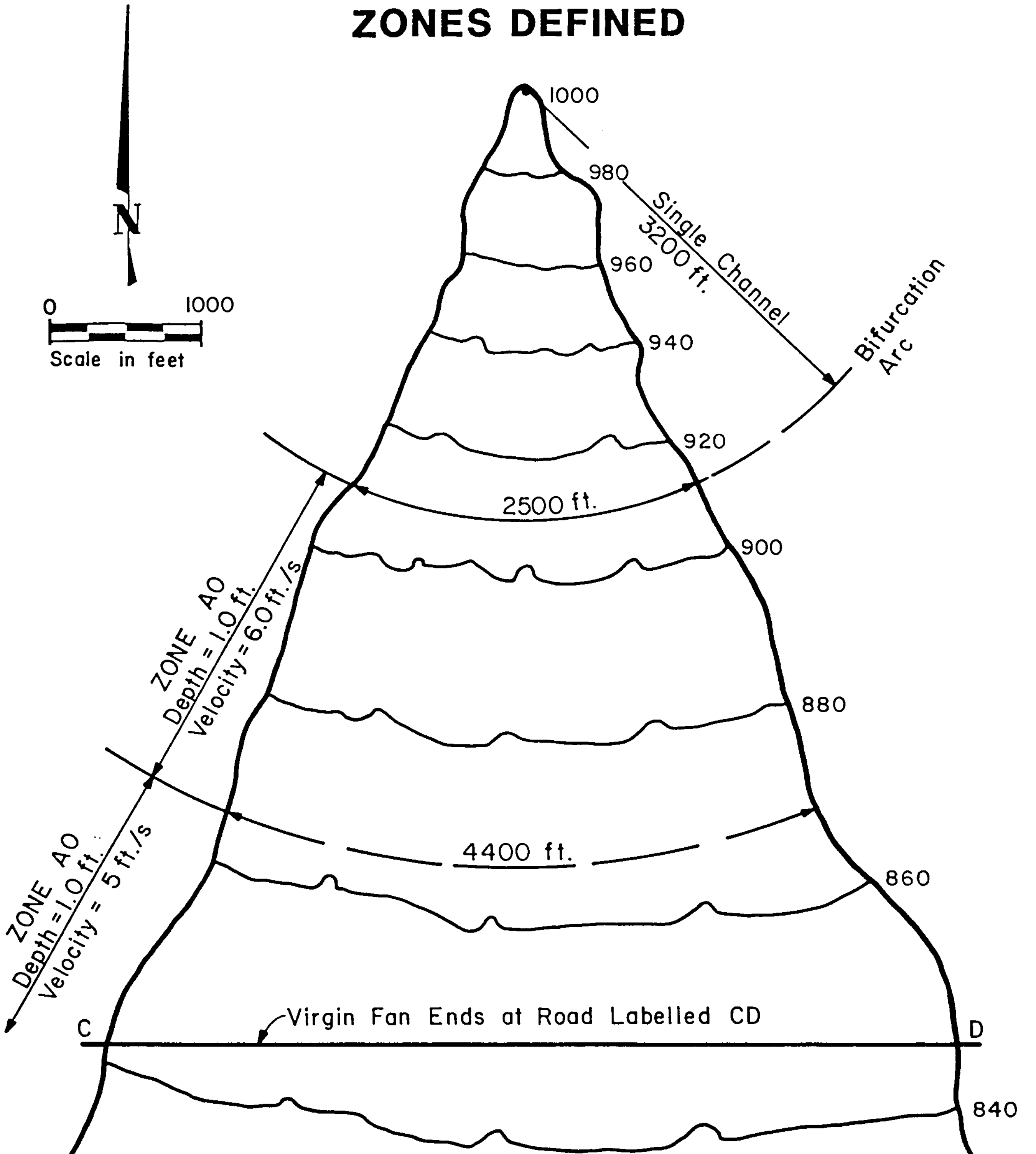
EFFECTS OF TYPICAL PENINSULA DEVELOPMENT

(FOR DEVELOPMENTS THAT DO NOT PASS FLOW THROUGH THE DEVELOPMENT)



Revision	Date

EXAMPLE VIRGIN FAN WITH FLOOD HAZARD ZONES DEFINED



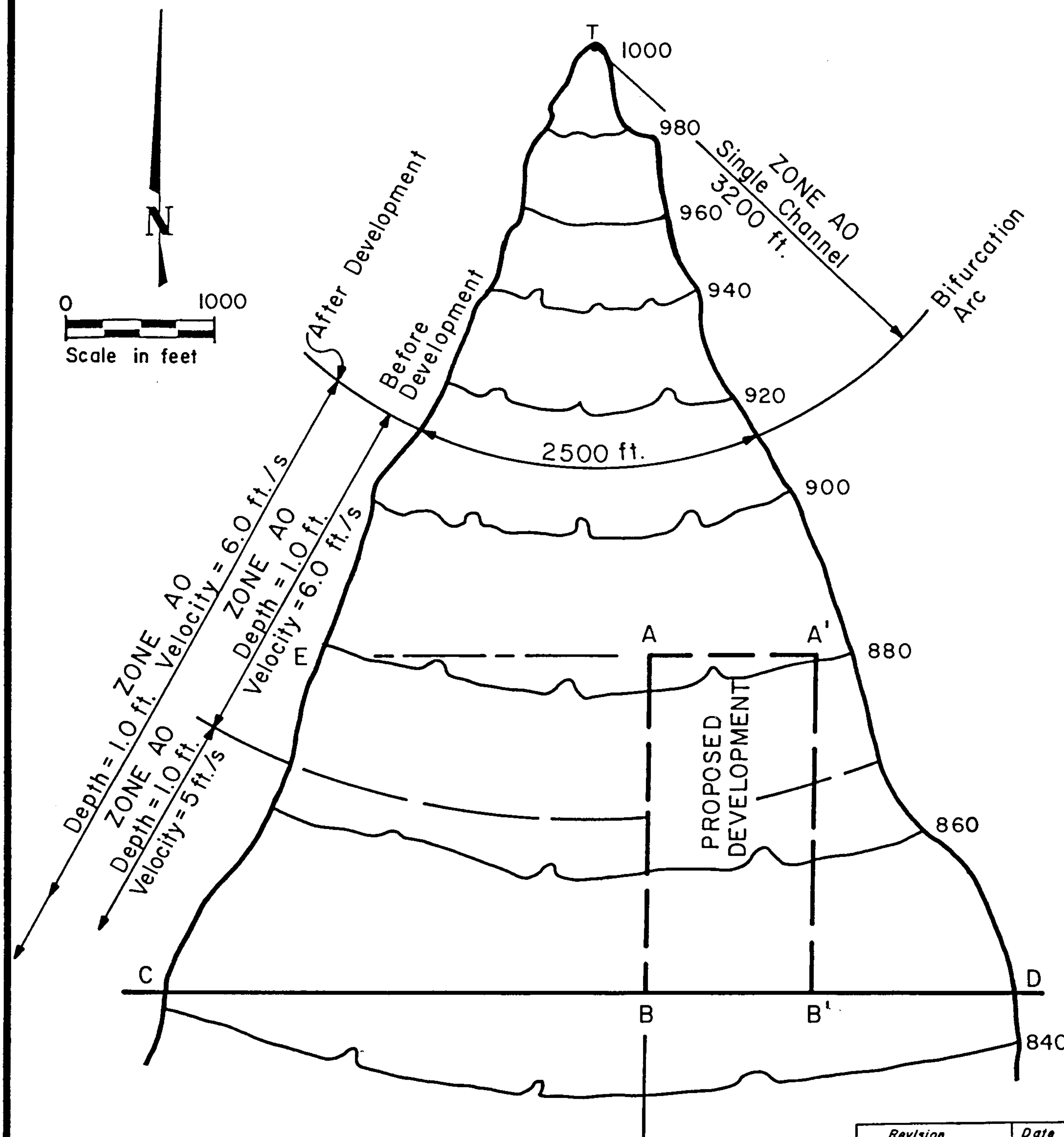
Revision	Date

WRC ENGINEERING

REFERENCE:

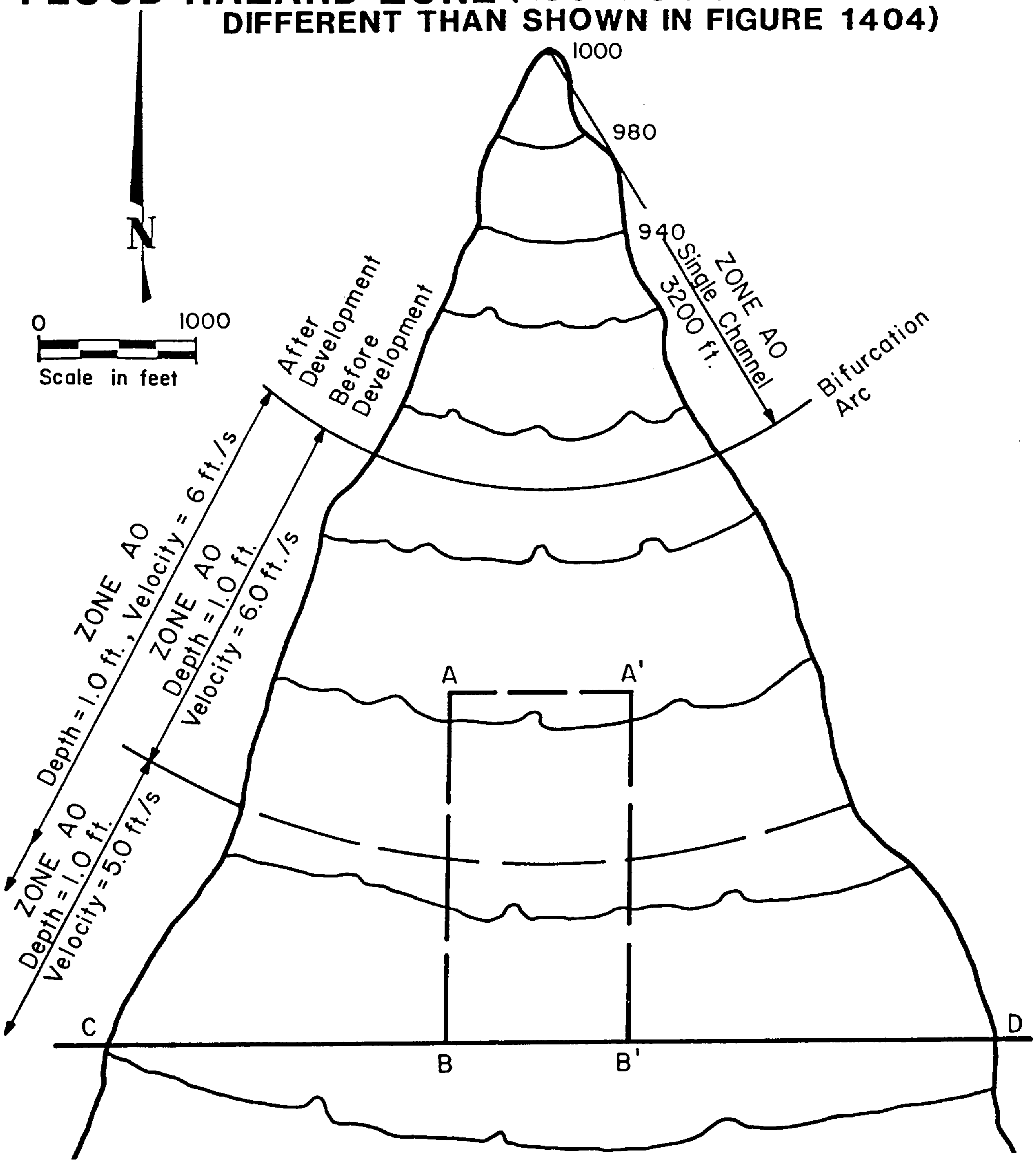
FIGURE 1403

EXAMPLE IMPACT OF DEVELOPMENT ON FLOOD HAZARD ZONES



Revision	Date

IMPACT OF DEVELOPMENT POSITIONING ON FLOOD HAZARD ZONE (LOCATION OF DEVELOPMENT DIFFERENT THAN SHOWN IN FIGURE 1404)



Revision	Date

CLARK COUNTY REGIONAL FLOOD CONTROL DISTRICT
HYDROLOGIC CRITERIA AND DRAINAGE DESIGN MANUAL

SECTION 1500
STRUCTURAL BEST MANAGEMENT PRACTICES

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Section 1500
Structural Best Management Practices

1501 INTRODUCTION

Presented in this section are design criteria for structural Best Management Practices (BMPs) for the control of surface water quality in Las Vegas Valley. These BMPs have been identified as having potential effectiveness in Las Vegas Valley based on the types of water quality conditions expected in this area, and on documented BMP performance in other areas. Criteria for the following BMPs are presented:

- Parking Lot Low Impact Development (LID)
- General Low Impact Development (LID)
  - Disconnected Impervious Areas
  - Landscape Swale
  - Depressed Medians
  - Depressed Landscaping
  - Buffer Strip
  - Pervious Overflow Parking
  - Sand / Media Filters
  - Oil / Water Separators
  - Oil / Grit Separators
  - On-Site Water Quality Basin
  - Infiltration Trench

There are currently no federal, state or local stormwater regulations which require the installation of specific water quality features *i*n most new or existing developments in the Las Vegas Valley. The exception is new and redeveloped non-residential sites larger than 1.0 acre, for which low impact development (LID) measures are required by the Storm Water Management Plan. LID measures for sites smaller than 1 acre are encouraged for developers *desiring to mitigate the impacts of urban development on surface water resources in* the *Las Vegas Valley.*

There are three general categories of conditions for which urban stormwater quality management practices may be applied: (1) existing urban development; (2) new urban development and substantial redevelopment; and (3) construction activity. The BMPs in this section are primarily applicable to areas of new urban development, where land is available to devote to installations of this type and where design flexibility exists. In certain cases, these structural BMPs may be effective in retrofitting existing developed areas to control water quality problems. However, it is generally more cost-effective to rely on nonstructural BMPs (e.g., source controls, housekeeping practices, public education, and employee training) in these applications. BMPs related to construction activity are primarily directed toward erosion control. This issue is covered separately in the Las Vegas Valley Construction Site Best Management Practices Guidance Manual.

The design criteria presented in this section provides general guidelines for design of the selected structural BMPs. They do not represent detailed plans or specifications for the improvements. The information presented herein is intended to assist the designer in selecting the best BMP for a particular application. For each BMP the following information is provided:

Description of Facility (including schematic drawing)
Water Quality Benefits
Applications
Limitations
Design Criteria
Maintenance

The BMPs presented in this section are by no means all-inclusive. New and creative methods of controlling pollution are continuously generated by owners and contractors. However, it is required that the Contractor monitor and prove the effectiveness of a new BMP when submitting the site design for approval. The local entity will require documentation of the effectiveness and design criteria for proposed BMPs that are not contained in this section.

If more detailed design information is desired for the structural BMPs discussed in this Manual reference may be made to the following documents:

Strategic Plan for Use of Regional Detention Basins for Water Quality Management, Las Vegas Valley Stormwater Quality Management Committee, November 2012. (http://lvstormwater.com/)

Proposed Parking Lot Low Impact Development Program for Las Vegas Valley, Las Vegas Valley Stormwater Quality Management Committee, February 2013. (http://lvstormwater.com/)

Stormwater Management Plan, Las Vegas Valley Stormwater Quality Management Committee, August 2011. (http://lvstormwater.com/)

County of San Diego SUSMP Standard Urban Stormwater Mitigation Plan Requirements for Development Applications; County of San Diego, January 2011.
(http://www.sdcounty.ca.gov/dpw/watersheds/susmp/susmp.html)
Storm Water Standards; City of San Diego, January 2011.
(http://www.sandiego.gov/stormwater/plansreports/standards.shtml)

Development Planning for Stormwater Management, A Manual for the Standard Urban Stormwater Mitigation Plan (SUSMP); Los Angeles

County Department of Public Works, September 2002.
(http://ladpw.org/wmd/NPDES/)
Center for Watershed Protection, numerous miscellaneous publications on BMP selection and design. (http://www.cwp.org/)

New York State Stormwater Management Design Manual, New York State Department of Environmental Conservation (Center for Watershed Protection), August 2010.
(http://www.dec.ny.gov/chemical/29072.html)

Urban Storm Drainage Criteria Volume 3 Best Management Practices; Urban Drainage and Flood Control District, Denver, Colorado, November 2010.
(http://www.udfcd.org/downloads/down_critmanual_home.htm)

Denver Regional Stormwater BMP Implementation Guidelines City & County of Denver Water Quality Management Plan, 2004.
(http://www.udfcd.org/downloads/down_sw_bmp.htm)

Green Industry BMP for the Conservation & Protection of Water Resources in Colorado; The Green Industries of Colorado (Wright Water Engineers, Inc), May 2008.
(http://www.greenco.org/current-bmps.html)

Caltrans Treatment BMP Technology Report; California Department of Transportation Division of Environmental Analysis, April 2008.
(http://www.dot.ca.gov/hq/env/stormwater/annual_report/2008/annual_report_06-07/attachments/Treatment_BMP_Technology_Rprt.pdf)

Stormwater Best Management Practice Handbook New Development and Redevelopment; California Stormwater Quality Association, January 2003.
(https://www.casqa.org/casqastore/entitiy/tabid/169/c-4-best-management-practice-bmp-handbooks.aspx)

Stormwater Best Management Practice Handbook Industrial and Commercial; California Stormwater Quality Association, January 2003.
(https://www.casqa.org/casqastore/entitiy/tabid/169/c-4-best-management-practice-bmp-handbooks.aspx)

Stormwater Best Management Practice Handbook Municipal; California Stormwater Quality Association, January 2003.
(https://www.casqa.org/casqastore/entitiy/tabid/169/c-4-best-management-practice-bmp-handbooks.aspx)

1502 Parking Lot LID Measures

1502.1 Description

Low impact development (LID) measures can be used for managing runoff from areas of new development and significant redevelopment (NDSR). LID measures must be applied to parking lots for all new non-residential sites of 1 acre and larger in the Las Vegas Valley. For the purpose of LID measures, a parking lot is defined as all of the impervious area outside the building footprint. Parking lot LID policies and design criteria are discussed in more detail in *Proposed Parking Lot Low Impact Development Program for Las Vegas Valley,* Las Vegas Valley Stormwater Quality Management Committee, February 2013.

1502.2 Design Criteria

Parking lot designs must be submitted to local entities with the technical drainage study required by Municipal code. A list of accepted parking lot BMPs that can be used in the Las Vegas Valley is provided in **Table 1501**.

BMP requirements shall adhere to this manual. A stepped approach to determine whether a parking lot is categorized as small, medium or large for the purpose of determining required BMPs is shown below. Any project site that is less than 1 acre in size is categorized as small; parking lot size is not a factor in determining required BMPs for small sites. For project sites that are greater than or equal to 1 acre, the parking lot size is used to determine if the category is medium or large. Sites having parking lots that are less than or equal to 1 acre are categorized as medium. Sites with parking lots greater than 1 acre are categorized as large.

Total Site Size	Parking Lot Size	Parking Lot Category
< 1 acre	Not Applicable	Small
≥ 1 acre	≤ 1 acre	Medium
≥ 1 acre	> 1 acre	Large

Once the parking lot category is determined, the BMP requirements for the parking lot design can be determined as shown in **Table 1502**. BMP requirements are more stringent as the parking lot size increases.

The designer may choose how to meet the parking lot BMP design requirements depending on specific site characteristics. In order to maximize the water quality benefit of the parking lot BMPs, the designer is encouraged to locate BMPs to capture runoff from portions of the site with the highest potential to generate pollutants.

There are no specific BMP requirements for small parking lots; however, the designer is encouraged to install any of the BMPs that are listed for parking lots in the medium or large categories.

For the Medium parking lot category, the design requirements consist of disconnecting impervious areas for at least 75 percent of the parking lot area from the onsite drainage network. This may be accomplished by directing runoff to pervious areas such as landscaping, depressed medians, parking strips, or stormwater retention basins. The design criteria are based on meeting the accepted minimum standards for the BMPs selected. The designer may choose to install a treatment BMP listed in the Large parking lot category.

For the Large parking lot category, treatment BMPs are required to treat stormwater runoff from at least 75 percent of the parking lot. Because typical site drainage layouts must account for application of flood control criteria, treatment BMPs may need to be sized to accommodate runoff from the contributing building area as well. BMPs are to be sized for the 85th percentile storm, and hydrologic and hydraulic calculations are required to be submitted for the BMP design.

The requirement to treat 75 percent of the parking lot area, either through disconnecting impervious areas or installing treatment devices, recognizes that grading and other site conditions will prevent directing all site runoff to some type of BMP. However, designers are encouraged to exceed the minimum standard when this can be done cost effectively in order to maximize the potential water quality benefits of the BMP system.

LID measures that are incorporated in the parking lot design should be planned during the site design process. Certain LID measures, including depressed medians, islands, and rock-lined swales, help to disconnect impervious areas and remove suspended particles in the stormwater runoff, prior to connection to the MS4. Parking Lot BMPs can be integrated into perimeter landscaped areas (e.g., buffers) and interior landscaped areas (e.g., medians) required by local development standards whenever possible. When incorporated into landscaping features, tributary areas to individual BMPs should be kept to less than 0.5 acres whenever possible, and should never exceed 1.0 acre.

Figure 1501 through **Figure 1503**, provide ideas on how parking lots can be designed to comply with the water quality regulations. The designer is required to comply with the planning code for the municipality in which the site is located. The conceptual layouts incorporate general planning requirements, however, specific planning requirements such as those for landscape buffers are determined based in part on the adjacent property. The conceptual layouts are designed to represent a commercial property that is approximately 5 acres in size, with the total parking lot area of approximately 3.3 acres.

Figure 1501, Conceptual Layout 1, represents a parking lot that can be designed so that the runoff can be directed towards the perimeter landscape buffer areas that incorporate a landscaped swale. These swales are designed as rock or xeriscaped swales to convey runoff to a low point, which discharges either directly to the MS4 or to a main drainage pipe and then into the MS4. **Figure 1502**, Conceptual Layout 2, represents a parking lot that is not designed to direct all runoff towards swales installed in the perimeter landscape buffer areas. One example could be that the adjacent right-of-way at this area is higher in elevation than the onsite grade. In order to still use the landscape swale BMP, the Developer could use swales in depressed medians within the site, combined with perimeter swales where feasible.

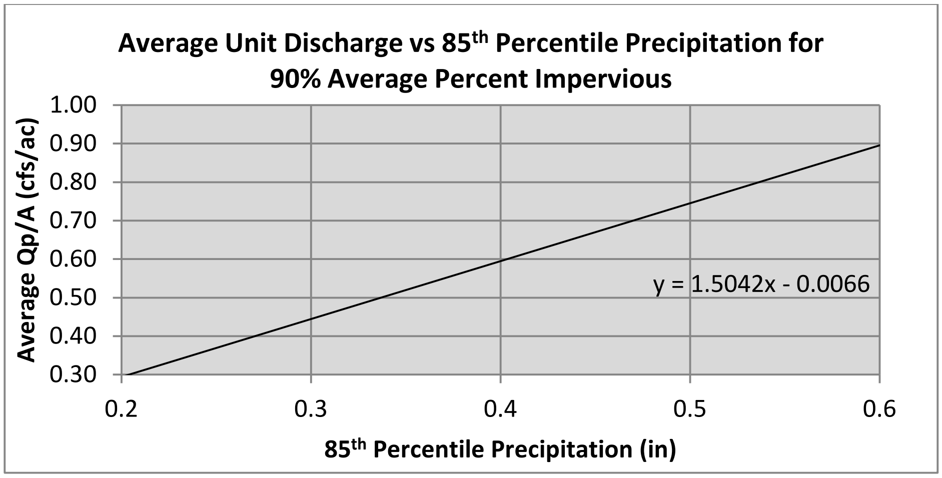
Treatment control devices rather than BMP swales may be used, however, the amount of parking lot area required to be treated would need to be calculated. **Figure 1503**, Conceptual Layout 3, shows a conceptual level layout for this scenario, which relies on a combination of modular pavers, sand/oil separators, and underground detention to provide treatment for at least 75 percent of the parking lot area. To size the treatment control BMPs, the approaches described in section 1502.3 should be used. Note that for treatment devices, hydraulic sizing may need to account for the contribution of runoff from building areas draining through the parking lot.

1502.3 Water Quality Design Storm

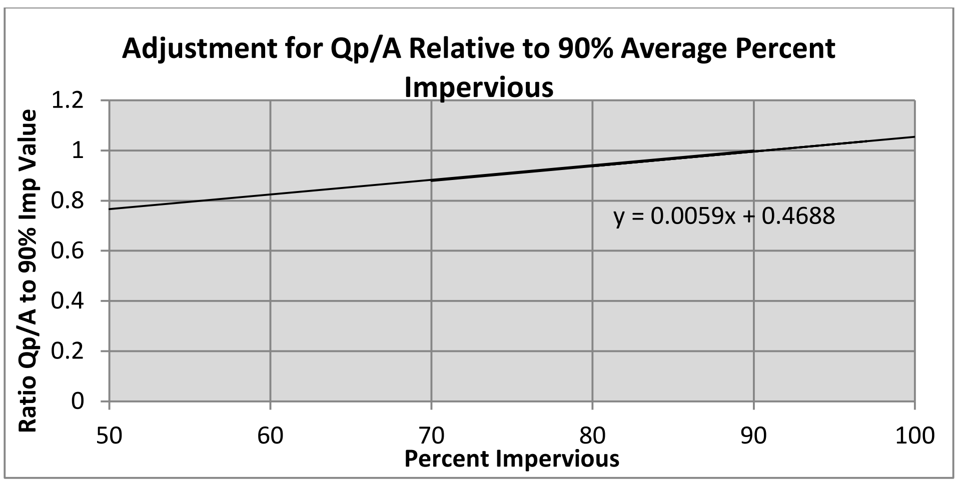
This section describes the steps to be followed to determine the design hydrology for sizing parking lot BMPs in the Las Vegas Valley.

1. Determine BMP Design Precipitation - 85th Percentile Rainfall Depth
   a. Locate site on CCRFCD Design Rainfall Map for 2-year, 6-hour storm
   b. Determine the adjusted 2-year rainfall depth (D_2) for site, per CCRFCD methods
   c. Compute ratio of D_2 site to D_2 for the McCarran Airport Area
   d. Compute 85th percentile rainfall depth (D_{85}) as 0.32 x D_2 ratio (0.32 is D_{85} for McCarran Area)

2. Calculate BMP Design Peak Discharge
   a. Use the following chart or regression equation to calculate the unit discharge (Q_P/A) for the 90 percent average percent impervious area condition based on the D_{85} value.

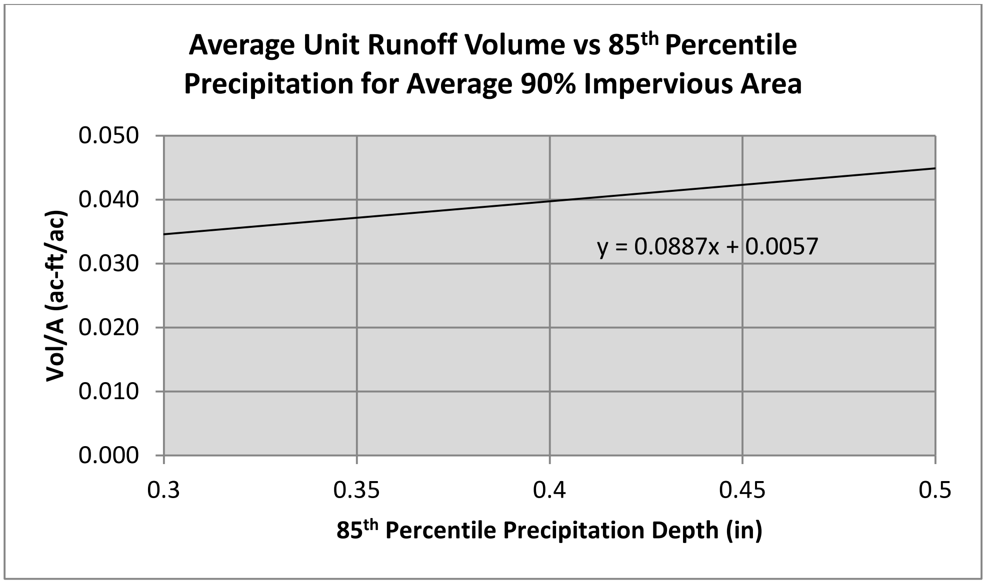


b. Use the following chart or regression equation to calculate the unit discharge for the site based on the actual percent impervious area of the parking lot and associated landscaped areas.



c. Calculate the peak design discharge in cfs as $Q_P = Q_P/A \times A$ where Q_P/A is from step (2b) and A is in acres.

3. Calculate BMP Design Runoff Volume
   a. Use the following chart or regression equation to calculate the unit runoff volume (Vol/A) for the 90 percent impervious condition based on the D_{85} value.



b. Use the chart or regression equation in step (2b) to calculate the unit runoff volume for the site based on the actual average percent impervious area of the parking lot and associated landscaped areas.

c. Calculate the design runoff volume in acre-feet as Vol = Vol/A x A where Vol/A is from step (3b) and A is in acres

Sample Calculation

Given: New Commercial Center in Summerlin area
Site Area = 100 acres
D_2 site / D_2 McCarran Area = 1.25
percent impervious area = 85% (parking lot and associated landscaped area)

1. Determine BMP Design Precipitation - 85th Percentile Rainfall Depth (D_{85})

D_{85} = 1.25 x 0.32 = 0.40 inches

2. Calculate BMP Design Discharge

Q_P/A (90%) = 1.5042 x (0.40) + 0.0066 = 0.60

Find Adjustment factor for percent impervious
Q_P/A = 0.60 x [(0.0059[85]) + 0.4688) = 0.58

Q_P = 0.58 x 100 acres = 58 cfs

3. Calculate BMP Design Runoff Volume

Vol/A (90%) = 0.0887 x 0.40 + 0.0057 = 0.04
Vol/A = 0.04 x (0.0059 x [85] + 0.4688) = 0.04; Vol = 0.04 x 100 acres = 4.0 ac-ft.

1503 LOW IMPACT DEVELOPMENT (LID)

1503.1 Description

LID measures can be used for managing runoff from areas of new development and significant redevelopment (NDSR). LIDs consist of a variety of site planning and site design measures or practices to minimize the impact of individual urban developments on stormwater quality and quantity. Developers proposing the use of measures that rely on infiltration to dispose of stormwater must perform studies to demonstrate that infiltration will be effective and will not adversely affect groundwater or downstream surface water quality. LID measures determined to be feasible for use in the Las Vegas Valley NDSR program can be found in **Table 1503**.

The BMPs presented in this section can be utilized for sites complying with the Parking Lot LID program or other site designs requiring BMPs. These BMPs are recommendations and the designer is not limited to the BMPs in this section. The designer is encouraged to contact the local entity when alternative BMPs are considered for use.

1503.2 Water Quality Benefits

Managing site runoff can significantly reduce the amount of pollutants conveyed to receiving waters. By incorporating the LID measures described below into site planning, runoff water quality can be improved by using simple technologies, employing basic practices, and educating those present on the site. Reducing pollutants in site runoff can reduce the need for downstream treatment to meet water quality goals.

1503.3 Disconnected Impervious Areas

1503.3.1 Description

This BMP consists of disconnecting impervious areas from downstream storm drainage infrastructure. Any surface in the landscape that cannot effectively absorb or infiltrate rainfall is an impervious area. This includes driveways, roads, parking lots, rooftops, and sidewalks. On natural landscapes, rainfall is absorbed into the soil and vegetation which naturally slows down, spreads out, and soaks up precipitation and runoff. A stable supply of groundwater is provided from water percolating into the soil, and the runoff is naturally filtered of impurities before it reaches creeks, streams, rivers, and bays.

The amount of impervious cover increases as areas become more developed. Natural filter systems are no longer in place to intercept the runoff which has serious implications for water quality and flood control. Typical pollutants in runoff from impervious areas include oil, litter, sediment, bacteria and in some cases, herbicides and pesticides.

1503.3.2 Water Quality Benefits

Concentrations and loads of the following pollutants are typically reduced by the LID BMP described in this section:

- Trace metals
- Hydrocarbons
- Litter

1503.3.3 Applications

This BMP applies to all urban areas. A beneficial way of mitigating stormwater runoff from impervious areas is to direct the runoff from parking lots and roads to pervious and vegetated soils.

1503.3.4 Limitations

Limitations may occur in areas with poor soils, where infiltration would not occur or could exacerbate groundwater quality problems. Limitations may also occur in ultra-urban areas where the site has no pervious area, including but not limited to high density areas.

1503.3.5 Design Criteria

Ways to mitigate the stormwater runoff by disconnecting impervious areas include:

- Breaking up flow directions from paved surfaces – Impervious surfaces are designed to allow stormwater to run off in a dispersed manner in several directions. The drainage off impervious surfaces is directed onto adjacent vegetated soil and not onto other pavements or into storm sewers.

- Minimize directly connected impervious areas - The impact of impervious surfaces is reduced by minimizing directly connected impervious areas, directing runoff from the impervious areas to pervious areas and/or small depressions, and in the process disconnecting hydrologic flow paths.

- Locating impervious surfaces to drain to natural systems - Existing zones of vegetation, from forested zones to scrub vegetation, are used for management of stormwater runoff, often with some sort of landforming to achieve volume control. The scale of this technique can vary from microcontrol by redirecting sidewalk and driveway runoff to adjacent vegetation to conveyance of runoff from larger impervious surfaces to natural areas on the development site.

1503.3.6 Maintenance

Sweep impervious areas regularly to collect loose particles and litter. Wipe up spills with rags and other absorbent material immediately, but do not hose down the area to a storm drain. Pervious areas receiving runoff should be cleared regularly of litter and accumulated sediment.

1503.4 Landscape Swale

1503.4.1 Description of Facility

The purpose of the swale is to settle out sediment, which has the potential to have urban pollutants bound to the particulate matter. The required length of the swale to settle out particulate pollutants is dependent on multiple factors, including velocity, characteristics of the stormwater, characteristics of the swale, and length of the swale. For the Parking Lot LID program, sizing of swales will be based on minimizing the slope of the swale to minimize the swale length and flow velocities. Xeriscaped or rock swales can be sized using the standard sizing requirements, as shown in Section 1503.4.5.

Runoff from any impervious areas on a developed site can be directed to swales, including parking lots, roof tops, parking covers, etc. The swales can be installed either within the parking lot area or at the perimeter of the site.

1503.4.2 Water Quality Benefits

Concentrations and loads of the following pollutants are typically reduced by the LID BMP described in this section:

- Sediment
- Total Phosphorus
- Trace Metals
- Hydrocarbons

Estimated pollutant removal efficiencies for vegetated landscape swales are given below. Pollutant removal efficiencies for rock lined or xeriscaped swales are expected to be lower.

Pollutant	Removal Efficiency
TSS	77%
Total Phosphorus	8%
Total Nitrogen	67%
Trace Metals	83-90%

Source: CASQA New Development and Redevelopment Handbook, 2003

1503.4.3 Applications

Landscape swales are typically located in parks, parkways, private landscaped areas, or in development buffers, and can also be used as pre-treatment devices for other structural treatment controls. They are primarily used as stormwater conveyance systems; however they are limited in their ability to mitigate large storms. Landscape swales are best utilized in low to moderate sloped areas as an alternative to ditches and curb and gutter drainage.

1503.4.4 Limitations

The effectiveness of landscape swales is decreased by compacted soils, frozen ground conditions, short grass heights, steep slopes, large storm events, high discharge rates, high velocities, and a short runoff contact time. Landscape swales also require a sufficient amount of available land area and they may not be appropriate for industrial sites or locations where spills may occur. In areas where burrowing animals are abundant, landscape swales may not perform effectively. Additionally, the infiltration rates of local soils can limit the application of landscape swales, unless underdrains are installed. Another possible issue is the formation of mosquito breeding habitat if water does not drain or infiltrate.

1503.4.5 Design Criteria

Tributary Area (ac)	Swale Slope	Minimum Swale Length (ft)	Size of Swale
≤ 0.5	≤ 1%	30	9” deep maximum, 6 ft wide at top average, 4ft wide at top minimum, 4:1 side slope
0.5 - 1.0	≤ 1%	40	9” deep maximum, 6 ft wide at top average, 4ft wide at top minimum, 4:1 side slope

Minimum criteria for sizing landscaped swales for parking lot BMPs using xeriscaped surface treatments are provided below. Sizing was based on minimizing flow velocities and maximizing flow paths to encourage settlement of sediment.

BMP swales should be as long as practically feasible to provide as much treatment as possible. The minimum length is 30 ft for tributary areas up to 0.5 acres, and 40 ft for tributary areas of up to 1.0 acre. When incorporating swales into landscaping features, tributary areas to individual BMPs should be kept to less than 0.5 acre whenever possible, and should never exceed 1.0 acre unless hydrologic and hydraulic calculations are provided. BMP swales should have a minimum average top width of 6 ft, with a minimum top width at any point of 4 ft and a maximum flow depth of 9 inches for water quality flows (approximately 2-yr storm). When combined with a larger flood control facility these dimensions should apply to the portion of the swale carrying low flows. When lined with rock, landscaped swale BMPs should have a minimum rock size of 3-inch D_{50}. When combined with a larger flood control facility, rock lining should be sized based on the anticipated flood velocities in the facility.

The Developer may propose to use an alternative size swale, if supporting calculations are submitted.

1503.4.6 Maintenance

Remove any sediment or debris build-up when the depths exceed 3 inches. Inspect for pools of standing water. At regular intervals, re-grade to restore design grade and re-vegetate.

Use of heavy equipment for mowing and removing plants/debris should be avoided to minimize soil compaction. Disturbed areas should be stabilized with seed and mulch, or rock lining, as necessary.

If a spill occurs and hazardous materials contaminate soils in the swales, the affected areas should be removed immediately and disposed of in a manner that complies with federal and state regulations. The swale soils and materials should be replaced as soon as possible.

1503.5 Depressed Median

1503.5.1 Description of Facility

Medians come in the depressed or raised form, or are made flush with the surface of the carriageways. Only the depressed medians are discussed in this section.

For stormwater management purposes, depressed medians are beneficial in reducing stormwater runoff as it collects in the depressed median before being carried to a drainage system. They also provide an open green space in urban areas.

1503.5.2 Water Quality Benefits

Concentrations and loads of the following pollutants are typically reduced by the LID BMP described in this section:

- Sediment
- Nutrients
- Metals
- Hydrocarbons

1503.5.3 Applications

In stormwater management applications, depressed medians serve as a means of collecting stormwater run-off from the roads and parking areas.

1503.5.4 Limitations

The depressed medians are limited in providing nutrient removal from the stormwater runoff and may even add to the nutrient loading due to landscape material such as wood chips applied to the edges of the depressed medians.

1503.5.5 Design Criteria

The depressed median configuration has more efficient drainage and is therefore normally used on rural roads.

The depressed median side slopes should be traversable and should preferably be ≤ 1V:20H; should not exceed 1V:10H, particularly when a median barrier is installed; and must not exceed 1V:6H.

Median width is measured between the edges of opposing traffic lanes or parking areas, including the adjacent offside (right hand) shoulders, if any. See **Figure 1504**. Median widths range from a minimum of 3 ft in urban areas to 75 ft or more in rural areas.

Generally, depressed medians should be kept clear of obstructions within the clear zone requirements of the road and need to avoid the use of head walls, unprotected culvert openings, solid sign foundations, fragile sign posts, and light poles. Where longitudinal culverts are required, ex. under cross overs, the ends facing traffic should be sloped at 1V:20H (preferably), no steeper than

1V:6H, and provided with traversable safety grates. All other drainage inlets should be designed with their tops flush with the ground.

Depressed medians should be designed to ensure that they are as maintenance free as possible. The amount of time that maintenance personnel will be required to spend on the median will be reduced thereby minimizing their exposure to traffic hazards. Planting should consist of xeriscaped species suitable for the Las Vegas Valley. Landscaping design and species selection will depend on specific circumstances and require specialist input. Features in medians that limit horizontal sight distance on curves should be located such that adequate sight distance is achieved.

1503.5.6 Maintenance

The depressed median should be inspected at regular intervals and sediment or debris buildup should be removed when necessary. Inspect for pools of standing water, re-grade to restore design grade, and re-vegetate when necessary. After large storm events, the depressed median should be inspected.

1503.6 Depressed Landscaping

1503.6.1 Description of Facility

Depressed landscaping consists of a low-lying vegetated area underlain by a sand reservoir and an underdrain system. A combination of soils and plants is utilized to remove pollutants from storm water runoff through physical and biological processes. A typical depressed landscape design includes a depressed ponding area, topsoil or mulch layer, an engineered soil mix of peat or leaf compost and clean sand, and a gravel sub-base layer with an underdrain system consisting of a perforated HDPE or PVC pipe in a gravel layer. As an option for pre-treatment, a vegetated buffer strip can be added. Designing depressed landscapes with slotted curb or curb cuts slows the velocity of the storm water runoff from small events as it passes through and distributes it evenly along the length of the ponding area. Water ponded to approximately 6 inches gradually infiltrates into the underdrain system, underlying soils or is evapotranspired over a period of days. To divert excess runoff from large events away from the surrounding area, the depressed landscape area should be graded for the flows to move towards the conventional storm drain system.

1503.6.2 Water Quality Benefits

Concentrations and loads of the following pollutants are typically reduced by the LID BMP described in this section:

- Sediment
- Nutrients

- Metals
- Hydrocarbons

1503.6.3 Applications

Depressed landscaping may be installed in commercial, residential, and industrial areas. In addition to providing benefits in water quality, other advantages of depressed landscaping are that it improves an area's aesthetics, reduces irrigation needs, and reduces or eliminates the need for an underground storm drain system.

1503.6.4 Limitations

Clogging may be a problem in depressed landscaping, especially in areas with high sediment loads in the runoff.

Sediment controls and fencing should be installed to prevent clogging and compaction of engineered and existing site soils from heavy equipment, if located in the vicinity of active construction sites.

1503.6.5 Design Criteria

The following design criteria have been adapted from the Truckee Meadows Structural Controls Design Manual. Other BMP manuals may be utilized for further information.

For an efficient depressed landscape, using the appropriate plant species can stabilize banks and increase the infiltration capacity and storm water treatment effectiveness. Sand and gravel must be rinsed with potable water prior to installation and construction of the sand filter since locally available sand and gravel is typically washed with a high pH, recycled water.

The size of the depressed landscape area is a function of the drainage area and the runoff generated from the area. The recommended minimum dimensions of the depressed landscape area are 15 ft wide by 40 ft long. For areas longer than 20 ft, the depressed landscape should be twice as long as they are wide.

To allow infiltration and prevent clogging, a liner such as a woven geotextile fabric layer should be used to mitigate migration of sediment into the underdrain system.

Flows in excess of the water quality volume (WQV) should drain out of the depressed landscape and flow to another treatment control or the conventional storm drain system.

The following equation is used to determine the ponding depth of the depressed landscape based on the available surface area (SA):

$$DWQV = \frac{WQV}{SA} \times 12$$

Where:

DWQV = ponding depth of the temporary ponded water (in)

WQV = Water Quality Volume (ft^3)

SA = Surface Area of ponding area based on the length and width at the toe of the sideslopes(ft^2)

The maximum recommended ponding depth is 12 inches and minimum ponding depth is 6 inches with water standing no longer than 7 days. This prevents problems with mosquito breeding and certain plants that cannot tolerate standing water.

The recommended engineered soil mixture is 50-60 percent clean sand (ASTM 33), 20- 30 percent peat or certified compost with a low P-index, and 20-30 percent topsoil.

The pH of the soil should be between 5.5 and 6.5.

Approximately 3 inches of shredded hardwood mulch should be applied to the area.

A general rule is 1 tree or shrub for each 50 ft^2 of landscape detention area.

Plant selection and layout should consider aesthetics, maintenance, native versus non-native invasive species, and regional landscaping practices.

Some trees should be planted on the perimeter to provide shade and shelter.

1503.6.6 Maintenance

Depressed landscaping should be inspected monthly and after large storm events upon installation. Once the depressed landscaping has proven to work efficiently and vegetation is established, inspections can be reduced to a semi-annual schedule. A health evaluation of the xeriscape and shrubs should be conducted biannually. Replacement of mulch is generally required every two to three years. If ponding is observed for seven (7) consecutive days or longer, cleaning of the underdrain system or replacement of engineered soils may be required. Key maintenance areas include inlet areas, under drain, and overflow structures.

If a spill occurs and hazardous materials contaminate soils in depressed landscaping area, the affected areas should be removed immediately and disposed of in a manner that complies with federal and state regulations. The landscaping soils and materials should be replaced as soon as possible.

1503.7 Buffer Strips

1503.7.1 Description of Facility

A buffer strip, also known as the vegetative buffer strip, is a gently sloping area of vegetative cover that runoff water flows through before entering a stream, storm sewer, or other conveyance. The buffer strip may be an undisturbed strip of natural vegetation or it can be a graded and planted area.

Buffer strips reduce the flow and velocity of surface runoff, promote infiltration, and reduce pollutant discharge by capturing and holding sediments and other pollutants carried in the runoff water. They act as a living sediment filter that intercepts and detains stormwater runoff. Buffer strips function much like vegetated or grassed swales, however, they are fairly level and treat sheet flow across them, whereas grassed swales are indentations that treat concentrated flows running along them.

1503.7.2 Water Quality Benefits

Concentrations and loads of the following pollutants are typically reduced by the LID BMP described in this section:

- Sediment
- Trace metals

Estimated pollutant removal efficiencies for buffer strips are given below. Pollutant removal efficiencies with xeriscape landscaping or rock lining are expected to be lower.

Pollutant	Removal Efficiency
TSS	74%
Total Phosphorus	-52%
Total Nitrogen	15%
Trace Metals	66-88%

Source: CASQA New Development and Redevelopment Handbook, 2003

1503.7.3 Applications

Buffer strips are usually used in conjunction with other sediment collection and slope protection practices for temporary or permanent control. Consider the use of buffer strips with level spreaders or diversion measures such as earth dikes and slope drains. Also, installing silt fences up gradient can prevent overloading of the buffer strip.

Buffer strips may be placed in various locations between the source of sediment (road surface, side slopes) and a natural or constructed waterway. They are inexpensive and easily constructed, and can be installed at any time if climatic conditions allow for planting.

Where a site can support vegetation, the buffer strip may be used, but is best suited for areas where the soils are well drained or moderately well drained and where the bedrock and the water table are well below the surface. Buffer strips also provide low to moderate treatment of pollutants in stormwater while providing a natural look to a site and can provide habitat for wildlife. They can also screen noise and views if trees or high shrubs are planted on the filter strips.

1503.7.4 Limitations

Buffer strips cannot treat high velocity flows and do not provide enough storage or infiltration to effectively reduce peak discharges to predevelopment levels for design storms. This lack of quantity control dictates use in rural or low density development or where peak discharge reduction is not an objective.

Buffer strips require a slope less than 5 percent and have a large land requirement. They also require low to fair permeability of natural subsoil and often concentrate water, which significantly reduces their effectiveness. Pollutant removal is unreliable in urban settings. The useful life of a buffer strip may be short due to clogging by sediments and oil and grease.

Vegetated buffer strips typically require supplemental irrigation. In the Las Vegas Valley use of xeriscaping is recommended.

1503.7.5 Design Criteria

The following design criteria have been adapted from the Idaho Department of Environmental Quality (IDEQ) Storm Water BMP Catalog and the Truckee Meadows Structural Controls Design Manual. Other BMP manuals may be utilized for further information.

Registered professional civil engineers and landscape architects should work together on the design of vegetated buffer strips. It is recommended that slopes should not be greater than 4 percent (2 to 4 percent is preferred). Maximum drainage area is 5 acres.

Channelized flow across buffer strips should not be permitted; sheet flow is preferred to be maintained across buffer strips. This can be created by installing a level spreader at the top edge of the buffer strip along a contour.

The top of the vegetated buffer strip should be installed 2 – 5 inches lower than the impervious surface that is being drained. If supplemental irrigation is not available, use drought tolerant species in the buffer strip to minimize irrigation in dry climates.

In many cases, a vegetative buffer strip will not effectively control runoff and retain sediments unless employed in conjunction with other control measures. Where heavy runoff or large volumes of sediment are expected, provide diversion measures or other filtering measures above or below the buffer strip.

1503.7.6 Maintenance

Regularly inspect the buffer strip to ensure it is functioning properly and remove sediments when necessary. Check for damage by equipment and vehicles. For areas which are newly planted, check progress of plant growth.

Ensure additional erosion is not caused by water flowing through the buffer strip and that it is not forming ponds from erosion of the buffer strip. Promptly repair any damage from equipment, vehicles, or erosion.

1503.8 Pervious Pavement

1503.8.1 Description of Facility

Pervious pavement is designed to infiltrate stormwater runoff instead of shedding it off the surface. The advantage of pervious pavement is decreasing the effective imperviousness of an urbanizing or redevelopment site, thereby reducing runoff and pollutant loads leaving the site. Although there are many types of pervious pavement, the only type approved for use in the Las Vegas Valley is modular block pavers.

1503.8.2 Water Quality Benefits

Concentrations and loads of the following pollutants are typically reduced by the LID BMP described in this section:

- Sediment
- Nutrients
- Trace Metals
- Hydrocarbons

1503.8.3 Applications

The modular block pavements are best suited for use in low vehicle movement zones, such as roadway shoulders, driveways, parking strips, parking lots, and particularly overflow parking areas. Vehicle movement (i.e., not parking) lanes that lead up to one of these types of porous pavement parking pads may be better served, but not always, by solid asphalt or concrete pavement.

1503.8.4 Limitations

Modular block pavers are less effective and are prone to clogging when used to receive runoff from other areas (i.e., not direct rainfall).

Unless underlying soils are extremely permeable, larger storms will either sheet flow off the site, or if not graded properly, will pond on the site. To address these concerns, the following limitations are recommended (source: Truckee Meadows Source Control Manual):

- Not to be applied in heavily trafficked areas or where speeds exceed 30 miles per hour.
- Care must be taken when applying in commercial or industrial areas.
- May become clogged if not properly installed and maintained.
- Maintenance costs can be relatively high if the blocks frequently become clogged with sediment from offsite sources. No additional area should drain onto the paver area.
- Porous pavements may cause uneven driving surfaces and may be problematic for high heel shoes.
- May not be suitable for areas that require wheelchair access because of the pavement texture.

1503.8.5 Design Criteria

Design criteria for pervious pavers are available in various reference manuals and should be designed by registered professional civil engineers. The following are conceptual level design criteria.

Pervious pavement can be designed with or without underdrains. Whenever underdrains are used, infiltrated water will behave similarly to interflow and will surface at much reduced rates over extended periods of time.

In the Las Vegas Valley, the only type of approved pervious pavement for overflow parking or other applications is modular block pavers. This consists of concrete block units with open surface voids laid on a gravel sub-grade. These voids occupy at least 20 percent of the total surface area and are filled with sand (ASTM C-33 sand fine concrete aggregate or mortar sand) or sandy loam turf that has at least 50 percent sand. The modular block pavement used in pervious overflow parking may be sloped or flat.

1503.8.6 Maintenance

Maintenance measures for modular block pervious pavers are provided below.

Required Action or Practice	Maintenance Objective and Action	Frequency of Action
Debris and litter removal	Accumulated material should be removed as a source control measure.	Routine – As needed.
Sod maintenance	If sandy loam turf is used, provide lawn care, irrigation system, and inlay depth maintenance as needed.	Routine – As dictated by inspection.
Inspection	Inspect representative areas of surface filter sand or sandy loam turf for accumulation of sediment or poor infiltration	Routine and during a storm event to ensure that water is not bypassing these surfaces on frequent basis by not infiltrating into the pavement.
Rehabilitating sand infill surface	To remove fine sediment from the top of the sand and restore its infiltrating capacity.	Routine – Sweep the surface annually and, if need be, replace lost sand infill to bring its surface to be ¼ below the adjacent blocks.
Replacement of surface filter layer	Remove, dispose, and replace surface filter media by pulling out turf plugs or vacuuming out sand media from the blocks. Replace with fresh ASTM C-33 sand or sandy loam turf plugs, as appropriate.	Non-routine – When it becomes evident that runoff does not rapidly infiltrate into the surface. May be as often as every two year or as little as every 5 to 10 years.
Replacement of modular block paver	Restore the pavement surface. Remove and replace the modular pavement blocks, the sand leveling course under the blocks and the infill media when the pavement Surface shows significant deterioration.	Non-routine – When it becomes evident that the modular blocks have deteriorated significantly. Expect replacement every 10 to 15 years dependent on use and traffic.

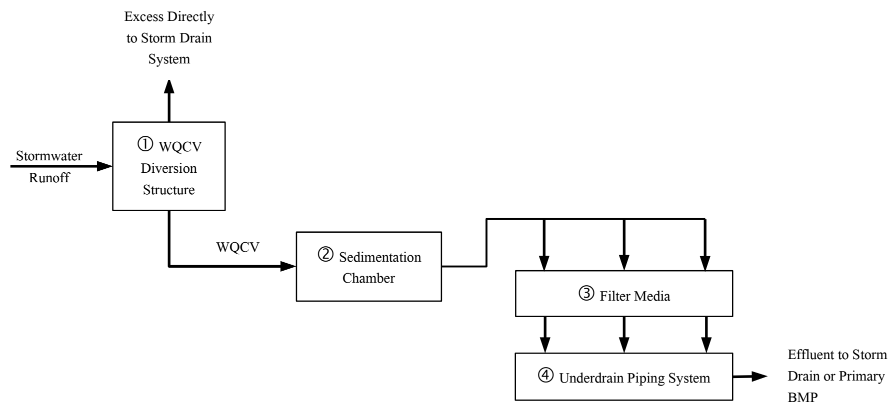
1503.9 Sand / Media Filters

1503.9.1 Description of Facility

Sand/media filters are used for filtering stormwater runoff through a sand layer into an underdrain system that conveys the treated runoff to the point of discharge or other stormwater system. The sand/media filters may be configured to be at the surface or if space is a constraint, then an underground sand/media filter is also an option.

A typical sand/media filter contains a two-stage treatment system which includes a pretreatment settling basin and a filter bed containing sand or other filter media. The filters are only designed to treat the water quality capture volume (WQCV) and not the entire storm volume. The WQCV represents the site runoff volume generated from the 85th percentile storm, which is 0.32 inches of rainfall in the McCarran Airport Rainfall Area.

There are a number of sand/media filter configurations but there are four basic components which most contain. These are shown schematically shown below.



(Courtesy of Los Angeles County Department of Public Works)

- Diversion Structure – This directs stormwater equivalent to the WQCV to the filter and bypasses the overflow directly to the point of discharge or other stormwater system. It is either incorporated into the filter itself or is a stand-alone device.
- Sedimentation Chamber – The removal of large grained sediments prior to exposure to the filter media is important to the long-term successful operation of any filtration system. The sedimentation chamber is typically integrated directly into the sand filter BMP but can also be a stand-alone unit if space permits.
- Filter Media – Typically consists of a 1-inch gravel layer over an 18- to 24-inch layer of washed sand. A layer of geotextile fabric can be placed between the gravel and sand layers.
- Underdrain System – Below the filter media is a gravel bed, separated from the sand by a layer of geotextile fabric, under which a series of perforated pipes is placed. The treated runoff is routed out of the BMP to the downstream stormwater facility.

The sand/media filtering systems are generally applied to land uses with a high percentage of impervious surfaces, which makes these BMPs suitable for the parking lot LID program.

Common sand/media filter alternatives are the surface sand filter, underground sand filter, perimeter sand filter, and the organic filter. These alternatives are illustrated in **Figures 1505 through 1509**. Each was developed and adapted by various governments and engineers to serve different water quality treatment goals or to accommodate different physical constraints. In addition to the filter alternatives described, there may be other alternative configurations which may also be useful for different land use applications or climatic conditions.

1503.9.2 Water Quality Benefits

Concentrations and loads of the following pollutants are typically reduced by the LID BMP described in this section. Sand/media filters are effective at removing total suspended solids with moderate removal effectiveness for total phosphorus. This is due to the physical straining, pollutant settling and pollutant adsorption to remove pollutants.

- Sediment
- Phosphorus
- Bacteria
- Trace Metals
- Hydrocarbons

Estimated pollutant removal efficiencies for media filters are given below. Compost filters use compost as the filter media. Multi-chamber filters have more than one chamber, either combining a settling chamber with a media chamber, or having two media chambers for improved removal efficiency.

Pollutant	Sand Filter Efficiency	Compost Filter Efficiency	Multi-Chamber Filter Efficiency
TSS	89%	85%	98%
TP	59%	4%	84%
Total Nitrogen	17%	-	-
Metals	72 - 86%	44 – 75%	83 – 89%

Source: CASQA New Development and Redevelopment Handbook, 2003

1503.9.3 Applications

The following recommendations in the application of the sand/media filter are adapted from the IDEQ Storm Water BMP Catalog.

Sand filters generally take up little space and can be added to retrofit existing sites. They can also be used on highly developed sites and sites with steep slopes. They are not recommended where high sediment loads are expected, unless pre-treatment (e.g. sedimentation chamber) is provided since the fine sediment clogs sand filters, or where the runoff is likely to contain high concentrations of toxic pollutants (e.g., heavy industrial sites).

Where there are smaller drainage areas, sand/media filtration trenches are used rather than sand filtration basins. A trench is typically placed along the perimeter of a parking lot. Trenches have experienced fewer problems with clogging than basins, perhaps because their use in the field has been limited to high-impervious cover sites where less suspended solids are generated. Sand/media filters should never be used as sediment basins during construction because of the potential for clogging.

1503.9.4 Limitations

The following limitations are adapted from the California Stormwater BMP Handbook.

Sand/media filters may be more expensive to construct than many other BMPs. They may also require more maintenance than some other BMPs depending upon the sizing of the filter bed. They generally require more hydraulic head to operate properly (minimum 4 feet). A high solids load will cause the filter to clog. Sand/media filters work best for relatively small, impervious watersheds. They can present aesthetic and safety problems if constructed with vertical concrete walls in residential areas. Some designs may maintain permanent sources of standing water where mosquito and midge breeding is likely to occur.

1503.9.5 Design Criteria

The following are design criteria for sand/media filters, according to the Truckee Meadows Structural Controls Design Manual. Other BMP Manuals may also be utilized to size the facility.

- Registered professional civil engineers should design the underground sand filters.
- The structure to detain the WQCV is to be designed based on the method in Section 1502.3.
- Flows in excess of the WQCV should be diverted around the underground sand filter with an upstream diversion structure.
- The maximum allowable depth of water in the underground sand filter (hmax) is determined by considering the difference between the inlet and outlet invert elevations.
- The sand filter layer should consist of a minimum 16-inch gravel bed (dg) covered with a minimum 18-inch sand filter layer (ds) and a minimum 2-inch gravel layer above the sand filter layer. Geotextile fabric

liners should be placed between the sand and gravel layers (e.g. above and below the sand layer).

- A woven geotextile fabric layer should be installed between the sand filter and the gravel under drain.
- The top of the sand and gravel filter should not have any slope or grade.
- Basins should be located off-line from the primary conveyance/detention system where possible and should be preceded by a pre-treatment sedimentation chamber in order to improve the effectiveness of sand filtration basins and to protect the media from clogging.
- In areas with high water table conditions and the possibility of groundwater contamination, liners are recommended for trenches and basins.
- Disturbed areas that contribute to sediments accumulating in the drainage area should be identified and stabilized.
- The locally available sand and gravel is typically washed with a high pH, recycled wastewater. (Sand and gravel must be rinsed with potable water prior to installation and construction of the sand filter.)

In order to size a sand/media filter bed two major underground filter types can be considered, the D.C. and Delaware.

The required bed area for the D.C. type underground sand filter is calculated using the following equation:

$$\boldsymbol{A_f} = \frac{(\boldsymbol{WQCV})\,(\boldsymbol{d_f})}{(\boldsymbol{k})(\boldsymbol{h_f} + \boldsymbol{d_f})(\boldsymbol{t_f})}$$

Where: Af = Surface area of filter bed (ft^2)

WQCV = Water Quality Control Volume (cf)

df = Filter bed depth (df=dg+ds) (ft)

k = Coefficient of permeability for the sand filter (typically 1.18 ft/hr for clean, well graded sand with d10 = 0.1 mm) (ft/hr)

hf = Average height of water above filter bed or one half of the maximum allowable water depth (2hf) over the filter bed (2hf=hmax-df) (ft)

tf = Design filter bed drain time or the time required for the WQCV to pass through the filter in hrs (Max: 48 hours) (hrs)

- Considering site constraints, assume a filter width (Wf) and calculate the filter length (Lf) using:

$$L_f = \frac{A_f}{W_f}$$

- Determine the volume of storage available above the filer bed using:

$$V_{tf} = A_f * 2h_f$$

- Compute the storage volume of the filter voids (Vv) by assuming a 40% void space and using:

$$V_v = 0.4A_f * d_f$$

- Compute the flow through the filter during filling assuming 1 hour to fill the voids using:

$$\boldsymbol{V_q = k * \frac{A_f(d_f+h_f)t_f}{d_f}} \text{ (assume k = 0.0833 ft/hr and } t_f = 1\text{ hr)}$$

- Compute net volume to be stored in the permanent pool awaiting filtration using:

$$V_{st} = WQCV - V_{tf} - V_v - V_q$$

- Compute the minimum length of the permanent pool using:

$$L_{pm} = \frac{V_{st}}{2h_f * W_f}$$

Furthermore, the following design criteria are applicable to the D.C. type underground sand filter:

Parameter	*Design Criteria*
Maximum drainage area	1.5 acres
Filter sand size	Concrete sand (Sec. 200.05.04 SSPWC*)
Typical sand coefficient of permeability (k) for sand with D_{max}=10 mm and Effective Size D_{10}=0.1	1.18 ft/hr**
Maximum diameter of gravel in upper gravel layer	1 in, Class C backfill (Sec. 200.03.04 SSPWC*)
Diameter of gravel in under drain gravel layer	½ to 1 in, Class B or C backfill (Sec. 200.03.03 & 200.03.04 SSPWC*)
Minimum size of under drain pipes	6-in Schedule 40 PVC
Minimum size of perforations in under drain pipes	3/8-in diameter
Minimum number of perforations per under drain pipe	6
Minimum spacing of perforations	6 in
Maximum spacing of under drain pipes	27 in center to center
Minimum volume of sediment chamber	20% of the WQCV

Minimum length of the clearwell chamber	3 ft

*SSPWC: Standard Specifications for Public Works Construction
**USCOE, 2001 EM 1110-2-1100 Part VI

The required bed area for the Delaware type underground sand filter is calculated as follows:

- When the maximum ponding depth above the filter (2hf) is less than 2.67 feet, the area of the sediment chamber (As) and the area of the filter chamber (Af) can be found using the following equation:

$$A_s = A_f = \frac{WQCV}{4.1h_f + 0.9}$$

- When the maximum ponding depth above the filter (2hf) is 2.67 feet or greater, use the following equation:

$$A_s = A_f = \frac{WQCV * d_f}{k(h_f + d_f)}$$

- Establish the dimensions of the facility assuming sediment chambers (As) and filter chambers (Af) are typically 18 to 30 inches wide. Use of standard grates requires a chamber width of 26 inches.

The following summarizes important design criteria that apply to the Delaware type underground sand filter:

Parameter	Design Criteria
Maximum drainage area	5 acres
Filter sand size	Concrete sand (Sec. 200.05.04 SSPWC*)
Typical sand coefficient of permeability (k) for sand with D_{max}=10 mm and Effective Size D_{10}=0.1	1.18 ft/hr**
Maximum diameter of gravel in upper gravel layer	1 in, Class C backfill (Sec. 200.03.04 SSPWC*)
Diameter of gravel in under drain gravel layer	½ to 1 in, Class B or C backfill (Sec. 200.03.03 & 200.03.04
Weir height between sedimentary chamber and sand filter	2 in above filter bed
Minimum size of under drain pipes	6-in Schedule 40 PVC
Minimum size of perforations in under drain pipes	3/8-in diameter
Minimum number of perforations per under drain pipe	6
Minimum spacing of perforations	6 in

Minimum weephole diameter	3 in
Minimum spacing between weepholes	9 in – center to center
Sedimentation chamber and sand filter trench width	18 to 30 in

*SSPWC: Standard Specifications for Public Works Construction
**USCOE, 2001 EM 1110-2-1100 Part VI

1503.9.6 Maintenance

The following guidelines for inspection and maintenance of sand filters have been adapted from the IDEQ Storm Water BMP Catalog.

Inspection Schedule:

Inspect the sand filters at least annually. Additionally, the observation well in a filtration trench should be monitored for water quality periodically. For the first year after completion of construction, the well should be monitored after every large storm (greater than 1 inch in 24 hours). A logbook should be maintained by the responsible person designated by the owner indicating the rate at which the facility dewaters after large storms and the depth of the well for each observation. Once the performance characteristics of the structure have been verified, the monitoring schedule can be reduced to an annual basis unless the performance data indicate that a more frequent schedule is required.

Sediment and Debris Removal:

Sediment build-up in the top foot of stone aggregate or the surface inlet should be monitored on the same schedule as the observation well. A monitoring well in the top foot of stone aggregate should be required when the trench has a stone surface. Sediment deposits should not be allowed to build up to the point where they will reduce the rate of infiltration into the device. As a general rule, remove silt when accumulation exceeds 0.5 inches and remove accumulated paper, trash and debris every 6 months or as necessary.

Sand Media Rehabilitation and Replacement:

Over time, a layer of sediment will build up on top of the filtration media that can inhibit the percolation of runoff. Experience has shown that this sediment can be readily scraped off during dry periods with steel rakes or other devices. Once sediment is removed, the design permeability of the filtration media can typically be restored by then forming grooves on the surface layer of the media. Eventually, however, finer sediments that have penetrated deeper into the filtration media will reduce the permeability to unacceptable levels, thus necessitating replacement of some or all of the sand. The frequency in which the sand media should be replaced is not well established and will depend on the suspended solids levels entering the system. Drainage areas that have disturbed areas containing clay soils will likely necessitate more frequent replacement. Properly designed and maintained sand filtration BMPs in arid climates have been found to function effectively, without complete replacement of the sand media, for at least 5 years and should have design lives of 10 to 20 years.

1503.10 Oil / Water Separators

1503.10.1 Description of Facility

Oil/water separators remove oil and other water insoluble hydrocarbons and settleable solids from stormwater runoff. Variations of this device include the Spill Control (SC), American Petroleum Institute (API) and the Coalescing Plate (CP). More detailed design criteria can be obtained for these special oil/water separators.

For the purposes of the parking lot LID program, the SC device (shown in **Figure 1510**) is likely to be sufficient and these types of separators are the least expensive and complex of the three. The SC device is a simple underground vault or manhole with a “T” outlet designed to trap small spills.

The owner of the site may elect to use an API or CP device; however, details of these devices are not included in this manual.

1503.10.2 Water Quality Benefits

Concentrations and loads of the following pollutants are typically reduced by the LID BMPs described in this section:

- Sediment
- Phosphorus
- Small Floatables
- Trace Metals
- Hydrocarbons

Oil / Water separators are appropriate "pretreatment facilities" for other BMPs such as on-site water quality basins, infiltration basins, and infiltration trenches.

1503.10.3 Applications

Oil/water separators have limited application in stormwater treatment. These treatment mechanisms are generally not well suited for stormwater runoff with high discharge rates, turbulent flow regime, low oil concentration or high suspended solids concentration. The primary use of oil/water separators will be in cases where oil spills are a concern or areas where oil and grease can accumulate, such as high traffic areas, loading docks, gas stations and parking lots. Oil/water separators should be located offline from the primary stormwater system. The contributing drainage area should be completely impervious and as small as necessary to contain the sources of oil. Under no circumstances should any portion of the contributing drainage area contain disturbed pervious areas that can be sources of sediment.

1503.10.4 Limitations

The following limitations are adapted from the IDEQ Storm Water BMP Catalog:

- Drainage area - 1 ac.
- Minimum bedrock depth – 8 ft
- NRCS soil type – A, B, C
- Drainage/flood control – no
- Max site slope – 15%
- Minimum water table – 8 ft
- Freeze/thaw - fair

1503.10.5 Design Criteria

The oil/water separator inflow design flow rate should be calculated using the hydrologic design criteria for parking lot BMPs for the Las Vegas Valley. Oil/water separators should be installed upstream of any pumps to prevent oils from emulsifying and also upstream of any other stormwater treatment facility. Stormwater from building rooftops and other impervious surfaces are not likely to be contaminated by oil and should not be discharged to the separator. Appropriate removal covers should be provided to allow access for observation and maintenance.

To size the oil water separator, consideration needs to be given on the wide distribution of sizes of the oil droplets in water. The oil/water separator is a propriety product which is sized to remove droplets of various sizes. Sizing of the oil/water separator is per the manufacturer's sizing requirements. Refer to manufacturer's guidelines to determine a suitable product for specific site.

1503.10.6 Maintenance

Oil/water separators should be cleaned frequently to keep accumulated oil from escaping during storms. As a general rule, they should be cleaned annually at a minimum to remove material that has accumulated, and again after any significant storm (i.e., 1 inch of rainfall within a 24 hour period).

General maintenance procedures should include the following:

- Weekly inspections of the facility by the owner.
- Replacement of the oil absorbent pads at least every 6 months, before both rainy seasons (July and December), or as needed.
- During cleaning operations, the effluent shutoff valve is to be closed.
- Dispose of waste oil and residuals in accordance with current local government health department requirements.
- Any standing water removed during the maintenance operation should be disposed to a sanitary sewer at a discharge location approved by the local government.
- Any standing water removed should be replaced with clean water to prevent oil carry-over through the outlet weir or orifice.

1503.11 OIL / GRIT SEPARATORS

1503.11.1 Description of Facility

Oil/grit separators are underground storage tanks with three chambers designed to remove heavy particulates, floating debris and hydrocarbons from stormwater, see **Figure 1511**. Stormwater enters the first chamber where heavy sediments and solids drop out. The flow moves into the second chamber where oils and greases are removed and further settling of suspended solids takes place. Oil and grease are stored in this second chamber for future removal. After moving into the third outlet chamber, the clarified stormwater runoff is then discharged to a pipe and onto a point of discharge or other stormwater system.

1503.11.2 Water Quality Benefits

Concentrations and loads of the following pollutants are typically reduced by the LID BMP described in this section.

- Sediment
- Phosphorus
- Small Floatables
- Trace Metals
- Hydrocarbons
- Nutrients

Oil and grit separators are appropriate "pretreatment facilities" for other BMPs such as on-site water quality basins, infiltration basins, and infiltration trenches.

1503.11.3 Applications

Oil/grit separators are best used in commercial, industrial and transportation land uses and are intended primarily as a pretreatment measure for high-density or urban sites, or for use in hydrocarbon hotspots, such as parking lots, gas stations and areas with high vehicular traffic. However, gravity separators cannot be used for the removal of dissolved or emulsified oils and pollutants such as coolants, soluble lubricants, glycols and alcohols.

Since re-suspension of accumulated sediments is possible during heavy storm events, gravity separator units are typically installed off-line. One of the most important selection criteria when considering an oil-grit separator is the long-term maintenance and operation costs, and the need for regular inspections and cleanout. Inspection and maintenance needs for such systems can be considered high relative to other stormwater BMPs. Therefore, the oil-grit separator system should only be constructed if the property owner or tenant of the site has both the physical and fiscal ability to perform regular inspection and maintenance of the system on a long-term basis.

Oil/grit separators are available as prefabricated proprietary systems from a number of different commercial vendors.

A hydrodynamic separator is primarily installed to separate floatables and grit/sediment from stormwater runoff from small urban areas. Generally these are precast concrete structures with a fiberglass insert which extends into the treatment chamber to provide dual wall containment of hydrocarbons. This system may be used in place of a traditional inlet structure upstream of a conventional treatment device in small drainage areas.

1503.11.4 Limitations

The oil/grit separator cannot effectively remove soluble pollutants, fine particles, or bacteria and can become a source of pollutants due to resuspension of sediment unless properly maintained. During large storms it is susceptible to flushing and is limited to being installed in relatively small contributing drainage areas of one acre or less of impervious cover. The oil/grit separator may be expensive to install and maintain compared to other BMPs. This is usually installed where the cost of land would be prohibitive or where resources are sensitive or valuable. Frequent maintenance is necessary and it requires proper disposal of trapped sediments and oils. It may also be an entrapment hazard for amphibians and other small animals

1503.11.5 Design Criteria

Sizing of the oil/grit separator is per the manufacturer's sizing requirements. Refer to manufacturer's guidelines to determine a suitable product for specific site.

The following is general outline of the design criteria to size a conventional oil/grit separator according to the Knox County Tennessee Stormwater

Management Manual. Other BMP Manuals may be utilized to size the facility; however, guidance provided below should be used to determine the WQCV.

Step 1

Calculate the WQCV based on the methods described for hydrologic design of parking lot BMPs in Las Vegas Valley.

Step 2

Calculate the rise velocity of oil droplets using Stokes Law:

$$V_p = \frac{1.79 \times 10^{-8}(S_w - S_p)D_p^2}{N}$$

Where:

Vp = Upward rise velocity of petroleum droplet (ft/s)

Sw = Specific gravity of water (0.998 to 1.000)

Sp = Specific gravity of the petroleum droplet (typically 0.85 – 0.95)

Dp = Diameter of petroleum droplet to be removed (microns)

N = Absolute viscosity of water (poises)

The expected temperature is generally chosen for cold winter months. Typical values for the specific gravity and absolute viscosity of water at various temperatures are shown below:

Temperature	S_w	N
32°F	0.999	0.01794
40°F	1.000	0.01546
50°F	0.999	0.01310
60°F	0.999	0.01129
70°F	0.998	0.00982

Step 3

Calculate the size of the oil/grit separator:

Error! Bookmark not defined. $$D = \left(\frac{Q}{RV_H}\right)^{0.5}$$

$$W = RD$$

$$L = \frac{V_H D}{V_p}$$

$$V_H = \frac{V_p D}{L} = 15\left(V_p\right)$$

Where:

D = Depth of unit (ft), generally between 3 and 8 ft

W = Width of unit (ft), usually twice the depth

L = Total Length of unit (ft), usually fifteen times the depth

Q = Design flow rate (cfs), i.e., the WQCV

R = Width to depth ratio, generally a value of 2 is recommended

VH = Allowable horizontal velocity (ft/s), maximum 0.05 ft/s

Vp = Upward rise velocity of petroleum droplet (ft/s)

- The total depth shall be adjusted by adding 1 foot of freeboard to the depth calculated using the equations above, or equations provided by a manufacturer.
- Top baffles should extend downward by 0.85D, and bottom baffles should extend upward by 0.15D, where D is the depth of the unit (in feet). The distribution baffle should be located at a distance of 0.10L from the inlet of the unit, where L is the length of the unit (in feet).

The following are general design criteria to consider when designing an oil/grit separator.

Location and Siting

- Any individual oil/grit separator shall have a contributing drainage area no greater than 1 acre.
- It is desirable to maintain reasonable dimensions by bypassing larger storm flows in excess of the design flow rates. It is preferred that oil-grit separators be located off-line where a separator can use an existing or proposed manhole with a baffle or other control.
- Oil/grit separator systems can be installed in almost any soil or terrain. Since these devices are underground, appearance is not an issue and public safety risks are low.
- The design loading rate for oil/grit separators is low; therefore, they can only be cost-effectively sized to detain and treat the WQCV. It is usually not economical or feasible to size an oil/grit separator to treat large design storms. Oil/grit separators require frequent maintenance for the life of the separator unit. Maintenance can be minimized (and performance can be increased) by careful planning and design, particularly upstream and downstream of the separator unit.

Physical Specifications/Geometry

- Design procedures for commercially available oil/grit separators are usually given by the manufacturer in simplified tables or graphs based on field testing and observed pollutant removal rates. Oil-grit separators must be constructed with watertight joints and seals.
- The separation chamber shall provide for three separate storage volumes, as follows:

  (1) A volume for separated oil storage at the top of the chamber;

  (2) A volume for settleable solids accumulation at the bottom of the chamber; and,

  (3) A volume required to give adequate flow-through detention time for separation of oil and sediment from the stormwater flow.
- Ideally, a gravity separator design will provide an oil draw-off mechanism to a separate chamber or storage area. This design is required where a gravity separator is utilized to treat oil, grease and/or petroleum hotspots.
- Oil/grit separators are typically designed to bypass runoff flows in excess of the WQCV peak flow. Some designs have built-in high flow bypass mechanisms, whereas others require a diversion structure or flow splitter located upstream of the device in the drainage system. Bypass mechanisms must minimize potential for captured pollutants from being washed-out or resuspended by large flows.
- Regardless of the bypass mechanism, an adequate outfall/outlet must be provided for both the discharge from the separator itself, and the bypassed discharge. Runoff shall be discharged in a non-erosive manner.
- The device shall be designed such that the velocity through the separation chamber does not exceed the entrance velocity.
- A trash rack shall be included in the design to capture floating debris, preferably near the inlet chamber to prevent debris from becoming oil impregnated.
- The total wet storage of the gravity separator unit shall be no less than 400 cubic feet per contributing impervious acre.

Manufactured Oil/Grit Separators

Manufactured separators should be selected on the basis of good design, suitability for the desired pollution control goals, durability, ease of installation, ease of maintenance, and reliability. Manufacturers generally provide design methods, installation guidelines, and proof of effectiveness for each application where used. These structures tend to include innovative methods of providing high flow bypass. However, it is incumbent upon the landowner to carefully investigate the suitability and overall trustworthiness of each manufacturer and/or subcontractor.

Variations of this device include the SC, API and the CP. More detailed design criteria can be obtained for these special oil/grit separators.

1503.11.6 Maintenance

Sediments and associated pollutants and trash are removed only when inlets or sumps are cleaned out, so regular maintenance is essential. Most studies have linked the failure of oil/grit separators to the lack of regular maintenance. The more frequent the cleaning, the less likely sediments will be resuspended and subsequently discharged. In addition, frequent cleaning also makes more volume available for future storms and enhances overall performance. Cleaning includes removal of accumulated oil and grease and sediment using a vacuum truck or other ordinary catch basin cleaning device. In areas of high sediment loading, inspect and clean inlets after every major storm. At a minimum, inspect oil/grit separators monthly, and clean them out at least twice per year. Polluted water or sediments removed from an oil grit separator should be disposed of in accordance with all applicable local, state and federal laws and regulations.

1503.12 On-Site Water Quality Basin

1503.12.1 Description of Facility

On-site water quality basins are depressed basins that can be utilized in the Las Vegas Valley to detain a portion of stormwater runoff following a storm event. See **Figure 1512**. Water is controlled by means of a hydraulic control structure to restrict outlet discharge. General objectives of on-site water quality basins are to remove particulate pollutants and to reduce maximum runoff values associated with development to their pre-development levels. Water quality basins may be berm-encased areas, excavated basins, or tanks.

On-site water quality basins do not maintain a permanent pool between storm events. A micropool is not necessary and likely cannot be sustained in the arid conditions of the Las Vegas Valley.

Outlets are designed to detain the volume of a water quality design storm for a minimum of 24 hours and a maximum of 72 hours to allow for the settling of particles and associated pollutants. In addition, on-site water quality basins provide flood control by either including additional temporary storage for peak flows above the WQCV or providing provisions for peak flows to pass through the outlet structure. On-site water quality basins are designed to detain and treat runoff from the smaller more frequent runoff events.

1503.12.2 Water Quality Benefits

On-site water quality basins provide moderate pollutant removal, provided that the recommended design features are incorporated. Although they can be effective at removing some pollutants through settling, they are less effective at removing soluble pollutants because of the absence of a permanent pool.
Concentrations and loads of the following pollutants are typically reduced by the LID BMP described in this section:

- Sediment
- Phosphorus
- Trace Metals
- Hydrocarbons
- Floatables

1503.12.3 Applications

On-site water quality basins require careful planning in order to function correctly. Of critical importance is the prediction of flow volumes and the design of an outlet structure to drain slowly enough to provide some water quality benefits but rapidly enough to be empty for the next storm. Since the basin drains completely between storms, the first flush of the next storm tends to resuspend sediments deposited during the last storm.

On-site water quality basins often serve multiple purposes. In addition to flood control and water quality benefits, the basin may be used for recreation, such as a playground or picnic area, when dry. Thus, aesthetic considerations are important in siting basins. Use of good landscaping principles is encouraged. The planting and preservation of xeriscaped vegetation should be an integral part of the storage facility design.

In a localized situation, an individual property owner can help to attenuate peak flows in the storm system by detaining stormwater runoff. However, uncontrolled installation of localized water quality basins within a watershed can severely alter natural flow conditions, causing compounded flow peaks or increased flow duration that can contribute to downstream degradation or capacity concerns. In addition, upstream impacts due to future land use changes should be considered when designing the structure. Land use planning and regulation may be necessary to preserve the intended function of the impoundment.

1503.12.4 Limitations

The following limitations have been adapted from the IDEQ Stormwater BMP Catalog:

- Drainage area – 10 to 50 ac.
- Minimum bedrock depth – 6 ft
- NRCS soil type – B, C, D
- Drainage/flood control – yes
- Max slope – 10%
- Minimum water table – 4 ft
- Freeze/thaw – good

Other limitations according to the California Stormwater BMP Handbook – Development are:

- Limitation of the diameter of the orifice may not allow use of extended detention in watersheds of less than 5 acres (would require an orifice with a diameter of less than 0.5 inches that would be prone to clogging).
- On-site water quality basins have only moderate pollutant removal when compared to some other structural stormwater practices, and they are relatively ineffective at removing soluble pollutants.

1503.12.5 Design Criteria

Failure of water quality basins can cause significant property damage and even loss of life. Only professional engineers registered in the State of Nevada who are qualified and experienced in impoundment design should design such structures. Local safety standards for flood control design should be followed. Below grade on-site water quality basins are preferred to above grade basins.

The following design criteria to size an on-site water quality basin are from the California Stormwater BMP Handbook – Development. Other BMP Manuals may be utilized to size the basin; however, guidance provided below should be used to determine the WQCV.

1. Facility Sizing – Calculate the WQCV based on the hydrologic design criteria for parking lot BMPs developed for Las Vegas Valley.

   Basin Configuration – A high aspect ratio may improve the performance of detention basins; consequently, the outlets should be placed to maximize the length of the flow path through the facility. The ratio of flow path length to width from the inlet to the outlet should be at least 1.5:1 (L:W). The flow path length is defined as the distance from the inlet to the outlet as measured at the surface. The width is defined as the mean width of the basin. Basin depths optimally range from 2 to 5 feet. The basin may include a sediment forebay to provide the opportunity for larger particles to settle out.

A micropool should not be incorporated in the design because of vector concerns. For online facilities, the principal and emergency spillways must be sized to provide 1.0 foot of freeboard during the 25-year event and to safely pass the flow from the 100-year storm.

2. Pond Side Slopes - Side slopes of the pond should be 3H:1V or flatter for grass stabilized slopes. Slopes steeper than 3H:1V must be stabilized with an appropriate slope stabilization practice.

3. Basin Lining – Basins must be constructed to prevent possible contamination of groundwater below the facility. Infiltration basins are not allowed in areas of shallow groundwater or poor permeability soils. If infiltration is planned as a means of disposing of stormwater, a special geotechnical study is required to assure there will be no adverse impacts on soil stability or downstream surface water quality. The local entity must approve the study in order to use this BMP.

4. Basin Inlet – Energy dissipation is required at the basin inlet to reduce re-suspension of accumulated sediment and to reduce the tendency for short-circuiting.

5. Outflow Structure - The facility's drawdown time should be regulated by a gate valve or orifice plate. In general, the outflow structure should have a trash rack or other acceptable means of preventing clogging at the entrance to the outflow pipes.

   The outflow structure should be sized to allow for complete drawdown of the water quality volume in 72 hours. No more than 50% of the water quality volume should drain from the facility within the first 24 hours. The outflow structure should be fitted with a valve so that discharge from the basin can be halted in case of an accidental spill in the watershed. This same valve also can be used to regulate the rate of discharge from the basin.

   The discharge through a control orifice is calculated from:

$$Q = CA(2gH - H_0)^{0.5}$$

   Where:

   Q = Discharge (ft^3/s)

   C = Orifice coefficient

   A = Area of the orifice (ft^3)

   g = Gravitational constant (32.2)

H = Maximum water surface elevation (ft)

H0= Orifice elevation (ft)

Recommended values for C are 0.66 for thin materials and 0.80 when the material is thicker than the orifice diameter.

6. Splitter Box - When the pond is designed as an offline facility, a splitter structure is used to isolate the water quality volume flowing from the parking lots to the basin.

7. Erosion Protection at the Outfall - For online facilities, special consideration should be given to the facility's outfall location. Flared pipe end sections that discharge at or near the stream invert are preferred. The channel immediately below the pond outfall should be modified to conform to natural dimensions, and lined with large stone riprap placed over filter cloth. Energy dissipation may be required to reduce flow velocities from the primary spillway to non-erosive velocities.

8. Safety Considerations - Safety is provided either by fencing of the facility or by managing the contours of the pond to eliminate dropoffs and other hazards. Earthen side slopes should not exceed 3:1 (H:V) and should terminate on a flat safety bench area. Landscaping can be used to impede access to the facility. The primary spillway opening must not permit access by small children. Outfall pipes above 48 inches in diameter should be fenced.

1503.12.6 Maintenance

On-site water quality basin structures should be regularly inspected for signs of failure, such as seepage or cracks in the berm.

The on-site water quality basin will require regular maintenance between rain events such as removal of debris. Any exposed soil on steep slopes should be promptly re-stabilized.

Safety, Signage and Fencing

Ponds that are readily accessible to populated areas, which includes all ponds adjacent to parking lots, should incorporate all possible safety precautions. Steep side slopes (steeper than 3H:1V) at the perimeter should be avoided and dangerous outlet facilities should be protected by enclosure. Warning signs should be used wherever appropriate. Signs should be placed so that at least one is clearly visible and legible from all adjacent streets, sidewalks or paths.

Heavy Metal Contamination

Dry ponds are less likely to build up excessive levels of heavy metals from sediments washed off impervious areas than wet ponds. However, routine maintenance should remove any significant sediment deposits.

1503.13 INFILTRATION TRENCH

1503.13.1 Description of Facility

An infiltration trench is a shallow excavation (generally 2 to 10 feet in depth) which is backfilled with sand or graded aggregates. Storm water from impervious surfaces can be directed to these facilities for infiltration and limited detention. The surface of the trench can be covered with stone, gabions, sand, or grass with a surface inlet. An alternative design is to build a vault or tank without a bottom. An Infiltration Trench is not allowed in areas of shallow groundwater or poor permeability soils. Permeable soils are a prerequisite for this BMP. A geotechnical analysis must be prepared and approved by the local entity in order to use this BMP. **Figure 1514** shows a schematic drawing of an infiltration trench.

1503.13.2 Water Quality Benefits

The infiltration trench provides adequate control for soluble and small particulate pollutants generated from small watersheds. It should not be used to trap large-sized sediments, as these will lead to premature clogging of the facility. The infiltration trench is particularly adaptable to retrofit projects for small tributary watersheds. It is easily integrated into the un-utilized portions of commercial and industrial sites.

Pollutant removal occurs through exfiltration of captured runoff into the soil layer. Removal mechanisms include sorption, precipitation, trapping, straining, and bacterial degradation or transformation. If trenches are sized to capture only low flows and initial first flush runoff volumes (the normal design condition), typical removal efficiencies can be expected in the following range.

Pollutant	Long-Term Maximum Removal Efficiency
Sediment	90%
Total Phosphorus	60%
Total Nitrogen	60%
Trace Metals	90%
Bacteria	90%

Source: CASQA New Development and Redevelopment Handbook, 2003

Use of buffer strips, swales, or detention basins for pretreatment is important to limit the amount of coarse sediment that may clog the trench.

1503.13.3 Applications

Infiltration trenches require careful planning in order to function. Although, historically they have high failure rates, performance should improve by incorporating pretreatment practices and improving on the design. An important benefit of infiltration trenches is the approximation of pre-development hydrology during which a significant portion of the average annual rainfall runoff is infiltrated rather than becoming runoff. Infiltration trenches may provide erosion protection if adequately sized for stormwater runoff.

1503.13.4 Limitations

Infiltration basins have a high failure rate if soil and subsurface conditions are not suitable. They may not be appropriate in the following instances:

- Not suitable for industrial sites or locations where spills may occur.
- Not suitable on fill sites or steep slopes
- Infiltration basins require a minimum soil infiltration rate of 0.5 inches/hour, and are not allowed in areas of shallow groundwater or poor permeability soils.
- If infiltration rates exceed 2.4 inches/hour, then the runoff should be fully treated prior to infiltration to protect groundwater quality.

1503.13.5 Design Criteria

- The maximum contributing area to an individual infiltration practice should generally be less than 5 acres.

- Infiltration trenches should not be located in areas receiving high sediment loads; on fill sites; within 100 feet of water supply wells; or under buildings or pavement. They should be a minimum of 20 feet downslope and 100 feet upslope from building foundations.

- The trench depth is generally between 2 and 10 feet. The bottom should be level. The normal configuration is with a long, narrow excavation. The water table should be at least 2 feet below the bottom of the trench.

- The volume should be based on accepting 0.5 inches of runoff from the tributary impervious areas. Void spaces are assumed to be in the range of 30 to 40 percent.

- Backfill material may be 1/2- to 3-inch aggregate. The trench may be backfilled to within 3 inches of the ground surface.

- A minimum 20-ft wide vegetated buffer strip should be provided to assist in removal of floatables, settleable solids, and oil and grease.

- A positive overflow pipe or bypass conveyance system should be provided for large storm events.

- An observation well should be located in the center of the facility, constructed of 4- to 6-inch PVC.

- The trench bottom and walls should be lined with a permeable geotextile filter fabric with a minimum 12-inch overlap. Filter fabric may also be installed one foot below the ground surface to trap large sediment and debris in the event the overlying cover material is removed.

- Typical trench width is 18 to 36 inches.

- The maximum infiltration or dewatering time is 72 hours.

- A minimum infiltration rate of 0.3 inches per hour should be obtainable to be effective. Use a safety factor of 2.0 when sizing the trench volume and dewatering time.
- The in-trench overflow drain should be formed of perforated or slotted pipe. Large pipes can be used to add to the storage in the trench. Typical perforations are 3/8-inch diameter holes with not less than 30 perforations per square foot of pipe. The pipe drain should be located a minimum of 2 feet above the trench bottom.

- For Median Strip Design: Sheet flow is accepted from both sides of the infiltration trench, and is filtered through a 20-ft wide vegetated buffer strip graded at a slope of 5 percent. An overflow pipe is required to pass excess flows.

- For Parking Lot Perimeter Design: Sheet flow is accepted from the lower end of the parking lot. Slotted curb spacers are used as a level spreader at the edge of the parking lot to evenly distribute flows to the 20-ft wide vegetated buffer strip. See Section 1502 for parking lot BMP design guidance.

- For Swale Design: The swale collection system longitudinal slope should not exceed 5 percent. The trench should be located in the invert of the swale. Check dams may be required across the swale to increase the retention volume and prevent "short-circuiting" of the infiltration trench. See Section 1503.4 for information on swales.

1503.13.6 Maintenance

It is critical that settleable particles and floatable organic materials be removed from runoff water before it enters the infiltration trench. The trench will clog and become nonfunctional if excessive particulate matter is allowed to enter the trench.

Maintenance requirements for infiltration trenches consist primarily of annual surface and water level inspections, buffer strip maintenance, and periodic surface sediment and debris removal. However, their small size and inconspicuous design can tend to leave them forgotten. Coarse sediment must be kept out of the trench to prevent premature clogging. If clogging does occur, a substantial portion of the backfill aggregate may have to be removed and replaced.

1504 ADDITIONAL MAINTENANCE MEASURES

1504.1 Description

Sites will require good housekeeping measures to help reduce the discharge of targeted constituents into the storm drain system. One important good housekeeping practice is regular cleaning. A parking lot sweeping frequency schedule should be established based on observation of waste accumulation. Before sweeping begins, any storm drains that may be affected should be protected and excess runoff water should be contained. Washwater shall be disposed of in the sanitary sewer or to a pervious surface. Debris accumulated from sweeping shall be disposed of properly in litter receptacles or at a landfill. Debris should be removed regularly from the parking lot. Litter receptacles should be cleaned out at regular intervals and should be covered to prevent spillage. Oily deposits should be cleaned with absorbent materials to prevent oil from entering the storm drain system. Rooftop drains should be cleaned out and arranged to prevent drainage directly on paved surfaces. Maintenance equipment should be inspected before cleanings. Maintenance logs should be kept to document materials removed and any improvements made.

ACCEPTED PARKING LOT BMPs IN THE LAS VEGAS VALLEY

	Site Design Measures	Treatment Control Measures	Source Control Measures	Site Maintenance Measures	Code Measures
Usage	• Site design measures accept drainage from impervious parking area to trap pollutants prior to stormwater leaving the site	• Infrastructure accepts site runoff and removes pollutants prior to stormwater leaving site	• Onsite control to reduce pollutants from reaching site design and treatment control measures	• Maintenance and pollution prevention measure to keep pollutants from the site design and treatment control measures.	• Pollution prevention associated with typical parking area appurtenances and activities, to keep potential pollutants from contacting stormwater
Measures	• Minimize Directly Connected Impervious Areas • Landscape Drainage Swale • Depressed Median • Depressed Landscaping • Buffer Strip • Pervious Overflow Parking (modular pavers)	• Sand / Media Filers • Oil & Water Separator • Onsite Water Quality Basin (no infiltration)	• Proposed use & disposal of sorbents • Proposed waste handling & disposal	• Building and Ground Maintenance • Parking Lot Sweeping • Sidewalk cleaning	• Trash Storage Areas • Vehicle & Equipment Washing Areas • Loading Dock Areas • Minimize Parking Requirements
Benefits	• Lower cost than treatment control • Easier construction than treatment control • Lower maintenance than treatment control • Easy to incorporate in most site plans	• Compact design allows for installation in small areas • Relatively efficient removal of many pollutants of concern	• Control measure to prevent pollutants from leaving site	• Aesthetics • Pretreatment control for Site Design and Treatment Control	• Reduce pollutants from high potential sources • Low implementation costs • Close gaps in exiting regulations

Revision	Date

REFERENCE: MWH

PARKING LOT BMP DESIGN REQUIREMENTS BY PARKING LOT CATEGORY

Parking Lot Category	BMP Requirement	Total Site Size	Parking Lot Size	Example BMP
Small	None	< 1 acre	Not Applicable	None specifically required. Developer has the option to install any BMPs listed in the Medium or Large Categories.
Medium (minor potential impacts on MS4)	Disconnect at least 75% of parking lot and contributing building area. Design based on accepted minimum standards.	> / = 1 acre	≤ 1 acre	Depressed medians Depressed planter areas Modular pavers Buffer strips Disconnect roof drains Developer has the option to install any BMPs listed in the Large Category.
Large (major potential impacts on MS4)	Provide treatment BMPs for at least 75% of parking lot and contributing building area. Design for 85th percentile storm. Provide supporting hydrologic and hydraulics calculations for BMP design.	≥ 1 acre	> 1 acre	Depressed medians Depressed planter areas Modular pavers Buffer strips Disconnect roof drains Xeriscaped swales Rock swales Retention ponds (if no adverse impacts) Settling basins Sand filters Sand/oil separators

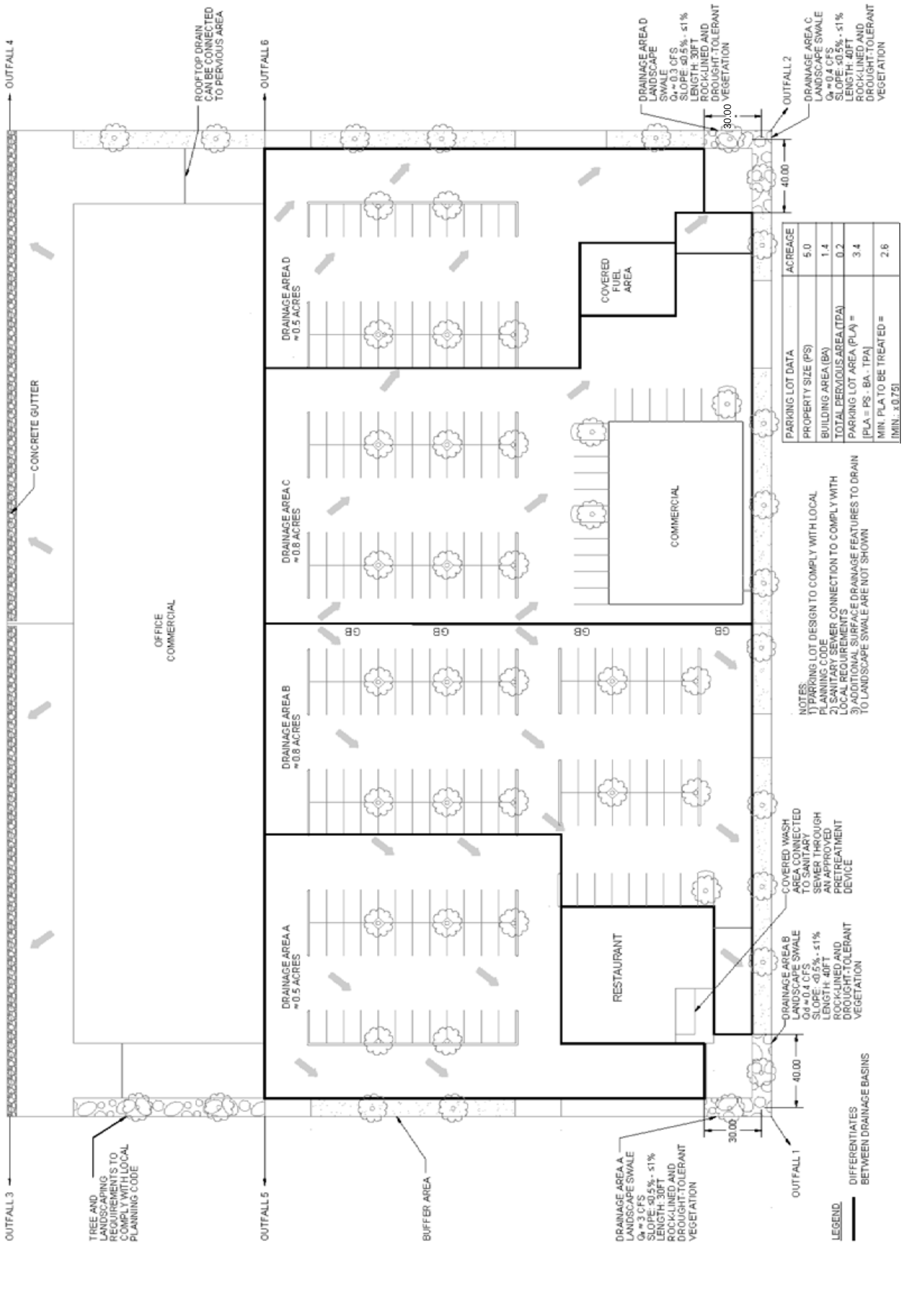
Revision	Date

FEASIBLE LID MEASURES IN THE LAS VEGAS VALLEY NDSR PROGRAM

LID Category	Specific Measure or Practice	Limitations
Minimize directly connected impervious area	Minimize overall impervious area	-
	Direct runoff onto properly designed unpaved surfaces	-
	Disconnect rooftop drains	-
Parking Lot Design	Depressed medians	Only outside Selenium Management Area; Xeriscaping only
	Buffer strips	Xeriscaping only
	Modular pavers	-
	Minimize parking requirements	-

Revision	Date

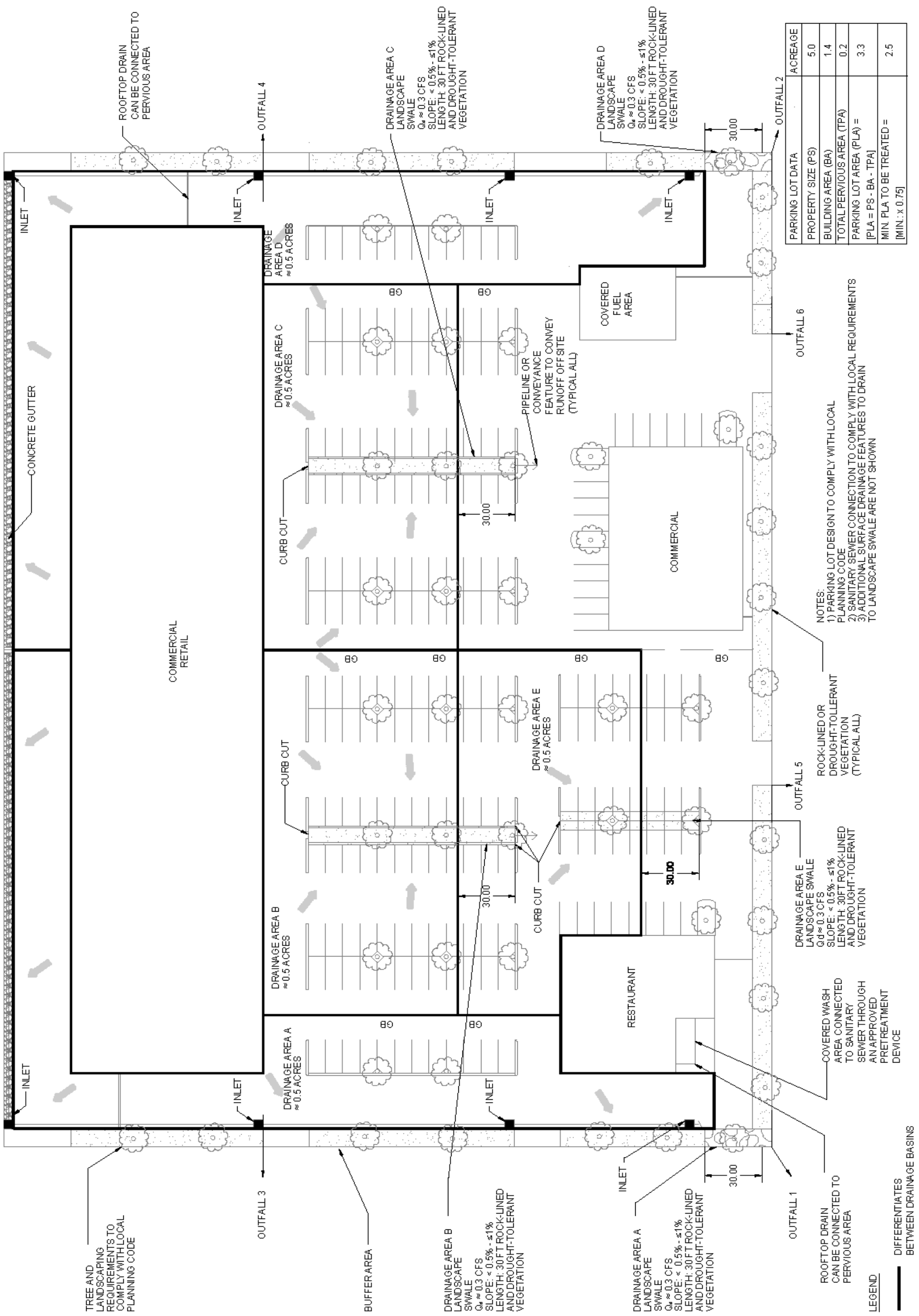
SCHEMATIC DRAWING OF CONCEPTUAL LAYOUT 1 - PERIMETER SWALE ONLY



Revision	Date

	REFERENCE: **MWH**	**Figure 1501**

SCHEMATIC DRAWING OF CONCEPTUAL LAYOUT 2 - PERIMETER AND INTERNAL SWALES

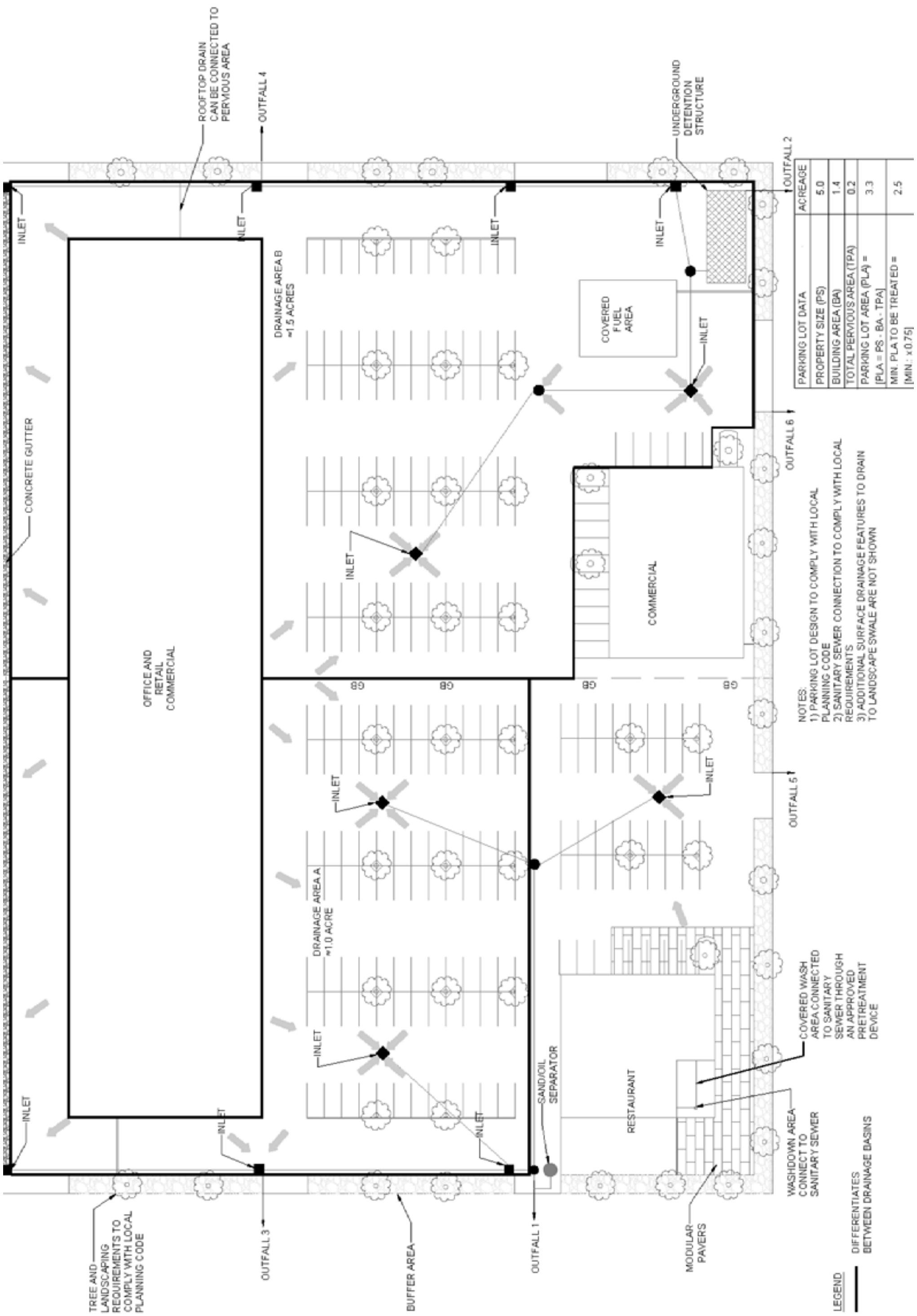


Revision	Date

REFERENCE: MWH

Figure 1502

SCHEMATIC DRAWING OF CONCEPTUAL LAYOUT 3 - NO SWALES

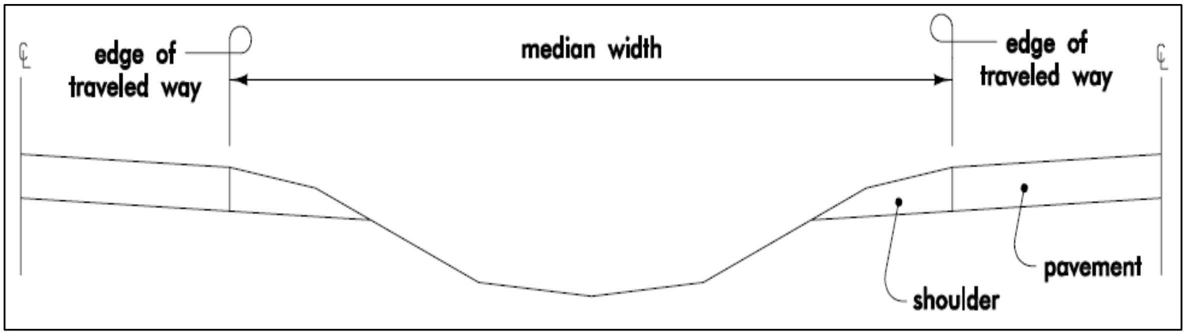


Revision	Date

REFERENCE: MWH

Figure 1503

SCHEMATIC DRAWING OF A DEPRESSED MEDIAN

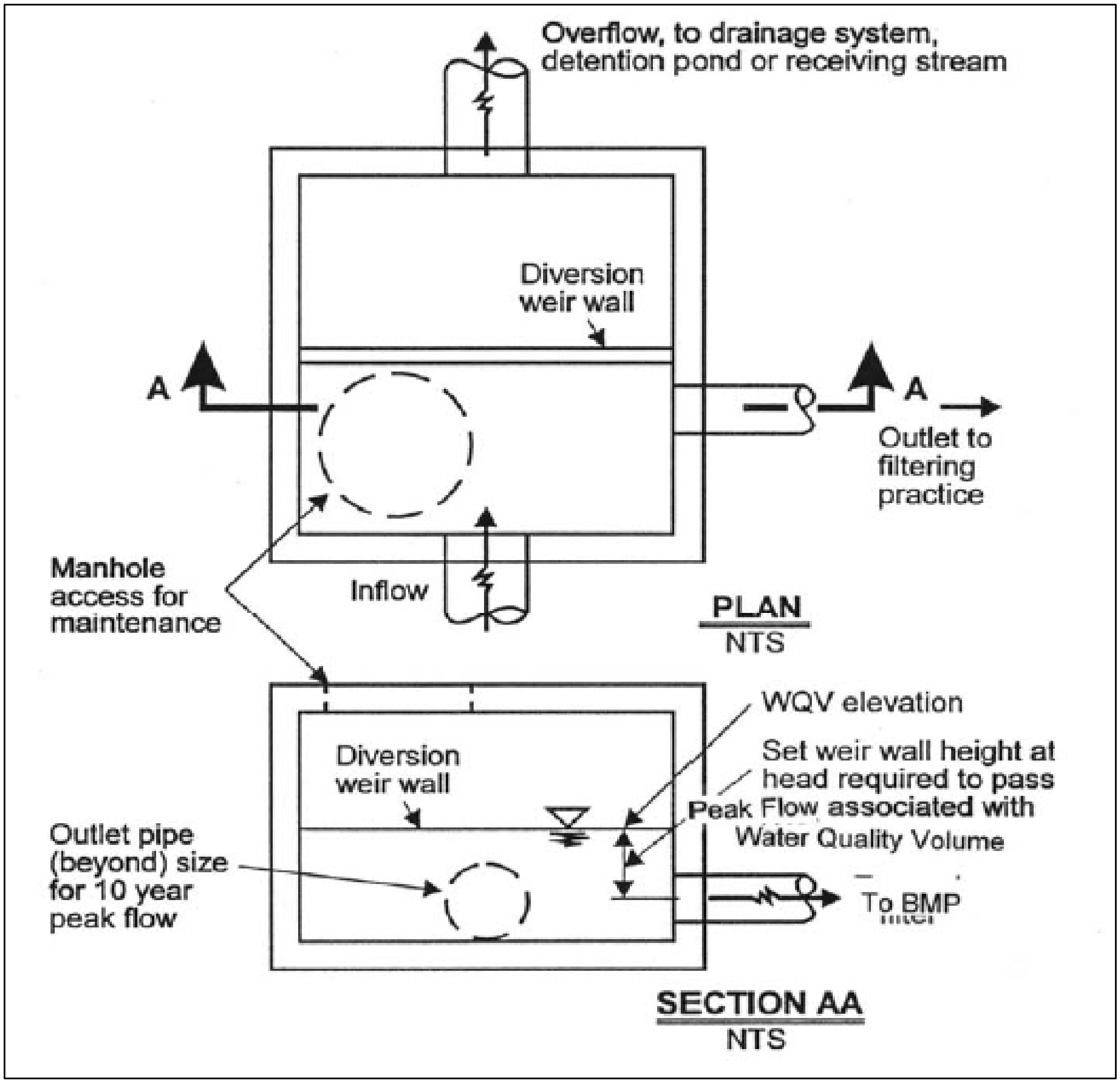


Revision	Date

REFERENCE: Iowa Department of Transportation, Office of Design

Figure 1504

SCHEMATIC DRAWING OF A BMP DIVERSION STRUCTURE

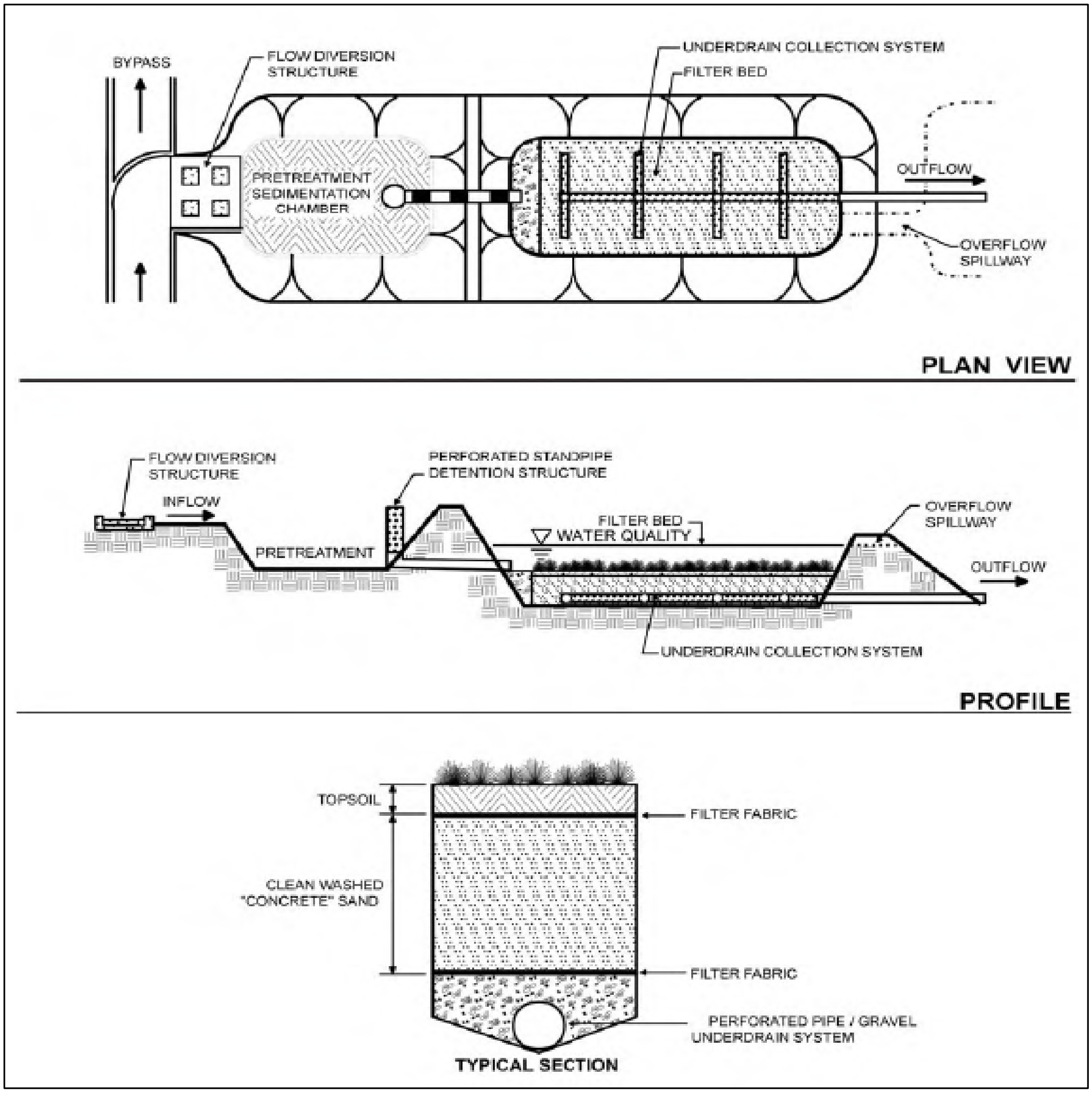


Revision	Date

REFERENCE: Chesapeake Research Consortium

Figure 1505

SCHEMATIC DRAWING OF A SURFACE SAND FILTER

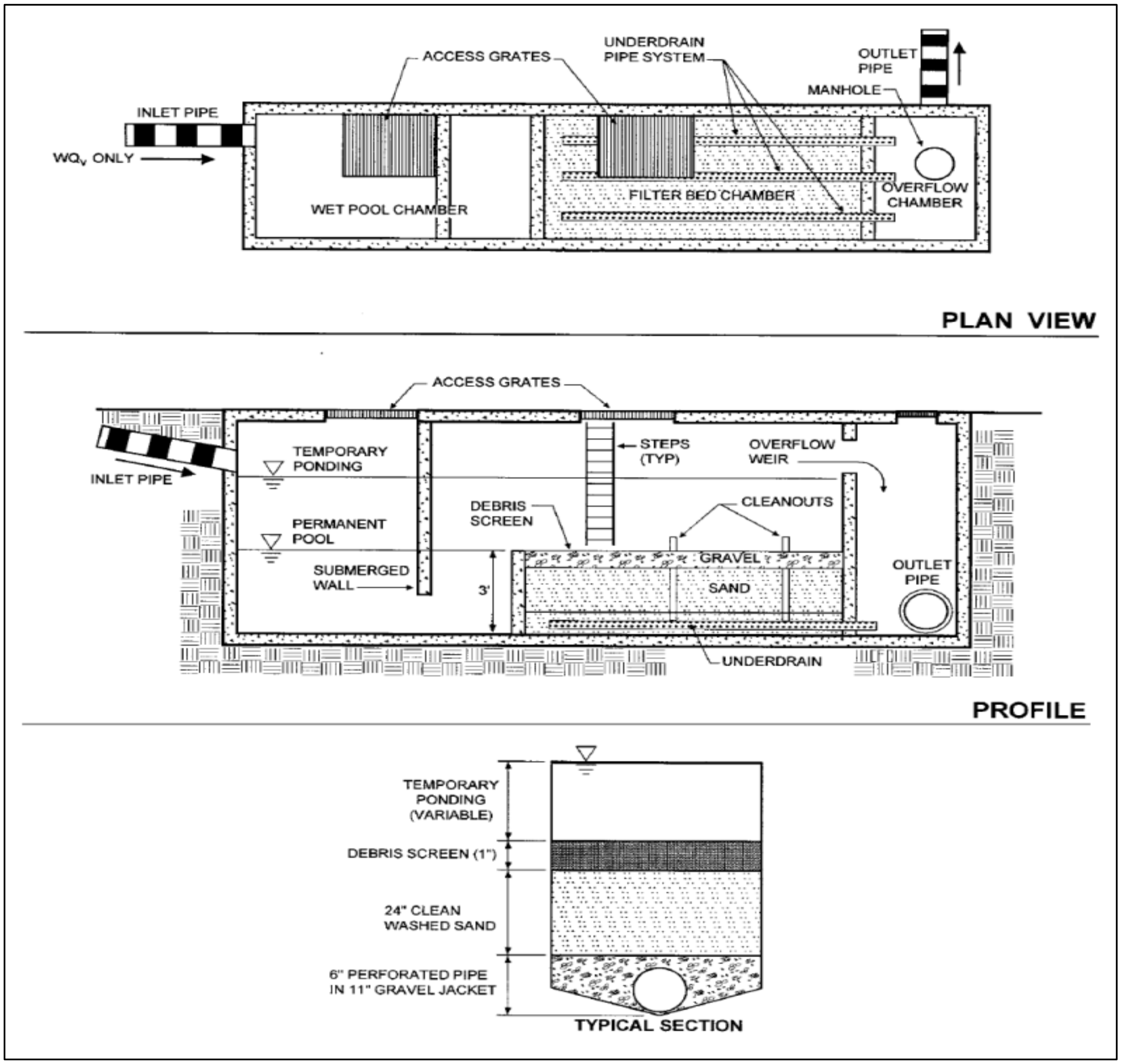


Revision	Date

REFERENCE: Chesapeake Research Consortium

Figure 1506

SCHEMATIC DRAWING OF AN UNDERGROUND SAND FILTER

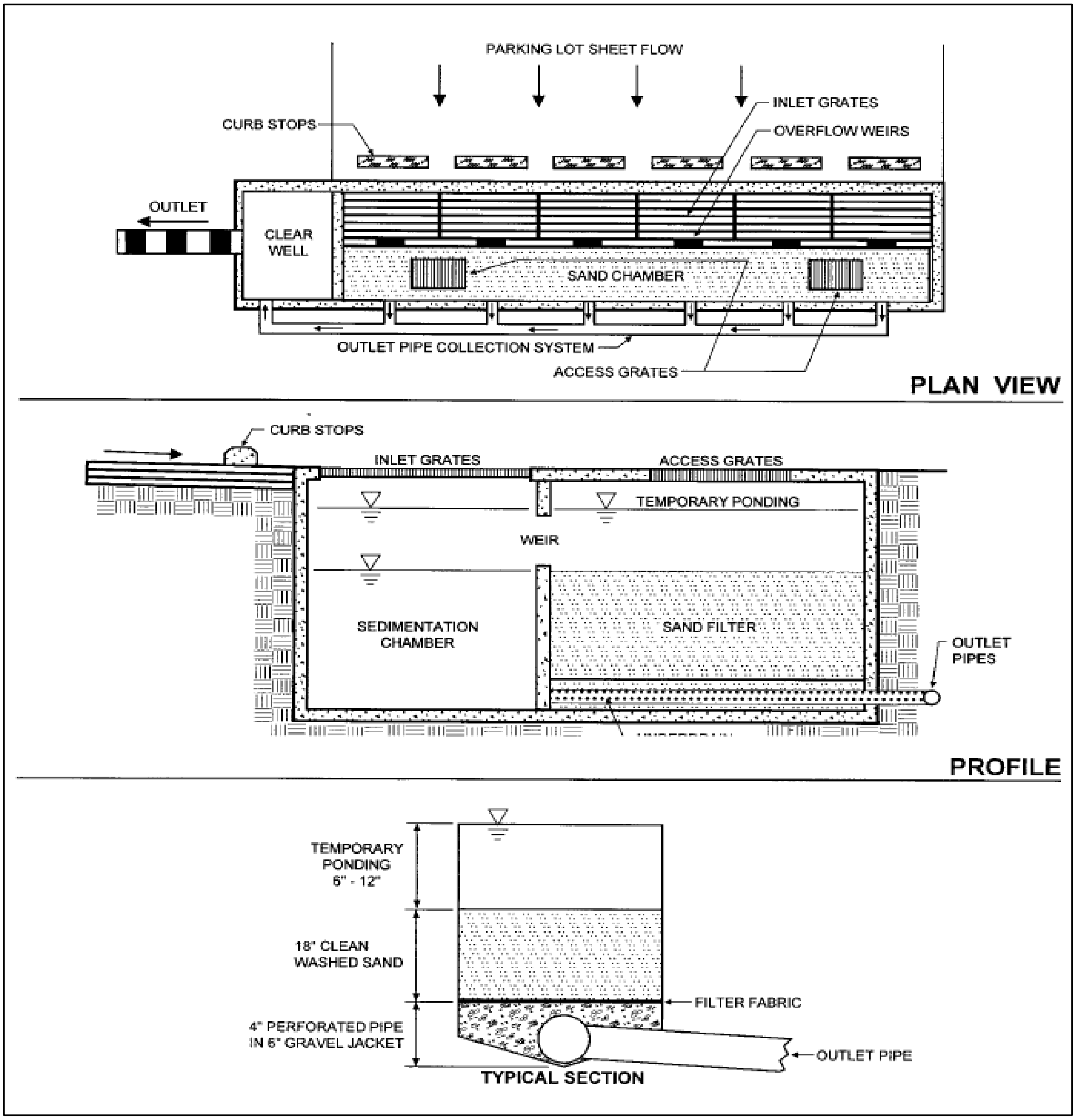


Revision	Date

REFERENCE: Chesapeake Research Consortium

Figure 1507

SCHEMATIC DRAWING OF A PERIMETER SAND FILTER

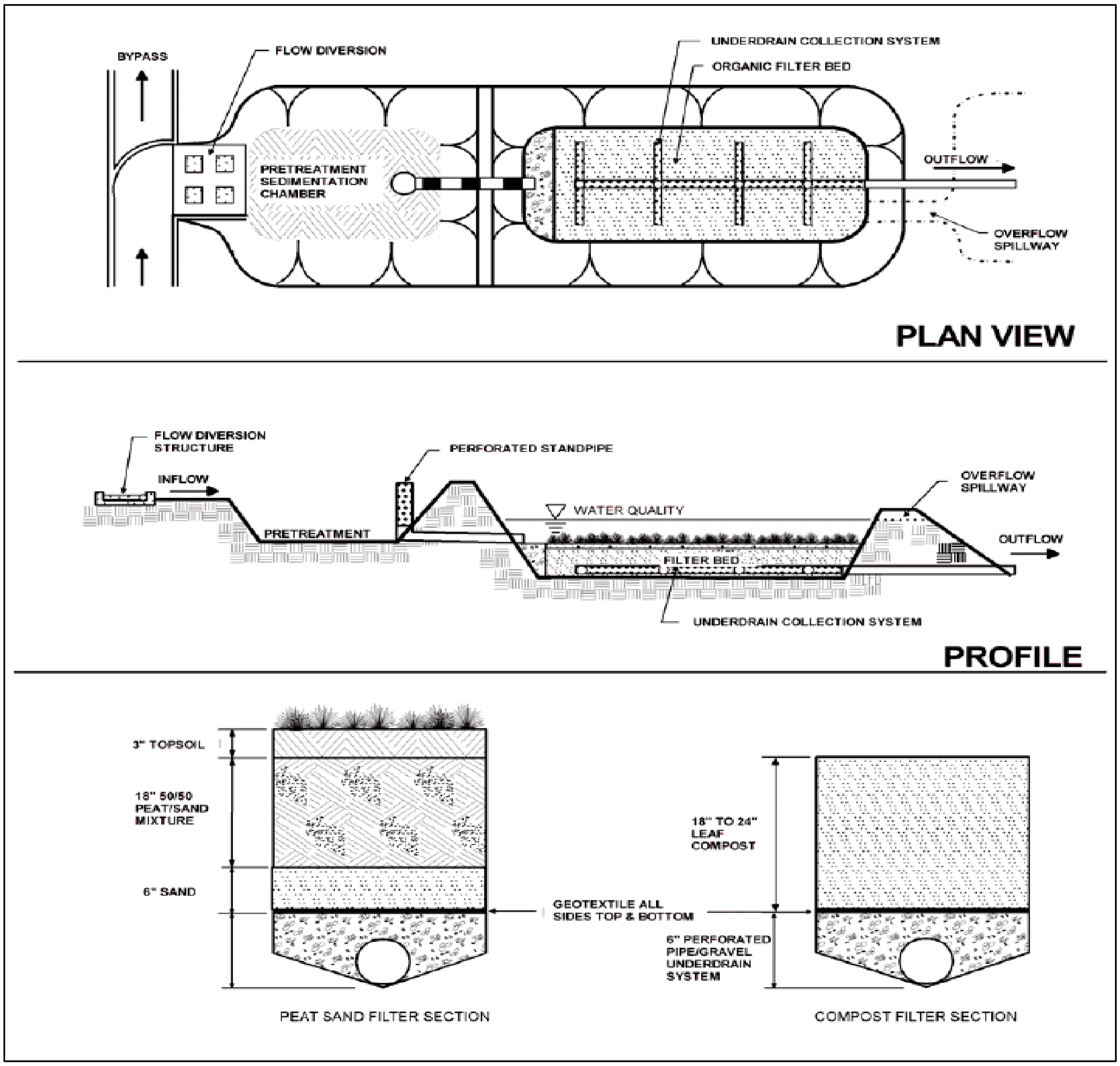


Revision	Date

REFERENCE: Chesapeake Research Consortium

Figure 1508

SCHEMATIC DRAWING OF AN ORGANIC FILTER

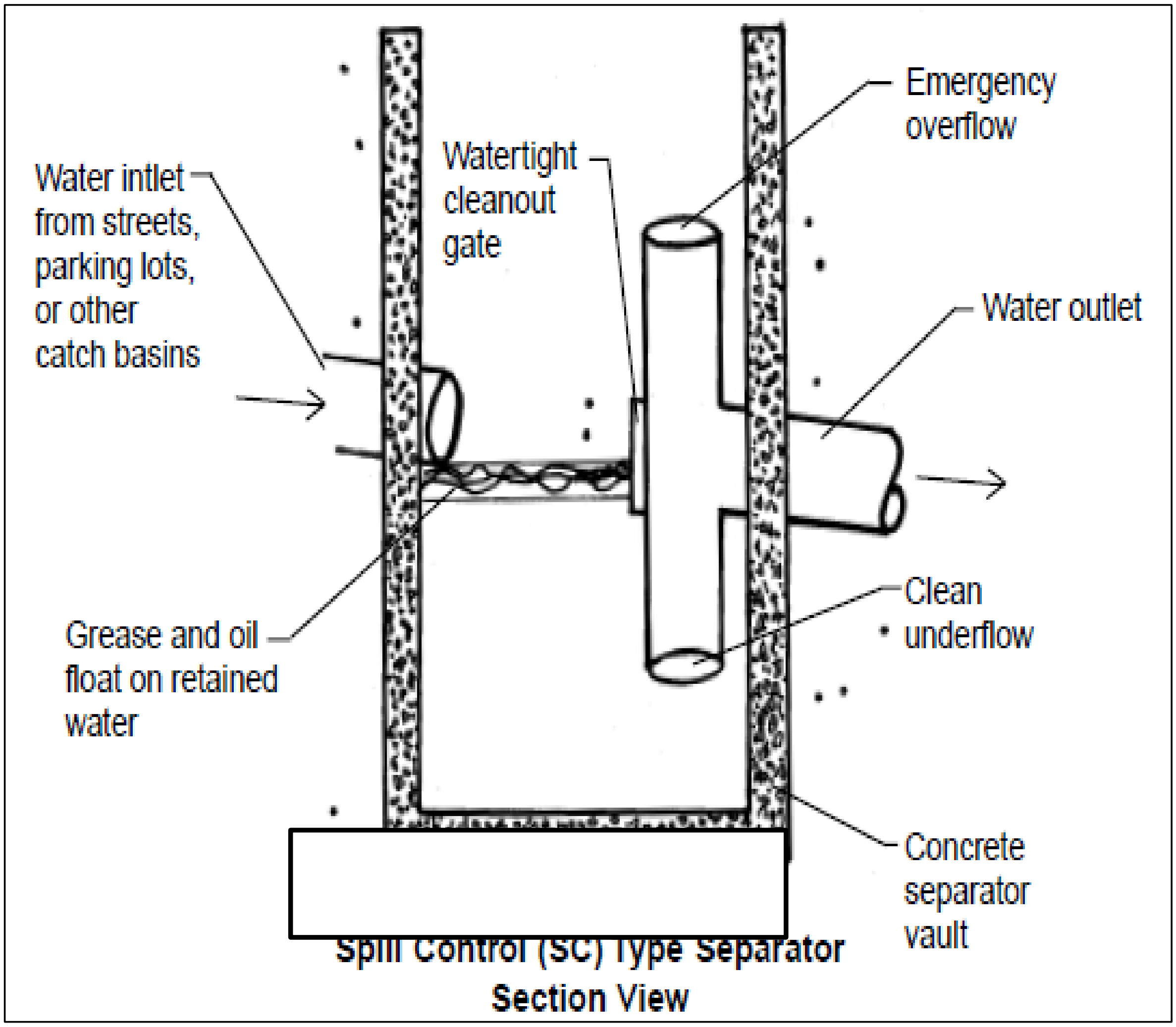


Revision	*Date*

REFERENCE: **Chesapeake Research Consortium**

Figure 1509

SCHEMATIC DRAWING OF A
SPILL CONTROL TYPE OIL / WATER SEPARATOR

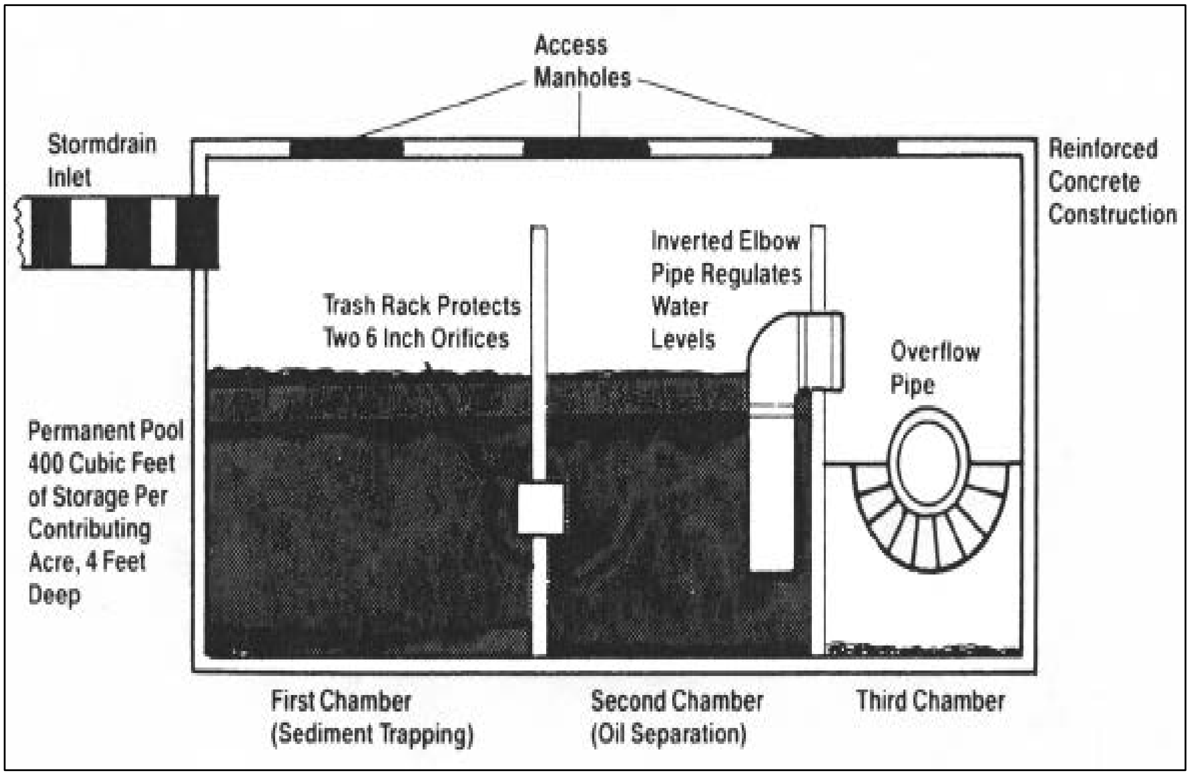


Revision	Date

REFERENCE: IDEQ Storm Water BMP Catalog

Figure 1510

SCHEMATIC DRAWING OF AN OIL AND GRIT SEPARATOR

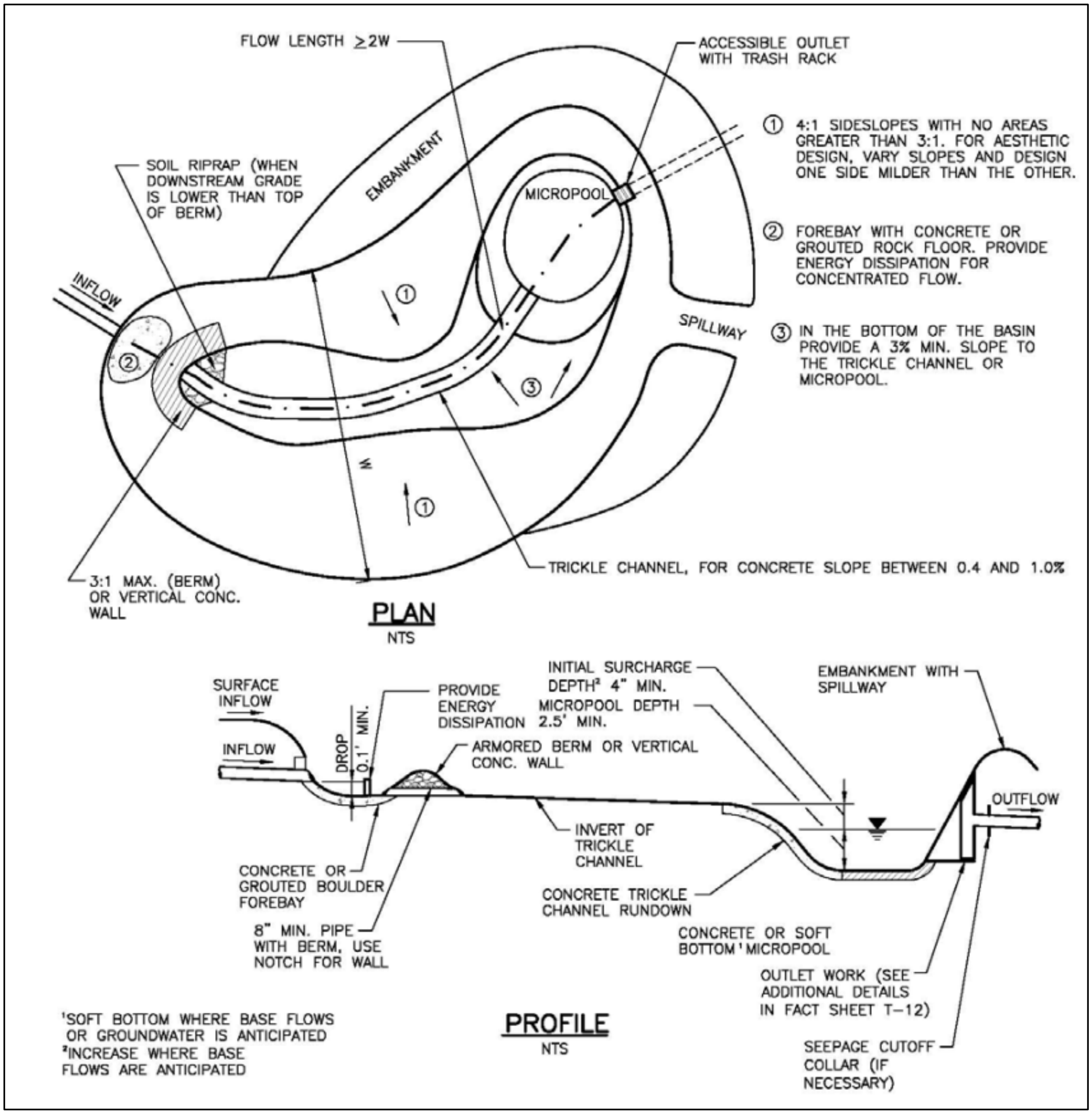


Revision	*Date*

REFERENCE: **Minnesota Urban Small Sites BMP**

Figure 1511

SCHEMATIC DRAWING OF ON-SITE WATER QUALITY BASIN PLAN AND PROFILE

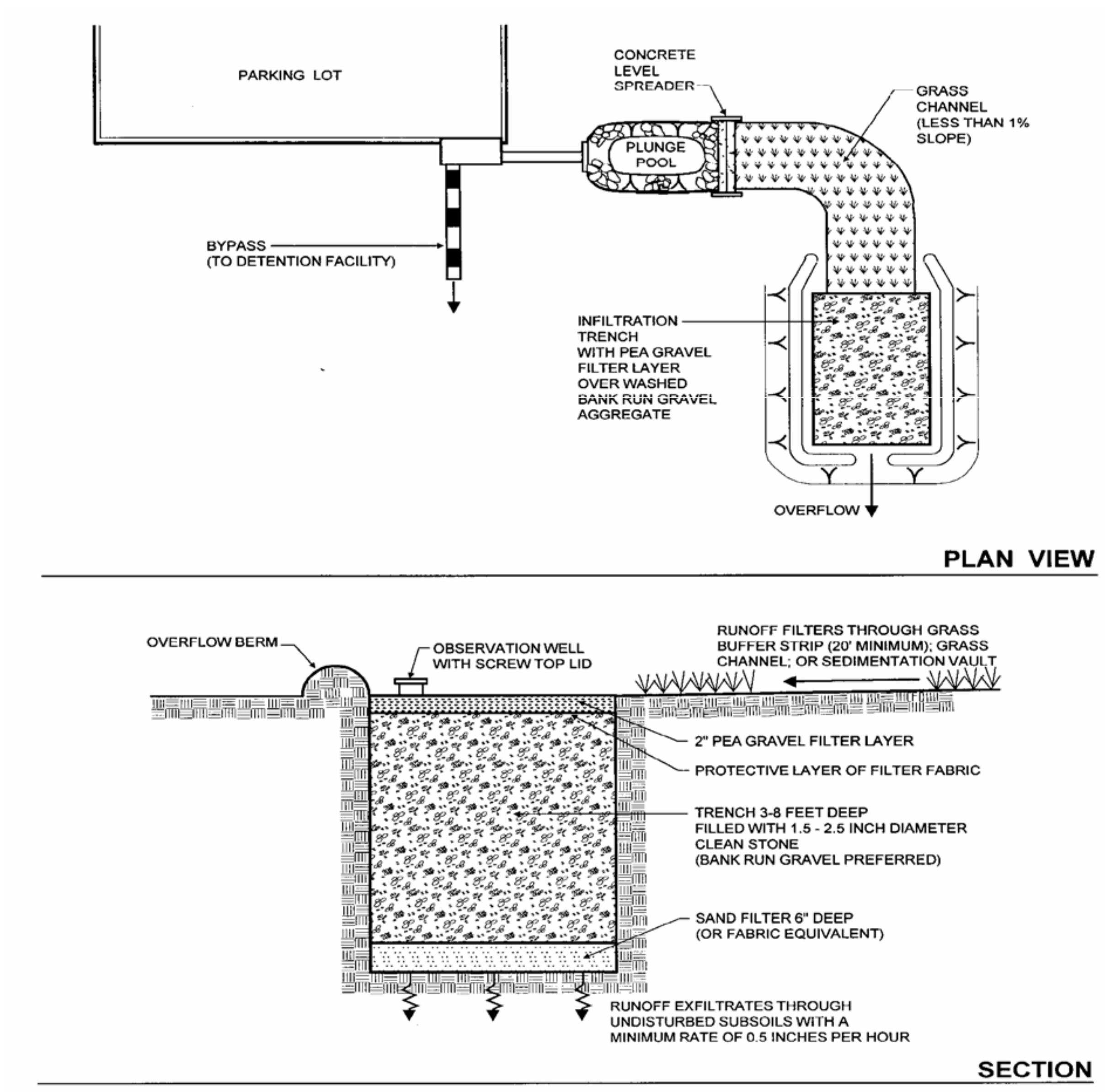


Revision	Date

REFERENCE: Courtesy of Urban Storm Drainage Criteria Manual Vol. 3

Figure 1512

SCHEMATIC DRAWING OF INFILTRATION TRENCH PLAN AND PROFILE



Revision	*Date*

REFERENCE: **CASQA Stormwater Best Management Practice Handbook New Development and Redevelopment, 2003**

Figure 1513

CLARK COUNTY REGIONAL FLOOD CONTROL DISTRICT
HYDROLOGIC CRITERIA AND DRAINAGE DESIGN MANUAL

SECTION 1600
LOCAL ENTITY CRITERIA

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1601	City of Henderson	1601
1602	City of Las Vegas	1605
1603	City of North Las Vegas	1608
1604	Clark County Public Works - County Policies	1610

LIST OF FIGURES

1601 City of Henderson Finished Floor Elevation Certificate

Section 1600
Local Entity Criteria

1601 CITY OF HENDERSON

Section 201

The City of Henderson shall require **Standard Form 1** be included with every submittal to the City.

The City of Henderson shall require the latest copy of all grading plans and any necessary improvement drawings to evaluate the control of drainage for the project are included with every submittal to the City.

The City of Henderson shall require an exhibit showing which lots are being protected by any proposed facilities. The City will not issue any building permits for any lots impacted by this exhibit prior to the associated facilities being completed.

Section 203.3

Parcel Map Drainage Study requirements:

a) Parcel Maps dividing land into parcels greater than 2 acres shall complete a Conceptual Study for the purpose of defining off-site flow impacts and to determine if any drainage easements are required.

b) Parcel Maps dividing any land into parcels less than or equal to 2 acres shall complete a Technical Drainage Study as defined in Section 204.

Section 303.1.3, Paragraph 1

The City of Henderson will allow nuisance water to travel a maximum length of 1,000 feet or across the front of 20 lots before it is required to be conveyed within a storm drainage system.

Section 303.10

The City of Henderson requires a minimum 20-foot wide easement for all publicly maintained facilities per the current City of Henderson Development Code.

The City of Henderson will require a surface overflow path with the capacity for the major storm in addition to any proposed underground facility. The overflow path will not be required to meet the same criteria as if it were the primary flow path; however, the adjacent structures need to be elevated above the calculated water surface elevation for the major storm without freeboard.

Section 303.4, Paragraph 3, Number 2

... The City of Henderson will require all new development to construct any facilities required to protect their site and mitigate any increase in runoff.

Section 303.6.1

The City of Henderson requires a finished floor elevation certificate be submitted to Public Works for approval prior to receiving any shear and ties inspections for any structure. A copy of the form is shown in **Figure 1601**.

For lots located in a FEMA designated flood zone, a FEMA elevation certificate is also required prior to receiving any shear and ties inspections.

The City of Henderson requires that a Letter of Map Revision (LOMR) be submitted to FEMA prior to approval of the elevation certificate for lots located within the 100-year floodplain.

Section 304.4

The City of Henderson will only allow bubbler laterals to convey storm water under streets as an interim solution to outlet the downstream end of a storm drain, which will be extended in the future. A minimum 6-inch temporary drain line will be required to drain the bubbler. No permanent bubblers will be allowed.

Section 303.4

B) Residential Streets: The City of Henderson requires that the depth of flow from a major storm in any street with residential lots fronting be contained within the street right-of-way.

Section 304.4, Paragraph 6, Letter D, Number 1

The City of Henderson requires that the finished floor elevation be set at a vertical distance above the adjacent curb of at least 18-inches. This shall be measured at the center of the structure.

Section 304.4, Paragraph 6, Letter D, Number 2

The City of Henderson requires that the finished floor elevation is set at a vertical distance above the adjacent curb of at least 18-inches or twice the depth of flow, which ever is higher. This shall be measured at the center of the structure.

Section 304.4, Paragraph 6, Letter E

The City of Henderson requires that the finished floor elevation be set at a vertical distance above the adjacent curb of at least 18-inches. This shall be measured at the center of the structure.

Section 705.7.1.13

For minor drainage channels the City of Henderson will accept one of the following as an alternate to the standard concrete section:

a) A 4-inch thick, 3,000 psi concrete with 3.0 pounds of 2-inch 100 percent virgin homopolymer polypropylene fibrillated fibers per cubic yard added at the batch.

b) A 2-inch AC Pavement section consistent with Standard Drawing No.209 of the Uniform Standard Drawings for Public Works Construction Off-Site Improvements, Clark County Area, Nevada.

Section 803.1.2

The City of Henderson will accept only reinforced concrete pipe/box for a publicly maintained storm drain.

Section 804.1

The City of Henderson will accept computer software in lieu of **Standard Form 6** for the hydraulic analysis of the proposed storm drain system; if and only if, the software calculates the losses per chapter 804.2. The engineer shall provide an output table, which includes all of the parameters included on **Standard Form 6**.

Section 905

The minimum street slope shall conform to the current City of Henderson Development Code.

Section 1002.1.3

Culvert crossings as part of an improved channel system or a future proposed channel system-conveying 500 cfs or greater shall be sized using the bridge criteria per Section 1005.

Where vertical constraints **DO NOT** exist, the minimum box culvert shall have a rise greater than or equal to 6-feet.

1602 CITY OF LAS VEGAS

Section 201, Paragraph 4

The City of Las Vegas requires only one copy of the drainage study and two copies of the Grading Plan.

Section 204, Technical Drainage Study

A grading plan is required with the Technical Drainage Study.

Section 204.2, Drainage Plan

A reference point on the drainage map must be shown for street capacity and drainage easement locations.

Section 207, Paragraph 3

Mitigation of nuisance water, both during construction and for the fully developed condition must be addressed in the drainage study.

Section 300, Drainage Policy

Concrete valley gutters are required in parking lots with slopes < 1 percent. Slopes through cul-de-sacs must be at a 1 percent minimum where flow is drained through the cul-de-sac.

Section 303.10, Paragraph 2

Ten-foot wide public drainage easements to be privately maintained are allowed for flow < 20 cfs. The depth of flow entering the easement must be checked by using the submerged weir calculation.

Section 304.4.D.1

The limits of the flood zones and the Base Flood Elevations (BFE) must be shown on all Grading Plans for all developments within a Special Flood Hazard Zone A, AO, AE, etc.

Section 304.4.E.1

Minimum finished floor elevation is 6 inches above highest adjacent top of curb.

Finished floor elevation calculations must include allowances for super elevations on curves and velocity head (head loss) for tee intersections.

Finished floor elevations for buildings adjacent to public drainage easements must be a minimum of 18 inches above the Q-100 weir or submerged weir elevation, whichever is greater.

Lots with "B and C Type Drainage" (drainage into the back yard) shall be required to install an underground nuisance drainage system or a 2-foot valley gutter through the lot. A 16-inch x 24-inch block wall opening between lots at the property boundaries with No. 4 - rebar 6 inches on center is required for both options. Improvements shall include a "private" drainage easement to be shown on the grading plan and granted by the final map, parcel map or separate document.

Block wall openings must be sized using a 50 percent clogging factor (i.e., assuming the lower half of the opening is clogged). The minimum block wall opening allowed is 16-inch x 48-inch for flows up to 10 cfs. For flows greater than 10 cfs a wrought iron fence is required at one end of the easement. Concrete bollards are required at the opposite end. However, a 10 foot gate is also acceptable in lieu of the bollards A 2-foot minimum scour pad must be provided at the opening entrance and exit. A 3-foot cutoff wall must be provided at the entrance to prevent erosion.

Finished floor elevations for each lot must be shown on the grading plan with top-of-curb elevations at the upstream end of the lot.

Section 706.1.3

Minimum freeboard of 1.5 feet from adjacent finished floor elevations of buildings/homes.

Section 706.2.4

Minimum freeboard of 1.5 feet from adjacent finished floor elevations of buildings/homes.

Section 803.5

Bubblers are required across 80 foot and 100 foot wide streets in-lieu ofvalley gutters. When flows exceed 10 cfs, bubbles larger than 18 inches will be required up to a maximum of 36 inches in diameter. Inlets sized to match the pipe size must be provided. Bubblers are considered interim solutions and are intended to have connected to neighborhood or regional storm drain facilities. Bubblers must be drained with 6-inch drains except when lengths become excessive.

Section 805, Paragraph 3

Hydraulic calculations must provide for a 50 percent clogging factor in the capacity calculations for all drop inlets.

Section 906, Paragraph 5

For street slopes < 0.4 percent an 18-inch storm drain must be installed. No cross gutters are permitted across streets $\geq$ 80 feet wide. A bubbler system consisting of a 36-inch pipe with inlets, inlets sized to match and a minimum 6-inch bleeder line must be provided. Where flows are less than 20 cfs, a 10 foot concrete lined drainage easement shall be allowed.

Standard Form 2, Section 1 - General Requirement

Grading Plan required in addition to a Drainage Plan as required in the MANUAL.

1603 CITY OF NORTH LAS VEGAS

Section 204

A completed *Drainage Submittal Checklist* (**Standard Form 2**) must be included with the initial technical drainage study submittal.

Section 303.6.1

The City of North Las Vegas does not permit the construction of permanent structures within a Federal Emergency Management Agency (FEMA) designated Special Flood Hazard Area (SFHA). Under this policy, any developer/builder proposing to place structures within a SFHA must meet the following requirements prior to the issuance of various permits and certificates-of-occupancy:

a. **Grading and off-site construction permits** may be issued by the City of North Las Vegas, Department of Public Works, once the improvement plans and drainage study have been approved and a copy of the completed Conditional Letter of Map Revision (CLOMR) application has been submitted to FEMA for processing.

b. **Building permits** can be issued once a CLOMR has been obtained from the FEMA.

c. **Certificates-of-Occupancy** can be issued once a Letter of Map Revision (LOMR) has been obtained from FEMA.

Section 304.4, Paragraph 4

Where downstream storm sewer facilities are not available, the City of North Las Vegas requires bubbler laterals for the conveyance of storm water under streets with right-of-way widths greater than or equal to 80 feet. The bubbler laterals must consist of a minimum 18-inch diameter reinforced concrete pipe. To accommodate the draining of the bubbler system prior to future downstream extension of the storm sewer system, a minimum 6-inch diameter PVC pipe must be daylighted downstream.

Section 304.4, Major Storm Street Capacity Limitations, Item A

Within the interior streets of a residential subdivision, the depth times velocity for the major storm event shall be less than or equal to 6.

Section 803.3

The City of North Las Vegas requires that stormwater drop inlet signage is obtained from the City's Resources/Environmental Division to be affixed to any installed drop inlets. Quality control inspectors will verify that the signs are properly installed.

General

Side Lot Drainage Easements

Side lot drainage easements are generally discouraged unless the engineer can demonstrate design constraints that render alternative site layout and drainage facility design options as impossible or impractical.

Block Wall Openings

All block wall openings must be designed to pass the 100-year storm event flow rates using the assumption that the bottom 50 percent of the openings are obstructed. Additionally, non-damaging emergency surface flow paths mustbe available to convey the 100-year flows.

Lot Drainage Beneath Air-Conditioning Pads

For any ground-mounted air-conditioning pad that encroaches to within three (3) feet of a property line, the engineer must indicate on the plot plans how lot drainage will be accommodated beneath the pad. This can include, but is not limited to, the placement of a 4-inchdiameter PVC pipe, with the inlet and outlet inverts of the pipe constructed to correspond with the flow line of the obstructed lot drainage swale.

1604 CLARK COUNTY PUBLIC WORKS - COUNTY POLICIES

1. Drainage Easements

   a. Public Drainage Easements - Public drainage easements are required for situations where a publicly maintained facility must drain through a private parcel. The easements must comply with the Clark County Public Works public drainage easement policy which follows:

      - Subdivisions are to be designed to minimize the need for drainage easements;

      - The drainage easement must be a minimum of 15 feet wide;

      - The drainage easements must be fully concrete lined, with a low flow area constructed to a minimum grade of 1 percent in 50 feet or less or 0.5 percent for lengths greater than 50 feet;

      - Block walls or combination of block wall and wrought iron to meet zoning's wall height requirements. Walls are to be located outside of the drainage easement;

      - At a minimum, removable locking bollards must be placed at each end of the easement. In easements 50 feet long or less, a single galvanized gate may be installed at approximately the midpoint. In easements greater that 50 feet, two galvanized gates may be installed but they must be recessed at least 10 feet or at the front yard set backs as determined by Zoning, whichever is greater, from the public rights-of-way. Gates are to be hinged to allow 180-degree movement;

      - Joint or multi-use easements are not acceptable, unless the above conditions are met;

      - Where existing storm drainage facilities exist, to provide an outlet, underground storm drains will be used through an underground drainage easement with overflow section. The minimum width for a public underground drainage easement is 10 feet;

   b. Private drainage easements are to be used to convey flows from one private parcel through an adjacent private parcel. The private drainage easements must comply with the following criteria:

- The minimum width of a private drainage easement is 5 feet;

- The private drainage easement must be lined with a 3-foot wide minimum concrete valley gutter;

- Any proposed or future walls crossing the private drainage easement must have wall openings designed to pass the flow and a detail of the opening(s) must be provided on the grading plan;

2. Calculations for block wall openings must be completed with a 50 percent clogging factor applied.

3. Calculations for storm drain inlets must be completed with a 50 percent clogging factor applied.

4. Floodwalls:

   a. For flow depths 1-foot or less, solid grouting is required.

   b. For flow depths 1.5-feet or less, a County Standard Flood Wall may be used provided the criteria set forth for the flood wall is met.

   c. For flow depths greater that 1.5-feet, a structurally designed flood wall is required.

5. Half street valley gutters must be constructed to the future spandrel on the opposite side of the street.

6. A completed **Standard Form 1** and the County minimum drainage criteria checklist signed and sealed by the engineer is required for each new submittal. Updates and amendments to approved drainage studies must have a completed **Standard Form 1** signed and sealed by the engineer. All submittals including addendums require a completed off-site submittal sheet.

Culverts and Bridges

1002.1.3 Minimum Size

For rectangular shaped culverts, the minimum size shall be 6 feet in height and 8 feet in width.

MUST BE TYPED MUST BE TYPED

FINISHED FLOOR ELEVATION CERTIFICATE

PERMIT # ______________________ DATE PERMIT ISSUED ____________

ADDRESS ______________________ ASSESSORS PARCEL # ____________

SUBDIVISION ______________________ LOT ________ BLOCK ________

OWNER OR BUILDER CONTACT ______________________ PHONE # ________

This form must be completed by a Nevada-registered Land Surveyor or Civil Engineer After the slab is poured and before any additional inspections.

I hereby acknowledge that I have reviewed the approved grading plans stamped and signed by ______________________ and dated ______________ and I certify the following:
(use stamp date)

According to the approved grading plans noted above, the elevation of the lowest habitable finished floor should be: ____________* COH Bench Mark________ Datum (29 or 88) ________

I also certify that the actual elevation of the lowest habitable finished floor (to the nearest tenth of a foot) is: ______________* COH Bench Mark________ Datum (29 or 88) ________

***Full elevation required; for example, 2045.8 not 45.8.**
(Note: The City of Henderson (COH) Bench Mark and Datum shown on the approved plans should be used for the actual elevation. If not available, explain below in comments)

Comment: ____
____

Affix Seal
In the Space to the Right
Then Sign and Date

Firm Name: ______________________ Phone # ____________ Fax # ____________

**Return this completed certificate to Public Works/Land Development
City Hall, 240 Water Street, Suite 210, Henderson, NV 89015
Phone: 565-2867, FAX: 565-5687**

8/10/1999 REVISION

Revision	Date

REFERENCE:

FIGURE 1601

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The following list of references are prefaced with the acronym and date format (i.e. FAA, 1970) used in the written test of the MANUAL. This format allows the user to easily find the corresponding document referenced in the various sections of the MANUAL.

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X

Y

Z

DRAINAGE STUDY INFORMATION FORM

Name of Development:______________________________ Date: ______________

Location of Development: a) Descriptive (Cross Streets) North/South: ______________________

East/West:______________________

b) Section:______________ Township:______________ Range: ______________

c) APN :______________________________

____

Name of Owner:______________________________

Telephone No.:______________ Fax No.:______________ E-Mail Address: ______________

Address:______________________________

Contact Person-Name: ______________________________ **Telephone No.:** ______________

* **E-Mail Address:** ______________________________ **Fax No.:** ______________

Firm: ______________________________

Address: ______________________________

Type of Land Development/Land Disturbance Process:

	Rezoning		Subdivision Map		Clearing and Grading Only
	Parcel Map		Planned Unit Development		Other (Please specify below)
	Large Parcel Map		Building Permit		

1. Total Owned Land Area: At Site:______________ Being Developed/Disturbed:______________
2. Is a portion or all of the subject property located in a designated FEMA Flood Hazard Area? ☐ **Yes**** ☐ **No**
3. Is the property bordered or crossed by an existing or proposed Clark County Regional Flood Control District Master Planned Facility? ☐ **Yes**** ☐ **No**
4. Proposed type of development (Residential, Commercial, Etc.):______________________
5. Approximate upstream land area which drains to the subject site:______________________
6. Has the site drainage been evaluated in the past? ☐ **YES** ☐ **NO** If yes, please identify documentation:______________

   ____
7. If known, please briefly identify the proposed discharge point(s) of runoff from the site:______________

   ____

   ____
8. Briefly describe your proposed schedule for the subject project:______________________

   ____

   ____

Submit this form as part of the required drainage study to the local entity which has jurisdiction over the subject property. This form may provide sufficient information to serve as the Conceptual Drainage Study.

***New Required Field**

****Review and concurrence of the Clark County Regional Flood Control District is required.**

Revision	Date

Engineer's Seal

____

Local Entity File No.

REFERENCE:

STANDARD FORM 1

Updated 05/01/2008

HYDROLOGIC CRITERIA AND DRAINAGE DESIGN MANUAL		
DRAINAGE SUBMITTAL CHECKLIST		
Project Name:	Map ID:	
Firm Name:	Engineer:	
Address: City:	State:	Zip:
Phone Number:	Fax Number:	
Property Owner:		
Address: City:	State:	Zip:
Reviewed By:	Date Received:	Date Accepted for Review:

The following checklist is intended as a guide for the engineer preparing a Technical Drainage Study to submit to the local entity and Clark County Regional Flood Control District (if necessary). The listed items are the minimum information required prior to the entity performing a review. The engineer will remain responsible to ensure the Technical Drainage Study is prepared within the guidelines as set forth in the Clark County Regional Flood Control District (CCRFCD) Hydrologic Criteria and Drainage Design Manual (MANUAL).

This document is intended as an aid in preparing Technical Drainage Studies. Each study submitted is reviewed for compliance with local and regional criteria. This form is not intended to be all inclusive and does not limit the extent of the information, calculations or exhibits which may be necessary to properly evaluate the intended land use.

If items are not applicable for the subject site, provide N/A.

I. GENERAL REQUIREMENT

Yes	No	
______	______	Design Manual **Standard Form 1** with the engineer's seal and signature.
______	______	Design Manual **Standard Form 4**.
______	______	2 copies of the 24” x 36” Drainage Plan.
______	______	A notarized letter from the adjacent property owner(s) allowing off-site grading or discharge.

II. MAPS AND EXHIBITS

Yes	No	
______	______	A copy of a current Flood Insurance Rate Map (FIRM) with the site delineated.
______	______	A copy of the current CCRFCD Master Plan Update Figure, (F-x), for Flood Control Facilities and Environmental areas with the site delineated.

REFERENCE:	**STANDARD FORM 2**

DRAINAGE SUBMITTAL CHECKLIST

II. MAPS AND EXHIBITS (Continued)

Yes	No	
______	______	Off-site drainage basin maps for existing, interim and future conditions showing the existing topography, basin boundaries, concentration points, and flows in cfs.
______	______	On-site drainage basin maps for existing and proposed conditions showing the existing topography, basin boundaries, concentration points, and on-site and off-site flows in cfs.
______	______	Vicinity Map with local and major cross streets identified and a north arrow.

III. DRAINAGE PLAN

Yes	No	
______	______	Sheet size: 24” x 36” sealed by a registered engineer in the State of Nevada.
______	______	Minimum scale: 1” = 60’.
______	______	Project name.
______	______	Vicinity Map with local and major cross streets.
______	______	Revision box.
______	______	North arrow and bar scale.
______	______	Engineer’s/consultant’s address and phone number.
______	______	Elevation datum and benchmark.
______	______	Legend for symbols and abbreviations.
______	______	Cut/fill scarps, where applicable.
______	______	Street names, grades, widths.
______	______	Proposed future and existing spot grades for top of curbs and street crowns at lot lines, grade breaks, and along curb returns on both sides of the street.
______	______	Existing contours encompassing the site and 100 feet beyond with spot elevations for important locations, where appropriate.
______	______	Minimum finish floor elevations with top-of-curb elevations at upstream end of lot.
______	______	Proposed typical street sections.

REFERENCE:	STANDARD FORM 2

DRAINAGE SUBMITTAL CHECKLIST

III. DRAINAGE PLAN (Continued)

Yes	No	
_____	_____	Streets with off-set crowns.
_____	_____	Proposed contours or spot elevations in sufficient detail to exhibit intended drainage patterns and slopes.
_____	_____	Property lines.
_____	_____	Right-of-way lines and widths, existing and proposed.
_____	_____	Existing improvements and their elevations.
_____	_____	Delineation of proposed on-site drainage basins indicating area and 10-year and 100-year storm peak flows at basin concentration points.
_____	_____	Concentration points and drainage flow direction with Q_{100} and V_{100} and D_{100} in streets.
_____	_____	Cumulative flows, velocity, and direction of flow at upstream and downstream ends of site for the 10-year and 100-year flows.
_____	_____	Location and cross-section of street capacity calculations.
_____	_____	Cross-sectional detail for channels, including cutoff wall locations.
_____	_____	Existing and proposed drainage facilities, appurtenances, and connections (i.e., sidewalk, ditches, swales, storm drain systems, unimproved and improved channels, and culverts, etc.) stating size, material, shape, and slope with plan and profile and HGL calculations.
_____	_____	Existing and proposed drainage easements and widths shown with sufficient detail. A cross sectional detail must be provided that shows appropriate lining and reinforcement.
_____	_____	Location and detail of existing, proposed, and future block wall openings. Minimum size is 16" x 48". Wrought iron gate is required for flows > 10 cfs.
_____	_____	Location and detail of flood walls illustrating depth of flow, proposed grouting height, etc.
_____	_____	Perimeter retaining wall locations. All existing and proposed walls (retaining screen and flood) must be shown with adjacent ground elevations. Flood walls with 8-inch concrete masonry unit.
_____	_____	Building and/or lot numbers.
_____	_____	Alignment of all existing, proposed, or future Regional Facilities adjacent to the site.
_____	_____	Limits of existing floodplain based on current FIRM or best available information; limits of proposed floodplains based on best available information.

REFERENCE:	STANDARD FORM 2

DRAINAGE SUBMITTAL CHECKLIST

III. DRAINAGE PLAN (Continued)

Yes	No	
______	______	For areas in Zone A, AE, AH, and AO, base flood elevations (BFEs) must be shown for each lot; BFEs may be listed on each lot, or in a table. Finish floor elevations must be a minimum of 18 inches above BFE.
______	______	Appropriately elevated "humps" 6 inches above the 100 year water surface elevation at site accesses where the intent is to protect the site from the Q_{100} flows.
______	______	Street slopes for perimeter and interior streets. The minimum slope is 0.4 percent.
______	______	Location and detail of best management practice (BMP) for parking lots and low impact development (LID) (if required).

IV. HYDROLOGIC ANALYSIS

Yes	No	
______	______	Appropriate soil information and Soils Map for existing and future conditions with subbasins and property delineated.
______	______	Input and output information for existing conditions from computer models (HEC-1 or TR-55). The flow routing diagram must be provided with HEC-1 models.
______	______	Input and output information for future conditions from computer models (HEC-1 or TR-55). The flow routing diagram must be provided with HEC-1 models.
______	______	Use of correct precipitation values in and around the McCarran Airport rainfall area.
______	______	A discussion in the text of the hydrologic analysis justifying subbasin boundaries and cutoffs, supporting assumptions, and calculations.
______	______	A summary table of stormwater flows showing basin area, Q_{10} and Q_{100} for both individual basins and combined basin flows, where applicable.
_______	______	Copies of supporting technical information referenced from a previously approved study and a statement accepting these results.
______	______	On-site facilities must perpetuate flows through or around the site without significantly impacting adjacent property owners in accordance with current Nevada Drainage Law.
______	______	Calculation for impervious area for parking lots and LIDs (if required).

REFERENCE:	STANDARD FORM 2

HYDROLOGIC CRITERIA AND DRAINAGE DESIGN MANUAL

DRAINAGE SUBMITTAL CHECKLIST

V. HYDRAULIC ANALYSIS

Yes No

______ ______ Flow split calculations and supporting documentation or reference for the method of flow split calculations used.

______ ______ Normal depth street flow calculations and cross section diagrams for all interior and perimeter streets. Provide "d x v" products for the Q_{100} and Q_{10} flows representing the worst case for interior and all perimeter streets. Q_{100} d x v $\leq$ 8. Q_{10} d x v $\leq$ 6 and 12 foot dry lane for rights-of-way $\geq$ 80 feet. Calculations must be labeled by street name as indicated on the Grading Plan.

______ ______ A summary table of interior and exterior street capacity calculations showing the street name, Q_{100} flow, slope, depth of flow, velocity and depth times velocity product and streets needing to meet 12 foot dry lane criteria.

______ ______ Appropriate hydraulic calculations for block wall openings assuming a 50 percent vertical clogging factor. (Assume the lower half of the opening is plugged.)

______ ______ Appropriate hydraulic calculations at drainage easement entrance and discharge locations to set finish floor elevations. Hydraulic calculations must include submerged weir, superelevation and tee intersection losses, where appropriate.

______ ______ Provide necessary freeboard requirements to set the finished floor elevations of all proposed buildings, 2 x depth of flow or depth of flow plus 18 inches of freeboard, whichever is less. The minimum requirement is 6 inches above adjacent upstream top of curb. Buildings adjacent to drainage easements must always be provided with 18 inches of freeboard above the Q_{100} weir height or flow depth, whichever is greater.

______ ______ A complete water surface profile analysis (HEC-2, HEC-RAS, etc.) for channel flows and FEMA Zone A flood zones.

- Field survey data.
- Input and output information.
- Plotted cross-sections based on survey with proper encroachments.
- A map showing the location of the cross-sections.
- Analysis of both sub and super-critical flow segments.
- A summary table and a discussion of the results in the text of the report.

______ ______ Provide a 50 percent clogging factor in the capacity calculation for drop inlets.

______ ______ Hydraulic calculations for culverts and storm drains. D-Load calculations must be provided for storm drain pipes in public rights-of-way, including headwater pool inundation.

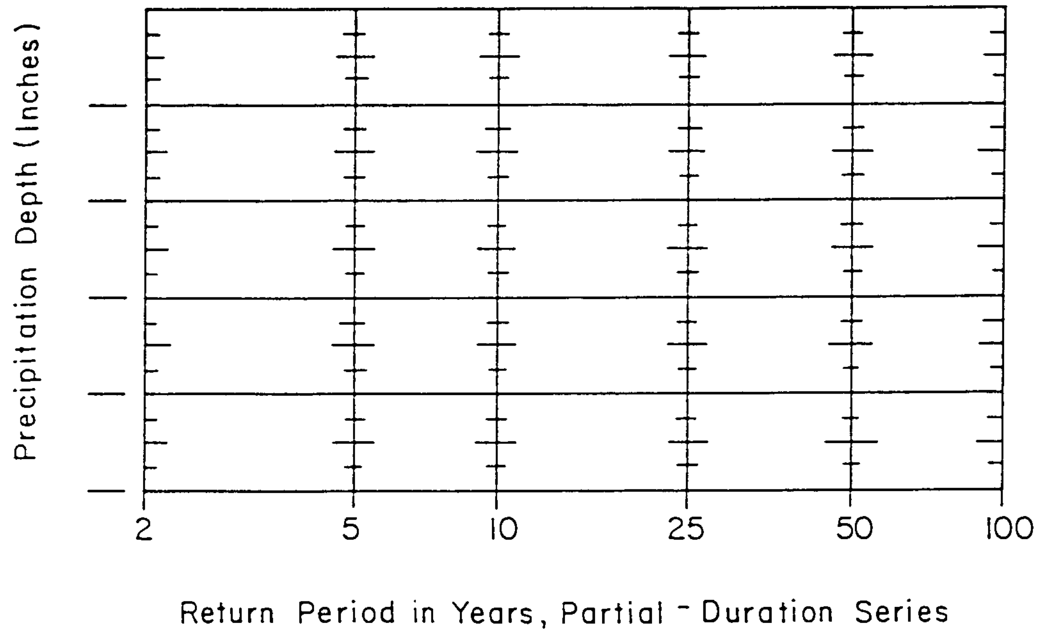
______ ______ The mitigation of nuisance water, both during construction and in the fully developed condition, must be addressed.

______ ______ Provide BMP type, size and supporting calculations for parking lots and LIDs (if required).

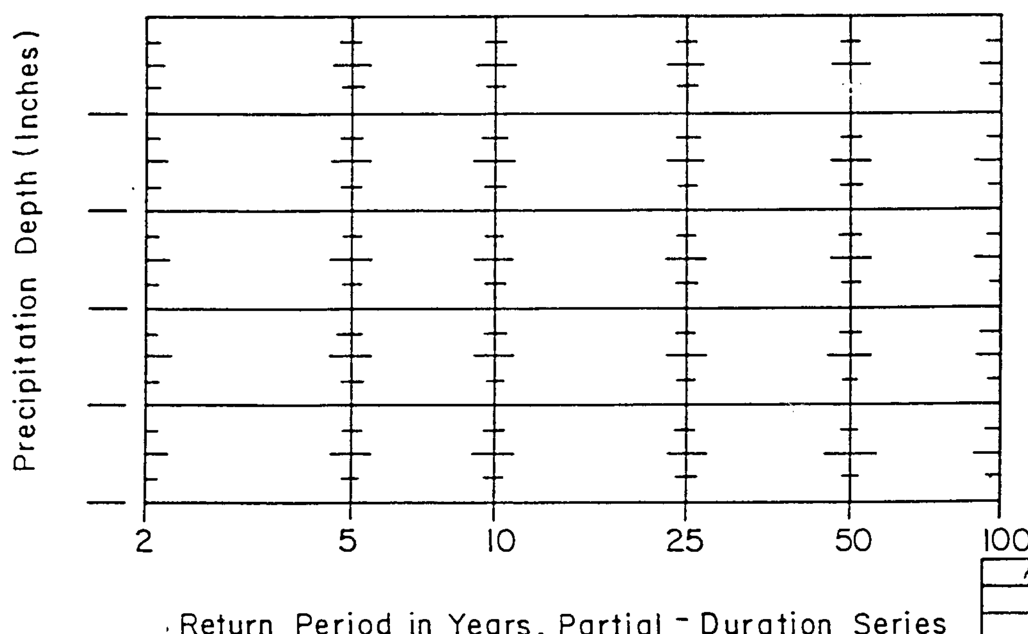
REFERENCE:	**STANDARD FORM 2**

PRECIPITATION DEPTH VERSUS RETURN PERIOD

A. PLOT OF 6-HOUR AND 24-HOUR PRECIPITATION DEPTH VALUES



B. PLOT OF 1-HOUR PRECIPITATION DEPTH VALUES



Revision	Date

REFERENCE: NOAA ATLAS 2, VOLUME VII NEVADA, 1973

STANDARD FORM 3

TIME OF CONCENTRATION AND HEC-1 INPUT

DEVELOPMENT ______________________________

CALCULATED BY ____________________ **DATE** ______________

Sub-Basin Data				Initial/Overland Time (t_i)			Travel Time (t_t)				tc Check (Urbanized Basins)		Final t_c			Remarks
KK			BA											UD	LS	HEC-1 Card
Desig: (1)	K (2)	Area Ac (3)	Area sq mi (4)	Length Ft (5)	Slope % (6)	t_i Min (7)	Length F1 (8)	Slope % (9)	Vel. FPS (10)	t_t Min (11)	Total Length Ft (12)	t_c = (L/180) + 10 Min (13)	Min (14)	LagTime hr (15)	CN (16)	

$$t_i = 1.8\,(1.1 - K)\,L^{1.2}/S^{1/3}$$

$$t_c = t_i + t_t$$

Revision	Date

REFERENCE:

TABULAR HYDROGRAPH DISCHARGE SUMMARY

Project ______________________ Location ______________________ By ________ Date ________

Circle one: Present Developed ______________________ Frequency (yr) ________ Checked ________ Date ________

Subarea name	Basic watershed data used: Sub-area T_c (hr)	ΣT_t to outlet (hr)	I_a/P	A Q (mi^2-in)	Select and enter hydrograph times in hours from Table 603											
					Discharges at selected hydrograph times 1 (cfs)											
Composite hydrograph at outlet																

Notes:
1. Hydrograph discharge for selected times is A*Q multiplied by tabular discharge from appropriate Table 603.

Revision	Date

REFERENCE:

STORM SEWER HYDRAULIC CALCULATIONS

CONDUIT DATA						FLOW DATA							ENERGY LOSS DATA							ENERGY AND HYDRAULIC DATA				COMMENTS	
STATION		SIZE / TYPE	INVERTS		LENGTH					$H_v =$		AVE.	PIPE	TRANSITIONS						PIPE		TRANSITION		PRESS. FLOW ?	OTHER
FROM	TO		D/S ELEV.	U/S ELEV.		A (FT^2)	K	Q (CFS)	V (FPS)	$V^2/2g$ (FT)	S_f (FT/FT)	S_f (FT/FT)	H_f (FT)	H_t (FT)	H_j (FT)	H_m (FT)	H_ (FT)	H_ (FT)	TOTAL (FT)	U/S EGL ELEV.	U/S HGL ELEV.	U/S EGL ELEV.	U/S HGL ELEV.		

Enter Starting EGL and HGL Here → ←

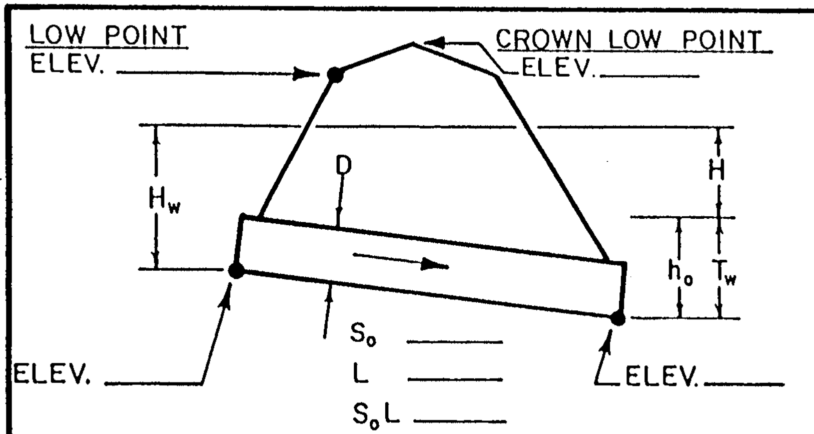
$$S_f = \frac{K\,H_v}{R^{1.33}} \qquad K = \frac{2g(n^2)}{2.21}$$

Revision	Date

WRC ENGINEERING, INC. | REFERENCE: | | STANDARD FORM 6

CULVERT RATING

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 $T_w < D$; $h_o = \dfrac{d_c + D}{2}$ 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

Revision	Date

REFERENCE:

STANDARD FORM 7