# MASTER

## TABLE OF CONTENTS

# SECTION 100

## INTRODUCTION

# SECTION 100
## INTRODUCTION

## TABLE OF CONTENTS

# SECTION 100
# INTRODUCTION

## 101 GENERAL INDEX



Each section of this Manual is numbered separately. All subsections are numbered in such a way as to allow additions and deletions with ease. All figures and tables are numbered and are placed either within the text or at the end of respective sections. The figure and table numbers are bold faced in the text for their ease of finding.

## 102 PURPOSE AND INTENT

The purpose of this Manual is to promote public health, safety, and general welfare and to minimize public and private losses due to flooding by adopting policies, procedures, standards, and criteria for storm drainage.

All new development and redevelopment, building permits for commercial and industrial uses, or any other proposed construction submitted for acceptance under the provisions of the Stormwater Management Manual shall include adequate storm drainage system analysis and appropriate drainage system design. Such analysis and design shall meet or exceed the criteria set forth herein.

## 103 AMENDMENTS TO THIS MANUAL

These policies, procedures, standards and criteria may be amended as new technology is developed and/or if experience gained in the use of this Manual indicates a need for revision.

### 103.1 Mesa County

Amendments and revisions will be adopted by Board of County Commissioners.

### 103.2 City of Grand Junction

Amendments and revisions may be made administratively by the City Manager or the City Manager's designee and shall have the same authority as the original document adopted by City Council. Amendments and revisions may also be made by City Council by ordinance.

### 103.3 Other Jurisdictions

Amendments and revisions may be made by the City of Fruita, the Town of Palisade, and the Grand Junction Drainage District, in accordance with their procedures.

## 104 PURCHASE OF A COPY OF THIS MANUAL

Any party desiring to purchase a copy of this Manual, provided such copies are available, may contact the City of Grand Junction Public Works Department at 970.244.1555. The entire copy of the Manual is also available for viewing or downloading from the County's website at: www.mesacounty.us/publicworks/stormwater.aspx
or the City's website at:
www.gjcity.org/CityDeptWebPages/PublicWorksAndUtilities/StormWater/StormWater.htm

## 105 ACKNOWLEDGEMENT

This Manual was prepared by WRC Engineering, Inc. working under the direction of Mesa County. Representatives from the Cities of Grand Junction and Fruita, Town of Palisade and Grand Junction Drainage District participated and provided valuable input during the preparation of this Manual.

We wish to acknowledge and thank the following individuals and organizations for their participation and contribution to the preparation of this Manual.

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## 106 UPDATING AND REVISIONS

This Manual was prepared in accordance with existing engineering methodologies and applicable criteria at the time of publication. The field of stormwater management has experienced rapid and extensive changes in recent years. Therefore, periodic review and revision of all material contained herein is recommended.

Users of this Manual are encouraged to notify Mesa County and/or local jurisdictions of any errors or omissions in this document. Comments and questions relating to the Manual may also be directed to the entities listed below.

Please submit input to:

Mesa County Engineering Department
c/o Ms. Julie Constan, P.E. (or successor)
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Applicable Local Jurisdiction(s)

# MASTER TABLE OF CONTENTS

**SECTION 300 – DRAINAGE PLANNING SUBMITTAL REQUIREMENTS**



**SECTION 400 – DRAINAGE POLICY**

**SECTION 600 – RAINFALL**



**SECTION 700 – STORM RUNOFF**

**SECTION 900 – ADDITIONAL HYDRAULIC STRUCTURES**

**SECTION 1100 – STREETS**



**SECTION 1200 – CULVERTS AND BRIDGES**

**SECTION 1300 – IRRIGATION / DRAINAGE STRUCTURES**

**SECTION 1400 – DETENTION AND RETENTION**

**SECTION 1500 – CONSTRUCTION SITE STORMWATER RUNOFF CONTROL**

## SECTION 1600 – POST CONSTRUCTION STORMWATER MANAGEMENT



## SECTION 1700 – STANDARD FORMS

## SECTION 1800 – REFERENCES

## APPENDIX A – DRAINAGE LAW

## APPENDIX B – EXCERPTS FROM UDSCD VOLUME 3

# SECTION 200

## GENERAL PROVISIONS

# SECTION 200
## GENERAL PROVISIONS

## TABLE OF CONTENTS

# SECTION 200
# GENERAL PROVISIONS

## 201 TITLE

This Manual and design standards with all future amendments and revisions shall be known as the **Stormwater Management Manual** (herein referred to as the Manual or SWMM).

## 202 JURISDICTION

This Manual and design standards shall apply to all new development and redevelopment within the boundaries of Mesa County, including any public lands, facilities constructed on rights-of-way, easements dedicated for public use, and to all privately owned and maintained drainage facilities, including but not limited to detention ponds, storm drains, inlets, manholes, culverts, swales, channels and water quality facilities.

### 202.1 Mesa County

Chapter 7.7 of the Mesa County Land Development Code (Code) requires that "*Drainage facilities shall be designed and installed in accordance with the Mesa County Stormwater Management Manual*".

### 202.2 City of Grand Junction

The City of Grand Junction Zoning and Development Code requires that "All proposed development must provide for on-site runoff collection and conveyance in accordance with the Stormwater Management Manual (SWMM) and applicable federal and state laws."

The City of Grand Junction adopted stormwater pollution prevention Ordinance Number 3824, which requires that: "All proposed development…must provide for on-site erosion and sediment control, control of illegal discharges, and runoff collection and conveyance in accordance with the Stormwater Management Manual and applicable federal and state laws."

### 202.3 Other Jurisdictions

The City of Fruita, the Town of Palisade, and the Grand Junction Drainage District may adopt the Stormwater Management Manual in whole or in part.

## 203 ADOPTION AUTHORITY

### 203.1 Municipality

Powers to regulate land use activities, including drainage, are granted to a municipality under Colorado Revised Statutes as noted below:

CRS 31-15-701 et seq. Grants municipalities the power to establish, improve, and regulate such improvements as streets and sidewalks, water and water works, sewers and sewer systems, and water pollution controls. In addition, a municipality may, among other powers, deepen, widen, pipe, cover, wall, alter or change the channel or watercourses.

CRS 31-25-501 et seq. Authorizes municipalities to construct local improvements and assess the cost of the improvements wholly or in part upon property specially benefited by such improvements. By ordinance, a municipality may order construction of district sewers for storm drainage in districts called storm sewer districts. (For the City of Grand Junction, see People's Ordinance No. 33.)

CRS 31-25-601 et seq, Authorizes municipalities to establish improvement districts as taxing units for the purpose of constructing or installing public improvements.

CRS 31-35-401 et seq. Authorizes municipalities to operate, maintain, and finance water and sewage facilities for the benefit of users within and without their territorial boundaries. Sewerage facilities are defined as "any one or more of the various devices used in the collection, treatment or disposition of sewerage or industrial wastes of a liquid nature or storm, flood or surface drainage waters…."

### 203.2 County

Powers to regulate land use activities, including drainage, are granted to a county under Colorado Revised Statutes as noted below:

CRS 24-67-101 et seq. (Planned Unit Development Act of 1972) power to encourage more efficient and innovative use of the land for public services and encourages integrated planning.

CRS 29-20-101 et seq. (Local Government Land Use Control Enabling Act of 1974) clarifies and provides broad authority to local governments to plan for and regulate the use of land within hazardous and environmentally sensitive areas.

CRS 30-20-401 et seq. Authorizes construction, maintenance, improvements and financing of water and sewerage facilities for the county's own use and for the use of the public and private consumers and users within and without the county's territorial limit.

CRS 30-20-501 et seq. (County Public Improvement District Act of 1968) authorizes creation of public improvement districts within any county as taxing units and for the purpose of implementing public improvements.

CRS 30-20-601 et seq. authorizes a county by resolution to construct local improvements and to assess the costs to properties especially benefited by improvements.

CRS 30-28-101 et seq. provides the County with planning authority, such as development of master plans and adoption of such plans by resolution; creation of a regional planning commission; and regulation of development density.

CRS 30-30-101 et seq. (Control of Stream Flow) provides power to remove any obstruction to the channel of any natural stream which causes a flood hazard and provides a right of access to any such natural stream

CRS 37-20-101 and 37-33-101 et seq. authorizes owner of agricultural lands susceptible to drainage problems from the same general system to petition the board of county commissioners to set up a drainage district.

## 204 ENFORCEMENT RESPONSIBILITY

### 204.1 Mesa County

The Director or an authorized representative is responsible for enforcing the provisions of this Manual.

### 204.2 City of Grand Junction

The City Manager or the City Manager's Designee is responsible for enforcing the provisions of this Manual.

### 204.3 Other Jurisdictions

Contact the City of Fruita, the Town of Palisade, or the Grand Junction Drainage District for information.

## 205 REVIEW AND ACCEPTANCE

All drainage submittals will be reviewed for general compliance with this Manual. An acceptance does not relieve the owner, engineer, or designer from responsibility of ensuring that the calculations, plans, specifications, construction, and record drawings comply with this Manual.

Adequate time must be allocated in development planning to permit a complete review. The intent of this Manual is to more clearly define the requirements and reduce the time and effort required to develop an acceptable drainage design.

### 205.1 Mesa County

Acceptance of Final Drainage Report and drainage facility construction plans shall be valid for 2 years. Documents with approvals more than 2 years old may require revision prior to development to comply with the provisions of the Manual in effect at that time. Amendments to this Manual will apply to all drainage reports submitted after the effective date of the amendment. Final drainage reports are exempt from an amendment provided they are submitted for approval within 60 days after the effective date of an amendment.

### 205.2 City of Grand Junction

Drainage submittals shall contain all information required by this Manual but the review process will be in accordance with the City's *Submittal Standards for Improvements and Development* (SSID) and the Zoning and Development Code.

### 205.3 Other Jurisdictions

Contact the City of Fruita, the Town of Palisade, or the Grand Junction Drainage District for information.

## 206 INTERPRETATION

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

1. **Minimum Standards.** This Manual shall be regarded as the minimum requirements for analysis and design of storm drainage facilities. Special site conditions or mitigation for potential impacts from new development or redevelopment may result in more stringent standards.

2. **Higher Standards.** For Mesa County if provisions of the Code, any law, ordinance, resolution, rule, or regulation contain restrictions covering the same subject matter, the more stringent standards or requirements shall govern. For the City of Grand Junction if provisions of the Zoning and Development Code, any law, ordinance, resolution, rule, or regulation contain restrictions covering the same subject matter, the more stringent standards or requirements shall govern.

3. **Flexibility**. There may on occasion be need for site specific application and interpretation of this Manual. The Director in Mesa County or the City Manager or the City Manager's designee in the City of Grand Junction may deviate from the requirements of this Manual, provided that the approved plan is compatible with surrounding in-place improvements and is sufficiently protective. The burden of responsibility shall be on the applicant to show that the requested deviation/variance from standards does not create a public hazard.

4. **Abrogation**. This Manual shall not abrogate or annul any permits or approved drainage reports, construction plans, easements, or covenants issued before the effective date of this Manual.

## 207 DEVIATION/VARIANCE PROCEDURES

Deviations/variances from specific standards, procedures, or criteria in this Manual may only be requested for:

1. Unusual situations where strict compliance with the Manual may not protect the public health and safety, or

2. Unusual situations which require additional analysis outside the scope of the Manual for which the additional analysis shows that strict compliance with the Manual may not protect the public health and safety, or

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

### 207.1 Mesa County

A variance from the technical provision of this Manual may be granted by the Director. All requests for variances shall be submitted in writing, (normally with the drainage report, see Section 300), shall state the provision for which the variance is requested, and shall provide evidence, data or other information in support of the request. The Director will review and rule on the request and provide his findings in writing.

### 207.2 City of Grand Junction

A deviation from any requirement of this Manual may be granted by the City Manager or the City Manager's designee. A request for deviation shall be submitted in writing as a separate letter attached to the drainage report. The request shall state the provision for which the deviation is requested and shall provide supporting evidence, data, or other appropriate information. The City Manager or the City Manager's designee shall review and rule on the request and provide the findings in writing.

### 207.3 Other Jurisdictions

Contact the City of Fruita, the Town of Palisade, or the Grand Junction Drainage District for information.

## 208 ACRONYMS

The following acronyms are used within the context of this Manual.

| | |
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| BMP | Best Management Practice |
| CAP | Corrugated Aluminum Pipe |
| CAPA | Corrugated Aluminum Pipe Arch |
| CMP | Corrugated Metal Pipe |
| CMPA | Corrugated Metal Pipe Arch |
| CDOT | Colorado Department of Transportation |
| CDPHE | Colorado Department of Public Health and Environment |
| CDPS | Colorado Discharge Permit System |
| CRS | Colorado Revised Statues |
| CSP | Corrugated Steel Pipe |
| CSPA | Corrugated Steel Pipe Arch |
| CSWMP | Construction Stormwater Management Plan |
| CWA | Clean Water Act |
| CWCB | Colorado Water Conservation Board |
| EC | Erosion Control |
| ECP | Erosion Control Plan |
| EGL | Energy Grade Line |
| EPA | Environmental Protection Agency |
| FEMA | Federal Emergency Management Agency |
| GIS | Geographic Information System |
| HDS | Hydraulic Design Series |
| HEC | Hydrologic Engineering Center or Hydraulic Engineering Circular (FHWA) |
| HERCP | Horizontal Elliptical Reinforced Concrete Pipe |
| HGL | Hydraulic Grade Line |
| MS4 | Municipal Separate Storm Sewer System |
| NAVD | North American Vertical Datum |

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| NFIP | National Flood Insurance Program |
| NOAA | National Oceanic and Atmospheric Administration |
| NRCS | Natural Resource Conservation Services, formerly the SCS |
| NPDES | National Pollutant Discharge Elimination System |
| NWS | National Weather Service |
| PE | Professional Engineer Licensed by the State of Colorado |
| PMF | Probable Maximum Flood |
| PVC | Poly Vinyl Chloride |
| RCBC | Reinforced Concrete Box Culvert |
| RCP | Reinforced Concrete Pipe |
| ROW | Right-of-Way |
| SCS | Soil Conservation Service |
| SPP | Structural Plate Pipe |
| SSID | Submittal Standards for Improvements and Development (City of Grand Junction) |
| SWMM | Stormwater Management Manual |
| SWMP | Stormwater Management Plan (either the Construction SWMP or the final drainage report containing post-construction BMPs) |
| TAC | Technical Advisory Committee |
| TRC | Technical Review Committee |
| UDFCD | Urban Drainage and Flood Control District (Denver, Co.) |
| USACE | United States Army Corps of Engineers |
| USBR | United States Bureau of Reclamation |
| USDCM | Urban Storm Drainage Criteria Manual, prepared by the Urban Drainage & Flood Control District in three volumes. |
| USGS | United States Geological Survey |
| WQCD | Water Quality Control Division of the Colorado Department of Public Health and Environment |
| WQCV | Water Quality Capture Volume |

## 209 DEFINITIONS

| Term | Definition |
|---|---|
| Applicant | A qualified agent, individual or firm acting on behalf of the owner of property requesting approval of plans for new development and redevelopment. |
| Authority or Drainage Authority | Mesa County, the Cities of Grand Junction and Fruita, the Town of Palisade, and the Grand Junction Drainage District have contracted through intergovernmental agreements (IGA) to form the 5-2-1 Drainage Authority (Authority), under Colorado Revised Statutes, specifically CRS 29-1-204.2. |
| Best Management Practices | Schedules of activities, prohibitions of practices, maintenance procedures, and other management practices to prevent or |

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| | reduce the pollution of "state waters." BMPs also include treatment requirements, operating procedures and practices to control site runoff, spillage or leaks, sludge or waste disposal, or drainage from raw material storage. |
| CDPS Permit | Permit issued by the state of Colorado under Part 5 of the Colorado Water Quality Control Act that authorizes the discharge of pollutants to waters of the state, whether the permit is applicable to a person, group or area. |
| Channel | A natural or artificial low-lying area with a definite bed and banks, which confines and conveys continuous or periodic flows of water. (Mesa County Land Development Code @ 12.1 & the City of Grand Junction Zoning and Development Code, Chapter Nine Definitions) |
| Clean Water Act | Federal act of 1977, 33 U.S.C. section 466 et seq. as amended. |
| Colorado Water Quality Control Act | Title 25, Article 8 of the Colorado Revised Statutes. |
| Commercial | Any business, trade, industry or other activity engaged in profit. |
| Construction Activity | Includes clearing, grubbing, filling, grading, and excavation. Does not include routine maintenance performed by public agencies, private parties, or their agents to maintain original line and grade, hydraulic capacity, or original purpose of the facility. Also see land disturbance. [(Adapted from CDPS General Permit No. COR-030000 @ Part I A(2)(a)] . |
| Construction Site | Any location where construction or construction related activity occurs. |
| Construction Stormwater Management Plan | A specific individual construction plan that describes the BMPs to be implemented at a site to prevent or reduce the discharge of pollutants during construction activities. |
| Director | Mesa County Planning Director or Designee when referring to a Mesa County official, but the City Manager or his designee when referring to a City of Grand Junction official. |
| Discharge | The addition or release of any pollutant, stormwater, subsurface, groundwater or any substance whatsoever to the storm drainage system. |
| Downstream | Refers to locations which are hydraulically lower in elevation than the location at which the comparison is being made. Downstream may include locations outside of the stream, channel, pipe, etc. such as sheet flow, which often meanders in several directions. |

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| Drainageway | Any natural or artificial (man-made) channel which provides a course for water flowing either continuously or intermittently to downstream areas. |
| Flood | Temporary rise in a watercourse, flow, or stage, that results in water over-topping its banks and inundating areas adjacent to the channel. (Mesa County Land Development Code @ 12.1. See also the City of Grand Junction Zoning and Development Code, Chapter Nine Definitions) |
| Final Stabilization | Means when all soil disturbing activities at the site have been completed, and uniform vegetative cover has been established with a density of at least 70 percent of pre-disturbance levels, or equivalent permanent, physical erosion reduction methods have been employed. For purposes of the Construction SWMP, establishment of a vegetative cover capable of providing erosion control equivalent to pre-existing conditions at the site will be considered final stabilization. |
| Grand Valley | The portion of Mesa County whose boundary is approximated by the 5,000 foot elevation contour and extends west of the community of Mack, along the north eastern edge of the Colorado National Monument, south of Whitewater, to the east of Palisade, and along the face of the Bookcliffs to the north. |
| Land Disturbance | A man made change in the existing cover or topography of the land, including grading, excavation, filling, building, paving, and other activities that may result in or contribute to soil erosion or sedimentation in the discharge of pollutants. |
| Local Facility | Detention and/or water quality facility that has been sized based on the area and imperviousness of the watershed that includes all the development that drains to the facility, but is not publicly owned and maintained. Also see "regional facility" and "on-site facility" |
| Local Jurisdiction | Within the context of the SWMM, means Mesa County, City of Grand Junction or Fruita, Town of Palisade, or the Grand Junction Drainage Authority. |
| Major Drainage System | A stormwater facility, such as a channel, large conduit, detention or retention, which receives storm runoff from a watershed generally 160-acres in size or larger. |
| Municipal Separate Storm Sewer System (MS4) | A conveyance or the system of conveyances, including roads with drainage systems, municipal streets, curbs, gutters, ditches, drainage inlets, catch basins, pipes, tunnel, culverts, channels, detention basins and ponds owned and operated by a municipality or county and designed or used for collecting or conveying stormwater that is not a combined sewer or used for collecting or conveying sanitary sewage. |

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| New Development and Redevelopment | For Mesa County definition, refer to the "Mesa County Land Development Code". For the City of Grand Junction definition, refer to the "City of Grand Junction Zoning and Development Code". For the City of Fruita, Town of Palisade and the Grand Junction Drainage District definitions, refer to the applicable development codes. |
| NPDES | The National Pollutant Discharge Elimination System under section 402 of the Clean Water Act. |
| On-site Facility | A Local Facility that is contained within and only serves the development in question and not other developments. |
| Outfall drainage system | The drainage system typically consisting of swales, curb and gutter, storm drains, and sometimes small open channels that discharge to a major drainage system. Also called "local drainage system". |
| Owner | A person having dominant and/or servient interest in property, having sufficient interest to convey property, and/or having possessory interest in property. The term owner also includes the owner's agent. |
| Part of a larger common plan of development or sale | A contiguous area where multiple separate and distinct construction activities will take place at different times on different schedules under one plan. An example would be a commercial development with multiple separate buildings constructed over the course of multiple construction schedules. |
| Pollutant | Dredged spoil, dirt, slurry, solid waste, incinerator residue, sewage, sewage sludge, garbage, trash, chemical waste, biological nutrient, biological material, radioactive material, heat, wrecked or discarded equipment, rock, sand, or any industrial, municipal, or agricultural waste. [CRS 25-8-103(15)] |
| Pollution | The alteration of the physical, thermal, chemical, or biological quality of, or the contamination of any water that renders the water harmful, detrimental, or injurious to humans, animal life, plant life, property or public health, safety or welfare, or impairs the usefulness or the public enjoyment of the water for any lawful or reasonable purpose. |
| Post-Construction Stormwater Management Plan | The final drainage report, in accordance with SWMM Section 303 *Final Drainage Report*, which includes a combination of structural and/or non-structural BMPs, further described in Section 1600 *Post-Construction Stormwater Management*, that reduce the discharge of pollutants after construction is complete. |

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| Pre-developed or pre-existing | Conditions that existed as of the adoption date of this Manual. |
| Private Drainage System | All privately owned ground, surfaces, structures or systems, excluding the MS4 that contribute to or convey stormwater including but not limited to roofs, gutters, downspouts, lawns, driveways, pavement, roads, streets, curbs, gutters, inlets, drains, catch basins, pipes, tunnels, culverts, channels, detention basins, ponds, draws, swales, streams and any ground surfaces. |
| Public Improvement | Any improvement, facility or service together with its associated public site, right-of-way or easement necessary to provide transportation, drainage, public private utilities, parks or recreational, energy or similar essential services. (Mesa County Land Development Code @ 12.1) |
| Qualified Erosion Control Specialist | A Qualified Person, as defined by City of Grand Junction Ordinance Number 3824, with specialized training, education, or experience in the field of erosion control methods, planning, and inspection. |
| Recurrence Interval | The recurrence interval corresponds to the statistical return period of an event of the same intensity (e.g. a 100 year recurrence interval flood has 1% chance to occur each year, which does not mean that it will occur every 100 years...). |
| Regional Facility | Detention and/or water quality facility that is publicly owned and maintained and serves all properties within the tributary watershed. |
| Sediment | Soil, mud, dirt, gravel and rocks that have been disturbed, eroded and/or transported naturally by water, wind or gravity and/or mechanically by any person, vehicle or equipment. |
| Significant Materials | Include but are not limited to: raw materials; fuels; materials such as metallic products; hazardous substances designated under section 101(14) of CERCLA; any chemical the facility is required to report pursuant to section 313 of title III of SARA; fertilizers; pesticides; and waste products such as ashes, slag and sludge that have the potential to be released with stormwater discharge. |
| Storm Drainage System | All surfaces, structures and systems that contribute to or convey stormwater including private drainage systems and the MS4, and any non-municipal drain or pipe, channel or other conveyance, including natural and artificial (man-made) washes and ditches for conveying water, groundwater, drainage water or unpolluted water from any source, excluding sewage and industrial wastes, to waters of the state and United States. |

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| Storm Water or Stormwater | Within the context of the Stormwater Management Manual, stormwater, whether one or two words, shall mean surface runoff resulting from precipitation. |
| Stormwater Management Plan | Within the context of the Stormwater Management Manual, a Stormwater Management Plan (SWMP) means the Construction SWMP (CSWMP) and/or the Post-Construction SWMP. |
| Stormwater Construction Permit | The permit issued by the CDPHE called the general permit for stormwater discharges associated with construction activities. Mesa County or the City of Grand Junction will also issue a Stormwater Construction Permit for land disturbances related to new development. |
| Stream Channel | The area of the floodplain which carries the normal flow of the watercourse. (Mesa County Land Development Code @ 12.1). |
| Urbanized Area | The area identified by the Colorado Department of Public Health and Environment based on the 2000 census and called the urban area of Mesa County. Within the context of the SWMM, urban area also includes the Redlands within the Urban Growth Boundary of the County. |
| Waters of the State | Any groundwater, percolating or otherwise, lakes, bays, ponds, impounding reservoirs, springs, rivers, streams, creeks, estuaries, marshes, inlets, canals, wells, watercourses, drainage systems, and irrigation systems; all sources of water such as snow, ice, and glaciers; and all other bodies or accumulations of water, surface and underground, natural or artificial, navigable or non-navigable, and including beds and banks of all water courses and bodies of surface water, public or private, located wholly or partly within or bordering upon this state and within the jurisdiction of this state. |
| Watershed Master Plan | The Grand Valley Stormwater Management Master Plan or other stormwater master plans approved by the Drainage Authority representing applicable municipalities and Mesa County. |
| Water Quality Capture Volume | The minimum storage volume, based on the 80th percentile event, which is retained and released over a specified period of time, depending on the specific structural BMP and is based on the time it takes to fully drain the brim-full volume contained in storage |
| Water Quality Outlet | Structure designed to release the water quality capture volume over the specified period of time for the specific BMP. |
| Wetland | Any area that is inundated or saturated by surface or |

groundwater at a frequency and duration sufficient to support, under normal circumstances, a prevalence of vegetation typically adapted to the saturated soil conditions. Wetlands generally include swamps, marshes, bogs and similar areas.

# SECTION 300

## DRAINAGE PLANNING SUBMITTAL REQUIREMENTS

# SECTION 300
## DRAINAGE PLANNING SUBMITTAL REQUIREMENTS

## TABLE OF CONTENTS

## LIST OF TABLES



## LIST OF FIGURES

# SECTION 300
# DRAINAGE PLANNING SUBMITTAL REQUIREMENTS

## 300 REVIEW PROCESS

All development within the jurisdiction of this Manual shall submit drainage reports, construction drawings/specifications, and record drawing information for review and approval in accordance with the requirements of this Section. The City of Fruita, the Town of Palisade, or the Grand Junction Drainage District may alter the requirements of this Section, in accordance with their guidance documents. **Figure 301** provides a flow chart of the general review process.

### 300.1 Mesa County

The County will review reports and plans for completeness of the submittal to the Planning Department. The County will provide written or oral review comments of the submittal. The County will make every effort to affect a complete review and comment within a reasonable period, however, the County cannot approve reports or plans by default.

The applicant or his designated representative is required to attend a pre-application conference to review processing steps in accordance with the Mesa County Land Development Code (Code @ 3.1.6). The applicant shall consult with the Director or representative for general information regarding land development regulations, required procedures, possible drainage problems and deviations/variances, and specific submittal requirements. As a minimum, a conceptual and final drainage report will be required, unless modified at the pre-application conference.

### 300.2 City of Grand Junction

The City shall review and approve drainage reports and construction plans in accordance with Submittal Standards for Improvements and Development (SSID), Zoning and Development Code, and the pollution prevention stormwater ordinance.

### 300.3 Other Jurisdictions

Refer to requirements of the City of Fruita, the Town of Palisade or the Grand Junction Drainage District for specific information regarding review and acceptance of drainage reports and plans.

## 301 SUBMITTAL REQUIREMENTS

A conceptual/preliminary and final drainage report is required for all new development and redevelopment, except as otherwise determined at the pre-application conference. The number of report copies to be submitted will be determined at the pre-application conference. One copy will be returned to the applicant or his representative with comments.

All submitted reports shall be clearly and cleanly reproduced. Copies of charts, tables, nomographs, calculations, or any other referenced material shall be legible. Washed out, blurred or unreadable portions of the report are unacceptable and may warrant resubmittal of the report. The submittal shall include a declaration of the type of report submitted (i.e., Conceptual/Preliminary or Final).

Table 302 shall be used to determine the adequacy of the submittal, in addition to those requirements identified at the pre-application meeting, which may alter the checklist. Incomplete or absent information may result in the report being rejected for review. Revision dates must be included on all re-submittals.

All development and redevelopment must provide construction site best management practices (see Section 1500), onsite detention (see Section 1400), and post-construction BMPs (see Section 1600), unless otherwise modified at the pre-application conference.

## 302 CONCEPTUAL/PRELIMINARY DRAINAGE REPORT

The purpose of the Conceptual/Preliminary Drainage Report is to:

- Identify drainage conditions prior to proposed development, including designated floodplain boundaries (see Table 302).
- Identify existing and potential drainage problems, which may occur on-site or off-site because of the development.
- Identify proposed solutions to drainage problems, including location of detention storage and water quality requirements, in sufficient detail to verify technical feasibility.

Text shall be typed on 8-1/2" x 11" paper. All pages, including appendices, shall be numbered in a fashion that identifies the report (e.g. PDR-#). Text, tables, figures, charts, calculations, and appendices shall be bound to form a formal report. Drawings shall be included in a pocket attached with the report, but shall not be smaller than 11" x 17" or larger than 24" x 36" in size. The report shall include a cover letter presenting the conceptual/preliminary design for review and shall be stamped and signed by an engineer licensed in Colorado.

### 302.1 Conceptual/Preliminary Report Contents

The Conceptual/Preliminary Drainage Report shall contain general information regarding the proposed drainage facilities for the development. For instance, only identify that a channel or storm drain is proposed for conveyance, and not the size, slope, velocity or other more detailed information. Also, it is only required to identify the location and type of detention (i.e.: drainage and water quality), and not the volumes or release rates, however, post-development flow rates shall be calculated and submitted with the Preliminary Report.

The report shall be prepared in accordance with the outline provided as part of the checklist in Table 302 (see Section 305). Grading and erosion control information are not required. The checklist must be completed by the applicant and included with the drainage report. The checklist will be used to determine the completeness of the report. If information provided is lacking or incomplete; the information may be required prior to further review. It is understood that information in the *Conceptual/Preliminary Drainage Report* is subject to change. More detail can be required if necessary due to the complexity of a development.

### 302.2 Conceptual/Preliminary Report Drawing Contents

a. **General Location Map**: A map (8-1/2" x 11") shall be provided in sufficient detail at a scale not larger than 1" = 1,000' and included with the report. The map shall identify:

   i. Drainage flows entering and leaving the development and general drainage patterns within the development.
   ii. Path of all drainage from the upper end of any off-site basins to the defined major drainageways (see Drainage Policy).
   iii. Major construction (i.e., development, irrigation ditches, existing detention facilities, culverts, and storm drains) along the entire path of drainage.
   iv. All major basins. Topographic contours are optional.

b. **Conceptual/Preliminary Drainage Plan**: Map(s) of the proposed development at a scale of 1" = 20' to 1" = 200' on a 24" x 36" drawing shall be included. The plan shall show the following:

   i. Existing and (if available) proposed contours at 5-foot maximum intervals. The contours shall extend beyond the property boundaries the distance necessary to show how the development interacts hydraulically with the surrounding area, but no less than 50 feet. Mesa County contours are acceptable provided they are representative of actual field conditions.
   ii. All existing drainage facilities. If existing (pre-development) conditions cannot be adequately addressed on the Proposed Drainage Plan, then a separate Existing Conditions Drainage Plan shall be prepared and included with the submittal document.
   iii. Approximate flooding limits based on available information.
   iv. Conceptual major drainage facilities including detention basins, storm drains, swales, riprap, and outlet structures in the detail consistent with the proposed development plan.
   v. Major drainage boundaries and sub-boundaries.
   vi. Any off-site feature influencing development, including both upstream and downstream structures.
   vii. Proposed flow directions entering, within, and exiting the development and, if available, proposed contours. Identify the drainage path from the development to the nearest MS4 facility or major drainageway
   viii. Existing and proposed irrigation facilities.
   ix. Legend to define map symbols.
   x. Title block in lower right corner.
   xi. All elevations shall be NAVD 1988.

## 303 FINAL DRAINAGE REPORT

The purpose of the Final Drainage Report is to:

- Identify drainage conditions prior to proposed development, including floodplain boundaries.
- Identify existing and potential drainage problems, which may occur on-site or off-site because of the development.

- • Investigate or refine conceptual solutions to drainage problems, including detention storage and water quality requirements, in sufficient detail to verify their technical feasibility.
- • Present final design and details for drainage facilities discussed in the Conceptual/Preliminary Drainage Report.
- • Identify post-construction BMPs to control the discharge of pollutants in stormwater to the maximum extent practicable.

Text shall be typed on 8-1/2" x 11" paper. All pages, including appendices, shall be numbered in a fashion that identifies the report (e.g. FDR-#). Text, tables, figures, charts, calculations, and appendices shall be bound to form a formal report. Drawings shall be 24" x 36" and included in a pocket attached with the report. The report shall include a cover letter presenting final design for review and shall be stamped and signed by an engineer licensed in Colorado.

### 303.1 Final Report Contents

The Final Drainage Report shall provide final details of proposed drainage facilities, including grading, erosion control, and water quality enhancement, and is to be submitted along with construction documents (see Section 305.3).

The *Final Drainage Report* shall be prepared by an engineer registered in Colorado in accordance with the outline provided as part of the checklist in **Table 302** (see Section 305). The checklist must be completed by the applicant and included with the drainage report. The checklist will be used to determine the completeness of the report. If information provided is lacking or incomplete: the information may be required prior to further review.

The report shall contain the following certifications:

"I hereby certify that this *Final Drainage Report* (plan) for the design of (Name of Development) was prepared by me (or under my direct supervision) in accordance with the provisions of the *Stormwater Management Manual* for the owners thereof. I understand that the (local jurisdiction) does not and will not assume liability for drainage facilities designed by others."

__________________________
Registered Professional Engineer
State of Colorado No._________
(Affix Seal)

I, (Name of Developer) hereby certify that the drainage facilities for (Name of Development) shall be constructed according to the design presented in this report. I understand that the (local jurisdiction) does not and will not assume liability for the drainage facilities designed and/or certified by my engineer. I understand that the (local jurisdiction) reviews drainage plans but cannot, on behalf of (Name of Development), guarantee that final drainage design review will absolve (Name of Developer) and/or their successors and/or assigns of future liability for improper design. I further understand that approval of the Final Plat and/or Final Development Plan does not imply approval of my engineer's drainage design.

Name of Developer

Authorized Signature Date

### 303.2 Final Report Drawing Contents

a. **General Location Map**: An 8-1/2" x 11" map shall be provided in sufficient detail at a scale not larger than 1" = 1,000' and included with the report. The map shall identify:

i. Drainage flows entering and leaving the development and general drainage patterns.
ii. Path of all drainage from the upper end of any off-site basins to the defined major drainageways (see Drainage Policy).
iii. Major construction (i.e., development, irrigation ditches, existing detention facilities, culverts, and storm drains) along the entire path of drainage.
iv. All major basins. Topographic contours are optional.

b. **Final Drainage Plan**: Map(s) of the proposed development at a scale of 1" = 20' to 1" = 200' on a 24" x 36" drawing shall be included. The plan shall show the following:

i. Existing and proposed contours at 2-foot maximum intervals. The contours shall be based on a USGS datum and extend a minimum of 50 feet beyond property lines, or further if required to show how the development interacts with the surrounding area.
ii. Location of benchmarks, which must be one of the following:
  a) Mesa County monumentation, including GIS reference points.
  b) USGS NAVD 1988
  c) City of Grand Junction; or
  d) Colorado Department of Transportation
iii. Property lines and easements with purposes noted.
iv. Existing street names and proposed streets, indicating names, ROW width, flowline width, curb type, sidewalk, and approximate slopes.
v. All existing drainage boundaries and peak flows entering and within the exiting development. Existing facilities such as irrigation and roadside ditches, drainageways, culverts, and detention sites. Include pertinent characteristics, such as location, size, shape, slope, and material. A separate Existing Conditions Drainage Plan shall be prepared and included with the submittal document that details the pre-development conditions listed in this line item.
vi. Any off-site feature influencing development, including both upstream and downstream structures.
vii. Proposed drainage boundaries and sub-boundaries.
viii. Proposed type of street flow (i.e., vertical or combination curb and gutter), roadside ditches, gutter slope, and flow directions, and cross-pans.

ix. Proposed storm drains and open drainageways, including inlets, manholes, culverts, and other appurtenances, including erosion/riprap protection.
x. Proposed outfall point for runoff from the developed area and facilities to convey flows to the final outfall point without damage to downstream properties.
xi. Routing and accumulation and flows at various critical points for the initial storm runoff listed on the drawing.
xii. Volumes and release rates for detention storage facilities and information on outlet works.
xiii. Location and elevations of all existing floodplains affecting the property.
xiv. Location and elevations of all existing and proposed utilities affected by or affecting the drainage design.
xv. Routing of off-site drainage flow through the development.
xvi. Definition of flow path leaving the development through the downstream properties ending at a major drainageway.
xvii. Legend to define map symbols (see **Table 301** for symbol criteria).
xviii. Title block in lower right hand corner.
xix. Identify each post-construction BMP including the WQCV provided or other bases for sizing BMPs.
xx. All elevations shall be NAVD 1988.

## 303.3 Construction Plans

### 303.3.1 Mesa County

The Final Construction Plans and Final Drainage Report must be submitted for acceptance to the Director at least 20 working days prior to the consideration of the plat before the County Commissioners. Before final subdivision plats and site plans can be submitted to County Commissioners for approval, the following conditions must be met:

a. Drainage reports and/or construction plans must be accepted by the Director without conditions,
b. All required easements and licenses with the County must be approved by the Director and the County Attorney, and the appropriate title insurance provided, and
c. Easements and other agency approvals must be fully executed and copies provided to the County.

Acceptance of the Final Construction Plans and Final Drainage Report are required prior to issuance of a permit.

Construction plans shall be prepared in accordance with sound engineering principles, this Manual and the Mesa County Land Development Code for subdivision designs. Construction documents shall include geometric, dimensional, structural, foundation, bedding, hydraulic, landscaping, and other details as needed to construct the storm drainage facility. The approved Final Drainage Plan shall be included as part of the construction document for all facilities affected by the drainage plan. Construction plans shall be signed by a registered professional engineer as being in accordance with the County approved drainage report, drawings, and this Manual. Requirements for construction plans are outlined in the checklist in **Table 303**.

#### 303.3.2 City of Grand Junction

Refer to the Submittal Standards for Improvements and Development for specific requirements for construction plans.

#### 303.3.3 Other Jurisdictions

Refer to requirements of the City of Fruita, Town of Palisade or the Grand Junction Drainage District for construction plans.

### 303.4 Post-Construction BMPs

Mesa County, the City of Grand Junction, the Grand Junction Drainage District, and the Town of Palisade, who are members of the Drainage Authority, have obtained permits to discharge stormwater under the Colorado Discharge Permit System (permit #'s COR-090031, COR-090077, COR-090006, and COR-090005, respectively). The terms and conditions of the permits set forth minimum requirements for stormwater management programs including Construction Site Stormwater Runoff Control and Post-Construction Stormwater Management for new development and redevelopment to reduce pollutants in all stormwater runoff to the MS4.

These permit conditions are the basis for requirements identified in Section 1600. The Final Drainage Report shall identify and include design bases, calculations, and construction details for post-construction BMPs, in accordance with requirements of Section 1600.

### 303.5 Construction SWMP

The permit conditions described in Section 303.4 are the basis for requirements identified in Section 1500. The Construction SWMP requirements, which are described in Section 1500, meet the conditions of the permit. The Construction SWMP is a separate document from the Final Drainage Report, must be prepared and certified by a qualified erosion control specialist, and is generally submitted for approval after the Final Drainage Report. Contact local jurisdiction for exact timing of submittal.

## 304 <u>RECORD DRAWINGS AND ACCEPTANCES</u>

### 304.1 Mesa County

#### 304.1.1 Record Drawings

Record drawings for all improvements are to be submitted to the County. Drawings shall be submitted in electronic format, along with Mylar (minimum 3 mil.) reproducible copy and paper prints. Drawings shall include appropriate seals and signatures in accordance with current state law, with the request for Probationary Acceptance of public improvements or prior to requesting a Certificate of Occupancy for commercial, industrial or multi-family residential building sites. Certification of the record drawings is required as follows:

a. Registered Professional Engineer (PE): A registered PE in the state of Colorado shall certify, based on survey from a registered land surveyor, the as-built detention pond volumes and surface areas at the design depths, outlet structure sizes and elevations, storm drain sizes and invert elevations at inlets, manholes, and discharge location, and representative open channel cross-sections, and dimensions of all the drainage structures.

b. Registered Professional Engineer: The responsible design engineer shall state that "*to the best of my knowledge, belief, and opinion, the drainage facilities were constructed in accordance with the design intent of the approved drainage report and construction drawings*".

The Director will compare the certified record drawing information with the construction drawings to ensure that:

a. The record drawing information demonstrates that the construction is in compliance with the design intent.

b. The record drawings are certified by a professional engineer licensed in Colorado.

### 304.1.2 Probationary Acceptance

All public storm drainage facilities shall be guaranteed by the developer to the County for a minimum 18-month warranty period (with the exception of drains over 20' deep which shall require a two-year warranty period or other special cases).

The developer is responsible for routine maintenance, any workmanship defects, and for removal and clean-up of construction debris, dirt and mud in the system during the warranty period.

### 304.1.3 Acceptances

a. For newly constructed public drainage improvements, the County may consent to a reduction of the improvements guarantee provided by the developer when the drainage improvements are granted Probationary Acceptance.

b. For new commercial, industrial and residential building sites, the drainage portion of the Certificate of Occupancy shall be accepted when the Record Drawings are determined by the County to comply with the above criteria.

## 304.2 City of Grand Junction

Refer to the Submittal Standards for Improvements and Development (SSID) for specific requirements regarding record drawings and acceptance.

### 304.3 Other Jurisdictions

Refer to requirements of the City of Fruita, Town of Palisade or the Grand Junction Drainage District regarding record drawings and acceptance.

## 305 SUBMITTAL CHECKLIST

### 305.1 Mesa County

To aid the designer and reviewer, a summary of the required certifications and approvals is presented below.

| ITEM | CERTIFICATION REQUIRED | COUNTY ACCEPTANCE REQUIRED |
|---|---|---|
| Conceptual Drainage Report | Engineer | No |
| Final Drainage Report | Engineer and Developer | Yes |
| Construction Drawings | Engineer | Yes |
| Construction SWMP (see Section 1500) | Qualified EC Specialist | Yes |
| Record Drawings | Engineer | Yes |

Checklists for Conceptual/Preliminary and Final drainage reports, and for construction plans, are provided in **Tables 302** and **303**. These checklists contain recommended report outline and contents for all drainage reports. A copy of the completed checklist shall be bound with the Conceptual/Preliminary and Final Drainage report.

The applicant is to identify with a "✔" if information is provided with the appropriate submittal. If applicant believes information is not required, indicate with "n/a". County will review the submittal to determine if information is required and whether information must be submitted. Due to the nature of a conceptual/preliminary report, not all information listed in the outline/checklist may be required for a conceptual/preliminary drainage report, such as those items listed with an asterisk (*). If the applicant is uncertain if information is required, the applicant is encouraged to contact the County.

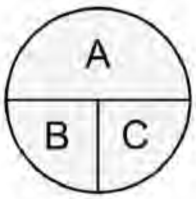
### 305.2 City of Grand Junction

The submittal checklist for the City of Grand Junction follows the same steps and procedures set forth for Mesa County presented in Section 305.1. In addition the designer and reviewer are to refer to the Submittal Standards for Improvements and Development (SSID) for specific requirements regarding checklist requirements.

### 305.3 Other Jurisdictions

Refer to requirements of the City of Fruita, Town of Palisade or the Grand Junction Drainage District regarding checklist requirements.

## DRAWING SYMBOL AND HYDROLOGY SUMMARY TABLE




A = BASIN DESIGNATION

B = AREAS IN ACRES

C = COMPOSITE RUNOFF COEFFICIENTS


D = DESIGN POINT DESIGNATION

**SUMMARY RUNOFF TABLE**
(to be placed on drainage plan)

| DESIGN POINT | CONTRIBUTING AREA (ACRES) | RUNOFF PEAK Minor Storm * (CFS) | Major Storm (CFS) |
|---|---|---|---|
| XX | XX .XX | XX.X | XX.X |

*2-year storm for storm drain design, street capacity
10-year storm for culvert and detention design

| Revision | Date |
|---|---|
| ORIGINAL ISSUE | 3/27/06 |
| MINOR STORM CHANGES | 12/19/07 |

Table 302
Stormwater Management Manual
**Drainage Report Checklist**

**Instructions:**
1. Applicant to identify with a "check-mark" if information is provided with report. If applicant believes information is not required, indicate with "n/a" and attach separate sheet with explanation
2. The reviewer will determine if information labeled "n/a" is required and whether information must be submitted.
3. Those items noted with an "asterisk" are not typically required for conceptual/ preliminary report. Applicant shall confirm this with local jurisdiction.
4. Submit three (3) copies of report and include copy of check list bound with report.

**TITLE PAGE**

A. Type of report (Conceptual/Preliminary or Final Drainage Report).
B. Project Name.
C. Preparer name, firm, address, number, and date.
D. Professional Engineer's seal of preparer.
E. Certifications (see SWMM Section 303.1)

**I. INTRODUCTION**

A. Background
- ________ 1. Identify report preparer and purpose.
- ________ 2. Identify date of letter with previous County comments.

B. Project Location
- ________ 1. Identify Township, Range, and Section.
- ________ 2. Identify adjacent street and subdivision names.
- ________ 3. Reference to General Location Map.

C. Property Description
- ________ 1. Identify area in acres of entire contiguous ownership.
- ________ 2. Describe existing ground cover, vegetation, soils, topography and slopes.
- ________ 3. Describe existing drainage facilities, such as channels, detention areas, or structures.
- ________ 4. Describe existing irrigation facilities, such as ditches, head-gates, or diversions.
- ________ 5. Identify proposed types of land use and encumbrances.

D. Previous Investigations
- ________ 1. Identify drainage master plans that include the project area, including floodplain studies.
- ________ 2. Identify drainage reports for adjacent development.

**II. DRAINAGE SYSTEM DESCRIPTION**

A. Existing Drainage Conditions
- ________ 1. Describe existing topography and provide map with contours extending a minimum of 100 feet beyond property limits.
- ________ 2. Identify major drainageway or outfall drainageway and describe map showing location of proposed development within the drainageways.
- ________ 3. Identify pre-developed drainage patterns and describe map showing pre-developed sub-basins and concentrated discharge locations. Provide calculations of pre-developed peak flows entering and leaving the site.

B. Master Drainage Plan
- ________ 1. Describe location of the project relative to a previously prepared master drainage plan, including drainage plans prepared for adjacent development.

C. Offsite Tributary Area

1. Identify all offsite drainage basins that are tributary to the project.
2. Identify assumptions regarding existing and future land use and effects of offsite detention on peak flows.

D. Proposed Drainage System Description
1. Identify how offsite stormwater is collected and conveyed through the site and ultimately to the receiving water(s).
2. Identify sub-basins and describe, in general terms, how onsite stormwater is collected and conveyed through the site for each location where stormwater is discharged from the site.
3. * Describe detention volumes, release rates and pool elevations.
4. * Identify the difference in elevation between pond invert and the groundwater table.
5. Describe how stormwater is discharged from the site, including both concentrated and dispersed discharges and rates.
6. Describe stormwater quality facilities.
7. * Describe maintenance access aspects of design.
8. * Describe easements and tracts for drainage purposes, including limitation on use.

E. Drainage Facility Maintenance
1. * Identify responsible parties for maintenance of each drainage and water quality facility.
2. * Identify general maintenance activities and schedules.

**III. DRAINAGE ANALYSIS AND DESIGN CRITERIA**

A. Regulations
1. Identify that analysis and design was prepared in accordance with the provisions of the Manual.
2. Identify other regulations or criteria which have been used to prepare analysis and design.

B. Development Criteria
1. Identify drainage constraints placed on the project, such as by a major drainage study, floodplain study or other drainage reports relevant to the project.
2. Identify drainage constraints placed on the project, such as from major street alignments, utilities, existing structures, and other developments.

C. Hydrologic Criteria

(If Manual was followed without deviation, then a statement to that effect is all that is required. Otherwise provide the following information where the criteria used deviates from the Manual.)
1. Identify developed storm runoff peak flows and volumes and how they were determined, including rainfall intensity or design storm.
2. Identify which storm events were used for minor and major flood analysis and design.
3. Identify how and why any other deviations from the Manual occurred.

D. Hydraulic Criteria

(If Manual was followed without deviation, then a statement to that effect is all that is required. Otherwise provide the following information where the criteria used deviates from the Manual.)
1. * Identify type(s) of streets within and adjacent to development and source for allowable street capacity.
2. * Identify which type(s) of storm inlets were analyzed or designed and source for allowable capacity.
3. Identify which type of storm sewers which were analyzed or designed and

__________* Manning's n-values used.

4. Identify which method was used to determine detention volume requirements and how allowable release rates were determined. __________*

5. Identify how the capacity of open channels and culverts were determined. __________*

6. Identify any special analysis or design requirements not contained with the Manual. __________*

7. Identify how and why any other deviations from the Manual occurred. __________

E. Variance from Criteria

1. Identify any provisions of the Manual for which a variance is requested. __________

2. Identify pre-existing conditions which cause the variance request. __________

***IV. POST CONSTRUCTION STORMWATER MANAGEMENT. See Manual Section 1600 for requirements.**

**Note:** This section of the Final Drainage Report identifies additional information required by Mesa County's, City of Grand Junction's, and Town of Palisade's, Permit for Stormwater Discharges Associated with Municipal Separate Storm Sewer Systems (MS4s), permit No. COR-090000. The Final Drainage Plan and the Construction SWMP (see SWMM Section 1500) meets the requirements of the MS4s Permit. In general, this section identifies permanent BMP practices to control the discharge of pollutants after construction is complete.

*A. Stormwater Quality Control Measures

1. Describe the post-construction BMPs to control discharge of pollutants from the project site. __________*

2. If compensating detention is provided, discuss practices to address water quality from area not tributary to detention area. __________*

3. If underground detention is proposed, discuss how water quality facilities will be provided on the surface. __________*

4. If proprietary BMPs are proposed, provide the justification and sizing requirements (see SWMM Section 1603.3). __________

*B. Calculations

1. Provide methods and calculations for WQCV, sediment storage, and water quality outlet structure. __________

**V. CONCLUSIONS**

A. Compliance with Manual

Compliance with Manual and other approved documents, such as drainage plans and floodplain studies. __________

B. Design Effectiveness

Effectiveness of drainage design to control impacts of storm runoff. __________

C. Areas in Flood Hazard Zone

Meet requirements of Floodplain Regulations: Mesa County Land Development Code, Section 7.13; City of Grand Junction Zoning and Development Code, Section 7.1. __________

D. Variances from Manual

Applicant shall identify any requested variances and provide basis for approving variance. If no variances are requested, applicant shall state that none are requested. __________

**VII. REFERENCES**

Provide a reference list of all criteria, master plans, drainage reports, and technical information used. __________

**TABLES**

Include copy of all tables prepared for report. __________

FIGURES

A. General Location Map (See Section 303.2a) __________

________ B. Flood Plain Information
________ C. Drainage Plan (See Section 303.2b)
________ D. Other pertinent figures.

**APPENDICIES**

A. DESIGN CHARTS

________ 1. Provide copy of all design charts (i.e.: tables, figures, charts from other criteria) used for the report.

B. HYDROLOGIC CALCULATIONS (see Manual Sections 600 and 700)

________ 1. Land use assumptions for off-site runoff calculations.
________ 2. Time of concentration and runoff coefficients for pre-existing and post development conditions.
________ 3. Pre-developed hydrologic computations.
________ 4. Developed conditions hydrologic computations.

C. HYDRAULIC CALCULATIONS

________ 1. Capacity of existing channels, streets, storm sewers, inlets, culverts and other facilities.
________ 2. Calculations for existing storm sewer and open channel.
________ 3. Irrigation ditch flows and ditch system capacity.
________* 4. Detention pond design (see Manual, Section 1400 for requirements).
________* a. Storage volume, release rates, and pool elevations for 10-year and 100-year storm.
________* b. Outlet structure dimensions, orifice diameter, weir lengths, pipe headwater and other data.
________* c. Outlet velocity and energy dissipation requirements.
________* d. Routing of outlet flows and emergency spillway flows.
________* 5. Street capacity calculations, if data in Manual not used (see Section 1100).
________* 6. Storm inlet capacity calculations, if data in Manual not used (see Section 1100).
________* 7. Storm sewer capacity calculations, if data in Manual not used (see Section 1000).
________* 8. Channel capacity calculations, if data in Manual not used (see Section 800).
________* 9. Culvert capacity calculations (see Manual, Section 1200).
________* 10. Other hydraulic structure calculations (see Manual, Section 900).

D. STORMWATER QUALITY CALCULATIONS

________ 1. Water Quality Capture Volume (WQCV).
________* 2. Storage volume for sediment volume and pool elevations for WQCV.
________* 3. Outlet calculations for required area per row, diameter of individual holes, number of holes per row, and number of holes per column.

________ **CERTIFICATION – PROFESSIONAL ENGINEER'S SEAL AND SIGNATURE**

**ACKNOWLEDGEMENTS**

Drainage Report checklist was prepared by: ____________________________

Table 303
Stormwater Management Manual
# Drainage Plan Checklist

**Instructions:**
1. Applicant to identify with a "check-mark" if information is provided. If applicant believes information is not required, indicate with "n/a".
2. County will determine if information labeled "n/a" is required and whether information must be submitted.

## I. EXISTING FACILITIES

_________ A. Contours at two foot intervals, based on USGS datum. Contours to extend at least 50 feet past property line.
_________ B. Location and elevation of USGS benchmarks or benchmarks referenced to USGS.
_________ C. Property lines.
_________ D. Drainage easements.
_________ E. Street names.
_________ F. Major and minor channels and floodplains.
_________ G. A historic drainage plan including historic basin boundaries and flow paths.

## II. PROPOSED FACILITIES

_________ A. Contours at two-foot intervals, based on USGS datum.
_________ B. Property lines.
_________ C. Drainage easements.
_________ D. Street names and grades.
_________ E. Right of way and easement.
_________ F. Finished floor elevations for protection from major storm run-off.
_________ G. Detention pond information:
_________ 1. Location of each detention pond with site at 1"=50' scale or larger with 2-foot contour intervals.
_________ 2. Inlet and outlet structure, and trickle channel design details.
_________ 3. Details of emergency spillway and channel.
_________ 4. Landscape information, including side slopes, vegetation and planting requirements.
_________ 5. Details of water quality outlet structure.
_________ H. Channel Information:
_________ 1. Profiles with existing and proposed grades.
_________ 2. Cross sections on 100-foot stations showing existing and proposed topography and required rights of way.
_________ 3. Locations and size of all existing and proposed structures.
_________ 4. Locations and profiles of adjacent utilities.
_________ 5. Typical channel section and lining details.
_________ I. Storm sewer information:
_________ 1. Alignment and location of manholes, inlets, and outlet structures.
_________ 2. Profile of invert and pipe crown.
_________ 3. Invert elevations at manholes and inlets.
_________ 4. Lengths and grades between manholes and inlets.
_________ 5. Locations and elevations of utilities adjacent to and crossing storm sewer.
_________ 6. Easement and other O&M access geometry.
_________ 7. Outlet details, such as end sections, headwall and wingwalls, erosion control, and vegetation.
_________ J. Street cross sections with design 100-year flood depth.
_________ K. Other drainage related structures and facilities, including underdrains and sump pump discharge lines.
_________ L. Other permanent BMP measures to control pollutant discharges to the County's MS4 system.

**III. HYDRAULIC AND HYDROLOGIC INFORMATION**

A. Routing and accumulative runoff peaks at upstream and downstream ends of the site and at various critical points onsite for initial and major storms. Inflow and outflow from each subbasin shall be shown for both initial and major storms.

B. Street cross sections showing 100-year flood levels.

C. Major and minor channels and floodplains.

D. Detention pond data:

1. Release rates for 10- and 100-year storm events.
2. Required and provided volumes for 10- and 100-year storm events.
3. Design depths for 10- and 100-year storm events.
4. Water quality capture volume and pool elevation.

E. Channel data:

1. Water surface profiles.
2. Representative 100-year flow velocity and Froude number.

F. Storm sewer data:

1. Profile of water surface for design flow rate.
2. Peak flows for design flow, 2-year and 100-year storm events.

**IV. STANDARD NOTES**

A. No building, structure, or fill will be placed in the detention areas and no changes or alterations affecting the hydraulic characteristics of the detention areas will be made without the approval of the County.

B. Maintenance and operation of the detention and water quality areas is the responsibility of property owner. If owner fails in this responsibility, the County has the right to enter the property, maintain the detention areas, and be reimbursed for costs incurred.

C. Detention pond volumes, all drainage appurtenances, and basin boundaries shall be verified. As-built drawings shall be prepared by a registered professional engineer prior to issuance of certificate of occupancy for any structure within the development.

D. Permission to reproduce these plans is hereby given to Mesa County for County purposes associated with plan review, approval, permitting, inspection and construction of work.

**V. PROFESSIONAL ENGINEER'S SEAL AND SIGNATURE**

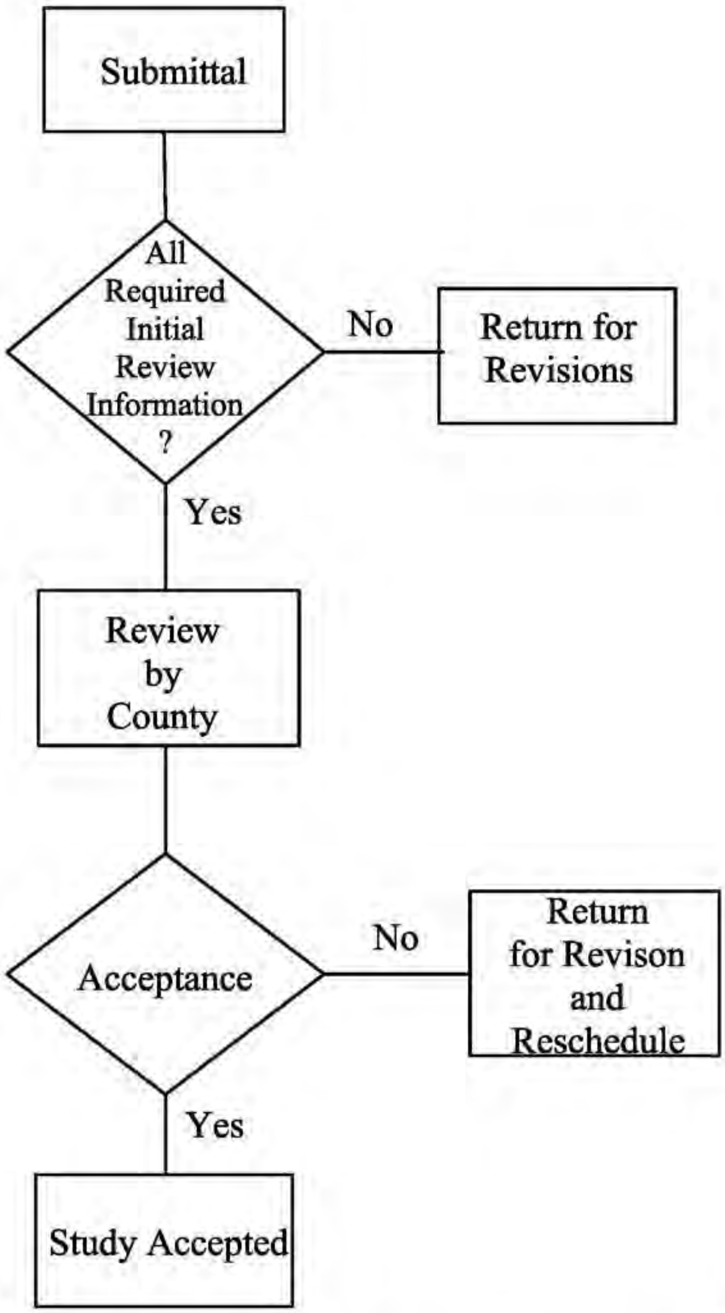
**VI. OTHER**

A. Horizontal and vertical control information and ties to existing and proposed features.

**ACKNOWLEDGEMENTS**

Drainage Plan checklist was prepared by: ____________________________

# SUBMITTAL AND REVIEW FLOW CHART




NOTE: Review time increases if any "NO" is received and the submittal is returned for revisions.

| Revision | Date |
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| ORIGINAL ISSUE | 3/27/06 |
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WRC ENGINEERING INC.

REFERENCE:

FIGURE 301

# SECTION 400

## DRAINAGE POLICY

# SECTION 400
## DRAINAGE POLICY

## TABLE OF CONTENTS

## LIST OF FIGURES

# SECTION 400
# DRAINAGE POLICY

## 401 INTRODUCTION

Presented in this Chapter of the Manual are policies that govern development of specific standards and criteria for the design, evaluation, and construction of drainage facilities. These policies are based on industry standards for stormwater management that have evolved through experience gained. Policy statements are indicated by ***bold italic*** text to distinguish them from background information, which is presented to assist with decisions regarding special circumstances that may arise during development.

## 402 BASIC PRINCIPLES

### 402.1 Jurisdictional Cooperation

Mesa County, the Cities of Grand Junction and Fruita, the Town of Palisade, and the Grand Junction Drainage District have contracted through intergovernmental agreements (IGA) to form the 5-2-1 Drainage Authority (Authority), under Colorado Revised Statutes, specifically CRS 29-1-204.2. The stated purpose of the Authority is to pursue unified stormwater management planning that meets the requirements of the Colorado Water Quality Control Act (25-8-101 *et seq.*, CRS) and the Federal Water Pollution Control Act, as amended (33 U.S.C. 1251 *et seq.*) for the discharge of stormwater associated with municipal separate storm sewer systems.

***The Drainage Authority was created to pursue a jurisdictionally unified effort to promote integrated, comprehensive, regional stormwater management planning, while recognizing local requirements may have minor differences in policies, standards and criteria presented in this Manual.***

### 402.2 Drainage Planning and Required Space

***All new development and redevelopment shall provide storm drainage planning that includes allocation of space for drainage facilities, construction and maintenance, and dedication of rights-of-way and/or easements.***

### 402.3 Multi-Purpose Resource

Stormwater runoff is an integral part of Mesa County's and the City of Grand Junction's water resources and may have potential for beneficial uses, such as groundwater recharge, recreation, and wildlife habitat. These uses, however, must be compatible with adjacent land uses and applicable State Water Laws.

***Stormwater runoff shall be considered an integral part of Mesa County's water resources.***

### 402.4 Water Rights

A drainage design must be planned and constructed with recognition given to adjudicated water rights and applicable water laws.

***The existence of adjudicated water rights and all applicable state laws related thereto shall be recognized in stormwater management plans.***

### 402.5 Drainage Analysis Tributary Area

Stormwater runoff will follow pathways established by physical boundaries and not political or property boundaries. Therefore:

***Any drainage analysis must consider all tributary area and the potential for future land use changes within the tributary area.***

## 403 REGIONAL AND LOCAL PLANNING

### 403.1 Reasonable Use Rule

The "Reasonable Use Rule" is defined for drainage planning purposes as permitting the use of 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.

***Stormwater discharges from new development and redevelopment shall be:***

1. ***Discharged to downstream properties within the pre-developed drainage path and in a manner and quantity and quality that approximates pre-developed conditions. If developed stormwater discharges occur in a more concentrated manner, then additional measures are required to protect downstream properties.***

2. ***Limited to pre-developed rates, unless downstream drainage facilities can accommodate increased flow rates from development and permission/easements are granted.***

### 403.2 Regional Master Planning

***It is the intent of Mesa County and the other cooperating governmental entities to develop, adopt, and use Watershed Master Plans to identify stormwater requirements. Once adopted, these plans will be reviewed and updated regularly. When Watershed Master Plans are not available, then requirements in the Stormwater Management Manual will govern.***

### 403.3 Drainage Improvements

Drainage facilities are categorized as part of either the "major drainage system" or the "outfall system". Recommended public improvements to major drainage and outfall drainage systems are defined in Watershed Master Plans, if they exist. If a Master Plan does not exist, other information will be used to determine the scope of public improvements.

***All new development and redevelopment shall participate in drainage improvements as set forth below:***

**403.3.1 Outfall Drainage System**

1) ***Design and construct that portion of the outfall drainage system, as defined by the approved Final Drainage Report (Section 300 of this Manual).***

2) ***If the outfall system is defined in a Watershed Master Plan, and traverses the development, the developer shall design and construct that portion of the outfall system within the development, in accordance with the Manual.***

3) ***If the outfall system defined in a Watershed Master Plan does not traverse the development, but is required to convey stormwater from the development to the major drainageway, the developer shall design and construct that portion of the outfall system within the development, in accordance with the Manual. The local jurisdiction may participate in the connection of the outfall to the major drainageway at their sole discretion.***

**403.3.2 Major Drainage System**

1) ***If new development encroaches (i.e.: the placement of fill or structures) into a 100-year floodplain (whether mapped or not), the developer will be required to construct improvements as described in the Watershed Master Plan. If a Watershed Master Plan is not available, the developer shall have prepared a channel stabilization analysis, under the guidance of the local jurisdiction, to identify required improvements and shall implement the mitigation plan.***

2) ***Additional improvements to protect health, safety, and welfare may be required by the local jurisdiction if new development is within the vicinity of a 100-year floodplain, whether mapped or not. The developer may be required to participate in a channel stabilization analysis, under the guidance of the local jurisdiction, and may be required to participate in the implementation of the mitigation plan. For the purpose of this policy "vicinity" shall mean any portion of the property that lies within a setback area defined by a slope of 10-foot horizontal to 1 foot vertical (10:1) from the channel invert to the point where the slope daylights.***

### 403.4 Drainage Fee Calculation

Fees may be collected by local jurisdiction and used by local jurisdiction solely for the planning, design and construction of drainage improvements to the outfall drainage system and major drainage systems.

When drainage fees are assessed for new development and redevelopment, the drainage fee will be determined based on the following equation:

$$\text{Drainage Fee} = B(C_D - C_H)A^{0.7} \qquad (401)$$

where: $B$ = Fee Constant (US Dollars, see local jurisdiction for value)
$C_D$ = 100-year runoff coefficient (expressed as a decimal) based on developed land use conditions
$C_H$ = 100-year runoff coefficient (expressed as a decimal) based on pre-developed land use conditions
$A$ = Area of Development (acres)

### 403.5 Proposed Drainage Improvements

***All investigations, reports, and construction plans prepared for drainage improvements shall be submitted for review and acceptance before construction of said improvements, and shall be consistent with this Stormwater Management Manual and Watershed Master Plans.***

### 403.6 Floodplain Management

***Floodplains within Mesa County, Grand Junction, Fruita and Palisade shall be regulated in accordance with the provisions of the applicable Land Development / Use Code.***

### 403.7 Storm Runoff Detention

Since urban development can increase the rate, volume, duration, and frequency of stormwater runoff, measures must be implemented to avoid harm to downstream properties. Detention is considered a viable method to reduce development impacts and drainage costs. Temporarily detaining storm runoff can significantly reduce downstream flood hazards as well as reduce pipe and channel sizes in developed areas. Temporary storage also provides for sediment and debris collection which helps to maintain water quality in downstream channels and streams. However, detention may not be necessary where downstream drainage facilities have adequate capacity to convey runoff from fully developed upstream areas without negatively impacting downstream properties.

#### 403.7.1 Detention Requirements

***1) On-site detention storage is required for all new development or redevelopment to limit 10-year peak stormwater discharges to the 10-year storm runoff based on pre-developed conditions. In addition to the 10-year detention, detention of the 100-year storm event and release at pre-developed rates will be required, if the***

***capacity of the downstream drainage system will be exceeded. With local jurisdiction approval, detention may be considered for off-site flows draining to a site. See Section 1400 for details.***

***2) A fee-in-lieu of detention may be required in lieu of on-site detention that is not required (see Section 1404). Granting fee-in-lieu of detention does not exempt the development from providing post-construction BMPs.***

**403.7.2 Detention Exemption**

***At the sole discretion of the local jurisdiction, exemptions to on-site detention may be granted for the following conditions. Note that in all cases, post-construction BMPs are required*:**

1. A development which discharges to a regional drainage facility is exempt provided the regional facility is completed in accordance with a Watershed Master Plan, the regional facility was designed to include runoff from the proposed development, and there is adequate conveyance capacity for the drainage system from the development to the regional facility.

2. A development lying within the limits of a Watershed Master Plan which explicitly exempts on-site detention for development.

3. A development which discharges to a local detention facility is exempt provided the local detention is completed in accordance with the Final Drainage Plan for the development and was designed to include runoff from the proposed development.

4. A development which discharges to an outfall drainage system that has adequate conveyance capacity for the 100-year flood from a fully developed watershed.

5. When it can be demonstrated that the peak flows from the development will not increase the peak flows from the watershed for storm events up to the 100-year flood. The burden of proof is on the developer to demonstrate this condition.

***A fee-in-lieu of detention may be required in-lieu of on-site detention that is not required.***

## 403.8 Storm Runoff Retention

Storm runoff retention or over-detention may be used when downstream drainage facilities lack adequate conveyance capacity or are essentially nonexistent and construction of an outfall drainage system is impractical.

***Retention or over-detention may be used in those instances where there are severe limitations on the downstream conveyance capacity or where there is essentially no outfall or drainage system to convey storm runoff from the development. The acceptability of retention will be determined on a case-by-case basis.***

### 403.9 Stormwater Quality

Studies by the Environmental Protection Agency (EPA) and others have shown that land disturbances due to construction activities and the resulting development decrease the quality of storm runoff. The CDPS stormwater discharge permit requires that construction activities and new development be controlled to minimize the discharge of pollutants to the maximum extent practicable. As such, it is recognized that construction sediment and erosion control, stream stabilization, and permanent best management practices (BMPs) are necessary to protect the quality of the waters of the state.

***All significant development and redevelopment disturbing more than 1-acre within the urban areas of Mesa County shown on Figure 401 shall implement:***

1. ***Sediment and erosion control measures during construction activities,***

2. ***Stream stabilization measures for the major drainageways,***

3. ***Post-construction best management practices to control the discharge of pollutants to the municipal separate stormwater system (MS4).***

### 403.10 Drainage Facilities Maintenance

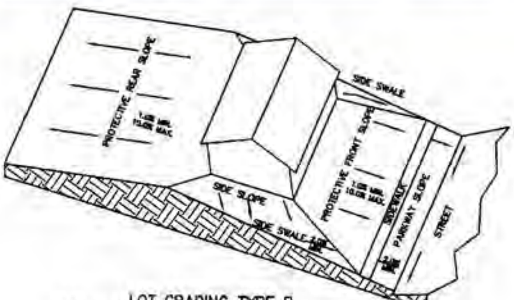
An important part of all storm drainage facilities is continued maintenance of the facilities to insure they will function as designed. Maintenance of drainage facilities includes a number of routine tasks, such as removal of debris and sediment, and non-routine tasks, such as restoring damaged structures.

***All drainage facilities will be maintained to preserve their function, and shall:***

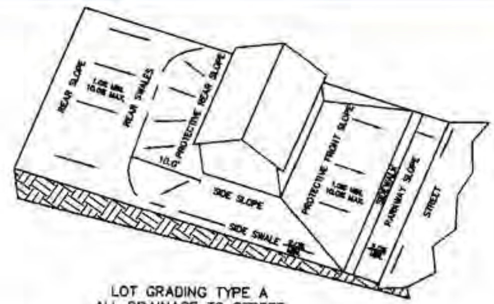
1. ***Be designed to minimize and facilitate maintenance.***

2. ***Include access to the entire drainage facility by dedication of rights-of-way, easements and tracts of land specifically for drainage purposes. Tracts or easement dedications shall prohibit uses and the construction of permanent improvements that restrict or block access. Specifically, detention and retention basins in subdivisions shall be located in tracts owned by the property owners' association with an easement granted to the local jurisdiction. Basins for individual sites require neither a tract nor an easement but do require a maintenance agreement.***

3. ***Be incorporated in the lot grading for residential development in conformance with FHA lot grading TYPE-A (all drainage to street) or TYPE-B (drainage to front and rear lot line). Typical FHA three dimension TYPE-A and TYPE-B grading plans are shown below:***




LOT GRADING TYPE B
DRAINAGE TO FRONT & REAR LOT LINE




LOT GRADING TYPE A
ALL DRAINAGE TO STREET

***4. Be maintained by the property owner, the developer and/or a home owners association. Should the property owner fail to adequately maintain drainage facilities, the right is reserved to enter the property, upon proper notice, for the purpose of performing drainage maintenance. All maintenance costs shall be assessed against the owner(s) of the property.***

***5. Include v-pans (2' minimum width) or storm sewer systems with an inlet in each yard when the grading plan creates backyard swales (regardless of swale slope). These must be installed prior to curb and gutter installation. Check with local municipality to confirm this requirement.***

### 403.11 Watershed Transfer of Storm Runoff

Drainage law recognizes the inequity of transferring the burden of managing storm drainage from one location or property to another. Liability questions also arise when the historic drainage continuum is altered. The diversion of storm runoff from one basin to another shall be avoided unless specific and prudent reasons justify and dictate such a transfer.

Stormwater diversion may be investigated during development of Watershed Master Plans and may be shown, in some instances, to be a viable option to address drainage and flooding problems. Tests for reasonableness of the potential transfer include:

(1) Determine where the runoff would go if the diversion system were to fail. If the flow stays in the "historic" basin, then the diversion may be reasonable.

(2) Determine if the downstream channel to which the drainage flows are being transferred is adequate to handle increased flows. If the channel has capacity, the diversion may be reasonable.

(3) Check condition where diversion fails during the major flood. If the downstream natural channel would have adequate capacity without adverse effects, the diversion may be reasonable.

(4) Determine if there would be any adverse water quality impact in either drainageway. If there is little or no impact, the diversion may be reasonable.

The development process can and will significantly alter the historic or natural drainage paths. When these alterations result in development outfall system that discharges back into the natural drainageway at or near the historic location, then the alterations (i.e. intra-basin transfer) are generally acceptable. However, when the subdivision outfall system does not return to the historic drainageway, an inter-basin transfer may result.

***Inter-basin transfer as a drainage solution for new development is prohibited, unless allowed as a deviation/variance to the Manual (see Section 200) or as part of the Watershed Master Plan. During development of Watershed Master Plans where retrofit facilities are evaluated, inter-basin transfer may be a viable alternative in certain instances and will be reviewed on a case-by-case basis.***

## 404 TECHNICAL CRITERIA

### 404.1 Stormwater Management Technology

***All storm drainage facilities shall be planned and designed in accordance with this Manual. Criteria in the Manual may be revised or amended as new technology is developed and/or experienced gained in the use of these criteria. Exemptions to criteria and standards may be approved on a case-by-case basis.***

***Construction of any drainage facility not initiated within a two-year period from the time of construction plan approval must be reevaluated for approval and must receive re-approval.***

### 404.2 Design Storm Events

***All new development and redevelopment shall plan for, design and construct drainage facilities to convey storm runoff from the minor storm and major storm events. Emergency overflow paths for drainage may be required for storms greater than the 100-year flood event as evaluated on a case-by-case basis.***

***Analysis and design of all drainage improvements for new development and significant redevelopment shall be based on projected future land-use conditions, in accordance with local jurisdictional requirements and plans, except as may be modified by adopted Watershed Master Plans.***

***Culverts for local categorized streets may be designed for a flood more frequent than the 100-year), subject to the following conditions:***

1. ***The overtopping of the roadway during the 100-year event is limited to depths no greater than 1-foot and the downstream embankment is protected from erosion during overtopping.***

2. ***The culvert must pass the 10-year flood with a minimum of 1-foot of freeboard from the water surface to the lowest point in the roadway.***

3. ***The water surface elevation during the 100-year flood shall not cause additional flooding of adjacent properties and shall be compliant with local floodplain regulations.***

***All drainage systems shall be sized without accounting for peak flow reductions from on-site or local detention unless noted otherwise in the tables below. When required to size best management practices, the mean annual storm event shall be used.***

***A summary of requirements and design storms for the different types of stormwater facilities is presented in the following table.***

| STREETS | |
|---|---|
| Minor Storm | Design Requirements |
| 2-year | Flow depth ≤ 0.5 feet at gutter flow line |
| | Flow depth ≤ top of curb |
| | Flow velocity ≤ 8.0 feet per second |
| Major Storm | Design Requirements |
| 100-year | Flow depth ≤ 1.0 feet at gutter flow line |
| | Flow velocity ≤ 8.0 feet per second |
| | Freeboard ≥ 1.0 feet at all finished floor elevations |

| STORM DRAIN SYSTEMS | |
|---|---|
| Minor Storm | Design Requirements |
| 2-year* | Flow velocity between 2.5 and 15 feet per second |
| | EGL must remain below manhole rims and inlet throats |
| | |
| Major Storm | Design Requirements |
| 100-year* | Flow velocity between 2.5 and 15 feet per second |
| | EGL must remain below manhole rims and inlet throats |

****These flow rates will be whatever the street inlets cumulatively accept during the 2-year and 100-year storm events.***

| CULVERTS | |
|---|---|
| Minor Storm | Design Requirements for Roadways with ROW ≤ 80 feet |
| 10-year † | Freeboard ≥ 1.0 feet at lowest point of drive lane(s) |
| | Approval of local jurisdiction to use minor storm design |
| | |
| Major Storm | Design Requirements for Roadways with ROW ≤ 80 feet |
| 100-year † | Overtopping ≤ 1.0 feet at crown of roadway |
| | No additional flooding of adjacent properties |
| | Comply with all floodplain regulations |
| | Freeboard ≥ 1.0 feet at all finished floor elevations |
| | Design Requirements for Roadways with ROW ≥ 80 feet |
| | Flow depth ≤ 0.0 feet at crown of roadway |
| | Freeboard ≥ 1.0 feet at all finished floor elevations |

***†Design of culverts for may consider the effects of detention.***

| ***DETENTION/WATER QUALITY*** | |
|---|---|
| ***WATER QUALITY*** | ***Design Requirements*** |
| ***WQCV*** | ***Design is per Volume 3 of the USDCM (see Appendix B)*** |
| | |
| ***DETENTION*** | |
| ***Minor Storm*** | ***Design Requirements*** |
| ***10-year*** | ***Design volume is the detention volume plus the WQCV*** |
| | ***Allowable release rates apply*** |
| | |
| ***Major Storm*** | ***Design Requirements*** |
| ***100-year*** | ***Design volume is the detention volume (do not add WQCV)*** |
| | ***Allowable release rates apply*** |

### 404.3 Storm Runoff Determination

***Storm runoff peaks and volumes for the design and evaluation of storm drainage facilities shall be determined using statistical analysis of recorded stream gage data, the Rational Method, or synthetic rainfall runoff models.***

### 404.4 Streets

***Streets and street rights-of-way may be used to convey storm runoff from the minor and major events, subject to limitations set forth in this Manual.***

***Storm runoff is permitted to cross street crowns at intersections, subject to limitations set forth in this Manual.***

***Storm runoff is permitted to cross street crowns at culverts and bridges, subject to limitations set forth in this Manual.***

### 404.5 Flood Proofing

Flood proofing can be defined as those measures which reduce the potential for flood damages to properties within a floodplain. Measures can range from elevating structures to intentional flooding of non-critical building spaces (i.e., basement) to minimize structural damages.

***Flood proofing is permitted in accordance with the provisions of the Mesa County Land Development Code. For other jurisdictions, contact the specific jurisdiction for specific guidance.***

### 404.6 Alluvial Fans

Alluvial fans, consisting of sand and fine sediment, are subject to radical changes in shape, direction, depth, and flow carrying capacity during storm events. These changes increase the potential flood hazards of developing on alluvial fan areas.

***Additional analysis and specialized design is required for any development located within an active alluvial fan. Requirements for analysis and design will be determined on a case-by-case basis.***

## 405 IRRIGATION FACILITIES

Irrigation conveyance facilities and reservoirs in the Mesa County area have historically intercepted the storm runoff from the rural and agricultural watersheds, generally without major problems. With urbanization of the watersheds, however, the storm runoff has increased in rate, quantity and frequency, as well as changing water quality. In developed areas, the irrigation facilities can no longer be utilized indiscriminately to convey storm runoff. New developments and redevelopment are prohibited from using irrigation facilities to convey storm runoff, except as described below.

### 405.1 Watershed Analysis

***Drainage analysis and design shall be based on the assumption that an irrigation conveyance facility does not intercept storm runoff from the upper watershed and that the upper watershed is tributary to the watershed downstream of the irrigation conveyance facility, unless such irrigation conveyance facility has been designed to accommodate storm runoff and is part of an adopted Watershed Master Plan.***

### 405.2 Discharge to Irrigation Conveyance Facility

***Storm runoff shall be directed into pre-developed drainageways downstream of the irrigation conveyance facility. Storm runoff shall not be discharged into an irrigation conveyance facility, except as required by water rights law or the discharge is part of an adopted Watershed Master Plan or negotiated with an irrigation company.***

***Where irrigation ditches will serve as the outfall for a detention facility, the ditch water surface elevation shall be determined for the maximum irrigation flow of the ditch, and the storm water surface elevation shall be determined for the combination of the maximum irrigation flow and the 100-year storm water discharge of the detention facility.***

### 405.3 Irrigation Reservoirs

***New development is restricted to areas outside of:***

1. ***The reservoir's high water line created by the design flood for the emergency spillway.***

2. ***The high water line created by the breach of a dam (except high hazard classified dams which have passed inspection by the state engineer's office in accordance with 37-87-105 et seq. CRS). For dams without the breach high waterline identified, the developer shall analyze and define the high waterline resulting from such a breach.***

3. ***Existing or potential future emergency spillway paths, beginning at the dam and proceeding to the point where the flood water returns to the natural drainage course.***

## 406 PRESERVATION OF NATURAL DRAINAGEWAYS

Natural drainageways are considered an important environmental feature that contributes to the image and livability of a community. Their value includes the ability to convey floodwater, to provide opportunities for trails and open space corridors, and to maintain natural vegetation and wildlife habitat to the greatest degree possible.

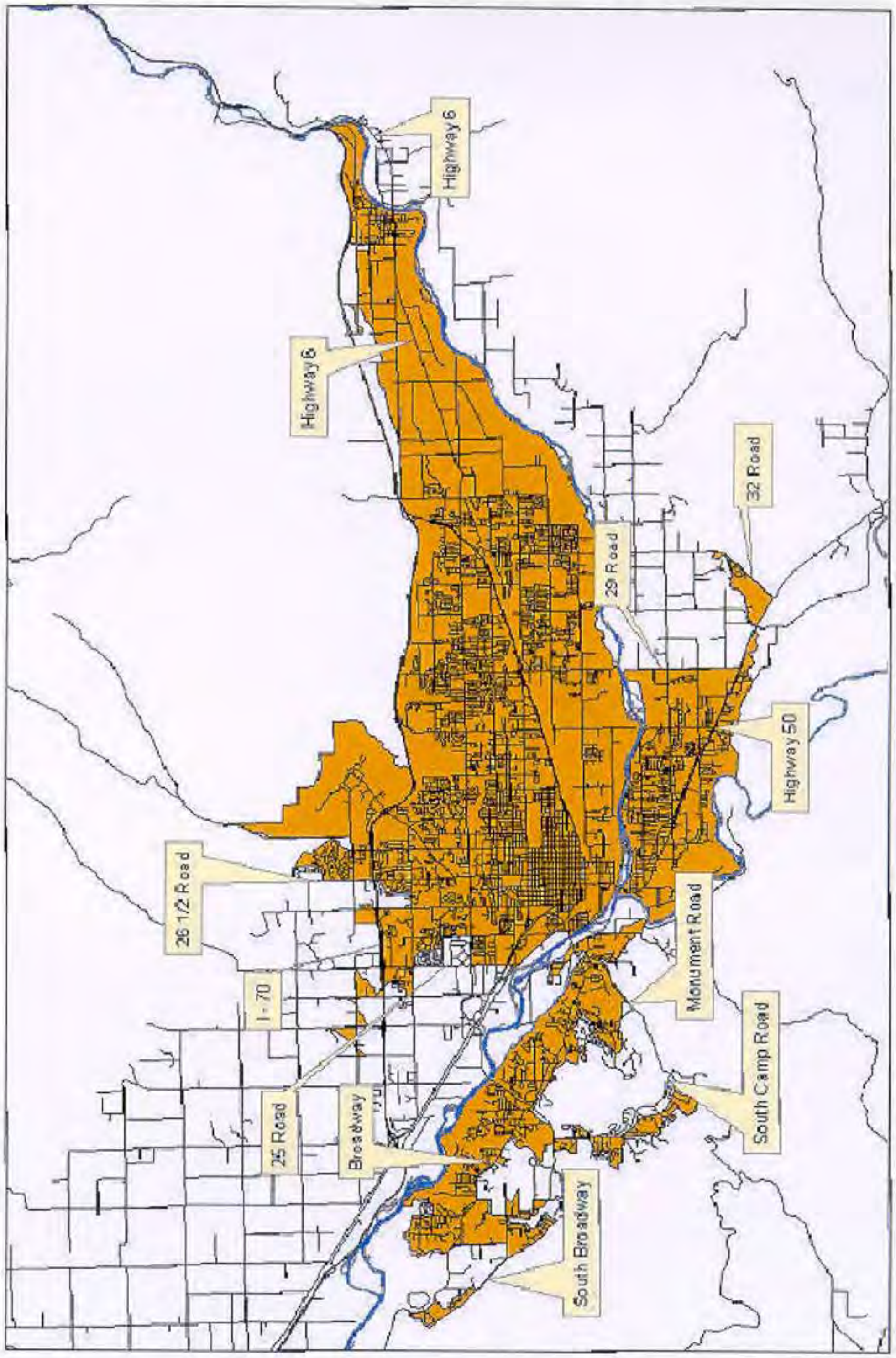
***Preservation of natural drainageways, based on developed land-use hydrology, is encouraged. Development of property shall not adversely affect any natural drainage facility or natural watercourse, and shall be subject to the following provisions:***

1. ***Drainageways shall remain in as near a natural state as is practicable. All proposed modification to the natural drainageway shall be subject to approval.***

2. ***When the flow rates, velocities, side slopes or other characteristics indicate a potential negative impact to the natural drainageway, the impact shall be mitigated in accordance with criteria set forth in this Manual.***

STORMWATER MANAGEMENT MANUAL

# MESA COUNTY URBANIZED AREA




NOTES:

1. PHASE II STORMWATER REGULATIONS WILL BE IMPLEMENTED WITHIN THE URBANIZED AREA.
2. URBANIZED BOUNDARY INCLUDES ALL OF THE CITY OF GRAND JUNCTION.

| Revision | Date |
|---|---|
| ORIGINAL ISSUE | 3/27/06 |
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WRC ENGINEERING, INC

REFERENCE:

**FIGURE 401**

# SECTION 500

## RESERVED

# SECTION 500

## RESERVED

# SECTION 600

## RAINFALL

# SECTION 600
## RAINFALL

## TABLE OF CONTENTS



## LIST OF TABLES



## LIST OF FIGURES

# SECTION 600
# RAINFALL

## 601 INTRODUCTION

Presented in this Section are design rainfall data for the mean annual, minor, and major storm events. These data are used to determine storm runoff peak flows and volumes in conjunction with the runoff models described in Section 700. All hydrologic analyses within the jurisdiction of this Manual shall utilize the rainfall data presented herein for calculating storm runoff.

### 601.1 Data Sources

Rainfall data is presented for three areas within Mesa County, the Grand Valley, as approximately defined by the area below 5,000 feet in elevation, area outside the Grand Valley, and the Leach Creek/Horizon Drive watersheds.

**Rainfall data - Grand Valley Area**. The rainfall data for the Grand Valley was obtained from the Henz Meteorological Services report prepared for Mesa County (Henz, 1992), which included a detailed analysis of the gage at Walker Field. Tables, figures, and equations are provided for point rainfall, intensity, and storm distributions for various storm recurrence intervals.

**Rainfall data - Outside Grand Valley**. For the area outside of the Grand Valley, 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 III - Colorado*" (NOAA, 1973) is recommended to develop point rainfall values, storm intensities, and distributions for the remainder of Mesa County. Procedures to obtain and interpret the data are provided herein.

**Rainfall Data Both Areas**. In the case where a watershed includes both areas (or areas outside of Mesa County), it is recommended that both sets of rainfall data be used to generate peak runoff rates and volumes, depending on which area the sub-watershed lies within. In addition, if point rainfall values from NOAA Atlas 2 vary throughout the watershed by more than 10%, the rainfall values used for analysis should also vary depending on where the sub-watershed lies.

**Rainfall Data - Leach Creek/Horizon Drive Watershed**. For projects within the Leach Creek/Horizon Drive watershed (see **Figure 617**), the depth-duration-frequency data approved by FEMA (FEMA October 1, 2002) are recommended because these data were used for floodplain analysis and for design purposes. However, since the FEMA-approved data did not include all durations and frequencies, and since the Henz data are similar, the Henz data was used to supplement the FEMA data.

**Mean Annual Precipitation**. Mean annual precipitation and mean annual storm events for design of water quality BMPs were obtained from Driscoll (Driscoll, et. al. November 1989. *Analysis of Storm Event Characteristics for Selected Rainfall Gages Throughout the United States,* EPA 8919148B 1100). *These values are based on recording gage number 3488 and can be used throughout Mesa County.*

**Probable Maximum Precipitation**. 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) is recommended.

### 601.2 Comparison to NOAA Atlas 14.

The National Weather Service (NWS) has updated the precipitation-frequency atlas for the semiarid, southwestern region of the United States (including the Mesa County area) from NOAA Atlas 2 to NOAA Atlas 14. A comparison was made for the 100- and 2-year storms for durations of 6- and 24-hour between NOAA Atlas 2 and NOAA Atlas 14.

The comparison shows that NOAA Atlas 14 is generally lower than NOAA Atlas 2 (i.e.: from 4 to 16% in the Grand Valley and from 0 to 46% in the mountains). NOAA Atlas 14 does appear to be more consistent with the data developed by Henz for the Grand Valley area. However, since NOAA Atlas 14 has not been accepted by the Colorado Water Conservation Board, precipitation values for areas outside of the Grand Valley were not obtained from NOAA Atlas 14, but were obtained from NOAA Atlas 2. When preparing Watershed Master Plans, the County will evaluate whether NOAA Atlas 14 values are more appropriate and may decide to use NOAA Atlas 14. For new developments and redevelopment, rainfall / runoff relationships shall be in accordance with Section 601.1 of this Manual.

## 602 RAINFALL ANALYSIS

### 602.1 Rainfall Depth - Duration - Frequency

For areas within the Grand Valley, point precipitation values are provided in **Table 601** for various recurrence interval storms. These data were used to develop intensity-duration-frequency values and are used in developing storm distributions.

**Table 601 - Point Rainfall Values for the Grand Valley Area**

| Storm Duration | Precipitation Depth (inches) | | | | | |
|---|---|---|---|---|---|---|
| | 2-year Recurrence | 5-year Recurrence | 10-year Recurrence | 25-year Recurrence | 50-year Recurrence | 100-year Recurrence |
| 5-min | 0.10 | 0.14 | 0.18 | 0.25 | 0.31 | 0.39 |
| 10-min | 0.15 | 0.22 | 0.28 | 0.38 | 0.48 | 0.60 |
| 15-min | 0.19 | 0.28 | 0.36 | 0.48 | 0.61 | 0.76 |
| 30-min | 0.27 | 0.39 | 0.50 | 0.67 | 0.85 | 1.06 |
| 1-hr | 0.34 | 0.49 | 0.63 | 0.85 | 1.07 | 1.34 |
| 2-hr | 0.42 | 0.58 | 0.72 | 0.94 | 1.15 | 1.40 |
| 3-hr | 0.47 | 0.63 | 0.77 | 0.99 | 1.19 | 1.44 |
| 6-hr | 0.55 | 0.73 | 0.87 | 1.10 | 1.31 | 1.56 |
| 12-hr | 0.55 | 0.83 | 0.98 | 1.22 | 1.44 | 1.69 |
| 24-hr | 0.70 | 0.93 | 1.12 | 1.42 | 1.69 | 2.01 |

For the area outside of the Grand Valley, the NOAA Atlas 2 Rainfall Depth-Duration-Frequency Maps are reproduced for the Mesa County area in **Figures 602 through 613**. Maps are presented for the 6- and 24-hour durations for the 2-, 5-, 10-, 25-, 50-, and 100-year recurrence intervals. Depending on the location of the project, point rainfall values are read from these figures, converted to other storm durations, and used to generate intensity-duration-frequency curves and design storms, following procedures described below.

For the areas within the Leach Creek/Horizon Drive watershed, point precipitation values are provided in **Table 602** for various frequencies and durations.

**Table 602 - Point Rainfall Values for the Leach Creek/Horizon Drive Watershed**

| Storm Duration | Precipitation Depth (inches) | | | | | |
|---|---|---|---|---|---|---|
| | 2-year Recurrence | 5-year Recurrence | 10-year Recurrence | 25-year Recurrence | 50-year Recurrence | 100-year Recurrence |
| 5-min | 0.10 | 0.14 | 0.18 | 0.25 | 0.31 | 0.39 |
| 10-min | 0.15 | 0.22 | 0.28 | 0.38 | 0.48 | 0.60 |
| 15-min | 0.19 | 0.28 | 0.36 | 0.48 | 0.61 | 0.76 |
| 30-min | 0.27 | 0.39 | 0.50 | 0.67 | 0.85 | 1.06 |
| 1-hr | ***0.29*** | ***0.41*** | ***0.52*** | 0.85 | ***0.86*** | ***1.07*** |
| 2-hr | 0.42 | 0.58 | 0.72 | 0.94 | 1.15 | 1.40 |
| 3-hr | ***0.46*** | ***0.63*** | ***0.78*** | 0.99 | ***1.24*** | ***1.52*** |
| 6-hr | ***0.55*** | ***0.72*** | ***0.86*** | 1.10 | ***1.28*** | ***1.52*** |
| 12-hr | ***0.67*** | ***0.85*** | ***0.99*** | 1.22 | ***1.39*** | ***1.61*** |
| 24-hr | ***0.79*** | ***1.01*** | ***1.19*** | 1.42 | ***1.71*** | ***1.99*** |

NOTE: The ***italicized*** values are the actual FEMA-approved data. The remaining data was obtained from **Table 601**.

### 602.2 Rainfall Depths for Durations from 1- to 6-Hours and Less Than 1 Hour

For areas within Grand Valley, point rainfall values for different storm durations are obtained from **Table 601** above.

For areas outside the Grand Valley, the rainfall values are determined using procedures recommended in NOAA Atlas 2 and summarized below:

**Step 1**. Read and record the point rainfall values for the 6- and the 24-hour durations obtained from **Figures 602 to 613** for the 2- and 100-year recurrence intervals at the desired location(s) within Mesa County.

**Step 2**. Calculate the 2-year, 1-hour and 100-year, 1-hour rainfall amounts using the following equations:

$$Y2 = 0.011 + 0.942 \, [ \, (x1) \, (x1 \, / \, x2) \, ] \qquad (601)$$
$$Y100 = 0.494 + 0.755 \, [ \, (x3) \, (x3 \, / \, x4) \, ] \qquad (602)$$

Where:
- Y2 = 2-yr, 1-hr estimated value (in)
- Y100 = 100-yr, 1-hr estimated value (in)
- x1 = 2-yr, 6-hr value (in)
- x2 = 2-yr, 24-hr value (in)
- x3 = 100-yr, 6-hr value from (in)
- x4 = 100-yr, 24-hr value from (in)

**Step 3**. Plot the 2- and 100-year, 1-hour rainfall (Y2 and Y100) values on **Figure 601** and draw a straight line connecting these points. The 1-hour point rainfall values for the 5-, 10-, 25-, and 50-year storm events are then read from the graph.

**Step 4**. Calculate the 2- and 3-hour duration rainfall for the various recurrence intervals using the following equations:

$$P2\text{-}hr = 0.341 \, P6\text{-}hr + 0.659 \, P1\text{-}hr \qquad (603)$$
$$P3\text{-}hr = 0.569 \, P6\text{-}hr + 0.431 \, P1\text{-}hr \qquad (604)$$

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

**Step 5**. Calculate the point rainfall values for durations less than 1-hour by multiplying the 1-hour precipitation depths by the factors listed in **Table 603**:

**Table 603 Factors for Durations of Less than One Hour**

| Duration (min) | 5 | 10 | 15 | 30 | 60 |
|---|---|---|---|---|---|
| Ratio to 1-hour | 0.29 | 0.45 | 0.57 | 0.79 | 1.00 |

The results from the steps above will result in a table of point rainfall values for the project area, similar to **Table 601**. These data can then be adjusted for the size of the watershed if needed (see Section 603) and used to develop Intensity-Duration-Frequency curves (see Section 604) design storm (see Section 605).

For areas within the Leach Creek/Horizon Drive watershed, point rainfall values for different storm durations are obtained from **Table 602** above.

### 602.3 Mean Annual Precipitation and Storm Events

For analysis and design of water quality BMPs, the mean annual precipitation and storm events are used, which are summarized below:

**Table 604 - Mean Storm Statistics**

| | Average | Coefficient of Variation |
|---|---|---|
| Annual Statistics | | |
| No. of Storms | 25 | 0.27 |
| Precipitation (in/year) | 6.76 | 0.35 |
| Independent Storm Events | | |
| Duration (hr) | 9.4 | 0.73 |
| Intensity (in/hr) | 0.044 | 1.15 |
| Volume (in) | 0.28 | 0.70 |
| Interval (hr) | 370 | 1.21 |

Note that the coefficient of variation is defined as the ratio of the standard deviation to the mean.

## 603 DEPTH-AREA REDUCTION FACTORS

The area adjustment to the point rainfall values described below applies to both the Grand Valley area and areas outside of the Grand Valley. In general, watersheds of less than 10 square miles do not require adjustment.

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 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.

The depth-area reduction factors used for rainfall analysis in the Mesa County area are provided in **Figure 614**.

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 with the Director to determine the appropriate method of analysis and design rainfall area reduction factors for the specific location and basin under study.

## 604 INTENSITY-DURATION-FREQUENCY CURVES FOR RATIONAL METHOD

For areas within the Grand Valley Area, rainfall intensities as a function of storm duration and recurrence interval are provided in **Table 605** and **Figure 616**. These data were derived from the investigation by Henz Meteorological Services (Henz 1992).

**Table 605 Intensity-Duration-Frequency Data for Grand Valley**

| Storm Duration | Precipitation Intensity – Grand Valley Area (inches/hour) | | | | | |
|---|---|---|---|---|---|---|
| | 2-year Recurrence | 5-year Recurrence | 10-year Recurrence | 25-year Recurrence | 50-year Recurrence | 100-year Recurrence |
| 5 | 1.20 | 1.68 | 2.16 | 3.00 | 3.72 | 4.68 |
| 10 | 0.90 | 1.32 | 1.68 | 2.28 | 2.88 | 3.60 |
| 15 | 0.76 | 1.12 | 1.44 | 1.92 | 2.44 | 3.04 |
| 30 | 0.54 | 0.78 | 1.00 | 1.34 | 1.70 | 2.12 |
| 60 | 0.34 | 0.49 | 0.63 | 0.85 | 1.07 | 1.34 |

Intensity values for any time of concentration up to 60 minutes can also be determined from the following equation (Froehlich 1995)

$$I_x = a^*P_1/(10+Tc)^b$$

Where:

$I_x$ = Intensity for recurrence interval "x", in/hour
$P_1$ = Point rainfall value for 1-hour duration (see **Table 601**) (inches)
Tc = Time of concentration, minutes
a, b = coefficients, a = 28.9, b = 0.786 for all of Mesa County

Using the coefficients above, this equation will generally match the values in **Table 601** to within 2%. Whereas the coefficients have not been verified for areas outside the Grand Valley, since the equation is based on the same ratios provided in **Table 602**, the results will be comparable.

For areas outside of the Grand Valley, first determine the 2- and 24-hour point rainfall values for the desired storm recurrence intervals, then determine the 1-hour point rainfall value, as described in section 602.2. Finally, use the above equation to compute the intensity at any time of concentration and for all storm recurrence intervals.

For areas within the Leach Creek/Horizon Drive watershed, determine the 2- and 24-hour point rainfall from **Table 602**, and then follow the procedures described above.

## 605 DESIGN STORMS

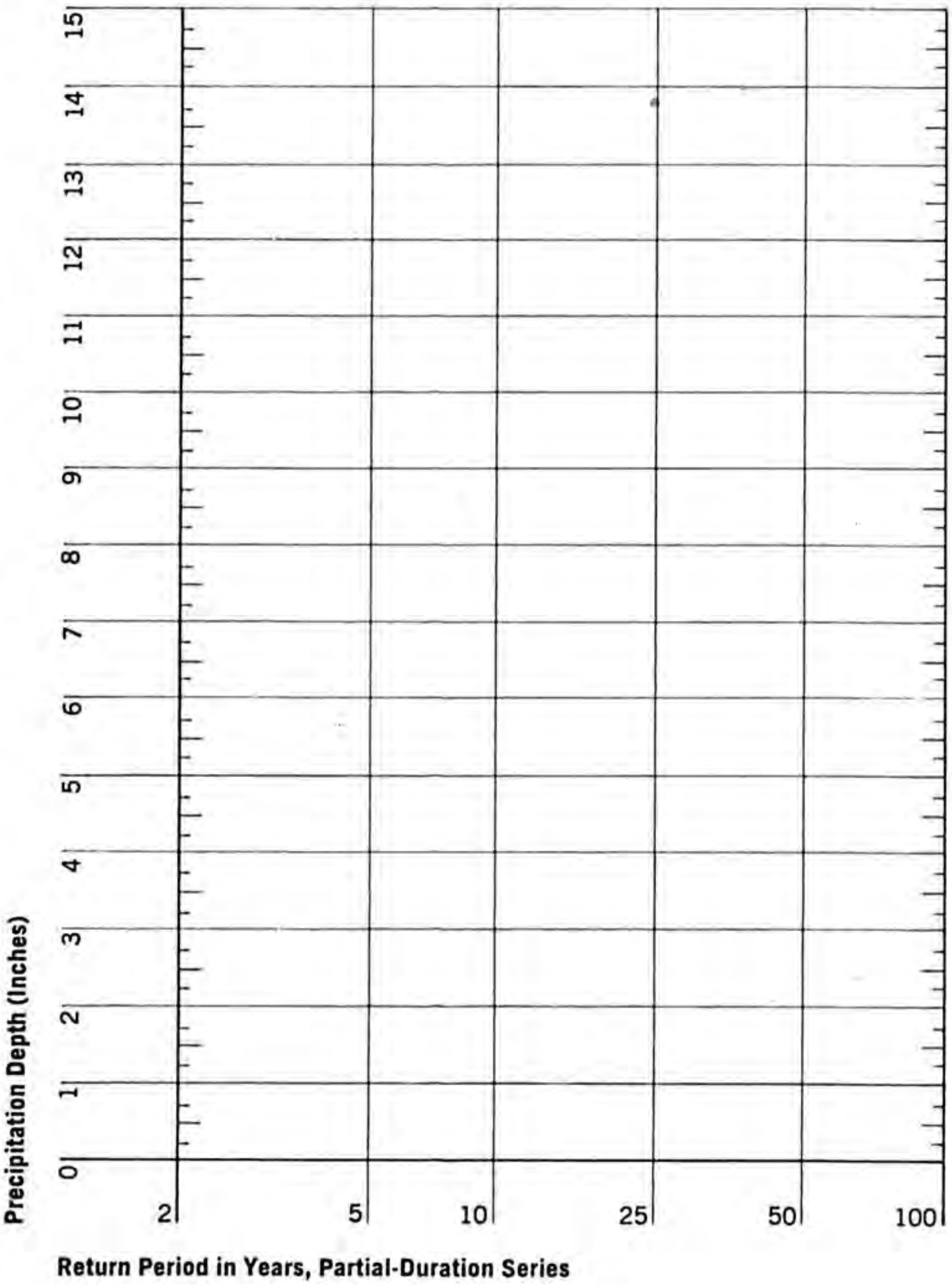
For hydrograph analysis, the recommended minimum design storm duration is 3-hour (Henz 1992). Approximately 90% of the 3-hour total rainfall is distributed within the second hour of the storm, in accordance with recommendations of Henz (Henz 1992). The remaining 10% of the storm total is divided equally between the first and third hour.

An example of this distribution is provided in **Table 606** and on **Figure 615**, which also compares the recommended design storm to the NRCS 2- and 6-hour distributions. As can be seen from the figure, the recommended storm is more severe in that 90% of the total precipitation occurs in a 1-hour period compared to 80% for the NRCS 2-hour and a little more than 50% for the NRCS 6-hour event.

**Table 606 Design Rainfall Distribution**

| Duration (minutes) | Cumulative Proportion of 3-Hour Precipitation | Duration (minutes) | Cumulative Proportion of 3-Hour Precipitation | Duration (minutes) | Cumulative Proportion of 3-Hour Precipitation |
|---|---|---|---|---|---|
| 5 | 0.004 | 65 | 0.089 | 125 | 0.960 |
| 10 | 0.007 | 70 | 0.199 | 130 | 0.964 |
| 15 | 0.011 | 75 | 0.391 | 135 | 0.967 |
| 20 | 0.015 | 80 | 0.546 | 140 | 0.971 |
| 25 | 0.018 | 85 | 0.642 | 145 | 0.974 |
| 30 | 0.022 | 90 | 0.728 | 150 | 0.978 |
| 35 | 0.026 | 95 | 0.792 | 155 | 0.982 |
| 40 | 0.029 | 100 | 0.847 | 160 | 0.985 |
| 45 | 0.033 | 105 | 0.892 | 165 | 0.989 |
| 50 | 0.036 | 110 | 0.929 | 170 | 0.993 |
| 55 | 0.040 | 115 | 0.947 | 175 | 0.996 |
| 60 | 0.044 | 120 | 0.956 | 180 | 1.000 |

## PRECIPITATION DEPTH VS. RETURN PERIOD

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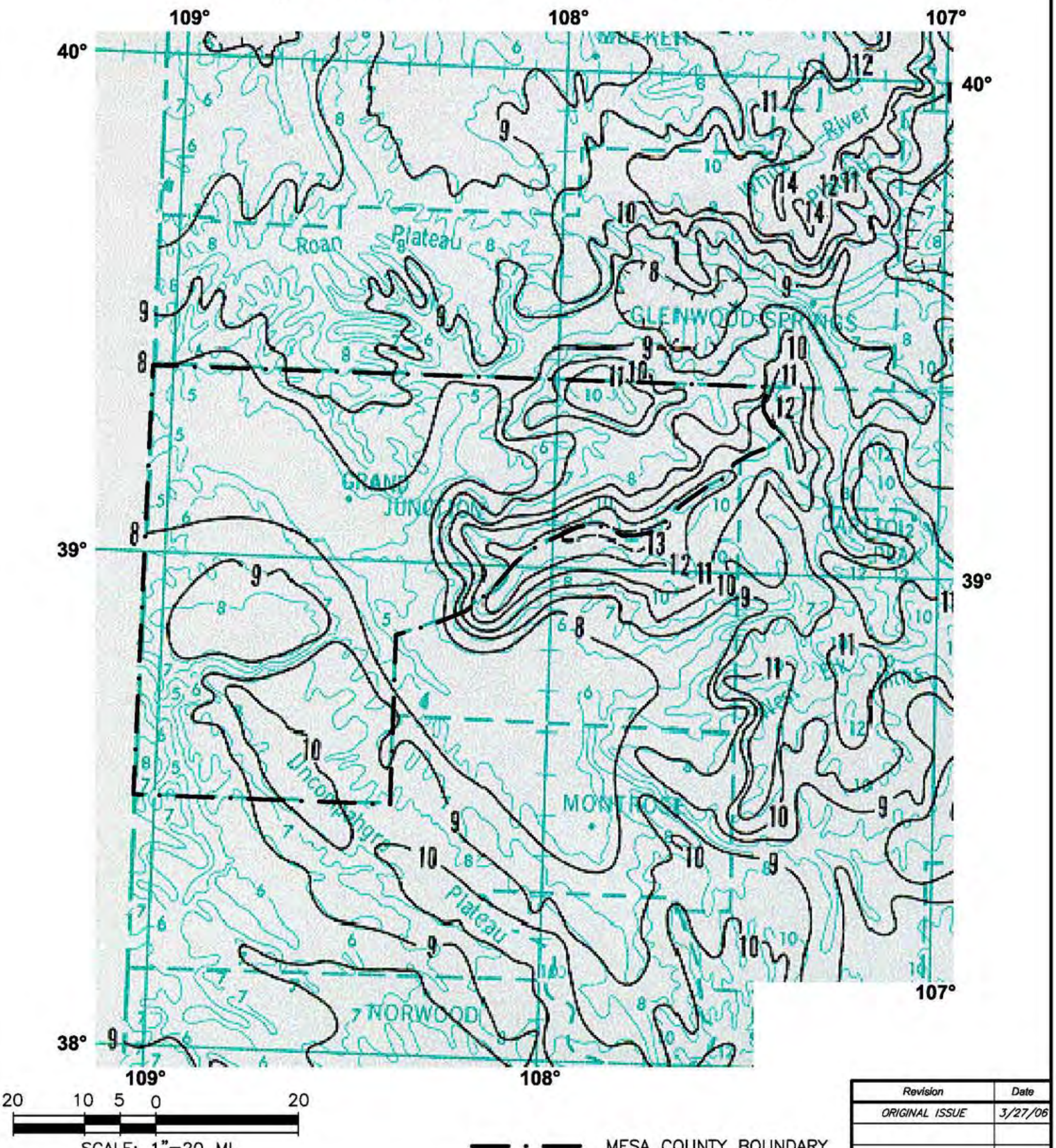
REFERENCE:
NOAA ATLAS 2, VOLUME III COLORADO, 1973

**FIGURE 601**

# STORMWATER MANAGEMENT MANUAL

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




| Revision | Date |
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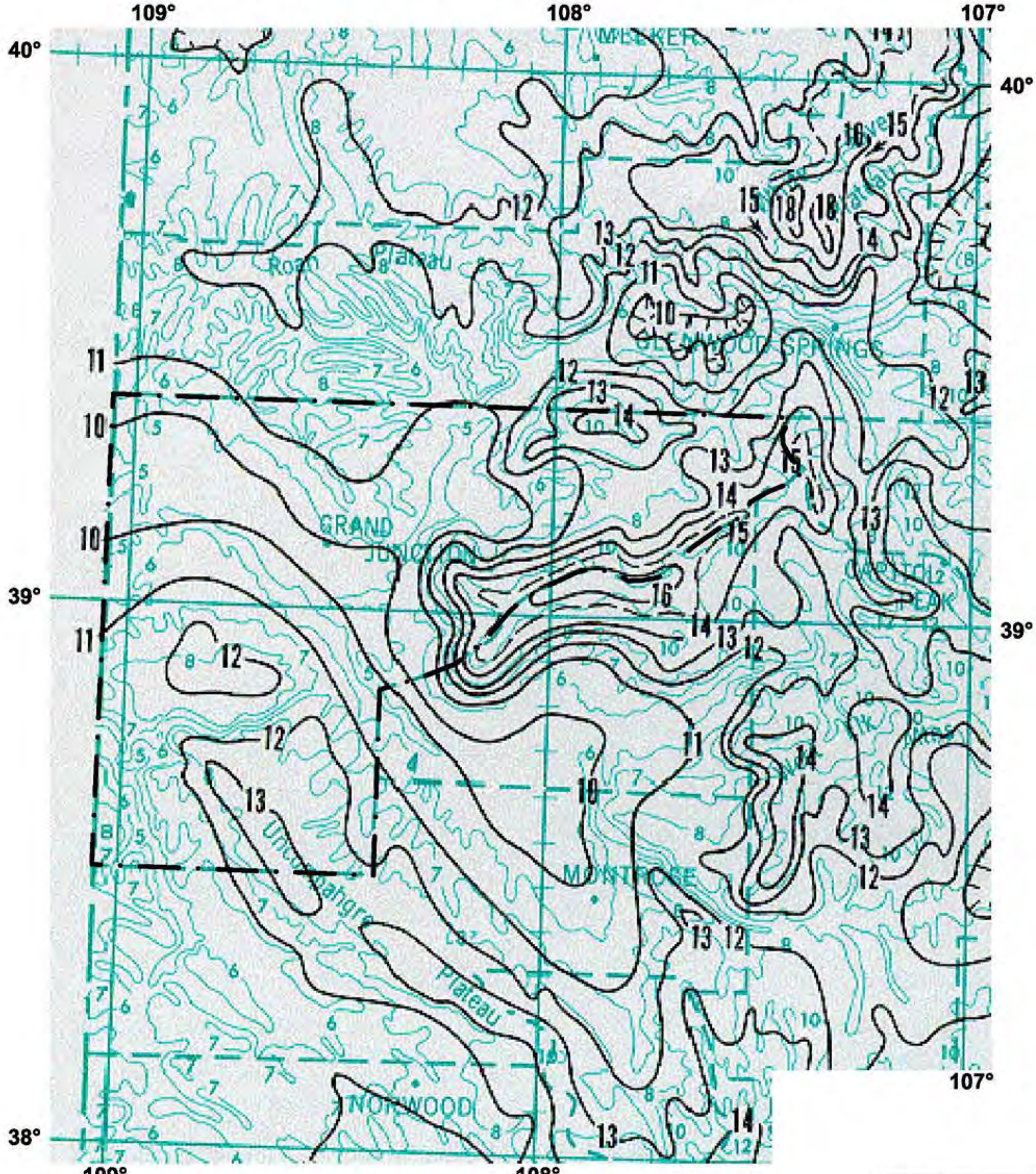
REFERENCE:

NOAA ATLAS 2, VOLUME III COLORADO, 1973

**FIGURE 602**

STORMWATER MANAGEMENT MANUAL

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




20 10 5 0 20

SCALE: 1"=20 MI.

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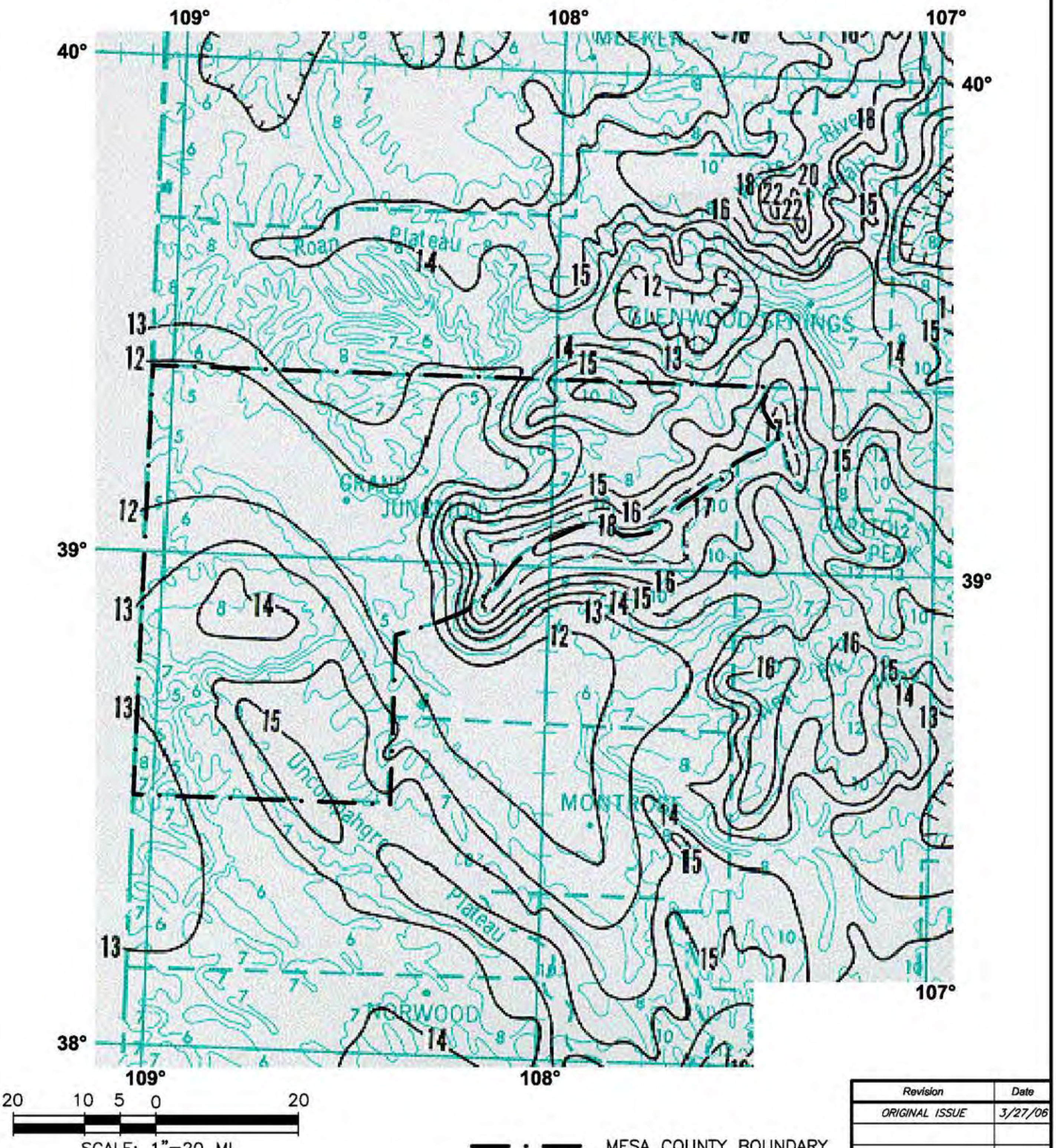
**FIGURE 603**

STORMWATER MANAGEMENT MANUAL

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

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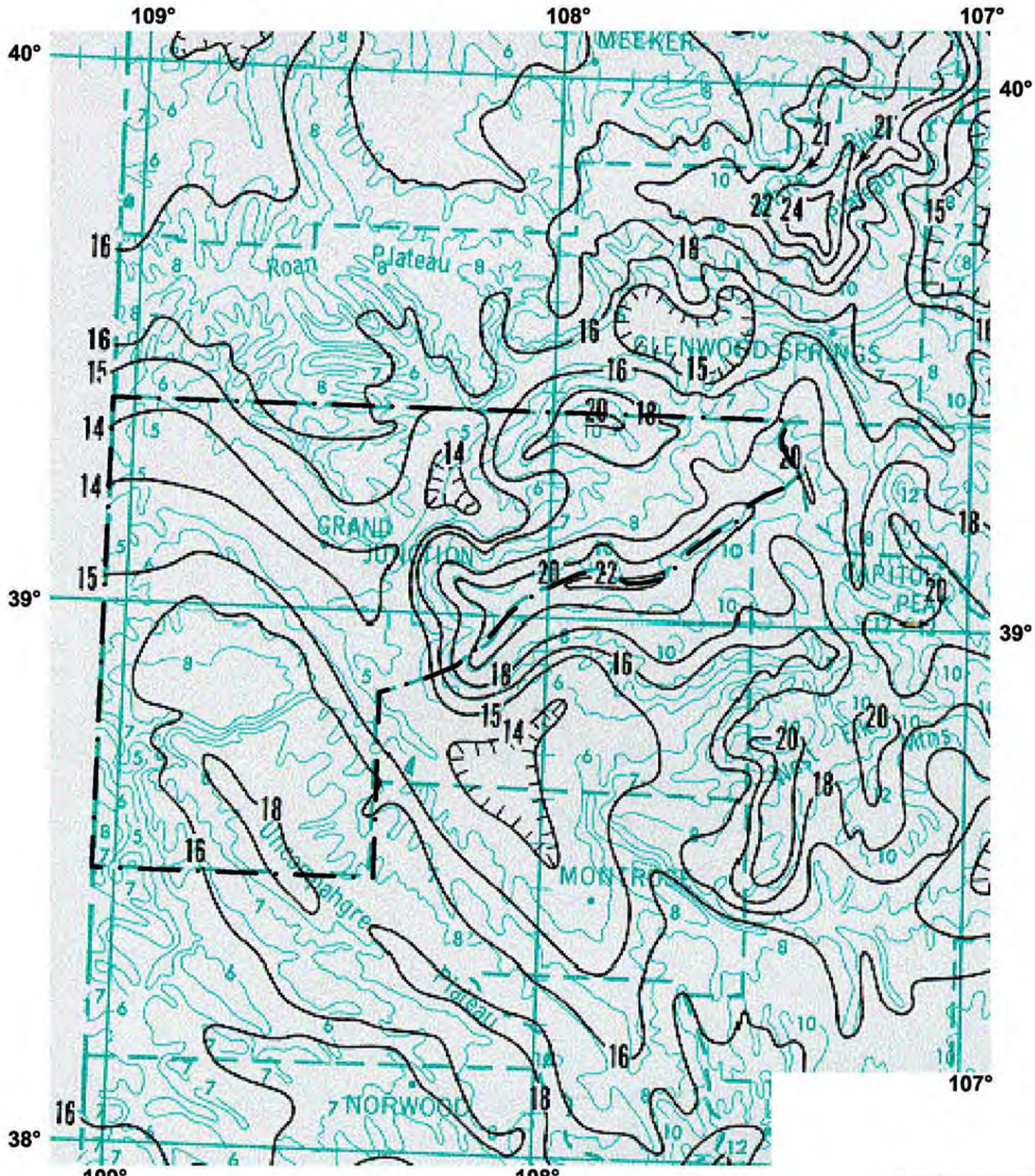
REFERENCE:

NOAA ATLAS 2, VOLUME III COLORADO, 1973

**FIGURE 604**

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

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SCALE: 1"=20 MI.

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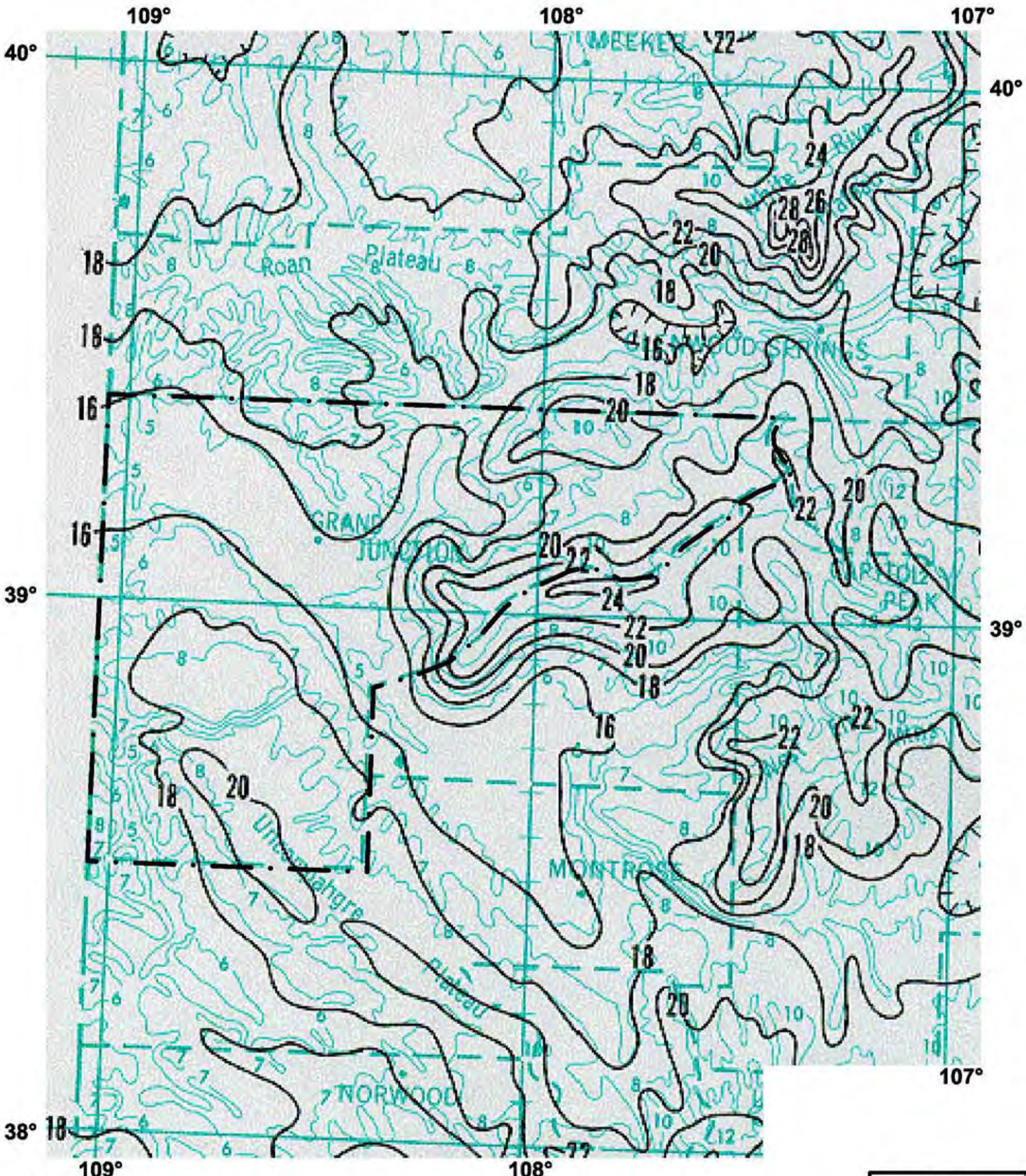
REFERENCE:

NOAA ATLAS 2, VOLUME III COLORADO, 1973

**FIGURE 605**

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




20 10 5 0 20

SCALE: 1"=20 MI.

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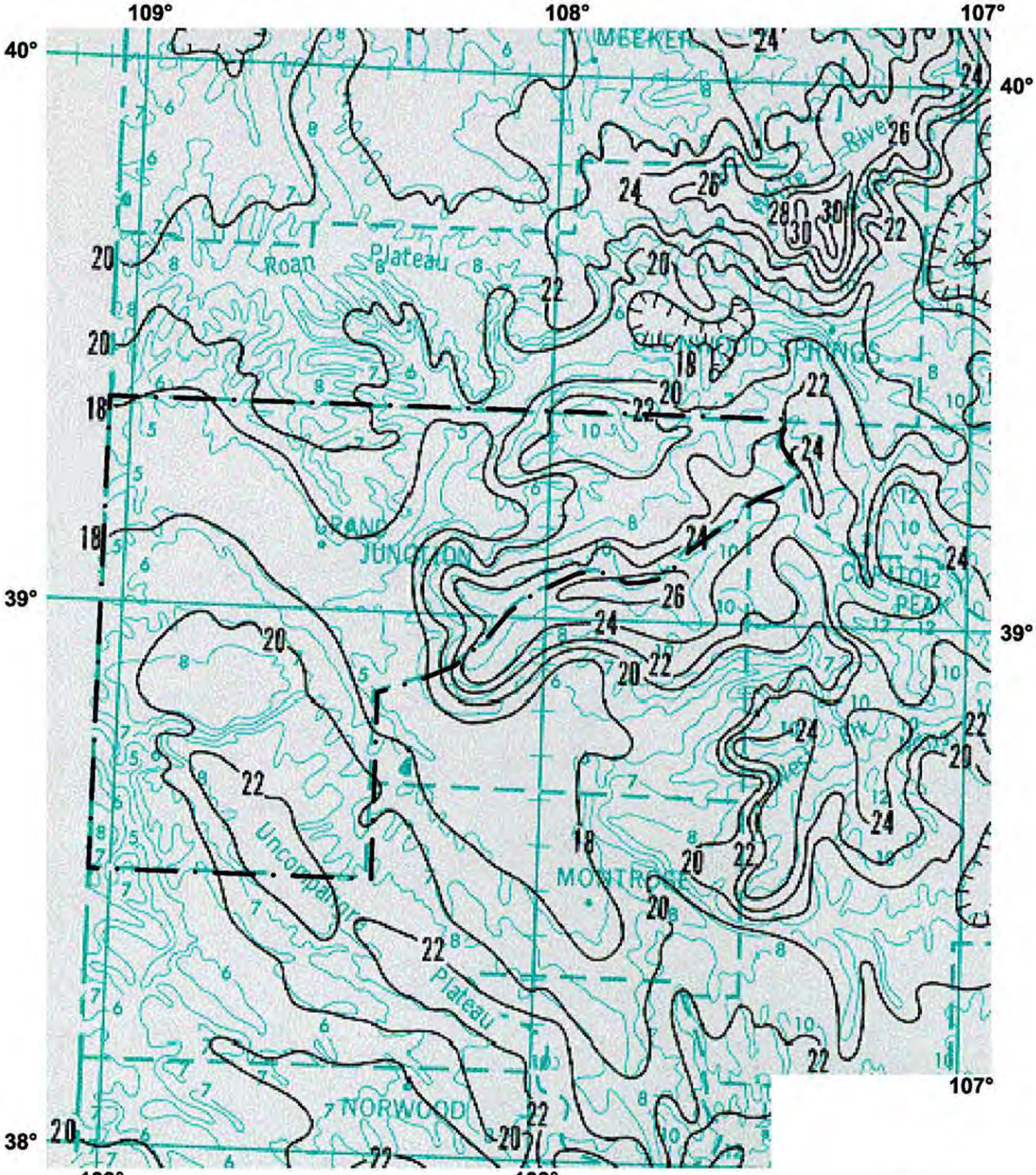
**FIGURE 606**

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




20 10 5 0 20

SCALE: 1"=20 MI.

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**FIGURE 607**

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

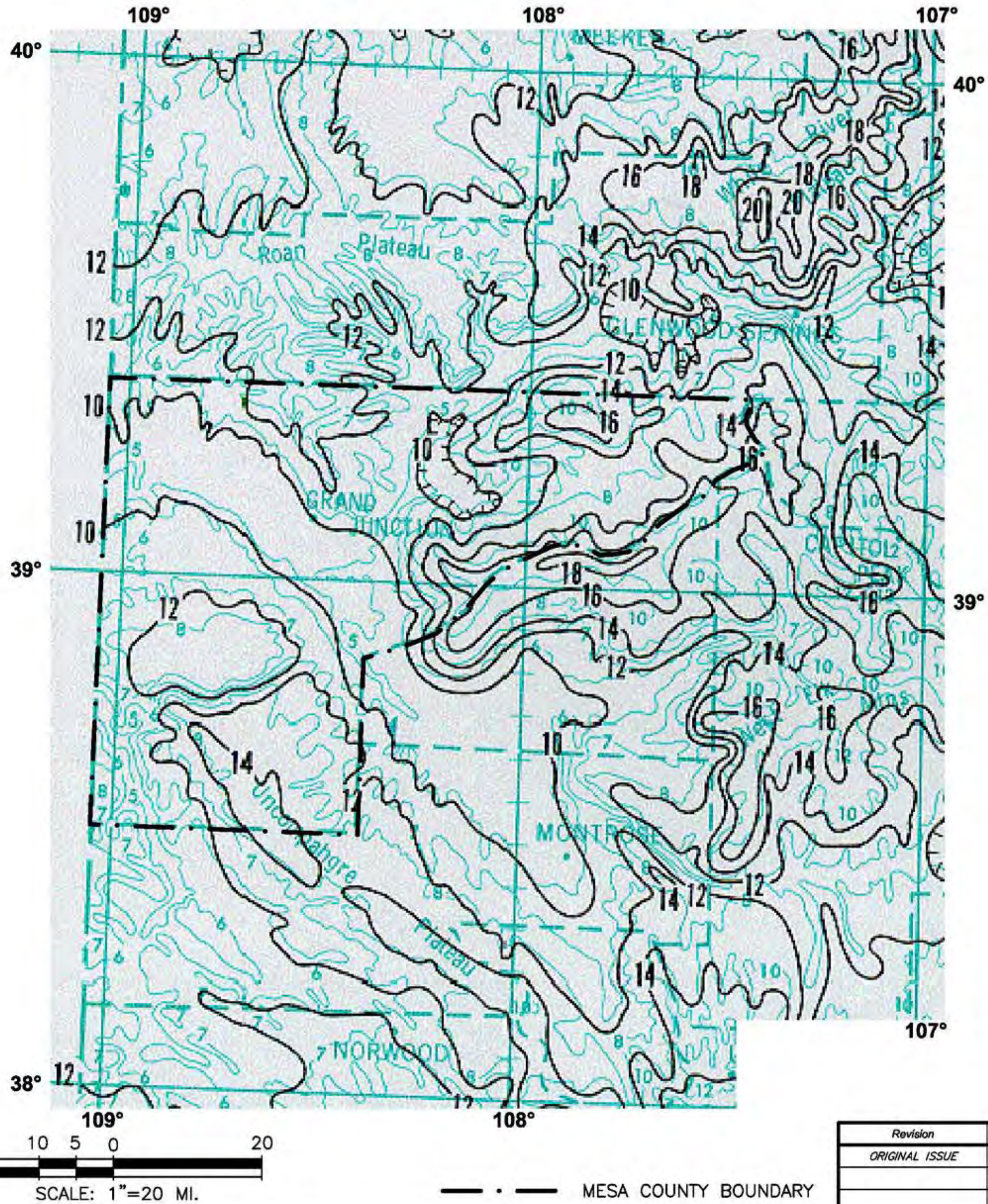


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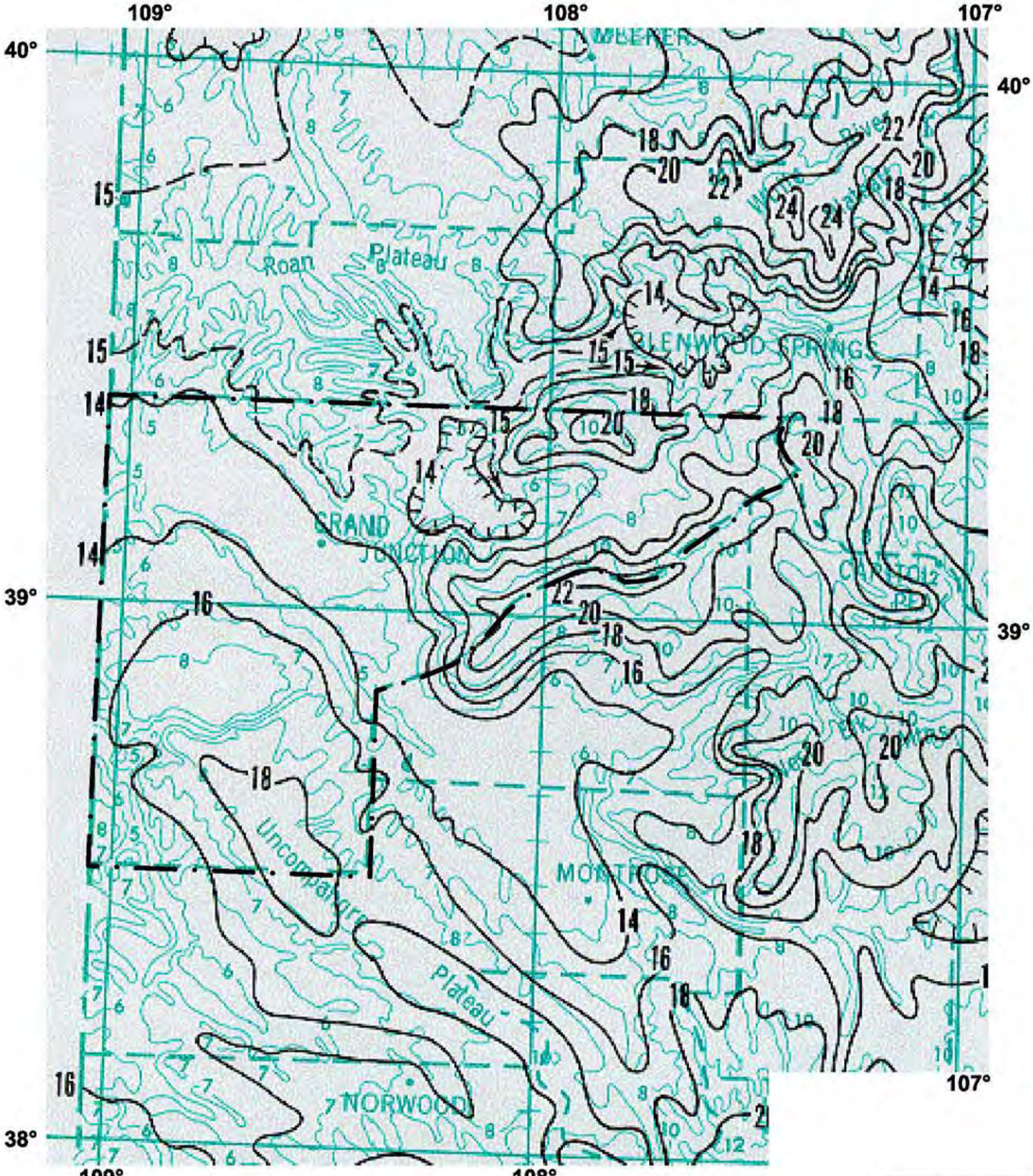
**FIGURE 608**

STORMWATER MANAGEMENT MANUAL

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




20 10 5 0 20

SCALE: 1"=20 MI.

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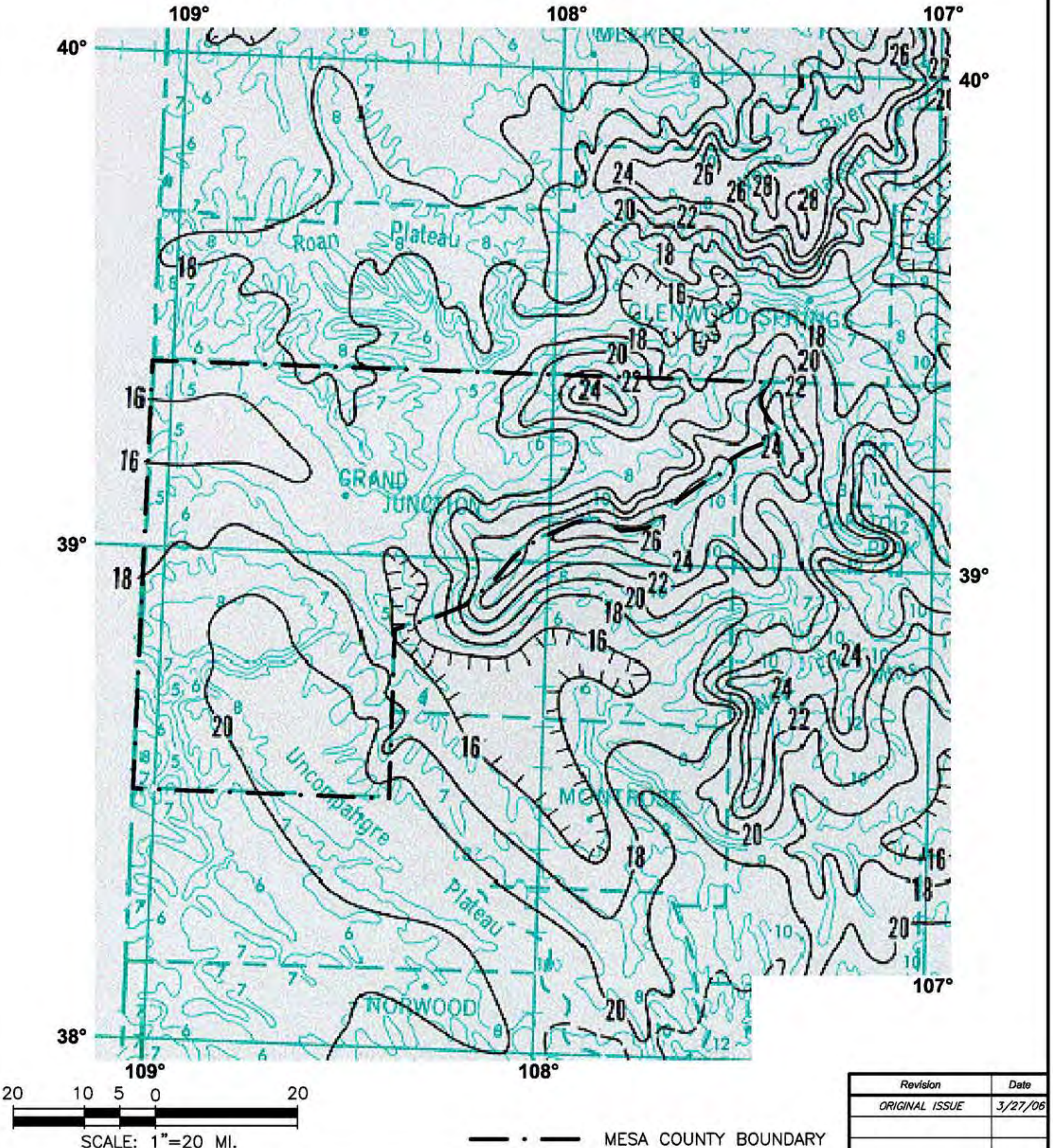
**FIGURE 609**

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

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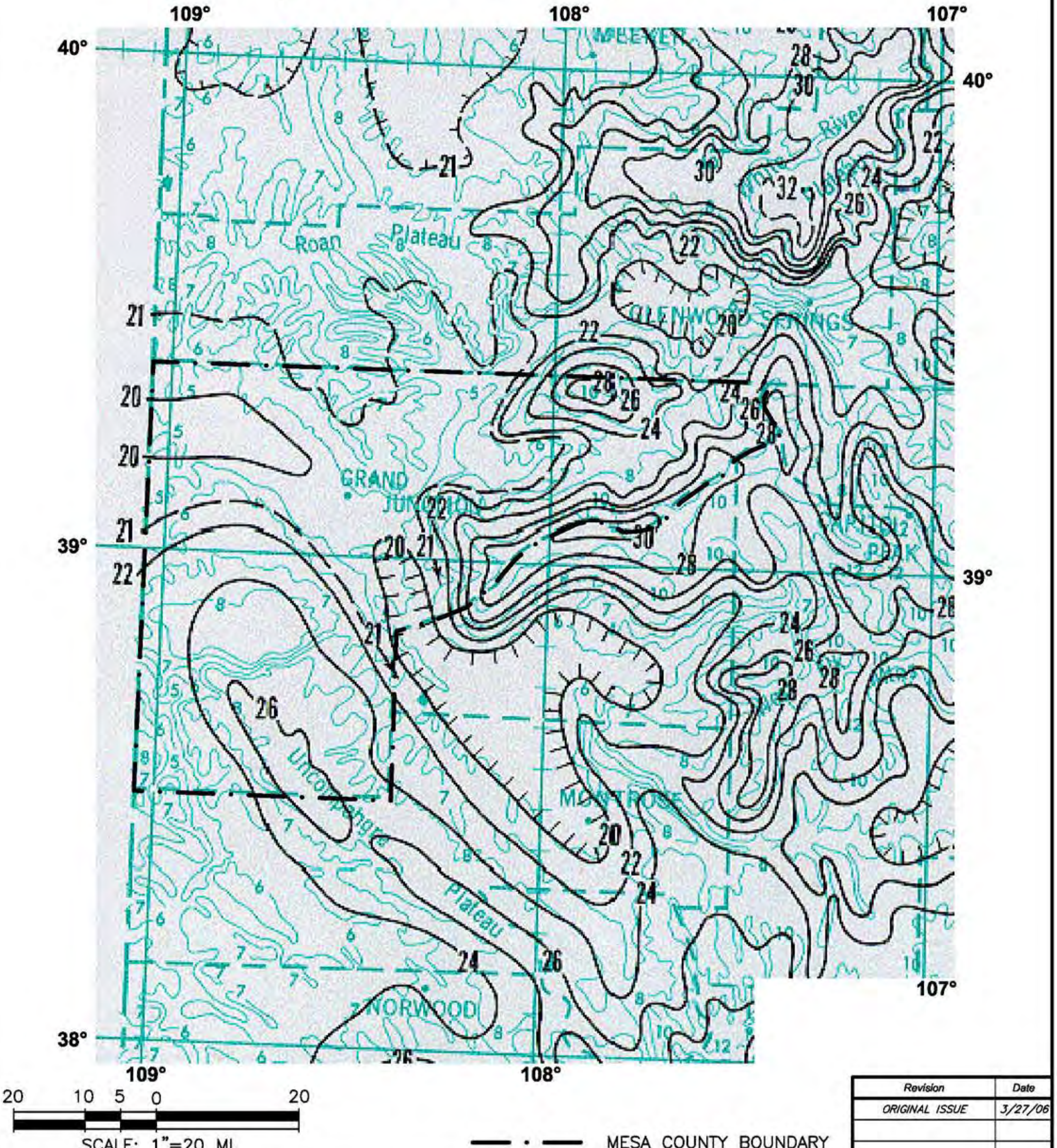
REFERENCE:

NOAA ATLAS 2, VOLUME III COLORADO, 1973

**FIGURE 610**

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

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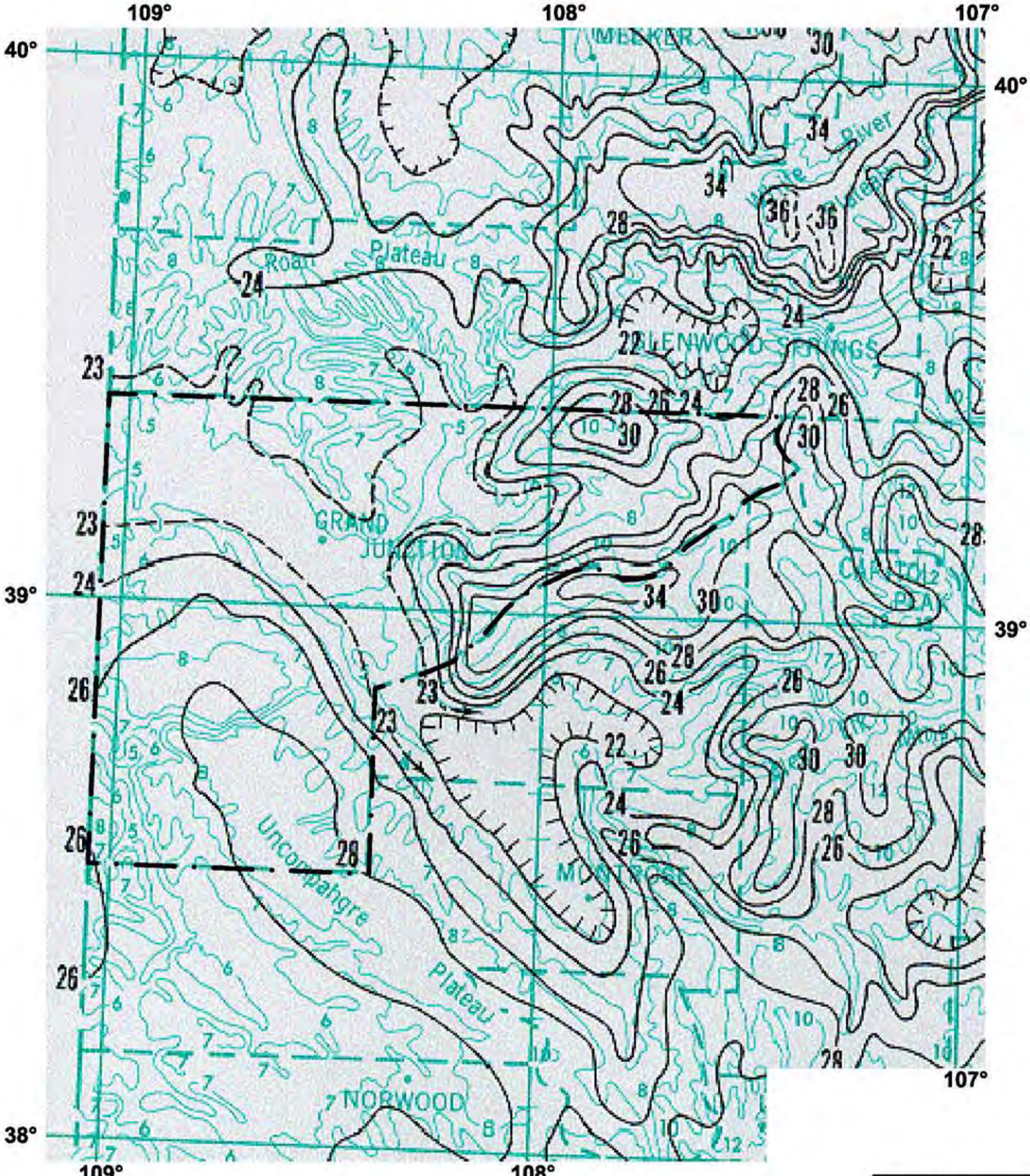
REFERENCE:

NOAA ATLAS 2, VOLUME III COLORADO, 1973

**FIGURE 611**

# STORMWATER MANAGEMENT MANUAL

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




20 10 5 0 20
SCALE: 1"=20 MI.

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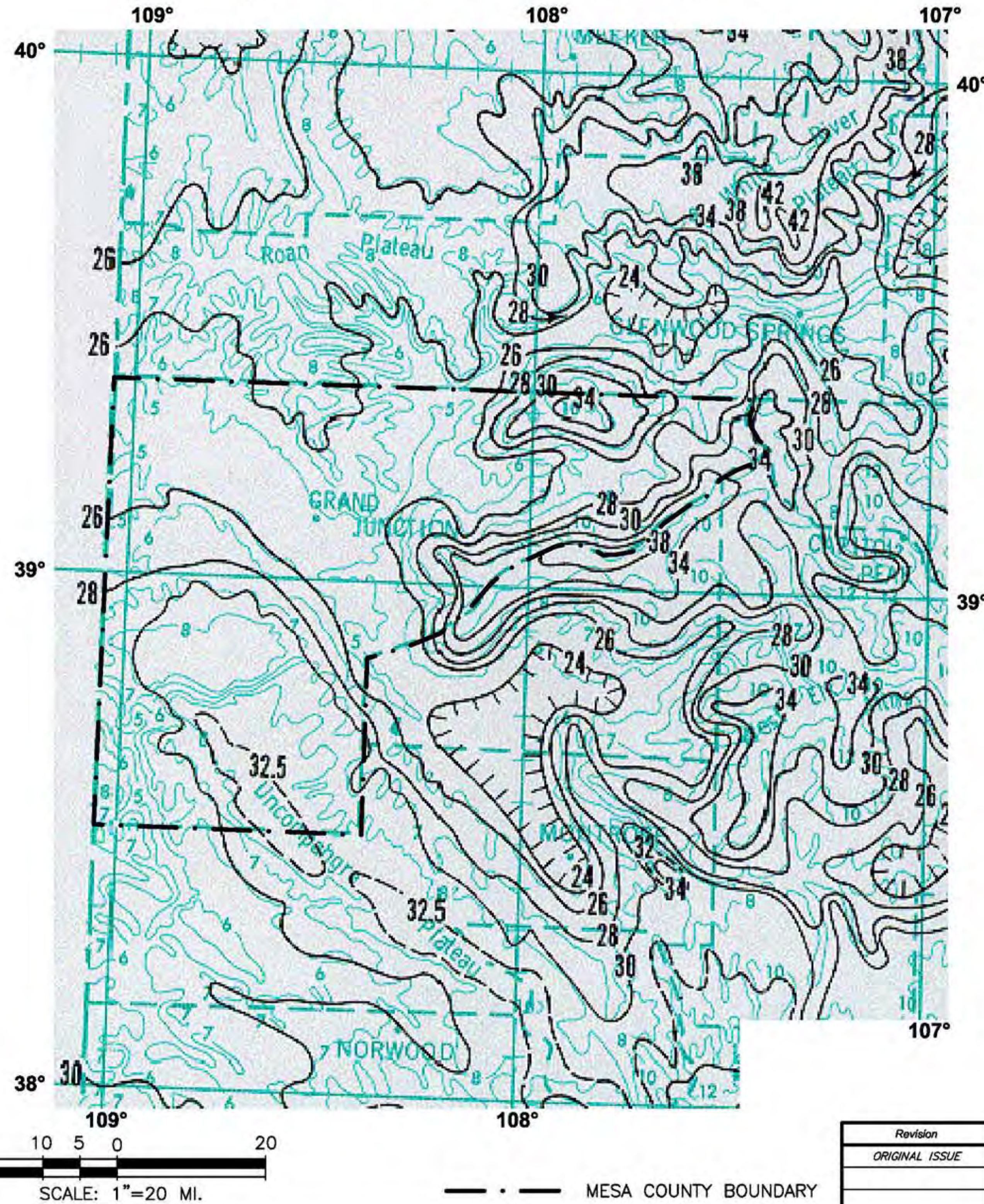
REFERENCE:
NOAA ATLAS 2, VOLUME III COLORADO, 1973

FIGURE 612

STORMWATER MANAGEMENT MANUAL

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

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REFERENCE:

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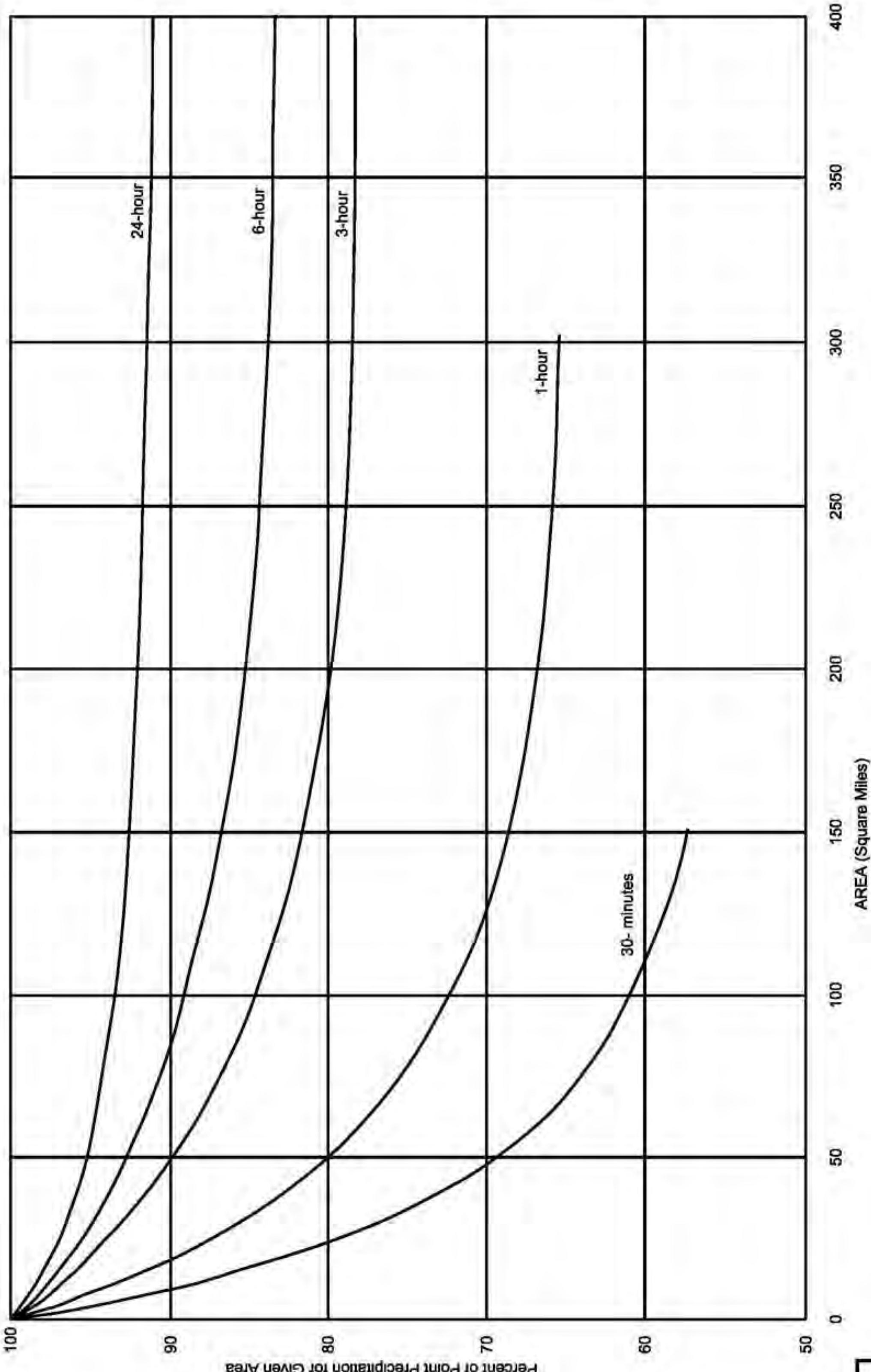
**FIGURE 613**




DEPTH AREA CURVES

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WRC ENGINEERING, INC.

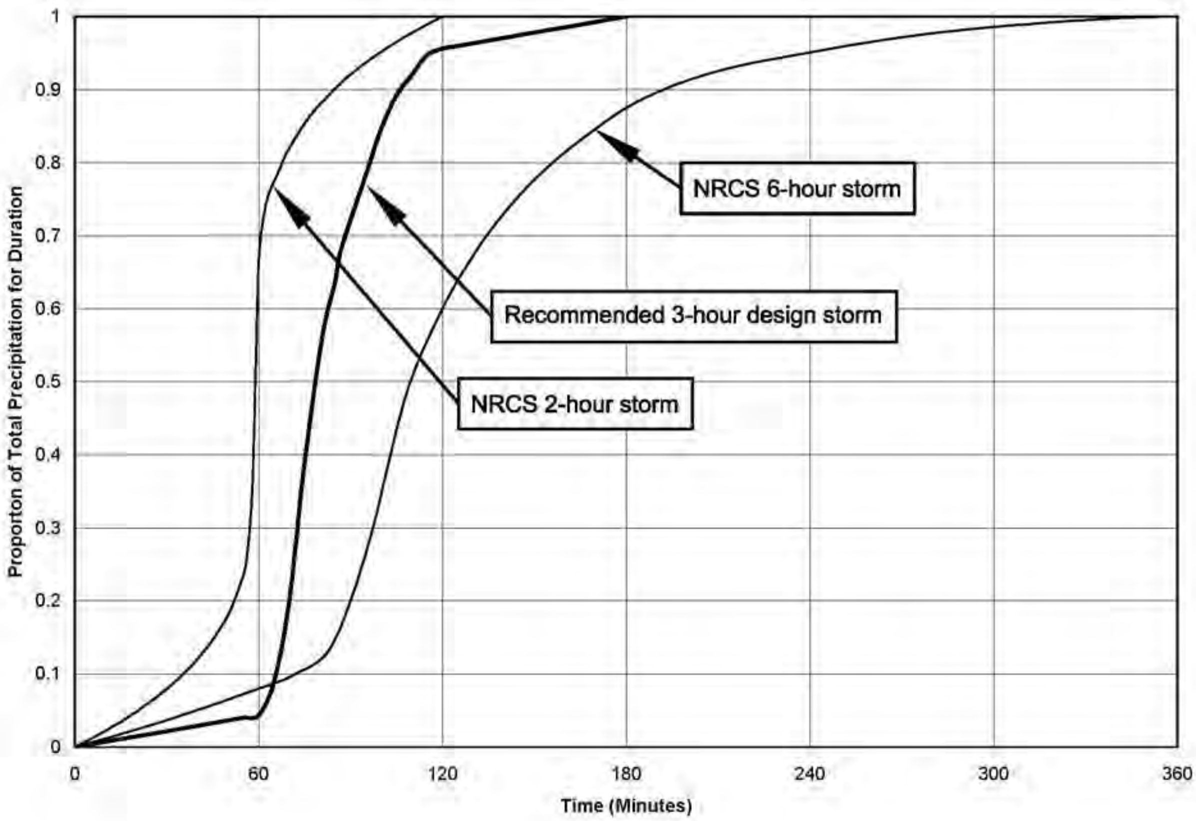
REFERENCE:
National Weather Service, NOAA 1973. Precipitation Frequency Atlas of the Western United States Volume III-Colorado. NOAA Atlas 2

**FIGURE 614**

# STORMWATER MANAGEMENT MANUAL

## DESIGN STORM DISTRIBUTION

NOTE: Use recommended storm distribution for entire Mesa County

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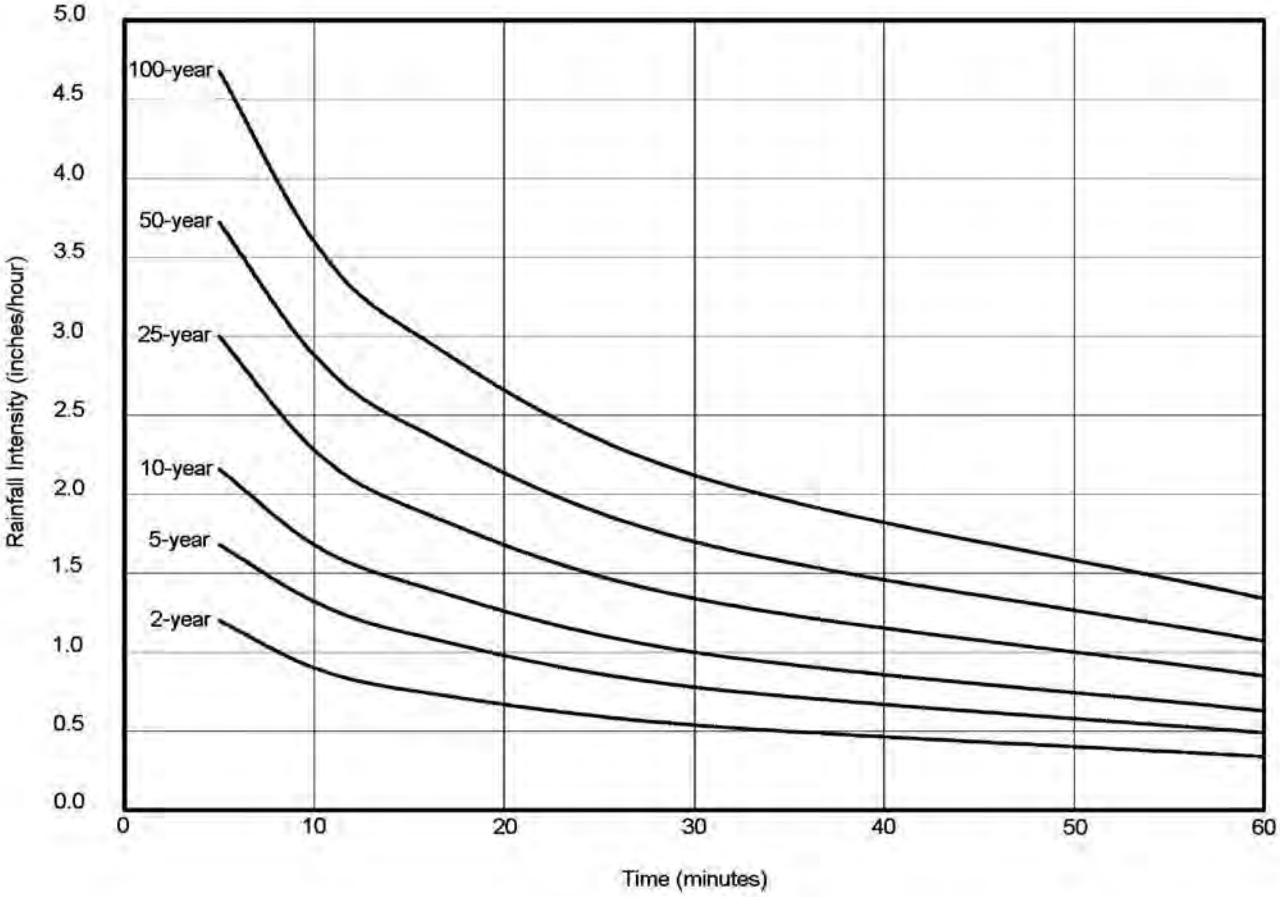
REFERENCE: Adapted from Henz Meteorological Services 1992. Mesa County Storm Drainage Criteria Manual Technical Memorandum 1 and 2

**FIGURE 615**

# INTENSITY-DURATION-FREQUENCY CURVES
## GRAND VALLEY AREA

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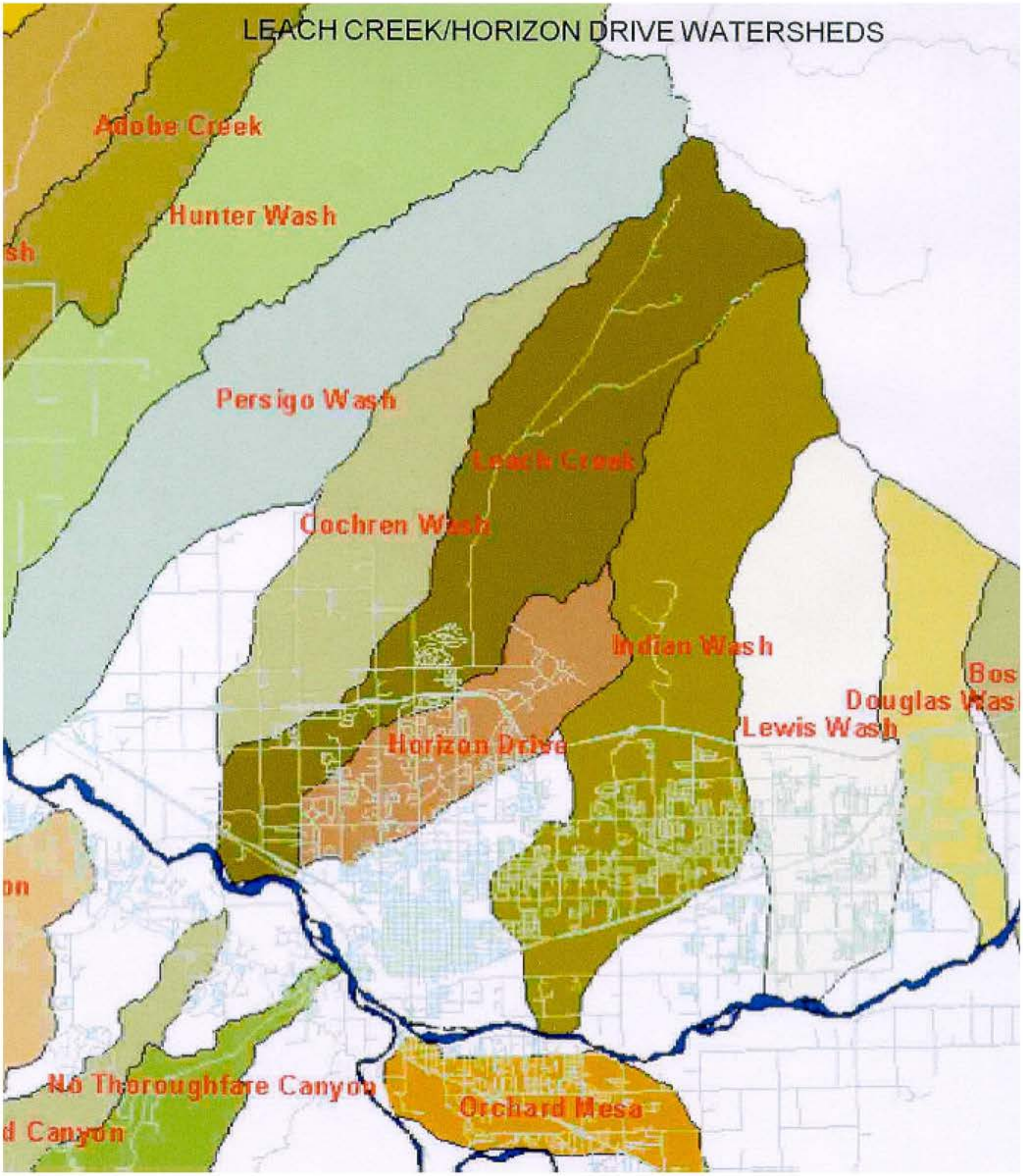
REFERENCE:
Henz Meteorological Services 1992. Mesa County Storm Drainage Criteria Manual Technical Memorandum 1 and 2

**FIGURE 616**




| Revision | Date |
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| ORIGINAL ISSUE | 3/27/06 |
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REFERENCE:

MESA COUNTY GIS WEBSITE

**FIGURE 617**

# SECTION 700

## STORM RUNOFF

# SECTION 700
## STORM RUNOFF

## TABLE OF CONTENTS

## LIST OF TABLES



## LIST OF FIGURES

# SECTION 700
# STORM RUNOFF

## 701 INTRODUCTION

For the area within the jurisdiction of this Manual, two deterministic hydrological models can be used to estimate storm runoff, Rational Formula and the SCS Unit Hydrograph method. The techniques for these methods are presented in this section.

The Rational and SCS Unit Hydrograph methods require the user to determine watershed characteristics (i.e.: area, length, slope, imperviousness, etc.) and the characteristic response time for the watershed (time of concentration). The SCS Unit Hydrograph method also requires calculation of the precipitation losses. Procedures for these parameters are presented first, followed by specific requirements for each method.

For certain circumstances, where adequate recorded stream flow data are available and the watershed area is large (> 10 square miles), a statistical analysis may be required to predict the storm runoff peaks or for calibration of deterministic models.

## 702 WATERSHED CHARACTERISTICS

The watershed characteristics needed for the runoff computation methods include the watershed area, the various flow path lengths, slopes, and surface conditions (i.e., overland, grassed channel, gutter), soil infiltration rates, and land use or imperviousness.

### 702.1 Physical Features

Watershed boundaries and areas can be determined from topographic maps, but a field investigation is recommended to verify boundaries. Land use and flow path characteristics can be obtained from zoning maps, aerial photographs, field investigations, or detailed topographic maps. Soil characteristics can be determined from NRCS (formerly SCS) soil reports for Mesa County.

### 702.2 Watershed Imperviousness

Imperviousness, $I_{mp}$, is a basic characteristic that is direct measure of the watershed's potential to produce storm runoff in terms of peak rates and total runoff volume. It is also a predictive indicator of watershed response to urbanization and the subsequent impact on stream stabilization and water quality.

Imperviousness is used in the Rational Method to derive the runoff coefficient and can be used in the SCS Unit Hydrograph method to derive runoff curve number. Imperviousness is typically determined in one of two ways, either by identification of watershed surface conditions (i.e.: natural ground versus hard surfaces) and direct computation or by characterization of the land uses (i.e.: undeveloped, residential, commercial, industrial) which have typical values of imperviousness.

Recommended impervious values for surface conditions and for typical land use types are provided in **Table 701**. The more precise approach is to divide the watershed into pervious and impervious surfaces and then compute the overall imperviousness directly, *which is the required method for determining imperviousness for new development*. This approach can be used when aerial photographs or detailed development plans (i.e.: for planned unit developments) are available, or field investigations of existing conditions are performed.

When this information is not available or for projecting watershed imperviousness for future land uses, then typical imperviousness values from **Table 701** are used. For single family residential development when the housing density (i.e. units per acre) and the size and type of housing (i.e.: ranch, split level, and two story) can be determined, then the use of **Figures 703, 704** and **705** is recommended. These figures are indicative of the existing development trend to place larger houses, including driveways, walks, patios, and out-buildings, on smaller lots. However, imperviousness values have also been found to be representative of older developments with small lots where additional impervious surfaces were added over time.

Note that in undeveloped areas that are to remain undeveloped, 2% imperviousness may be used for all areas that have soil and vegetative cover. However, these undeveloped areas often have significant geological features such as rock outcroppings that must be accounted for by calculating a composite imperviousness for these basins. Rock outcroppings are typically assigned an imperviousness of 100% for these calculations.

## 703 TIME OF CONCENTRATION

The time of concentration, $t_c$, is the time required for runoff to flow from the most remote part of the watershed area to the point of interest. For the Rational Formula, the time of concentration is calculated so that the average rainfall rate for a corresponding duration can be determined from the rainfall intensity-duration-frequency curves. For the 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.

For consistency between runoff analyses, the time of concentration equations presented in this section shall be used for all small watershed (less than 1.0 square miles) runoff calculations. For large watershed calculations, the watershed lag equation for the SCS Unit Hydrograph method is recommended (see Section 706.3)

Time of concentration consists of an initial time or overland flow time, $t_i$, plus travel time, $t_t$, which is in a combined form. In both urban and non-urban environments, the initial or overland flow is assumed to occur as sheet flow and as a function of surface type and slope, with an upper limit on the distance which this type of flow can occur.

### 703.1 Non-Urbanized Watersheds

For non-urban areas, the travel time occurs in a combined form, such as a small swale, channel, or wash. In urban areas, the travel time occurs in a combined form, such as in the storm drain, paved gutter, roadside drainage ditch, or drainage channel. Travel time can be estimated from the hydraulic properties of the storm drain, 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.

The time of concentration for both urban and non-urban areas is calculated as follows:

$$t_c = t_i + t_t \tag{701}$$

Where:

$t_c$ = Time of concentration (min)
$t_i$ = Initial, Inlet, or overland flow time (min)
$t_t$ = Travel time in the ditch, channel, gutter, storm drain, etc. (min)

Standard Form #2 has been developed to organize the calculation of $t_c$.

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

$$t_i = 1.8\ (1.1 - K)\ L_o^{1/2} / S^{1/3} \qquad (702)$$

Where:

$t_i$ = Initial or Overland Flow Time (min)
K = Flow Resistance Coefficient
$L_o$ = Length of Overland Flow, (ft, 300-ft maximum)
S = Average Watershed Slope (percent)

Equation 702 was originally developed by the Federal Aviation Administration (FAA, 1970) for use with the Rational Formula method. However, the equation is also valid for computation of the initial or overland flow time for the SCS Unit Hydrograph method. The 5-year runoff coefficient, $C_5$, presented in **Table 702** is recommended for the flow resistance coefficient, K.

The overland flow length, $L_o$, is generally defined as the length over which the flow characteristics appear as sheet flow or very shallow flow in broad, 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, which results in higher flow velocities and shorter travel times. Therefore, the initial flow time is limited to the time to travel a distance of 300 feet.

For watersheds longer than 300 feet, the travel time, $t_t$, must be added to the overland flow time. Travel time can be calculated using Manning's equation and the hydraulic properties of the storm drain, gutter, swale, ditch, or channel or can be approximated from Equation 703 and **Figure 701**:

$$V = C_v\ S_w^{0.5} \qquad (703)$$

Where:

V = Velocity, fps
$S_w$ = watercourse slope, ft/ft
$C_v$ = Conveyance coefficient

**Table 703 Travel Time Conveyance Coefficients**

| Land Surface | Conveyance Coefficient $C_v$ |
|---|---|
| Heavy meadow | 2.5 |
| Tillage/Field | 5.0 |
| Short pasture and lawns | 7.0 |
| Nearly bare ground | 10 |
| Grassed waterways | 15 |
| Paved areas and shallow swales | 20 |

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

### 703.2 Urbanized Watersheds

Overland flow in urbanized watersheds can occur from the back of the lot to the street, in parking lots, in greenbelt areas, or within park areas and can be calculated using the procedure described in Section 703. Travel time, $t_t$, to the first design point or inlet is often determined based on the conveyance coefficient for "Paved Areas and shallow swales", but can be estimated using Manning's equation.

The time of concentration for the first design point in an urbanized watershed using this procedure should not exceed the time of concentration calculated using Equation 704, which was developed using rainfall/runoff data collected in urbanized regions (USDCM,1969).

$$t_c = L / 180 + 10 \qquad (704)$$

Where:

$t_c$ = Time of Concentration at the first design point (min)
L = Watershed Length (ft)

Equation 704 may result in a lesser time of concentration at the first design point and thus would govern in an urbanized watershed. The recommended minimum $t_c$ to the first urban design point is 5 minutes. For subsequent design points, the time of concentration is calculated by accumulating the travel times in downstream reaches.

## 704 PRECIPITATION LOSSES

Two methods for calculating precipitation losses are presented below, the SCS Curve Number and the Green-Ampt methods, both which can be used for the SCS Unit Hydrograph Method.

Land surface interception, depression storage and infiltration are referred to as precipitation losses. Interception and depression storage represent the surface storage of water by trees or grass, in local depressions in the ground surface, in cracks and crevices in parking lots or roofs, or in a surface area where water is not free to move as overland flow. Infiltration represents the movement of water through the soil beneath the land surface.

Three important factors should be noted about precipitation loss computations. 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 (i.e.: contribute to surface runoff); therefore, the calculated amount of runoff is conservatively high. Finally, precipitation losses are considered to be a uniformly distributed over an entire sub-watershed.

### 704.1 SCS Curve Number Method

The National Resources Conservation Service (NRCS, formerly SCS), 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 watershed characteristics of soil groups to a curve number, CN (SCS, 1985). The NRCS provides information relating soil group type to the curve number as a function of soil cover, land use type and antecedent moisture conditions, which can be determined from the soil name.

Precipitation loss is calculated based on supplied values of CN and IA, which are related to a total runoff depth for a storm by the following relationships:

$$Q = (P - IA)^2 / ((P - IA) + S) \qquad (705)$$

$$S = (1{,}000 / CN) - 10 \qquad (706)$$

$$IA = 0.2\ S \qquad (707)$$

Where:

Q = Accumulated Excess (in)
P = Accumulated Rainfall Depth (in)
IA = Initial abstraction (in)
S = Currently Available Soil Moisture Storage Deficit (in)
CN = SCS Curve number

Note that initial abstraction, IA, (i.e.: soil surface storage capacity) is based on empirical evidence established by the NRCS, and is the default value in HEC-1 Program (HEC, 1988).

Since the SCS method results in 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.

### 704.2 CN Determination

The SCS Curve Number Method uses a soil cover curve 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, which correlates each soil name with the soil group. Land use and treatment class are determined during field visits or from aerial photographs. Procedures for determining land use and treatment class are found in Chapter 8 of National Engineering Handbook, Section 4 (SCS, 1985). Antecedent moisture condition of the watershed is explained as follows:

The amount of rainfall in a period of 5 to 30 days preceding a particular storm is referred to as antecedent rainfall, and the resulting condition of the watershed in regard to potential runoff is referred to as an antecedent moisture condition. In general, higher amounts of antecedent rainfall result in greater amounts of runoff from a given storm. The effects of infiltration and evapo-transpiration during the antecedent period are also important, as they may increase or lessen the effect of antecedent rainfall. Because of the difficulties of determining antecedent storm conditions from data normally available, the conditions are reduced to three cases, AMC-I, AMC-II and AMC-III. For the Mesa County area, an AMC-II condition is recommended 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 704**, which is reproduced from Table 2-2 in TR-55 (SCS, 1986).

When land uses shown in **Table 704** are not applicable or when more detailed land use information is available, CN values can be calculated directly from imperviousness estimates using the following equation.

$$CN = 98*Imp + X*(1-Imp) \qquad (708)$$

Where:

Imp. = Imperviousness as a decimal

X = Adjustment factor based on NRCS Soil Type

| NRCS Soil Type | Adjustment Factor |
|---|---|
| A | 39 |
| B | 61 |
| C | 74 |
| D | 80 |

Note that Equation 708 was derived from the data plotted on Figure 2-3 in TR-55 (SCS 1986) and applies when impervious surfaces are connected. Adjustment for disconnected impervious surfaces can be made using Figure 2-4 in TR-55. This adjustment is not required as the connected impervious surface assumption will result in conservatively high CN values.

### 704.3 Green and Ampt Method

The Green-Ampt method models infiltration by combining an unsaturated flow form of Darcy's law with requirements of mass conservation. The Green-Ampt method involves the simulation of rainfall loss as a two-phase process. The first phase of rainfall loss is called initial abstraction (IA) or surface retention loss, which involves vegetation interception, evaporation, and surface depression storage. Typical surface retention loss values are shown in **Table 705**.

**Table 705 Surface Retention Loss**

| Land Use and/or Surface Cover | Surface Retention Loss, IA (inches) |
|---|---|
| Natural | |
| Desert and rangeland, flat slope | 0.35 |
| Desert Hillslopes | 0.15 |
| Mountain with vegetated surface | 0.25 |
| Developed (Residential/commercial) | |
| Lawn and turf | 0.20 |
| Desert landscape | 0.10 |
| Pavement | 0.05 |
| Agricultural, tilled fields and irrigate pasture | 0.50 |

The second phase of the rainfall loss process is infiltration of rainfall into the soil. The infiltration is assumed to begin after the surface retention loss is completely satisfied. Excess precipitation is computed using the Green-Ampt equations after the initial loss is satisfied. Required parameters are then the hydraulic conductivity of the soil at saturation, volumetric moisture deficit at the beginning of rainfall, and wetting front capillary action.

Typical values for Green-Ampt parameters were obtained from the *Drainage Design Manual for Maricopa County Hydrology* (Maricopa County 2003 (draft)), available at http://www.fcd.maricopa.gov/Resources/HydrologyManual.asp.

704.3.1 Soil Hydraulic Conductivity

The soil hydraulic conductivity is based on soil texture classification. Typical values for different soil type are listed in **Table 706**. For watershed areas of sub-watersheds consisting of several different soil textures, a composite value for Green-Ampt parameters should be used.

**Table 706 Soil Hydraulic Conductivity Values**
(for bare ground)

| Soil Texture Classification | Hydrologic Soils Group | Soil Hydraulic Conductivity (in/hr) |
|---|---|---|
| Loamy-sand and sand | A | 1.20 |
| Sandy-Loam | B | 0.40 |
| Loam | B | 0.25 |
| Silty-loam | C | 0.15 |
| Sandy clay-loam | C | 0.06 |
| Clay-loam | D | 0.04 |
| Silty clay-loam | D | 0.04 |
| Sandy-clay | D | 0.02 |
| Silty-clay | D | 0.02 |
| Clay | D | 0.01 |

Hydraulic conductivity is affected by several factors besides soil texture, including soil crusting, tillage, ground cover, and canopy cover and should be adjusted. Ground cover, such as grass, litter, and gravel, will generally increase the infiltration rate over that of bare ground conditions.

Canopy-cover, such as from trees, brush, and tall grasses, also increases the bare ground infiltration rate. A simplified procedure for adjusting bare ground hydraulic conductivity to account for vegetation cover is provided in Equation 709:

$$Ck = (Vc\text{-}10)/90 + 1.0 \qquad (709)$$

Where:

Ck = Hydraulic conductivity ratio to bare ground conductivity

Vc = Vegetated cover as a percentage

704.3.2 Volumetric Soil Moisture Deficit

The soil moisture deficit is a volumetric measure of the soil moisture-storage capacity that is available at the start of the rainfall. The volumetric moisture deficit is a function of the effective porosity of the soil and its value is in the range of near zero to the effective porosity. If the soil is saturated at the start of rainfall, then the moisture deficit equals zero. If the soil is devoid of moisture at the start of rainfall, then the moisture deficit is essentially the effective porosity of the soil.

Three conditions for soil moisture deficit have been defined for use based on the antecedent soil moisture condition that could be expected to exist at the start of the design rainfall. These three conditions are:

- "Dry" for antecedent soil moisture near the vegetation wilting point;
- "Normal" for antecedent soil moisture condition near field capacity due to previous precipitation or land irrigation; and
- "Saturated" for antecedent soil moisture near effective saturation due to recent precipitation or land irrigation.

Typical volumetric soil moisture deficit values for the above three defined conditions may be taken from **Figure 702**. For Mesa County, the recommended condition is "normal", which is consistent with antecedent moisture condition II.

704.3.3 Wetting Front Capillary Action

This parameter is relatively insensitive to ground cover, and is a function of the average bare soil type hydraulic conductivity. Typical values may be taken from **Figure 702**.

## 705 RATIONAL FORMULA METHOD

For watersheds that are not complex and have small watershed areas, the design storm runoff may be analyzed using the Rational Formula method. This method is widely used due to its simplicity and level of general acceptance. The Rational Formula method, when properly applied in a consistent manner, results in drainage facilities sizes that adequately convey storm runoff with minimal consequences.

### 705.1 Methodology

The Rational method is based on the formula:

$$Q = CIA \quad (710)$$

Where:

Q = Maximum rate of runoff in cubic feet per second (cfs)
C = Runoff coefficient
I = Average intensity of rainfall in inches per hour
A = Contributing watershed area in acres.

### 705.2 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.

### 705.3 Limitations on Methodology

The Rational Formula method adequately estimates the peak rate of runoff from a rainstorm in a given watershed, but does not provide information on the full hydrograph and only approximates the runoff volume.

Typical design procedures assume 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. The problem becomes one of routing the surface and subsurface hydrographs which have been separated by the storm drain system. In general, this sophistication is not warranted and a conservative assumption is made that the entire routing occurs through the storm drain system when the system is present.

Because of the limitations of the Rational Method, the following guidelines on its application are provided:

- The individual sub-watershed sizes should not be greater than 20 acres.

- The aggregate of all sub-watershed areas should not be greater than 160 acres.

- The sub-watersheds should be reasonably homogeneous for existing and for projected land use.

### 705.4 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. Development of rainfall intensities for the Rational Method is described in Section 600.

### 705.5 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.

The first step is to determine the composite imperviousness of the watershed using recommended values in **Table 701**. Once the imperviousness is determined, recommended C values for various recurrence interval storms are determined from **Table 702** or can be calculated using the following equations:

$$C_{CD} = K_{CD} + (0.858*i^3 - 0.786*i^2 + 0.774*i + 0.04) \qquad (711)$$
$$C_A = K_A + (1.31*i^3 - 1.44*i^2 + 1.135*i - 0.12) \qquad (712)$$
$$C_B = (C_A + C_{CD})/2 \qquad (713)$$

Where:

$C_{CD}$ = Runoff coefficient for C and D soils*
$C_A$ = Runoff coefficient for A soils*
$C_B$ = Runoff coefficient for B soils*
i = Imperviousness as a decimal
$K_{CD}$ = Coefficient adjustment for C and D Soils
$K_A$ = Coefficient adjustment for A soils

*If C is not greater than or equal to zero, it should be set to 0.

**Table 707 Adjustment Factors for Runoff Coefficient Equations**
(values of $K_A$ and $K_{CD}$ based on rainfall intensity (i) in inches/hour)

| NRCS Soil | 2-year | 5-year | 10-year | 25-year | 50-year | 100-year |
|---|---|---|---|---|---|---|
| C and D | 0 | -0.10i+0.11 | -0.18i+0.21 | -0.28i+0.33 | -0.33i+0.40 | -0.39i+0.46 |
| A | 0 | -0.08i+0.09 | -0.14i+0.17 | -0.19i+0.24 | -0.22i+0.28 | -0.25i+0.32 |

Composite imperviousness is computed on the basis of the percentage of different types of surfaces or land uses in the watershed 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.

The equations for runoff coefficients as a function of imperviousness and recurrence interval were obtained from the *Urban Storm Drainage Criteria Manual* by the Urban Drainage & Flood Control District (UDFCD 2001), which can be obtained in pdf format on the following website: http://www.udfcd.org/.

### 705.6 Application of the Rational Formula Method

The first step in applying the Rational Formula method is to obtain a topographic map and define the boundaries of all the relevant watersheds. Watersheds to be defined include all watersheds tributary (i.e.: offsite areas) to the area of study and sub-watersheds in the study area. A field check is recommended and field surveys may be required in some cases. At this stage of planning, the possibility for the diversion of watershed runoff should be identified.

The major storm watershed does not always coincide with the minor storm watershed. This is often the case in urban areas where a low flow will stay within the gutter and follow the lowest grade, but when a large flow occurs the flow will be sufficiently deep or flowing with a high velocity such that part of the runoff will overflow street crown and flow into a new sub-watershed.

### 705.7 Major Storm Analysis

When analyzing the major runoff occurring on an area that has a storm drain system sized for the minor storm, care must be used when applying the Rational Formula method. Normal application of the Rational Formula method assumes that all of the runoff is collected by the storm drain. For the minor storm design, the time of concentration is dependent upon the flow time in the drain. However, during the major storm runoff, the drains 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 drains.

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 drain. The analysis of this aspect of the interaction between the storm drain system and the major storm runoff is complex. The simplified and conservative approach of using the minor storm time of concentration for all frequency analyses is acceptable for the Mesa County area.

## 706 SCS UNIT HYDROGRAPH METHOD

The SCS Unit Hydrograph was derived from a large number of natural unit hydrographs from watersheds varying widely in size and geographic location.

### 706.1 Methodology

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

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

## 706.2 Assumptions

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

a. The effects of all physical characteristics of a given watershed are reflected in the shape of the storm runoff hydrograph for that watershed.

b. 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.

c. A hydrograph of storm discharge that would result from a series of bursts of excess rain or from continuous excess rain of variable intensity may be constructed from a series of overlapping unit graphs each resulting from a single increment of excess rain of unit duration.

## 706.3 Lag Time

Input data for the SCS Dimensionless Unit Hydrograph method (SCS, 1985) consists of a single parameter, $T_{LAG}$, 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 watersheds (less than 1 square mile) in the Mesa County area, the lag time is related to the time of concentration, $t_c$, by the following empirical relationship:

$$T_{LAG} = 0.6\ tc \qquad (714)$$

Where:

$T_{LAG}$ = The time (hours) between the center of mass of the rainfall excess and the peak of the unit hydrograph.
$t_c$ = The time of concentration (minutes, see section 703).

For large watersheds (greater than 1 square mile), 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 watershed gets increasingly larger, the average flow velocity (and associated travel time) becomes more difficult to estimate. Therefore, for these watersheds, the following lag equation is recommended for use in computing $T_{LAG}$:

$$T_{LAG} = 22.1\ K_n\ (L\ L_c\ /\ S^{0.5})^{0.33} \qquad (715)$$

Where:

$K_n$ = Manning's roughness factor for the watershed channels
L = Length of longest watercourse (miles)
$L_c$ = Watercourse length from the outflow point to a point on the channel nearest the centroid of the watershed (miles)
S = Average slope of the longest watercourse (ft per mile)

This lag equation is based on the United States Bureau of Reclamation's (USBR) analysis of the above parameters for several watersheds 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.

For watersheds 1.0 square miles in size, it is recommended that the method which results in the shortest $T_{LAG}$ value be used to be conservative.

### 706.4 Roughness Factor

For consistency, recommended roughness factors, $K_n$, used in the lag time calculation are presented in **Table 708**. 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 Mesa County area. The reader is referred to these documents for further discussion on selection of a proper roughness factor. For multiple land uses within a watershed, composite roughness factors should be determined.

**Table 708 Lag Equation Roughness Factors**

| Land Use | Imperviousness % | $K_n$ |
|---|---|---|
| Commercial, industrial, office and business | 70-85 | 0.05 |
| Residential | | |
| Rural | 10-15 | 0.08 |
| Low density | 20-25 | 0.07 |
| Medium and high density | 30-65 | 005 |
| Irrigated grass, golf courses, parks and cemeteries | 0-5 | 0.10 |
| Undeveloped areas: | | |
| Rock outcroppings | | 0.04 |
| Irrigated agriculture | | 0.10 |
| Rangelands: | | |
| Grass lands | | 0.08 |
| Grass and shrubs | | 0.09 |
| Shrub and brush | | 0.10 |
| Forest, evergreen | | 0.15 |

### 706.5 Unit Storm Duration

The minimum unit duration, Δt, is dependent on the time of concentration of a given watershed. As a general rule, the unit storm duration, Δt, should be no greater than $T_c/3$. For small watersheds (i.e., < 1 square mile) the recommended duration is 5 minutes. For larger watersheds, larger unit durations may be used, but should not exceed 15 minutes.

### 706.6 Sub-watershed Sizing

The determination of the peak rate of runoff at a given design point is affected by the number of sub-watersheds within a larger watershed. Typically, the more discrete the analysis of a given watershed (more sub-watersheds), the more representative the resulting peak flow is of actual runoff conditions. The improved predictive capability of multiple sub-watersheds is due to better homogeneity of the sub-watershed characteristics, as compared to analysis of the watershed with no sub-watersheds. Recommended guidelines are:

a. For watersheds up to 100 acres in size, the maximum sub-watershed size should be approximately 20 acres.

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

## 707 CHANNEL ROUTING OF HYDROGRAPHS

Whenever a large or a non-homogeneous watershed is being investigated, the watershed should be divided into smaller and more homogeneous sub-watersheds. The storm hydrograph for each sub-watershed is then routed through the channel and combined with individual sub-watershed hydrographs to develop a storm hydrograph for the entire watershed. For hydrograph routing, the Muskingum, Kinematic Wave, and Muskingum-Cunge methods are recommended for use in the Mesa County area.

The Kinematic Wave method is recommended when the watershed has well defined channels. The Muskingum-Cunge method is recommended for poorly defined channels that have cross sections that can be determined from detailed topography or from a cross section survey, otherwise the Muskingum method is recommended.

### 707.1 Muskingum Method

The Muskingum method estimates some channel storage effects 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:

$$O_2 = (C_1 - C_2)I_1 + (1 - C_1)O_1 + C_2I_2 \qquad (716)$$

Where:

$O_2$ = 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 defined in Equations 717 and 718, respectively.

And

$$C_1 = (2\Delta t)/[2K(1-X)] + \Delta t \qquad (717)$$

$$C_2 = (\Delta t\text{-}2KX)/[2K(1\text{-}X)] + \Delta t \quad (718)$$

$$K = L/(3{,}600V) \quad (719)$$

Where:

$K$ = Muskingum storage time constant in hrs
$L$ = Channel reach length in ft
$V$ = Translation velocity in fps
$\Delta t$ = Unit time increment in hrs
$X$ = Muskingum weighting factor

The velocity used in Equation 719 is 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:

**Wave Velocities for Muskingum Method**

| Channel Shape | Wave Velocity |
|---|---|
| Wide rectangular | 5/8 V |
| Triangular | 4/3 V |
| Wide parabolic | 11/9 V |

Where V is the velocity from application of the Manning's equation.

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.

**Weighting Factors for Muskingum Method**

| Channel Condition | Weighting Factor "X" |
|---|---|
| Some storage in over bank but few obstructions (i.e. alluvial fans) | 0.15 |
| Overbanks with severe obstruction | 0.10 |

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 719. An estimate of the mean channel flow velocity may be obtained using Manning's formula with the hydraulic 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 sub-reaches 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

$$1/[2(1-X)] \leq K/(\Delta T \cdot NSTPS) \leq 1/(2X) \tag{720}$$

Where $\Delta T$ is the time increment in hours.

### 707.2 Kinematic Wave Method

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.

Thus flow at any point in the channel can be computed from Manning's formula:

$$Q = (1.486/n)\, R^{2/3}\, S_o^{1/2}\, A \tag{721}$$

Where:
- $Q$ = Flow rate, cfs,
- $R$ = Hydraulic radius, ft
- $S_o$ = Channel bed slope, ft/ft.
- $A$ = Cross sectional area, square feet,
- $n$ =Manning's resistance factor

Equation 721 can be simplified to:

$$Q = \alpha A^m \tag{722}$$

Where $\alpha$ and $m$ are related to flow geometry and surface roughness. This is the form of the equation used to route hydrographs through channels.

The Kinematic Wave method in HEC-1 does not allow for explicit separation of main channel and overbank areas. Therefore, a flood wave routed by the Kinematic Wave technique through a channel reach is translated, but does not attenuate (although a small 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.

### 707.3 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.

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 reasonableness. 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.

## 708 RESERVOIR ROUTING OF HYDROGRAPHS

Storm runoff detention is required for new development (Section 403.7) and therefore detention reservoirs will be required (see Section 1400). 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 modified Puls methodology for reservoir routing, which is computerized in the HEC-1 and other programs, is recommended.

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 Hydrology for Engineers (Linsley, 1975).

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

$$(I-D)\Delta t = \Delta S \qquad (723)$$

I is the inflow rate, D is the discharge rate, Δt 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:

$$(I_1 + I_2)\,\Delta t / 2 - (D_1 + D_2)\,\Delta t / 2 = S_2 - S_1 \qquad (724)$$

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:

$$I_1 + I_2 + ({}_2S_1 / \Delta t - D_1) = ({}_2S_2 / \Delta t + D_2) \qquad (725)$$

# RECOMMENDED IMPERVIOUSNESS VALUES

| Land Use or Surface Characteristic | Percentage Imperviousness |
|---|---|
| Business | |
| Commercial Areas | 85 |
| Neighborhood Areas | 70 |
| Residential | |
| Single Family | (see figures) |
| Multi-unit (detached) | 60 |
| Multi-unit (attached) | 75 |
| Half-acre lot or larger | (see figures) |
| Apartments | 80 |
| Industrial | |
| Light industrial | 80 |
| Heavy industrial | 90 |
| Parks, cemeteries | 5 |
| Playgrounds | 10 |
| Schools | 50 |
| Railroad yards | 15 |
| Undeveloped Areas | |
| Historic flow analysis | 2 |
| Greenbelts, agriculture | 2 |
| Off-site flow analysis (when land use not defined) | 45 |
| Streets | |
| Paved (concrete/asphalt) | 100 |
| Gravel | 40 |
| Drives and walks | 90 |
| Roofs | 90 |
| Lawns (all soils) | 0 |

NOTES:
1. The imperviousness values are representative of land uses shown and are for future development projections only. Impervious values for existing land uses may vary.
2. For areas that will not be developed, 2% imperviousness is an appropriate assumption where soil and vegetative cover are present. Areas with geological features, including significant rock outcroppings, need to be accounted for. See Section 702.2.

| Revision | Date |
|---|---|
| ORIGINAL ISSUE | 3/27/06 |
| CHANGED BUS. VALUES | 12/6/07 |
| ADDED NOTE 2 | 1/25/08 |



REFERENCE:
UDFCD 2001. Urban Storm Drainage Criteria Manual, Volume 1 (revised)

# RATIONAL FORMULA RUNOFF COEFFICIENTS

**Equation:**

$$C_{CD} = K_{CD} + (0.858i^3 - 0.786i^2 + 0.774i + 0.04$$
$$C_A = K_A + (1.31i^3 - 1.44i^2 + 1.135i - 0.12)$$
$$C_B = (C_A + C_{CD})/2$$

| KCD VALUES | | | | | | |
|---|---|---|---|---|---|---|
| NRCS Soil | 2-year | 5-year | 10-year | 25-year | 50-year | 100-year |
| C and D | 0 | -0.10i+0.11 | -0.18i+0.21 | -0.28i+0.33 | -0.33i+0.40 | -0.39i+0.46 |
| A | 0 | -0.08i+0.09 | -0.14i+0.17 | -0.19i+0.24 | -0.22i+0.28 | -0.25i+0.32 |

| Impervious Decimal | Type A | | | | | |
|---|---|---|---|---|---|---|
| | 2-year | 5-year | 10-year | 25-year | 50-year | 100-year |
| 0.0 | 0.00 | 0.00 | 0.08 | 0.14 | 0.18 | 0.23 |
| 0.1 | 0.00 | 0.06 | 0.14 | 0.20 | 0.24 | 0.28 |
| 0.2 | 0.06 | 0.13 | 0.20 | 0.26 | 0.30 | 0.33 |
| 0.3 | 0.13 | 0.19 | 0.25 | 0.31 | 0.34 | 0.37 |
| 0.4 | 0.19 | 0.25 | 0.30 | 0.35 | 0.38 | 0.41 |
| 0.5 | 0.25 | 0.30 | 0.35 | 0.40 | 0.42 | 0.45 |
| 0.6 | 0.33 | 0.37 | 0.41 | 0.45 | 0.47 | 0.50 |
| 0.7 | 0.42 | 0.45 | 0.49 | 0.53 | 0.54 | 0.56 |
| 0.8 | 0.54 | 0.56 | 0.60 | 0.63 | 0.64 | 0.66 |
| 0.9 | 0.69 | 0.71 | 0.73 | 0.76 | 0.77 | 0.79 |
| 1.0 | 0.89 | 0.90 | 0.92 | 0.94 | 0.95 | 0.96 |

| Impervious Decimal | Type B | | | | | |
|---|---|---|---|---|---|---|
| | 2-year | 5-year | 10-year | 25-year | 50-year | 100-year |
| 0.0 | 0.00 | 0.08 | 0.17 | 0.27 | 0.32 | 0.36 |
| 0.1 | 0.06 | 0.14 | 0.22 | 0.31 | 0.36 | 0.40 |
| 0.2 | 0.12 | 0.20 | 0.27 | 0.35 | 0.40 | 0.44 |
| 0.3 | 0.18 | 0.25 | 0.32 | 0.39 | 0.43 | 0.47 |
| 0.4 | 0.23 | 0.30 | 0.36 | 0.42 | 0.46 | 0.50 |
| 0.5 | 0.29 | 0.35 | 0.40 | 0.46 | 0.50 | 0.52 |
| 0.6 | 0.37 | 0.41 | 0.46 | 0.51 | 0.54 | 0.56 |
| 0.7 | 0.45 | 0.49 | 0.53 | 0.58 | 0.60 | 0.63 |
| 0.8 | 0.57 | 0.59 | 0.63 | 0.66 | 0.69 | 0.70 |
| 0.9 | 0.71 | 0.73 | 0.75 | 0.78 | 0.80 | 0.81 |
| 1.0 | 0.89 | 0.90 | 0.92 | 0.94 | 0.95 | 0.96 |

| Impervious Decimal | Type C and D Soil | | | | | |
|---|---|---|---|---|---|---|
| | 2-year | 5-year | 10-year | 25-year | 50-year | 100-year |
| 0.0 | 0.05 | 0.16 | 0.26 | 0.38 | 0.46 | 0.51 |
| 0.1 | 0.11 | 0.21 | 0.30 | 0.41 | 0.48 | 0.53 |
| 0.2 | 0.17 | 0.26 | 0.34 | 0.44 | 0.50 | 0.55 |
| 0.3 | 0.22 | 0.30 | 0.38 | 0.47 | 0.53 | 0.57 |
| 0.4 | 0.28 | 0.35 | 0.42 | 0.50 | 0.55 | 0.58 |
| 0.5 | 0.34 | 0.40 | 0.46 | 0.53 | 0.57 | 0.60 |
| 0.6 | 0.41 | 0.46 | 0.51 | 0.57 | 0.61 | 0.63 |
| 0.7 | 0.49 | 0.53 | 0.57 | 0.62 | 0.66 | 0.68 |
| 0.8 | 0.60 | 0.63 | 0.66 | 0.70 | 0.73 | 0.74 |
| 0.9 | 0.73 | 0.75 | 0.77 | 0.80 | 0.83 | 0.83 |
| 1.0 | 0.89 | 0.90 | 0.92 | 0.94 | 0.96 | 0.96 |

| Revision | Date |
|---|---|
| ORIGINAL ISSUE | 3/27/06 |
| ADDED 0% IMPERVIOUS | 12/6/07 |
| CORRECTED FORMULA | 1/25/08 |

WRC ENGINEERING, INC.

REFERENCE:

UDFCD 2001. Urban Storm Drainage Criteria Manual, Volume 1 (revised)

**TABLE 702**

# RUNOFF CURVE NUMBERS

| Land Use or Surface Characteristic | Average Imperv. (%) | Runoff Curve Number Soil Complex | | | |
|---|---|---|---|---|---|
| | | A | B | C | D |
| Business | | | | | |
| Commercial Areas | 85 | 89 | 92 | 94 | 95 |
| Neighborhood Areas | 70 | 80 | 87 | 91 | 93 |
| Residential | | | | | |
| Single Family (note 1) | (note 1) | | | | |
| Multi-unit (detached) | 60 | 74 | 83 | 88 | 91 |
| Multi-unit (attached) | 75 | 83 | 89 | 92 | 94 |
| Apartments | 80 | 86 | 91 | 93 | 94 |
| Industrial | | | | | |
| Light | 80 | 86 | 91 | 93 | 94 |
| Heavy | 90 | 92 | 94 | 96 | 96 |
| Parks, cemeteries | 5 | 42 | 63 | 75 | 81 |
| Playgrounds | 10 | 45 | 65 | 76 | 82 |
| Schools | 50 | 69 | 80 | 86 | 89 |
| Railroad yards | 15 | 48 | 67 | 78 | 83 |
| Irrigated Areas | | | | | |
| Lawns, parks, golf course | 0 | 39 | 61 | 74 | 80 |
| Agriculture | 0 | 39 | 61 | 74 | 80 |
| Undeveloped Areas | | | | | |
| Pre-development conditions | 2 | 40 | 62 | 74 | 80 |
| Greenbelts, agriculture | 2 | 40 | 62 | 74 | 80 |
| Off-site analysis when land use Unknown | 45 | 66 | 78 | 85 | 88 |
| Outcrops | 70 | 80 | 87 | | 94 |
| Streets/Roads | | | | | |
| Paved | 100 | 98 | 98 | 98 | 98 |
| Gravel | 40 | 63 | 76 | 84 | 87 |
| Drives/Walks | 90 | 92 | 94 | 96 | 96 |
| Roofs | 90 | 92 | 94 | 96 | 96 |

**NOTE:**
**ESTIMATE IMPERVIOUS FROM FIGURES 703, 704, 705. THEN COMPUTE CURVE NUMBER, CN, FROM EQUATION 708, BASED ON NRCS SOILS TYPE. USE OF THIS TABLE IS LIMITED TO EVALUATION OF IMPERVIOUSNESS FOR FUTURE DEVELOPMENT PROJECTIONS WITHIN REGIONAL WATERSHED MASTER PLANS, OR IN CONCEPTUAL DRAINAGE PLANS.**

| Revision | Date |
|---|---|
| ORIGINAL ISSUE | 3/27/06 |
| | |
| | |

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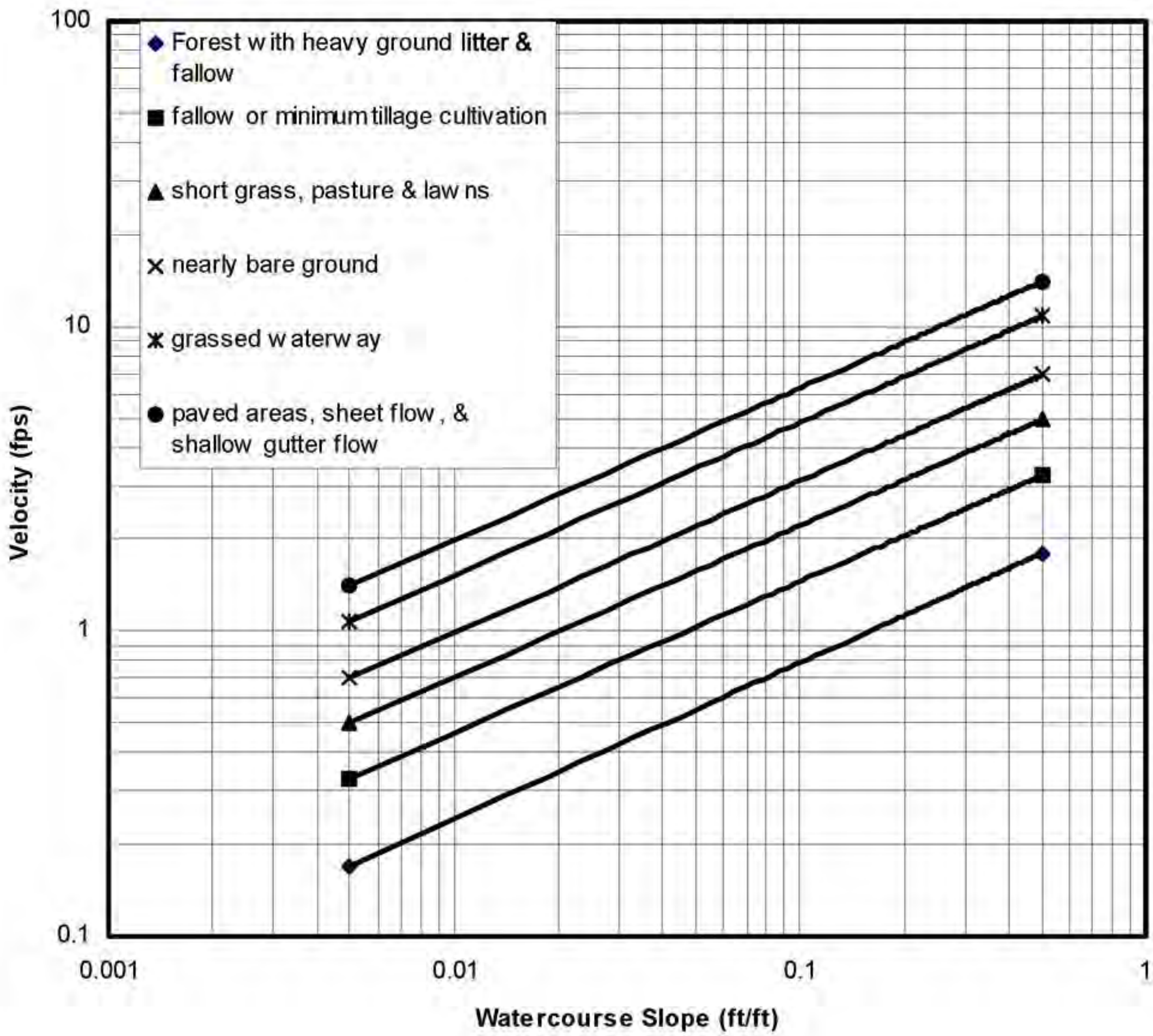
REFERENCE:
SCS TECHNICAL RELEASE NO. 55 (1986)

**TABLE 704**

# TRAVEL VELOCITY FOR RATIONAL METHOD




| Revision | Date |
|---|---|
| ORIGINAL ISSUE | 3/27/06 |
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WRC ENGINEERING, INC.

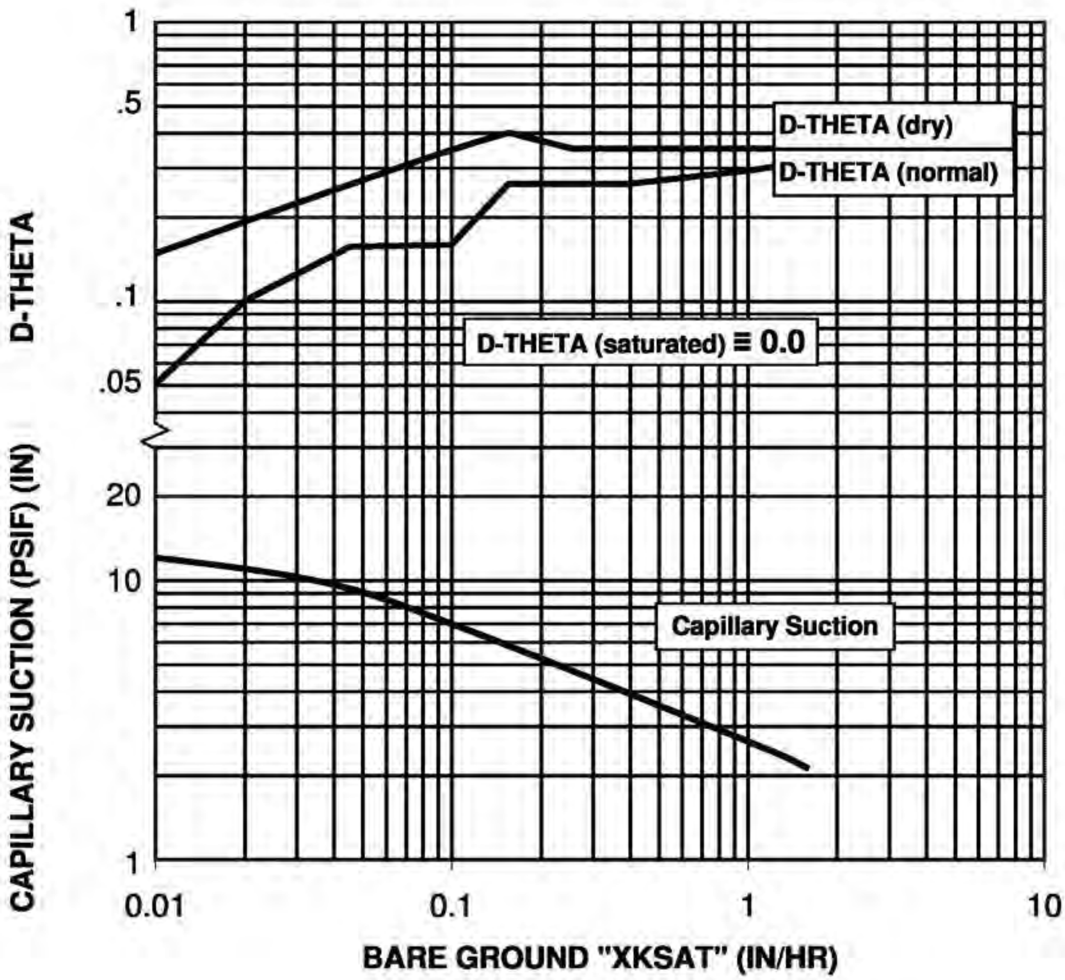
REFERENCE:
Adapted from USDA, SCS 1975. Urban Hydrology for Small Watersheds, TR 55

**FIGURE 701**

# ADJUSTMENT FOR SOIL MOISTURE DEFICIT AND CAPILLARY ACTION

SELECTION OF DTHETA CONDITION:

DRY – USE THIS FOR NON–IRRIGATED LANDS, SUCH AS DESERT AND RANGELAND.
NORMAL – USE THIS FOR IRRIGATED LAWN, TURF, AND PERMANENT PASTURE.
SATURATED – USE THIS FOR IRRIGATED AGRICULTURAL LAND.




NOTE:

DO NOT USE "XKSAT" VAU=LUES THAT HAVE BEEN ADJUSTED FOR VEGETATION COVER.

| Revision | Date |
|---|---|
| ORIGINAL ISSUE | 3/27/06 |
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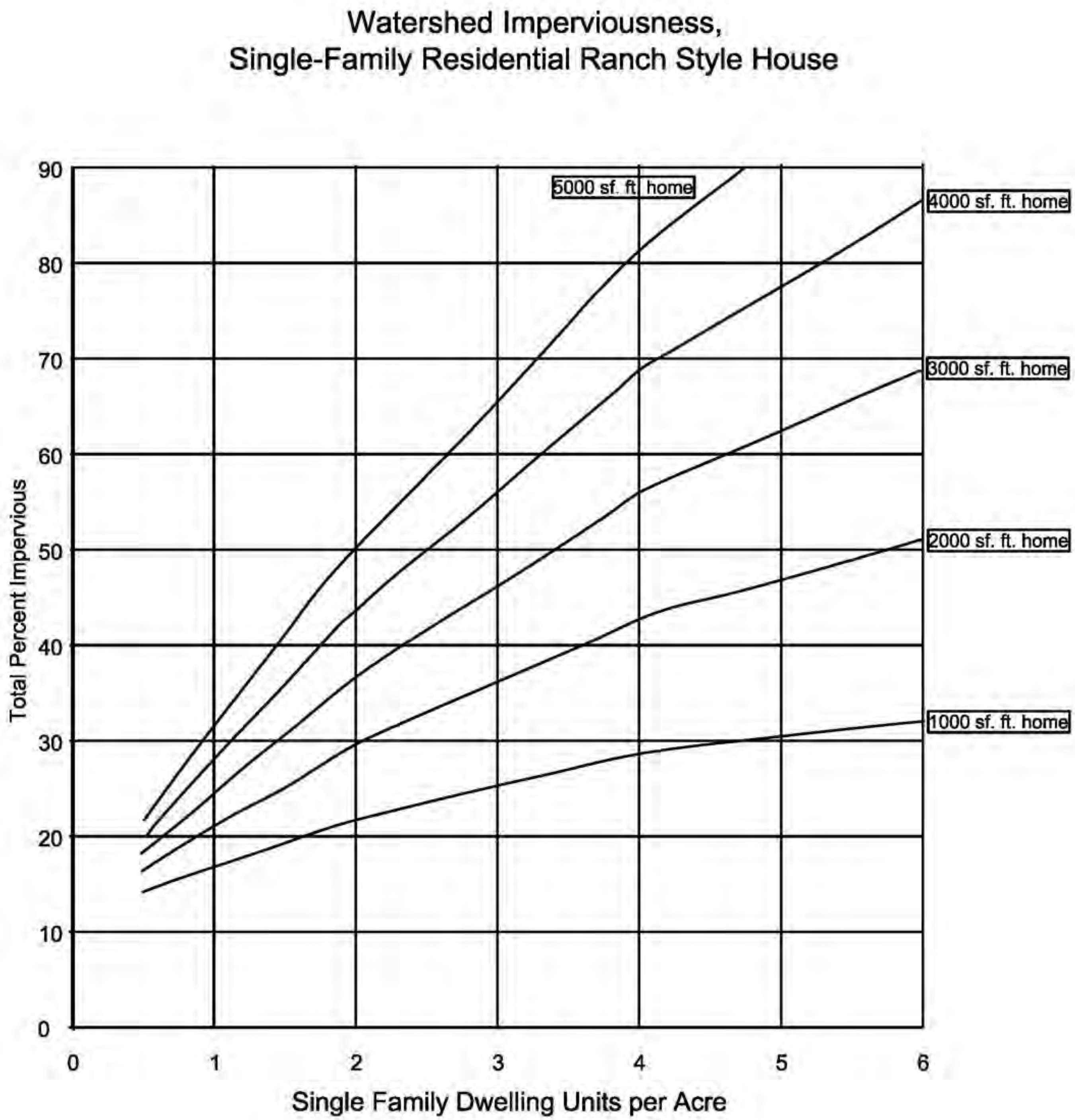


| Revision | Date |
|---|---|
| ORIGINAL ISSUE | 3/27/06 |
| | |
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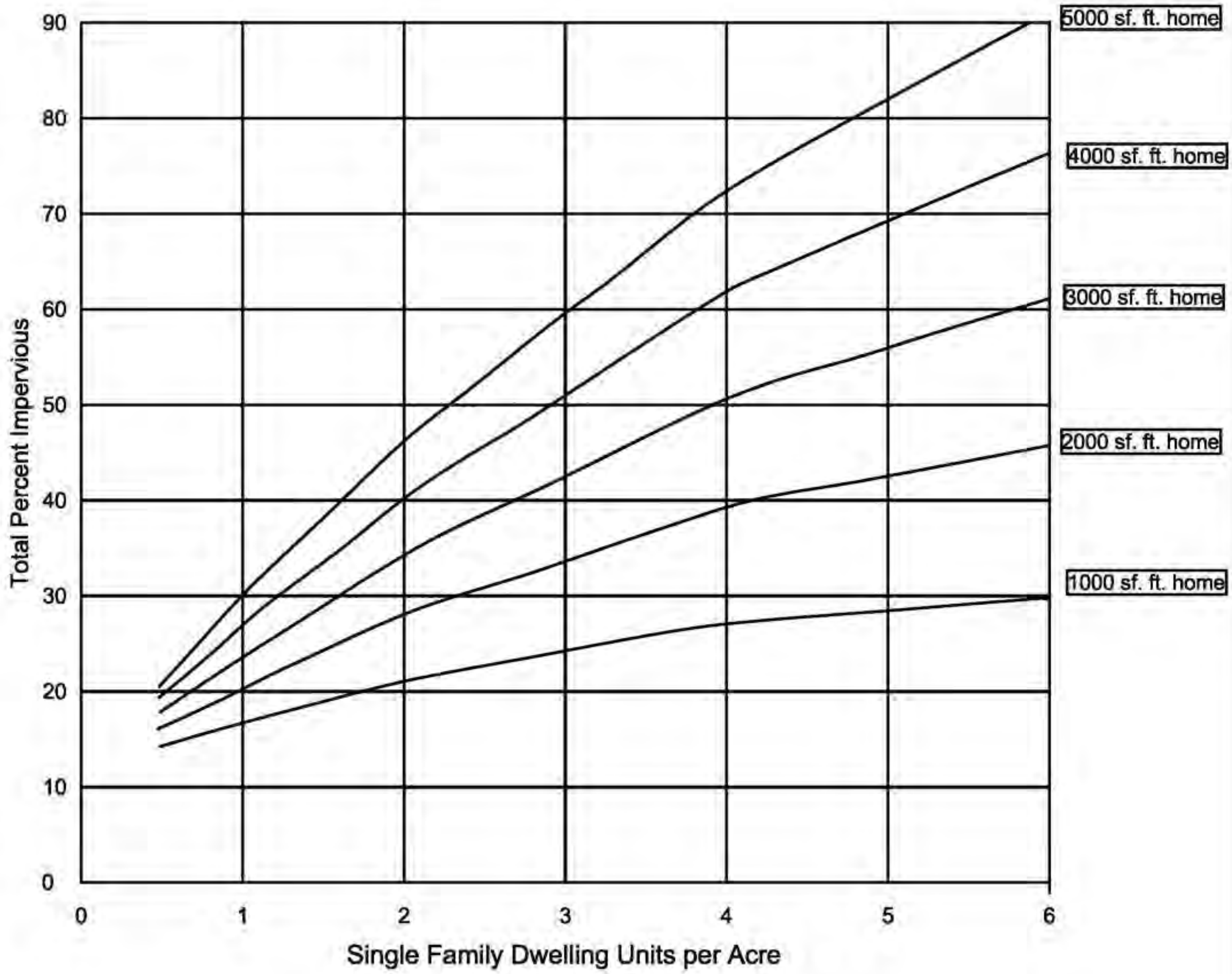
REFERENCE:
UDFCD 2001. Urban Storm Drainage Criteria Manual, Volume 1

**FIGURE 703**

## Watershed Imperviousness, Single-Family Residential Split-Level House




| Revision | Date |
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| ORIGINAL ISSUE | 3/27/06 |
| | |
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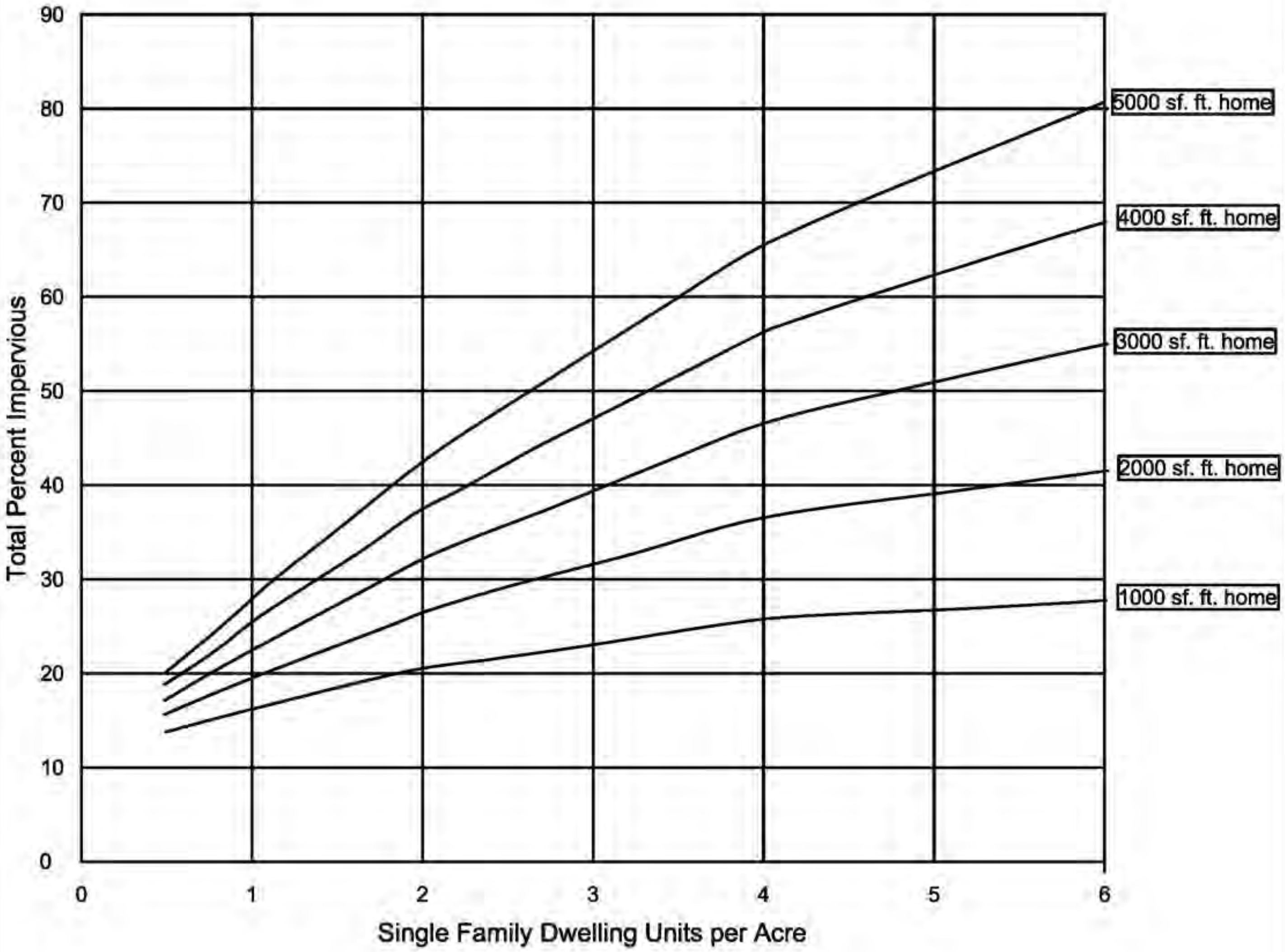
REFERENCE:
UDFCD 2001. Urban Storm Drainage Criteria Manual, Volume 1

**FIGURE 704**

## Watershed Imperviousness, Single-Family Residential Two Story House




| Revision | Date |
|---|---|
| ORIGINAL ISSUE | 3/27/06 |
| | |
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REFERENCE:
UDFCD 2001. Urban Storm Drainage Criteria Manual, Volume 1

**FIGURE 705**

# SECTION 800

## OPEN CHANNELS

# SECTION 800
## OPEN CHANNELS

## TABLE OF CONTENTS

## LIST OF TABLES



## LIST OF FIGURES

# SECTION 800
# OPEN CHANNELS

## 801 INTRODUCTION

Presented in this Section are the technical criteria and design standards for hydraulic evaluation and design of open channels (natural and artificial). Open channel flow can be extremely complex, and often entire textbooks are devoted to the subject. Discussions and hydraulic standards are provided for different channel types that are most likely to be found in Mesa County.

The ultimate responsibility for the design of a safe and stable channel rests with the professional design engineer. A good understanding of the site conditions is vital to the production of a stable channel design. The information presented in this Section must be considered to be the minimum standards upon which channel evaluation and design shall be based. Additional analysis that goes beyond the scope of this Manual may be necessary for unique or unusual channel conditions. For additional information, the users of this Manual are encouraged to refer to other textbooks and technical publications addressing this subject.

## 802 PARAMETERS AND TERMS COMMONLY USED IN OPEN CHANNEL ANALYSIS AND DESIGN

Several terms and parameters are used and must be understood when analyzing open channel flows. These are described below.

**Area (A)** The area always means the cross-sectional area of the flow, and is measured perpendicular to the direction of flow.

**Wetted Perimeter (Pw)** The wetted perimeter is the portion of the perimeter of a flow conveyance facility that is in contact with the flowing water.

**Hydraulic Radius (Rh)** The hydraulic radius is the cross-sectional area of flow divided by the wetted perimeter, or Rh=A/Pw.

**Depth (d)** If not specified otherwise, depth of flow refers to the maximum depth of water in the cross-section.

**Surface Spread (T)** The surface spread is the width at the top of the flow, measured perpendicular to the flow direction.

**Hydraulic Depth (Dh)** The hydraulic depth is the ratio of area in flow to the width of the channel at the fluid surface, or Dh=A/T.

**Slope (S)** Slope may refer to the channel bed, the hydraulic grade line, or energy grade line.

**Hydraulic Grade Line (HGL)** In an open channel, the hydraulic grade line is the profile of the free water surface.

**Hydraulic Gradient (Hg)** The slope of the hydraulic grade line is the profile of the free water surface.

**Energy Grade Line (EGL)** The grade line of the water surface profile plus the velocity head, or the specific energy line.

**Critical Flow** This refers to flow at critical depth or velocity, where the specific energy is a minimum for a given discharge. Critical flow is very unstable.

**Critical Depth (dc)** This refers to the depth of flow under critical flow conditions.

**Critical Velocity** This refers to the velocity of flow under critical flow conditions.

**Critical Slope** This refers to the slope which, for a given cross-section and flow rate, results in critical flow.

**Froude Number (Fr)** This is a dimensionless number, equal to the ratio of the velocity of flow to the velocity of very small gravity waves, the latter being equal to the square root of the product of the acceleration of gravity and the flow depth, or

$$Fr = \frac{V}{\left(\frac{gA}{T}\right)^{0.5}} = \frac{V}{(gDh)^{0.5}} \qquad (801)$$

Where:

Fr<1.0, flow is subcritical;
Fr = 1.0, flow is critical; and
Fr>1.0, flow is supercritical
V = velocity (fps)
A = cross-sectional area of flow (sf)
T = top width of flow (ft)
Dh = hydraulic depth (ft)

**Normal Depth** When the flow depth is constant along a channel reach; that is, when neither the flow depth nor velocity is changing, the depth is said to be normal.

**Uniform Flow** Uniform flow occurs when flow has a constant water area, depth, discharge, and average velocity through a reach of channel.

**Gradually Varied Flow** Varied flow in which the depth does not change abruptly over a comparatively short distance.

## 803 GENERAL OPEN CHANNEL FLOW

Any water flow that is conveyed in such a manner that top surface is exposed to the atmosphere is defined as open channel flow. This type of flow occurs in all channel types described in Section 804 of this Manual including canals, ditches, drainage channels, culverts, and pipes under partially full flow conditions. The hydraulics of an open channel can be very complex, encompassing many different flow conditions from steady-state uniform flow to unsteady, rapidly varying flow. Most of the problems in storm water drainage involve uniform, gradually varying or rapidly varying flow states. Examples of these flow conditions are illustrated in **Figure 801**. Steady uniform flow is most commonly treated flow in open channel hydraulics, in which the depth of flow remains constant over the time interval studied. The calculations for uniform and gradually varying flow are relatively straight forward and are based upon similar assumptions (e.g., parallel streamlines). Rapidly varying flow computations (e.g., 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 written on this subject.

### 803.1 Uniform Flow Computation

Open-channel flow is uniform if the depth of flow is the same at every section of the channel. For a given channel geometry, roughness, discharge and slope, there is only one possible depth for maintaining uniform flow. This depth is referred to as the "normal depth." For uniform flow within a prismatic channel (i.e., uniform cross section), the water surface will be parallel to the channel bottom. While uniform flow rarely occurs in nature and is difficult to achieve in a laboratory, a uniform-flow approximation is generally adequate for planning and design purposes.

The computation of uniform flow and normal depth shall be based upon the Manning or Uniform Flow Equation:

$$Q = \frac{1.49}{n} A^{5/3} P^{-2/3} \sqrt{S} = \frac{1.49}{n} A R^{2/3} \sqrt{S} \qquad (802)$$

Where:

- Q = flow rate ($ft^3/s$);
- n = Manning roughness coefficient;
- A = area ($ft^2$);
- P = wetted perimeter (ft);
- R = hydraulic radius, R = A/P (ft); and
- S = slope of the energy grade line (ft/ft).

For prismatic channels, the energy grade line (EGL), hydraulic grade line (HGL), and the bottom can be assumed parallel for uniform, normal depth flow conditions.

The variables dependent on channel cross-section geometry (i.e., area and hydraulic radius) can be lumped together as the *conveyance* (K) of the channel. This simplifies the Uniform Flow Equation to the following expression:

$$Q = K\sqrt{S} \qquad (803)$$

**Table 801** presents equations for calculating many of the parameters required for hydraulic analysis of different uniform channel sections.

**Tables 802A, 802B, 802C, and 802D** provide a list of Manning roughness coefficient values for many types of conditions that may occur in Mesa County. The Uniform Flow Equation and its constituent parameters are readily computed using handheld calculators and personal computers.

803.1.1 Gradually Varying Flow

The most common occurrence of gradually varying flow in storm drainage is the backwater created by culverts, storm drain inlets, or channel constrictions. For these conditions, the flow depth will be greater than normal depth in the channel, and the water surface profile (a.k.a. "backwater curve") is computed using either the direct-step or standard step method.

803.1.1.1 Direct-Step Method

The Direct-Step Method is best suited to the analysis of open prismatic channels. Water surface profiles in simple prismatic channels can be computed manually. Chow (1959) presents the basic method for applying the direct-step analysis. The Direct-Step Method is also available in many handheld and personal computer programs. The most general and widely used programs are the U.S. Army Corps of Engineers' *HEC-2 Water Surface Profiles* and *HEC-RAS River Analysis System*. The design engineer may use these programs or proprietary computer software to compute water surface profiles for channel and floodplain analyses.

803.1.1.2 Standard-Step Method

The Standard-Step Method is required for the analysis of irregular or non-uniform cross-sections. Because the Standard Step Method involves a more tedious iterative process, this Manual recommends that design engineers use computer programs such as HEC-RAS to accomplish these calculations.

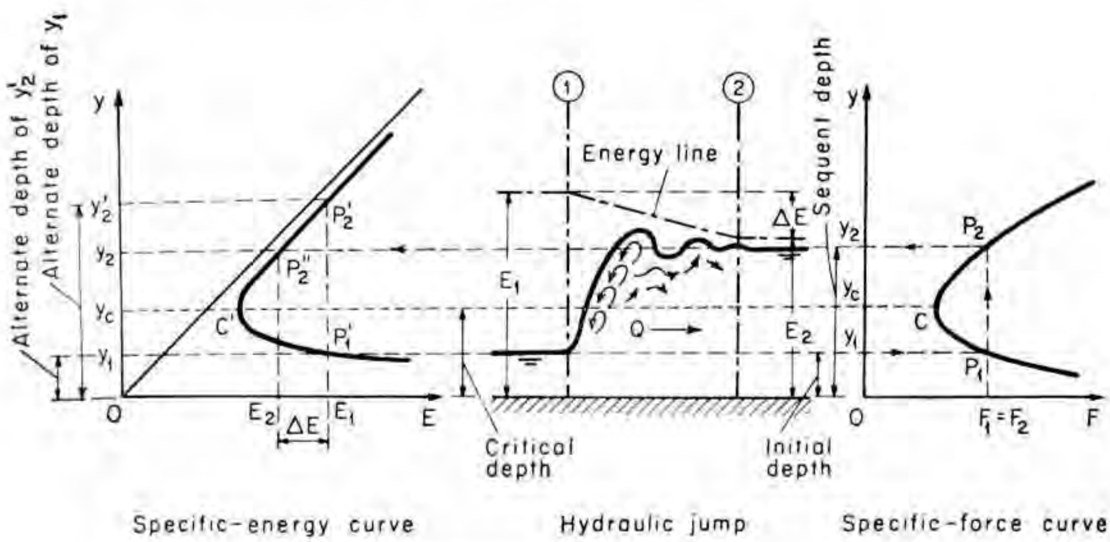



**Figure 802** Specific Energy Curve

803.1.2 Rapidly Varying Flow

Rapidly varying flow is characterized by very pronounced curvature of the water surface profile. The change in water surface profile may become so abrupt to result in a state of high turbulence. Calculation methods for gradually-varying flow (e.g., direct-step and standard-step methods) do not apply for rapidly-varying flow. There are mathematical solutions to some specific cases of rapidly varying flow, but the solutions to most rapidly varying flow problems rely on empirical data.

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

### 803.2 Critical Flow Computation

The critical flow through a channel is characterized by several important conditions regarding the relationship between the flow, specific energy, and slope of a particular hydraulic cross-section (**Figure 802**). Critical state is characterized by the following conditions.

1. The specific energy ($E=y+v^2/2g$) is at a minimum for a given discharge (Q).
2. The discharge (Q) is a maximum for a given specific energy (E).
3. The specific force is a minimum for a given discharge (Q).
4. The velocity head ($v^2/2g$) is equal to half the hydraulic depth (D/2) in a channel of small slope.
5. The Froude Number (Fr) is equal to 1.0.

**Typically, channels must not be designed to flow at or near critical state (0.80<Fr<1.2).** 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 subcritical flow. A slope steeper than $S_c$ will cause supercritical flow. A flow at or near the critical state is unstable. Factors creating minor changes in specific energy, such as channel debris or minor variation in roughness, will cause a major change in depth.

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

$$Fr = \frac{v}{\sqrt{gD}} \qquad (804)$$

Where:

- Fr = Froude number (dimensionless)
- v = velocity (ft/s)
- g = gravitational acceleration (32.2 ft/$S^2$)
- A = channel flow area (ft$^2$)
- T = top width of flow area (ft)
- D = hydraulic depth, D=A/T (ft)

The critical depth in a given trapezoidal channel section with a known flow rate can be determined using the following method:

**Step 1.** Compute the section factor (Z).

$$Z = \frac{Q}{\sqrt{g}} \tag{805}$$

Where:

$Z$ = section factor
$Q$ = flow rate (cfs)
$g$ = gravitational acceleration (32.2 ft/s$^2$)

**Step 2.** Determine the critical depth in the channel ($d_c$) from **Figure 803**, using appropriate values for the section factor (Z), the channel bottom width (b), and the channel side slope (z).

For other prismatic channel shapes, Equation 805 determines the critical depth using the section factors provided in **Table 801**.

### 803.3 Design Procedures – Subcritical Flow

All open channels shall be designed with the limits as stated in Sections 806 through 813. The following design procedures shall be used when the design runoff in the channel is flowing in a subcritical condition (Fr<1.0).

803.3.1 Transitions

Subcritical transitions occur when transitioning one subcritical channel section to another subcritical channel section (expansion or contraction), or when a subcritical channel section is steepened to create a supercritical flow condition downstream (e.g., a sloping spillway entrance).

**Figure 804** presents a number of typical subcritical transition sections. The warped transition section, although most efficient, shall 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 shall only be used when either a straight-line transition or a cylinder-quadrant transition cannot be used due to topographic constraints or utility conflicts.

803.3.2 Contractions

The energy loss created by a contracting section may be calculated using the following equation:

$$H_t = K_{tc}\left(\frac{V_2^2}{2g} - \frac{V_1^2}{2g}\right) \tag{806}$$

Where:

$H_t$ = energy loss (ft);
$K_{tc}$ = contraction transition coefficient
$V_1$ = upstream velocity (ft/s);
$V_2$ = downstream velocity (ft/s); and
$g$ = gravitational acceleration (32.2 ft/s$^2$).

**Figure 804** shows contraction loss coefficient ($K_{tc}$) values for the typical open-channel transition section.

803.3.3 Expansions

The energy loss created by an expanding transition section may be calculated using the following equation:

$$H_t = K_{te}\left(\frac{V_1^2}{2g} - \frac{V_2^2}{2g}\right) \qquad (807)$$

**Figure 804** also shows expansion loss coefficients ($K_{te}$) values for typical open-channel transition sections.

803.3.4 Transition Length

The length of the transition section shall be long enough to keep the streamlines smooth and nearly parallel throughout the expanding (contracting) section. Experimental data and performance of existing structures have been used to estimate the minimum transition length necessary to maintain the stated flow conditions. Based on this information, guidelines for the minimum length of the transition section are as follows:

$$L_t \geq 0.5 L_c (\Delta T_w) \qquad (808)$$

Where:

$L_t$ = minimum transition length (feet);
$L_c$ = length coefficient (dimensionless); and
$\Delta T_w$ = difference in the top width of the normal water surface upstream and downstream of the transition (feet).

**Table 803** below summarizes the transition length coefficients for subcritical flow conditions. These transition length guidelines are not applicable to cylinder-quadrant or square-ended transitions.

For flow approach velocities of 12 ft/sec or less, the transition length coefficient ($L_c$) shall be 4.5. This represents a 4.5L:1W expansion or contraction, or about a 12.5-degree divergence from the channel centerline. For flow approach velocities of more than 12 ft/sec, the transition length coefficient ($L_c$) shall be 10. This represents a 10L:1W expansion or contraction, or about a 5.75-degree divergence from the channel centerline.

**Table 803 Transition Length Coefficients for Subcritical Open Channels**

| Flow Approach Velocity (v) (ft/s) | Transition Length Coefficient ($L_c$) |
|---|---|
| = 12 | 4.5 |
| > 12 | 10 |

## 803.4 Design Procedures – Supercritical Flow

Mesa County and the City of Grand Junction do not encourage supercritical channels, which typically are concrete-lined. The information presented herein is for completeness and in anticipation that analysis of a supercritical channel may be necessary in the future. All supercritical channels shall be designed within the limits as stated in Sections 806 through Section 813. The following design procedures shall be used when channels are designed to flow in a supercritical condition (Fr>1.0).

Supercritical flow can become unstable in response to relatively minor disturbances to the channel cross-section; even small obstruction can sometimes cause a hydraulic jump. Good design practice is to test supercritical flow stability during events smaller than the design flow by evaluating flows of specific more frequent storm events (e.g., 10-year, 2-year, etc.), or testing successive fractions (e.g., one half, one quarter, and further if necessary) of the design flow. Also, the designer shall test for small variations in n-value as well. However, only calculations for the full design flow are required to be submitted for review.

### 803.4.1 Transitions

The design of supercritical flow transitions is more complicated than subcritical transition design due to the potential damaging effects of the oblique jump created by the transition. The oblique jump results in cross waves and higher flow depths that can cause damage if not properly accounted for in the design. Supercritical transitions can be avoided by designing a hydraulic jump, which must also be carefully designed to assure the jump will remain where the jump is designed to occur. Hydraulic jumps shall be designed to take place only within concrete-lined portions of the channel, such as energy dissipation or drop structures.

### 803.4.2 Contractions

**Figure 805** presents an example of a supercritical contracting transition, with upstream flow contracted from width $b_1$ to $b_3$ and a wall diffraction angle of θ. The oblique jump occurs at the points A and B where the diffraction angles start. Wave fronts generated by the oblique jumps on both sides propagate toward the centerline with a wave angle ß1. Since the flow pattern is symmetric, the centerline acts as if there was a solid wall that causes a subsequent oblique jump and generates a backward wave front toward the wall with another angle ß2. These continuous oblique jumps result in turbulent fluctuations in the water surface.

To minimize the turbulence, the first two wave fronts are designed to meet at the center and then end at the exit of the contraction. Using the contraction geometry, the length of the transition shall be as follows:

$$L_T = \frac{b_1 - b_3}{2\tan\theta} \qquad (809)$$

Where:

- $L_T$ = transition length (ft);
- $b_1$ = upstream top width of flow (ft);
- $b_3$ = downstream top width of flow (ft);

$\theta$ = wall angle as related to the channel centerline (degrees).

Using the continuity principle,

$$\frac{b_1}{b_3} = \left(\frac{y3}{y1}\right)^{3/2}\left(\frac{F_{R_3}}{F_{R_1}}\right) \tag{810}$$

Where:

$y_1$ = upstream depth of flow (ft)
$y_3$ = downstream depth of flow (ft)
$F_{R1}$ = upstream Froude number
$F_{R3}$ = downstream Froude number

Also, by the continuity and momentum principals, the following relationship between the Froude number, wave angle, and wall angle is:

$$\tan\theta = \frac{\tan\beta_1\left[\left(1+8F_{R1}^2\sin^2\beta_1\right)^{1/2}-3\right]}{2\tan^2\beta_1+\left(1+8F_{R1}^2\sin^2\beta_1\right)^{1/2}-1} \tag{811}$$

Where:

ß1 = Initial wave angle (degrees)

By trial and error, this design procedure can be used to determine the transition length and wall angle. **Figure 806** offers a faster solution than trial and error using Equation 810 and Equation 811 (above). **Figure 806** can also be used to determine the wave angle (ß), or may be used with the equations to determine the required downstream depth or width parameter if a certain transition length is desired or required.

To minimize the length of the transition section, the ratio of downstream and upstream flow depths should generally be greater than 2.0 and less than 3.0 ($2.0<y_3/y_1<3.0$). The downstream Froude number should generally be greater than 1.7 to help avoid undulating hydraulic jumps downstream. For further discussion on oblique jumps and supercritical contractions, refer to Chow (1959).

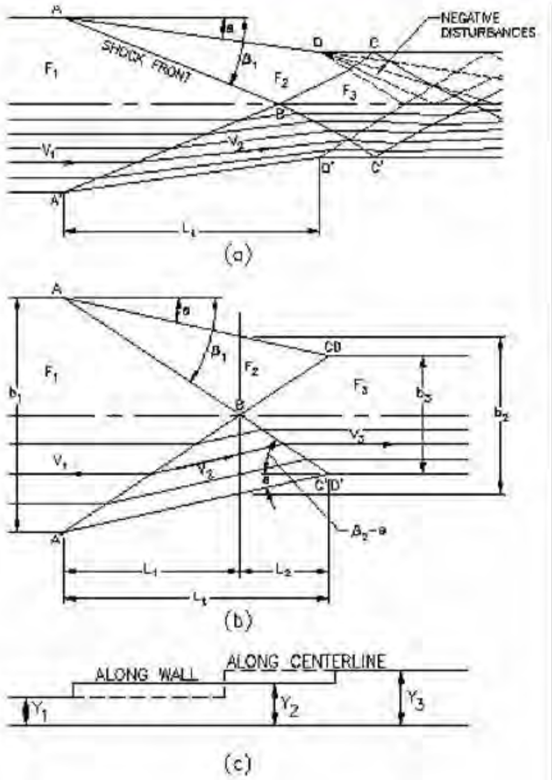



**Figure 805** Supercritical Contraction Transition and Angle Definitions

803.4.3 Expansions

A properly designed expansion transition expands the flow boundaries at approximately the same rate as the natural flow expansion. Based on experimental and analytical data results, the minimum length of a supercritical expansion shall be as follows:

$$L_t \geq 1.5(\Delta W)Fr \qquad (812)$$

Where:

$L_t$ = minimum transition length (ft)

$\Delta W$ = difference in the top width of the normal water surface upstream and downstream of the transition

Fr = Upstream Froude number

803.4.4 Transition Curves

A transition curve may be used to reduce the required amount of freeboard or radius of curvature in a rectangular channel. The length of the transition curve measured along the channel centerline shall be determined as follows:

$$L_c = 2D = 0.32\frac{WV}{\sqrt{y}} \qquad (813)$$

Where:

$L_c$ = length of transition curve (ft);

D = distance from the start of curve to point of first maximum superelevation (ft). Typically $D=3L_W$; see description of how to apply superelevation allowance in Section 803.5;

W = top width of design water surface (ft);

V = mean design velocity (ft/s); and

y = depth of design flow (ft).

The radius of the transition curves shall be twice the radius of the main bend. Transition curves shall be located both upstream and downstream of the main bend.

803.4.5 Slug Flow and Roll Waves

Steep channels with significantly rapid flows (Fr>2.0) are prone to developing pulsating flow profiles, often called slug flows or roll waves. These standing waves can cause flow to exceed freeboard limits and possible damage to the channel lining. The design engineer may resolve pulsating flow issues either by adjusting the channel slope to prevent the development of these waves or providing additional freeboard to account for the height of the standing waves.

Theoretically, slug flow will not occur when the Froude number is less than 2.0. To avoid slug flow when the Froude number is greater than 2.0, the channel slope shall be as follows:

$$S \leq \frac{12}{R_E} \tag{814}$$

Where:

S = channel slope (feet per feet);

$R_E$ = Reynolds Number, $R_E = \frac{uR}{v}$;

u = mean design velocity (feet per second);

R = hydraulic radius (feet); and

v = kinematic viscosity of water ($ft^2/s$).

More detailed discussion of pulsating flow is beyond the scope of this Manual. Several references, including Chow (1959) and Clark County (2000) provide further discussion of this topic. The Los Angeles County Flood Control District (1982) has developed nomographs for determining the appropriate freeboard allowance for roll wave height based on empirical research at the California Institute of Technology (Brock, 1967).

## 803.5 Design Procedures – Superelevation

Superelevation is the transverse rise in water surface that occurs around a channel bend, measured between the theoretical water surface at the centerline of a channel and the water surface elevation on the outside of the bend. Superelevation in bends shall be estimated from the following equation:

$$\Delta y = \frac{CV^2W}{rg} \tag{815}$$

Where:

| | | |
|---|---|---|
| r | = | radius of curvature at centerline of channel (ft); |
| C | = | curvature coefficient (see **Table 804**); |
| $\Delta y$ | = | rise in water surface between design water surface at centerline of channel and outside water surface elevation (ft); |
| W | = | top width at the design water surface at channel centerline (ft); |
| V | = | mean channel velocity (ft/s); and |
| g | = | gravitational acceleration (ft/s$^2$). |

The curvature coefficient C shall be 0.5 for subcritical flow conditions. For supercritical flow conditions, the curvature coefficient shall be 1.0 for all trapezoidal channels and for rectangular channels without transition curves, and 0.5 for rectangular channels with transition curves. **Table 804** provides superelevation curvature coefficients for various flow regimes, cross-section shapes, and types of curves.

Bends in supercritical channels create cross-waves and superelevated flow in the bend section as well as further downstream from the bend. In order to minimize these disturbances, best design practice is to design the channel radius of curvature to limit the superelevation of the water surface to 2.0 feet or less. This can be accomplished by modifying Equation 815 to determine the allowable radius of curvature of a channel for a given superelevation value.

**Table 804 Superelevation Curvature Coefficients**

| Flow Type | Cross Section | Type of Curve | Curvature C |
|---|---|---|---|
| Subcritical | Rectangular | No Transition | 0.5 |
| Subcritical | Trapezoidal | No Transition | 0.5 |
| Supercritical | Rectangular | No Transition | 1.0 |
| Supercritical | Trapezoidal | No Transition | 1.0 |
| Supercritical | Rectangular | with Spiral Transition | 0.5 |
| Supercritical | Trapezoidal | with Spiral Transition | 1.0 |
| Supercritical | Rectangular | with Spiral Banked Transition | 0.5 |
| Source: Corps EM 1110-2-1601 (July 1991) | | | |

## 803.6 Transitions

Channel transitions occur in open channel design whenever there is a change in channel slope or shape and at junctions with other open channels or storm sewers. The goal of a good transition design is to minimize the loss of energy as well as minimize surface disturbances from cross-waves and turbulence. Special cases of transitions where excess energy is dissipated by design are drop structures and hydraulic jumps. Channel drop structures are discussed in Section 814.4.2 of this Manual.

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

1. Subcritical flow to subcritical flow.
2. Subcritical flow to supercritical flow.

3. Supercritical flow to subcritical flow (Hydraulic Jump)
4. Supercritical flow to supercritical flow.

For definition purposes, conditions 1 and 2 will be considered as subcritical transitions and are later discussed in Section 805. Conditions 3 and 4 will be considered as supercritical transitions and are later discussed in Section 805.

## 804 TYPES OF OPEN CHANNELS AND THEIR SELECTION

Open channels can be categorized as either natural or engineered (artificial). Natural channels include all watercourses that are carved and shaped by the erosion and sediment transport process. Engineered channels are those constructed by human efforts. Open channels can be separated into six different types:

**804.1 Natural Channels** - Watercourses are carved and shaped by natural erosion processes before urbanization occurs. As the channel's tributary watershed urbanizes, natural channels often experience erosion and may need grade control checks and localized bank protection to stabilize.

Natural channels are also strongly influenced by urbanization in the watershed, which significantly alters the hydrology and therefore, the geometry of the natural channels. If the watershed imperviousness exceeds around 10%, it is likely that channel geometry will be altered such that a natural channel is no longer viable and mitigation measures will be required, such as bank and bed stabilization measures.

**804.2 Grass-lined Channels** - Among various types of constructed or modified drainageways grass-lined channels are most desirable. They provide channel storage, lower velocities, groundwater recharge, and various multiple use benefits. Low flow areas may need to be concrete, rock-lined, or otherwise reinforced with vegetation to minimize erosion and maintenance problems.

**804.3 Wetland Vegetation Bottom Channels** - A subset of grass-lined channels that are designed to encourage the development of wetlands or certain types of riparian vegetation in the channel bottom. These channels offer potential benefits that may include wildlife habitat, groundwater recharge and water quality enhancement. In low-flow areas, the banks may need supplemental reinforcement to protect against undermining.

**804.4 Concrete-lined Channels** - Concrete-lined channels are high velocity artificial drainageways that are not encouraged. However, in retrofit situations where existing flooding problems need to be solved and where right-of-way is limited, concrete channels may offer advantages over other types of open drainageways. Special attention shall be taken to provide safety measures (i.e. fence) around the concrete-lined channels.

**804.5 Riprap-lined Channels** - Riprap-lined channels offer a compromise between a grass-lined channel and a concrete-lined channel. They can reduce right-of-way needs as compared to grass-lined channels and avoid the higher costs of concrete-lined channels. Riprap-lined channels are not encouraged.

**804.6 Other Lined Channels** - A variety of artificial channel liners are on the market, all intended to protect the channel walls and bottom from erosion at higher velocities. These include gabion, interlocked concrete blocks, concrete revetment mats formed by injecting concrete into double layer fabric forms, and various types of synthetic fiber liners. As with rock and concrete liners, all of these types are best considered for helping to solve existing urban flooding problems and are not recommended for new developments. Each type of liner has to be scrutinized for its merits, applicability, how it meets other community needs, its long term integrity, and maintenance needs and costs. Channels lined with artificial materials are not permitted in new development areas of Mesa County, including the City of Grand Junction, except by variance to or deviation from these criteria.

**804.7 Selection of Channel Type** - Mesa County and the City of Grand Junction do not have a preference for any particular channel-lining system, as long as it is properly evaluated on its merits. Each type of channel must be evaluated for its longevity, integrity, maintenance requirements and costs, and general suitability for community needs, among other factors. Selection of a channel type that is most appropriate for the conditions that exist at a project site shall be based on a multi-disciplinary evaluation, which may include hydraulic, structural, environmental, sociological, maintenance, economic, and regulatory factors.

## 805 NATURAL CHANNEL SYSTEMS

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

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

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

### 805.1 Channel Morphology

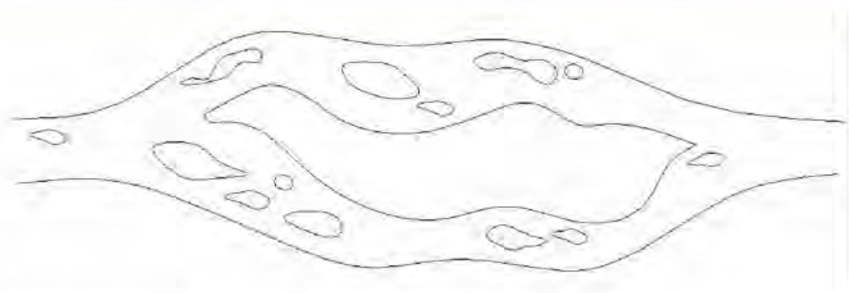
When seeking to utilize or modify a natural channel, an understanding of the mechanism of its morphology is important. Without incorporating thorough understanding of the geomorphic conditions of the stream and the watershed, alterations to channels or to their watersheds can lead to unexpected instabilities,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

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

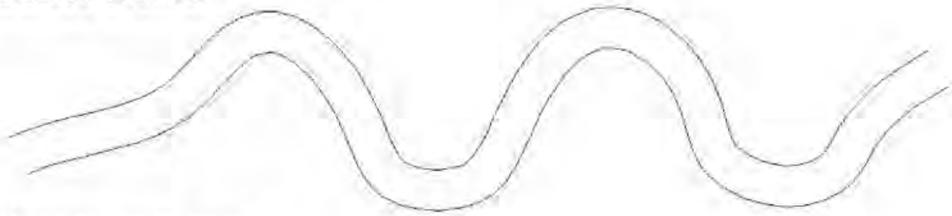
- Dave Rosgen, illustrated by Hilton Lee Silvey, Applied River Morphology, 1996
- Lane, E. W., 14957. A study of the shape of channels formed by natural streams flossing in erodible material: M.R., D. Sediment Series No. 9, US. Army Engineer Division, Missouri River, Corps of Engineers, Omaha, NE.
-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

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

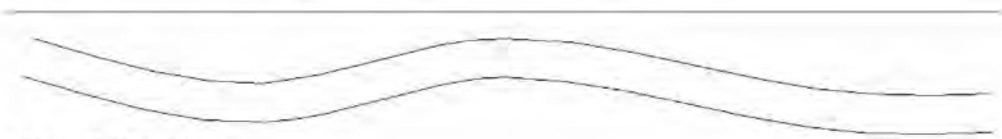
Natural streams can be classified generally into three prevailing patterns. These patterns, straight, meandering and braided, are characteristics of the responses of a system to its prevailing discharge and load.



Braided Channel



Meandering Channel



Straight Channel

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

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

$$Q \cong \frac{b, d, \lambda}{S} \qquad (816)$$

and

$$Qs \cong \frac{b,\lambda,\lambda}{d,P} \quad \text{(SLA, 1982)} \qquad (817)$$

Where:

Q = Average discharge
Qs = Sediment supply
b = Channel width
d = Channel depth
$\lambda$ = Meander wavelength
S = Bed slope
P = Sinuosity

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

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

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

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

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

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

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

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

### 805.2 Channel Restoration

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

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

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

## 806 GENERAL DESIGN CRITERIA FOR IMPROVED OPEN CHANNELS

This section presents general design standards that apply to all improved channels. Section 807 provides specific design criteria for natural and alluvial bed channels. Sections 808 through Section 812 provide specific design criteria for fixed-bed type channels that include: grass-lined channels, wetland bottom channels, riprap-lined channels, concrete-lined channels, and channels with other types of linings.

Depending on the local conditions, the specific requirements for a particular type channel may be more strict than the general criteria outlined in this Section. In addition, unique and unusual site conditions may require additional design analysis be performed to verify the suitability of the proposed channel design for the project site.

### 806.1 Channel Type Selection

Six general different types of open channels were presented in Section 804 of this Manual. In general, the use of concrete-lined and riprap-lined channels is discouraged. The selection of a channel type was presented in general terms in Section 804.7. The following multi-disciplinary factors shall be used if selecting the channel that is most suitable for a specific site.

Hydraulic Factors

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

Structural Factors

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

Environmental Factors

1. Neighborhood character

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

Sociological Factors

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

Maintenance Factors

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

Regulatory Factors

1. Federal regulations
2. State regulations
3. Local regulations

### 806.2 Hydraulic Capacity

All new open channels shall be designed, at a minimum, to safely confine and convey the runoff from the 100-year design event.

### 806.3 Manning Roughness Coefficient

Selection of an appropriate channel roughness value for a given channel section is important for the hydraulic capacity analysis and design of open channel. The roughness value can vary significantly depending on the channel type and configuration, density and type of vegetation, depth of flows, and other hydraulic properties.

**Tables 802A, 802B, 802C, and 802D** show recommended values for the Manning roughness coefficient for various channel types and conditions. Manning roughness coefficients for riprap channels shall be computed based on the criteria outlined in Section 810.2.

### 806.4 Uniform Flow

Open channel drainage systems shall be designed assuming uniform flow conditions. Section 803.1 presents the uniform flow equation and methods for calculating uniform flow.

### 806.5 Vertical and Horizontal Alignment

Open channels shall have a minimum longitudinal gradient of 0.5 percent whenever practical. Flatter grades may be approved with prior consultation with Mesa County or applicable governing agencies. Open channels with grades flatter than 0.5 percent shall have provisions for the drainage of nuisance low flows.

Horizontal alignment changes of two degrees or less may be accomplished without the use of a circular curve for subcritical flow designs ($Fr<1.0$, see 803.3). Curves must be used for supercritical flow designs ($Fr>1.0$), no matter the degree of change in horizontal alignment. Curved channel alignments shall have superelevated banks in accordance with Section 803.5.

Spiral transition curves shall be used upstream and downstream of curves for supercritical channel designs with reverse curves or horizontal alignments with consecutive circular curves. Spiral curves may also be used to reduce required superelevation allowances and cross-wave disturbances.

### 806.6 Maximum Permissible Velocity

The design of open channels shall be governed by maximum permissible velocity. This design method assumes that a given channel section will remain stable up to a maximum permissible velocity, provided that the channel is designed in accordance with the standards presented in this Manual. **Table 805** presents the maximum permissible velocities for several types of natural, improved, unlined, and lined channels.

Regardless of these maximum permissible velocities, the channel section shall be designed to remain stable at the final design flow rate and velocity. The design flow may not always be based on the highest flow velocity. Therefore, best practice is to confirm channel section stability during events smaller than the design flow. This may be accomplished by evaluating flows of specific more frequent storm events (e.g., 10-year, 2-year, etc.), or testing successive fractions (e.g., one half, one quarter, and further if necessary) of the design flow. However, only calculations for the full design flow are required to be submitted for review.

Additional geotechnical and geomorphologic investigation and analyses may be required for natural channels or improved unlined channels to verify that the channel will remain stable based on the maximum design velocities.

### 806.7 Subcritical and Supercritical Flow

Flow can be classified as critical, subcritical, or supercritical according to the level of energy in the flow. This energy is commonly expressed in terms of a Froude Number (Fr) and critical depth ($d_c$). Section 803.2 discusses the characteristics of critical flow and describes methods for determining Froude Number and critical depth. All channel design submittals shall include the calculated Froude Number (Fr) and critical depth ($d_c$) for each unique reach of channel to identify the flow state and verify compliance with these criteria.

Flow at or near the critical state (Fr=1.0 or $d=d_c$) is unstable. As a result, minor factors such as channel debris have the potential to cause severe and acute changes in flow depth. Whenever practicable, channels shall be designed to convey their design flow following the flow energy limitations described in **Table 806**. When necessary to convey flows at or near critical state (0.86<Fr<1.13), flow instabilities may be accommodated by providing additional freeboard.

**Table 806 Limitations on Flow Energy for Rectangular and Trapezoidal Channels**

| Design Flow Condition | Froude Number |
|---|---|
| Subcritical | Fr<0.86 |
| Supercritical | Fr>1.13 |

In rare cases, the specific energy relationship of a cross-section might result in a situation where flows less than the design flow may have a greater depth than the depth calculated for the design flow. The design engineer shall check supercritical channel designs to evaluate whether the channel will maintain freeboard requirements (Section 806.8) during flows less than the design flow (see suggested method in Section 806.6).

## 806.8 Freeboard

In the context of this Manual, freeboard is the additional height of a flood control facility (e.g., channel, levee, or embankment) measured above the design water surface elevation. In this way, the freeboard will provide a factor of safety when designing open channels.

Open channel facilities conveying a design flow of less than 10 cfs shall have a minimum freeboard of 0.5 feet. Open-channel facilities conveying a design flow of 10 cfs or more shall have a design freeboard based on a minimum freeboard of 1.0 foot, with allowances for velocity, superelevation, standing waves, and/or other water surface disturbances such as slug flow. Section 803.3 and Section 803.4 provide design methods for calculating these allowances. Equation 818 and Equation 819 describe the minimum design freeboard for subcritical and supercritical flow design, respectively:

$$(h_{fr})_{SUBCRITICAL} = \max\left(1.0, 0.5 + \frac{v^2}{2g} + \frac{Cv^2W}{rg} + \Delta y\right) \qquad (818)$$

$$(h_{fr})_{SUPERCRITICAL} = 1.0 + 0.025vd^{1/3} + \frac{Cv^2W}{rg} + \Delta y \qquad (819)$$

Where:

$h_{fr}$ = minimum required freeboard (ft);
$v$ = flow velocity (ft/s);
$g$ = gravitational acceleration (32.2 ft/s$^2$);

$\frac{Cv^2W}{rg}$ = superelevation allowance (ft), see Section 803.5; and

$\Delta y$ = allowances for other hydraulic phenomenon (ft), (e.g., standing waves, slug flow – see Section 803.4.5).

Superelevation allowance is a function of flow velocity, channel geometry, and channel alignment. Applying transition curves to the alignment may reduce the required superelevation allowance. Section 803.5 discusses the calculations of superelevation allowance in more detail. The superelevation allowance shall be applied to both banks of the channel. The superelevation allowance shall be applied to channel bends in the following manner:

- Begin at a point 5.0 times the characteristic wave length of the design flow ($5L_W$), measured from the downstream tangent point of the curve, with no superelevation allowance.
- Taper uniformly to the full superelevation allowance at a point 3.0 times the characteristic wave length of the design flow (3.0Lw), measured from the downstream tangent point of the curve.
- Maintain the full superelevation allowance through the curve.
- Continue the top of bank elevation level from the upstream tangent point of the curve to its intersection with the normal top of bank.

Equation 820 and Equation 821 describe the characteristic wavelengths for subcritical and supercritical flow, respectively.

$$(L_W)_{SUBCRITICAL} = 2W \qquad (820)$$

$$(L_W)_{SUPERCRITICAL} = 2W\sqrt{F_r^{\,2} - 1} \qquad (821)$$

The freeboard under the lowest chord of bridge deck (i.e., the soffit elevation) shall be a minimum of 1.0 foot during the 100-year design event. In cases where the bridge has been designed to withstand hydraulic forces of floodwaters and impact from large floating debris, the water surface elevation upstream of the bridge shall maintain a freeboard of at least 1.0 feet below the roadway crest and the finished floors of structures within the zone influenced by the bridge headwater.

This Manual only describes the Mesa County's and the City of Grand Junction's minimum freeboard requirements for open channel design. Major drainageways involving road crossings or other types of crossings, streams that Federal Emergency Management Agency (FEMA) has mapped as Special Flood Hazard Areas, might have significantly different freeboard requirements.

### 806.9 Flow Transitions

Channel transitions occur in open channel design whenever there is a change in channel slope or shape and at junctions with other open channels or storm drains. Properly designed flow transitions mimic the expansion or contraction of natural flow boundaries as best as possible, as well as minimize surface disturbances from cross-waves and turbulence. Drop structures and hydraulic jumps are special transitions where excess energy is dissipated by design. Transitions in open channels are generally designed for either subcritical or supercritical flow transitions.

Hydraulic jumps shall be designed to take place only within energy dissipation or drop structures, and not within an erodible channel. Subcritical transitions shall satisfy the minimum transition lengths described in Section 803.3. Supercritical transitions shall satisfy the minimum transition lengths described in Section 803.4. Special transitions such as drop structures and hydraulic jumps shall satisfy the specifications described in Section 814.

## 806.10 Access and Safety

806.10.1 Access

Any easement encompassing a channel shall be wide enough to provide for the channel structure and adequate maintenance access. Easements shall be placed on one side of lot or ownership lines in new developments and in existing developments where conditions permit.

- The minimum width of any channel easement shall be the top width of channel plus 4 feet on each side of channel.
- Channels with a top width of less than 40 feet require a minimum 12-foot wide service road parallel to one side of the channel and a 4-foot wide access on the opposite side, whenever practicable.
- Channels 40 feet or more in top width require service roads on both sides of the channel that are a minimum of 12 feet wide, whenever practicable.

In all cases, vehicular access to the channel facility must be provided at intervals of 1,000 feet or less, whenever practical. Access easements must be at least 12 feet wide, with a maximum grade of ten percent (10%).

Access ramps shall slope down in the down-gradient direction whenever practicable. Access ramps designed for personnel access shall have a maximum slope of 10 percent. When designed to accommodate vehicular traffic, maintenance access ramps shall be designed to Mesa County or the local jurisdiction private road standards, whichever is applicable.

806.10.2 Safety

Specific safety requirements shall be determined on a case-by-case basis in consultation with Mesa County or the applicable local jurisdiction. As a minimum, guardrails or other approved traffic barriers as described in the Mesa County Road and Bridge Specifications shall be provided when a channel is located next to traffic and according the AASHTO official guidance.

Fencing or access barriers, as required by the governing Agency, is required for channels abutting residential developments, schools, parks, and pedestrian walkways as follows:

- Fencing is required for all concrete-lined or riprapped channels where the design frequency storm produces a velocity that exceeds 5.0 feet per second or 2.0 feet in depth. Fencing is also required when velocity times the depth exceeds 10.0 within 5.0 feet of the flow waters edge.
- Fencing or access barriers required for all unlined Alluvial-bed, grass-lined, and wetland-bottom channels with side slopes steeper than 4H:1V where the design frequency storm produces a velocity that exceeds 5.0 feet per second or 2.0 feet

in depth. Fencing is also required when velocity times the depth exceeds 10.0 within 5.0 feet of the flow waters edge.

Gates shall be provided for maintenance and emergency access at regular intervals, with 20-foot wide gates placed 1,000 feet on center and 4-foot gates placed 500 feet on center or portion thereof. Fencing or access barriers shall be located at a minimum of 6 inches inside the easement boundary lines unless otherwise approved.

### 806.11 Environmental Permitting

Open channel facilities are often located within or adjacent to sensitive environmental areas. The design engineer must investigate which permits might be necessary from various Agencies, such as the U.S. Army Corps of Engineers (e.g., Section 404 Wetland Permit). It is important that the final permits and/or permit conditions allow for the future and perpetual maintenance of a channel facility without the necessity of returning to a permitting Agency for regular maintenance activities.

### 806.12 Maintenance

Where failure of an open-channel facility might cause flooding of a public road or structure, the facility shall have an operation and maintenance plan. These operation and maintenance plans shall specify regular inspection and maintenance at specific time intervals (e.g., annually before the wet season) and/or maintenance "indicators" when maintenance will be triggered (e.g., vegetation more than 6 inches in height). Operation and maintenance plans shall ensure that vegetation is removed or maintained on a regular basis to maintain the function of the facility.

Flood control channels require lifetime maintenance. The project owner and design engineer shall consult with Mesa County and the local jurisdiction to determine which maintenance mechanism is required for a particular project. At a minimum, privately owned and maintained detention facilities shall have a recorded easement agreement with a covenant binding on successors or other mechanism acceptable to Mesa County and the local jurisdiction.

## 807 DESIGN CRITERIA FOR NATURAL AND ALLUVIAL-BED (ERODIBLE) CHANNELS

Natural open channels are important drainage elements that contribute to the image and livability in an urban environment. The areas around open channels may have other uses that facilitate trails, open space areas, and wildlife habitat.

### 807.1 Typical Open Channel Design Sections for Natural Channels

Typical open channel design sections for natural channels are presented in **Figure 807**. The selection of a design section for a natural channel is generally dependent on the value of developable land versus the cost to remove the land from a floodplain. The costs for 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 section discussed herein varies from no encroachment to the level of encroachment at which point an improved channel (unlined or lined) becomes more economical or is required to adequately protect the proposed development. The design standards of natural channels are the same for both major and minor drainageways.

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

### 807.2 General Design Considerations and Evaluation Techniques for Natural Channels

1. The channel and overbank areas shall have adequate conveyance capacity for the major storm runoff.

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

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

4. Filling of the floodplain fringe may reduce valuable storage capacity and may increase downstream runoff peaks, and therefore shall be avoided.

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

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

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

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

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

If a natural channel is to be modified or encroached upon for a development, then the applicant shall meet with the agencies with jurisdiction over the channel to discuss the design concept and to obtain the requirements for planning, design analysis, and documentation. Channel stability analysis must be based on peak runoff rates from the long-term projected, urbanized watershed tributary to the channel.

### 807.3 Natural Unencroached Channels

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

### 807.4 Natural Encroached Channels

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

### 807.5 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 shall prepare a design that addresses scour depths at the lining interface to assure that the lining extends below this depth to avoid undermining of the lining.

### 807.6 Partially Lined Channels

Partially lined channels are defined as channels in which half of the channel is lined 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 only be allowed if:

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

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

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

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

### 807.7 Bio-Engineered Channels

Bio-engineering is an applied science that integrates structural, biological and ecological principles to construct living structures (plant communities) for erosion, sediment, and flood control purposes. In many instances, "bio-engineered" channel stabilization measures can be safely utilized to supplement other stabilization measures. Successful application of "bio-engineered" stabilization measures depends upon accurate diagnosis of the causes of channel stability problems, rather than just treating visible problems areas. The references section of the Manual (Section 1800) provides useful resources on bio-engineered solutions for natural channel stabilization.

## 808 GRASS-LINED CHANNELS

Grass-lined channels are desirable artificial channels from an aesthetic point of view. The engineer shall design grass-lined channels in accordance with the criteria presented herein, and any special considerations due to project site specific requirements.

Applicable design parameters include:

### 808.1 Longitudinal Channel Slopes

In a grass-lined channel, slopes are determined by the maximum permissible velocity requirements. A minimum longitudinal gradient of 0.5% shall be provided whenever practical.

### 808.2 Roughness Coefficients

**Tables 802C** and **802D** present roughness coefficients for grass-lined channels. The design engineer is to assume a mature channel that has substantial vegetation with minimal maintenance.

### 808.3 Trickle and Low Flow Channels

After urbanization, because of lawn irrigation return flows and runoff from directly connected impervious areas, it is not uncommon to see dry waterways having a continuous flow. A trickle channel is required on all urban grass-lined channels. Properly designed earth trickle channels are acceptable. Because of low maintenance and limited infiltration, concrete trickle channels are also acceptable. In the case of larger streams and rivers, a low flow channel may be more appropriate than a trickle channel.

808.3.1 Trickle Channels

Trickle channels are recommended for grass-lined channels with a 100-year design flow less than or equal to 200 cfs. The trickle channel capacity shall convey 5 percent of the 100-year design flow rate or 5 cfs, whichever is greater. There is no freeboard requirement for trickle channels. The flow capacity of the main channel shall be determined without considering the flow capacity of the trickle channel. Care shall be taken to ensure that low flows enter the trickle channel without flow paralleling the trickle channel or bypassing inlets to the channel.

Trickle channels are not typically required for swales and other minor drainageways and grass-lined channels conveying a 100-year peak runoff of 20 cfs or less. For these smaller channels, the design engineer shall evaluate the factors such as drainage slope, flow velocity, soil type, and upstream impervious area, and specify a trickle channel when needed based on their engineering judgment.

808.3.1.1 Concrete Trickle Channel

Concrete trickle channels help minimize erosion, silting, and excessive plant growth. **Figure 808** illustrates a typical concrete trickle channel. The concrete trickle channel shall have a minimum depth of 6 inches. A Manning roughness coefficient value of n=0.015 will be used to design the concrete trickle channel.

808.3.1.2 Rock-Lined Trickle Channel

Rock-lined trickle channels shall have a minimum depth of 12 inches, with the Manning roughness coefficient determined as described in Section 806.3. The minimum stone size for rock-lined trickle channels shall be 6 inches.

808.3.2 Low Flow Channels

Low-flow channels are used to contain relatively frequent flows within a recognizable channel section. Low-flow channels are recommended for channels with a 100-year flow greater than 200 cfs, and at a minimum have the capacity to convey the 2-year flow event with no freeboard. The overall flow capacity of the channel shall include the capacity of the low flow channel.

Low-flow channels shall have a minimum depth of 12 inches. The maximum side slopes of the low-flow channel shall be 2.5H:1V. The main channel depth limitations (Section 805.5) do not apply to the low-flow channel area of the overall channel cross-section.

## 808.4 Bottom Width

The bottom width selected shall be based on factors such as possible wetland mitigation requirements, constructability, channel stability, maintenance requirements, and multi-use purposes.

The minimum channel bottom width shall be 5.0 feet for channels with a concrete trickle channel, 20 feet in channels with a riprap trickle channel, and 30 feet in channels with a low flow channel.

### 808.5 Flow Depth and Freeboard

Swales and grass-lined channels conveying a 100-year flow less than or equal to 10 cfs shall have a minimum freeboard of 6 inches. Grass-lined channels conveying larger discharges shall meet the minimum freeboard requirements outlined in Section 806.8.

The recommended design depth of flow for a grass-lined channel (outside the low flow channel area) is 5.0 feet for a 100-year flow of 1,500 cfs or less whenever practical. Excessive depths shall also be avoided in channels with greater design flows to the maximum extent practicable. Section 806.8 discusses access and safety for open channels, including thresholds for flow depth and velocity.

### 808.6 Side Slopes

Side slopes of a grass-lined channel shall be not a steeper than 3H:1V.

### 808.7 Grass Lining

Satisfactory performance of a grass-lined channel depends on constructing the channel with the proper shape and preparing the area in a manner to provide conditions favorable to vegetative growth. Between the time of seeding and the actual establishment of the grass, the channel is unprotected and subject to considerable damage unless interim erosion protection is provided. See Section 1500 Construction Erosion Control for requirements.

The grass lining for channels may be seeded or sodded with a grass species that is adapted to the local climate and will flourish with minimal irrigation. Channel vegetation is usually established by seeding. In the more critical sections of some channels, it may be required to provide immediate protection by transplanting a complete sod cover. All seeding, planting, and sodding shall conform to local landscape recommendations.

### 808.8 Horizontal Channel Alignment and 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 supplemental erosion protection at bends in natural or grass-lined channels.

The minimum radius for channels with a 100-year runoff of 20 cfs or less shall be 25 feet. For channels carrying larger flows, horizontal channels alignment shall be limited based on the presence of erosion protection.

No channel bend protection is required along bends where the radius is greater than two times the top width of the 100-year water surface. Channels without bend protection shall have a radius of curvature greater than two times the 100-year flow top width or 100 feet, whichever is greater.

When erosion protection is provided the minimum radius of curvature shall be 1.2 times the 100-year flow top width, or 50 feet, whichever is greater.

Erosion protection shall extend downstream from the end of the bend a distance that is equal to the length of the bend measured along the channel centerline.

### 808.9 Maintenance

Grass-lined channels shall be maintained to ensure that vegetation is removed or maintained on a regular basis to maintain the function of the facility. The project owner shall ensure that appropriate mechanism is in place to provide maintenance for the lifetime of the facility.

## 809 WETLAND-BOTTOM CHANNEL

This section presents minimum design criteria for wetland-bottom channels. The design engineer shall design wetland-bottom channels in accordance with the criteria presented herein, and any special considerations for a particular design situation.

Under certain circumstances, such as when existing wetland areas are affected or natural channels are modified, the Corps of Engineers Section 404 permitting process may mandate the use of channels with wetland vegetation. In other cases, a wetland bottom channel may better suit individual site needs if used to mitigate wetland damages somewhere else of if used to enhance urban runoff quality. Wetland-bottom channels are similar to grass-lined channels, except that wetland vegetation growth is encouraged by eliminating the concrete-lined trickle channel and flattening longitudinal slope so that low flows have low velocities.

There are potential benefits associated with a wetland bottom channel, such as wildlife and water quality enhancement as the base flows move through the marshy vegetation.

### 809.1 Longitudinal Channel Slope

The longitudinal channel slope shall be set to the maximum permissible velocity criteria provided in **Table 805** is not violated. To prevent channel degradation, the channel slope shall be determined assuming there is not wetland vegetation on the bottom (i.e., "New Channel"). In addition to the velocity requirements, the Froude Number for the New Channel condition shall be less than 0.7.

### 809.2 Roughness Coefficients

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

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

$$n_c = \frac{\left(n_0^2 P_0 + n_w^2 P_w\right)^{0.5}}{\left(P_0 + P_w\right)} \qquad (822)$$

Where:

$n_c$ = Manning's roughness coefficient for the composite channel (Dimensionless)
$n_o$ = Manning's roughness coefficient for areas above the wetland area (Dimensionless)
$n_w$ = Manning's roughness coefficient for the wetland area (Dimensionless)
$P_0$ = Wetland perimeter of channel cross-section above the wetland area (feet)
$P_w$ = Wetland perimeter of the wetland channel bottom (feet)

For grass-lined areas above the wetland area, use a Manning's roughness coefficient $n_0$ = 0.035. Manning's roughness coefficients for the wetland area ($n_w$) can be obtained from **Figure 809**.

### 809.3 Low-Flow and Trickle Channel

Concrete trickle channels are not permitted in wetland bottom channels. Low-flow channels may be used when the 100-year flow exceeds 1,000 cfs. The design of the low-flow channel shall be according to Section 808.3.2 of this Section.

### 809.4 Bottom Width

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

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

### 809.5 Flow Depth and Freeboard

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

Wetland bottom channels shall meet the minimum freeboard requirements outlined in Section 806.8.

### 809.6 Side Slopes

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

### 809.7 Grass-Lining

The side slopes may be grass-lined according to the guidelines provided previously in Section 808.7.

### 809.8 Channel Bend Protection

Channel Bends shall be designed according to the criteria discussed previously in Section 808.8.

### 809.9 Channel Crossings

Whenever a wetland bottom channel is crossed by a road, railroad or a trail requiring a culvert or a bridge, a drop structure shall be provided immediately downstream of such a crossing. This will help reduce the silting-in of the crossing with sediments. A 1-foot to 2-foot drop is recommended. The designer shall determine the hydraulics of the crossing and the drop structure and design the structures to ensure the stability of the channel.

### 809.10 Life Expectancy

Wetland bottom channels are expected to fill with sediment over time. This occurs because the bottom vegetation traps some of the sediments carried by the flow. The life expectancy of such a channel will depend primarily on the land use of the tributary watershed and could range anywhere from 20 to 40 years before major channel dredging is needed. However, life expectancy can be dramatically reduced, to as little as two to five years, if land erosion in the tributary watershed is not controlled. Therefore, land erosion practices need to be strictly controlled during new construction within the watershed and all facilities need to be built to minimize soil erosion in the watershed to maintain a reasonable economic life of a wetland bottom channel.

## 810 DESIGN CRITERIA – RIPRAP-LINED CHANNELS

This section presents minimum design criteria for riprap-lined channels. The engineer shall design riprap-lined channels in accordance with the criteria presented herein, and any special considerations arising out of a particular design situation.

Riprap-lined channels are defined as channels in which riprap is used for lining of the channel banks and the channel bottom to control erosion. Design standards for riprap-lined channels shall also be used for transitions and bends when lined with riprap. Riprap is used to control erosion in both channel banks and beds, transition sections upstream and downstream of hydraulic structures, at bends, and bridges.

Loose or grouted riprap is cost effective for short channel reaches. Riprap lining might be appropriate for 1) flows that produce channel velocities in excess of allowable values; 2) channels where side slopes need to be steeper than 3:1; 3) at channel bends and transitions; and 4) low-flow channels.

Specific design parameters for riprap-lined channels include:

### 810.1 Longitudinal Channel Slope

The maximum permissible velocity for riprap-lined channels is presented in **Table 805**. In steeper terrain, drop structures may be used to achieve the desired design velocities.

### 810.2 Roughness Coefficients

The Manning's roughness coefficient, n, for hydraulic computations may be estimated for loose riprap using the following equation.

$$n = .0395(d_{50})^{1/6} \quad (823)$$

Where:

$d_{50}$ = mean stone size (feet)

This equation (Anderson, 1968) does not apply to grouted riprap (n=.023 to .030) or to very shallow flow (hydraulic radius is less than or equal to 2 times the maximum rock size) where the roughness coefficient will be greater than indicated by the formula.

### 810.3 Low-Flow Channel

The design criteria for the low-flow channel are discussed in Section 808.3.2.

### 810.4 Bottom Width

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

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

### 810.5 Flow Depth

As preliminary criteria, the design depth of flow for the major storm runoff flow shall not exceed 7.0 feet in areas of the channel cross-section outside the low-flow or trickle channel.

### 810.6 Side Slopes

Due to stability, safety, and maintenance considerations, riprap-lined side slopes shall be 2 horizontal to 1 vertical or flatter.

### 810.7 Toe Protection

Where only the channel sides are to be lined, additional riprap is needed to provide for long-term stability of the lining. In this case, the riprap blanket shall extend a minimum of 3 feet below the proposed channel bed, and the thickness of the blanket below the proposed channel bed shall be increased to a minimum of 3 times $d_{50}$ to accommodate possible channel scour during floods. If the velocity exceeds the permissible velocity requirements of the soil comprising the channel bottom, a scour analysis shall be performed to determine if the toe requires additional protection.

## 810.8 Beginning and End of Riprap-Lined Channel

At the upstream and downstream termination of a riprap-lining, the thickness shall be increased 50 percent for at least 3 feet to prevent undercutting. Depending on the site-specific conditions, concrete cutoff walls at both ends may be necessary.

## 810.9 Loose Riprap Lining

Loose riprap, or simply riprap, refers to a protective blanket of large loose angular stones that are usually placed by machine to achieve a desired configuration. The term loose riprap has been introduced to differentiate loose stones from grouted riprap.

Many factors govern the size of the rock necessary to resist the forces tending to move the riprap. For the riprap itself, this includes the size and weight of the individual rock, the shape of stones, the gradation of the particles, the blanket thickness, the type of bedding under the riprap, and slope of the riprap layer. Hydraulic factors affecting riprap include the velocity, current direction, eddy action, and waves. **Figure 810** provides typical cross-sections for riprap-lined channels.

Experience has shown that riprap failures generally result from undersized individual rocks in the maximum size range, improper gradation of the rock which reduces the interlocking of individual particles and improper bedding for the riprap which allows leaching of channel particles through the riprap blanket.

810.9.1 Riprap Material

Rock used for loose riprap, grouted riprap, or wire enclosed riprap shall be hard, durable, angular in shape, and free from cracks, overburden, shale and organic matter. Neither breadth nor thickness of a single stone shall be less than 1/3 of its length and rounded stone shall be avoided. Rock having a minimum specific gravity of 2.65 is preferred; however, in no case shall the specific gravity of the individual stones be less than 2.50.

Classification and gradation for riprap are shown in **Table 807** and are based on a minimum specific gravity of 2.50 for the rock. Because of its relatively small size and weight, riprap Class 150 must be buried with native topsoil and revegetated to protect the rock from vandalism.

Riprap lining requirements for a stable channel lining are based on the following relationship which resulted from model studies by Smith and Murray (Smith, 1965)

$$d_{50} = \frac{0.05V^2S^{0.34}}{(S_s-1)^{1.332}} \qquad (824)$$

Where:

$d_{50}$ = Rock size for which 50 percent of riprap by weight is smaller (feet)
$V$ = Mean channel velocity (fps)
$S$ = Longitudinal channel slope (feet/feet)
$S_s$ = Specific gravity of rock (minimum $S_s$ = 2.50) (dimensionless)

The riprap blanket thickness shall be at least 2.0 times $d_{50}$ and shall extend up the side slopes to an elevation of the design water surface plus the calculated freeboard.

810.9.2 Bedding Requirements

Long term stability of riprap erosion protection is strongly influenced by proper bedding conditions. A large percentage of all riprap failures is directly attributable to bedding failures. Gradations for granular riprap bedding are shown in **Table 808**.

Properly designed bedding provides a buffer of intermediate sized material between the channel bed and the riprap to prevent movement of soil particles through the voids in the riprap. Three types of bedding are in common use: generic single-layer granular bedding, granular bedding based on the T-V (Terzughi-Vicksburg) methodology, and filter fabric.

1) Granular Bedding - Generic Design

The gradation of a single layer bedding specification is based on the assumption that said bedding will generally protect the underlying soil from displacement during a flood event. The single layer bedding design does not require any soil information, but in order to be effective covering a wide range of soil types and sizes, this method requires a greater thickness than the T-V method.

A single 12-inch layer of said granular bedding can be used except at drop structures. At drop structures, filter fabric must be added below the 12-inch layer of granular bedding.

2) Granular Bedding - T-V Design

The T-V design establishes an optimum granular bedding gradation for a specific channel soil. Since this method designs the granular bedding for a particular soil, the allowable granular bedding thickness may be much less than the generic design.

The specifications for the T-V reverse filter method relate the gradation of the protective layer (filter) to that of the bed material (base) by the following inequalities:

$$D_{15(filter)} \leq 5d_{85(base)} \qquad (825)$$

$$4d_{15(base)} \leq D_{15(filter)} \leq 20d_{15(base)} \qquad (826)$$

$$D_{50(filter)} < 25d_{50(base)} \qquad (827)$$

Where the capital "D" refers to the filter grain size and the lower case "d" to the base grain size. The subscripts refer to the percent by weight which is finer than the grain size denoted by either "D" or "d". For example, 15 percent of the filter material is finer than $D_{15(filter)}$ and 85 percent of the base material is finer than $d_{85(base)}$.

When the T-V method is used, the thickness of the resulting layer of granular bedding may be reduced to six inches. However, if a gradation analysis of the existing soils shows that a single layer of T-V Method designed granular bedding cannot bridge the gap between the riprap specification and the existing soils, then two or more layers of granular bedding shall be used. The design of the bedding layer closest to the existing soils shall be based on the existing soil gradation. The design of the upper bedding layer shall be based on the gradation of the lower bedding layer. The thickness of each of the two or more layers shall be 4 inches.

3) Filter Fabrics

Filter fabric is not a complete substitute for granular bedding. Filter fabric provides filtering action only perpendicular to the fabric and has only a single equivalent pore opening between the channel bed and the riprap. Filter fabric has a relatively smooth surface which provides less resistance to stone movement. As a result, it is recommended that the use of filter fabric in place of granular bedding be restricted to slopes no steeper than 2.5 horizontal to 1 vertical, and that such filter fabric only replace the bottom layer in a multi-layer T-V Method granular bedding design. The granular bedding shall be placed on top of the filter fabric to act as a cushion when placing the riprap. Tears in the fabric greatly reduce its effectiveness so that direct dumping of riprap on the filter fabric is not allowed and due care must be exercised during construction. Nonetheless, filter fabric has proven to be an adequate replacement for granular bedding in many instances. Filter fabric provides adequate bedding for channel linings along uniform mild sloping channels where leaching forces are primarily perpendicular to the fabric.

At drop structures and sloped channel drops, where seepage forces may run parallel with the fabric and cause piping along the bottom surface of the fabric, special care is required in the use of filter fabric. Seepage parallel with the fabric may be reduced by folding the edge of the fabric vertically downward about 2 feet (similar to a cutoff wall) at 12-foot intervals along the installation, particularly at the entrance and exit of the channel reach. Filter fabric has to be lapped a minimum of 12 inches at roll edges with upstream fabric being placed on top of downstream fabric at the lap.

Fine silt and clay has been found to clog the openings in filter fabric. This prevents free drainage which increases failure potential due to uplift. For this reason, a granular filter is often more appropriate bedding for fine silt and clay channel beds.

### 810.10 Grouted Riprap-Lining

Grouted riprap provides a relatively impervious channel lining which is less subject to vandalism than loose riprap. Grouted riprap requires less routine maintenance by reducing silt and trash accumulation and is particularly useful for lining low-flow channels and steep banks. The appearance of grouted riprap is enhanced by exposing the tops of individual stones and by cleaning excess grout from the projecting rock with a wet broom prior to curing. **Figure 811** provides a typical cross-section for a grouted riprap lining.

810.10.1 Riprap Material

The rock used for grouted riprap is different from the standard gradation of riprap in that the smaller rock has been removed from the groundwater to allow larger void spaces and, therefore greater penetration by the grout. The riprap specifications are shown on **Table 809**. Riprap smaller than Class 400 shall not be grouted.

810.10.2 Bedding Material

The bedding material requirements are the same as for loose riprap.

810.10.3 Cutoff Trench

As the riprap layer is placed, a cutoff trench shall be excavated around the rock section at the top of the slope and at the upstream and downstream edges. The trench shall be, at a minimum, the full depth of the riprap and bedding layer and at least 1-foot wide. This trench is filled with grout to prevent water from undermining the grouted rock mass.

810.10.4 Grout

After the riprap has been placed to the required thickness and the trench excavated, the rock is sprayed with clean water which cleans the rock and allows better adherence by the grout. The rock is then grouted using a low pressure (less than 10 psi) grout pump with a 2" maximum diameter hose. Using a low pressure grout pump allows the work crew time to move the hose and vibrate the grout. Vibrating the grout with a pencil vibrator assures complete penetration and filling of the voids. After the grout has been placed and vibrated, a small hand broom or gloved hand is used to smooth the grout and remove any excess grout from the rock. The finished surface is sealed with a curing compound.

The grout shall consist of 6 sacks (564 pounds) of cement per cubic yard, and the aggregate shall consist of 30% of 3/8-inch coarse gravel and 70% natural sand. The grout shall contain 7.5% +/- 1.5% air entrainment, have a 28-day compressive strength of at least 2,000 psi, and have a slump of 7 inches +/- 2 inches. Fiber reinforcement shall be used such as 1.5 pounds per cubic yard of Fibermesh or an approved equivalent amount. A maximum of 25% fly ash maybe substituted for the cementations material.

### 810.11 Channel Bend Protection

When riprap protection is required for a straight channel, increase the rock size by one category (e.g., Class 300 to Class 400) through bends. The minimum radius for a riprap-lined bend is 1.2 times the top width and in no case less than 50 feet. Riprap protection shall extend downstream from the end of the bend a distance that is equal to the length of the bend measured along the channel centerline.

## 810.12 Transition Protection

Scour potential is amplified by turbulent eddies in the vicinity of rapid changes in channel geometry such at transitions and bridges. For these locations, the riprap lining thickness shall be increased by one size category.

Protection shall extend upstream from the transition entrance at least 5.0 feet and extend downstream from the transition exit at least 10 feet. See Sections 803.3 and 803.4 for further discussions on transitions.

## 810.13 Concrete Cutoff Walls

Transverse concrete cutoff walls may be required for riprap lined channels where a resulting failure of the riprap lining may seriously affect the health and safety of the public. The designer shall consult with the local officials prior to design of riprap lined-channels to determine if concrete cutoff walls are required as well as their sizing and spacing, if required.

## 810.14 Riprap-Lined Channels on Steep Slopes

Achieving channel stability on steep slopes usually requires some type of channel lining. The only exception is a channel constructed in durable bedrock.

On mild slopes, the water velocity is slow enough and the depth of flow is large enough (relative to the riprap size) that a reasonable estimate of the resistance to flow can be made. On steep channels, the riprap size required to stabilize the channel is on the same order of magnitude or greater than the flow depth, which invalidates the Manning's relation. Since the resistance to flow is unknown, an estimate of the velocity needed for the design of the riprap cannot be accurately estimated.

A graphically based methodology was developed for the U.S. Department of Interior, Office of Surface Mining Reclamation and Enforcement (SIMONS, 1989) to design riprap-lined channels on steep slopes (supercritical flow). This methodology was based on a study by BATHURST, 1979 that analyzed the hydraulics of mountain rivers where roughness elements are on the same order of magnitude as the depth of flow. Using the resistance equation developed by Bathurst, the velocity can be estimated for a given riprap size. The velocity is then used to predict the stability of the riprap.

This procedure shall be used for all riprap lined channels whose depth of flow is equal to or less than $d_{50}$ as computed initially using Equation 824.

810.14.1 Rock Size

Five sets of design curves **(Figures 812 through 816)** have been developed from Bathurst's relationship to simplify riprap design for steep channels. The design curves were developed for channels with 2 to 1 side slopes and bottom widths of 0 feet, 6 feet, 10 feet, 14 feet, and 20 feet. The curves were terminated at the point where flow velocity exceeded 15 fps. A median rock diameter could be determined that would be stable at higher flows and velocities; however, rock durability at velocities greater than 15 fps becomes of greater concern.

For a given flow, channel slope, and channel width, **Figures 812 through 816** will provide the median riprap size. When the channel slope is not provided by one of the design curves, linear interpolation is used to determine the riprap size. This is done by extending a horizontal line at the given flow through the curves with slopes bracketing the design slope. A curve at the design slope is then estimated by visual interpolation. The design $D_{50}$ size is then chosen at the point that the flow intercepts the estimated design curve. Linear interpolation can also be used to estimate the $D_{50}$ size for bottom widths other than those supplied in the figures.

For practical engineering purposes, the $D_{50}$ size specified for the design shall be given in 0.25-foot increments. The final minimum design size is determined using **Table 810**.

810.14.2 Riprap Gradation for Steep Slopes

**Table 811** provides ratios used to determine the $D_{10}$, $D_{20}$, and $D_{max}$ rock sizes from the $D_{50}$ rock size determined in the previous section. It is important to establish a smooth gradation from the largest to the smallest sizes to prevent large voids between rocks.

810.14.3 Riprap Thickness for Steep Slopes

For riprap linings on steep slopes, a thickness of 1.25 times the median rock size is recommended. The maximum resistance to the erosive forces of flowing water occurs when all rock is contained within the riprap layer thickness. Oversize rocks that protrude above the riprap layer reduce channel capacity and reduce riprap stability.

810.14.4 Riprap Placement on Steep Slopes

Improper placement is another major cause of failure in riprap-lined channels. To prevent segregation of rock sizes, riprap shall be dumped directly from trucks from the top of the embankment, and draglines with orange peel buckets, backhoes, and other power equipment to place riprap with minimal handwork.

810.14.5 Freeboard

**Figures 812 through 816** also provide the depth of flow for a given flow rate, channel slope, and channel dimensions. The minimum required freeboard is given by Equation 818 for subcritical flow or 819 for supercritical flow. The velocity can be estimated by dividing the flow rate by the area of flow.

810.14.6 Bedding Requirements on Steep Slopes

Either a granular bedding material or filter fabric may be used on steep slopes according to the requirements previously specified in Section 810.9.

## 811 DESIGN CRITERIA – CONCRETE-LINED CHANNELS

This section presents minimum design criteria for concrete-lined channels. The design engineer is responsible for confirming that a channel design meets these criteria, the general open channel criteria outlined in Section 806, and any specific considerations due to project specific site requirements.

Applicable design parameters include:

### 811.1 Longitudinal Channel Slope

Concrete-lined channels have the ability to accommodate supercritical flow conditions and thus can be constructed almost on any naturally occurring slope. The maximum slope is determined by the maximum permissible velocity shown in **Table 805**.

### 811.2 Roughness Coefficients

See figures presented in **Tables 802A thru 802C** for appropriate Manning roughness coefficient for concrete-lined channels. For subcritical flow for concrete-lined channels, check the Froude Number using a Manning roughness coefficient of 0.011.

### 811.3 Low-Flow Channel

The bottom of the concrete-lined channel shall be sloped to confine the low flows to the middle or one side of the channel. Low flow channels are defined in Section 808.3.2.

### 811.4 Bottom Width

The bottom width of the concrete-lined channel shall be a minimum of 4.0 feet.

### 811.5 Flow Depth

There are no flow depth requirements for concrete-lined channels.

### 811.6 Side Slopes

Vertical or flatter side slopes may be designed for concrete-lined channels.

### 811.7 Concrete-Lining Section

811.7.1 Thickness

Concrete-lined channels shall have a minimum thickness of 6 inches for flow velocities less than 30 feet per second (fps). For flow velocities of 30 fps and greater, the minimum thickness shall be 7 inches.

811.7.2 Concrete Joints

The following design standards, found to work in similar conditions, are suggested. Alternatives will be considered on a case-by-case basis.

- Channels shall be continuously reinforced without transverse joints. Expansion/contraction joints (without continuous reinforcement) shall only be installed where the new concrete-lining is connected to a rigid structure or to an existing concrete-lining which is not continuously reinforced. The design of the expansion joint shall be coordinated with local officials.

- Longitudinal joints, where required, shall be constructed on the sidewalls at least 1.0 feet vertically above the channel invert.
- All joints shall be designed to prevent differential movement.
- Construction joints are required for all cold joints and where the lining thickness changes. Reinforcement shall be continuous through the joint and the concrete-lining shall be thickened at the joint.

811.7.3 Concrete Finish

The surface of the concrete-lining shall be provided with a wood float finish unless the design requires additional finishing treatment.

Excessive working or wetting of the finish shall be avoided if additional finishing is required.

811.7.4 Concrete Curing

It is suggested that concrete-lined channels be cured by the application of a liquid membrane-forming curing compound (white pigmented) upon completion of the concrete finish. All curing shall be completed in accordance with the standard specifications of the local government agency.

811.7.5 Reinforcement Steel

- Steel reinforcement shall be at a minimum grade 40 deformed bars. Wire mesh shall not be used.
- Ratio of longitudinal steel area to concrete cross-sectional area shall be greater than .0905 but not less than a #4 rebar placed at 12-inch spacing. The longitudinal steel shall be placed on top of the transverse steel.
- Ratio of transverse steel area to concrete cross-sectional area shall be greater than .0025 but not less than a #4 rebar placed at 12-inch spacing.
- Reinforcing steel shall be placed near the center of the section with a minimum clear cover of 3 inches adjacent to the earth.
- Additional steel shall be added as needed. If a retaining wall structure is used, the structure shall be designed by a registered structural engineer with structural design calculations submitted for review and approval.

811.7.6 Earthwork

As a minimum, the following areas shall be compacted to at least 90 percent of maximum density as determined by ASTM 1557 (Modified Proctor). Additional requirements may be required by the geotechnical report.

- The 12 inches of subgrade immediately beneath concrete lining (both channel bottom and side slopes).
- Top 12 inches of maintenance road.
- Top 12 inches of earth surface within 10 feet of concrete channel lip.
- All fill material.

811.7.7 Bedding

A Geotechnical report shall be submitted which addresses the required bedding necessary for the specific concrete section under consideration.

811.7.8 Underdrain and Weepholes

A transverse concrete cutoff shall be installed at the beginning and end of the concrete-lined section of channel and at a maximum spacing of 90 feet. The concrete cutoffs shall extend a minimum of 3.0 feet below the bottom of the concrete slab and across the entire width of the channel lining. Longitudinal cutoffs, a minimum of 3.0 feet in depth, at top lining are required to ensure integrity of the concrete lining.

If the channel is continuously reinforced without transverse joints then a concrete cutoff is required to be incorporated into the expansion/concrete joint.

### 811.8 Special Consideration for Supercritical Flow

Supercritical flow in an open channel in an urbanized area creates hazards which the designer shall take into consideration. Careful attention shall be taken to insure against excessive waves which may extend down the entire length of the channel from only minor obstructions. Imperfections at joints may rapidly cause a 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 shall be taken to minimize the change in wetted area of the cross-section at bridges and culverts. Bridges and other structures crossing the channel shall be anchored satisfactorily to withstand the full dynamic load which might be imposed upon the structure in the event of major debris plugging.

The concrete-lining shall 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 shall be designed to be free draining. With supercritical flows, minor downstream obstructions do not create any backwater effect. Backwater computation methods are applicable for computing the water-surface profile or the energy gradient in channels having a supercritical flow; however, the computations shall proceed in a downstream direction. The designer shall take care to insure against the possibility of unanticipated hydraulic jumps forming in the channel.

## 812 <u>DESIGN CRITERIA – OTHER CHANNEL LININGS</u>

Other channel linings include all channel linings that are not discussed in the previous sections. These include composite-lined channels, which are channels in which two or more different lining materials are used (i.e., riprap bottom with concrete side slope lining). They also include gabions, soil cement linings, synthetic fabric and geotextile linings, preformed block linings, reinforced soil linings, and flood walls (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 these linings not discussed in this Manual, supporting documentation will be required to support the use of the desired lining. A guideline of some of the items which should be addressed in the supporting documentation is as follows:

a. Structural integrity of the proposed lining
b. Interfacing between different linings
c. The maximum velocity under which the lining will remain stable
d. Potential erosion and scour problems
e. Access for operations and maintenance
f. Long term durability of the product under the extreme meteorological and soil conditions
g. Ease of repair of damaged section
h. Past case history (if available) of the lining system in other arid areas
i. Potential groundwater mitigation issues (i.e. weepholes, underdrains, etc.).

These linings will be allowed on a case-by-case basis. Mesa County and/or other jurisdictions may reject the proposed lining system in the interest of operation, maintenance, and protecting the public safety.

## 813 DESIGN STANDARDS FOR MINOR ARTIFICAL DRAINAGEWAYS

A minor drainageway is defined as a channel/drainageway with a contributing tributary area of less than 160 acres. Additional flexibility and less stringent standards may be allowed for minor drainageways. **Only the differences in a channel type's design as a minor drainageway versus that of a major drainageway are presented in this section**.

### 813.1 Freeboard

For grass, concrete and riprap-lined swales and drainageways with a 100-year flow of equal to or less than 10 cfs, the minimum freeboard requirement is 6 inches.

### 813.2 Curvature(Horizontal)

For grass, wetland bottom, concrete and riprap-lined swales and drainageways, the minimum radius with a 100-year runoff of 20 cfs or less shall be 25 feet.

### 813.3 Trickle Channel

For grass, concrete and riprap-lined swales and drainageways, with 100-year runoff peaks of 20 cfs or less, trickle channel requirements will be evaluated for each case. Trickle channels help preserve swales crossing residential property. Factors to be considered when establishing the need for trickle channels are: drainage slope, flow velocity, soil type, and upstream impervious area.

## 814 CHANNEL APPURTENANCES

Presented in this section are the design standards for appurtenances to improved channels. All improved channels shall be designed to include these appurtenances.

### 814.1 Maintenance Access Road

A maintenance access road with a minimum passage width of 12 feet shall be provided along the entire length of all improved channels with 100-year design capacity equal to or greater than 50 cfs. For such channels with less than 50 feet in top width, one maintenance access shall be provided as part of the channel improvements. For channels with greater than 50 feet in top width, the maintenance road shall be located in or within 10 feet horizontal distance from the bottom of the channel or on both sides of the channel top.

For channels with the maintenance access road at or near the channel bottom, ramps to said road shall be provided at a maximum 10 percent slope. Said ramps shall slope down in the down gradient direction of the channel.

### 814.2 Safety Requirements

The following safety requirements are required for concrete-lined channels. Similar safety requirements may be required for all other channels.

a. A six-foot high galvanized-coated chain link or comparable fence shall be installed to prevent unauthorized access. The fence shall be located at the edge of the ROW or on the top of the channel lining. Gates, with top latch, shall be placed at major access points or 1,320-foot intervals, whichever is less.
b. Ladder-type steps shall be installed not more than 1,200 feet apart and shall be staggered on alternating sides of the channel to provide a ladder every 600 feet. The bottom rung shall be placed approximately 12 inches vertically above the channel invert.

### 814.3 Culvert Outlet Protection

If the flow velocity at a culvert or storm sewer outlet exceeds the maximum permissible velocity for the local soil or channel lining, channel protection is required. This protection usually consists of an erosion resistant reach, such as riprap and stilling basin, to provide a stable reach at the outlet in which the exit velocity is reduced to a velocity allowable in the downstream channel.

The following basin sizing procedure shall be used for culvert sizes less than or equal to 36 inches in diameter or equivalent open area and outlet velocities less than 15 fps. For larger culverts or outlet velocities greater than 15 fps, the outlet protection design provided for in USDOT, 1983 shall be used.

814.3.1 Basin Configuration

The length of the outlet protection ($L_a$) is determined using the following empirical relationships that were developed for the U.S. Environmental Protection Agency (USEPA, 1976):

$$L_a = \frac{1.8Q}{D_o^{3/2}} + 7D_o, \text{ for } TW < \frac{D_o}{2} \qquad (828)$$

and

$$L_a = \frac{3Q}{D_o^{3/2}} + 7D_o, \text{ for TW} \geq \frac{D_o}{2} \tag{829}$$

Where:

| | | |
|---|---|---|
| $D_0$ | = | maximum inside culvert width (ft); |
| Q | = | pipe discharge (cfs); |
| TW | = | tailwater depth (ft) |

Where there is no well defined channel downstream of the apron, the width, W, of the outlet and of the apron (as shown in **Figure 1209**) shall be as follows:

$$W = 3D_o + 0.4L_a, \text{ for TW} \geq \frac{D_o}{2} \tag{830}$$

and

$$W = 3D_o + L_a, \text{ for TW} < \frac{D_o}{2} \tag{831}$$

The width of the apron at the culvert outlet shall be at least 3 times the culvert width.

Where there is a well-defined channel downstream of the apron, the bottom width of the apron shall be at least equal to the bottom width of the channel and the lining shall extend at least 1.0 feet above the tailwater elevation and at least two thirds of the vertical conduit dimension above the invert.

The apron side slopes shall be 2:1 or flatter, and the bottom grade shall be level.

814.3.2 Rock Size

The median stone diameter, $d_{50}$ is determined from the following equation:

$$d_{50} = 0.02\frac{(Q)^{4/3}}{TW(D_o)} \tag{832}$$

Existing scour holes may be used where flat aprons are impractical. **Figure 817** shows a general design of a scour hole. The stone diameter is determined using the following equations.

$$d_{50} = \frac{0.0125(Q)^{4/3}}{TW(D_o)}, \text{ for y} = \frac{D_o}{2} \tag{833}$$

Also,

$$d_{50} = \frac{0.0082(Q)^{4/3}}{TW(D_o)}, \text{ for y} = D_o \tag{834}$$

Where y = depth of scour hole below culvert invert

The other riprap requirements are as indicated in the previous sections for channel lining.

## 814.4 Low Flow Grade Control Structures

814.4.1 Introduction

With the advent of floodplain management programs, developers and local governments frequently decided to preserve the floodplain. Since urbanization causes more frequent and sustained flows, the trickle/low flow channel becomes more susceptible to erosion even through the overall floodplain may remain stable and able to resist major flood events.

Erosion of the low flow channel, if left uncontrolled, can cause degradation and destabilization of the entire floodplain. Low flow check structures are designed to provide control points and establish stable bed slopes within the base flow channel. The check structures can be small versions of the drop structures described in Section 902 or in many instances simply control sills across the floodplain. Low flow check structures are not appropriate in instances such as completely incised floodplains or very steep channels.

In addition, low flow structures inherently permit a considerable amount of sediment to be transported downstream as urbanization takes place, which negatively impacts stormwater quality. Therefore, complete channel stabilization using drop structures in conjunction with channel bed and bank stabilization at the time of development may be required.

814.4.2 Drop Structure Grade Control Structures

The grouted sloping boulder drop structure and the vertical riprap drop structure designs can be adapted for use as check structures. The analysis steps are the same with the additional consideration of 1) stable bed slope for the unlined trickle or low flow channel and 2) potential overflow erosion during submergence of the check structure and were flow converges back from the main channel sides or below the check structure.

The basic design steps for this type of structure include the following:

a. Determine a stable slope and configuration for the low flow zone. For unlined channels, discharges from full floodplain flow to the dominant discharge shall first be considered. The dominant discharge is more fully explained in sediment transport texts such as Simons, Li and Associates (1982).
b. The configuration of the low flow zone, and number and placement of the check structures has to be reviewed. Typically, the floodplain slope is steeper, often on the order of critical conditions. If the checks are widely spaced, the trickle channel depth can be quite deep downstream of the check, leading to concentration of higher flows into the trickle channel and the check. A good rule of thumb is to not have the trickle channel more than 2 feet deep at the crest of the check, or more than 4 feet deep below the check structure (relative to the overbank).
c. A hydraulic analysis shall be performed using the discharge that completely fills the check structure at its crest (the primary design flow).

d. The secondary design flow is that flow which causes the worst condition for lateral overflow around the abutments and back into the basin or trickle channel below. The goal is to have the check structure survive such an event with minimal or reasonable damage to the floodplain below. The best approach is to estimate unit discharges, velocities and depths along overflow paths. The unit discharges can be estimated at the crest or critical section for the given total flow. Estimating the overflow path around the check abutment is difficult and requires practical judgment. Slopes can be derived for the anticipated overflow routes and protective measures devised such as grouted rock.
e. Seepage control is also important, as piping and erosion through or around these structures is a frequent problem. It is advisable to provide a cutoff which extends laterally at least 5.0 to 10 feet into undisturbed bank at minimum and has cutoff depth appropriate to the profile dimensions of the check.

Additional information related to design of drop structures can be found in Section 902 of this Manual.

814.4.3 Control Sill Grade Control Structures

Another type of check structure that can be used to stabilize low flow channels within wide, relatively stable floodplains is the control sill shown in **Figure 818**. The sill can be constructed by filling an excavated trench with concrete, if soil conditions are acceptable for trenching, or forming a simple wall if a trench will not work.

The sill crosses the low flow channel and shall extend a significant distance into the adjacent floodplain on both sides. The top of the sill conforms to the top of the ground at all points along its length. Riprap or other erosion control methods can then be added as erosion occurs.

The basic design steps are:

a. Determine a stable slope as described above
b. Determine spacing of the sills based on the difference in slope between the natural and projected stable slope and the amount of future drop to be allowed (not to exceed 3 feet).

## 815 EXAMPLE APPLICATION

An open channel is to be constructed for Doe Creek downstream of John Boulevard and north of Rose Subdivision.

Assume the following conditions for this problem.

$Q_{100}$ = 191 cfs
Invert elevation downstream of John Boulevard = 4,918
Invert elevation downstream of Rose Subdivision = 4,917
Channel improvement length = 900 feet

Due to aesthetics and sufficient right-of-way, a grass-lined channel should be constructed.

Side slope = z = 3
Bottom Width = b = 10 feet
n = 0.035 for grass-lined channel

Since the 100-year, 24-hour flow is less than 200 cfs, a trickle channel should be constructed in the proposed channel bottom.

**Solution:**

**Step 1:** Determine the depth of water during a 100-year flow event

$$\text{Slope} = \frac{4918 - 4917}{900} = 0.0011 \text{ feet/foot}$$

The Manning equation can be re-written so that the depth of flow, y, in a trapezoidal channel is on one side of the equation.

$$\frac{\left(by + zy^2\right)^{5/3}}{\left(b + 2y\left(1 + z^2\right)^{1/2}\right)^{2/3}} = \left(\frac{Q}{S^{1/2}}\right)\left(\frac{n}{1.49}\right)$$

Solving by trial and error,

Y = 3.7 feet

**Step 2:** Calculate the water velocity in the proposed channel during a 100-year flow event using the Manning equation.

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

$$= \left(\frac{1.49}{0.035}\right) * (0.0011)^{1/2} * \left(\frac{(10 + 3 * 3.7) * 3.7}{10 + 2 * 3.7 * \left(1 + 3^2\right)^{1/2}}\right)^{2/3}$$

= 2.5 fps

Since the water velocity of the proposed channel (2.5 fps) is less than the maximum permissible water velocity in a grass-lined channel, a grass-lined channel can be used at this location.

**Step 3** Design the trickle channel.

Assume dimensions for a concrete trickle channel:

Bottom width = 5 feet
Depth = 1 foot
Side Slopes = vertical

The capacity of the trickle channel is:

$$Q = \left(\frac{1.49}{n}\right)\left(S^{1/2}\right)\left(R^{2/3}\right)(A)$$

$$Q = \left(\frac{1.49}{n}\right)\left(S^{1/2}\right)\left(\frac{by}{b+2y}\right)^{2/3}(by)$$

$$Q = \left(\frac{1.49}{0.015}\right) * (.0011)^{1/2} * ((5*1)/(5+(2*1)))^{2/3} * (5*1)$$

Q = 13.16 cfs

**Step 4:** Verify that trickle channel has sufficient capacity.

The minimum capacity of the trickle channels is:

Min. $Q_{T_c} = 0.05 * Q_{100}$

Min. $Q_c = 9.6\text{cfs}$

Since the capacity of the proposed trickle channel (13.2 cfs) is greater than the required capacity (9.6 cfs), the proposed trickle channel is adequate.

**Step 5:** Determine the freeboard required for the proposed channel.

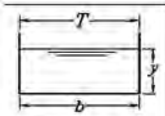
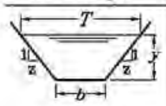
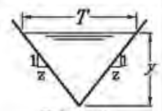
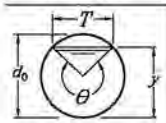
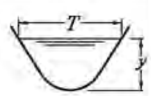
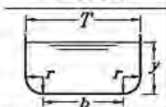
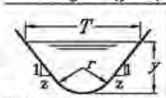
$$F_b = 0.5 + \frac{V^2}{2g}$$

$F_b = 0.5 + \dfrac{(2.5)^2}{2*32.2}$ = 0.6 feet,
but minimum = 1.0 feet

Therefore use $F_b$ = 1.0 feet.

**Step 6:** The cross-section of the proposed channel is shown in **Figure 819**.

# GEOMETRIC ELEMENTS OF CHANNEL SECTIONS

| SECTION | AREA, $A$ | WETTED PERIMETER, $P$ | HYDRAULIC RADIUS, $R$ | TOP WIDTH, $T$ | HYDRAULIC DEPTH, $D$ | SECTION FACTOR, $Z$ |
|---|---|---|---|---|---|---|
|  Rectangle | $by$ | $b+2y$ | $\frac{by}{b+2y}$ | $b$ | $y$ | $by^{1.5}$ |
|  Trapezoid | $(b+zy)y$ | $b+2y\sqrt{1+z^2}$ | $\frac{(b+zy)y}{b+2y\sqrt{1+z^2}}$ | $b+2zy$ | $\frac{(b+zy)y}{b+2zy}$ | $\frac{[(b+zy)y]^{1.5}}{\sqrt{b+2zy}}$ |
|  Triangle | $zy^2$ | $2y\sqrt{1+z^2}$ | $\frac{zy}{2\sqrt{1+z^2}}$ | $2zy$ | $\frac{y}{2}$ | $\frac{\sqrt{2}}{2}zy^{2.5}$ |
|  Circle | $\frac{1}{8}(\theta-\sin\theta)d_0^2$ | $\frac{1}{2}\theta d_0$ | $\frac{1}{4}\left(1-\frac{\sin\theta}{\theta}\right)d_0$ | $(\sin\frac{1}{2}\theta)d_0$ or $2\sqrt{y(d_0-y)}$ | $\frac{1}{8}\left(\frac{\theta-\sin\theta}{\sin\frac{1}{2}\theta}\right)d_0$ | $\frac{\sqrt{2}}{32}\frac{(\theta-\sin\theta)^{1.5}}{(\sin\frac{1}{2}\theta)^{0.5}}d_0^{2.5}$ |
|  Parabola | $\frac{2}{3}Ty$ | $T+\frac{8}{3}\frac{y^2}{T}$ * | $\frac{2T^2y}{3T^2+8y^2}$ * | $\frac{3}{2}\frac{A}{y}$ | $\frac{2y}{3}$ | $\frac{2}{9}\sqrt{6}\,Ty^{1.5}$ |
|  Round-cornered rectangle (y>r) | $\left(\frac{\pi}{2}-2\right)r^2+(b+2r)y$ | $(\pi-2)r+b+2y$ | $\frac{(\pi/2-2)r^2+(b+2r)y}{(\pi-2)r+b+2y}$ | $b+2r$ | $\frac{(\pi/2-2)r^2}{b+2r}+y$ | $\frac{[(\pi/2-2)r^2+(b+2r)y]^{1.5}}{\sqrt{b+2r}}$ |
|  Round-bottom triangle | $\frac{T^2}{4z}-\frac{r^2}{z}(1-z\cot^{-1}z)$ | $\frac{T}{z}\sqrt{1+z^2}-\frac{2r}{z}(1-z\cot^{-1}z)$ | $\frac{A}{P}$ | $2[z(y-r)+r\sqrt{1+z^2}]$ | $\frac{A}{T}$ | $A\sqrt{\frac{A}{T}}$ |

*Satisfactory approximation for the interval $0<x\leq 1$, where $x=4y/T$. When $x>1$, use the exact expression $P=(T/2)[\sqrt{1+x^2}+1/x\ln(x+\sqrt{1+x^2})]$

| Revision | Date |
|---|---|
| ORIGINAL ISSUE | 3/27/06 |
| | |
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WRC ENGINEERING, INC.

REFERENCE:

CHOW, V.T., OPEN CHANNEL HYDRAULICS
McGRAW HILL BOOK COMPANY 1959

TABLE 801

# TYPICAL ROUGHNESS COEFFICIENTS FOR OPEN CHANNELS

| TYPE OF CHANNEL AND DESCRIPTION | MINIMUM | NORMAL | MAXIMUM |
|---|---|---|---|
| EXCAVATED OR DREDGED | | | |
| a. Earth, straight and uniform | | | |
| 1. Clean, recently completed | 0.016 | 0.018 | 0.020 |
| 2. Clean, after weathering | 0.018 | 0.022 | 0.025 |
| 3. Gravel, uniform section, clean | 0.022 | 0.025 | 0.030 |
| 4. With short grass, few weeds | 0.022 | 0.027 | 0.033 |
| b. Earth, winding and sluggish | | | |
| 1. No vegetation | 0.023 | 0.025 | 0.030 |
| 2. Grass, some weeds | 0.025 | 0.030 | 0.033 |
| 3. Dense weeds or aquatic plans in deep channels | 0.030 | 0.035 | 0.040 |
| 4. Earth bottom and rubble sides | 0.028 | 0.030 | 0.035 |
| 5. Stony bottom and weedy banks | 0.025 | 0.035 | 0.040 |
| 6. Cobble bottom and clean sides | 0.030 | 0.040 | 0.050 |
| c. Dragline-excavated or dredged | | | |
| 1. No vegetation | 0.025 | 0.028 | 0.033 |
| 2. Light brush on banks | 0.035 | 0.050 | 0.060 |
| d. Rock cuts | | | |
| 1. Smooth and uniform | 0.025 | 0.035 | 0.040 |
| 2. Jagged and irregular | 0.035 | 0.040 | 0.050 |
| e. Channels not maintained, weeds and brush | | | |
| 1. Dense weeds, high as flow depth | 0.050 | 0.080 | 0.120 |
| 2. Clean bottom, brush on sides | 0.040 | 0.050 | 0.080 |
| 3. Same as above, but highest state of flow | 0.045 | 0.070 | 0.110 |
| 4. Dense brush, high state | 0.080 | 0.100 | 0.140 |

| Revision | Date |
|---|---|
| ORIGINAL ISSUE | 3/27/06 |

# TYPICAL ROUGHNESS COEFFECIENTS FOR OPEN CHANNELS

| TYPE OF CHANNEL & DESCRIPTION | MINIMUM | NORMAL | MAXIMUM |
|---|---|---|---|
| Brass, smooth | 0.009 | 0.010 | 0.013 |
| Steel: | | | |
| Lockbar and welded | 0.010 | 0.012 | 0.014 |
| Riveted and spiral | 0.013 | 0.016 | 0.017 |
| Cast Iron: | | | |
| Coated | 0.010 | 0.013 | 0.014 |
| Uncoated | 0.011 | 0.014 | 0.016 |
| Wrought Iron: | | | |
| Black | 0.012 | 0.014 | 0.015 |
| Galvanized | 0.013 | 0.016 | 0.017 |
| Corrugated Metal: | | | |
| Sub-drain | 0.017 | 0.019 | 0.021 |
| Storm Drain | 0.021 | 0.024 | 0.030 |
| Lucite | 0.008 | 0.009 | 0.010 |
| Glass | 0.009 | 0.010 | 0.013 |
| Cement: | | | |
| Neat, surface | 0.010 | 0.011 | 0.013 |
| Mortar | 0.011 | 0.013 | 0.015 |
| Concrete: | | | |
| Culvert, straight and free of debris | 0.010 | 0.011 | 0.013 |
| Culvert with bends, connections, and some debris | 0.011 | 0.013 | 0.014 |
| Finished | 0.011 | 0.012 | 0.014 |
| Sewer with manholes, inlet, etc., straight | 0.013 | 0.015 | 0.017 |
| Unfinished, steel form | 0.012 | 0.013 | 0.014 |
| Unfinished, smooth wood form | 0.012 | 0.014 | 0.016 |
| Unfinished, rough wood form | 0.015 | 0.017 | 0.020 |
| Wood: | | | |
| Stave | 0.010 | 0.012 | 0.014 |
| Laminated, treated | 0.015 | 0.017 | 0.020 |
| Clay: | | | |
| Common drainage tile | 0.011 | 0.013 | 0.017 |
| Vitrified sewer | 0.011 | 0.014 | 0.017 |
| Vitrified sewer with manholes, inlet, etc. | 0.013 | 0.015 | 0.017 |
| Vitrified subdrain with open joint | 0.014 | 0.016 | 0.018 |
| Brickwork: | | | |
| Glazed | 0.011 | 0.013 | 0.015 |
| Lined with cement mortar | 0.012 | 0.015 | 0.017 |
| Sanitary sewers coated with sewage slime with bends and connections | 0.012 | 0.013 | 0.016 |
| Paved invert, sewer, smooth bottom | 0.016 | 0.019 | 0.020 |
| Rubble masonry, cemented | 0.018 | 0.025 | 0.030 |

| Revision | Date |
|---|---|
| ORIGINAL ISSUE | 3/27/06 |
| | |
| | |

# TYPICAL ROUGHNESS COEFFICIENTS FOR OPEN CHANNELS

| TYPE OF CHANNEL AND DESCRIPTION | MINIMUM | NORMAL | MAXIMUM |
|---|---|---|---|
| **LINED OR BUILT-UP CHANNELS** | | | |
| a. CONCRETE | | | |
| 1. TROWEL FINISH | 0.011 | 0.013 | 0.015 |
| 2. FLOAT FINISH | 0.013 | 0.015 | 0.016 |
| 3. GUNITE, GOOD SECTION | 0.016 | 0.019 | 0.023 |
| 4. GUNITE, WAVY SECTION | 0.018 | 0.022 | 0.023 |
| b. CONCRETE BOTTOM FLOAT FINISHED WITH SIDE OF | | | |
| 1. DRESSED STONE IN MORTAR | 0.015 | 0.017 | 0.020 |
| 2. RANSOM STONE IN MORTAR | 0.017 | 0.020 | 0.024 |
| 3. DRY RUBBLE OR RIPRAP | 0.020 | 0.030 | 0.035 |
| c. GRAVEL BOTTOM WITH SIDES OF | | | |
| 1. FORMED CONCRETE | 0.017 | 0.020 | 0.025 |
| 2. RANDOM STONE IN MORTAR | 0.020 | 0.023 | 0.026 |
| 3. DRY RUBBLE OR RIPRAP | 0.023 | 0.033 | 0.036 |
| d. ASPHALT | | | |
| 1. SMOOTH | 0.013 | 0.013 | -- |
| 2. ROUGH | 0.016 | 0.016 | -- |
| e. GRASSED | 0.030 | 0.040 | 0.050 |

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|---|---|
| ORIGINAL ISSUE | 3/27/06 |

WRC ENGINEERING, INC.

REFERENCE:
CHOW, V.T., OPEN CHANNEL HYDRAULICS
McGRAW HILL BOOK COMPANY 1959

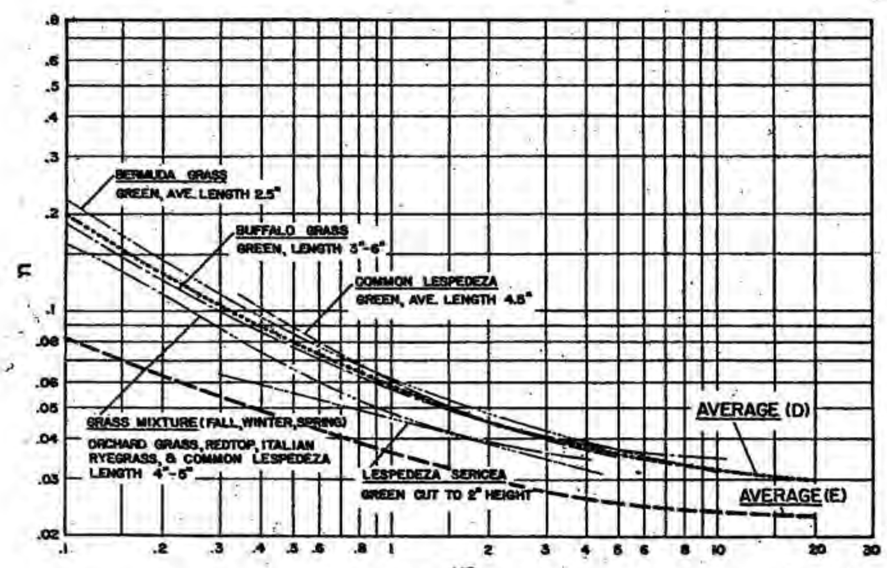
**TABLE 802C**

# TYPICAL ROUGHNESS COEEFICIENTS FOR OPEN CHANNELS

## GRASSED CHANNELS WITH LOW RETARDANCE COEFFICIENT*

| Retardance | Cover | Condition |
|---|---|---|
| D Low | Bermuda grass | Good stand, cut to 2.5 in. height |
| | Common lespedeza | Excellent stand, uncut (av 4.5 in.) |
| | Buffalo grass | Good stand, uncut (3 to 6 in.) |
| | G ass-legume mixture—fall, spring (orchard grass, redtop, Italian rye grass, and common lespedeza) | Good stand, uncut (4 to 5 in.) |
| | Lespedeza sericea | After cutting to 2 in. height, very good stand before cutting |
| E Very low | Bermuda grass | Good stand, cut to 1.5 in. height |
| | Bermuda grass | Burned stubble |

STAND=DENSITY OF GRASS(NUMBER OF STEMS PER SQUARE FOOT)




V=MEAN VELOCITY
R=HYDRAULIC RADIUS

| Revision | Date |
|---|---|
| ORIGINAL ISSUE | 3/27/06 |

* THE 'n' ROUGHNESS COEFFICIENT FOR GRASSED CHANNELS IS KNOWN AS RRETARDANCE COEFFICIENT.

## MAXIMUM PERMISSIBLE MEAN CHANNEL VELOCITY

| MATERIAL / LINING | MAXIMUM PERMISSIBLE MEAN VELOCITY (FPS) |
|---|---|
| NATURAL AND IMPROVED UNLINED CHANNELS | |
| Erosive Soils: | |
| Loams, Sands, Noncolloidal Silts | 3.0 |
| Less Erosive Soils: | |
| Clays, Shales, Cobbles, Gravel | 5.0 |
| FULLY-LINED CHANNELS | |
| Unreinforced Vegetation | 5.5 |
| Loose Riprap | |
| Angular Rock | 15.0 |
| Semi-Angular Rock | 12.0 |
| Rounded Rock | See Note #4 |
| Grouted Riprap | 15.0 |
| Gabions | 15.0 |
| Soil Cement | 15.0 |
| Concrete | 20.0 |

NOTES:

1. For composite lined channels, us the lowest of the maximum mean velocities for the materials used in the composite lining.
2. Deviations from the above values are only allowed with appropriate engineering analysis and/or suitable agreements for maintenance responsibilities.
3. Maximum permissible velocities based upon non-clear water conditions.
4. Suitability of rounded rock as loose riprap material shall be determined by rock particle resistance to movement as a result of shear forces as calculated with a factor of safety of 1.5.

| Revision | Date |
|---|---|
| ORIGINAL ISSUE | 3/27/06 |
| | |
| | |

# GRADATION OF LOOSE RIPRAP

| STONE SIZE d50[1] (INCHES) | % OF MATERIAL SMALLER THAN TYPICAL STONE[2] | TYPICAL STONE DIMENSIONS[3] (INCHES) | TYPICAL STONE WEIGHT[4] (POUNDS) |
|---|---|---|---|
| 6 | 70–100 | 12 | 85 |
| | 50–70 | 9 | 35 |
| | 35–50 | 6 | 10 |
| | 2–10 | 2 | 0.4 |
| 9 | 70–100 | 15 | 160 |
| | 50–70 | 12 | 85 |
| | 35–50 | 9 | 35 |
| | 2–10 | 3 | 1.3 |
| 12 | 70–100 | 21 | 440 |
| | 50–70 | 18 | 275 |
| | 35–50 | 12 | 85 |
| | 2–10 | 4 | 3 |
| 18 | 100 | 30 | 1280 |
| | 50–70 | 24 | 650 |
| | 35–50 | 18 | 275 |
| | 2–10 | 6 | 10 |
| 24 | 100 | 42 | 3500 |
| | 50–70 | 33 | 1700 |
| | 35–50 | 24 | 650 |
| | 2–10 | 9 | 35 |

[1] d50 = NOMINAL STONE SIZE
[2] BASED ON TYPICAL ROCK MASS
[3] EQUIVALENT SPHERICAL DIAMETER
[4] BASED ON A SPECIFIC GRAVITY = 2.5

| Revision | Date |
|---|---|
| ORIGINAL ISSUE | 3/27/06 |
| | |
| | |

# GRADATION FOR GRANULAR RIPRAP BEDDING

PERCENTAGE PASSING DESIGNATED SIEVES

| U.S. STANDARD SIEVE SIZE (INCHES) | COARSE AGGREAGATE (1 ½" TO #4) | FINE AGGREGATE (#4 TO #100) |
|---|---|---|
| 2 | 100 | – |
| 1 1/2 | – | – |
| 1 | 95–100 | – |
| 3/4 | – | – |
| 1/2 | 25–60 | – |
| 3/8 | – | 100 |
| #4 | 0–10 | 95–100 |
| #8 | 0–5 | 80–100 |
| #16 | – | 50–85 |
| #30 | – | 25–60 |
| #50 | – | 10–30 |
| #100 | – | 2–10 |

| Revision | Date |
|---|---|
| ORIGINAL ISSUE | 3/27/06 |
| | |
| | |

WRC ENGINEERING, INC.

REFERENCE: COLORADO DEPARTMENT OF TRANSPORTATION STANDARD SPECIFICATIONS FOR ROAD AND BRIDGE CONSTRUCTION, 2005

**TABLE 808**

# GRADATION FOR GRANULAR RIPRAP BEDDING

PERCENTAGE PASSING DESIGNATED SIEVES

| U.S. STANDARD SIEVE SIZE (INCHES) | COARSE AGGREAGATE (1 ½" TO #4) | FINE AGGREGATE (#4 TO #100) |
|---|---|---|
| 2 | 100 | – |
| 1 1/2 | – | – |
| 1 | 95–100 | – |
| 3/4 | – | – |
| 1/2 | 25–60 | – |
| 3/8 | – | 100 |
| #4 | 0–10 | 95–100 |
| #8 | 0–5 | 80–100 |
| #16 | – | 50–85 |
| #30 | – | 25–60 |
| #50 | – | 10–30 |
| #100 | – | 2–10 |

| Revision | Date |
|---|---|
| ORIGINAL ISSUE | 3/27/06 |

# CLASSIFICATION AND GRADATION OF ROCK FOR GROUTED RIPRAP

| STONE SIZE d50 (INCHES) | % SMALLER THAN GIVEN SIZE BY WEIGHT | INTERMEDIATE ROCK DIMENSION (INCHES) |
|---|---|---|
| d50=24"(TYPE HG) | 100 | 30 |
| | 50–70 | 24 |
| | 0–5 | 18 |
| d50=18"(TYPE HG) | 70–100 | 21 |
| | 50–70 | 18 |
| | 0–5 | 12 |

| Revision | Date |
|---|---|
| ORIGINAL ISSUE | 3/27/06 |
| | |
| | |

# DESIGN $D_{50}$ VALUES FOR STEEP CHANNELS

| $D_{50}$ DETERMINED FROM DESIGN CURVE (FT) | MINIMUM DESIGN $D_{50}$ (FT) |
|---|---|
| < 0.25 | 0.25 |
| 0.26 - 0.50 | 0.50 |
| 0.51 - 0.75 | 0.75 |
| 0.76 - 1.00 | 1.00 |
| 1.01 - 1.25 | 1.25 |
| 1.26 - 1.50 | 1.50 |
| 1.51 - 1.75 | 1.75 |
| 1.76 - 2.00 | 2.00 |
| 2.01 - 2.25 | 2.25 |
| 2.26 - 2.50 | 2.50 |
| 2.51 - 2.75 | 2.75 |
| 2.76 - 3.00 | 3.00 |

| Revision | Date |
|---|---|
| ORIGINAL ISSUE | 3/27/06 |
| | |
| | |

# RIPRAP GRADATION FOR STEEP SLOPES

$$\frac{D_{MAX}}{D_{50}} = 1.25$$

$$\frac{D_{50}}{D_{20}} = 2.0$$

$$\frac{D_{50}}{D_{10}} = 3.0$$

| Revision | Date |
|---|---|
| ORIGINAL ISSUE | 3/27/06 |
| | |
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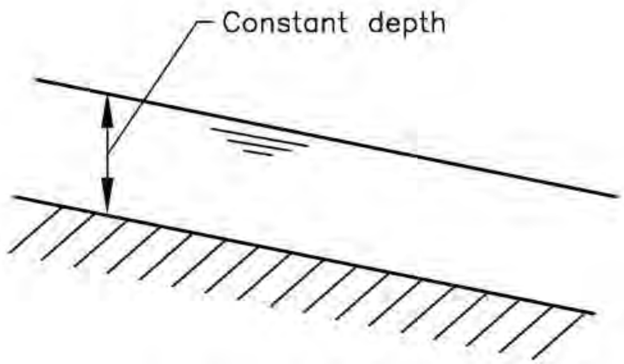
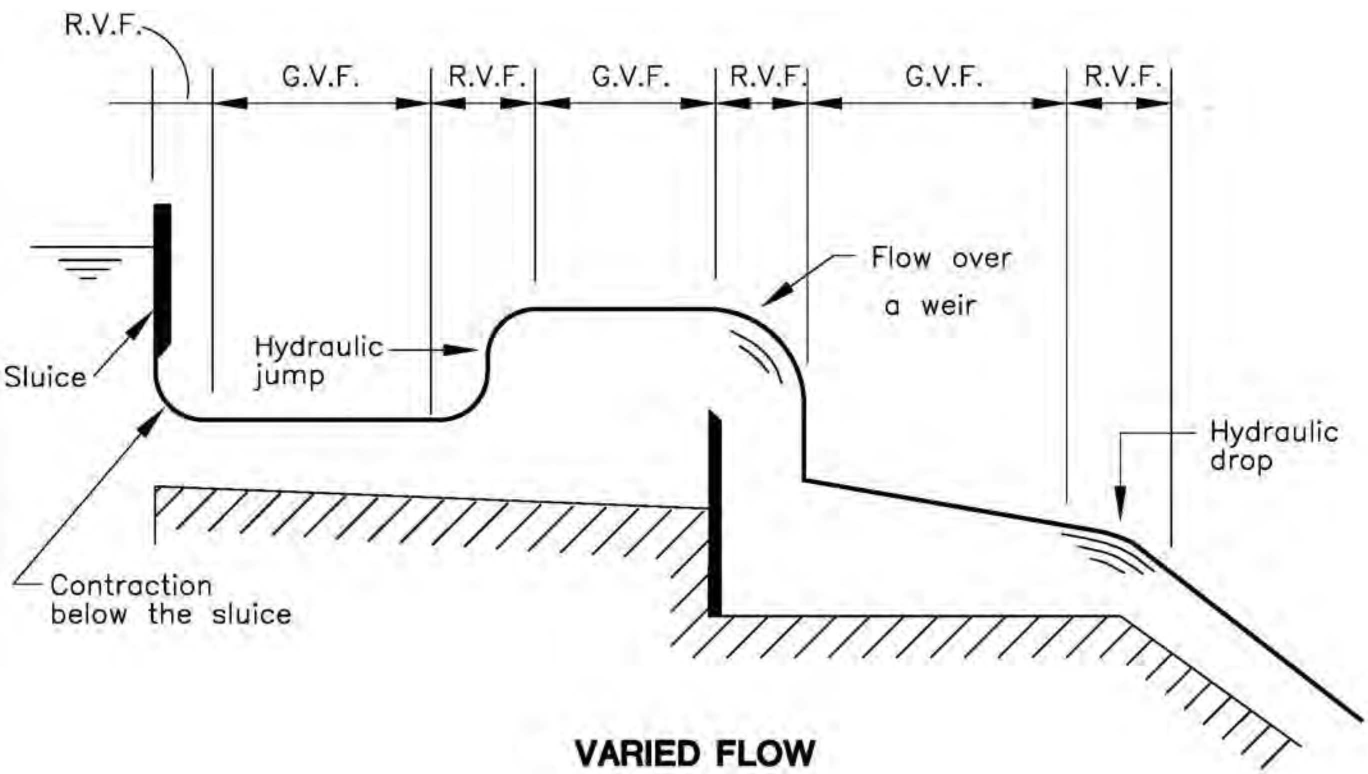
REFERENCE: SIMONS, 1989

# OPEN CHANNEL FLOW CONDITIONS




**UNIFORM FLOW**
Flow in a laboratory channel

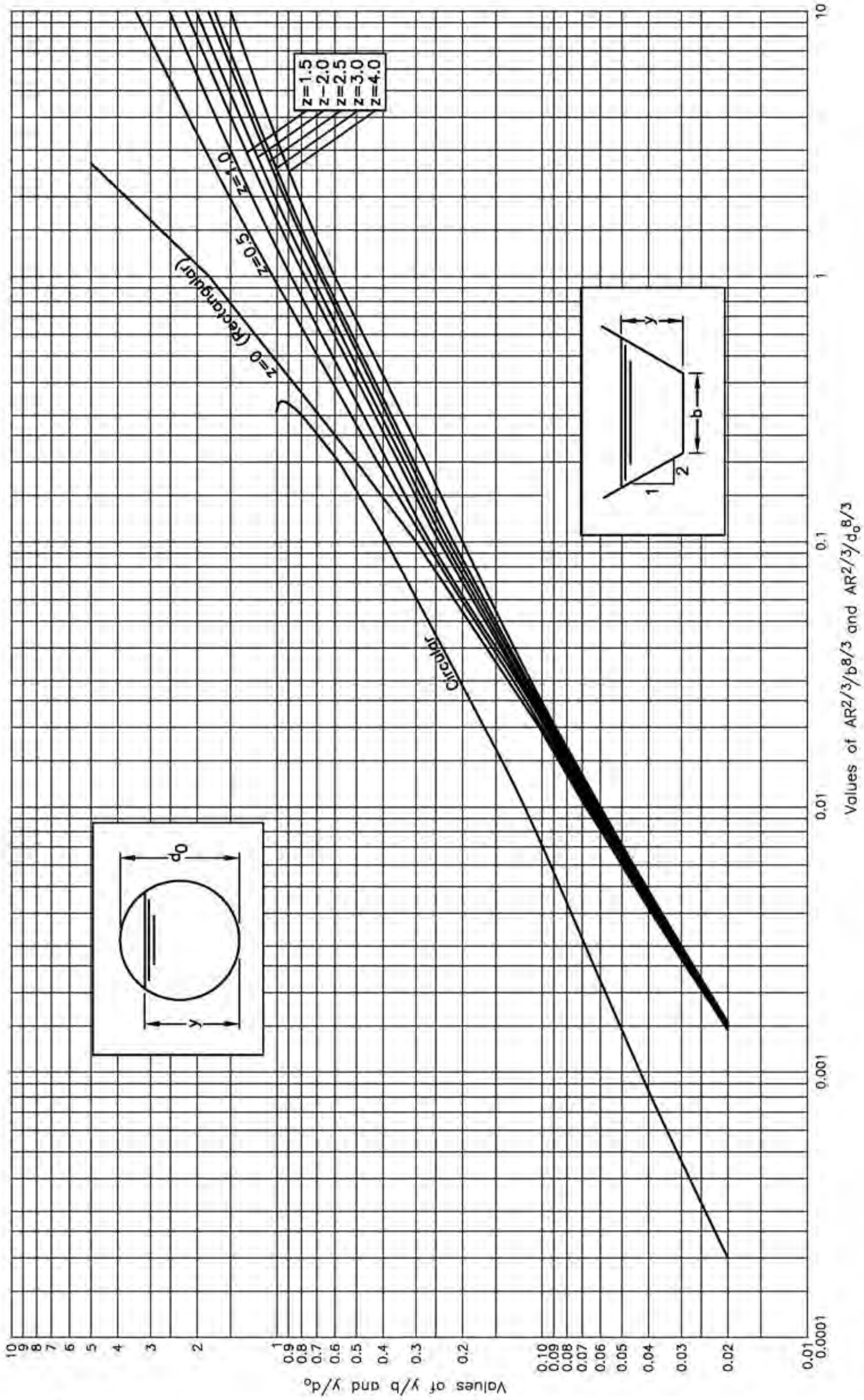



**VARIED FLOW**

G.V.F. — Gradually Varying Flow
R.V.F. — Rapidly Varying Flow

| Revision | Date |
|---|---|
| ORIGINAL ISSUE | 3/27/06 |
| | |
| | |

# CRITICAL DEPTH FOR TRAPEZOIDAL AND CIRCULAR SECTIONS

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|---|---|
| ORIGINAL ISSUE | 3/27/06 |
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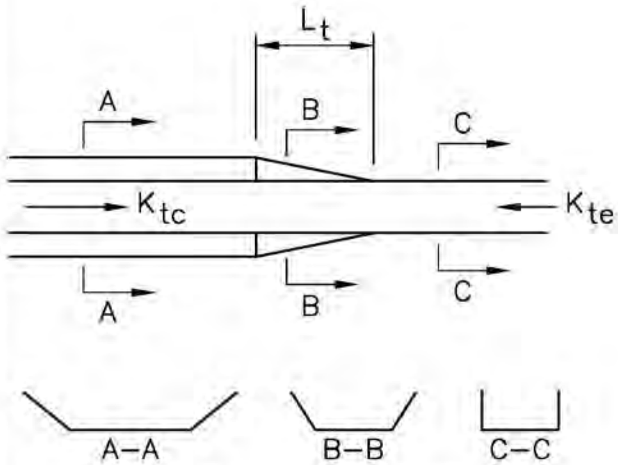
REFERENCE:

CHOW, V.T., OPEN CHANNEL HYDRAULICS
McGRAW HILL BOOK COMPANY 1959

**FIGURE 803**

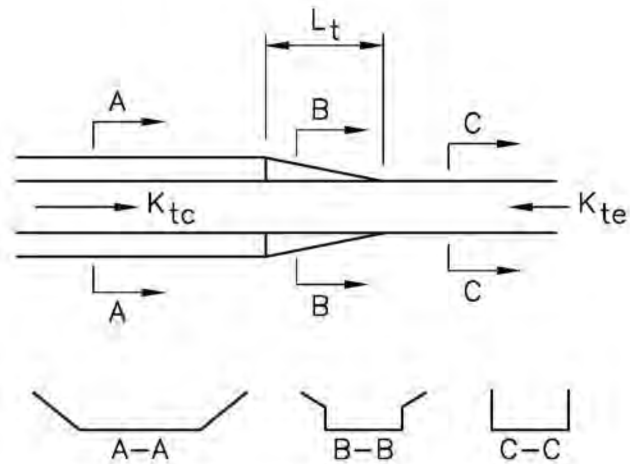
# TYPICAL SUBCRITICAL TRANSITION SECTIONS AND LOSS COEFFICIENTS ($K_{tc}$)




WARPED TRANSITION

$K_{tc}$ (CONTRACTION)=0.1

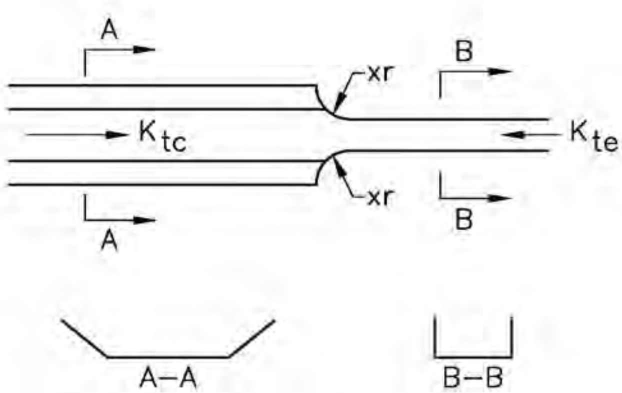
$K_{te}$ (EXPANSION)=0.2




STRAIGHT–LINE TRANSITION

$K_{tc}$ (CONTRACTION)=0.3

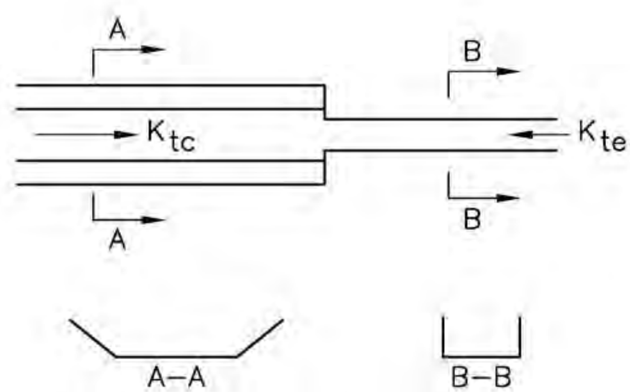
$K_{te}$ (EXPANSION)=0.5




CYLINDER–QUADRANT

$K_{tc}$ =0.15

$K_{te}$ =0.25




SQUARE–ENDED TRANSITION

$K_{tc}$ =0.30

$K_{te}$ =0.75

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|---|---|
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| | |
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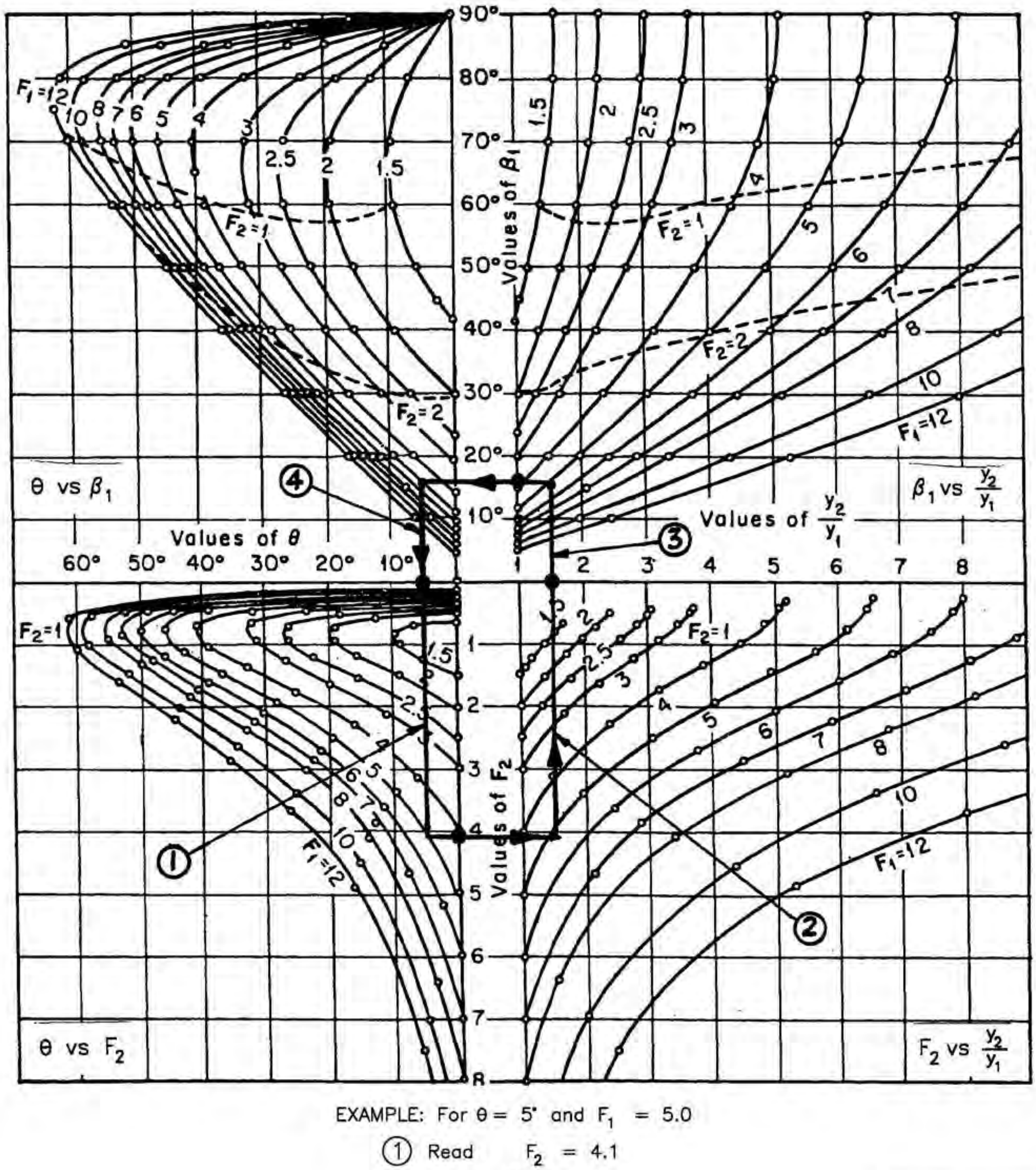
REFERENCE:

**FIGURE 804**

# DESIGN NOMOGRAPH FOR SUPERCRITICAL CONTRACTION TRANSITION LENGTH AND WAVE ANGLE




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 |
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| ORIGINAL ISSUE | 3/27/06 |
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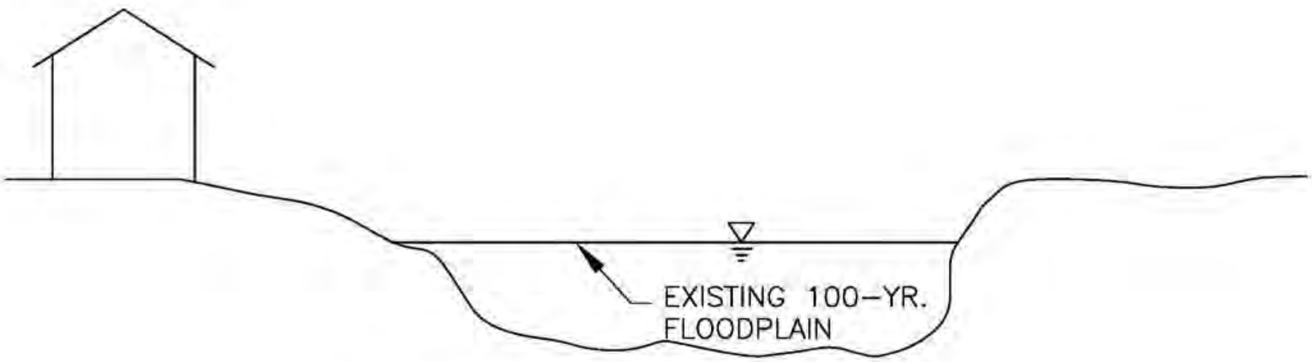
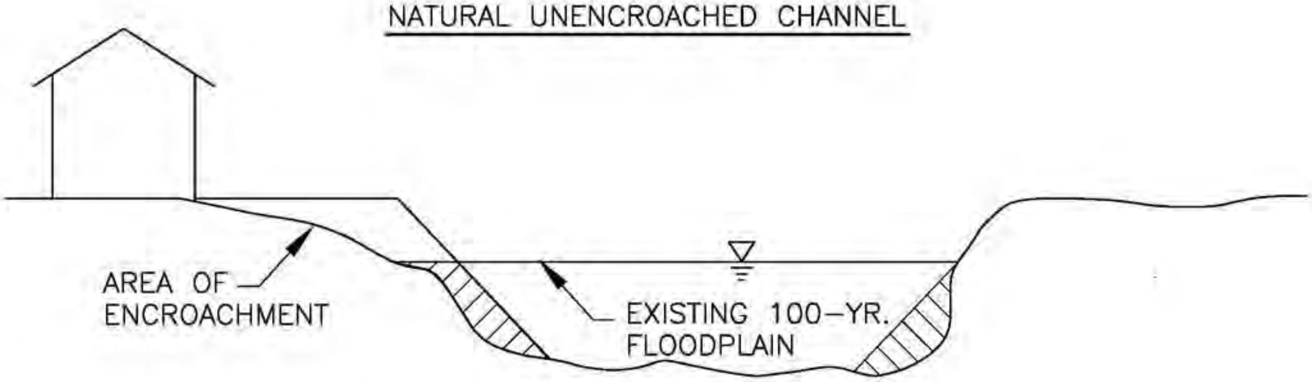
REFERENCE:

CHOW, V.T., OPEN CHANNEL HYDRAULICS, McGRAW HILL BOOK COMPANY, 1959

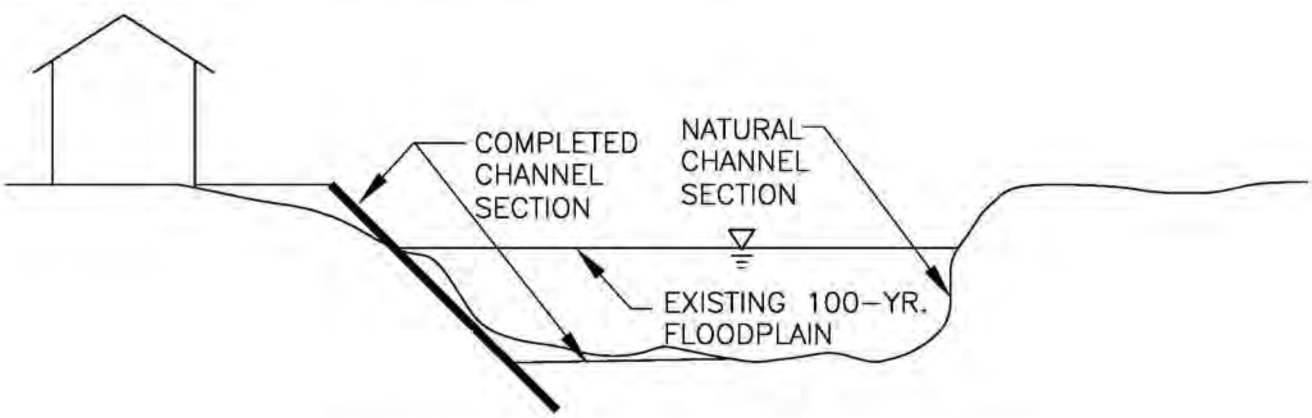
# TYPICAL NATURAL OPEN CHANNEL DESIGN SECTIONS




NATURAL UNENCROACHED CHANNEL




NATURAL ENCROACHED CHANNEL




BANK LINED AND TEMPORARY UNLINED CHANNEL

| Revision | Date |
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| ORIGINAL ISSUE | 3/27/06 |
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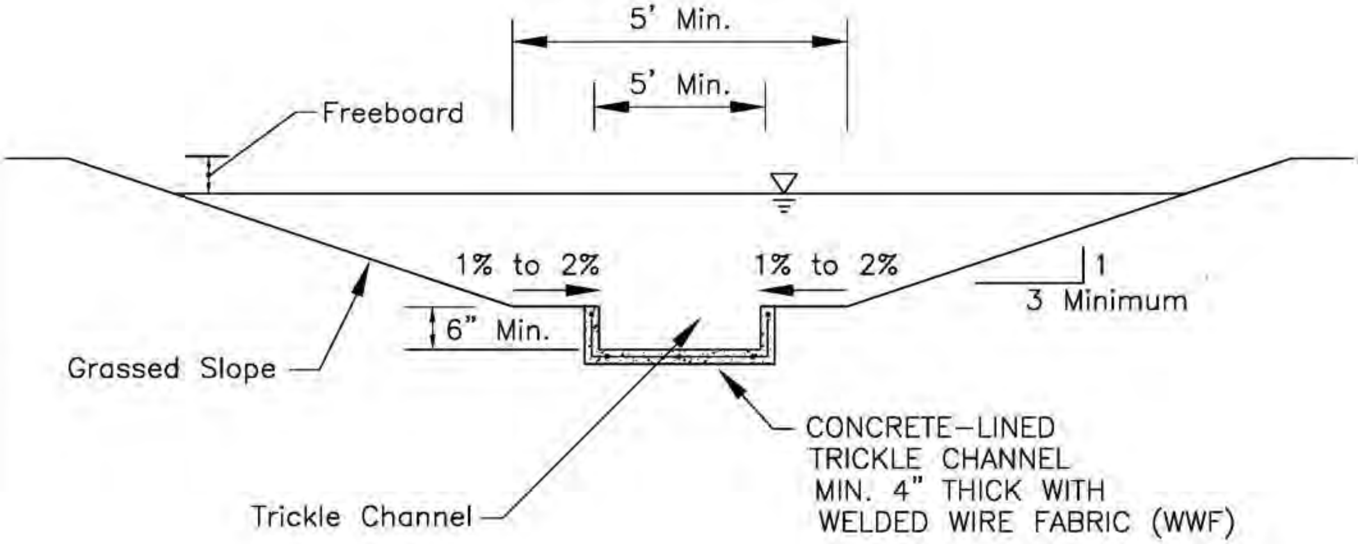
REFERENCE:

**FIGURE 807**

# TYPICAL CROSS-SECTION OF CONCRETE-LINED TRICKLE CHANNEL

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| ORIGINAL ISSUE | 3/27/06 |
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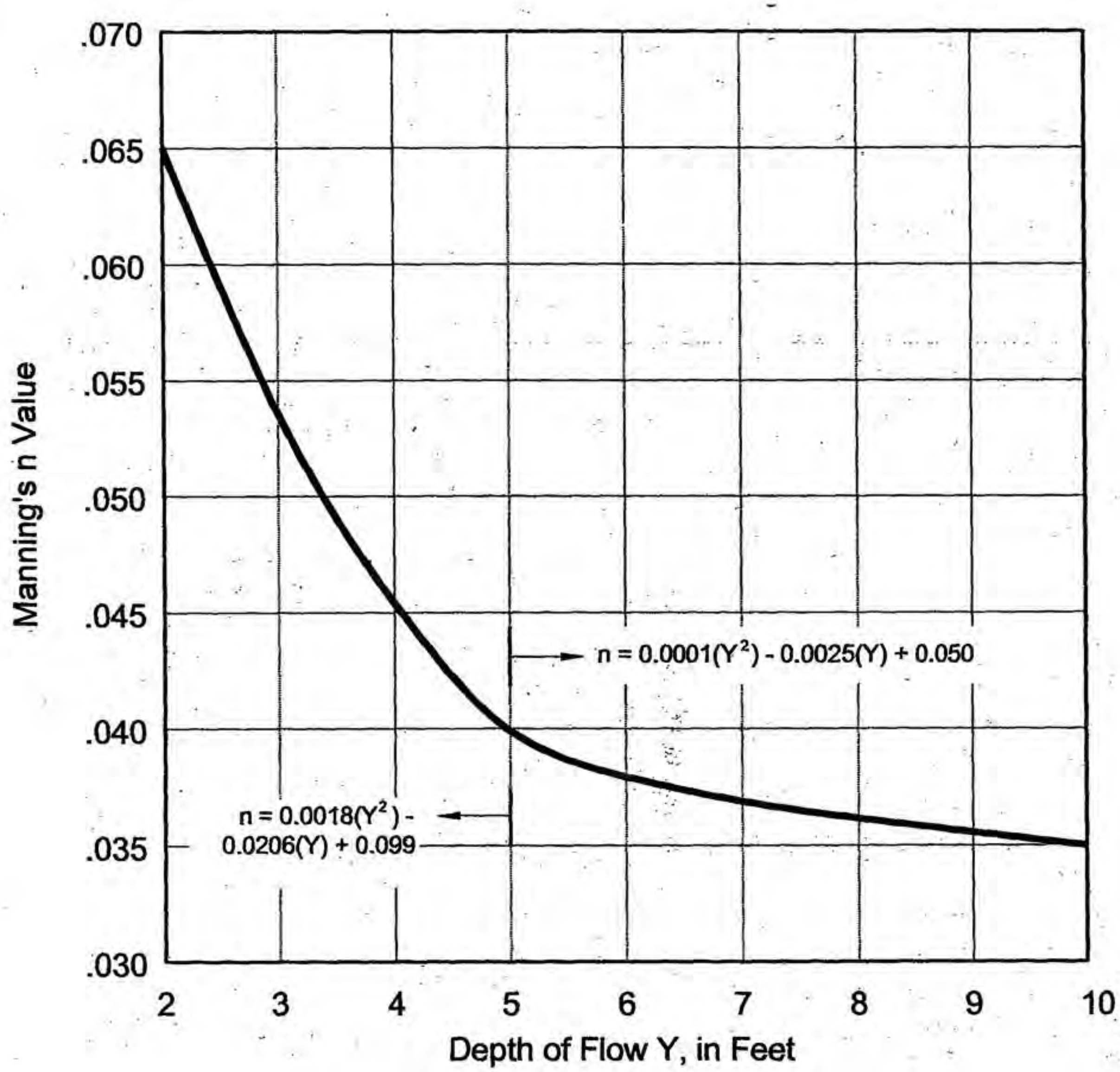
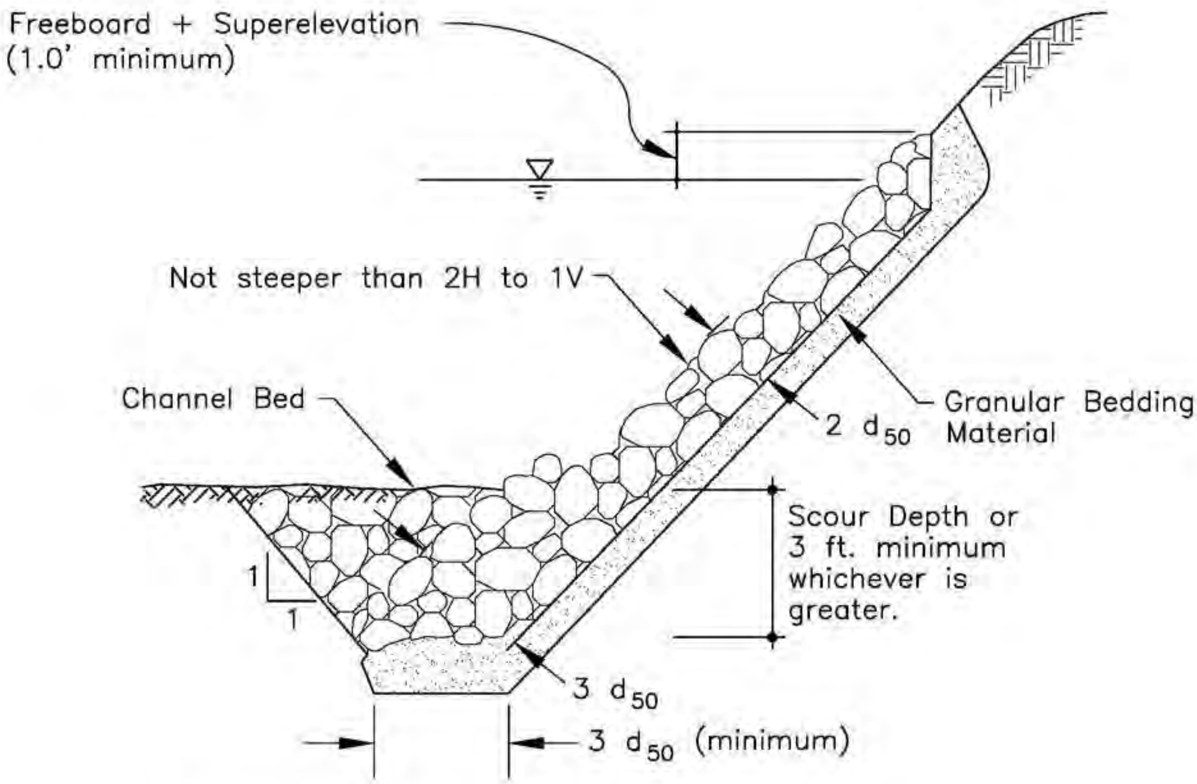
REFERENCE: UDFCD MANUAL (V.1)

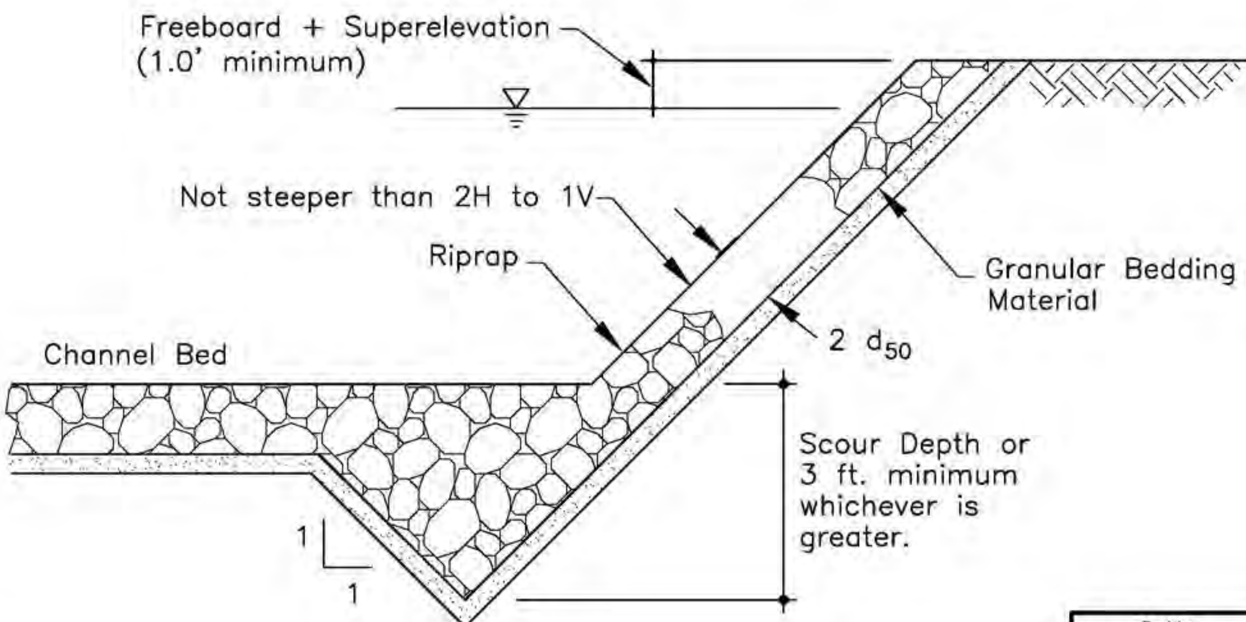
**FIGURE 808**

# MANNING n VS. DEPTH FOR COMPOSITE CHANNEL AND LOW-FLOW SECTION DESIGN



# TYPICAL CROSS SECTIONS FOR RIPRAP-LINED CHANNELS

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| ORIGINAL ISSUE | 3/27/06 |
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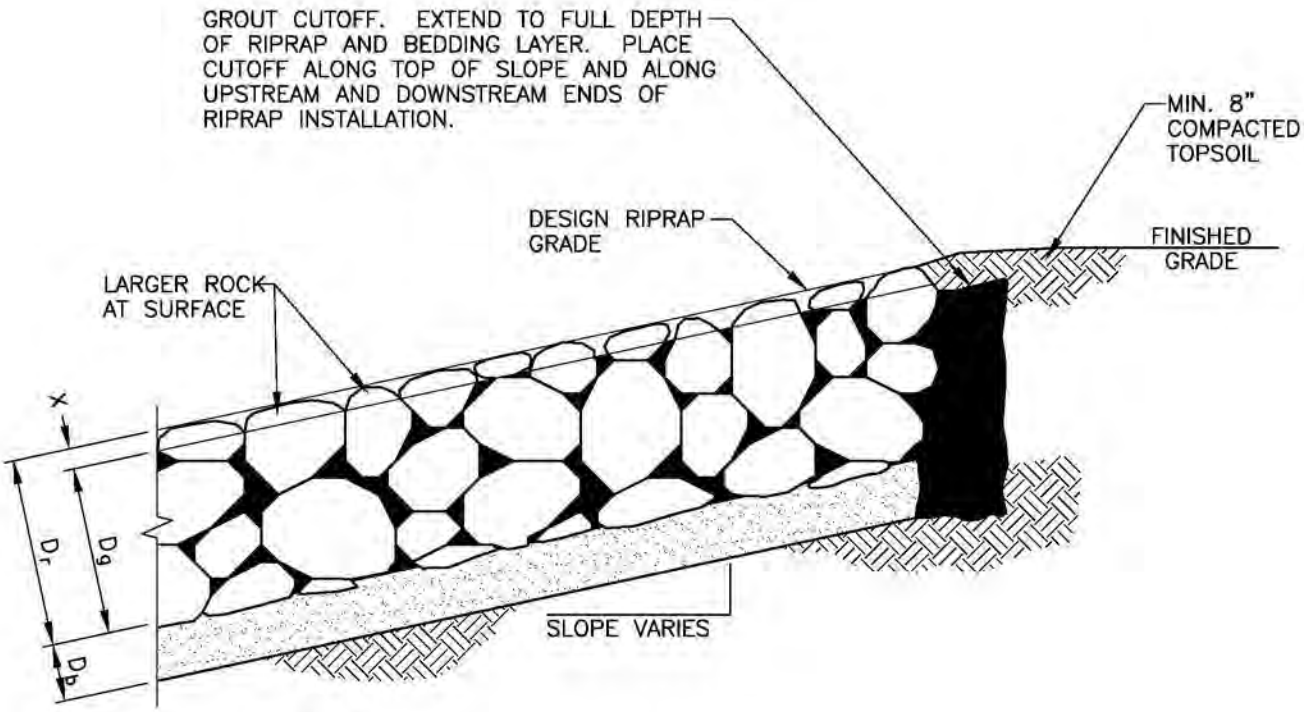
REFERENCE:

MODIFIED FROM H.E.C. NO. 11, USDOT, 1989

**FIGURE 810**

# TYPICAL CROSS SECTION FOR GROUTED RIPRAP LINING




LEGEND:

$D_b$ = DEPTH OF BEDDING MATERIAL

$D_g$ = DEPTH OF GROUT LAYER

$D_r$ = DEPTH OF RIPRAP LAYER

X = DEPTH FROM RIPRAP SURFACE TO GROUT SURFACE

NOTES:

1. FINAL PLACEMENT OF RIPRAP TO BE APPROVED BY ENGINEER PRIOR TO GROUTING.
2. BEFORE GROUTING, CLEAN ALL DIRT AND MATERIALS FROM ROCK THAT COULD PREVENT THE GROUT FROM BONDING TO ROCK.
3. PLACE GROUT BY INJECTION METHODS AND USE A PENCIL VIBRATOR TO FILL VOIDS TO THE SPECIFIED GROUT DEPTH. CLEAN EXCESS GROUT FROM ALL EXPOSED SURFACES. PROVIDE A BROOM FINISH FOR GROUT SURFACE.
4. THE CONTRACTOR SHALL CONTROL GROUT MIX AND PLACEMENT PROCEDURES TO ACHIEVE THE SPECIFIED THICKNESS, PENETRATION AND GRADE OF THE GROUT LAYER.

| Revision | Date |
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| ORIGINAL ISSUE | 3/27/06 |
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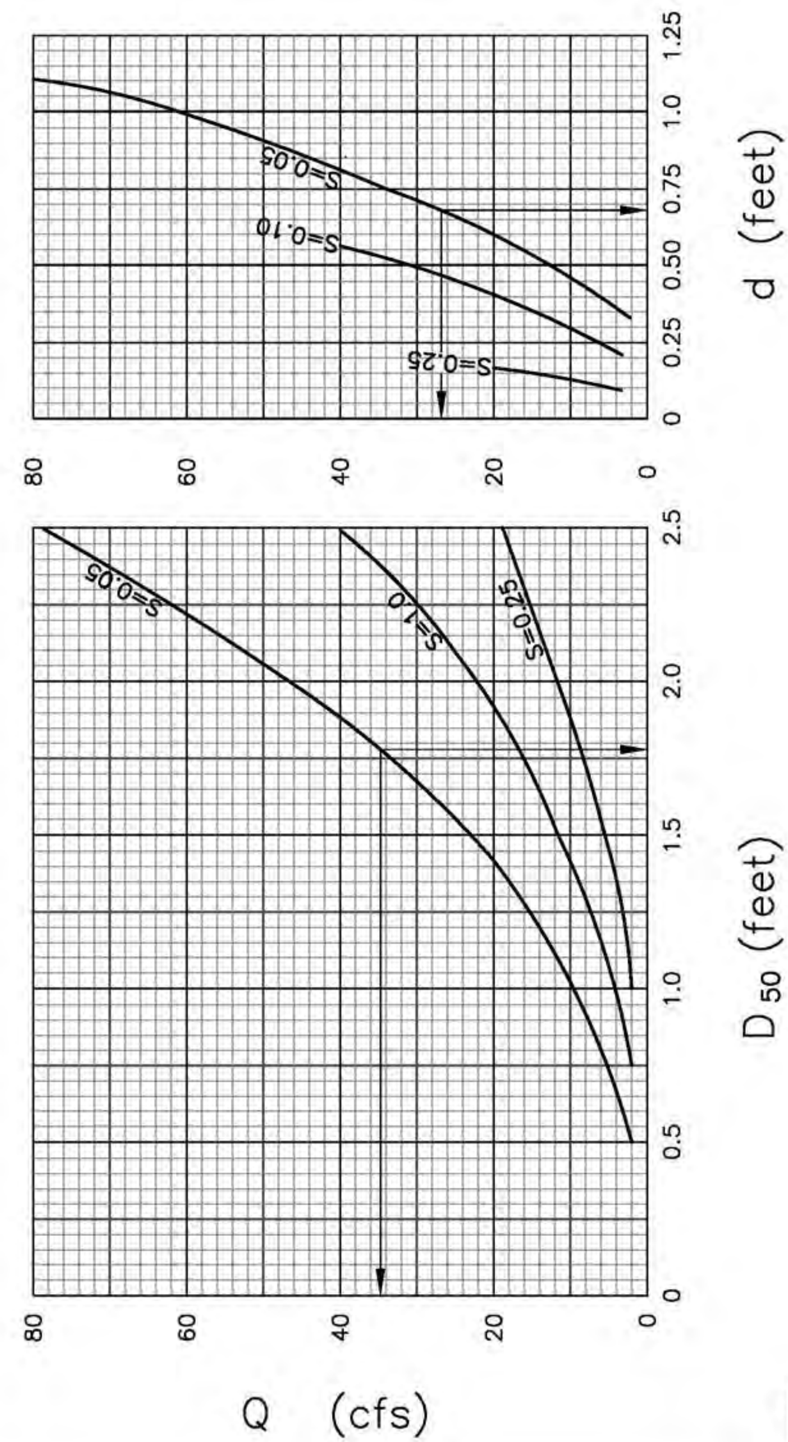
REFERENCE: UDFCD, 1990

**FIGURE 811**

# STEEP SLOPE RIPRAP DESIGN, TRIANGULAR CHANNEL, 2:1 SIDE SLOPES

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| ORIGINAL ISSUE | 3/27/06 |
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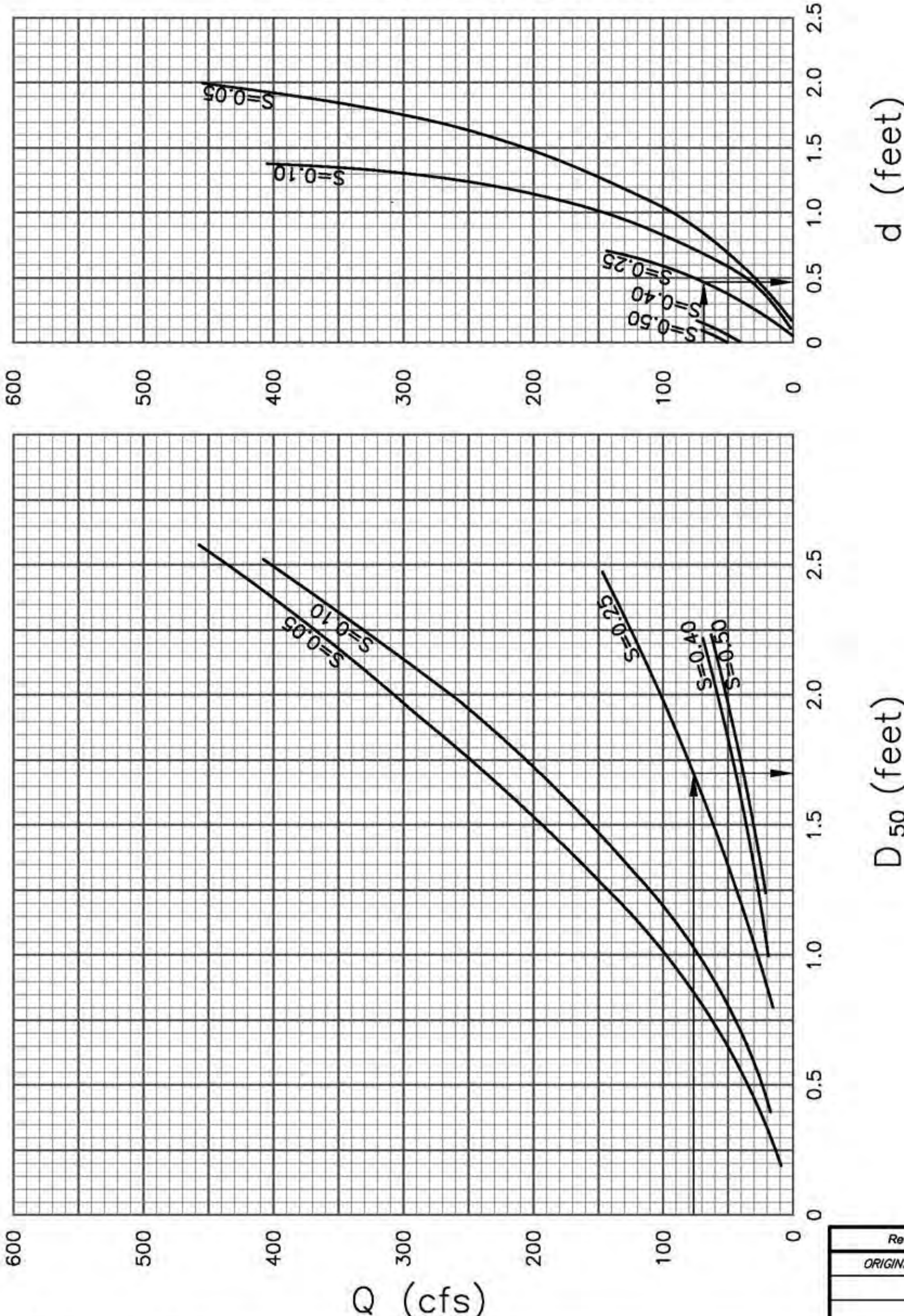
WRC ENGINEERING, INC

REFERENCE:
SIMON, LI AND ASSOC., 1989

**FIGURE 812**

# STEEP SLOPE RIPRAP DESIGN, TRAPEZOIDAL CHANNEL, 2:1 SIDE SLOPES, 6-FOOT BOTTOM WIDTH

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| ORIGINAL ISSUE | 3/27/06 |
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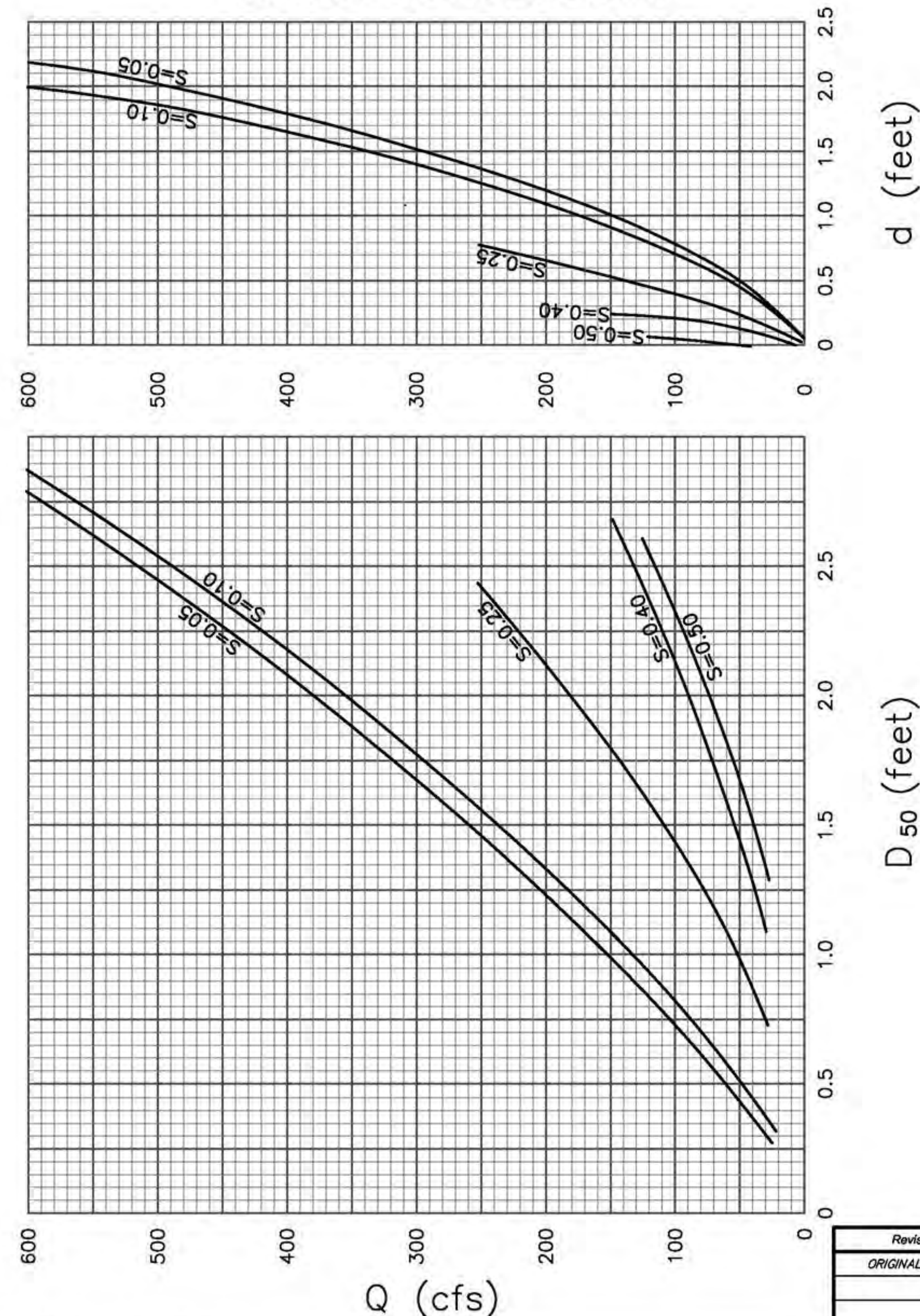
REFERENCE:

SIMON, LI AND ASSOC., 1989

**FIGURE 813**

# STEEP SLOPE RIPRAP DESIGN, TRAPEZOIDAL CHANNEL, 2:1 SIDE SLOPES, 10-FOOT BOTTOM WIDTH

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| ORIGINAL ISSUE | 3/27/06 |

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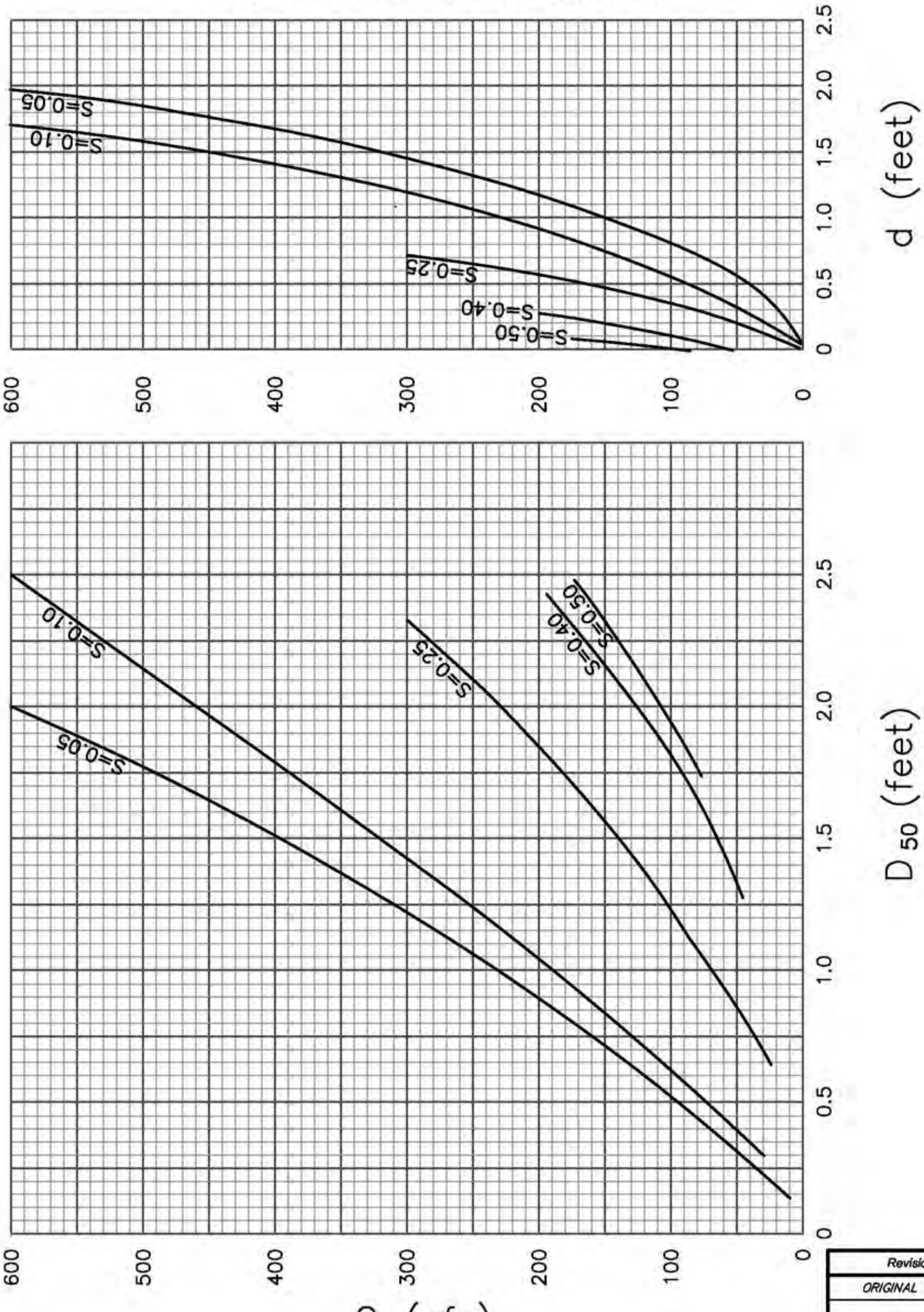
REFERENCE:

SIMON, LI AND ASSOC., 1989

**FIGURE 814**

# STEEP SLOPE RIPRAP DESIGN, TRAPEZOIDAL CHANNEL, 2:1 SIDE SLOPES, 14-FOOT BOTTOM WIDTH

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| ORIGINAL ISSUE | 3/27/06 |
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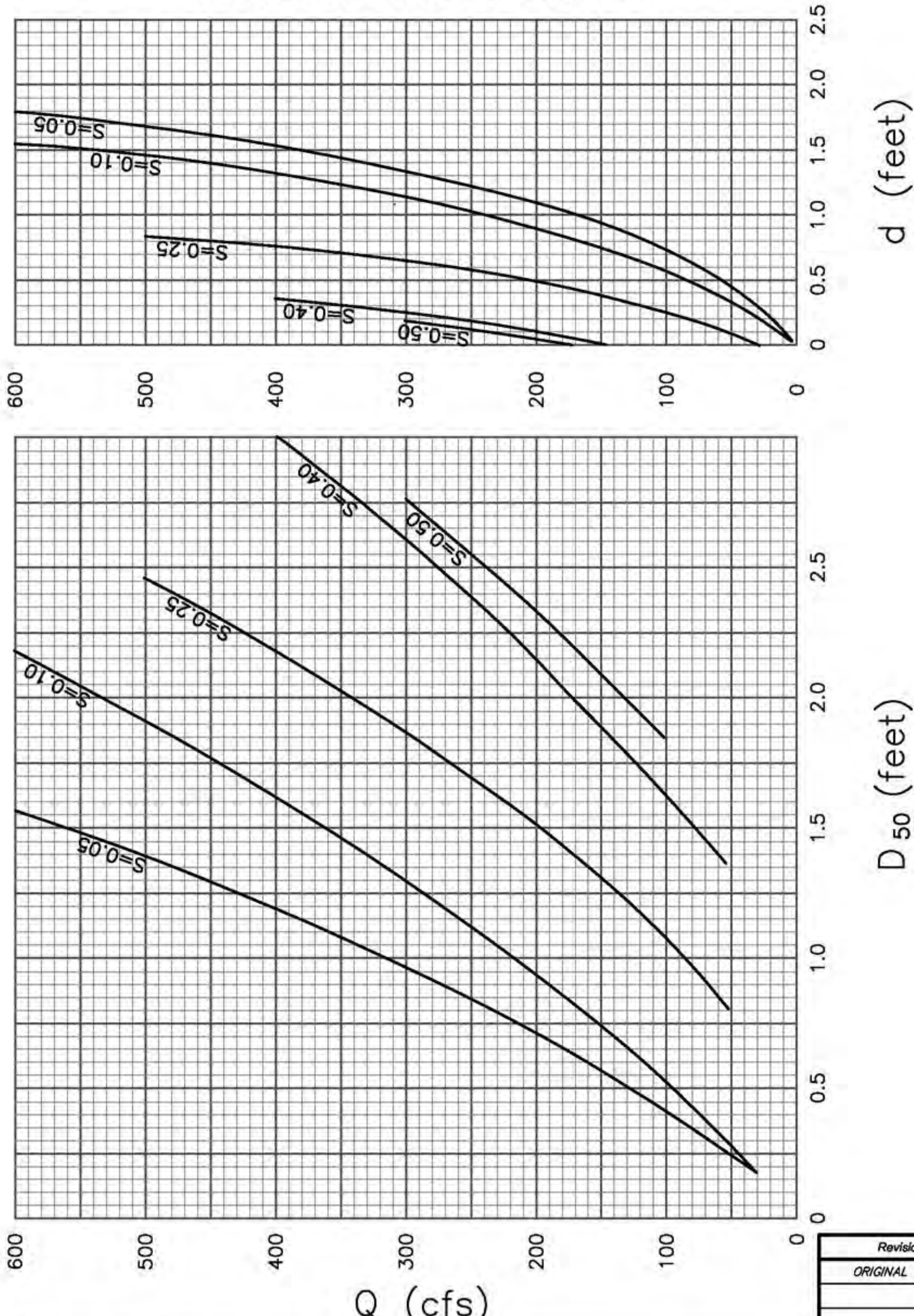
REFERENCE:

SIMON, LI AND ASSOC., 1989

**FIGURE 815**

# STEEP SLOPE RIPRAP DESIGN, TRAPEZOIDAL CHANNEL, 2:1 SIDE SLOPES, 20-FOOT BOTTOM WIDTH




| Revision | Date |
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| ORIGINAL ISSUE | 3/27/06 |
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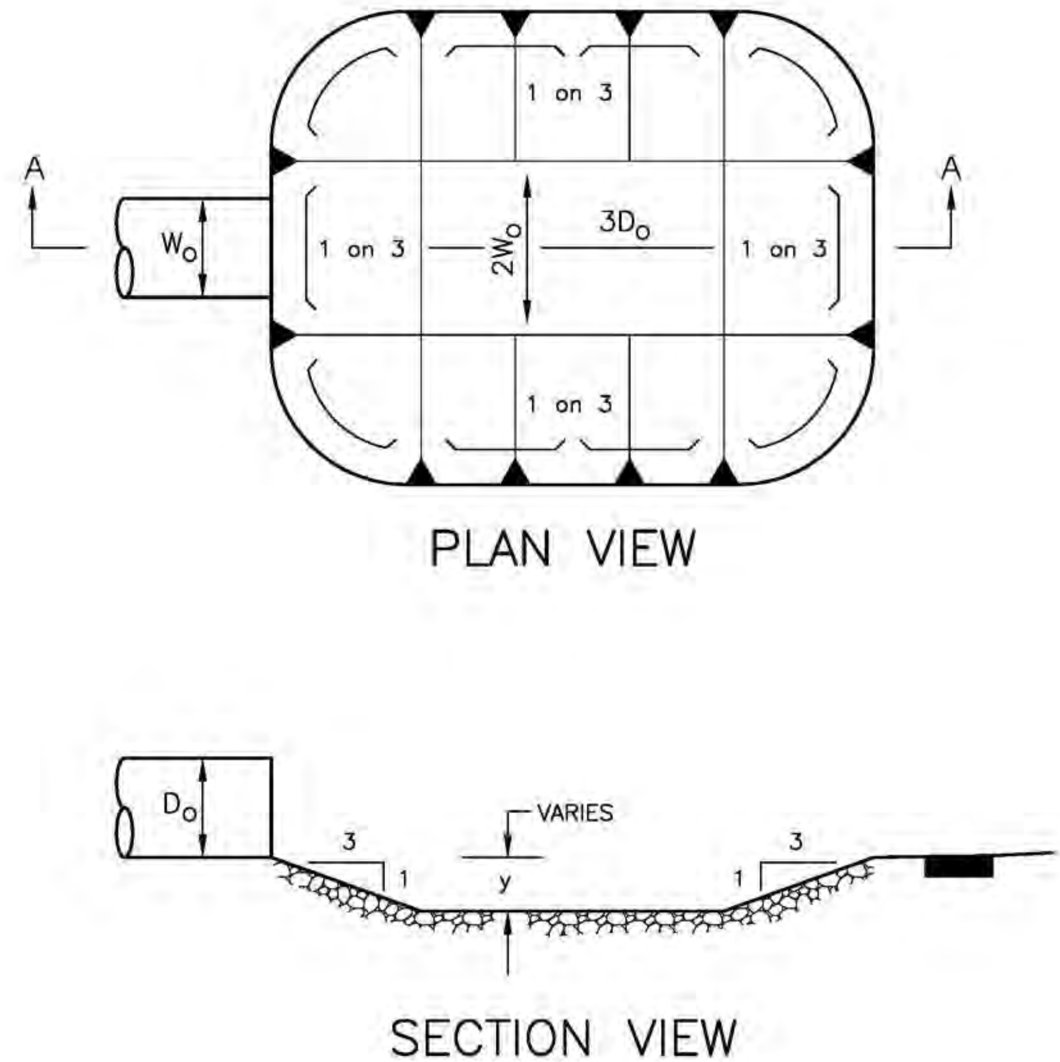
# STORMWATER MANAGEMENT MANUAL

## PREFORMED SCOUR HOLE




| Revision | Date |
|---|---|
| ORIGINAL ISSUE | 3/27/06 |
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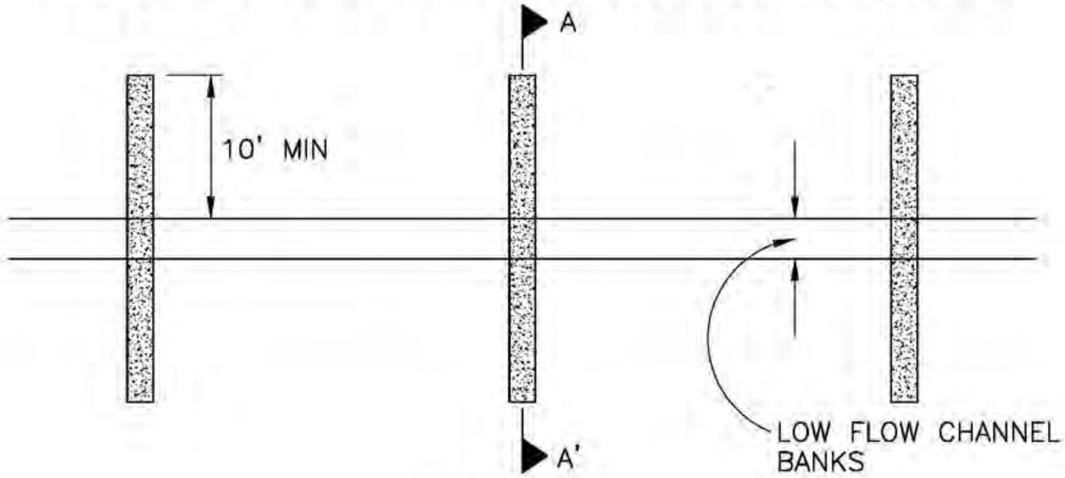
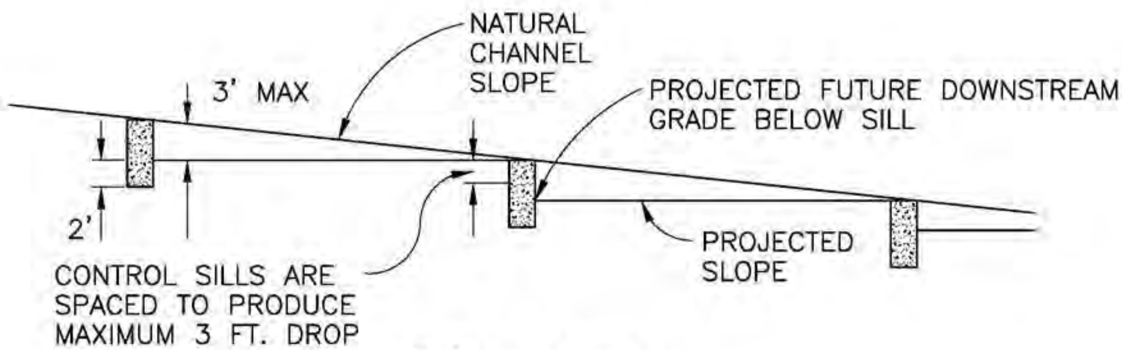
REFERENCE: ASCE, 1975

**FIGURE 817**

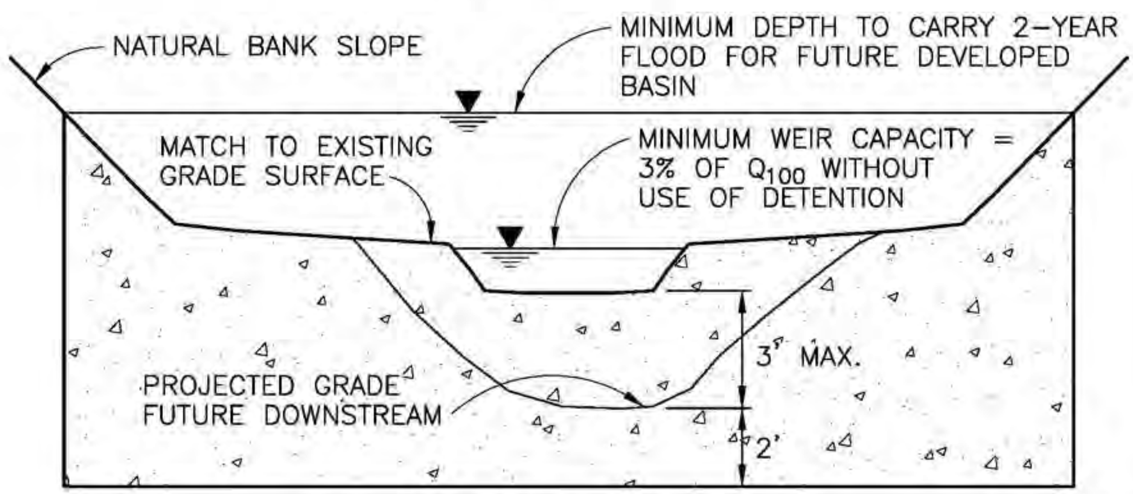
# CONTROL SILL GRADE CONTROL STRUCTURE




(a) PLAN




(b) PROFILE




(c) SECTION A–A'

| Revision | Date |
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| ORIGINAL ISSUE | 3/27/06 |
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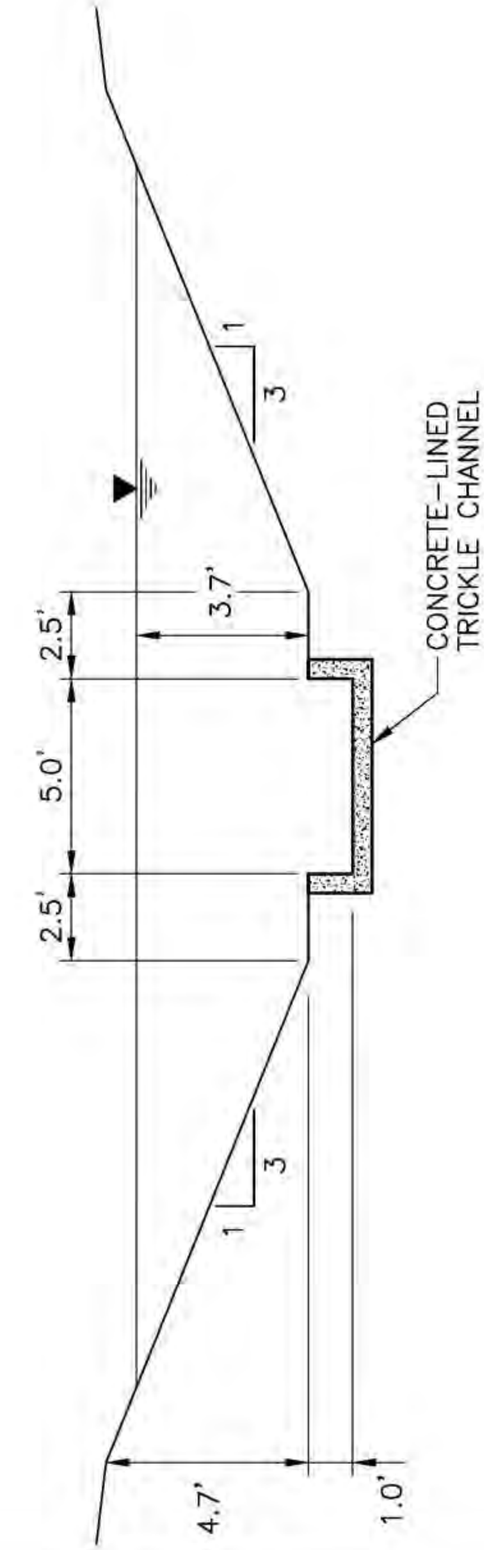
# EXAMPLE: CROSS SECTION OF DOE CREEK

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| ORIGINAL ISSUE | 3/27/06 |
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REFERENCE:

**FIGURE 819**

# SECTION 900

## ADDITIONAL HYDRAULIC STRUCTURES

# SECTION 900
## ADDITIONAL HYDRAULIC STRUCTURES

## TABLE OF CONTENTS

## LIST OF TABLES



## LIST OF FIGURES

# SECTION 900
# ADDITIONAL HYDRAULIC STRUCTURES

## 901 INTRODUCTION

This section presents criteria regulating the design and construction of certain hydraulic structures commonly encountered in Mesa County at storm drain and culvert outlets as well as in open channels that are intended to convey storm runoff. Many of the structures discussed herein vary widely in physical and hydraulic parameters, and are thus presented with general design criteria only. The designer is referred to the applicable references for further detail on the hydraulic theory and design processes for these structures.

Many of the structures discussed in this section are highly visible and easily accessible to the general public. In some cases, these structures may attract onlookers who are unaware of the inherent dangers associated with their operation. Therefore, it is imperative that the design of such structures incorporate posted warnings, pedestrian barriers, fences, and/or other safety apparatuses. It is recommended that the designer consult with Mesa County and/or other local jurisdictions to coordinate the planning and design of the structures in this section.

## 902 CHANNEL DROP STRUCTURES

The design of open channels for conveyance of stormwater runoff is governed by the maximum permissible velocities for a given channel type. These velocities, presented in Section 800 *Open Channels*, are primarily related to the erosive potential of non-clear stormwater flow. In some locations, such as those adjacent to schools or parks, it may be advisable to further reduce design velocities to diminish safety risks to children. Drop structures are used at locations where the use of channel lining materials is undesirable or does not sufficiently reduce design velocity in the channel (see Section 800 for the design of lined channels). A properly designed drop structure will effectively reduce the design slope in a channel segment and dissipate the energy produced by the drop without adverse erosive effects to the channel bed.

Control Sill Grade Control Structures, or Low-Flow Check Structures, are used for velocity and grade control in wide, relatively stable floodplains and wetland areas. These structures are addressed in Section 814.4.3 of this Manual.

The first three structures addressed in this section are typically selected for drops of 5.0 feet or less, but may be used in series (stepped drops). The vertical riprap drop structure (hard drop basin) is limited to 3.0 feet per drop for safety reasons. For larger single drops, the straight drop spillway, the baffled apron, or one of the structures found in Section 903 may be employed.

### 902.1 Grouted Sloping Boulder (GSB) Drop Structure

The GSB drop structure has recently become one of the most commonly installed drop structures in both new construction and channel retrofit situations. Relatively good hydraulic effectiveness and generally pleasing aesthetics likely contribute to this trend. However, the local availability of rock that meets the size and quality requirements for this structure weighs heavily on the economic viability of GSB drops. Much of the design data presented herein is attributed to the *Urban Drainage Criteria Manual* for the Denver-area UDFCD.

The excess energy created by the invert drop is dissipated in two ways in a GSB structure. The additional channel roughness of the grouted boulders themselves is secondary in energy dissipation to the hydraulic jump formed in the drop basin downstream. However, improper design or construction of the grouted boulders, including faulty rock or grout selection and placement, may result in sweepout of the hydraulic jump due to excessive velocity in the drop basin.

The GSB structure is intended for use only in grass-lined channels with upstream velocities within the limits set forth in Section 800 of this Manual. With some variation in design (as outlined in this Section), GSB structures may be used with channels containing or not containing a trickle channel or a low-flow channel.

Plans, profiles, sections, and details of typical GSB structures are found in **Figures 901a, 901b, 901c, 902a, 902b,** and **902c**. Design of these structures involves five components: Rock and grout, upstream channel and approach apron, the drop face, the drop basin, and the exit apron. Seepage concerns are addressed in the design of the approach apron and associated cutoff wall.

Rock and Grout

Grouted boulders must be placed upstream of, and along, the crest of the drop, in the drop face and basin, and along the sill at the end of the basin.

Boulder sizing is based upon the critical velocity, $V_{c1}$, in the channel upstream from the drop structure. If a trickle channel or low-flow channel is present, the maximum critical velocity of that channel and the main channel is used to find the rock-sizing parameter:

$$R_p = \frac{V_{c1} S^{0.17}}{\left(G_{s,rock} - 1\right)^{0.66}} \tag{901}$$

Where:

$R_p$ = Rock - Sizing Parameter

$V_{c1}$ = Maximum Upstream Critical Velocity (fps)

$S$ = Longitudinal Slope (ft/ft)

$G_{s,rock}$ = Specific Gravity of Rock = 2.55 unless otherwise certified by quarry

This parameter is used in **Table 901** to find minimum boulder dimension, $D_r$.

**Table 901 Grouted Sloping Boulder (GSB) Drop Structure Rock Sizing**

| Rock Sizing Parameter, $R_p$ | Minimum Boulder Dimension, $D_r$ |
|---|---|
| Less than 4.50 | 18 inches |
| 4.50 to 4.99 | 24 inches |
| 5.00 to 5.59 | 30 inches |
| 5.60 to 6.39 | 36 inches |
| 6.40 to 6.99 | 42 inches |
| 7.00 to 7.50 | 48 inches |

Adapted from *USDCM* (UDFCD) Table HS-5

Note that standard riprap rock gradation is not utilized in GSB structures. Instead, the boulders are placed in one layer directly on the graded and compacted subgrade (compaction per Section 811.7.6), as close together as is possible, and in such a manner so as not to adversely disturb the subgrade. The flattest surface of each boulder shall be oriented upward and shall be as horizontal as possible. The boulders shall be cleaned with water before grouting to improve grout-rock adherence. The boulders shall be placed by such methods that are less likely to cause breakage or significant blemishes and shall be checked for significant cracking before grouting. Damaged boulders shall be replaced before grouting.

Grout is used to fill the voids between the boulders from the subgrade to one half of the boulder height from the subgrade. In the drop basin only, this is increased to three quarters of the boulder height from the subgrade to promote draining. Excessive grouting leads to a reduction in hydraulic capacity and energy dissipation, and may endanger the structural stability of the drop. Selection, mixing, placement, and finishing of grout shall comply with the specifications set forth in **Table 902**.

Upstream Channel and Approach Apron

Grouted boulders shall be placed on grade with the upstream channel for a minimum of 8.0 feet upstream from the drop crest (this shall be referenced as the "approach apron"). Buried riprap shall be installed from the upstream end of the approach apron to a point at least 8.0 feet upstream along the channel flowline. The riprap shall be $D_{50}$ = 12 inches (UDFCD Type M) and shall be installed per the criteria described in Section 810 of this Manual. The grouted boulder approach apron shall be continuous across the width of the channel (except as described in the following paragraph) and up each bank to the elevation of the normal depth for the design flow at that location. The buried riprap shall be installed across the channel bottom and up each bank to the elevation of one half the normal depth for the design flow at that location.

For grass-lined channels with a concrete or rock-lined trickle channel (see Section 808.3.1 of this Manual), the approach apron and upstream riprap protection are discontinuous across the channel cross-section to allow the trickle channel flowline to continue unimpeded to the drop crest (see **Figure 901A**). While this is necessary to retain the effectiveness of the trickle channel in conveying base or nuisance flows, it tends to create a concentrated jet at the location of the trickle channel during higher flow periods. The additional energy introduced to the basin in these cases may be partially dissipated by the installation of large boulders or baffles in the trickle channel and/or a meandering trickle channel through the drop basin itself. These options are not shown in the GSB details (**Figures 901 and 902**), but are similar to the trickle channel/drop basin controls used for the vertical riprap drop structure (see **Figure 903**).

Grouted rock is particularly susceptible to failure from undermining and the subsequent loss of the supporting bank material. (HEC-11) This refers to the high potential for seepage and piping under and around the drop structure. Since the GSB structure is rigid and essentially monolithic, seepage under the grouted boulders and the resultant transport of subgrade particles will eventually lead to structural failure. Therefore, a seepage cutoff section is required as shown in **Figures 901 and 902**. As noted in the details, the dimensions of the vertical cutoff shall be determined based on geotechnical investigations and seepage analysis or shall comply with the minimum cutoff criteria set forth in the appropriate figures. The seepage cutoff shall be installed prior to the placement of the grouted boulders at the drop crest, and shall include a keyway for the grout/cutoff interface as shown in the details.

Drop Face

The drop face shall consist of grouted boulder "steps" of vertical dimension no greater than one half of the minimum boulder dimension, $D_r$, from **Table 901**. The overall drop face slope must not exceed 4H:1V; flatter slopes are permissible and encouraged due to improved aesthetics and energy dissipation. Slopes steeper than 4H:1V may reduce structural stability.

The grouted boulders are continuous across the entire bottom width on the drop face – the trickle channel flowline equals the main channel flowline in the drop section. The grouted boulders also continue up each bank to the elevation equivalent to the downstream channel normal depth (sequent subcritical depth) plus freeboard or the channel critical depth plus 1.0 feet, whichever is greater.

A weep drain system shall be installed behind the drop face to relieve hydrostatic pressure in drops exceeding 5.0 vertical feet. See details in **Figures 901 and 902**.

Drop Basin

The basin area shall be constructed of continuous grouted boulders of the same dimensions as the drop face section (boulder size, crest and basin width, height of bank protection). However, the grout level is increased to three quarters of the boulder height in the basin, and shall be sloped to drain to the centerline of the channel (or trickle channel if applicable).

The basin is depressed below the downstream channel invert by 2.0 feet for drops of 5.0 feet or less. This helps to stabilize the hydraulic jump. For drops exceeding 5.0 feet, a sequential depth analysis is necessary to determine basin depression depth (a minimum of 2.0 feet applies). Sequential depth analysis is not presented in this Manual; refer to a hydraulics text such as *Open-Channel Hydraulics* (Chow, 1959) for explanation.

Basin length shall be a minimum of 15 feet for non-flexible downstream channel lining (concrete, grouted riprap, geosynthetic linings) and a minimum of 20 feet for downstream channels with flexible linings. A row of 36-inch or larger grouted boulders shall be placed at the downstream end of the basin. The top of this sill shall be equal to the invert of the downstream channel. For channels with a concrete or rock-lined trickle channel, there shall be a break in the end sill of width equal to that of the trickle channel. The trickle channel shall continue downstream through the sill and exit apron with scour protection as specified in Section 800 of this Manual.

Exit Apron and Downstream Channel

The exit apron shall consist of buried riprap of size $D_{50}$ = 12 inches (UDFCD Type M) and shall be installed per the criteria described in Section 810 of this Manual. The riprap shall extend across the channel (except in the trickle channel as applicable) and up the banks to an elevation equal to the top of the adjacent grouted boulders. This riprap protection shall extend downstream from the end sill a minimum distance of twice the drop height or 10 feet, whichever is greater.

### 902.2 Vertical Riprap Drop Structure (Vertical Hard Drop Basin)

This type of drop structure consists of an approach apron (grouted rock), a vertical concrete crest wall, a jump basin with end sill (grouted rock or concrete), and downstream channel scour protection. While an effective method for drop design, these structures shall be avoided if possible in areas of significant public use or in highly visible locations due to safety concerns and low aesthetic appeal. Vertical drop structures shall be avoided in channel reaches which may be utilized for boating or other recreation activities in or adjacent to the water. The maximum allowable drop for a vertical drop structure of this type shall be 3.0 feet.

Rock and Grout

Rock used upstream of the crest wall shall have a minimum dimension of 12 inches in any direction. Rock used downstream of the drop shall have a minimum dimension of 18 inches in any direction. Grouting requirements are identical to those presented in Section 902.1 for the GSB drop structure.

Approach Apron

A grouted rock apron shall be installed across the entire bottom width (including trickle and low-flow channels) and up each bank to the elevation equal to upstream channel normal depth plus 1.0 feet. The rock shall be buried to a depth such that the top of the grout is equal to the invert of the upstream channel at every point across the channel. This approach apron shall extend upstream from the crest wall a minimum of 10 feet.

Vertical Crest Wall

The concrete crest wall conforms to the upstream inverts for the trickle or low-flow channel and the main channel across the bottom width. The wall shall extend a minimum of 5.0 feet into the undisturbed banks. However, all design dimensions including minimum structural width, wall thickness, footer size and geometry, and reinforcement shall be determined using accepted structural analysis methods and determination of potential creep, heave, buoyancy and uplift due to seepage pressures, and all other considerations associated with the design of a retaining wall.

An impervious backfill material is recommended both upstream and downstream adjacent to the crest wall and footers to act as a horizontal seepage cutoff. In lieu of this material, other appliances may be employed to ensure minimized seepage around/under the crest wall. Piping, the transport of structural supporting material away from its intended location, is a common cause of structure instability and failure.

Basin

The basin is a depressed, hard-surface area which redirects the plunging flow from the crest horizontally. At lower flows, the energy dissipated by this redirection may be sufficient to return the flow to a subcritical state. However, the primary energy dissipation method for this structure is a hydraulic jump formed in the basin. When the upstream channel is composite (utilizing a trickle or low-flow channel), the approach velocity tends to be higher in the smaller sub-channel zone than in the main channel zone. Therefore, for the design flow, the basin length and downstream protection requirements may differ for the two zones. By placing large boulders (60% to 80% of critical depth in height) between the location of nappe impingement on the basin floor and a point at least 10 feet from the end sill, the required basin length for the sub-channel zone may be reduced to that of the main channel zone. Otherwise, the following calculations must be applied to both zones independently.

The drop will be treated hydraulically as a straight-drop spillway and analyzed per Chow's (1959) method:

A "drop number", $D_N$, must first be calculated in order to relate other associated lengths and depths:

$$D_N = \frac{q^2}{gh^3} \quad (902)$$

Where:

$q$ = Discharge per Unit Width for the Subject Zone (cfs/ft)

$g$ = Gravitational Constant = 32.2 $ft^2/s$

$h$ = Effective Height of Drop (ft)

Note that the effective drop height must include the basin depression depth. Using the drop number, the following relationships can be solved:

$$\frac{L_d}{h} = 4.30 \cdot D_N^{0.27} \quad (903)$$

$$\frac{y_p}{h} = 1.00 \cdot D_N^{0.22} \quad (904)$$

$$\frac{y_1}{h} = 0.54 \cdot D_N^{0.425} \quad (905)$$

$$\frac{y_2}{h} = 1.66 \cdot D_N^{0.27} \quad (906)$$

Where:

$L_d$ = Drop Length (ft)

$y_p$ = Pool Depth Under Nappe (ft)

$y_1$ = Depth Upstream of the Hydraulic Jump (ft)

$y_2$ = Subcritical Sequent Depth (ft)

See **Figure 903** for illustration of these variables. These values assume that atmospheric pressure is maintained under the nappe, thus the designer is responsible for incorporation of aeration devices as necessary. Drop length, $L_d$, refers to the horizontal distance from the crest wall to the location of depth $y_1$, upstream of the hydraulic jump.

The basin design length, for the subject zone, is given by Equation 907:

$$L_b = L_d + D_j + 0.60 \cdot L_j \quad (907)$$

Where:

$L_b$ = Basin Design Length (ft)

$D_j$ = Distance from Location of Depth $y_1$ to Jump (ft)

$L_j$ = Length of Jump $\cong 6 \cdot y_2$

The distance from the point of nappe impingement on the basin floor to the upstream end of the hydraulic jump is determined by a water surface profile analysis as presented in most hydraulic design texts.

Basin depression depth below the downstream channel invert is determined by comparing the subcritical sequent depth, $y_2$, with the tailwater depth in the downstream channel, $y_{TW}$. If $y_2$ exceeds $y_{TW}$, the jump will be swept downstream and possibly out of the basin. This situation is to be avoided since significant erosion may take place if the jump occurs in an unarmored location in the channel. If $y_{TW}$ exceeds $y_2$, the jump is pushed upstream toward the wall, potentially submerging jump. Hydraulically, this is not problematic, but the structural design of the crest wall may be affected by the additional forces. Basin depression effectively adds to the tailwater depth in the downstream channel, controlling the location of jump formation. Therefore, the minimum basin depression depth, B, is the maximum of the following:

$$B = y_2 - y_{TW}$$
$$\text{or} \qquad (908)$$
$$B = 1.5 \text{ ft}$$

This is the height of the end sill and downstream invert above the downstream end of the depressed basin. The end sill shall be constructed of reinforced concrete or grouted boulders of a minimum 36-inch dimension. This acts as a protected transition back to the channel invert.

Downstream Channel Protection

The channel directly downstream from the end sill shall be protected for a minimum of 10 feet in the direction of flow with buried riprap of size $D_{50}$ = 12 inches (UDFCD Type M) or grouted rock with a minimum dimension of 12 inches.

In cases where the sub-channel zone basin length is longer than the main channel zone (no additional boulders or baffles placed in the basin to dissipate the center jet), the additional protection shall extend a lateral distance equal to the bottom width of the trickle channel from each edge of the trickle channel. This results in an extended protection zone with a width equal to three times the trickle channel bottom width.

### 902.3 Straight Drop Spillway

The straight drop spillway is very similar hydraulically to the vertical hard drop basin presented in Section 902.2. The primary difference exists in the shaping of the spillway downstream from the crest to closely resemble the shape of the lower nappe, i.e., the bottom of the jet formed by the flow suddenly departing the crest. This results in a "classic" spillway shape, as used for major reservoir spillways and channel drops alike. The straight drop spillway itself is not a significant energy dissipation structure, and must be paired with an induced hydraulic jump basin as presented in Section 903.2.

The shape of a straight-drop spillway is dependent on the shape of the nappe, which varies with head over the crest and the shape of the approach to the spillway. The reader is referred to *Open-Channel Hydraulics* (Chow, 1959), *Hydraulic Design of Spillways* (USACOE, 1992), or other texts for design of these structures.

Figure 904 shows a typical straight-drop spillway configuration.

### 902.4 Baffled Aprons (USBR Type IX, Baffle Chute Drop)

The fixed costs associated with the construction of a baffled apron structure (hereafter referred to as a baffle chute drop) typically limit their use to larger drops from an economic standpoint, although the actual minimum size is limited to that length required to incorporate the minimum number of baffle rows. These drop structures are most effective at unit discharge rates between 35 and 60 cfs per foot. However, a value in this range can often be attained by altering the width of the chute. Most often, transition walls are employed to direct wider upstream channel flow to a narrower chute, decreasing the cost of the drop structure. When designed and built correctly, these structures are effective and last for many years with minimal maintenance requirements.

While the baffle chute drop can pass most sediment and debris, larger debris may become caught behind the baffles or in the narrowed chute, disabling the structure's ability to dissipate energy. This can lead to an effectively higher invert in the chute and overtopping, and can also allow the nearly unimpeded flow in the chute to exit the structure at erosive velocities. Therefore, debris-control structures are recommended upstream of the drop, and regular inspection and maintenance may be necessary.

The baffle chute drop structure does not rely on the formation of a hydraulic jump as its primary energy dissipation process. Instead, excess energy in the chute flow is dissipated by redirection over and around baffle blocks, which are arranged in offset rows to avoid the passing of high-velocity jets between the blocks. Since a hydraulic jump is not part of the design, there are no tailwater requirements for this structure. However, potential scour due to relatively high velocities at the end of the chute and in the downstream transition section necessitate a protected exit apron and/or scour hole.

Figure 905 presents an isometric view of a baffle chute drop with typical dimensional requirements. Note that this figure does not indicate structural requirements such as concrete thickness or reinforcement, footer depths and dimensions, or seepage control. These factors shall be assessed and approved by qualified professionals.

<u>Upstream Channel Transition</u>

Typically, the design width of the baffle chute drop is less than the upstream channel width for economic and sizing reasons as well as to attain unit discharge rates in the desired range. The headwalls and/or wingwalls associated with this transition are subject to design constraints set forth in this Manual and shall be designed using proper structural analysis techniques. The designer should note that the effective width of a conduit or channel is often considerably smaller than the physical width due to the separation of flow from the abutment/conduit interface.

The approach section downstream from the transition is designed to maintain an approach velocity of less than the critical velocity at the crest. Recommended approach velocities are presented in **Figure 905**. The concrete flow alignment apron, reaching from the abutment/conduit interface to the chute crest, shall be a minimum of 5.0 feet in length and shall be equal to the chute in width along its entire length. In certain cases, the transition section may not sufficiently reduce the specific energy of the flow to achieve the proper approach (alignment) apron velocity. In these situations, the crest may be raised by up to 12 inches above the approach apron invert.

If a trickle channel is present in the upstream channel, it shall continue through the transition section and apron, and shall maintain a continuous flowline through any raised crest.

Transition and apron wall heights are determined by backwater analysis at peak flow, with a freeboard equal to or greater than that of the upstream channel.

Baffled Chute

The chute floor, walls, and baffles shall be constructed of reinforced concrete and shall be structurally designed to withstand all geotechnical, hydrostatic, and hydrodynamic (impact and frictional) forces imposed by the specific site conditions, including a reasonably conservative factor of safety for all loading. The chute floor shall have a slope no steeper than 2H:1V (Z:1, $Z_{MAX} = 2$). The chute walls shall be vertical and shall be tied to the floor, upstream wall or abutments, and downstream abutments with properly sized and installed steel reinforcement.

The baffle blocks shall be reinforced concrete of the dimensions shown in **Figure 905**. Baffle blocks shall be adequately reinforced and tied with steel reinforcement to the chute floor. A key-in interface is recommended to stabilize the blocks on the chute floor. The block height normal to the chute floor is defined in Equation 909:

$$H = 0.8y_c \qquad (909)$$

Where:

$H$ = Block Height Normal to Chute Floor

$y_c$ = Critical Depth at Peak Flow

There shall be at least four rows of baffle blocks. Baffle block rows shall be spaced at $Z \cdot H$ along the direction of flow, and shall be staggered such that jets of water not directly impinging on a baffle block within a two-row distance are minimized. The blocks and the spaces between the blocks shall be equal to 1.5H except where the width is limited by the chute wall. All baffle rows shall be symmetrical along the centerline of the chute. When a trickle channel exists, the top row of baffles shall be aligned such that the maximum percentage of the trickle channel flow width is not impingent upon any baffles in the first row.

Chute walls shall be at least 3H in height normal to the chute floor. Other dimensional requirements may be found in **Figure 905**.

Where a hard-surface exit apron is not employed, at least 1.5 rows of baffles shall be buried in riprap. This allows for the exposure of additional baffle blocks as loose rock is displaced to form a scour hole or to adapt to a lower downstream channel invert.

Downstream transition walls (headwalls and/or wingwalls) shall be of a height equal to the design normal depth in the downstream channel plus 1.0 feet of freeboard. They shall extend from the chute walls at an angle of 45 to 90 degrees for a distance necessary to contain any eddies that may form in this area.

Basin / Exit Apron

There exist two primary design options for the basin downstream from the baffle chute. The first, a hard-surface basin, is used if the invert of the downstream channel is expected to remain approximately constant over the life of the drop structure. This basin is constructed of either reinforced concrete tied to the downstream end of the chute floor or grouted rock, the latter of which further dissipates energy in the flow and protects the downstream channel from excessive degradation.

Even more energy dissipation is often achieved with the installation of a preformed or non-preformed scour hole at the chute exit. The former is a riprap-lined depressed basin that approximately imitates the dimensions of the scour hole that would form if loose rock was placed as backfill. The riprap and basin sizing requirements are found in Section 814.3 and **Figure 817**. The designer may substitute the downstream design flow normal depth for $D_o$ in the relevant equations and figures. $W_o$ is equal to the width of the chute for the purpose of this design.

A non-preformed scour hole is constructed by backfilling up to the existing downstream channel invert with loose rock. The loose rock shall be at least 2.0 feet deep and shall extend a minimum of 4H feet horizontally parallel to the chute. The rock backfill area shall be of such a width to reach the ends of the downstream abutment walls. Rock size is based on the riprap selection criteria set forth in Section 800 of this Manual. Placement of the rock must not damage the buried baffle blocks. With sufficient operation time, the force of the flow from the baffle chute will displace the loose rock in such a way so as to form a stable scour hole.

The scour hole options, especially the latter, tend to adapt somewhat automatically to changing conditions in the downstream channel, including a gradually lowered invert elevation. However, it is still recommended that a protective channel lining be installed in the downstream channel for an appropriate distance to allow flow to return to a nearly steady state.

## 903 ENERGY DISSIPATION STRUCTURES

The structures described in this section are similar in many ways to the channel drop structures of Section 902. However, while the drop structures' primary purpose is to allow a channel to quickly change elevation without excessively increasing the specific energy of the flow in the downstream channel, these structures are designed to dissipate excess energy already present in the upstream channel. These energy dissipation structures are often employed at transitions from nonflexible channels or conduits to channels with flexible linings or other velocity restrictions. This includes culvert and storm drain outlets to open channels. They are also occasionally used at locations where the energy produced by a channel drop exceeds the limitations of the channel drop structures. As mentioned in Section 902.3, straight-drop spillways must be paired with one of the structures in this section to dissipate the energy associated with the high-velocity flows.

The structures in this section are divided into three categories:

1. Increased Roughness Basins
2. Induced Hydraulic Jump Basins
3. Impact Basins

### 903.1 Increased Roughness Basins

Increased roughness basins are designated for use in locations where the upstream Froude number does not exceed 3.0. Further restrictions apply to each type, including maximum velocities and maximum cross-sectional flow areas. These basins include the riprap basin (preformed scour hole) and the array of drop structures introduced in Section 902.

The FHWA's *Hydraulic Design of Energy Dissipators for Culverts and Channels* (HEC-14) also presents methods for the design of increased resistance devices for pipes, box culverts, and channels. These devices are intended to create a tumbling flow pattern along steep reaches of conduits and channels, thereby maintaining an allowable average velocity. However, due to the economic advantages of other options and to the relatively flat terrain in developed portions of Mesa County, these structures are not included in this Manual.

903.1.1 Riprap Basin

The riprap basin / preformed scour hole is effective for the dissipation of excess energy from upstream conduits and channels complying with the following:

1. Maximum allowable upstream flow area must be equal to or less than the equivalent full-flow area of a 36-inch pipe.
2. Maximum upstream flow velocity must be equal to or less than 15 feet per second at any flow depth.

The design procedure for this structure is presented in Section 814.3 and **Figure 817** of this Manual.

903.1.2 Drop Structure as an Energy Dissipator

While the first two drop structures listed in Section 902 are intended to dissipate only energy produced by the drop itself, they can in certain cases be used as dissipators of upstream energy. The most significant restriction is that flow in the upstream channel must be in a subcritical state before reaching the structure. The combination of a small channel drop and local energy dissipation can effectively reduce the velocity in the downstream channel.

## 903.2 Induced Hydraulic Jump Basins

Induced hydraulic jump basins are commonly used for large and small projects alike. They are highly effective at utilizing the hydraulic jump phenomenon to dissipate excess energy and return the flow to a subcritical depth. While the space required for these structures is relatively large, they are typically less expensive than impact-type basins on a unit-discharge basis. Five distinct induced hydraulic jump basin designs are presented herein. The designer shall incorporate adequate seepage controls as part of the design of all structures. Riprap protection shall be provided for an appropriate distance downstream of all structures in this section where the receiving channel has a flexible lining.

903.2.1 CSU Rigid Boundary Basin

The Colorado State University Rigid Boundary Basin (CSU RBB) utilizes offset rows of baffles (roughness elements) to force supercritical flow from a conduit into a hydraulic jump. The only basin in this category to be designed as entirely on-grade, the CSU RBB is useful for locations with restrictive vertical alignment criteria. However, the upstream Froude number is restricted to a value of 3.0. **Figures 906a** and **906b** present sketches and data for the design of this structure.

The design procedure for the CSU RBB is presented in HEC-14, Chapter VII-A.

903.2.2 USBR Type II Basin

This basin utilizes chute blocks and a dentated sill to induce a hydraulic jump in the basin. An isometric sketch of this basin is presented in **Figure 907**. Unlike the CSU Rigid Boundary Basin, the USBR Type II Basin does not allow for subcritical upstream flow by forcing the flow through critical depth prior to the jump basin. Therefore, this basin requires an upstream Froude number between 4.0 and 14.0. The required tailwater depth for this basin varies with the Froude number per **Figure 908**, in which the solid "design curves" incorporate the required 10% factor of safety. This basin is intended for rectangular sections only, thus transitions may be required upstream of the structure. The design procedure herein is intended for unit discharge rates of up to 500 cfs per foot width. The incoming chute to the basin can be of any slope, but slopes greater than 2:1 shall incorporate a radius curve to allow for a smooth transition to the basin floor. Sequent depths for a free hydraulic jump, USBR Type II Basin, and USBR Type III Basin are shown in **Figure 909**.

The design procedure for the USBR Type II Basin is presented in HEC-14, Chapter VII-D.

903.2.3 USBR Type III Basin

This basin utilizes chute blocks, baffle piers, and a solid end sill (no dentates) to induce a hydraulic jump in the basin. An isometric sketch of this basin is presented in **Figure 910**. This basin requires an upstream Froude number between 4.5 and 17.0. This can in part be controlled by the height and slope of the upstream chute, but the designer shall be aware that lower basin elevations can cause the jump to move upstream and submerge the chute, negating its ability to increase the influent Froude number. The required tailwater depth for this basin is at least full conjugate depth as indicated in **Figure 908**. This basin is intended for rectangular sections only, thus transitions may be required upstream of the structure.

The USBR Type III Basin is limited to a unit discharge rate of 200 cfs per foot width, but can handle velocities up to 50 or 60 feet per second. The design is intended to effectively initiate and shorten the hydraulic jump, thereby reducing the space requirements for the structure. However, the baffle piers, which are essential for controlling the jump, must be carefully designed to comply with the procedure outlined below. The incoming chute to the basin can be of any slope, but slopes greater than 2:1 shall incorporate a radius curve to allow for a smooth transition to the basin floor.

The design procedure for the USBR Type III Basin is presented in HEC-14, Chapter VII-E.

903.2.4 USBR Type IV Basin

At locations where the upstream flow is supercritical but still in the relatively low range of Froude numbers, the USBR Type IV Basin can be employed. Designated for Froude numbers between 2.5 and 4.5, the jump is defined by Chow (1959) as an "oscillating jump." This type of hydraulic jump can produce potentially destructive downstream wave action, so the recommended tailwater depth for this structure is higher than that for the Type III basin.

Like the Type II Basin, this structure utilizes chute blocks and an end sill. However, the end sill in this case is solid, not dentated. This basin is intended for rectangular sections only, thus transitions may be required upstream of the structure. An isometric sketch with general dimensions is presented in **Figure 911**.

The design procedure for the USBR Type IV Basin is presented in HEC-14, Chapter VII-F.

903.2.5 SAF Stilling Basin

The Saint Anthony Falls (SAF) Stilling Basin is similar to the USBR Type III Basin in that it utilizes chute blocks, baffle piers (floor blocks), and an end sill to induce and maintain a steady hydraulic jump in the basin. Also similar to the Type III, it produces a jump that is significantly shorter than a natural hydraulic jump (approximately 80% of the length), thereby reducing the required length of the basin and downstream protection

Plan and profile views of the SAF Basin are provided in **Figure 912**. Note that the basin itself may be laterally flared to better fit the downstream channel. This flare is labeled as z (longitudinal):1 (lateral), wherein the variable z is limited to values equal to or greater than 2.0. However, all side walls, headwalls, and wingwalls shall be vertical.

The SAF Basin may be used at the base of straight-drop spillways, at culvert and storm drain outlets, and in canals. It is required that flow entering the basin be supercritical, but this can usually be achieved by proper upstream chute design. The allowable range of Froude numbers for this structure is 1.7 to 17.0.

The design procedure for the SAF Stilling Basin is presented in HEC-14, Chapter VII-G.

## 903.3 Impact Basins

Impact basins dissipate energy by causing the high-velocity flow to encounter an obstruction, redirecting the flow in directions other than the influent path. This action effectively negates a large percentage of the velocity head that would otherwise potentially cause damage to the downstream channel. While these structures tend to be costly on a unit-discharge basis, they require far less space than many other dissipation options. Three types of impact basins are presented in this section.

The designer of the energy dissipators discussed herein is responsible for ensuring adequate structural design, including the analysis of all forces incident on the structure, calculation of creep and overturning potential, and design and installation of seepage controls. Necessary seepage controls may include cutoff walls, liners, weep drains, and/or other devices. The designer is referenced to applicable texts concerning subgrade compaction, concrete mixing, steel reinforcement, calculation of external forces, and retaining wall design.

903.3.1 Contra Costa Energy Dissipator

This structure is intended for use with small to medium culverts with medium to high velocity flows. It is also designed to operate with minimal tailwater, although some tailwater improves the dissipator's performance. Tailwater depth is limited to one half of the culvert height. The Contra Costa Dissipator is best for locations where the design flow depth at the culvert outlet is less than the culvert height. Therefore, culvert effluent depth is limited to one half of the culvert height. The Froude number of the culvert outlet flow is limited to a maximum of 3.0.

The Contra Costa Energy Dissipator is a concrete structure designed to be placed in a trapezoidal channel with side slopes of 1:1 and a bottom width between one and three times the culvert height $(D \leq W \leq 3D)$. If a natural channel exists at the structure location, the structure width shall conform to that channel, with a maximum width of 3D. The structure consists of two continuous baffles of different heights across the basin floor as well as a vertical end sill. All parts of the structure shall be reinforced concrete and shall be tied to the downstream end of the culvert with steel reinforcement bars if possible. Profile and section views with dimensional definitions are provided in **Figure 913**.

The design procedure for the Contra Costa Energy Dissipator is presented in HEC-14, Chapter VIII-A.

903.3.2 Hook-Type Energy Dissipator

The Hook-Type Dissipator, also called Aero-Type, is used at culvert outlets with Froude numbers in the range of 1.8 to 3.0. Each dissipator utilizes three "hook" structures in the basin that redirect a portion of the high-velocity flow up and back into the basin flow. This action creates a large amount of turbulence, thereby dissipating some of the excess energy in the flow. At Froude numbers exceeding about 3.0, the dissipation effects are greatly diminished.

This energy dissipation structure is designed to use either of two basin configurations. The first type contains vertical wingwalls at the culvert exit which are warped smoothly to side slopes of 1.5:1 at the end sill (see **Figure 914a**). The second configuration is a trapezoidal channel with a constant cross section throughout the basin (see **Figure 915a**). Hook details for the two configurations are found in **Figures 914b** and **915b**, respectively.

The design procedure for the Hook-Type Energy Dissipator is presented in HEC-14, Chapter VIII-B.

903.3.3 Impact-Type Energy Dissipator (USBR Type VI)

Also called the Baffle-Wall Energy Dissipator or Baffled Outlet, this structure is compact and highly effective for the control of high-energy flows exiting a conduit or rectangular channel section. Consisting of a vertical-walled basin with a single large vertical hanging baffle, energy is dissipated by impact with the baffle and secondarily by eddies formed in the basin. At the design flow, this structure dissipates energy more effectively than a hydraulic jump (See **Figure 916b**), and has no minimum tailwater depth. However, its debris-handling capability and maximum tailwater depth limit (discussed later) limit the locations at which the structure can be used. Further limitations include a maximum discharge of 400 cfs per structure and a maximum upstream velocity of 50 feet per second. This latter value is intended to minimize damage to the baffle due to cavitation. Where these limits are exceeded, two or more structures may be built adjacent to one another to accommodate the excess flow.

For upstream conduits with a slope greater than 15% and for all open channels, it is recommended that there be a horizontal section from the outlet brink to a point at least four conduit widths upstream. Rectangular upstream channels shall have sidewalls of a height equal to or greater than the walls of the dissipator basin and shall always have a zero longitudinal slope for a minimum of three channel widths upstream from the entrance to the basin.

**Figure 916a** presents the configuration and necessary dimensions for the design of the USBR Type VI structure. Note that the optional notches near the edges of the basin are included to create concentrated jets for self-cleaning purposes.

One of the most important design features of this structure is its ability to pass the entire design discharge over the top of the baffle. This is important to prevent upstream flooding in the case of complete clogging of the area under the baffle. However, this flow configuration is not nearly as effective and shall not be relied upon as an alternative energy dissipation method. Therefore, the debris and ice buildup potential at a given location shall be analyzed prior to selection of this structure as the energy dissipator for that outlet.

While some tailwater (up to $h_3+h_2/2$) improves the performance of the dissipator, depths over this height shall be avoided. Significant degradation of performance occurs with tailwater depths greater than $h_3+h_2$, thus the USBR Type VI structure shall not be installed in these conditions.

The design procedure for the Impact-Type Energy Dissipator is presented in HEC-14, Chapter VIII-C.

## 904 OVERBANK PREVENTION STRUCTURES AND WASTEWAYS

Every channel has a maximum allowable flow depth which, when exceeded, may cause damage to the banks and eventually failure of the channel. Occasionally, overflow from a storm drainage system enters an irrigation canal (this shall be avoided unless specific consent is granted by the owner/operator of the canal). In these situations, it is typically necessary to remove the overflow from the canal at some downstream location. The structures in this section are intended to remove excess water from a channel to maintain a specified water surface elevation or to allow the water in a channel section to be drained. The latter may be necessary to inspect, maintain, or repair the channel, or in the event of an embankment failure, to redirect some of the escaping flow to an acceptable location.

Wasteway is the term commonly applied to the channel to which the main channel excess flow is diverted. A wasteway shall have the capacity to convey the maximum flow that can be diverted through all diversion structures located upstream, and shall deliver the excess flow to an acceptable disposal point.

Two types of diversion structures which can act as overbank prevention structures are presented herein: the side-channel spillway and the gated turnout.

### 904.1 Side-Channel Spillways

A side-channel spillway is the most effective structure for automatic removal of excess flow in a channel since its capacity increases with depth over its crest. The spillway crest is usually parallel to the channel alignment except at terminal wasteways (at the end of a canal). Typically, the spillway crest is set approximately 0.2 feet above the normal design depth for the channel to allow for normal wave action. The length of the spillway is then controlled by the required overflow discharge capacity and the maximum allowable water surface elevation in the channel. A standard rule of thumb is to ensure no more than 50% encroachment on the freeboard of the channel banks in the vicinity of the spillway. A detailed procedure for the design of a side-channel spillway turnout and wasteway is not presented in this Manual due to the infrequent application of such a structure in stormwater runoff designs. However, Equation 910 is the basic design equation for the side-channel spillway (suppressed rectangular weir):

$$Q = 3.33 L_c H^{3/2} \qquad (910)$$

Where:

$Q$ = Design Flow Over the Spillway (cfs)

$L_c$ = Crest Length (ft)

$H$ = Height of Channel Water Surface over Crest (ft)

### 904.2 Gated Turnouts

To allow for manual release of water from a channel for the purpose of water level control, maintenance access, et cetera, gated turnouts are often installed at wasteways. It is common practice to include at least one gated turnout at any side-channel spillway location for flushing and additional water level control. Again, specific design procedures are not presented, but the general orifice equation is given:

$$Q = CA\sqrt{2gh} \qquad (911)$$

Where:

$Q$ = Design Flow through Gate (cfs)

$C$ = Orifice Coefficient $\cong 0.6$

$h$ = Height of Water Surface over Gate Centerline (ft)

$A$ = Area of orifice ($ft^2$)

$g$ = Gravetational Constant ($32.2\ ft/s^2$)

## 905 PIPE APPURTENANCES

This Section presents appurtenances for use in conjunction with pipe systems, specifically those designed for the transport of stormwater.

### 905.1 Pipe Collars

Pipe collars are transverse fins that extend from the pipe into the surrounding earth and function as barriers to percolating water and burrowing rodents. (USBR 1974) Due to the relative smoothness and impermeability of pipe, percolated water tends to collect and move along the soil adjacent to a pipe's outer wall. This action, typically called piping, tends to transport soil particles away from the pipe, potentially causing the pipe to experience structural problems. Failure of the backfill and ultimately the pipe itself can lead to hydraulic failure of the pipe system as well as the failure of surface structures such as roadways and buildings.

While percolation is expected around many storm drains and culverts, especially near pipe inlets, those with higher (5H:1V or greater) percolation gradients are often candidates for pipe collars. The percolation gradient is the slope of a line from an inlet water surface to a point of relief for the percolated water. The difference in water surface elevations between the upstream end of the percolation path and the point of relief is $\Delta H_{perc}$. Lane's weighted creep method is used to determine a percolation factor (weighted-creep ratio), which is compared to the allowable ratio for the soil type at a given site. First, determine the weighted-creep length:

$$L_{wc} = y_{steep} + (x_{mild}/3) + 2(L_{sc}) \qquad (912)$$

Where:

$L_{wc}$ = Weighted creep length (ft)

$y_{steep}$ = Vertical path distance along the structure (steeper than 45°) (ft)

$x_{mild}$ = Horizontal path distance along the structure (flatter than 45°) (ft)

$L_{sc}$ = Percolation path distance that short-cuts through the soil (ft)

Then determine the percolation factor, $R_{wc}$ to 1:

$$R_{wc} = \frac{L_{wc}}{\Delta H_{perc}} \qquad (913)$$

**Table 903** presents minimum recommended weighted-creep ratios for a range of soil types:

**Table 903 Lane's Minimum Recommended Weighted-Creep Ratios**

| Material | Minimum Ratio |
|---|---|
| Very fine sand or silt | 8.5:1 |
| Fine sand | 7.0:1 |
| Medium sand | 6.0:1 |
| Course sand | 5.0:1 |
| Fine gravel | 4.0:1 |
| Medium gravel | 3.5:1 |
| Course gravel w/cobbles | 3.0:1 |
| Boulders w/some cobbles & gravel | 2.5:1 |
| Soft clay | 3.0:1 |
| Medium clay | 2.0:1 |
| Hard clay | 1.8:1 |
| Very hard clay / Hardpan | 1.6:1 |

Adapted from USBR 1974, Untitled Table, Page 364

Where the weighted-creep ratio calculated in Equation 913 does not exceed the applicable recommended ratio from **Table 903**, or does not exceed 2.5:1, pipe collars shall be installed.

**Figure 917** presents basic dimensions for pipe collar fittings on reinforced concrete pipe (RCP) and corrugated metal pipe (CMP).

### 905.2 Thrust Blocks at Pipe Bends

Every horizontal or vertical pipe bend in a storm drain, culvert, inverted siphon or other pipe structure shall be analyzed for stability. As the momentum of flow changes around a bend, forces are exerted on the bend that must be countered by the pipe walls, soil pressure, pipe joints, and friction. When the dynamic thrust exceeds the allowable force on any of these resistance devices, a thrust block is installed at the bend. A thrust block typically consists of a rough block of concrete poured around the outside of a pipe bend in direct contact with the outer wall of the pipe.

The thrust force on a pipe bend is calculated by vector components (x, y, and z) to simplify the process. In the equations below, "x" represents the horizontal direction of flow upstream of the bend, "y" represents the horizontal direction of flow normal to "x", and "z" represents the vertical direction along which gravity acts. Equations 914 through 916 are adapted from Roberson et al, 1998, using conservation of momentum to find reaction forces. Pipe cross-sectional area and internal pressure are assumed to be constant through the bend, with pressure assumed to equal the surcharge depth above the pipe crown, if applicable.

$$F_{R_x} = \frac{\rho}{g} Q\left(V_{2_x} - V_{1_x}\right) + pA(\cos\theta - 1) \qquad (914)$$

$$F_{R_y} = \frac{\rho}{g} Q\left(V_{2_y} - 0\right) - pA \cdot \sin\theta \tag{915}$$

$$F_{R_z} = \frac{\rho}{g} Q\left(V_{2_z} - 0\right) - W_{bend} - W_{water} \tag{916}$$

Where:

$F_R$ = Reaction force required to hold bend in place, lbf

$\rho$ = Density of water $\cong 62.4\ {}^{lbs}/_{ft^3}$

Q = Flow rate in pipe, cfs

$V_{1_x}$ = Average pipe velocity upstream of the bend, fps

$V_{2_x} = V_{1_x} \cos\theta,\ V_{2_y} = V_{1_x} \sin\theta,\ V_{2z} = V_{1_x} \sin\theta$

p = Internal pipe pressure, psf

g = Gravitatio nalcons tan t, 32.17 ft/s$^2$

A = Cross sectional flow area of pipe, sf

$\theta$ = Total bend angle (vertical OR horizontal )

$W_{bend}$ = Weight of the pipe in the bend, lbs

$W_{water}$ = Weight of the water in the bend, lbs

Subscripts 1 and 2 indicate conditions just upstream and just downstream of the bend.

In addition to soil bearing pressure, force on a bend is resisted by friction between the pipe and the soil. A sliding coefficient of 0.35 is recommended for purposes of calculating the friction force. (USBR 1974)

Where calculations indicate that sliding or displacement of a horizontal bend may occur, a thrust block is installed to increase the effective bearing area on the soil such that the load is adequately dispersed. Vertical bends may require an anchor block to provide additional weight to resist the resultant vertical force. Calculation of resisting forces for a vertical bend may include full pipe weight and anchor block weight, but shall not include the weight of earth cover on the bend. This allows for safe operation of the pipe even with reduction or removal of cover material. (USBR 1974)

## 905.3 Valves

References for this section include USBR, 1974 and Linsley and Franzini, 1964 (see Section 906).

905.3.1 Drain (Blow-Off) Valves

A blow-off valve is intended to allow for the draining of a structure that typically will otherwise not fully drain. Most commonly used in long inverted siphons, blow-off valves may be gravity-fed, pumped, or a combination of both, depending on the invert of the discharge pipe. The design and installation of blow-off valves and related pipes shall incorporate pressure-rated joints and provisions for operation and maintenance access.

905.3.2 Pressure Relief Valves

Pressure relief valves are used to exhaust excess air pressure from a pipeline to protect the pipe from bursting and to remove large volumes of entrapped air that may significantly impact the hydraulic capacity of the pipe. The valves are set to open at a predetermined pressure so as to allow for a sealed pipeline under normal operating pressures.

These valves are commonly utilized in smaller pressure pipelines such as water supply lines to limit the effects of hydraulic transients (water hammers), but are occasionally used in stormwater systems. Inverted siphons (Section 1302.1) often require a venting system to prevent blowback of air entrained in the water, although an open air vent (no valve) is usually an acceptable solution given an exhaust point that is well above the hydraulic grade line.

An air venting system of some type is required at all locations where the crown of a pipe is higher than the crown elevations upstream and downstream from that point.

905.3.3 Air Inlet Valves

Air inlet valves operate in a similar fashion to pressure relief valves, but instead allow air into a pipeline to avoid internal pressure to drop too far below atmospheric pressure. As water drains from a sealed pipeline, a partial vacuum is created that can collapse or severely damage the pipe. Air inlet valves operate either by a float (water level) control or by opening at a set pressure difference like a pressure relief valve.

High points in a pipeline shall always be designed with an air venting system to avoid extreme positive or negative internal pressures as compared to atmospheric pressure.

# GROUTING CRITERIA

## GROUT NOTES

### Material Specifications

1. All grout shall have a minimum 28-day compressive strength equal to 3200 psi.
2. One cubic yard of grout shall have a minimum of six (6) sacks of Type II Portland cement.
3. A maximum of 25% Type F Fly Ash may be substituted for the Portland cement.
4. For Type A grout, the aggregate shall be comprised of 70% natural sand (fines) and 30% $^{3}/_{8}$ -inch rock (coarse).
5. For Type B grout, the aggregate shall be comprised of $^{3}/_{4}$ -inch maximum gravel, structural concrete aggregate.
6. Type B grout shall be used in streams with significant perennial flows.
7. The grout slump shall be 4-inches to 6-inches.
8. Air entrainment shall be 5.5%-7.5%.
9. To control shrinkage and cracking, 1.5 pounds of Fibermesh, or equivalent, shall be used per cubic yard of grout.
10. Color additive in required amounts shall be used when so specified by contract.

### Placement Specifications

1. All Type A grout shall be delivered by means of a low pressure (less than 10 psi) grout pump using a 2-inch diameter nozzle.
2. All Type B grout shall be delivered by means of a low pressure (less than 10 psi) concrete pump using a 3-inch diameter nozzle
3. Full depth penetration of the grout into the boulder voids shall be achieved by injecting grout starting with the nozzle near the bottom and raising it as grout fills, while vibrating grout into place using a pencil vibrator.
4. After grout placement, exposed boulder faces shall be cleaned with a wet broom.
5. All grout between boulders shall be treated with a broom finish.
6. All finished grout surfaces shall be sprayed with a clear liquid membrane curing compound as specified in ASTM C-309.
7. Special procedures shall be required for grout placement when the air temperatures are less than 40°F or greater than 90°F. Contractor shall obtain prior approval from the design engineer of the procedures to be used for protecting the grout.
8. Clean Boulders by brushing and washing before grouting.

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| ORIGINAL ISSUE | 3/27/06 |
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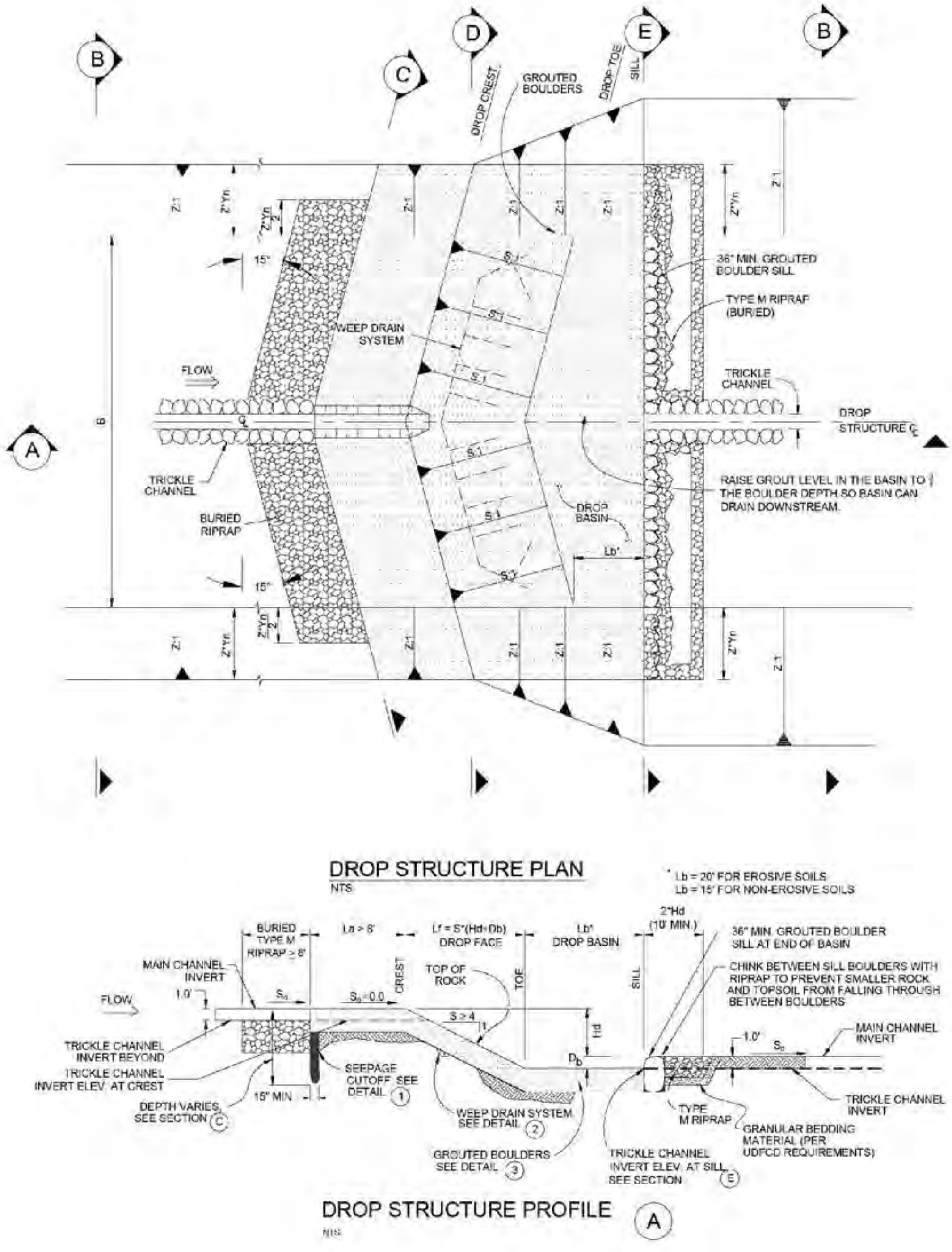
# GSB DROP W/ TRICKLE CHANNEL

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| ORIGINAL ISSUE | 3/27/06 |
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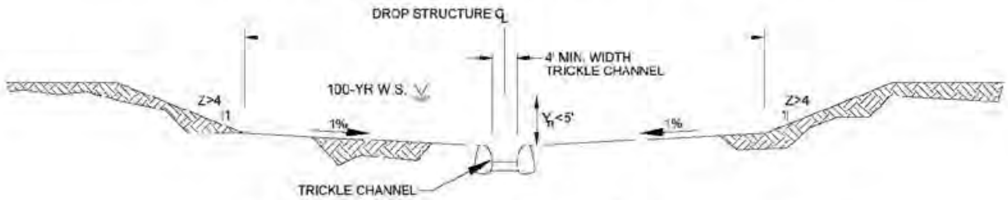
WRC ENGINEERING, INC.

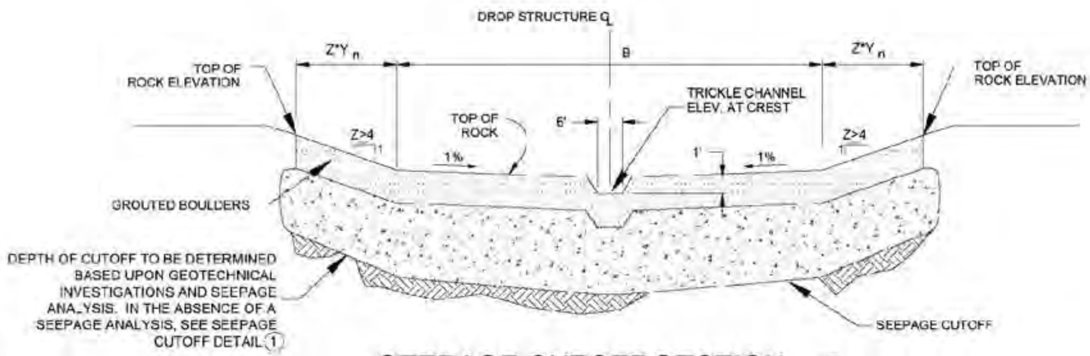
REFERENCE: USDCM VOL. 2 (UDFCD, 2001)

**FIGURE 901a**

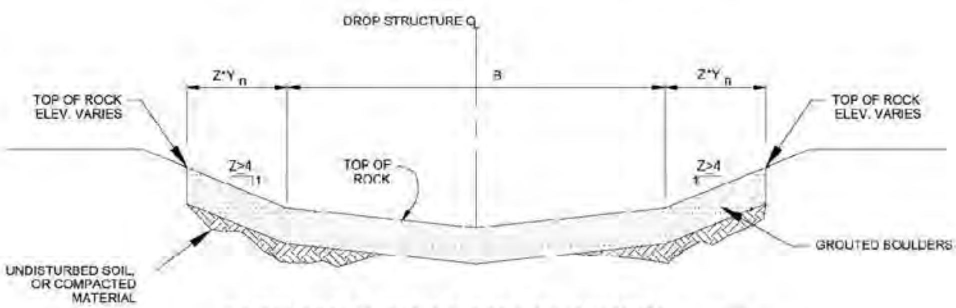
# GSB DROP W/ TRICKLE CHANNEL



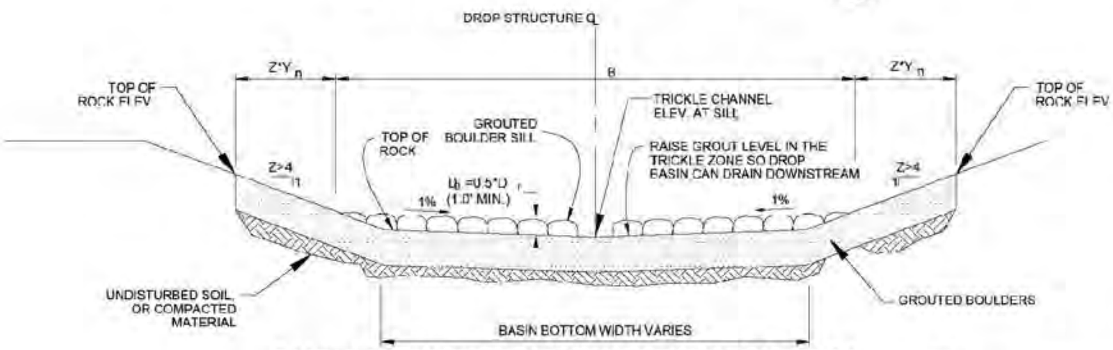

TYPICAL CHANNEL SECTION (B)
(UPSTREAM AND DOWNSTREAM OF DROP)
NTS




SEEPAGE CUTOFF SECTION (C)
NTS




TYPICAL DROP FACE SECTION (D)
NTS




TYPICAL DROP BASIN SECTION AND SILL (E)
NTS

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|---|---|
| ORIGINAL ISSUE | 3/27/06 |
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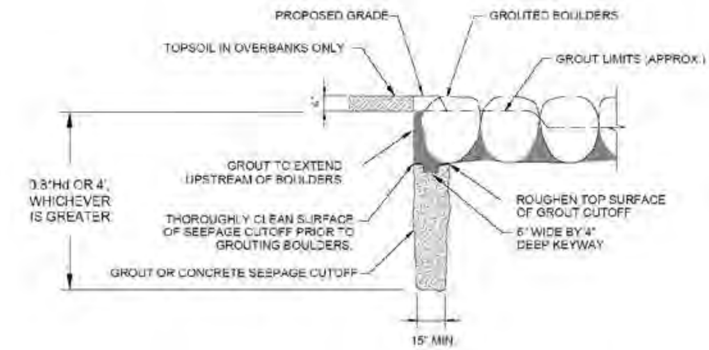
REFERENCE:
USDCM VOL. 2 (UDFCD, 2001)

**FIGURE 901b**

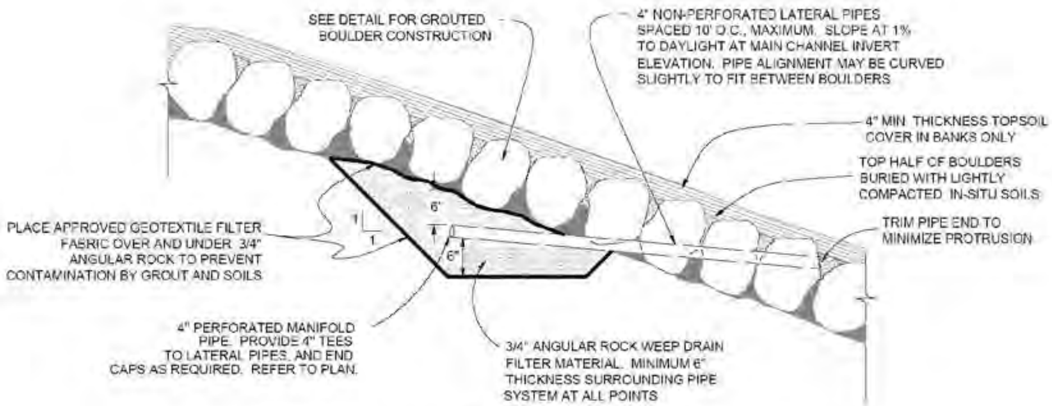
# STORMWATER MANAGEMENT MANUAL

## GSB DROP DETAILS



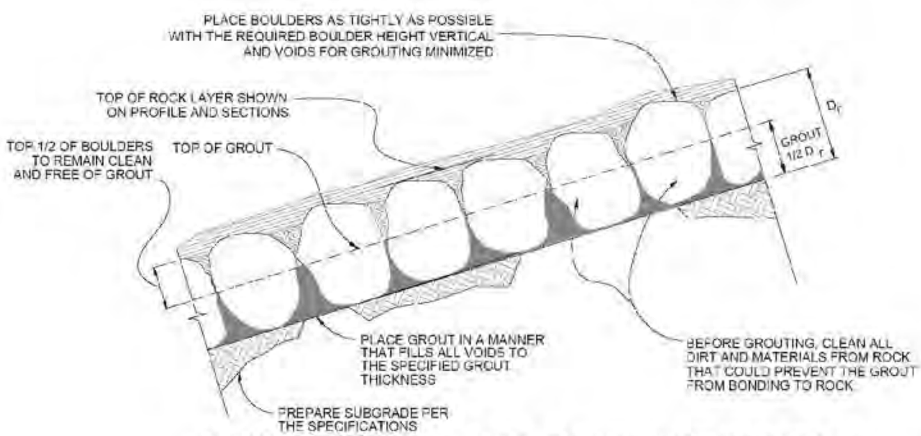

SEEPAGE CUTOFF DETAIL (1)

NTS




WEEP DRAIN SYSTEM DETAIL (2)

NTS USE ONLY IN DROPS HIGHER THAN 5 FEET




GROUTED BOULDER PLACEMENT DETAIL (3)

NTS

| Revision | Date |
|---|---|
| ORIGINAL ISSUE | 3/27/06 |
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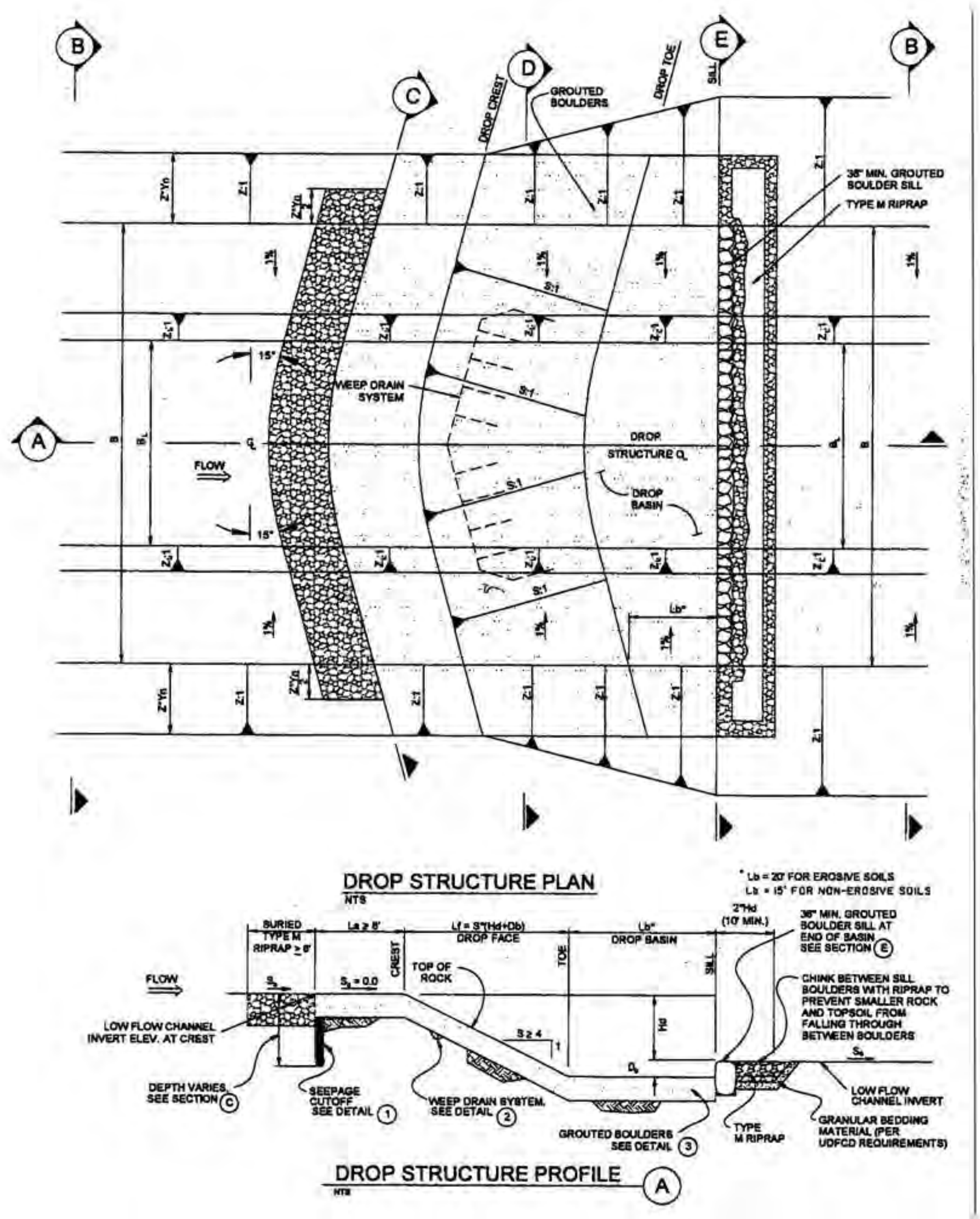
REFERENCE: USDCM VOL. 2 (UDFCD, 2001)

**FIGURE 901c**

# GSB DROP W/ LOW-FLOW CHANNEL

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|---|---|
| ORIGINAL ISSUE | 3/27/06 |
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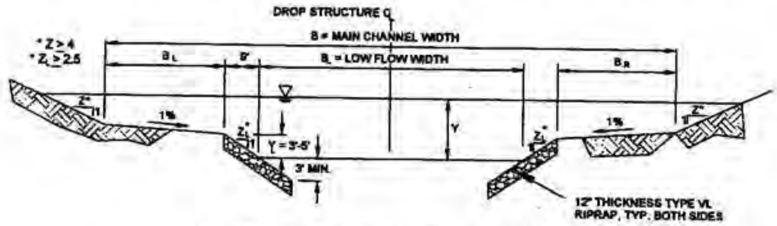
WRC ENGINEERING, INC

REFERENCE:

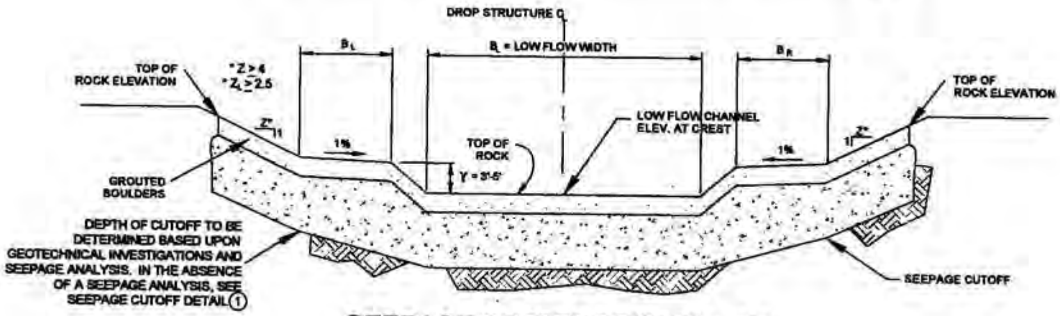
USDCM VOL. 2 (UDFCD, 2001)

**FIGURE 902a**

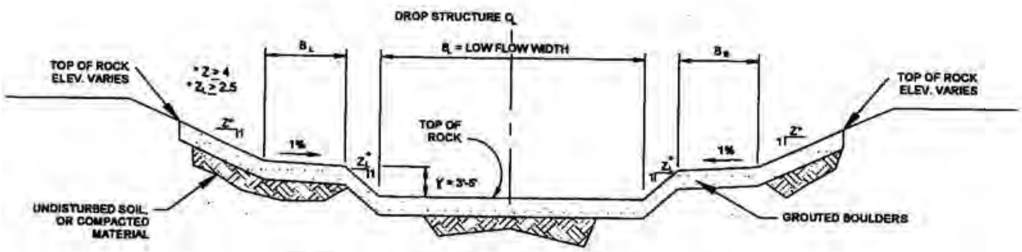
# GSB DROP W/ LOW-FLOW CHANNEL



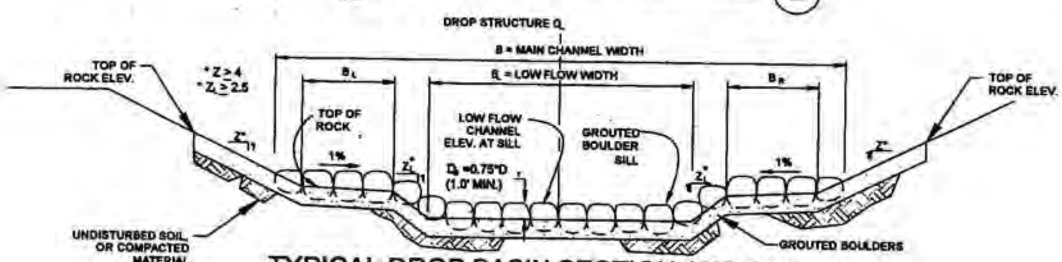

TYPICAL CHANNEL SECTION
(UPSTREAM AND DOWNSTREAM OF DROP
NTS
B




SEEPAGE CUTOFF SECTION
NTS
C




TYPICAL DROP FACE SECTION
NTS
D




TYPICAL DROP BASIN SECTION AND SILL
NTS
E

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|---|---|
| ORIGINAL ISSUE | 3/27/06 |
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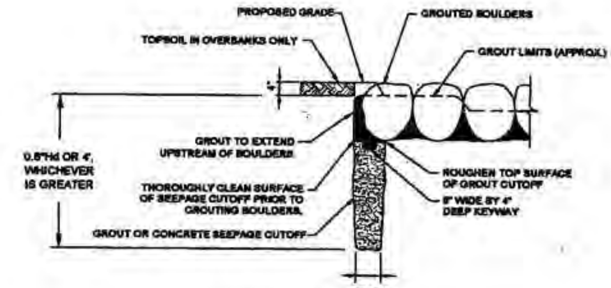
REFERENCE:

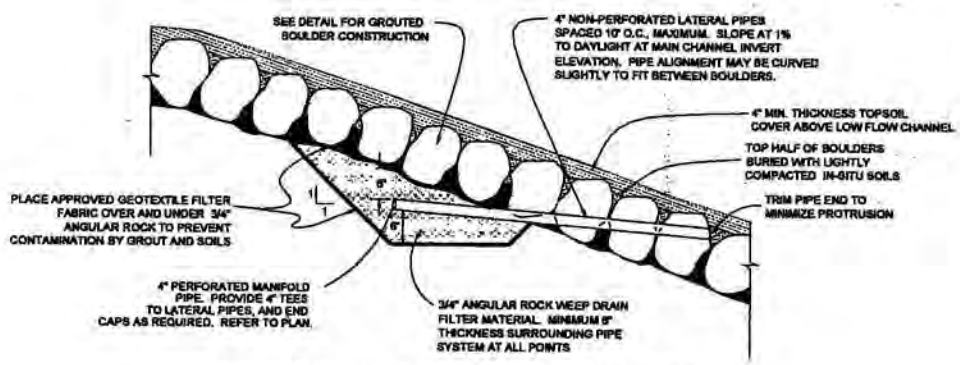
USDCM VOL. 2 (UDFCD, 2001)

**FIGURE 902b**



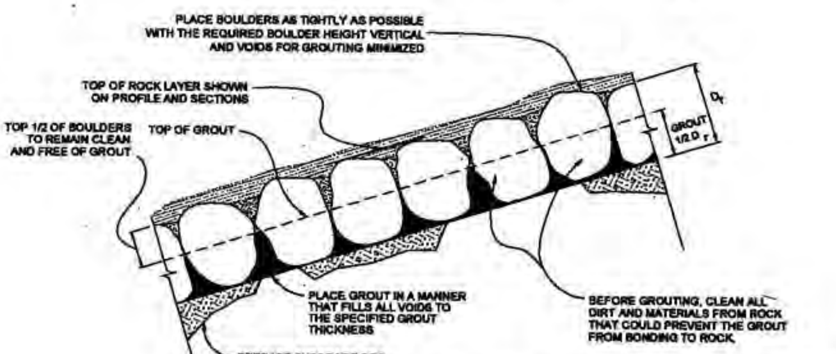

SEEPAGE CUTOFF DETAIL (1)

NTS




WEEP DRAIN SYSTEM DETAIL (2)

NTS USE ONLY IN DROPS HIGHER THAN 5 FEET




GROUTED BOULDER PLACEMENT DETAIL (3)

NTS

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| ORIGINAL ISSUE | 3/27/06 |
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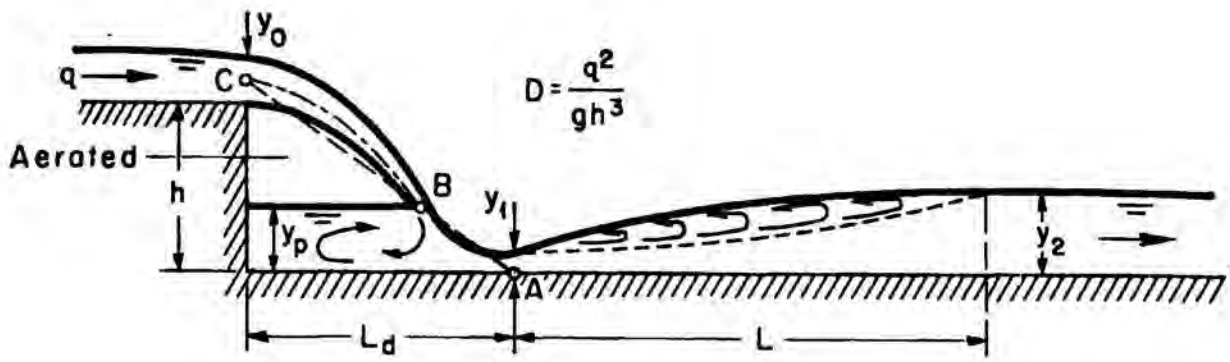
REFERENCE:

**FIGURE 902c**

# VERTICAL RIPRAP DROP STRUCTURE (HARD DROP BASIN) (DIMENSIONAL DEFINITIONS)

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|---|---|
| ORIGINAL ISSUE | 3/27/06 |
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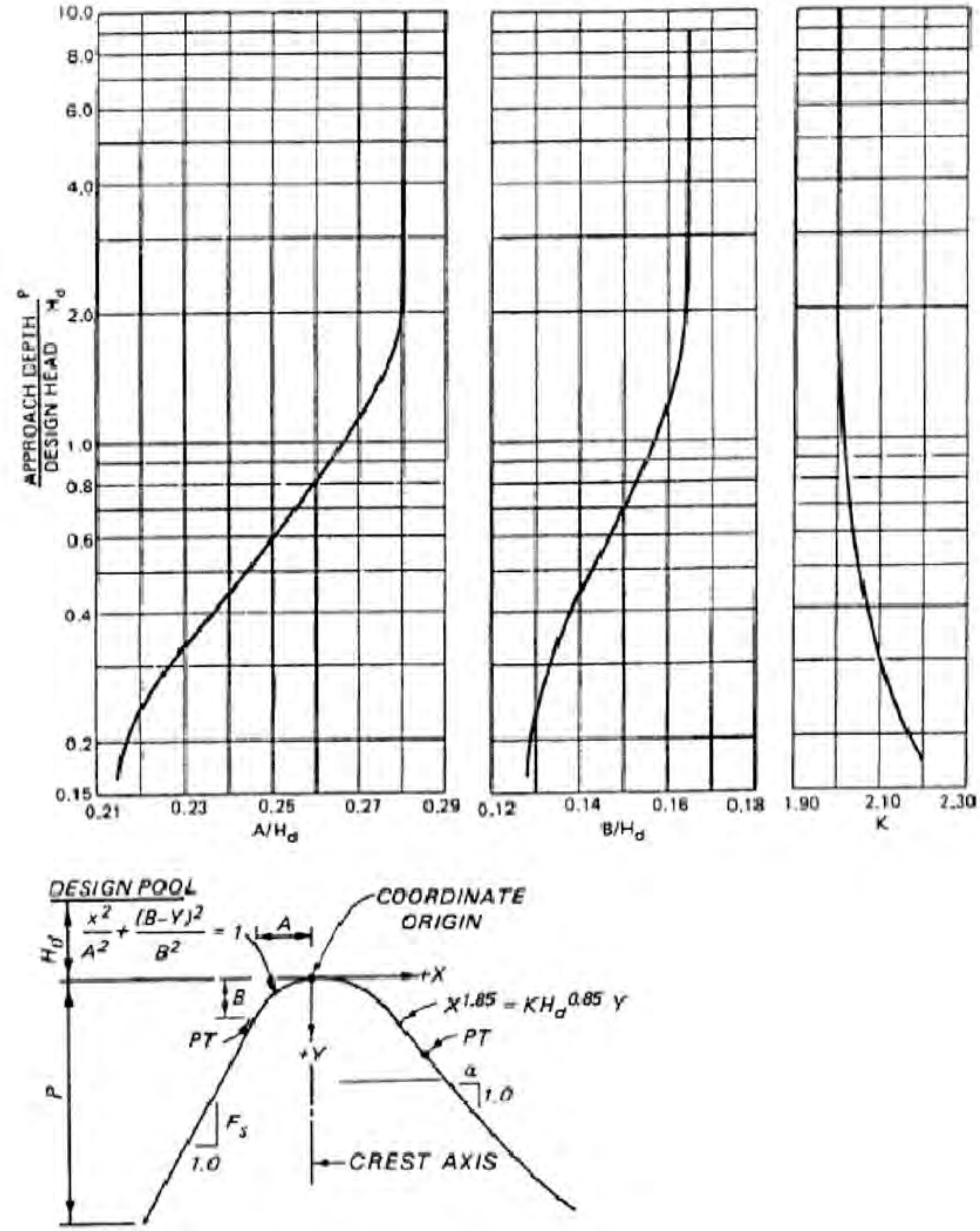
REFERENCE: OPEN-CHANNEL HYDRAULICS (CHOW, 1959)

FIGURE 903

# STRAIGHT - DROP SPILLWAY




DEFINITION SKETCH

ELLIPTICAL CREST SPILLWAY
COORDINATES
COORDINATE COEFFICIENTS
FROM HDC 111-20

| Revision | Date |
|---|---|
| ORIGINAL ISSUE | 3/27/06 |
| | |
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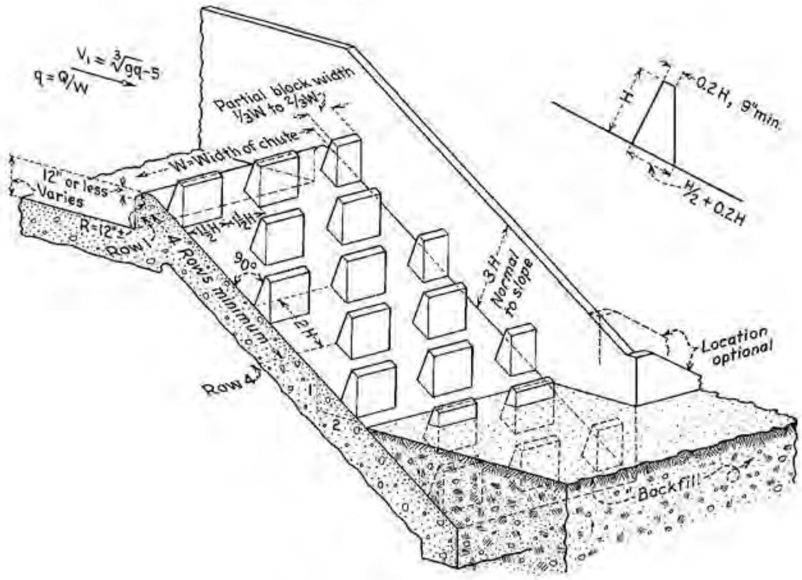
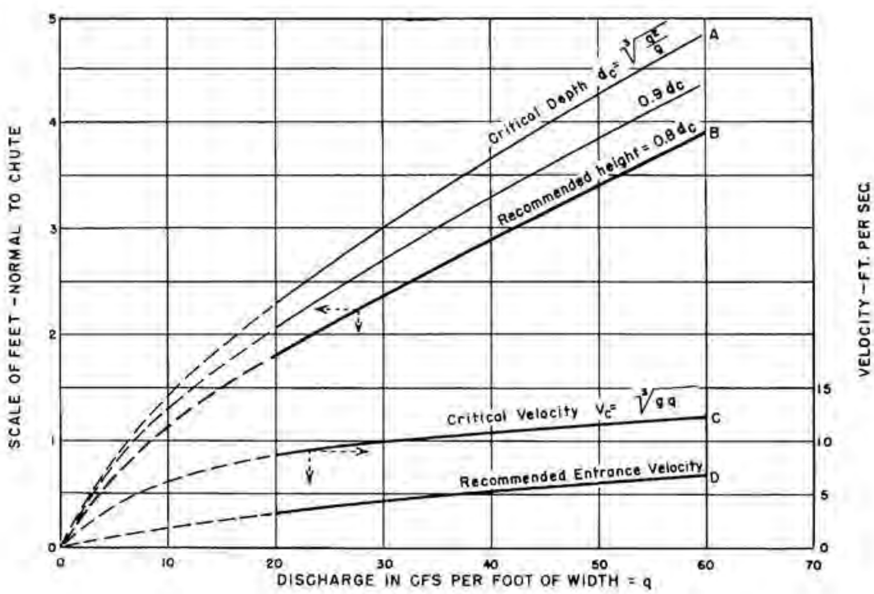
# BAFFLED APRON




(A) USBR ISOMETRIC




(B) DESIGN CRITERIA

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| ORIGINAL ISSUE | 3/27/06 |
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WRC ENGINEERING, INC.

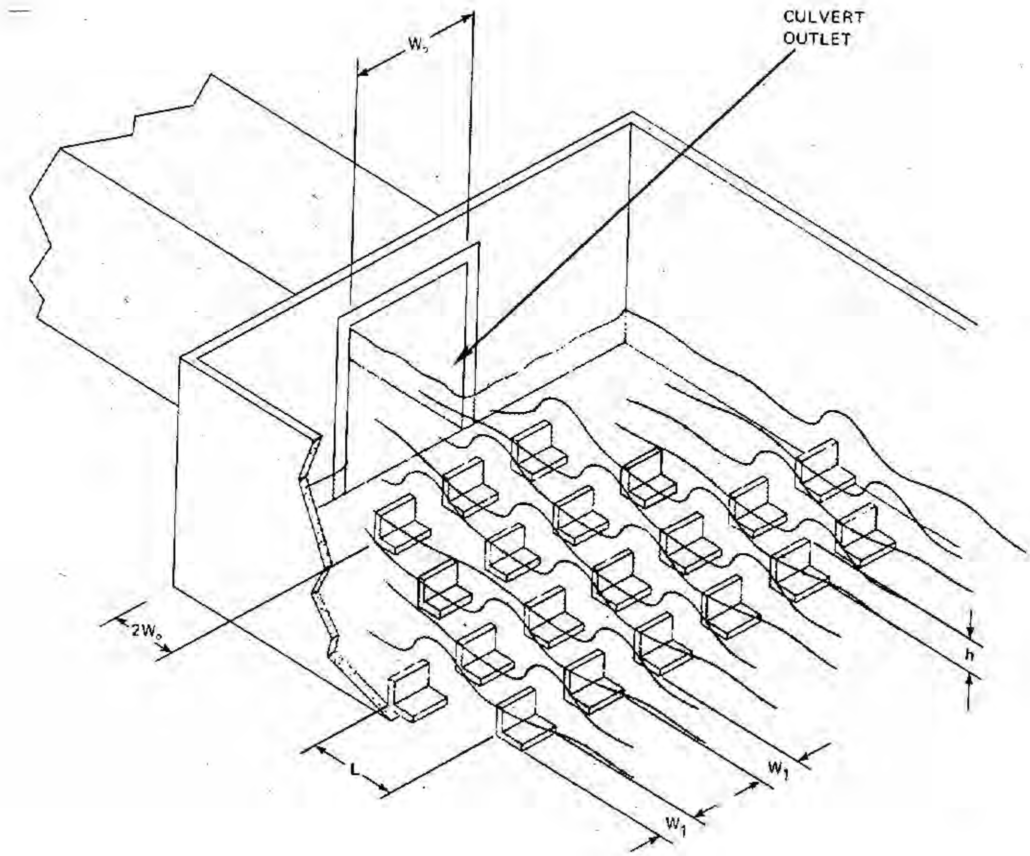
REFERENCE:

USDCM VOL 2 (UDFCD/USBR, 2001)

**FIGURE 905**

# STORMWATER MANAGEMENT MANUAL

## CSU RIGID BOUNDARY BASIN
## (ISOMETRIC)

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| ORIGINAL ISSUE | 3/27/06 |
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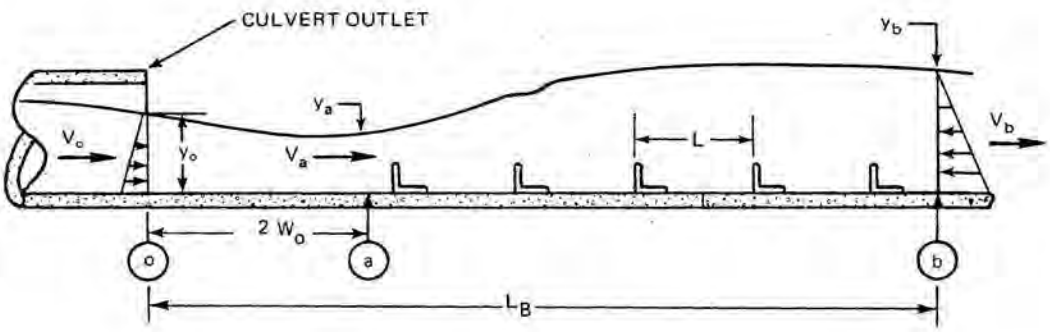
WRC ENGINEERING, INC.

REFERENCE: HYDRAULIC DESIGN OF ENERGY DISSIPATORS FOR CULVERTS AND CHANNELS (HEC-14) (1983)

**FIGURE 906a**

# STORMWATER MANAGEMENT MANUAL

## CSU RIGID BOUNDARY BASIN (PROFILE)

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| ORIGINAL ISSUE | 3/27/06 |
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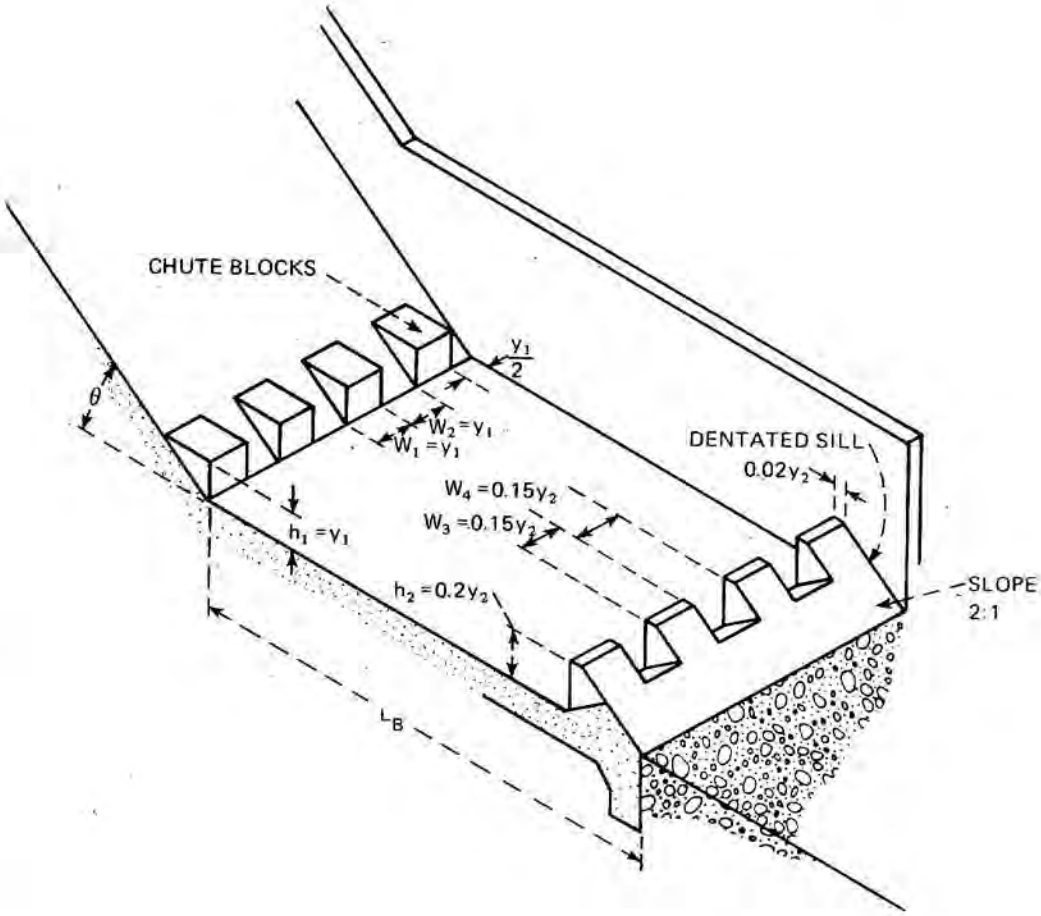
WRC ENGINEERING, INC.

REFERENCE: HYDRAULIC DESIGN OF ENERGY DISSIPATORS FOR CULVERTS AND CHANNELS (HEC-14) (1983)

**FIGURE 906b**

# USBR TYPE II BASIN

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| ORIGINAL ISSUE | 3/27/06 |
| | |
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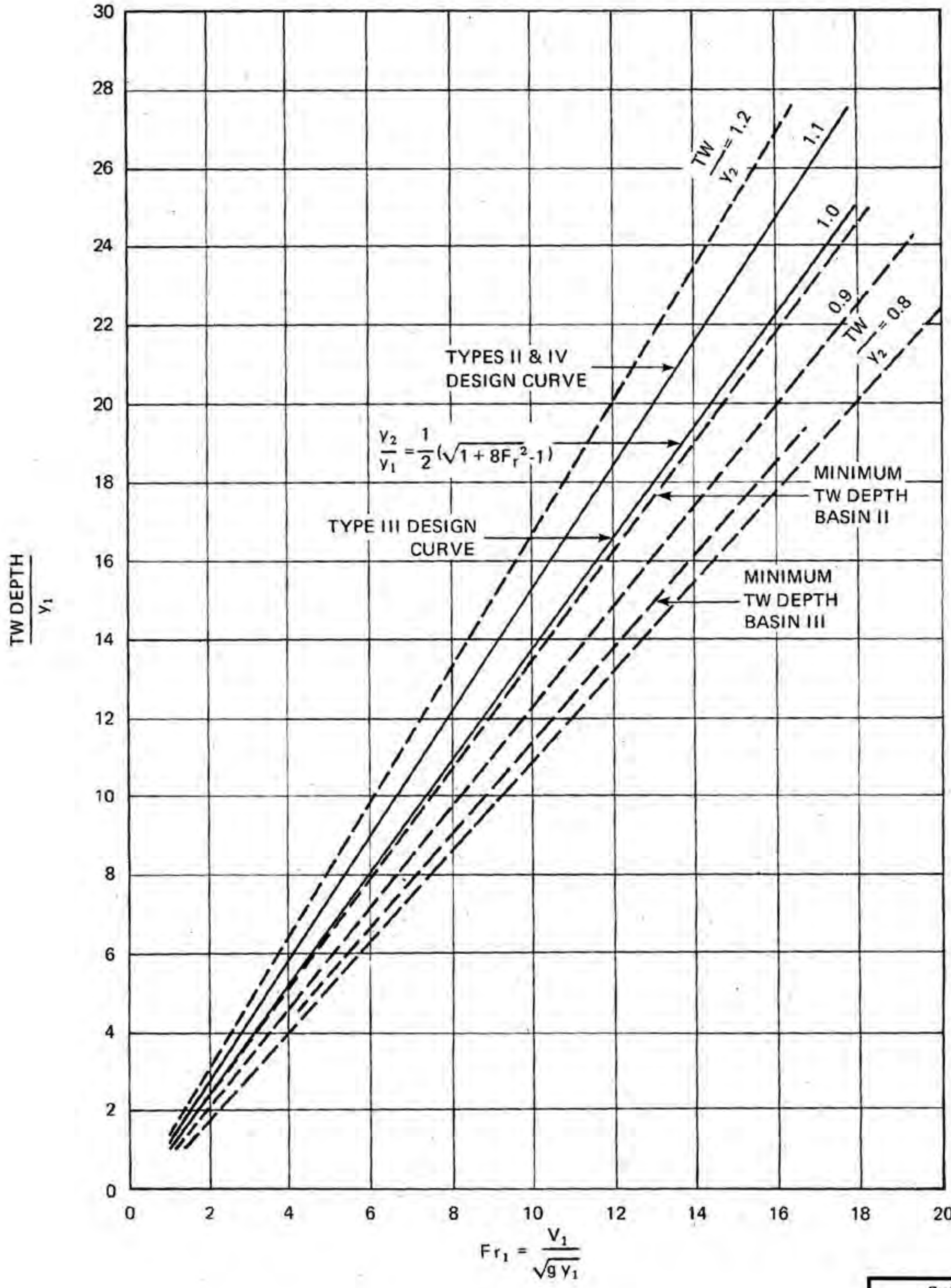
WRC ENGINEERING INC.

REFERENCE: HYDRAULIC DESIGN OF ENERGY DISSIPATORS FOR CULVERTS AND CHANNELS (HEC-14) (1983)

**FIGURE 907**

# REQUIRED TAILWATER DEPTHS (USBR TYPES II, III & IV)

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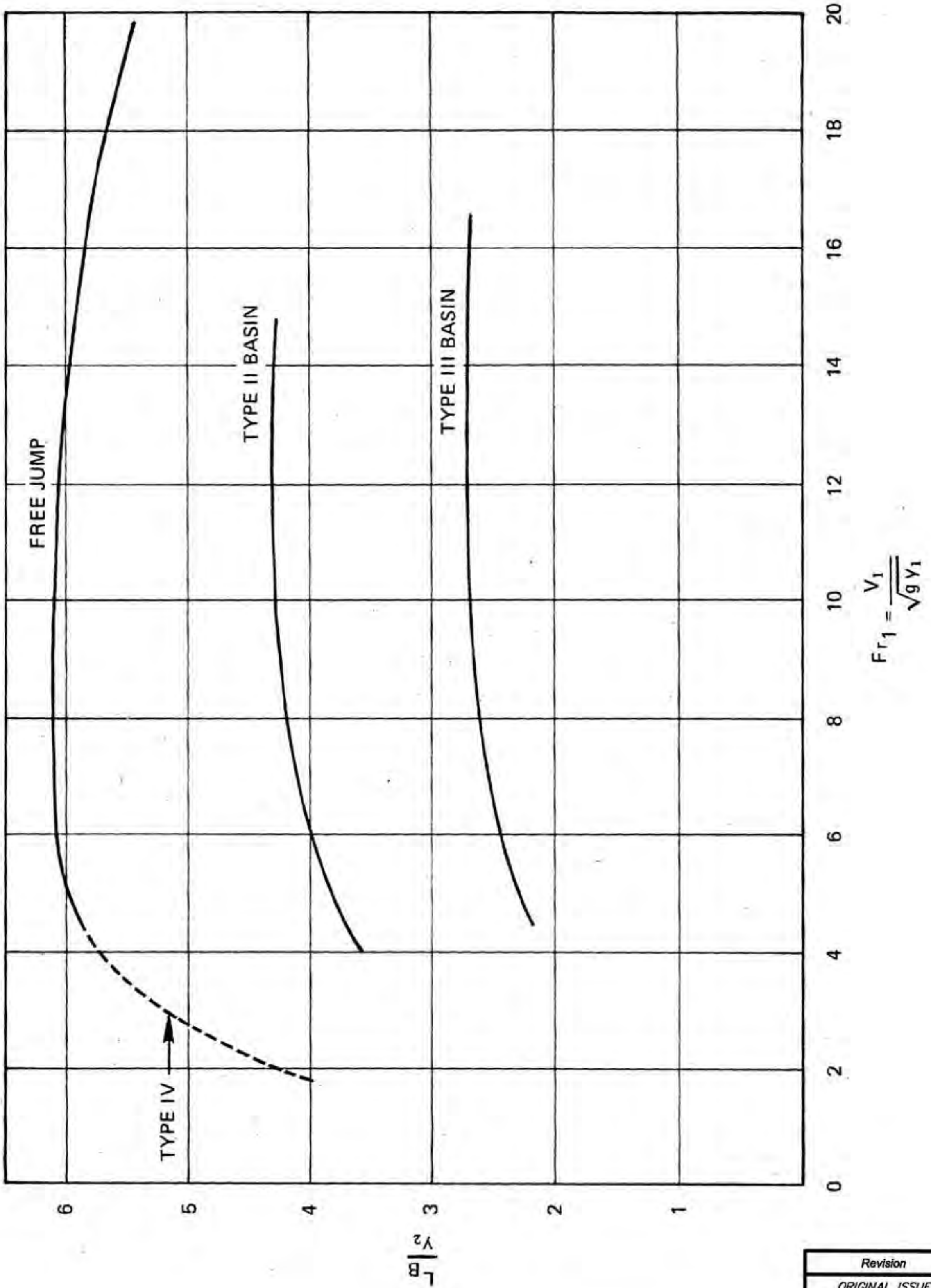
WRC ENGINEERING, INC.

REFERENCE: HYDRAULIC DESIGN OF ENERGY DISSIPATORS FOR CULVERTS AND CHANNELS (HEC-14) (1983)

**FIGURE 908**

## HYDRAULIC JUMP LENGTH

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| ORIGINAL ISSUE | 3/27/06 |
| | |
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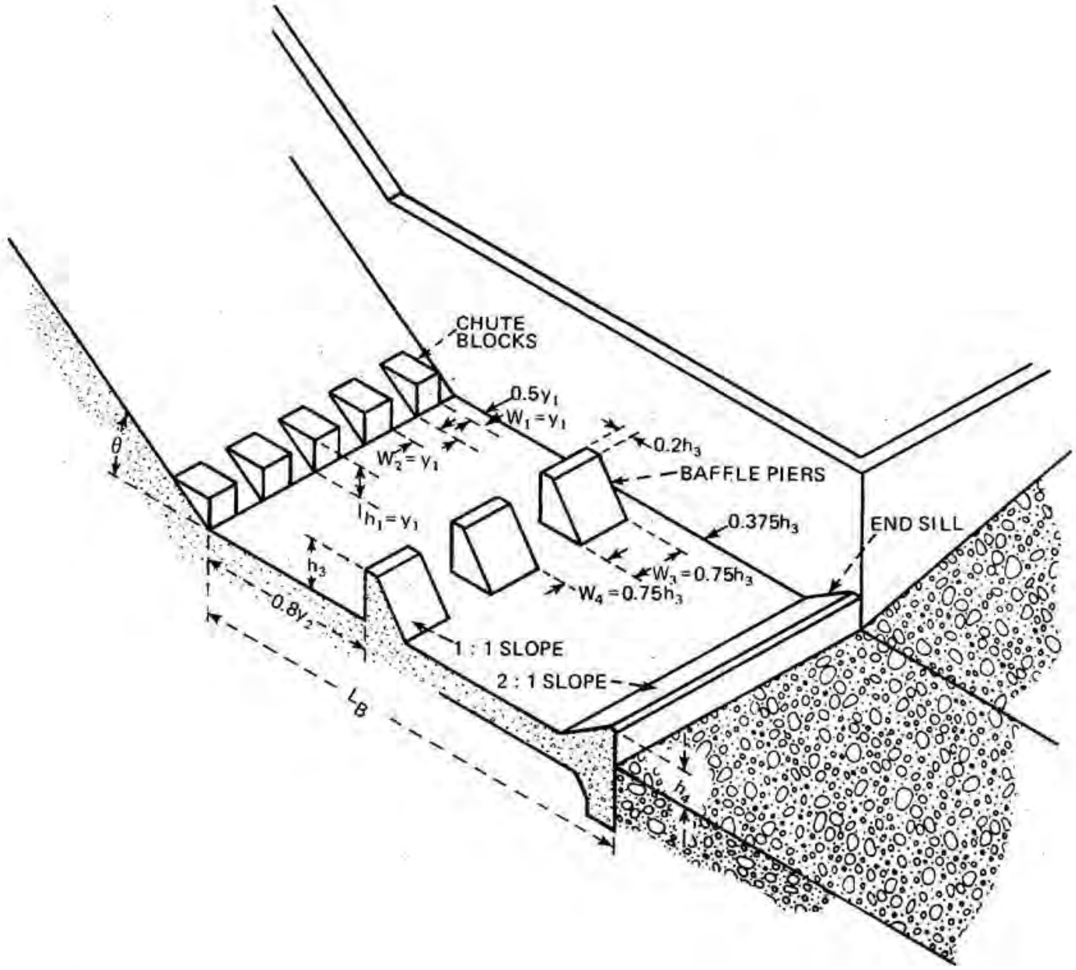
WRC ENGINEERING INC

REFERENCE: HYDRAULIC DESIGN OF ENERGY DISSIPATORS FOR CULVERTS AND CHANNELS (HEC-14) (1983)

**FIGURE 909**

# USBR TYPE III BASIN

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|---|---|
| ORIGINAL ISSUE | 3/27/06 |
| | |
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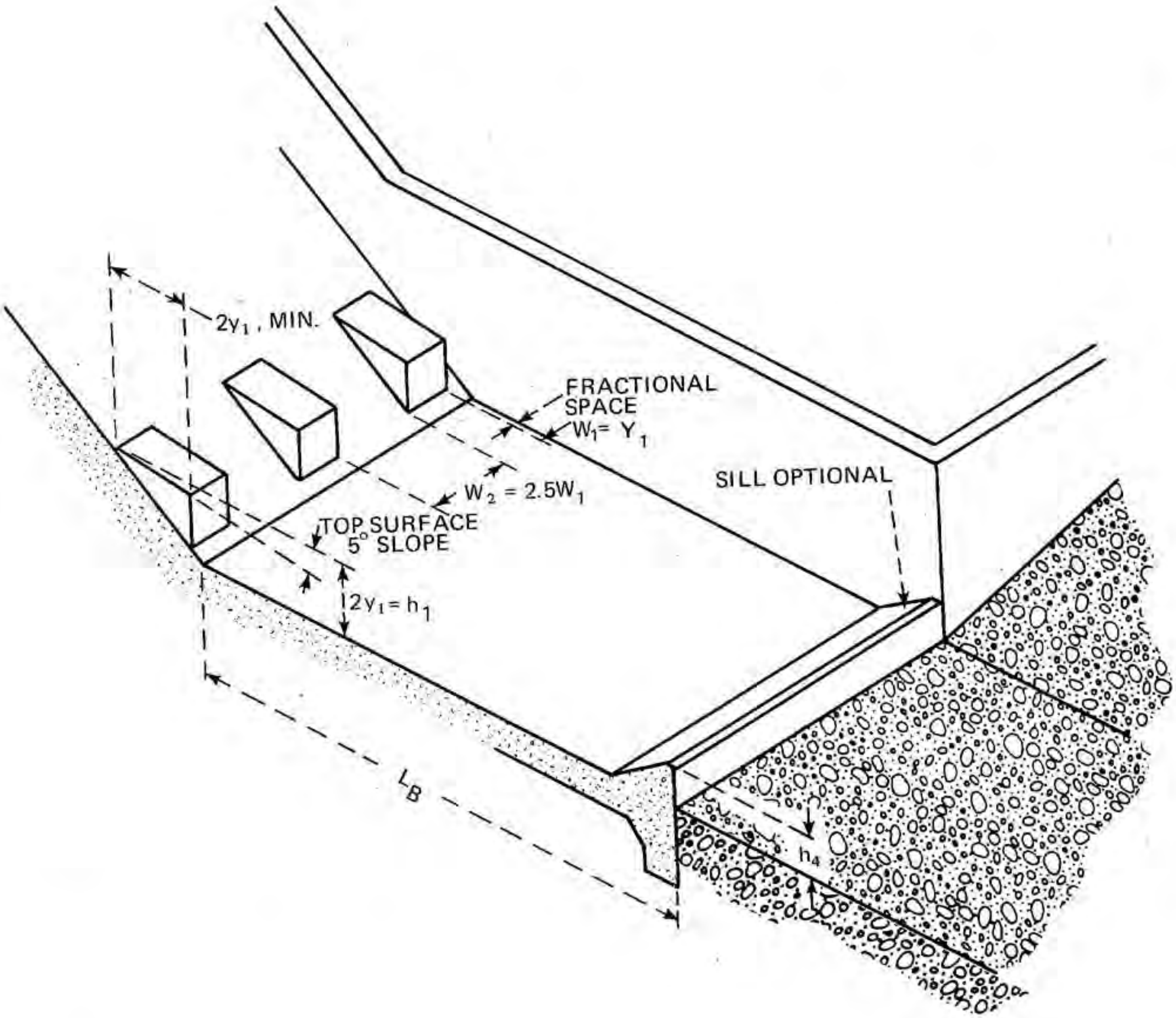
# USBR TYPE IV BASIN

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| ORIGINAL ISSUE | 3/27/06 |
| | |
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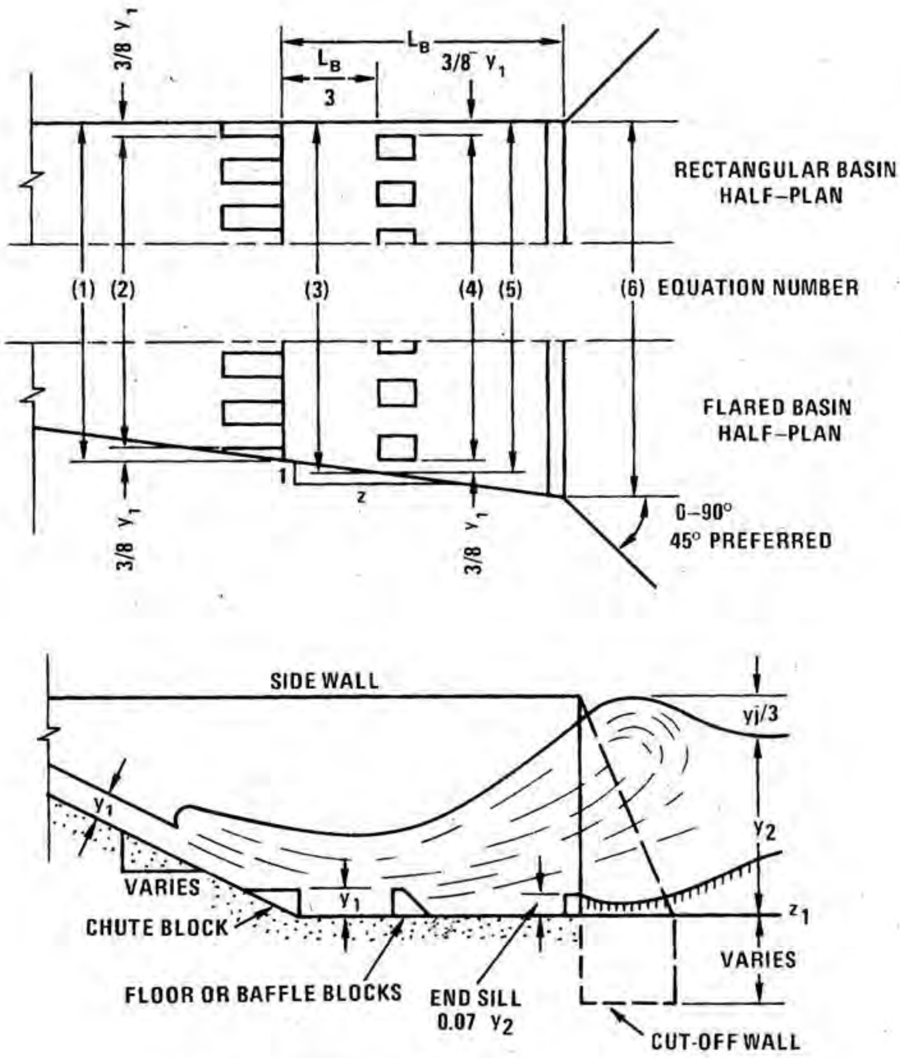
WRC ENGINEERING INC

REFERENCE: HYDRAULIC DESIGN OF ENERGY DISSIPATORS FOR CULVERTS AND CHANNELS (HEC-14) (1983)

**FIGURE 911**

# SAF STILLING BASIN




(1) $W_B$ = BASIN WIDTH UPSTREAM

(2) n BLOCKS AT 3/4 $y_1 \pm$

(3) $0.40\,W_{B2} \leq$ AGGREGATE BLOCK WIDTH $\leq 0.55\,W_{B2}$

(4) n BLOCKS AT 3/4 $y_1 \frac{W_{B2}}{W_B} \pm$

(5) $W_{B2} = W_B + 2L_B/3z$

(6) $W_{B3} = W_B + 2L_B/z$

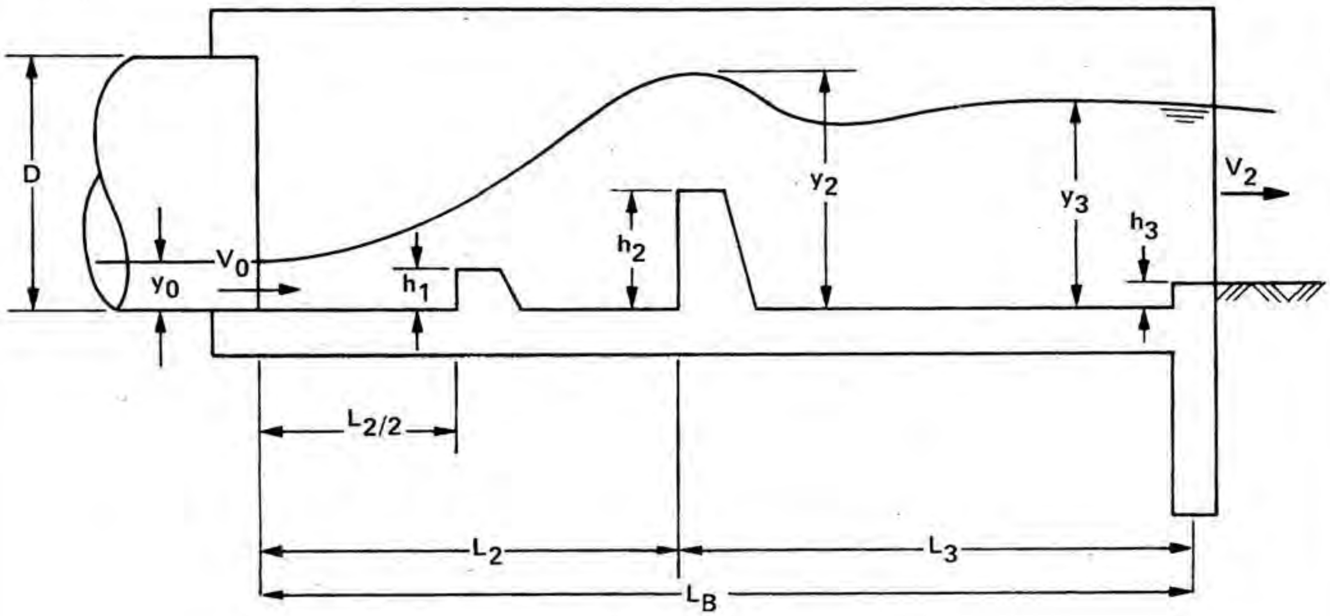
| Revision | Date |
|---|---|
| ORIGINAL ISSUE | 3/27/06 |
| | |
| | |

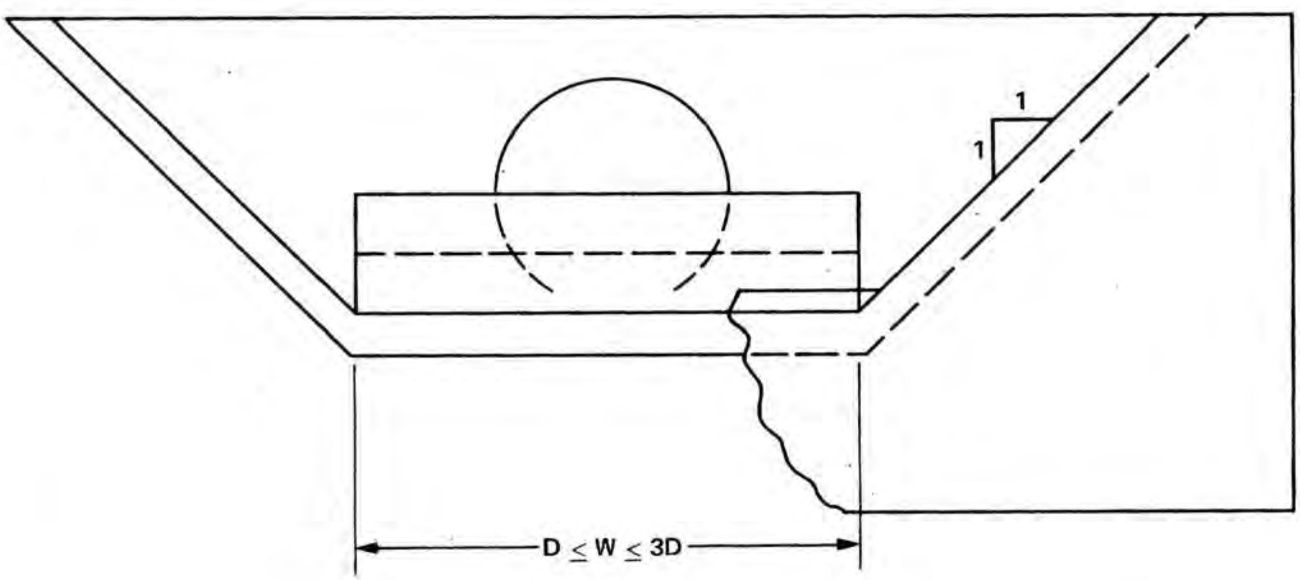
WRC ENGINEERING INC

REFERENCE: HYDRAULIC DESIGN OF ENERGY DISSIPATORS FOR CULVERTS AND CHANNELS (HEC-14) (1983)

**FIGURE 912**

# CONTRA COSTA ENERGY DISSIPATOR

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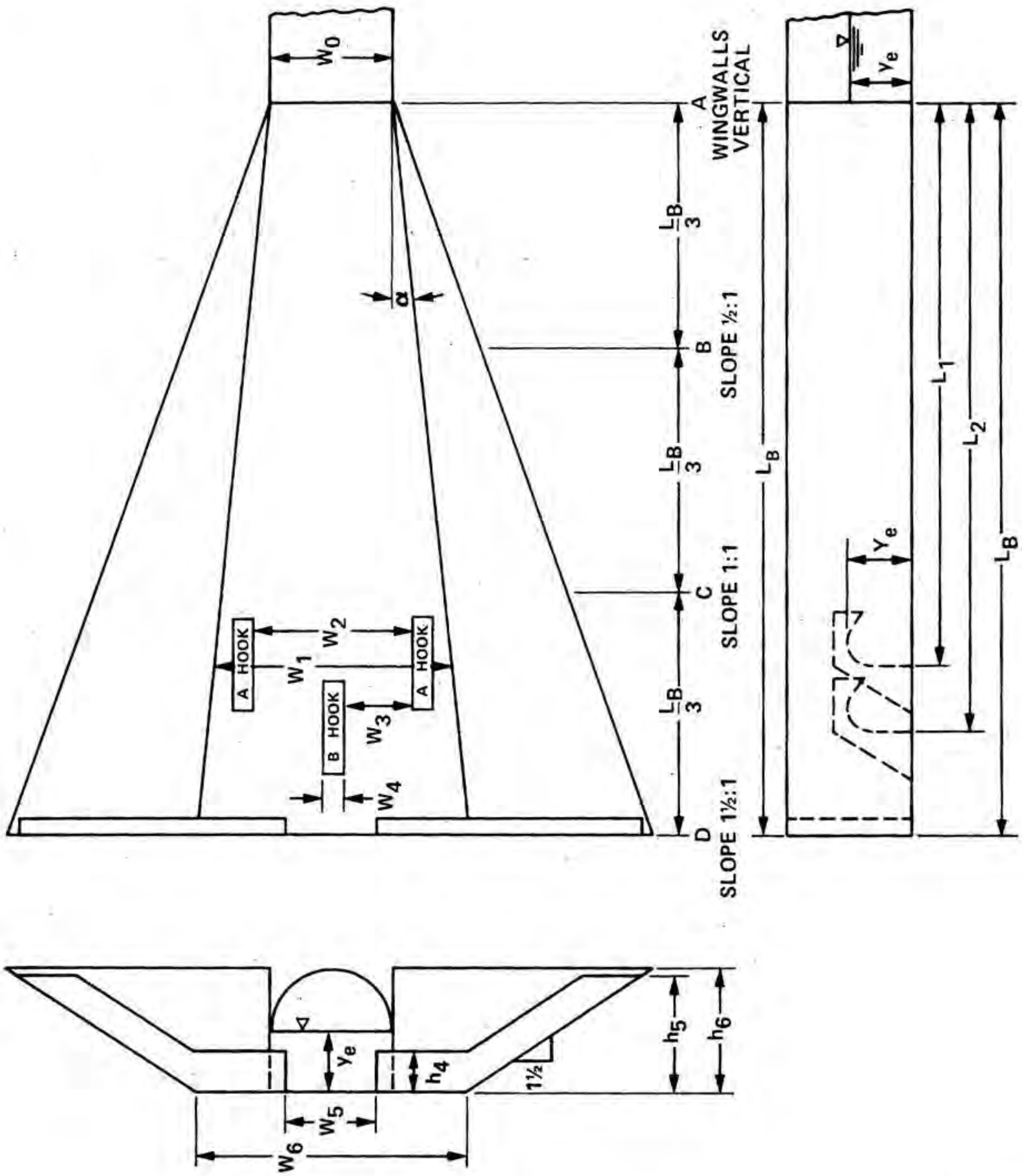
REFERENCE: HYDRAULIC DESIGN OF ENERGY DISSIPATORS FOR CULVERTS AND CHANNELS (HEC-14) (1983)

**FIGURE 913**

# HOOK-TYPE ENERGY DISSIPATOR (WARPED WINGWALL BASIN)

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| ORIGINAL ISSUE | 3/27/06 |
| | |
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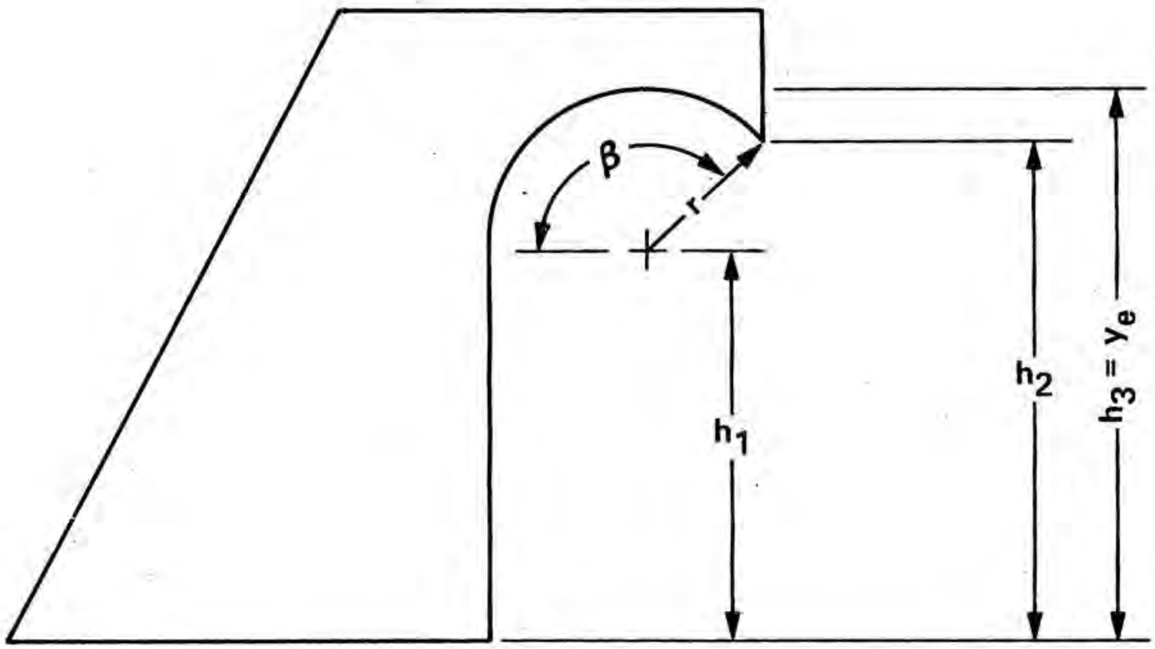
# HOOK-TYPE ENERGY DISSIPATOR
# (HOOK DETAIL - WARPED WINGWALL BASIN)




$$h_3 = y_e$$
$$h_2 = 1.28\,h_1$$
$$h_1 = y_e/1.4$$
$$\beta = 135^{o}$$
$$r = 0.4\,h_1$$

**HOOK FOR WARPED WINGWALL BASIN**

| Revision | Date |
|---|---|
| ORIGINAL ISSUE | 3/27/06 |
| | |
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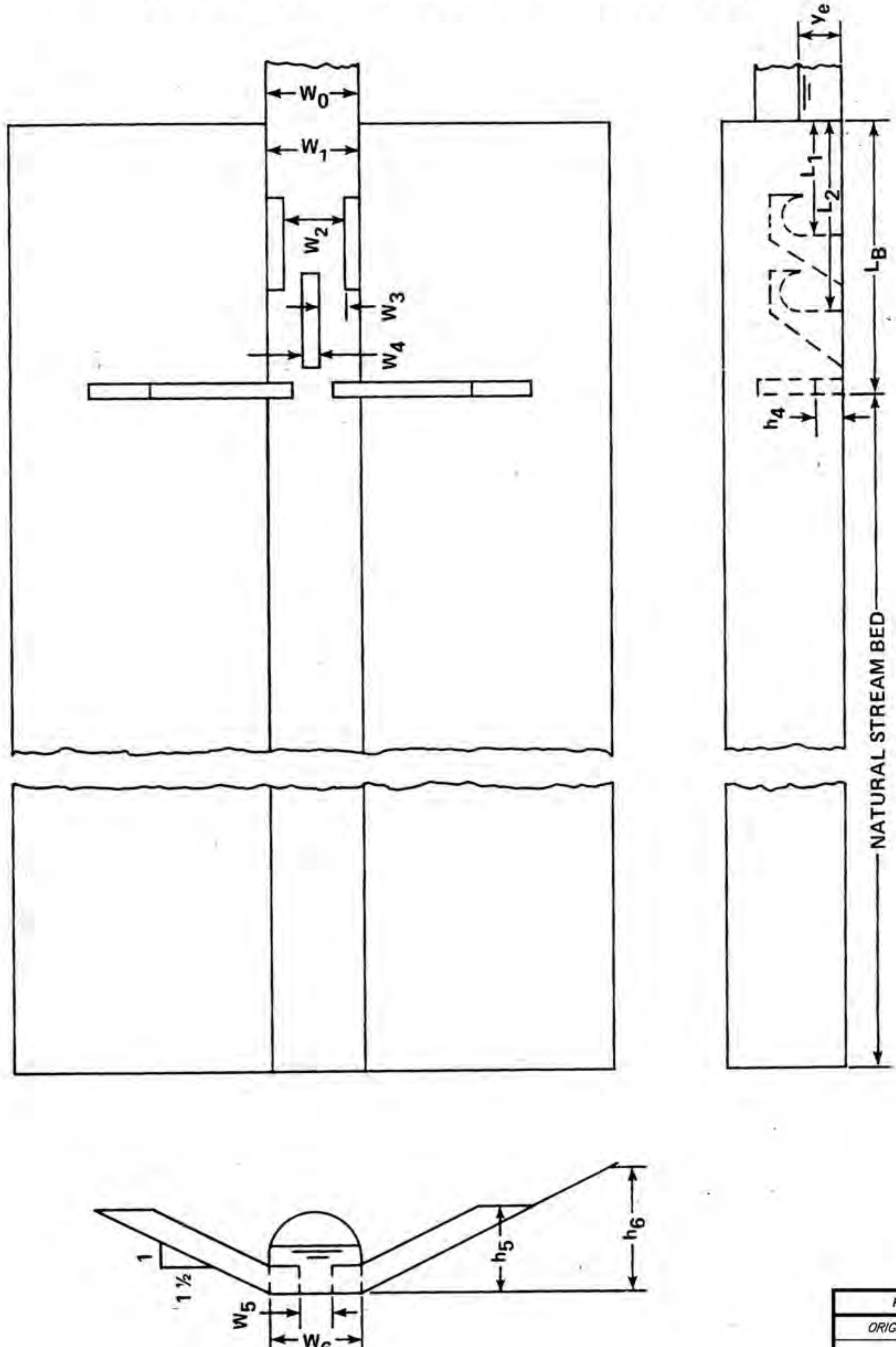
# HOOK-TYPE ENERGY DISSIPATOR (STRAIGHT TRAPEZOIDAL BASIN)

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| ORIGINAL ISSUE | 3/27/06 |
| | |
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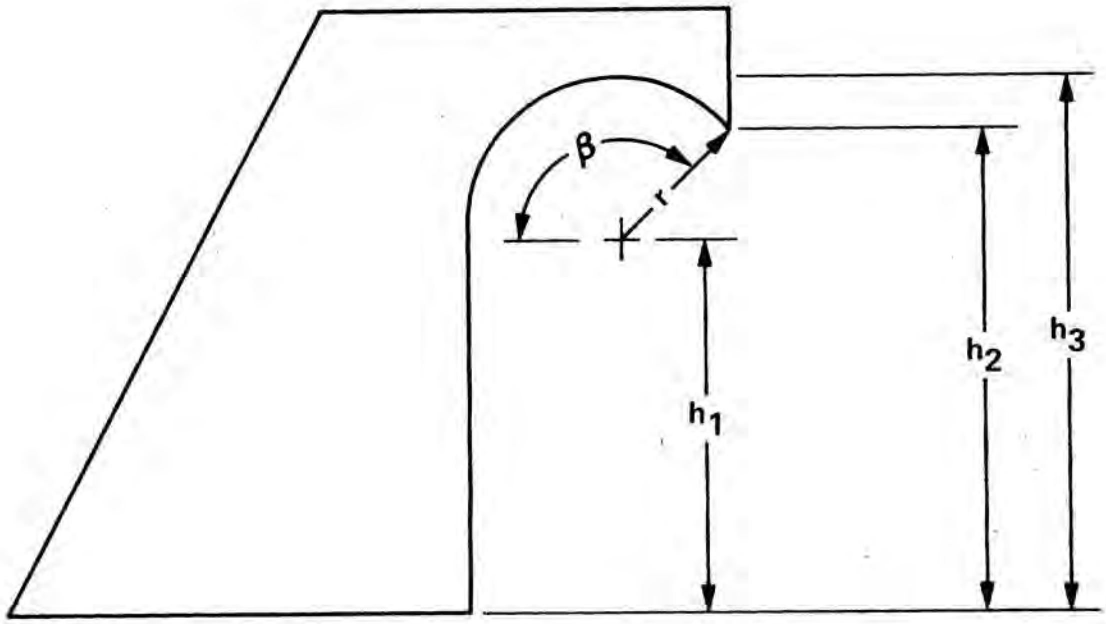
WRC ENGINEERING, INC.

REFERENCE: HYDRAULIC DESIGN OF ENERGY DISSIPATORS FOR CULVERTS AND CHANNELS (HEC-14) (1983)

**FIGURE 915a**

# HOOK-TYPE ENERGY DISSIPATOR (HOOK DETAIL - STRAIGHT TRAPEZOIDAL BASIN)




$h_2 = y_e$

$h_1 = 0.78\, y_e$

$h_3 = 1.4\, h_1$

$r = 0.4\, h_1$

$\beta = 135^{o}$

HOOK FOR STRAIGHT TRAPEZOIDAL BASIN

| Revision | Date |
|---|---|
| ORIGINAL ISSUE | 3/27/06 |
| | |
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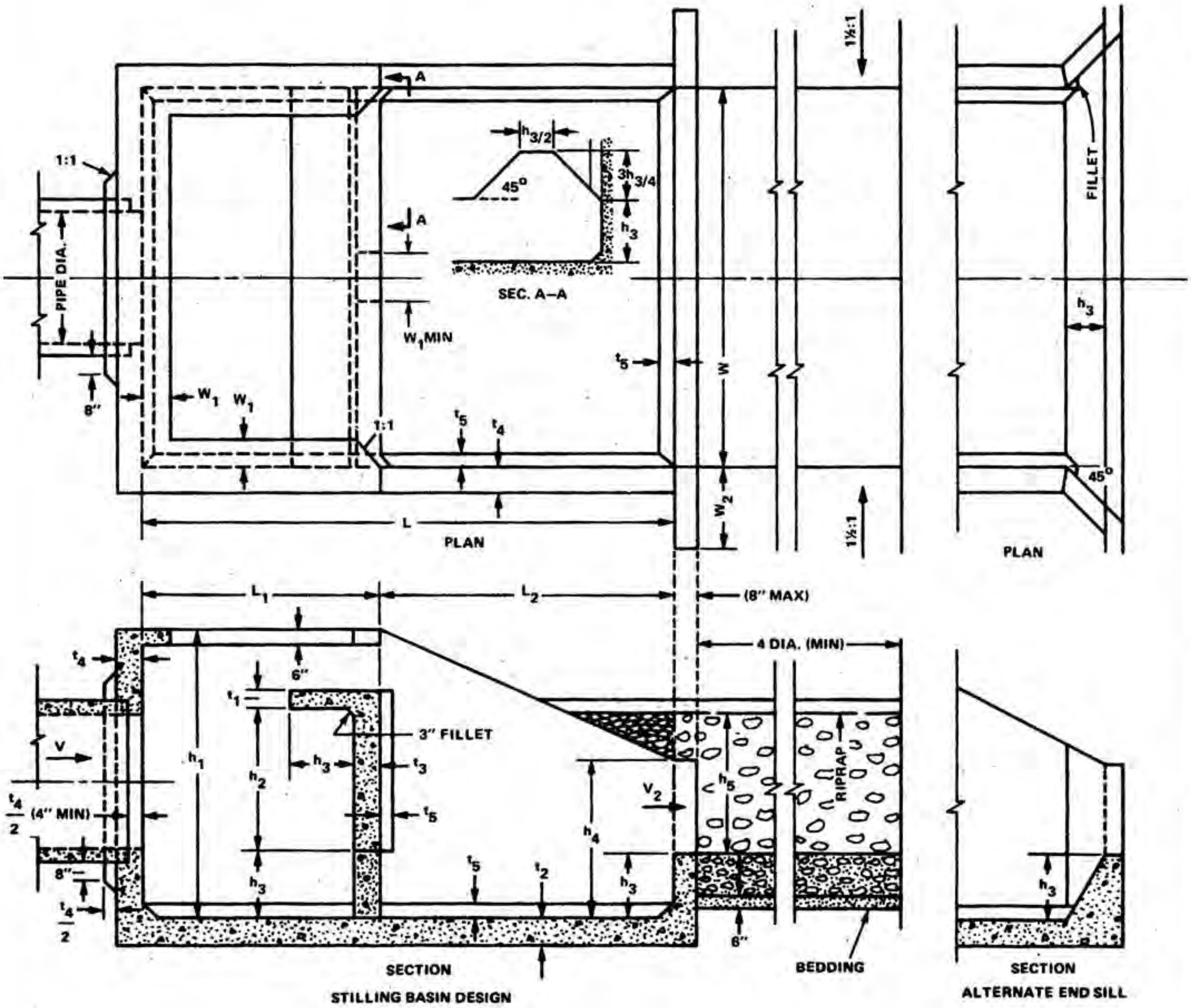
# IMPACT -TYPE ENERGY DISSIPATOR (USBR TYPE VI)

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| ORIGINAL ISSUE | 3/27/06 |
| | |
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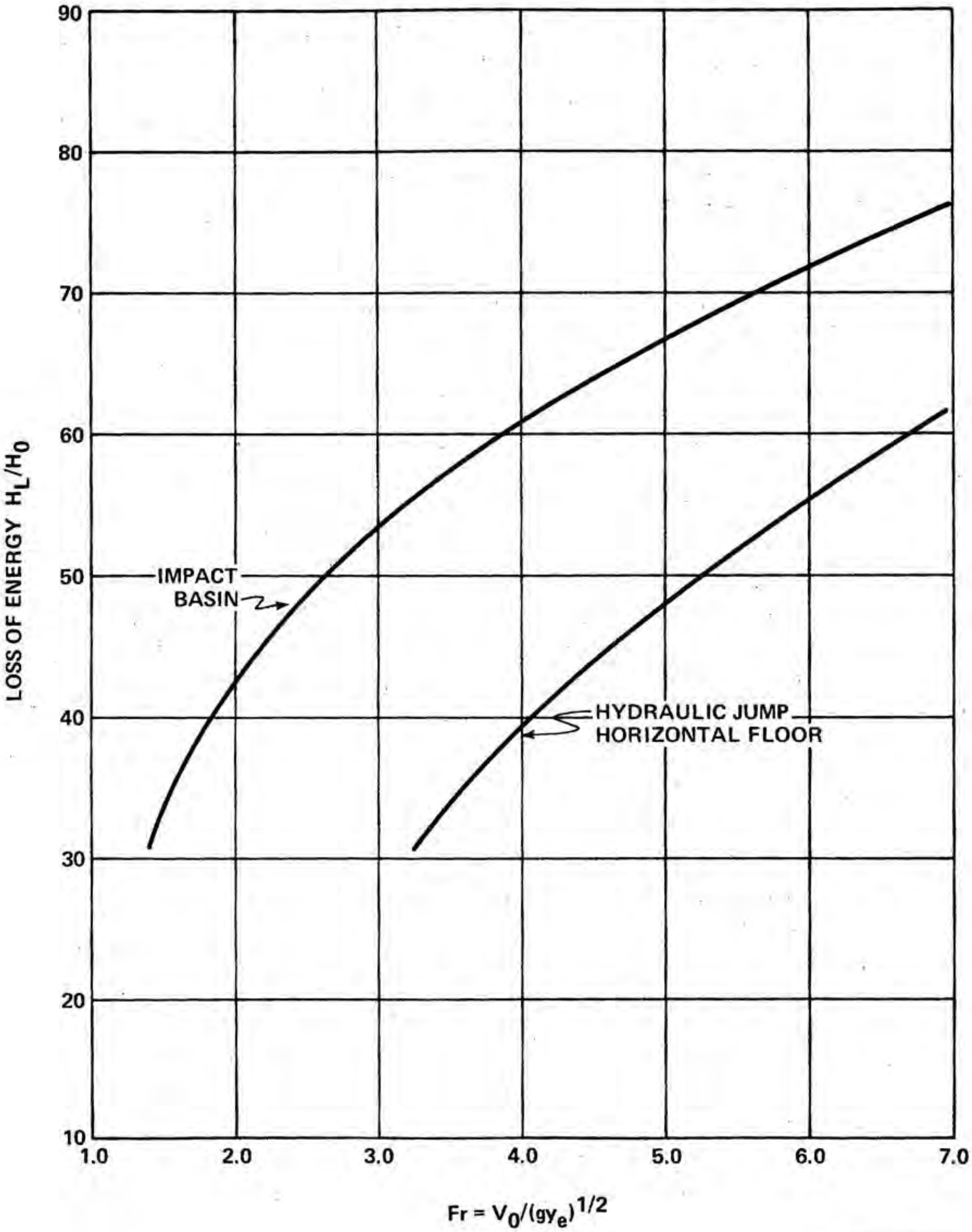
WRC ENGINEERING, INC.

REFERENCE: HYDRAULIC DESIGN OF ENERGY DISSIPATORS FOR CULVERTS AND CHANNELS (HEC-14) (1983)

**FIGURE 916a**

STORMWATER MANAGEMENT MANUAL

# IMPACT -TYPE ENERGY DISSIPATOR (ENERGY LOSS)

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|---|---|
| ORIGINAL ISSUE | 3/27/06 |
| | |
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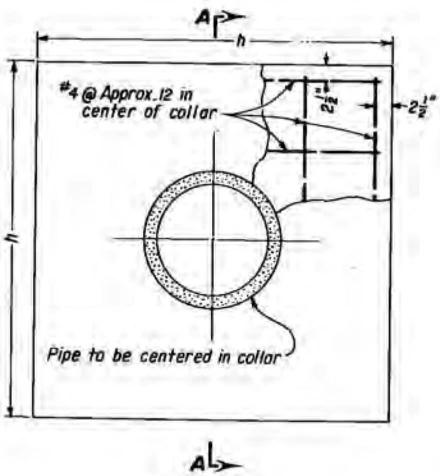
WRC ENGINEERING INC

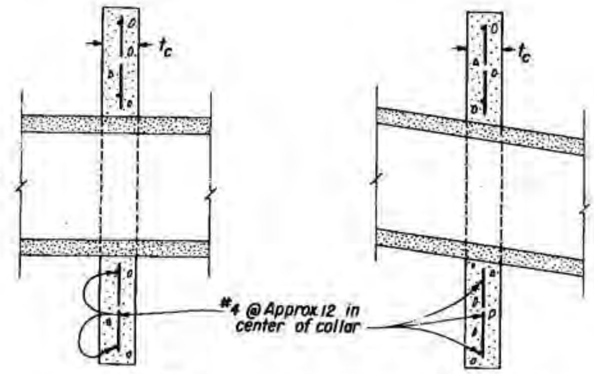
REFERENCE: HYDRAULIC DESIGN OF ENERGY DISSIPATORS FOR CULVERTS AND CHANNELS (HEC-14) (1983)

**FIGURE 916b**

# PIPE COLLAR DETAILS

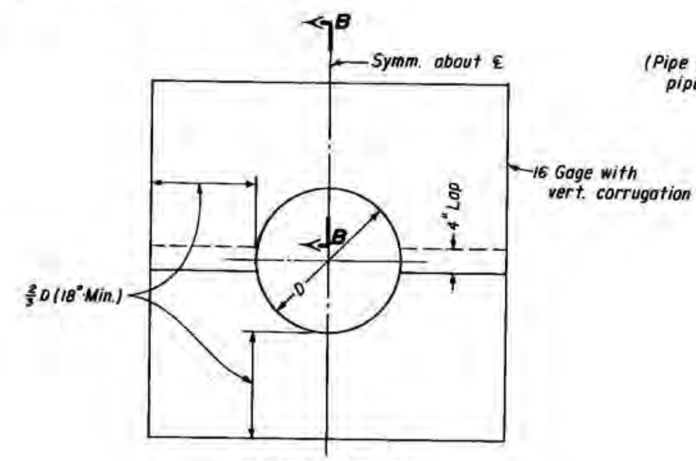



**ELEVATION**
*(Pipe collar for precast concrete pipe, asbestos-cement pipe and reinforced plastic mortar pipe)*

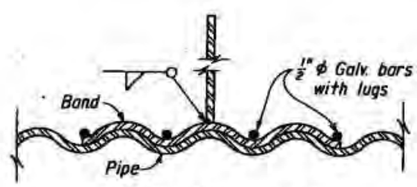



**LEVEL PIPE** **SLOPED PIPE**

**SECTION A-A**




**ELEVATION**
*(Pipe collar for corrugated metal pipe)*




**SECTION B-B**

**DIMENSIONS AND ESTIMATED QUANTITIES**
*(Pipe collar for precast concrete pipe, asbestos-cement pipe and reinforced plastic mortar pipe)*

| PIPE DIA. | h | $t_c$ | CONC. CU.YD. | REINF. LBS. |
|---|---|---|---|---|
| 18" | 5'-0" | 6" | 0.4 | 33 |
| 21" | 5'-3" | 6" | 0.4 | 35 |
| 24" | 5'-6" | 6" | 0.5 | 44 |
| 27" | 6'-3" | 6" | 0.7 | 50 |
| 30" | 7'-0" | 6" | 0.8 | 62 |
| 36" | 8'-6" | 6" | 1.2 | 96 |
| 42" | 9'-3" | 8" | 1.8 | 100 |
| 48" | 10'-0" | 8" | 2.2 | 120 |
| 54" | 11'-6" | 8" | 2.7 | 170 |
| 60" | 12'-0" | 8" | 2.9 | 180 |

| Revision | Date |
|---|---|
| ORIGINAL ISSUE | 3/27/06 |

WRC ENGINEERING INC.

REFERENCE:
DESIGN OF SMALL CANAL STRUCTURES (USBR, 1974)

**FIGURE 917**

# SECTION 1000

## STORM DRAIN SYSTEMS

# SECTION 1000
## STORM DRAIN SYSTEMS

## TABLE OF CONTENTS

## LIST OF TABLES



## LIST OF FIGURES

# SECTION 1000
# STORM DRAIN SYSTEMS

## 1001 INTRODUCTION

Storm drains are used to convey runoff in locations where streets or other drainage facilities exceed their designated capacity or are otherwise unable to drain. The most common method for the introduction of water into a storm drain is the street inlet, discussed in Section 1100. However, water may also enter the system via grated area inlets, culvert-type inlets (typically for the conveyance of drainage-channel flow into the drain), pump stations, or other entry points. The design of a storm drain system is dependant on topography, street rights-of-way and drainage easements, the need to convey flows from multiple locations, existing and proposed structures and utilities, outfall locations, local hydrology, and regional and local design criteria.

Typically, storm drains are sized to convey the peak runoff from the minor storm in excess of the contributing street flow capacity. This means the upper end of a storm drain branch will usually be located at the first inlet encountered by runoff in a given sub-watershed. As discussed in Section 1100, the first inlet will either be located at the point where street flow from the design storm exceeds street capacity for that storm (on-grade inlet) or where there is a vertical sag in the street (sump inlet). In some cases, however, street inlets discharge their intercepted flow to drainage facilities other than a storm drain (e.g., a drainage channel). Storm drains shall be sized to carry the maximum difference between street capacity and peak runoff for any given design storm. This could be the difference between the major storm peak runoff and the allowable street capacity for the major storm, or it could be the difference between the minor storm runoff and allowable street capacity for the minor storm. This is discussed further in Section 1005.

Occasionally, inlets and storm drains must be sized to convey the entire major storm event flow. Two examples of this situation follow:

1. Locations where street flow is not in the desired direction and there is no other feasible drainage solution (such as closed basins – natural ponding areas).
2. Locations where the standard allowable major storm street capacities do not apply, such as negative slopes outside the curb but within the right-of-way.

Peak runoff values are found using the methods set forth in Sections 600 and 700 of this Manual

## 1002 STORM DRAIN DESIGN CRITERIA

This section presents certain parameters relating to the design and construction of storm drain systems in Mesa County.

### 1002.1 Allowable Capacity

As described in Sections 1001 and 1005, a storm drain is designed to convey up to the entire design storm for all sub-watersheds tributary to it. The design of pressure-flow, or surcharged, storm pipes is allowed under certain restrictions as specified in this Section. These include the calculation of energy grade lines (EGLs) and hydraulic grade lines (HGLs) indicating all hydraulic losses due to friction, junctions, and other structures and phenomenon. The EGL for the storm drain design flow must at no time or location exceed the manhole rim or inlet throat elevation. More restrictive local criteria may apply; it is the responsibility of the designer to select the most restrictive of all applicable design criteria. Note that calculation of EGL and HGL is mandatory on all projects for the drainage plan submittal.

For the purpose of completing a conceptual storm drain system design, calculation of an EGL and HGL is not required. In these cases, the initial design methods presented in Section 1005.1 (using open-channel hydraulics as presented in Section 1003.1) are considered sufficient. The specific requirements for a Conceptual Drainage Report are detailed in Section 302 of this Manual.

### 1002.2 Allowable Velocity

Minimum velocities are required in storm drains to reduce sedimentation and promote positive drainage through the pipe at all depths. A minimum design-flow velocity of 2.5 feet per second is required for standard (positive-slope) storm drains. **Table 1001** provides required slope values needed to maintain this minimum velocity for different pipe sizes and roughness factors.

While concrete pipe itself "can carry clear water of extremely high velocities without eroding" (ACPA, 1996), there exist numerous other factors that indicate the need for a maximum velocity in storm drains. Among these are the use of other pipe materials and shapes, expected flow conditions, and the "type and quality of construction of joints, manholes, and junctions." (Washoe County, 1996) Therefore, storm drains shall have a maximum design-flow velocity of 15 feet per second. Note that maximum outfall velocities are more restrictive to protect those areas from extensive erosion. See Sections 800 *Open Channels*, 900 *Other Hydraulic Structures*, and 1200 *Culverts and Bridges* for details.

### 1002.3 Pipe Roughness

Roughness effects tend to vary with changes in flow depth and installation inconsistencies. To simplify design and ensure consistency, this Manual specifies roughness values and does not permit the use of pipe manufacturers' values. **Table 1002** provides a range of Manning's *n* values for many pipe materials and configurations as developed by Chow in 1959 and Normann in 1985 (adapted from tables found in HDS-4 and HEC-22). For purposes of storm drain design, hydraulic roughness shall be specified by the largest Manning's *n* value in the provided range.

The designer may choose to use a higher Manning's *n* value if conditions warrant.

### 1002.4 System Layout

The layout of a storm drain system is dependent on topography, hydrology, surface hydraulics, easements and rights-of-way, existing structures and utilities, outfall locations, and other factors. General criteria for the design of a storm drain layout follow.

1002.4.1 Vertical Alignment

- Minimum and maximum cover are determined by the size, material, and class of pipe, as well as the characteristics of the cover material and the expected surface loading. The designer shall consult the appropriate data sources to include:

  - Colorado Department of Transportation *Standard Specifications for Road and Bridge Construction*, Section 700 (Materials Details)
  - *Concrete Pipe Design Manual* (ACPA)
  - *Handbook of Steel Drainage and Highway Construction Products* (AISI)
  - Pipe Manufacturer Specifications
  - Other applicable references

  Storm drains crossing under railroads and highways must comply with any cover requirements specified for culverts (Section 1200).

- Pipes installed under any driving or parking area shall be designed for H-20 minimum live load. (City of Grand Junction *Standard Specifications for Construction of Underground Utilities – Waterlines, Sanitary Drain, Storm Drains, Underdrains and Irrigation Systems*)

- Storm drain mains (any storm drain to which laterals connect) shall have a minimum cover of 36 inches over the top of the pipe. This minimum includes any pavement thickness, but does not replace the minimum cover and compaction requirements designated by local standards and the application of valid structural loading computations.

1002.4.2 Horizontal Alignment

- Storm drain bends, whether completed using the pulled-joint method, bend pipe, or radius (curved) pipe, shall be avoided where possible. Bends are not allowed for storm drain pipe of less than 48-inch diameter. **Table 1003** shows the maximum allowed deflection for pulled – joint construction.

**Table 1003 Maximum Allowed Deflection for Pulled-Joint Construction**

| Pipe Diameter or Span (inches) | Allowed Deflection (Pull) Per Joint |
|---|---|
| 48 – 54 | 5/8 |
| 60 – 78 | 3/4 |
| 84 – 102 | 7/8 |
| 108 – 144 | 1 |

- Per the *City of Grand Junction General Utility Details*, storm drain manholes shall be located at the centerline of a traffic lane. Storm drain mains are to be located on the south or west side of a roadway, and must have a minimum horizontal clearance of 6.0 feet from the roadway centerline to the storm drain centerline. In cases where the sanitary drain main is not located at the street centerline, the designer shall consult with the appropriate local jurisdiction to determine the required horizontal and vertical clearances.

Maximum allowable spacing between manholes is presented in Section 1002.4.4 and **Table 1004** of this Manual.

1002.4.3 Utility Clearances

The designer shall consult with the most recent versions of the following documents to ensure compliance with the most restrictive (largest) utility clearance values applicable to the subject location:

- City of Grand Junction *Standard Specifications for Construction of Underground Utilities – Waterlines, Sanitary Drains, Storm Drains, Underdrains and Irrigation Systems*
- City of Grand Junction *Standard Details for Construction of Streets, Storm Drains, and Utilities*
- City of Grand Junction *Transportation Engineering Design Standards* (TEDS) Manual
- Any utility clearance requirements set forth by a local jurisdiction or special district

Pipe encasement may be required in some locations where minimum utility clearances are unable to be met. Standards for the design and installation of casing pipe and concrete encasement can be found in the *General Utility Details* of the City of Grand Junction *Standard Details for Construction of Streets, Storm Drains, and Utilities.*

1002.4.4 Manholes

Manholes are necessary to provide maintenance and inspection access to the storm drain. When designed correctly, they also provide more hydraulically efficient pipe junctions and other transitions. All manhole lids must bear the words storm water for identification purpose.

- For storm drain pipes of less than 48-inch diameter, a manhole must be located at all changes in main-line pipe size or grade, junctions where a lateral joins the main-line alignment at a higher elevation (vertical drops), main-line vertical drops (drop manhole), and main-line direction changes or bends. Manholes located at storm drain bends shall be located at either tangent intersection or within the bend itself.
- Pipes of 48 inches or larger diameter do not necessarily require manholes at all locations specified above. However, manholes in addition to those required by standard maximum spacing may be stipulated by local jurisdiction.
- **Table 1004** indicates maximum spacing for manholes. Noncircular pipes shall be converted to equivalent diameters based on pipe area.

**Table 1004 Maximum Manhole/Accessible Junction Spacing**

| Equivalent Pipe Diameter | Maximum Allowable Manhole Spacing |
|---|---|
| Less than 48 inches | 400 feet |
| 48 inches or larger | 600 feet |

## 1003 STORM DRAIN HYDRAULICS

This section presents the hydraulic methods used to calculate storm drain capacities and thereby to design a storm drain system. The actual design process is presented in Section 1005. The majority of the methods in this section are adapted from those presented in *HEC-22 (Urban Drainage Design Manual)* and *HDS-4 (Introduction to Highway Hydraulics).*

### 1003.1 Gravity-Flow Analysis

Initial storm drain design is completed by selecting pipe sizes based on "just full" capacity. This means that the drain capacity is calculated using open-channel (non-pressurized) flow computations. Starting at the uppermost reach of the storm drain (at the first inlet), the designer applies Manning's equation (Equation 1001) for each segment of drain. A segment is a reach of pipe with a junction, transition, grade change, horizontal bend, or pipe size change at each end.

$$Q_f = \frac{1.49}{n} A_f R_f^{2/3} S_o^{1/2} \qquad (1001)$$

Where:

$Q_f$ = Full Flow Discharge (cfs)

$n$ = Manning's Roughness Coefficient (see Sec. 1002.3)

$A_f$ = Full Flow Area $= \frac{\pi D^2}{4}$ for circular pipes (sf)

$R_f$ = Full Flow Hydraulic Radius $= D/4$ for circular pipes (ft)

$S_o$ = Pipe Slope ($S_o = S_f$ for full flow) (ft/ft)

$D$ = Pipe Diameter (feet)

Equation 1002 is a form of Manning's that can be used to directly solve for the minimum required pipe diameter for circular pipes. The designer shall always round up to the nearest available pipe size, keeping in mind that minor losses in the pipe may decrease available capacity. Initial pipe size, $D_i$ (ft), is based on the peak design flow for that pipe segment, $Q_P$ (cfs).

$$D_i = \left[\frac{2.16 n Q_P}{S_o^{1/2}}\right]^{3/8} \qquad (1002)$$

For noncircular pipes, Equation 1002 provides the equivalent diameter based on flow area.

To better account for energy losses that will occur in the system, the designer may choose to calculate preliminary head losses through inlet and manhole junctions. Application of these approximate losses will allow for better estimation of required pipe sizes during the initial design process, expediting the preliminary and final design phases. *HEC-22* presents the following equation and table for the calculation of approximate junction head loss:

$$H_{ah} = K_{ah}\left(\frac{V_o^2}{2g}\right) \tag{1003}$$

Where:

$H_{ah}$ = Preliminary Junction Head Loss Estimate (ft)

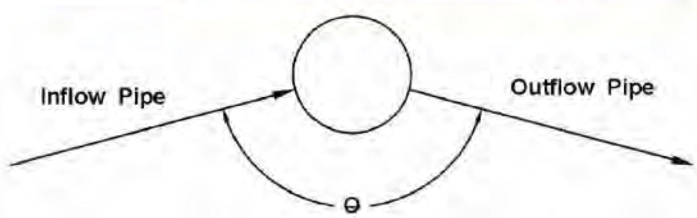
$K_{ah}$ = Head Loss Coefficient from **Table 1005**

$V_o$ = Flow Velocity = $Q_P / A_f$ (fps)

$g$ = Gravitational Constant = 32.2 ft$^2$/sec

**Table 1005 Preliminary Head Loss Coefficients (for Conceptual/Initial Design ONLY)**

| Structure Configuration | Coefficient, $K_{ah}$ |
|---|---|
| Inlet – Straight Run | 0.50 |
| Inlet – Angled Through (θ) | |
| 90° | 1.50 |
| 60° | 1.25 |
| 45° | 1.10 |
| 22.5° | 0.70 |
| Manhole – Straight Run | 0.15 |
| Manhole – Angled Through (θ) | |
| 90° | 1.00 |
| 60° | 0.85 |
| 45° | 0.75 |
| 22.5° | 0.45 |




From HEC-22, Table 7-5a and Figure 7-4.

**Figures 1001 through 1004** present relative velocities and flows for circular, elliptical (horizontal and vertical), and arch pipe under gravity-flow conditions. Similar charts for box sections can be found in the Concrete Pipe Design Manual and other design aids.

### 1003.2 Pressure-Flow Analysis

Following the initial "just-full" storm drain design, the system is analyzed using energy-momentum theory to account for specific energy losses. This method allows for the calculation of hydraulic and energy grade lines (HGL and EGL) for a given storm drain line by starting with the water surface elevation of the outfall and working upstream, accounting for all losses due to pipe friction, manholes, transitions, bends, junctions, and pipe entrances and exits. In cases where pressure flows exists, certain limitations exist on the maximum elevation of the EGL in relation to the ground surface (finished grade). Compliance with minimum and maximum flow velocities is based on peak design flow in the final selected pipe size for each segment. See Section 1002 for specific design criteria.

Energy-momentum theory is based upon the concept that energy, typically expressed in hydraulics as "head" in a linear dimension such as feet, is conserved along a given conduit segment. For a segment where *A* is the upstream end and *B* is downstream, the steady-flow energy equation can be expressed as:

$$z_A + \frac{p_A}{\gamma} + \frac{V_A^2}{2g} + h_p = z_B + \frac{p_B}{\gamma} + \frac{V_B^2}{2g} + \sum h_L \qquad (1004)$$

Where:

$z$ = Invert Elevation above any Horizontal Datum (ft)
$p$ = Fluid Pressure lbf/ft$^2$
$\gamma$ = Specific Weight of Water $\cong$ 62.4 lbf/ft$^3$
$V$ = Flow Velocity (fps)
$h_p$ = Head Added by a Pump (if applicable) (ft)
$\sum h_L$ = Sum of Head Losses in Segment A - B as calculated per the methods prescribed in this section.

Each term in Equation 1004, and thus the sum of the formula, has a linear dimension (e.g., feet). Each term represents the hydraulic head contributed to the total energy head by that term. For instance, the third term, $\frac{V^2}{2g}$, is the velocity head. The EGL elevation at a given point is equal to:

$$EGL = z + \frac{p}{\gamma} + \frac{V^2}{2g} \qquad (1005)$$

and the HGL elevation is simply the EGL minus the velocity head:

$$HGL = EGL - \frac{V^2}{2g} \text{ and conversely, } EGL = HGL + \frac{V^2}{2g} \qquad (1006)$$

In cases where outfall water surface is equal to or higher than the outlet flow elevation, the EGL and HGL are assumed to be equal, i.e. velocity is zero at the downstream point where calculations start. However, if the outfall water surface is lower than the outlet pipe flow elevation, the latter value is used as the outlet HGL. Note that the outfall water surface elevation used must be determined coincident with the time of peak flow from the storm drain.

The HGL at the next structure (e.g., manhole) is determined by the equations presented in **Table 1006**. The equations are separated by HGL at the pipe inlet downstream of the manhole and the pipe outlet at the inlet to the manhole. For non-surcharged flow (less than 80% pipe depth), the free water surface at the pipe inlet (downstream end of the manhole) is added to head loss across the manhole to find the pipe outlet HGL (upstream end of the manhole).

**Table 1006 Equations for Determining HGL**

| Surcharge Conditions | Outlet Submergence | HGL in Manhole/Junction | At | Equation Number |
|---|---|---|---|---|
| $d_n/D > 0.80$ | N/A | $= HGL_{\text{Pipe Outlet}} + h_f + h_{minor}$ | Pipe Inlet (D/S from MH) | (1007) |
| $d_n/D > 0.80$ | N/A | $= HGL_{\text{Pipe Inlet}} + h_{mh}$ | Pipe Outlet (U/S from MH) | (1008) |
| $d_n/D \le 0.80$ | Unsubmerged | $= WSE_{\text{Pipe Inlet}}$ | Pipe Inlet (D/S from MH) | (1009) |
| $d_n/D \le 0.80$ | Unsubmerged | $= WSE_{\text{Pipe Inlet}} + h_{mh}$ | Pipe Outlet (U/S from MH) | (1010) |
| $d_n/D \le 0.80$ | Submerged | = Larger of Equations 1007 and 1009 OR<br>= Larger of Equations 1008 and 1010 | | |

Where:

$d_n$ = Normal Flow Depth in Pipe (feet)

$HGL_{\text{Pipe Outlet}}$ = Larger of Tailwater Elevation, Flow Depth Elevation at Pipe Outlet, and HGL at Next Downstream Pipe Inlet

$WSE_{\text{Pipe Inlet}}$ = Free Water Surface Elevation at Pipe Inlet

$h_f, h_{mh}, h_{minor}$ = Head losses as described in this section

Occasionally, design flow through a pipe may be not only gravity-flow (non-surcharged) but also supercritical. Pipe losses ($h_f$ and $h_{minor}$) in a supercritical pipe section are not carried upstream. (HEC-22)

In locations where two adjoining pipe segments flow in supercritical conditions, manhole losses are also ignored for that line. The designer shall be careful to include these losses where only one of the pipes on the line under investigation contains supercritical flow.

Inlet pipes to a manhole must occasionally have an invert significantly above that of the outlet pipe. In locations where the outlet pipe water surface elevation (or HGL if pressure flow) is below the invert of an inlet pipe, that inlet pipe is treated as an outfall pipe. In this case, the outfall water surface elevation is always lower than the pipe outlet water level, so the latter elevation is used for the initial HGL of the new upstream reach. The outflow pipe from the manhole in such a situation acts as a culvert under either inlet or outlet control. See Section 1200 and/or FHWA *Hydraulic Design of Highway Culverts* (HDS-5) for information regarding the computation of an HGL at the manhole and calculation of head loss due to a culvert inlet.

The following sections prescribe methods for determining the energy losses induced by pipe friction, manholes, and other structures (minor pipe losses) that may be encountered by storm drain flows.

1003.2.1 Pipe Friction Losses

Pipe friction is a significant source of energy dissipation in storm drains, whether in gravity-flow or pressure-flow conditions. For the former, friction slope ($S_f$) can be assumed to be equal to the slope of the pipe invert ($S_o$). For pipes with a surcharge flow condition ($d_n/D > 0.80$), Equations 1011 and 1012 define friction slope (units for variables are the same as in Equation 1001 when using English units).

$$S_f = \frac{n^2 V_{avg}^2}{K_Q R^{4/3}} \qquad (1011)$$

Where:

$K_Q = 2.21$ (English Units)

$K_Q = 1.0$ (S.I. Units)

$$S_f = \left[\frac{Q_{avg} n}{K_Q D^{8/3}}\right]^2 \qquad (1012)$$

Where:

$K_Q = 0.46$ (English Units)

$K_Q = 0.312$ (S.I. Units)

Equation 1011 is a form of the Chezy-Manning Formula, and is based on average velocity in the pipe segment. Since flow rate and cross-sectional area typically remain constant through one segment of pipe, average velocity can be assumed to equal flow rate divided by flow area. Where flow rate and/or pipe size changes within one segment (such as at a pipe transition without a manhole or a no-access junction), this velocity is the average of those calculated at the ends of the pipe segment. (Linsley, 1992) Equation 1012 is based on the average flow rate in the pipe segment.

Once the friction slope is known, pipe friction head loss is calculated by multiplying the friction slope by the pipe segment length:

$$h_f = S_f L \qquad (1013)$$

1003.2.2 Manhole Junction Losses

This section details the Energy-Loss Method used by the HYDRAIN program (FHWA) as presented in HDS-4 for calculation of approximate head loss through a manhole. This method applies to any junction of two or more pipes accessible by a manhole. The approximate head loss coefficient values presented in **Table 1005** are replaced by the values computed herein.

For each manhole, the designer must first calculate the initial head loss coefficient ($K_o$) and all applicable coefficient correction factors ($C_x$). The adjusted head loss coefficient (K) and head loss in the manhole ($h_{mh}$) are then computed.

$$h_{mh} = K\left(\frac{V_o^2}{2g}\right) \tag{1014}$$

$$K = K_o C_D C_d C_Q C_p C_B \tag{1015}$$

$$K_o = 0.1\frac{b}{D_o}(1-\sin\theta) + 1.4\left(\frac{b}{D_o}\right)^{0.15}\sin\theta \tag{1016}$$

Where:

$\theta$ = Angle Between Inflow and Outflow Pipes ($\leq 180°$)

b = Manhole or Junction Diameter (at water level)

$D_o$ = Outlet Pipe Diameter

The coefficient correction factors are calculated using the equations presented below and are applied to the initial head loss coefficient per Equation 1015. Note that some correction factors do not apply to all manhole configurations. These non-applicable factors are set to unity.

- $C_D$ – Correction Factor for Pipe Diameter

  This applies to pressure flow when the ratio of water depth in the manhole above the outlet pipe invert to outlet pipe diameter is greater than 3.2. $d_{mho}/D_o > 3.2$

$$C_D = \left(\frac{D_o}{D_i}\right)^3 \tag{1017}$$

Where:

$D_o$ = Outlet Pipe Diameter

$D_i$ = Inlet Pipe Diameter

- $C_d$ – Correction Factor for Flow Depth

  This applies to gravity flow and low-pressure flow when the ratio of water depth in the manhole above the outlet pipe invert to outlet pipe diameter is less than 3.2. $d_{mho}/D_o < 3.2$

$$C_d = 0.5\left(\frac{d_{mho}}{D_o}\right)^{0.6} \tag{1018}$$

Where:

$d_{mho}$ = Water Depth in Manhole above Outlet Pipe Invert

$D_o$ = Outlet Pipe Diameter

For purposes of this calculation, water depth in the manhole is approximated as the vertical distance from the outlet pipe invert to the HGL at the upstream end of the outlet pipe.

- $C_Q$ – Correction Factor for Relative Flow

  This applies to manholes with three or more pipes entering the structure at similar elevations (one of these pipes will be the outlet pipe). This correction factor does not apply to the effects of inflow pipes with flowlines far enough above the outlet pipe to qualify as plunging flow (see Equation 1020 and explanation, this Section).

$$C_Q = (1 - 2\sin\theta)\left(1 - \frac{Q_i}{Q_o}\right)^{0.75} + 1 \qquad (1019)$$

Where:

$\theta$ = Angle between the Inflow Pipe of Interest and the Outflow Pipe

$Q_i$ = Flow in the Inflow Pipe of Interest

$Q_o$ = Flow in the Outflow Pipe

The "pipe of interest" is the inlet pipe to the manhole on the line being investigated. This factor accounts for streamline interference by flow from other pipes entering the manhole. See **Figure 1005** for an illustration of the relative flow effect.

- $C_p$ – Correction Factor for Plunging Flow

  This applies to manholes with an inflow pipe of interest that is affected by plunging flow from another inflow pipe with a higher flowline. The factor does not apply to the line with the pipe that is discharging the plunging flow, and only applies when the height of the plunging-flow pipe flowline above the outlet pipe center exceeds the manhole water depth above the outlet pipe invert: $h > d_{mho}$

$$C_p = 1 + 0.2\left(\frac{h}{D_o}\right)\left(\frac{h - d_{mho}}{D_o}\right) \qquad (1020)$$

Where:

$h$ = Vertical Distance of Plunging Flow (height of plunging flow pipe flowline above center of outlet pipe)

$d_{mho}$ = Water Depth in Manhole above Outlet Pipe Invert

$D_o$ = Outlet Pipe Diameter

A common application of this correction factor occurs at locations where inlets convey intercepted flow directly (vertically) to the storm drain main line (drop inlets) or where laterals enter a manhole well above the main line invert.

- $C_B$ – Correction Factor for Benching
  This applies to all flow conditions. See **Figure 1006** and **Table 1007** for proper correction factor selection.

**Table 1007 Benching Correction Factors**

| Bench Type (see Figure 1006) | Outlet Pipe Conditions | |
|---|---|---|
| | Fully Submerged, Pressure Flow* | Unsubmerged, Free Surface Flow** |
| Flat or Depressed | 1.00 | 1.00 |
| Benched: ½ Pipe Diameter | 0.95 | 0.15 |
| Benched: 1 Pipe Diameter | 0.75 | 0.07 |
| Improved Bench | 0.40 | 0.02 |

Adapted from *Mesa County SWMM* 1996, Figure "H-4"

*Applies for $d_{mho}/D_o \geq 3.2$
**Applies for $d_{mho}/D_o \leq 1.0$

As can be seen in **Table 1007**, benching in manholes significantly reduces head loss due to outlet inefficiency, especially in unsubmerged conditions. Note that in this case, the submerged pressure-flow factors do not apply until flow depth in the manhole has exceeded 3.2 times the outlet pipe diameter. Therefore, for depths between free surface (gravity) flow and full pressure-flow conditions ($1.0 > d_{mho}/D_o < 3.2$), the designer shall use a linear interpolation to compute the benching correction factor.

1003.2.3 Minor Pipe Losses

This section describes the methods used in Mesa County for the calculation of head losses caused by pipe transitions (expansions or contractions), bends (curved drains), no-access junctions, on-grade inlets, and exits (outlets). The minor losses are added together for a given pipe segment per Equation 1021:

$$h_{minor} = \sum h_{L,minor} = h_e + h_c + h_b + h_j + h_i + h_o \qquad (1021)$$

- $h_e$ and $h_c$ – Transition Losses

  Transition losses occur when pipe size is changed at a location other than a manhole. Expansions may be necessary due to changes in flow rate or slope. Contractions are locations where pipe size is decreased, and are allowed only thru a variance. Methods for head loss calculation through a pipe contraction are included in this Manual.

The calculation of head loss through a transition differs for non-pressure flow and pressure flow.

- Non-Pressure Flow Transitions

$$h_e = K_e\left(\frac{V_1^2}{2g} - \frac{V_2^2}{2g}\right) \tag{1022}$$

$$h_c = K_c\left(\frac{V_2^2}{2g} - \frac{V_1^2}{2g}\right) \tag{1023}$$

Where:

$K_e$ = Expansion Coefficient (see Table 1008a)
$K_c$ = Contraction Coefficient (see Table 1008b)
$K_c = 0.5 \cdot K_e$ for Gradual Contractions
$V_1$ = Velocity Upstream of the Transition
$V_2$ = Velocity Downstream of the Transition

- Pressure-Flow Transitions

$$h_e = K_{ep}\left(\frac{V_1^2}{2g}\right) \tag{1024}$$

$$h_c = K_{cp}\left(\frac{V_2^2}{2g}\right) \tag{1025}$$

Where:

$K_{ep}$ = Expansion Coefficient (see Tables 1009a,b)
$K_{cp}$ = Contraction Coefficient (see Table 1009c)
$V_1$ = Velocity Upstream of the Transition
$V_2$ = Velocity Downstream of the Transition

See **Figure 1007** for illustration of the "Angle of Cone" variable used in **Tables 1008 – 1009**.

- $h_b$ – Bend Losses (Curved Drains)

The minor loss that accompanies a storm drain bend can be approximated by:

$$h_b = 0.0033 \cdot (\Delta)\left(\frac{V^2}{2g}\right) \tag{1026}$$

Where: $\Delta$ = Angle of Curvature (degrees)

This equation does not apply to bends located at manholes. Head losses due to manhole bends and deflections are addressed in Section 1003.2.2.

- $h_j$ – No-Access Junctions

This term applies to head loss associated with locations where a lateral pipe connects to a larger trunk pipe without the use of a manhole structure. While these junctions are not recommended for trunk pipes of less than 48 inches in diameter, it is sometimes physically or economically inefficient to place a manhole at every junction location. At locations where more than one lateral joins the main line (trunk), a manhole is required. The head loss at no-access junctions is related to the relative flows and velocities of all three pipes, the angle between the lateral and trunk pipes, and the cross-sectional area of the trunk pipe.

$$h_j = \frac{(Q_o V_o) - (Q_i V_i) - (Q_L V_L \cos\theta)}{0.5 \cdot g \cdot (A_o + A_i)} + h_{vi} - h_{vo} \qquad (1027)$$

Where:

$Q_o, Q_i, Q_L$ = Outlet, Inlet, and Lateral Flow Rates

$V_o, V_i, V_L$ = Outlet, Inlet, and Lateral Velocities

$h_{vo}, h_{vi}$ = Outlet and Inlet Velocity Heads = $V^2/2g$

$A_o, A_i$ = Outlet and Inlet Cross - Sectional Areas

$\theta$ = Angle of Lateral with respect to Outflow Pipe

- $h_i$ – On-Grade Inlets (Culvert-Type Inlets)

In some locations, water may enter a storm drain system from a drainage channel, overflowing pond, or other conveyance with a flowline approximately equal to that of the storm drain inlet. These storm drain entrances are hydraulically equivalent to culvert inlets, thus the coefficient Ki in Equation 1028 is equal to the culvert entrance loss coefficient Ke provided in Section 1200, **Table 1201** of this Manual. (Note that Ke represents the expansion loss coefficient in this Section 1000.)

$$h_i = K_i \left( \frac{V^2}{2g} \right) \qquad (1028)$$

Where: $K_i$ = On - Grade Inlet Coefficient (see Table 1201)

- $h_o$ – Outlets (Pipe Exits)

This term applies to pipe outlets other than those which exit to a manhole. Outlet losses are always associated with a storm drain system outfall to an open channel, detention/retention basin, or other receiving waters. Outlets that discharge into a body of water with essentially zero velocity in the direction of the storm drain exit lose all velocity (one velocity head). This includes outlets perpendicular to an open channel and all submerged outlets. The storm drain flow is also assumed to lose all velocity when it discharges to open air and plunges to the receiving waters.

$$h_o = \frac{V_o^2}{2g} - \frac{V_d^2}{2g} \qquad (1029)$$

Where:

$V_o$ = Flow Velocity at storm drain exit

$V_d$ = Flow Velocity (in the direction of storm drain flow) in Receiving Waters

Allowable storm drain velocities often differ from those for open channels. Sections 800, 900, and 1200 present criteria for the proper design of outlets to open channels, including the design of riprap and other energy dissipation structures to reduce channel scour potential.

### 1003.3 Computer Hydraulic Modeling

Since the storm drain system design process tends to be somewhat iterative, computer programs are now commonly used to develop and/or model proposed and existing storm drainage networks. Numerous hydrologic modeling programs now exist that can often achieve more accurate results due to hydrograph routing capabilities. Many of these hydrologic programs also include hydraulic simulation modules based on hydrologic computations and system parameters. The tediousness of creating hydrographs for every convergence and divergence point in the system is avoided by using these programs, and hydrograph timing consistency is vastly improved. Other more simple programs are "stand-alone" hydraulic calculators, and are useful if peak flows have been previously determined.

HGL and EGL calculations may be prepared using computer software subject to review by the local jurisdiction. At this time, a list of approved or disapproved public or proprietary computer programs for hydrologic and hydraulic modeling is not maintained. However, the designer is urged to use sound professional judgment to select the program(s) that are most applicable to local design standards and the requirements of the given project. It is recommended that the designer consult with the local development review engineer before using any software that is newly released or has not already been broadly accepted by the engineering community.

## 1004 CONSTRUCTION STANDARDS

This section outlines standards for the construction of storm drain systems as based on the most recent versions of all reference publications. The designer is responsible for procuring and complying with the most current version of each applicable reference document. For hydraulic design, the most restrictive criteria among said references and this Manual are to be used.

## 1004.1 Storm Drain Pipe

### 1004.1.1 Minimum Size

Minimum pipe sizes are required in order to allow for maintenance and inspection and to reduce the effects of expected sedimentation and debris build-up. All storm drain pipes within the public right-of-way shall have a minimum diameter of 18". For noncircular pipes, these minimum diameters represent equivalent diameters based on cross-sectional areas.

### 1004.1.2 Maximum Size

There is no maximum pipe size specified. However, the designer shall consider the possibility of utilizing multiple barrels (pipes) where physically and economically advisable.

### 1004.1.3 Pipe Material and Shape

All storm drain pipes shall comply with the Grand Junction Standard Specifications for construction of underground utilities, well as the most recent revision of the Colorado Department of Transportation Standard Specifications for Road and Bridge Construction (CDOT).

Public storm drain mains (to which laterals are connected) may be circular, elliptical, arch, or box (rectangular – concrete only) pipe of reinforced concrete pipe, corrugated aluminized steel, corrugated aluminum, corrugated polymer coated galvanized steel, corrugated or profile wall polyethylene, or polyvinyl chloride. However, pipe material and shape shall be selected based on not only hydraulic capacity, but also "the ability of a pipeline to maintain full cross-sectional area and function without [excessive] cracking, breaking, or undergoing excessive deflection" (Mesa County SWMM, 1996). The designer should be aware that local jurisdictions may have varied regulations for allowable pipe materials.

### 1004.1.4 Joint Fillers, Sealants, and Gaskets

All pipe joint fillers, sealing compounds, and gaskets, and the installation thereof, shall be governed by the specifications set forth in Section 705 of the CDOT Standard Specifications. Rubber gaskets shall be used at pipe section joints where greater than 5.0 feet of pressure head is expected in the design storm. This is equivalent to locations where the HGL elevation is 5.0 feet higher than the pipe crown.

### 1004.1.5 Backfill Loading

Backfill and cover requirements for storm drain pipe are discussed in Section 1002.4 of this Manual.

### 1004.1.6 Pipe Bedding

Specifications for pipe trenching, bedding, and backfill may be obtained from the City of Grand Junction *General Utility Details and Standard Storm Drain Details*.

### 1004.2 Manholes

Construction details for a standard storm drain manhole are provided in the City of Grand Junction *Standard Storm Drain Details*. Non-standard manhole designs shall meet the design and construction criteria set forth in Section 604 of the CDOT *Standard Specifications*. The EGL for the all design flows must be at or below the manhole rim. Locking manhole covers are not permitted.

### 1004.3 Inlets

Section 1100 of this Manual describes the selection and placement criteria for storm drain inlets in Mesa County. Construction details for street inlets can be found in the City of Grand Junction Standard Storm Drain Details.

Culvert-type inlets, such as those directing ditch flows into a storm drain, are required to include a special end section to increase capacity and reduce erosive potential. See Section 1200 for culvert inlet design criteria.

### 1004.4 Outlets

Storm drain outlets typically discharge to a drainage channel, a natural stream or river, or a detention/retention basin. In order to increase storm drain capacity and reduce erosion potential, outlets are required to include a special end section equivalent to those required for culvert outlets per Section 1200 of this Manual.

Due to the erosive potential of high-velocity storm drain flow on unlined channels and detention/retention basins, a riprap apron and/or an energy-dissipation structure shall be constructed at all storm drain outlets per requirements set forth in Section 1200.

## 1005 <u>STORM DRAIN SYSTEM DESIGN</u>

Prior to starting storm drain design, the allowable minor and major street capacities must be determined and inlets preliminarily sized and located. In most cases, the storm drain design flow at a given point is equal to the cumulative minor storm runoff exceeding the minor storm street flow capacity to that point. However, since the street and storm drain must cumulatively carry the major storm event flow without exceeding the major storm street capacity, the storm drain must occasionally be sized to carry the runoff exceeding that capacity. Furthermore, in locations where a vertical sag exists in the street (sump inlets) and no overflow path exists for the major storm flow, the storm drain must be sized to accept the entire major storm flow minus street ponding allowances. Note that the latter two cases require that the inlets be resized to accommodate the larger flows.

### 1005.1 Initial Storm Drain Design

The following step-by-step procedure is for the initial layout and sizing of a storm drain. The results of this process must be validated by the procedures set forth in Section 1005.2 before the system can be deemed a viable design.

However, this design may be used for conceptual drainage report submittals per Section 302 of this Manual.

1. Choose a system layout based on street rights-of-way and other drainage easements, developed topography, utility locations, and likely cost and performance. This layout shall include preliminary inlet and manhole locations, if any.
2. Complete hydrologic analysis of the project area per Sections 600 and 700 of this Manual. Compute peak flow in each street (see Section 1100) starting at the upper end of the project area and working downstream. Typically, the runoff from multiple streets will converge at a point, so all streets that are tributary to that point must be completed before moving on downstream. An inlet shall be located wherever the minor storm peak street flow exceeds the allowable capacity for that street and at all sump locations.
3. Initial storm drain sizing starts at the uppermost inlet for each street, with individual street storm drains combining where appropriate. The design flow for a given storm drain segment is based on the sum of all flow from upstream pipes and the larger of the major or minor street flow exceeding the respective street capacity at the inlet just upstream from that segment.
4. Using gravity-flow analysis (Manning's open channel flow) as presented in Section 1003.1, including approximate junction head losses, compute required pipe size and slope for each pipe segment. In many locations, storm drain slope will be limited by topography or other design criteria including cover and utility clearance requirements, so slopes are often held constant during the initial design phase. It may be prudent to increase pipe size and/or slope at locations where the preliminary energy loss coefficient may not apply and significant energy losses may occur, such as large or complex pipe junctions and major pipe bends. Pipe size shall not be decreased in a downstream direction except in special situations.

### 1005.2 Preliminary/Final Storm Drain Design

Following the completion of an initial storm drain system design, the preliminary/final design may begin. The level of hydraulic analysis presented in this section shall be met before the design may be included in any final drainage reports (see Section 303).

1. The hydraulics for each system are recomputed using the energy-momentum theory presented in Section 1003.2, starting at each system's outfall point. All applicable energy losses must be included in the calculations, including head loss due to manhole/junction chambers, pipe transitions and bends, no-access junctions, and entrances/exits.
2. The HGL and EGL shall be calculated and plotted for each end of each pipe segment and each side of all locations of additional energy loss listed in Step 1. The EGL shall be limited to a maximum elevation of manhole rim or inlet throat at all locations along the storm drain.

While many designers may choose to utilize computer software to model storm drain systems, smaller projects are still often completed "by hand". Hand calculations are also useful for spot-checking of computer outputs to ensure that software is functioning properly. For this reason, Standard Form 3 in Section 1700 is provided to assist in tabulation of storm drain hydraulic calculations. **Figure 1009** (following this section) is Standard Form 3 showing input corresponding to the example design application presented herein.

### 1005.3 Example Design Application

This section presents an example of energy and hydraulic grade line computation through a simple storm drain system. It is assumed that initial design has previously been completed, the results of which are shown in **Figure 1008**.

Problem: Compute both the energy grade line (EGL) and hydraulic grade line (HGL) at Design Points 1 through 4 for the system shown in **Figure 1009** and check for locations where the EGL reaches any manhole rim or inlet throat.

Solution:

Step 1: Utilizing Standard Form 3 to organize data and calculations, enter "OUTFALL" in the STATION column for the first row. The "pipe" in this case is just the outlet itself, so calculate $h_o$ and enter it in column 19.

$$h_o = \frac{V_o^2}{2g} - \frac{V_d^2}{2g} = \frac{(10\text{cfs}/1.77\text{sf})^2}{2g} - \frac{0^2}{2g} = 0.50'$$

The outfall water surface elevation, 4500.0', exceeds the crown elevation of the outlet pipe, 4496.0'+1.5'=4497.5', so the pipe is flowing full at this point (outlet control). The outfall pool has no velocity component in the direction of the outlet pipe, so the EGL equals the HGL and water surface elevation (column 23). The U/S EGL (column 24) in this case represents the point just inside the outlet:

$$EGL_1 = EGL_0 + h_o = 4500.0' + 0.50' = 4500.5'$$

$$HGL_1 = EGL_1 - H_{v,1-2} = 4500.5' - \frac{(10\text{cfs}/1.77\text{sf})^2}{2g} = 4500.0'$$

Step 2: Enter stations 1 and 2 in columns 1 and 2 of the next row, as well as all known pipe and flow data. Since the pipe was already shown to be flowing full under outlet control, velocity (column 10) is:

$$V_{avg} = Q_{avg}/A_{full} = 10/1.77 = 5.65\text{fps}$$

Velocity head (column 11) and friction slope (column 12) are:

$$H_v = \frac{V^2}{2g} = \frac{5.65^2}{2g} = 0.50'$$

$$S_f = \left[\frac{Q_{avg} \cdot n}{0.46 \cdot D^{8/3}}\right]^2 = \left[\frac{10 \cdot 0.013}{0.46 \cdot 1.5^{8/3}}\right]^2 = 0.009$$

Pipe friction head loss is then found and entered in column 13:

$$h_f = S_f \cdot L = 0.009 \cdot 150' = 1.38'$$

The drain schematic (**Figure 1008**) also indicates a 30-degree bend in this pipe reach. Head loss due to the bend is entered in column 16:

$$h_b = 0.0033\Delta\left(\frac{V^2}{2g}\right) = 0.0033 \cdot 30 \cdot 0.50'$$

These total to 1.88 feet lost in the pipe reach (column 20), not including losses from the manhole at Design Point 2.

Step 3: At this point, columns 23, 24, and 25 can be entered. The downstream EGL is in this case simply equal to the upstream EGL (column 24) from the first row. The upstream EGL and HGL are:

$$EGL_{U/S,pipe} = EGL_1 + \Sigma h_{L,pipe} = 4500.5' + 1.88' = 4502.4'$$

$$HGL_{U/S,pipe} = EGL_{U/S,pipe} - H_v = 4502.4' - 0.50' = 4501.9'$$

Step 4: The calculation of losses through a manhole is completed per the procedure presented in Section 1003.2.2, and is dependent on the line being investigated. To find the maximum HGL in a manhole, losses for each line must be calculated and compared. The manhole at Design Point 2 has two inlet pipes, and thus two lines. The line to Station 3 is completed first:

$$K_o = 0.1\frac{b}{D_o}(1 - \sin\theta) + 1.4\left(\frac{b}{D_o}\right)^{0.15}\sin\theta = 0.1\frac{4}{1.5}(1 - \sin 180^\circ) + 1.4\left(\frac{4}{1.5}\right)^{0.15}\sin 180^\circ = 0.267$$

- Apply $C_D$? → If $d_{mho}/D_o > 3.2$ → $4.4/1.5 = 2.9$ → No (1.0)
- Apply $C_d$? → If $d_{mho}/D_o < 3.2$ → Yes

  $$C_d = 0.5\left(\frac{d_{mho}}{D_o}\right)^{0.6} = 0.5\left(\frac{4.4}{1.5}\right)^{0.6} = 0.954$$

- Apply $C_Q$? → If one or more other inlet pipes at manhole does not qualify as plunging flow. Plunging flow if $h_{2-4} > d_{mho}$ → (4503.0'-4497.5-1.5/2) = 4.75 > 4.4 → plunging flow exists → No (1.0)
- Apply $C_p$? → Yes (see $C_Q$)

$$C_p = 1 + 0.2\left(\frac{h}{D_o}\right)\left(\frac{h - d_{mho}}{D_o}\right) = 1 + 0.2\left(\frac{4.75}{1.5}\right)\left(\frac{4.75 - 4.4}{1.5}\right)$$

$$C_p = 1.148$$

- Apply $C_B$? → Yes (always)

  Use **Table 1007** and linearly interpolate with $d_{mho}/D_o = 2.9$.

  $C_B = 0.841$.

Now apply the correction factors to the initial head loss coefficient:

$$K_{2\text{-}3} = K_o C_D C_d C_Q C_p C_B = 0.267 \cdot 1.0 \cdot 0.954 \cdot 1.0 \cdot 1.148 \cdot 0.841 = 0.246$$

Then apply the corrected head loss coefficient to find the estimated head loss through the manhole (on this line):

$$h_{mh,2\text{-}3} = K\left(\frac{V_o^2}{2g}\right) = 0.246(0.50) = 0.12'$$

Note that the velocity used here is the average velocity in the outlet pipe from the manhole. This value is entered in column 21, and station number "3" in column 22 of the same row.

We now use the same procedure to find the head loss through the same manhole on the other line (2-4). While manhole diameter (b) and outlet pipe diameter ($D_o$) are the same as before, this pipe enters the manhole at a different angle and a different (invert) elevation:

$$K_o = 0.1\frac{b}{D_o}(1 - \sin\theta) + 1.4\left(\frac{b}{D_o}\right)^{0.15}\sin\theta = 0.1\frac{4}{1.5}(1 - \sin 120^\circ) + 1.4\left(\frac{4}{1.5}\right)^{0.15}\sin 120^\circ = 1.440$$

- Apply $C_D$? → If $d_{mho}/D_o > 3.2$ → $4.4/1.5 = 2.9$ → No (1.0)
- Apply $C_d$? → If $d_{mho}/D_o < 3.2$ → Yes

  $$C_d = 0.5\left(\frac{d_{mho}}{D_o}\right)^{0.6} = 0.5\left(\frac{4.4}{1.5}\right)^{0.6} = 0.954$$

- Apply $C_Q$? → If one or more other inlet pipes at manhole does not qualify as plunging flow. This pipe plunges to pipe 2-3, and no other pipes enter this manhole. → No (1.0)
- Apply $C_p$? → This correction factor is applied only to account for the effects of plunging flow by another pipe on the subject pipe. No pipe plunges to this pipe. → No (1.0)
- Apply $C_B$? → Yes (always)

  Use **Table 1007** and linearly interpolate with $d_{mho}/D_o = 2.9$.

  $C_B = 0.841$.

$$K_{2\text{-}4} = K_o C_D C_d C_Q C_p C_B = 1.440 \cdot 1.0 \cdot 0.954 \cdot 1.0 \cdot 1.0 \cdot 0.841 = 1.155$$

and

$$h_{mh,2-4} = K\left(\frac{V_o^2}{2g}\right) = 1.155(0.50) = 0.58'$$

This value is entered in column 21 in the row directly below that containing the 0.12' value. Station "4" is entered in the same row, column 22.

Step 5: The estimated manhole losses on each line (each row) are then added to the upstream EGL (column 24) and HGL (column 25) to get the EGL and HGL at the upper end of the manhole. The larger pair governs:

$$EGL_{U/S,mh,2-3} = EGL_{U/S,pipe(1-2)} + h_{mh,2-3} = 4502.4'+0.12' = 4502.5'$$
$$HGL_{U/S,mh,2-3} = EGL_{U/S,mh,2-3} - H_{vo} = 4502.5'-0.50' = 4502.0'$$
$$EGL_{U/S,mh,2-4} = EGL_{U/S,pipe(1-2)} + h_{mh,2-4} = 4502.4'+0.58' = 4503.0'$$
$$HGL_{U/S,mh,2-4} = EGL_{U/S,mh,2-4} - H_{vo} = 4503.0'-0.50' = 4502.5'$$

The hydraulic and energy grade line elevations for line 2-4 are used for the manhole freeboard check – the design storm maximum EGL is 4503.0', and the manhole rim at Design Point 2 is at 4505.0'. The rim is above the EGL, so the design is acceptable to this point.

Step 6: We now move to analysis of the upper pipe reaches, starting with the pipe between Design Points 2 and 3. As before, fill in known and computed data in columns 1 through 9. Average velocity in the pipe depends on flow conditions, so we must determine outlet conditions. The downstream EGL for this pipe is the larger of the following:

$$EGL_{D/S,pipe(2-3)} = EGL_{mh(2)} = 4503.0'$$

For full flow $(d > 0.80D)$:

$$EGL_{D/S,pipe(2-3)} = Invert_{D/S,pipe(2-3)} + D + H_{v(2-3)} = 4497.5'+1.25'+0.37' = 4499.1'$$

For partial flow $(d < 0.80D)$:

$$EGL_{D/S,pipe(2-3)} = Invert_{D/S,pipe(2-3)} + d_n + H_{v(2-3)} = 4497.5'+0.95'+0.56' = 4499.0'$$

The first value, 4503.0', is entered in column 23. The downstream HGL, then, is 4503.0'-Hv = 4503.0-0.37 = 4502.6', which is above the crown of pipe 2-3. Therefore, the pipe will be assumed to flow full under outlet control. Average velocity under full flow is 4.89 fps, resulting in a velocity head of 0.37'. Friction slope is:

$$S_f = \left[\frac{Q_{avg} \cdot n}{0.46 \cdot D^{8/3}}\right]^2 = \left[\frac{6 \cdot 0.013}{0.46 \cdot 1.25^{8/3}}\right]^2 = 0.009$$

This results in pipe friction head loss of:

$$h_f = S_f \cdot L = 0.009 \cdot 50' = 0.44'$$

Step 7: Since there is no known incoming pipe to the manhole at Design Point 3, we will not apply the full energy loss method as before. Instead, we can assume that the outlet pipe from the manhole will act as a culvert, with inlet losses as calculated below:

$$h_i = K_i\left(\frac{V^2}{2g}\right) = 0.5(0.37) = 0.19'$$

The value for $K_i$ was taken from **Table 1201**, assuming a square-edged headwall on concrete pipe. The value of $h_i$ is entered in column 18, and the sum of $h_f$ and $h_i$ is entered in column 20. This is then added to the downstream EGL value in column 23 to find the upstream EGL elevation (column 24):

$$EGL_{U/S,pipe} = EGL_{D/S,mh} + \Sigma h_{L,pipe} = 4503.0' + 0.44' + 0.19' = 4503.6'$$
$$HGL_{U/S,pipe} = EGL_{U/S,pipe} - H_v = 4503.6' - 0.37' = 4503.3'$$

The manhole rim elevation of 4505.0' is above the EGL of 4503.6', so this reach is acceptable.

Step 8: In a new row, enter stations "2" and "4" in columns 1 and 2. Enter data in columns 1 through 9. The full-pipe gravity-flow discharge for this reach is 3.56 cfs per Manning's equation, so pressure-flow conditions must exist to convey the 4 cfs design flow. However, there is a no-access junction 40 feet from Design Point 2 to which 1 cfs of the total 4 cfs is attributed. Above this junction, the main pipe is carrying 3 cfs under gravity-flow conditions. The following table organizes the computation of average values for the reach:

| Reach | Length (ft) | Flow (cfs) | Flow Area (sf) | Velocity (fps) |
|---|---|---|---|---|
| Downstream | 40 | 4 | 0.79 | 5.06 |
| Upstream | 30 | 3 | 0.59 | 5.08 |

Length-weighted averages for flow and velocity are needed for head loss calculations:

$$Q_{avg} = Q_{D/S}\frac{L_{D/S}}{L_{total}} + Q_{U/S}\frac{L_{U/S}}{L_{total}} = 4\frac{40}{70} + 3\frac{30}{70} = 3.57\text{cfs}$$
$$V_{avg} = V_{D/S}\frac{L_{D/S}}{L_{total}} + V_{U/S}\frac{L_{U/S}}{L_{total}} = 5.06\frac{40}{70} + 5.08\frac{30}{70} = 5.07\text{fps}$$

These values are entered in columns 9 and 10, respectively. Velocity head and friction slope (columns 11 and 12) are based on these averages:

$$H_v = \frac{V^2}{2g} = \frac{5.07^2}{2g} = 0.40'$$

$$S_f = \left[\frac{Q_{avg} \cdot n}{0.46 \cdot D^{8/3}}\right]^2 = \left[\frac{3.57 \cdot 0.013}{0.46 \cdot 1.0^{8/3}}\right]^2 = 0.010$$

Step 9: Determine the friction and minor pipe losses. Friction head loss (column 13) is:

$$h_f = S_f \cdot L = 0.010 \cdot 70' = 0.71'$$

Head loss at the no-access junction (column 17) is calculated as:

$$h_j = \frac{(Q_o V_o) - (Q_i V_i) - (Q_L V_L \cos\theta)}{0.5 \cdot g \cdot (A_o + A_i)} + \frac{V_i^2}{2g} - \frac{V_o^2}{2g}$$

$$h_j = \frac{(4 \cdot 5.06) - (3 \cdot 5.08) - (1 \cdot 6.40 \cos 120°)}{0.5 \cdot g \cdot (0.79 + 0.59)} + \frac{5.08^2}{2g} - \frac{5.06_i^2}{2g} = 0.37'$$

Like the manhole at Design Point 3, we will treat the outlet pipe from this manhole as a culvert inlet with a square-edged headwall (column 18):

$$h_i = K_i\left(\frac{V^2}{2g}\right) = 0.5(0.40) = 0.20'$$

Total pipe losses (column 20), then, are:

$$\Sigma h_{L,pipe(2-4)} = h_f + h_j + h_i = 0.71' + 0.37' + 0.20' = 1.28'$$

Step 10: Find the downstream and upstream EGLs and HGLs.

Downstream EGL (column 23) is the larger of:

$$EGL_{D/S,pipe(2-4)} = EGL_{mh(2)} = 4503.0'$$

$$EGL_{D/S,pipe(2-4)} = Invert_{D/S,pipe(2-4)} + D + H_{v(2-4)} = 4503.0' + 1.0' + 0.40' = 4504.4'$$

Upstream EGL (column 24) is:

$$EGL_{U/S,pipe(2-4)} = EGL_{D/S,pipe(2-4)} + \Sigma h_{L,pipe(2-4)} = 4504.4' + 1.28' = 4505.7'$$

$$HGL_{U/S,pipe(2-4)} = EGL_{U/S,pipe(2-4)} - H_v = 4505.7' - 0.40' = 4505.3'$$

The manhole rim elevation of 4507.0' is above the EGL of 4505.7', so this reach is acceptable.

# STORMWATER MANAGEMENT MANUAL

## SLOPES REQUIRED FOR V=2.5 FPS AT FULL AND HALF-FULL FLOW

| Pipe Diameter (Inches) | Required Slopes to Maintain Minimum Velocity (V = 2.5 fps) at Full and Half-Full Flow | | | | | |
|---|---|---|---|---|---|---|
| | n = 0.012 | n = 0.013 | n = 0.014 | n = 0.015 | n = 0.022 | n = 0.025 |
| 8 | 0.283% | 0.332% | 0.385% | 0.442% | 0.951% | 1.228% |
| 10 | 0.210% | 0.247% | 0.286% | 0.328% | 0.706% | 0.912% |
| 12 | 0.165% | 0.193% | 0.224% | 0.257% | 0.554% | 0.715% |
| 15 | 0.122% | 0.144% | 0.167% | 0.191% | 0.411% | 0.531% |
| | | | | | | |
| 18 | 0.096% | 0.113% | 0.131% | 0.150% | 0.322% | 0.416% |
| 21 | 0.078% | 0.092% | 0.106% | 0.122% | 0.263% | 0.339% |
| 24 | 0.065% | 0.077% | 0.089% | 0.102% | 0.220% | 0.284% |
| 27 | 0.056% | 0.066% | 0.076% | 0.087% | 0.188% | 0.243% |
| 30 | 0.049% | 0.057% | 0.066% | 0.076% | 0.163% | 0.211% |
| | | | | | | |
| 33 | 0.043% | 0.050% | 0.058% | 0.067% | 0.144% | 0.186% |
| 36 | 0.038% | 0.045% | 0.052% | 0.059% | 0.128% | 0.165% |
| 42 | 0.031% | 0.036% | 0.042% | 0.048% | 0.104% | 0.135% |
| 48 | 0.026% | 0.030% | 0.035% | 0.041% | 0.087% | 0.113% |
| 54 | 0.022% | 0.026% | 0.030% | 0.035% | 0.075% | 0.096% |
| | | | | | | |
| 60 | 0.019% | 0.023% | 0.026% | 0.030% | 0.065% | 0.084% |
| 66 | 0.017% | 0.020% | 0.023% | 0.027% | 0.057% | 0.074% |
| 72 | 0.015% | 0.018% | 0.021% | 0.024% | 0.051% | 0.066% |
| 78 | 0.014% | 0.016% | 0.018% | 0.021% | 0.046% | 0.059% |
| 84 | 0.012% | 0.014% | 0.017% | 0.019% | 0.041% | 0.053% |
| | | | | | | |
| 90 | 0.011% | 0.013% | 0.015% | 0.018% | 0.038% | 0.049% |
| 96 | 0.010% | 0.012% | 0.014% | 0.016% | 0.035% | 0.045% |
| 102 | 0.009% | 0.011% | 0.013% | 0.015% | 0.032% | 0.041% |
| 108 | 0.009% | 0.010% | 0.012% | 0.014% | 0.030% | 0.038% |
| 114 | 0.008% | 0.010% | 0.011% | 0.013% | 0.028% | 0.036% |
| | | | | | | |
| 120 | 0.008% | 0.009% | 0.010% | 0.012% | 0.026% | 0.033% |
| 126 | 0.007% | 0.008% | 0.010% | 0.011% | 0.024% | 0.031% |
| 132 | 0.007% | 0.008% | 0.009% | 0.011% | 0.023% | 0.029% |
| 138 | 0.006% | 0.007% | 0.009% | 0.010% | 0.021% | 0.028% |
| 144 | 0.006% | 0.007% | 0.008% | 0.009% | 0.020% | 0.026% |

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**TABLE 1001**

## MANNING'S ROUGHNESS COEFFICIENTS FOR STORM DRAIN CONDUITS

| Type of Conduit | Interior Wall Description | Manning's n |
|---|---|---|
| Concrete Pipes | Smooth | 0.011-0.013 |
| Concrete Boxes | | |
| Wood forms | Smooth | 0.012-0.014 |
| Steel forms | Smooth | 0.012-0.013 |
| Spiral-Rib Metal Pipes | Smooth | 0.012-0.013 |
| Corrugated Metal Pipes and Boxes | | |
| Annular Corrugations | 68mm x 13mm (2-2/3" x 1/2") corrugations | 0.022-0.027 |
| Helical Corrugations | 68mm x 13mm (2-2/3" x 1/2") corrugations | 0.011-0.023 |
| | 150mm x 25mm (6" x 1") corrugations | 0.022-0.025 |
| | 125mm x 25mm (5" x 1") corrugations | 0.025-0.026 |
| | 75mm x 25mm (3" x 1") corrugations | 0.027-0.028 |
| Structural Plate Corrugations | 230mm x 64mm (9" x 2-1/2") corrugations | 0.033-0.037 |
| | 150mm x 50mm (6" x 2") corrugations | 0.033-0.035 |
| Corrugated Polyethylene (HDPE) | Smooth | 0.009-0.015 |
| | Corrugated | 0.018-0.025 |
| Polyvinyl Chloride (PVC) | Smooth | 0.009-0.012 |
| Cast-Iron Pipe, uncoated | | 0.013 |
| Steel Pipe | | 0.009-0.013 |
| Vitrified Clay Pipe | | 0.012-0.014 |
| Vitrified Clay Liner Plates | | 0.015 |
| Cemented Rubble Masonry Walls | | |
| Concrete Floor and Top | | 0.017-0.022 |
| Natural Floor | | 0.019-0.025 |
| Brick | | 0.014-0.017 |
| Laminated Treated Wood | | 0.015-0.017 |

Note: The Manning's n values indicated in this table were obtained in the laboratory and are supported by the provided references. Actual field values for older existing pipelines may vary depending on the effects of abrasion, corrosion, deflection, and joint conditions. (HDS-4) The designer should take into account these potential issues when selecting roughness values.

| Revision | Date |
|---|---|
| ORIGINAL ISSUE | 3/27/06 |
| | |
| | |

# STORMWATER MANAGEMENT MANUAL

## TRANSITION COEFFICIENTS FOR GRAVITY-FLOW CONDITIONS

TABLE 1008a – VALUES OF $K_e$ FOR GRAVITY FLOW IN PIPE EXPANSIONS (FHWA 1996)

| $D_2/D_1$ | Angle of Cone | | | | | | |
|---|---|---|---|---|---|---|---|
| | 10° | 20° | 45° | 60° | 90° | 120° | 180° |
| 1.5 | 0.17 | 0.40 | 1.06 | 1.21 | 1.14 | 1.07 | 1.00 |
| 3 | 0.17 | 0.40 | 0.86 | 1.02 | 1.06 | 1.04 | 1.00 |

TABLE 1008b – VALUES OF $K_c$ FOR GRAVITY FLOW IN SUDDEN PIPE EXPANSIONS (LINSLEY AND FRANZINI 1964)

| $D_2/D_1$ | $K_c$ |
|---|---|
| 0.2 | 0.5 |
| 0.4 | 0.4 |
| 0.6 | 0.3 |
| 0.8 | 0.1 |
| 1 | 0 |

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|---|---|
| ORIGINAL ISSUE | 3/27/06 |
| | |
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**TABLE 1008**

## TRANSITION COEFFICIENTS FOR PRESSURE-FLOW CONDITIONS

### TABLE 1009a – VALUES OF $K_e$ FOR PRESSURE FLOW IN GRADUAL PIPE EXPANSIONS

| $D_2/D_1$ | Angle of Cone | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | 2° | 6° | 10° | 15° | 20° | 25° | 30° | 35° | 40° | 50° | 60° |
| 1.1 | 0.01 | 0.01 | 0.03 | 0.05 | 0.10 | 0.13 | 0.16 | 0.18 | 0.19 | 0.21 | 0.23 |
| 1.2 | 0.02 | 0.02 | 0.04 | 0.09 | 0.16 | 0.21 | 0.25 | 0.29 | 0.31 | 0.35 | 0.37 |
| 1.4 | 0.02 | 0.03 | 0.06 | 0.12 | 0.23 | 0.30 | 0.36 | 0.41 | 0.44 | 0.50 | 0.53 |
| 1.6 | 0.03 | 0.04 | 0.07 | 0.14 | 0.26 | 0.35 | 0.42 | 0.47 | 0.51 | 0.57 | 0.61 |
| 1.8 | 0.03 | 0.04 | 0.07 | 0.15 | 0.28 | 0.37 | 0.44 | 0.50 | 0.54 | 0.61 | 0.65 |
| 2.0 | 0.03 | 0.04 | 0.07 | 0.16 | 0.29 | 0.38 | 0.46 | 0.52 | 0.56 | 0.63 | 0.68 |
| 2.5 | 0.03 | 0.04 | 0.08 | 0.16 | 0.30 | 0.39 | 0.48 | 0.54 | 0.58 | 0.65 | 0.70 |
| 3.0 | 0.03 | 0.04 | 0.08 | 0.16 | 0.31 | 0.40 | 0.48 | 0.55 | 0.59 | 0.66 | 0.71 |
| inf | 0.03 | 0.05 | 0.08 | 0.16 | 0.31 | 0.40 | 0.49 | 0.46 | 0.60 | 0.67 | 0.72 |

### TABLE 1009b – VALUES OF $K_e$ FOR PRESSURE FLOW IN SUDDEN PIPE EXPANSIONS

| $D_2/D_1$ | Velocity, $V_1$, in feet Per Second | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | 2.0 | 3.0 | 4.0 | 5.0 | 6.0 | 7.0 | 8.0 | 10.0 | 12.0 | 15.0 | 20.0 | 30.0 | 40.0 |
| 1.2 | 0.11 | 0.10 | 0.10 | 0.10 | 0.10 | 0.10 | 0.10 | 0.09 | 0.09 | 0.09 | 0.09 | 0.09 | 0.08 |
| 1.4 | 0.26 | 0.26 | 0.25 | 0.24 | 0.24 | 0.24 | 0.24 | 0.23 | 0.23 | 0.22 | 0.22 | 0.21 | 0.20 |
| 1.6 | 0.40 | 0.39 | 0.38 | 0.37 | 0.37 | 0.36 | 0.36 | 0.35 | 0.35 | 0.34 | 0.33 | 0.32 | 0.32 |
| 1.8 | 0.51 | 0.49 | 0.48 | 0.47 | 0.47 | 0.46 | 0.46 | 0.45 | 0.44 | 0.43 | 0.42 | 0.41 | 0.40 |
| 2.0 | 0.60 | 0.58 | 0.56 | 0.55 | 0.55 | 0.54 | 0.53 | 0.52 | 0.52 | 0.51 | 0.50 | 0.48 | 0.47 |
| 2.5 | 0.74 | 0.72 | 0.70 | 0.69 | 0.68 | 0.67 | 0.66 | 0.65 | 0.64 | 0.63 | 0.62 | 0.60 | 0.58 |
| 3.0 | 0.83 | 0.80 | 0.78 | 0.77 | 0.76 | 0.75 | 0.74 | 0.73 | 0.72 | 0.70 | 0.69 | 0.67 | 0.65 |
| 4.0 | 0.92 | 0.89 | 0.87 | 0.85 | 0.84 | 0.83 | 0.82 | 0.80 | 0.79 | 0.78 | 0.76 | 0.74 | 0.72 |
| 5.0 | 0.96 | 0.93 | 0.91 | 0.89 | 0.88 | 0.87 | 0.86 | 0.84 | 0.83 | 0.82 | 0.80 | 0.77 | 0.75 |
| 10.0 | 1.00 | 0.99 | 0.96 | 0.95 | 0.93 | 0.92 | 0.91 | 0.89 | 0.88 | 0.86 | 0.84 | 0.82 | 0.80 |
| ∞ | 1.00 | 1.00 | 0.98 | 0.96 | 0.95 | 0.94 | 0.93 | 0.91 | 0.90 | 0.88 | 0.86 | 0.83 | 0.81 |

| $D_2/D_1$ | Velocity, $V_1$, in Meters Per Second | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | 0.6 | 0.9 | 1.2 | 1.5 | 1.8 | 2.1 | 2.4 | 3.0 | 3.7 | 4.6 | 6.1 | 9.1 | 12.2 |
| 1.2 | 0.11 | 0.10 | 0.10 | 0.10 | 0.10 | 0.10 | 0.10 | 0.09 | 0.09 | 0.09 | 0.09 | 0.09 | 0.08 |
| 1.4 | 0.26 | 0.26 | 0.25 | 0.24 | 0.24 | 0.24 | 0.24 | 0.23 | 0.23 | 0.22 | 0.22 | 0.21 | 0.20 |
| 1.6 | 0.40 | 0.39 | 0.38 | 0.37 | 0.37 | 0.36 | 0.36 | 0.35 | 0.35 | 0.34 | 0.33 | 0.32 | 0.32 |
| 1.8 | 0.51 | 0.49 | 0.48 | 0.47 | 0.47 | 0.46 | 0.46 | 0.45 | 0.44 | 0.43 | 0.42 | 0.41 | 0.40 |
| 2.0 | 0.60 | 0.58 | 0.56 | 0.55 | 0.55 | 0.54 | 0.53 | 0.52 | 0.52 | 0.51 | 0.50 | 0.48 | 0.47 |
| 2.5 | 0.74 | 0.72 | 0.70 | 0.69 | 0.68 | 0.67 | 0.66 | 0.65 | 0.64 | 0.63 | 0.62 | 0.60 | 0.58 |
| 3.0 | 0.83 | 0.80 | 0.78 | 0.77 | 0.76 | 0.75 | 0.74 | 0.73 | 0.72 | 0.70 | 0.69 | 0.67 | 0.65 |
| 4.0 | 0.92 | 0.89 | 0.87 | 0.85 | 0.84 | 0.83 | 0.82 | 0.80 | 0.79 | 0.78 | 0.76 | 0.74 | 0.72 |
| 5.0 | 0.96 | 0.93 | 0.91 | 0.89 | 0.88 | 0.87 | 0.86 | 0.84 | 0.83 | 0.82 | 0.80 | 0.77 | 0.75 |
| 10.0 | 1.00 | 0.99 | 0.96 | 0.95 | 0.93 | 0.92 | 0.91 | 0.89 | 0.88 | 0.86 | 0.84 | 0.82 | 0.80 |
| ∞ | 1.00 | 1.00 | 0.98 | 0.96 | 0.95 | 0.94 | 0.93 | 0.91 | 0.90 | 0.88 | 0.86 | 0.83 | 0.81 |

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|---|---|
| ORIGINAL ISSUE | 3/27/06 |
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## TABLE 1009c — VALUES OF $K_c$ FOR PRESSURE FLOW IN GRADUAL PIPE CONTRACTIONS

| $D_2/D_1$ | Velocity, $V_1$, in feet Per Second | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | 2.0 | 3.0 | 4.0 | 5.0 | 6.0 | 7.0 | 8.0 | 10.0 | 12.0 | 15.0 | 20.0 | 30.0 | 40.0 |
| 1.1 | 0.03 | 0.04 | 0.04 | 0.04 | 0.04 | 0.04 | 0.04 | 0.04 | 0.04 | 0.04 | 0.05 | 0.05 | 0.06 |
| 1.2 | 0.07 | 0.07 | 0.07 | 0.07 | 0.07 | 0.07 | 0.07 | 0.08 | 0.08 | 0.08 | 0.09 | 0.10 | 0.11 |
| 1.4 | 0.17 | 0.17 | 0.17 | 0.17 | 0.17 | 0.17 | 0.17 | 0.18 | 0.18 | 0.18 | 0.18 | 0.19 | 0.20 |
| 1.6 | 0.26 | 0.26 | 0.26 | 0.26 | 0.26 | 0.26 | 0.26 | 0.26 | 0.26 | 0.26 | 0.25 | 0.25 | 0.24 |
| 1.8 | 0.34 | 0.34 | 0.34 | 0.34 | 0.34 | 0.34 | 0.33 | 0.33 | 0.32 | 0.32 | 0.32 | 0.29 | 0.27 |
| 2.0 | 0.38 | 0.38 | 0.37 | 0.37 | 0.37 | 0.37 | 0.36 | 0.36 | 0.35 | 0.34 | 0.33 | 0.31 | 0.29 |
| 2.2 | 0.40 | 0.40 | 0.40 | 0.39 | 0.39 | 0.39 | 0.39 | 0.38 | 0.37 | 0.37 | 0.35 | 0.33 | 0.30 |
| 2.5 | 0.42 | 0.42 | 0.42 | 0.41 | 0.41 | 0.41 | 0.40 | 0.40 | 0.39 | 0.38 | 0.37 | 0.34 | 0.31 |
| 3.0 | 0.44 | 0.44 | 0.44 | 0.43 | 0.43 | 0.43 | 0.42 | 0.42 | 0.41 | 0.40 | 0.39 | 0.36 | 0.33 |
| 4.0 | 0.47 | 0.46 | 0.46 | 0.46 | 0.45 | 0.45 | 0.45 | 0.44 | 0.43 | 0.42 | 0.41 | 0.37 | 0.34 |
| 5.0 | 0.48 | 0.48 | 0.47 | 0.47 | 0.47 | 0.46 | 0.46 | 0.45 | 0.45 | 0.44 | 0.42 | 0.38 | 0.35 |
| 10.0 | 0.49 | 0.48 | 0.48 | 0.48 | 0.48 | 0.47 | 0.47 | 0.46 | 0.46 | 0.45 | 0.43 | 0.40 | 0.36 |
| ∞ | 0.49 | 0.49 | 0.48 | 0.48 | 0.48 | 0.47 | 0.47 | 0.47 | 0.46 | 0.45 | 0.44 | 0.41 | 0.38 |

| $D_2/D_1$ | Velocity, $V_1$, in Meters Per Second | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | 0.6 | 0.9 | 1.2 | 1.5 | 1.8 | 2.1 | 2.4 | 3.0 | 3.7 | 4.6 | 6.1 | 9.1 | 12.2 |
| 1.1 | 0.03 | 0.04 | 0.04 | 0.04 | 0.04 | 0.04 | 0.04 | 0.04 | 0.04 | 0.04 | 0.05 | 0.05 | 0.06 |
| 1.2 | 0.07 | 0.07 | 0.07 | 0.07 | 0.07 | 0.07 | 0.07 | 0.08 | 0.08 | 0.08 | 0.09 | 0.10 | 0.11 |
| 1.4 | 0.17 | 0.17 | 0.17 | 0.17 | 0.17 | 0.17 | 0.17 | 0.18 | 0.18 | 0.18 | 0.18 | 0.19 | 0.20 |
| 1.6 | 0.26 | 0.26 | 0.26 | 0.26 | 0.26 | 0.26 | 0.26 | 0.26 | 0.26 | 0.26 | 0.25 | 0.25 | 0.24 |
| 1.8 | 0.34 | 0.34 | 0.34 | 0.34 | 0.34 | 0.34 | 0.33 | 0.33 | 0.32 | 0.32 | 0.32 | 0.29 | 0.27 |
| 2.0 | 0.38 | 0.38 | 0.37 | 0.37 | 0.37 | 0.37 | 0.36 | 0.36 | 0.35 | 0.34 | 0.33 | 0.31 | 0.29 |
| 2.2 | 0.40 | 0.40 | 0.40 | 0.39 | 0.39 | 0.39 | 0.39 | 0.38 | 0.37 | 0.37 | 0.35 | 0.33 | 0.30 |
| 2.5 | 0.42 | 0.42 | 0.42 | 0.41 | 0.41 | 0.41 | 0.40 | 0.40 | 0.39 | 0.38 | 0.37 | 0.34 | 0.31 |
| 3.0 | 0.44 | 0.44 | 0.44 | 0.43 | 0.43 | 0.43 | 0.42 | 0.42 | 0.41 | 0.40 | 0.39 | 0.36 | 0.33 |
| 4.0 | 0.47 | 0.46 | 0.46 | 0.46 | 0.45 | 0.45 | 0.45 | 0.44 | 0.43 | 0.42 | 0.41 | 0.37 | 0.34 |
| 5.0 | 0.48 | 0.48 | 0.47 | 0.47 | 0.47 | 0.46 | 0.46 | 0.45 | 0.45 | 0.44 | 0.42 | 0.38 | 0.35 |
| 10.0 | 0.49 | 0.48 | 0.48 | 0.48 | 0.48 | 0.47 | 0.47 | 0.46 | 0.46 | 0.45 | 0.43 | 0.40 | 0.36 |
| ∞ | 0.49 | 0.49 | 0.48 | 0.48 | 0.48 | 0.47 | 0.47 | 0.47 | 0.46 | 0.45 | 0.44 | 0.41 | 0.38 |

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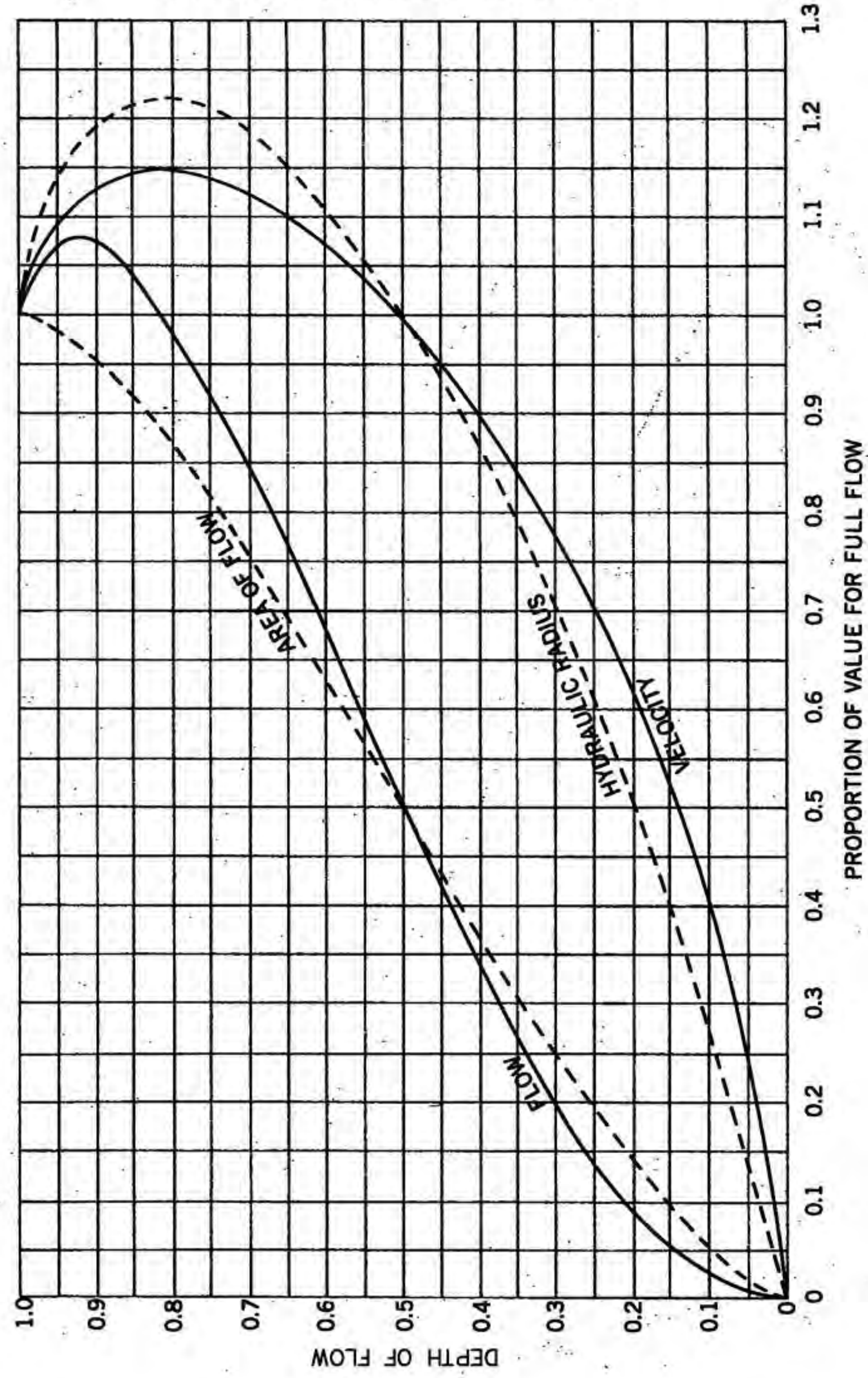
**RELATIVE VELOCITY AND FLOW IN CIRCULAR PIPE FOR ANY DEPTH OF FLOW**

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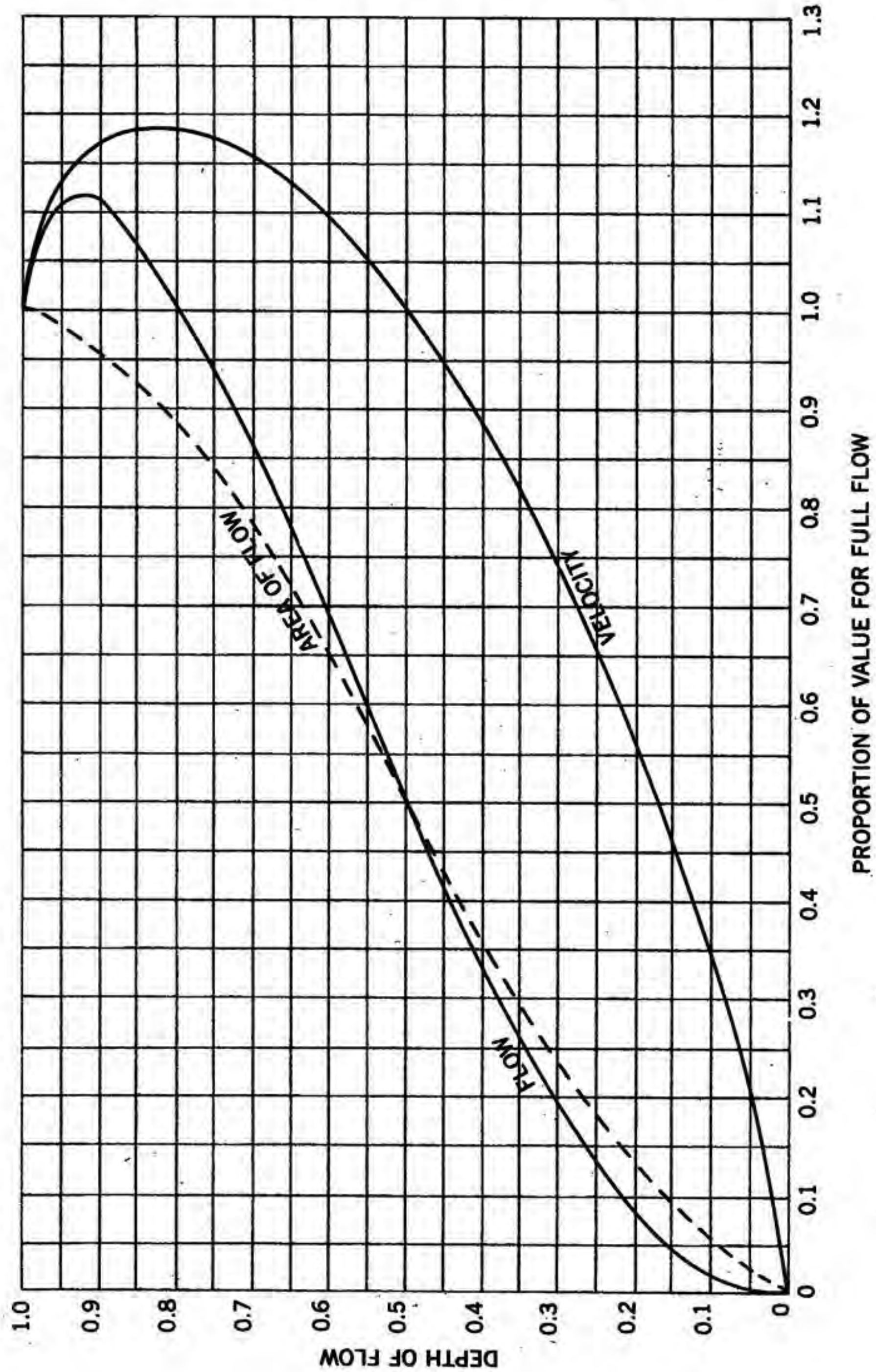
ACPA FIGURE 20

**FIGURE 1001**

# STORMWATER MANAGEMENT MANUAL

**RELATIVE VELOCITY AND FLOW IN**
**HORIZONTAL ELLIPTICAL PIPE FOR ANY DEPTH OF FLOW**

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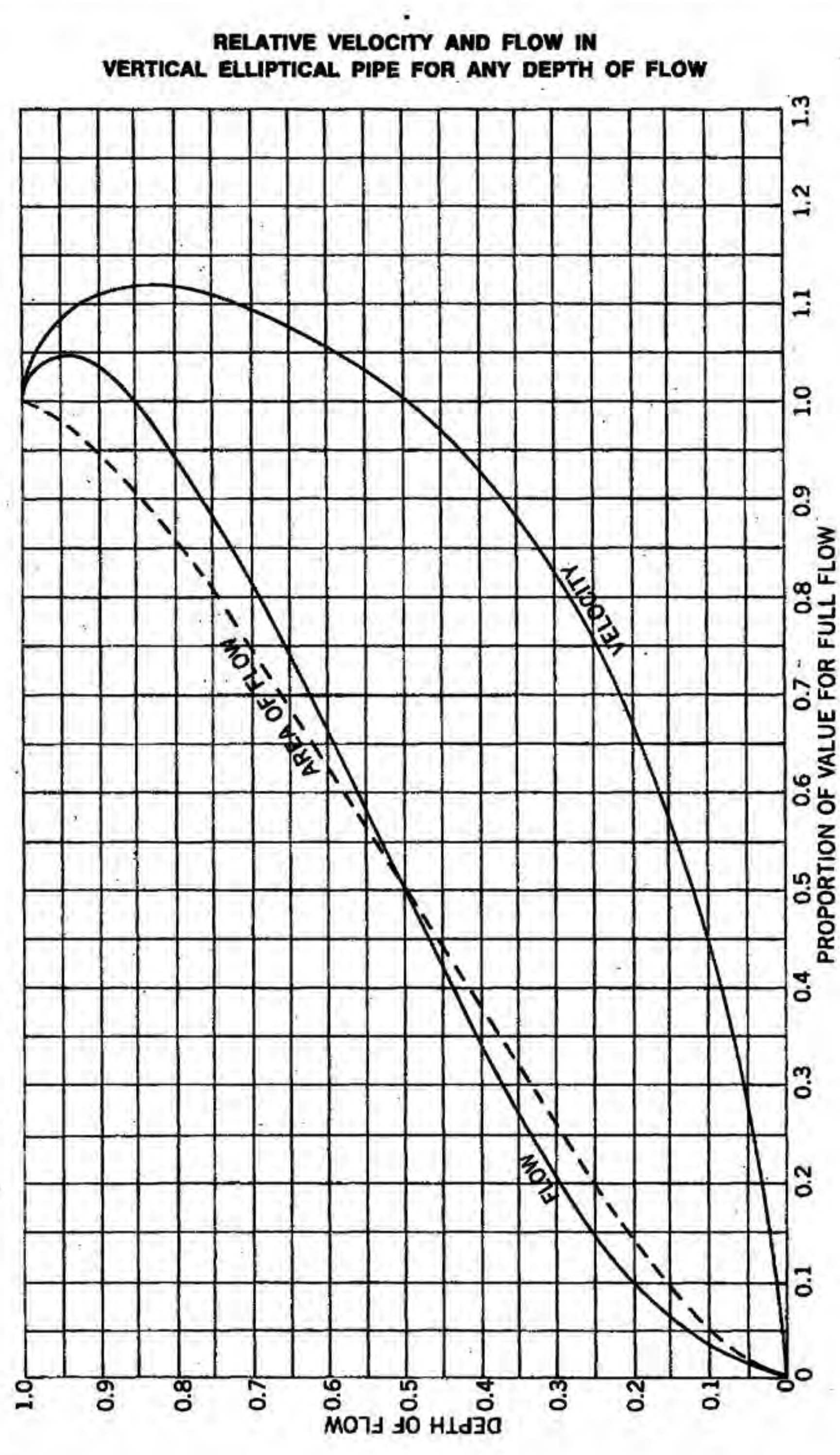
ACPA FIGURE 21

**FIGURE 1002**

# STORMWATER MANAGEMENT MANUAL

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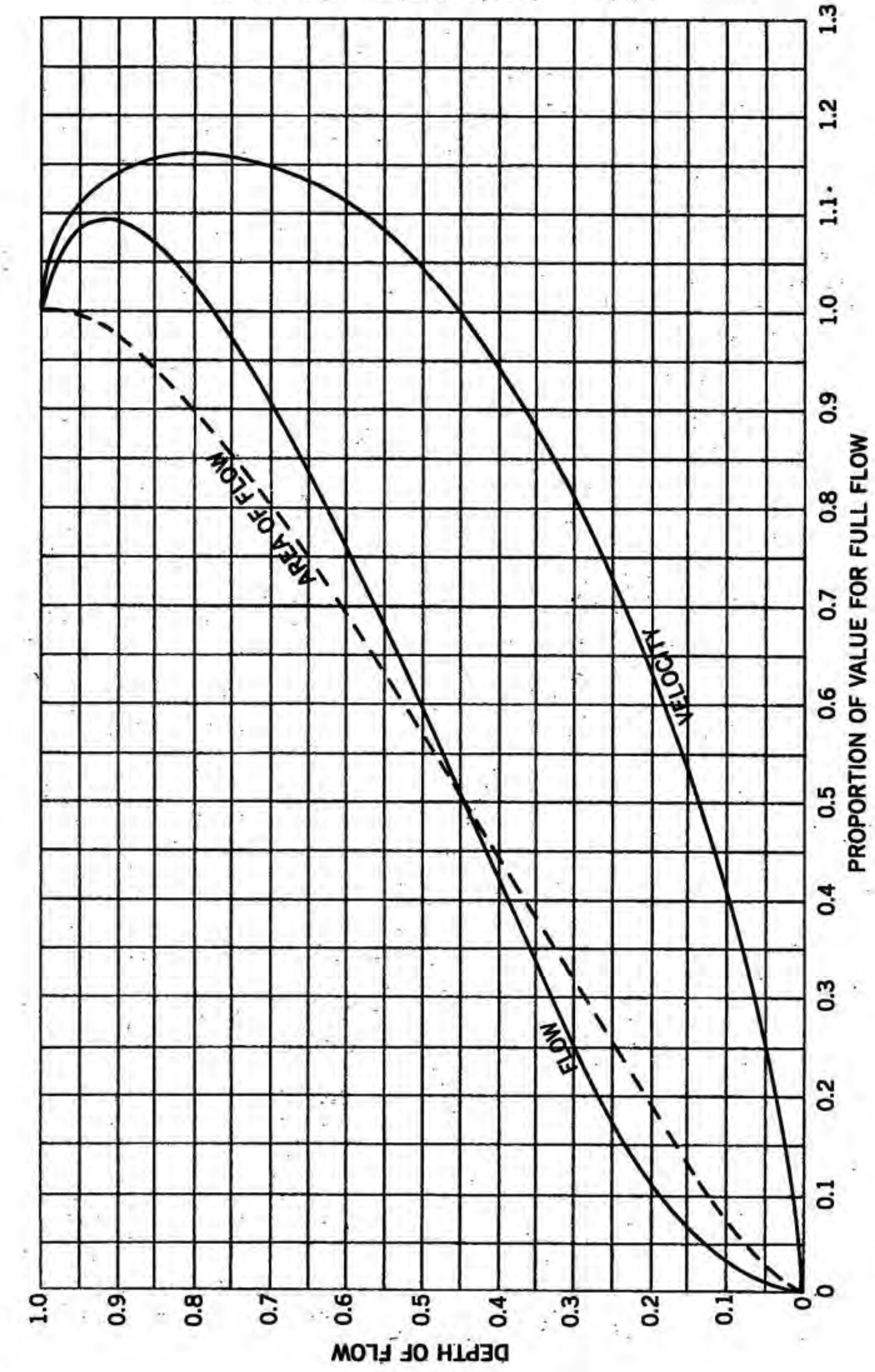
ACPA FIGURE 22

**FIGURE 1003**

**RELATIVE VELOCITY AND FLOW IN ARCH PIPE FOR ANY DEPTH OF FLOW**

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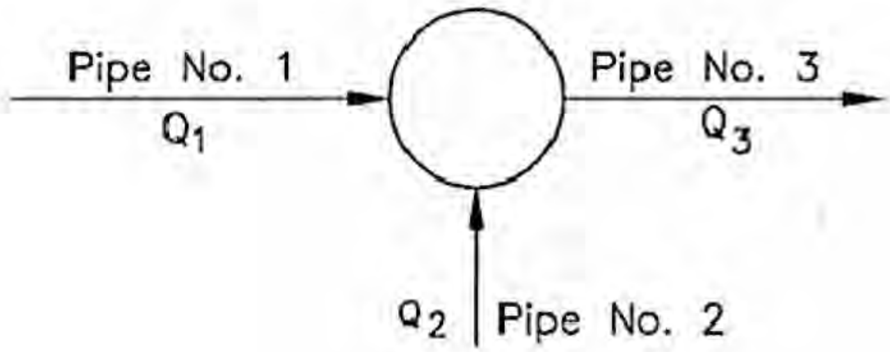
## RELATIVE FLOW EFFECT

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WRC ENGINEERING, INC.

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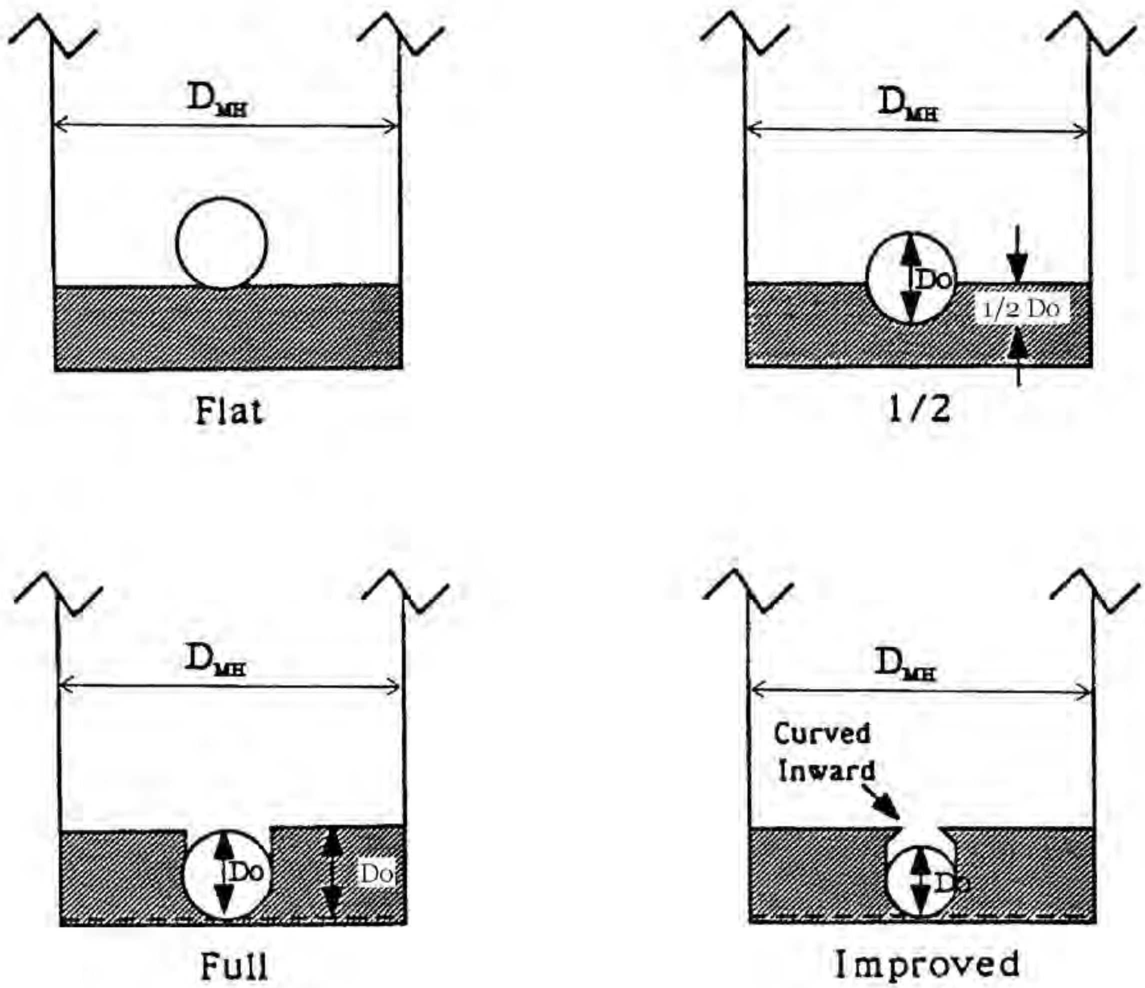
**FIGURE 1005**

## TYPES OF MANHOLE BENCHING




$D_{MH}$ = Diameter of Manhole, ft.
d = Depth of flow in manhole, taken at entrance of outflow pipe, ft.
Do = Diameter of outflow pipe, ft.

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| ORIGINAL ISSUE | 3/27/06 |
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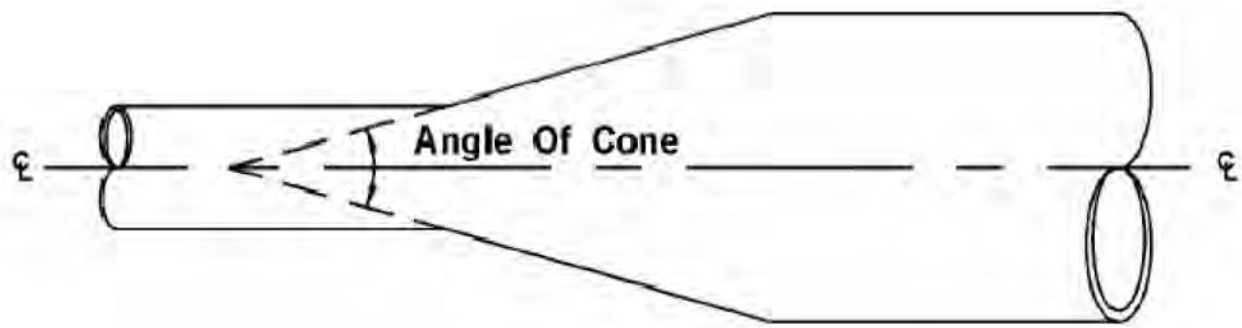
## ANGLE OF CONE FOR PIPE TRANSITIONS

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|---|---|
| ORIGINAL ISSUE | 3/27/06 |
| | |
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FROM HEC-22 FIGURE 7-3

**FIGURE 1007**

# STORMWATER MANAGEMENT MANUAL

## EXAMPLE PROBLEM: STORM DRAIN SCHEMATIC

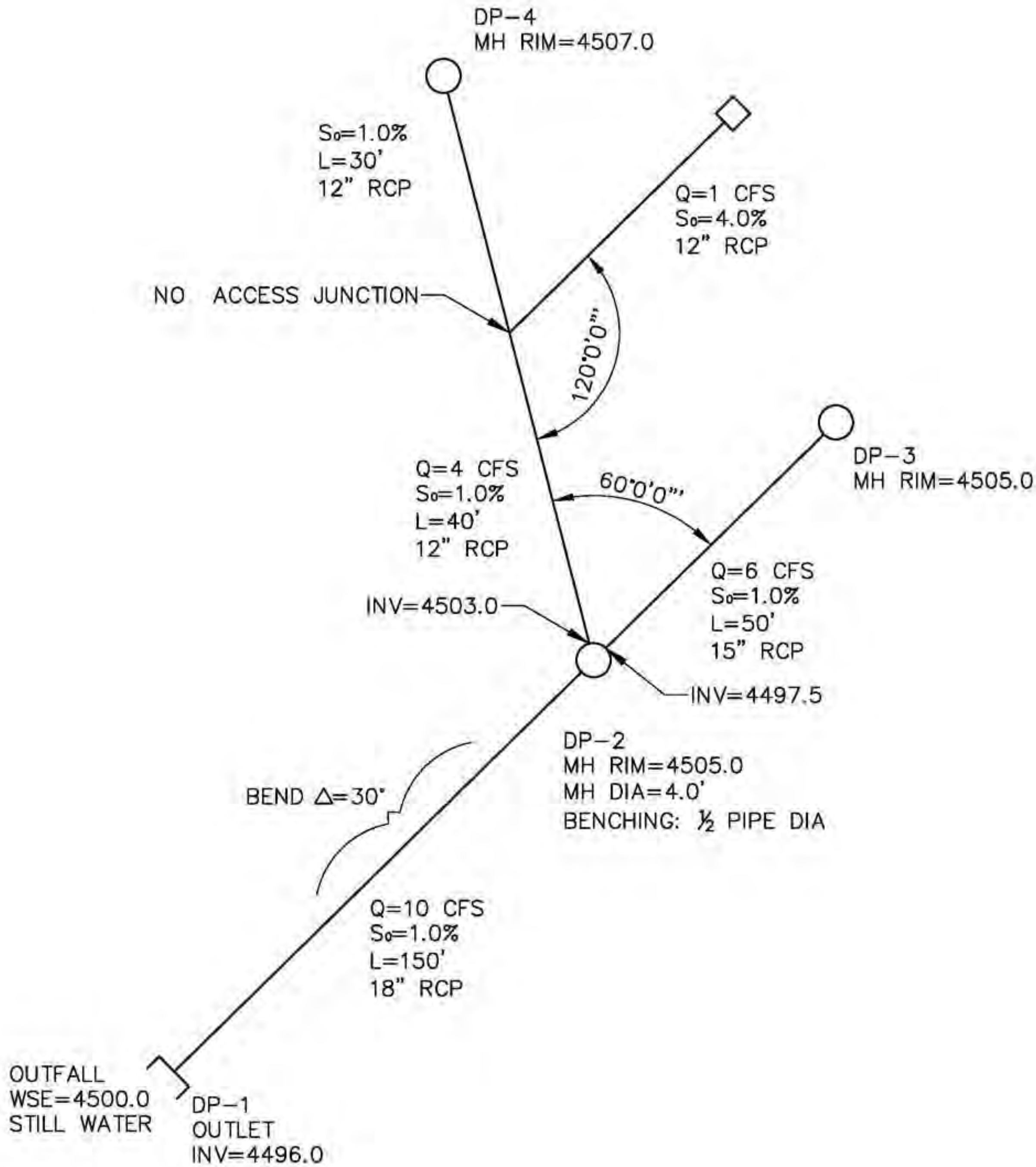


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**FIGURE 1008**

# STORMWATER MANAGEMENT MANUAL

## EXAMPLE PROBLEM: STORM DRAIN HYDRAULIC CALCULATIONS

| 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 11 | 12 |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | CONDUIT DATA | | | | | FLOW DATA | | | | |
| STATION | | SIZE / TYPE | INVERTS | | LENGTH | SLOPE | A (full) ($FT^2$) | Q (avg) (CFS) | V (avg) (FPS) | $H_V = V^2/2G$ (FT) | $S_f$ (avg) (FT/FT) |
| FROM | TO | | D/S ELEV. | U/S ELEV. | | | | | | | |
| OUTFALL | | POND | - | - | - | - | - | - | 0 | 0 | - |
| 1 | 2 | 18" RCP | 4496.0 | 4497.5 | 150 | 0.01 | 1.77 | 10.00 | 5.65 | 0.50 | 0.009 |
| 2 | 3 | 15" RCP | 4497.5 | 4498.0 | 50 | 0.01 | 1.23 | 6.00 | 4.89 | 0.37 | 0.009 |
| 2 | 4 | 12" RCP | 4503.0 | 4503.7 | 70 | 0.01 | 0.79 | 3.57 | 5.07 | 0.40 | 0.010 |

| MANHOLE HYDRAULICS | | | | | | |
|---|---|---|---|---|---|---|
| Manhole at Design Point | Next U/S Design Point | Initial $h_L$ coefficient $K_o$ | $C_D$ or $C_d$ | $C_Q$ or $C_p$ | $C_B$ | Adjusted $h_L$ coefficient K |
| 2 | 3 | 0.267 | 0.954 | 1.148 | 0.841 | 0.246 |
| 2 | 4 | 1.44 | 0.954 | 1 | 0.841 | 1.155 |

| 1 | 2 | 13 | 14 | 15 | 16 | 17 | 18 | 19 | 20 | 21 | 22 | 23 | 24 | 25 | 26 | 27 | 28 | 29 | 30 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | ENERGY LOSSES | | | | | | | | | | ENERGY / HYDRAULIC GRADE LINE ELEVATIONS | | | | | NOTES | | |
| STATION | | PIPE | MINOR PIPE LOSSES | | | | | | $\Sigma h_L$ (PIPE) (FT) | U/S MH | | PIPE | | | MANHOLE (U/S) | | PRESSURE FLOW? (Y/N) | MH RIM/INLET GRATE ELEV. | COMMENTS |
| FROM | TO | $h_f$ (FT) | $h_e$ (FT) | $h_c$ (FT) | $h_b$ (FT) | $h_j$ (FT) | $h_i$ (FT) | $h_o$ (FT) | | $h_{mh}$ (THIS LINE) (FT) | NEXT U/S STATION | D/S EGL ELEV. | U/S EGL ELEV. | U/S HGL ELEV. | U/S EGL ELEV. | U/S HGL ELEV. | | | |
| OUTFALL | | - | - | - | - | - | - | 0.5 | 0.5 | - | - | 4500.0 | 4500.5 | 4500.0 | - | - | - | - | |
| 1 | 2 | 1.38 | - | - | 0.50 | - | - | - | 1.88 | 0.12 | 3 | 4500.5 | 4502.4 | 4501.9 | 4502.5 | 4502.0 | YES | 4505.0 | FB = 2.0, OK |
| | | | | | | | | | | 0.58 | 4 | | | | 4503.0 | 4502.5 | | | |
| 2 | 3 | 0.44 | - | - | - | - | 0.19 | - | 0.62 | - | - | 4503.0 | 4503.6 | 4503.3 | - | - | YES | 4505.0 | FB = 1.7, OK |
| 2 | 4 | 0.71 | - | - | - | 0.37 | 0.20 | - | 1.28 | - | - | 4504.4 | 4505.7 | 4505.3 | - | - | MULTI | 4507.0 | FB = 1.7, OK |

| Revision | Date |
|---|---|
| ORIGINAL ISSUE | 3/27/06 |

# SECTION 1100

## STREETS

# SECTION 1100
## STREETS

## TABLE OF CONTENTS



## LIST OF TABLES



## LIST OF FIGURES

# SECTION 1100
# STREETS

## 1101 INTRODUCTION

This Section presents criteria regulating the design of public streets for storm flow drainage and allowable encroachment of these flows within public street rights-of-way. Also included herein are criteria for the selection and placement of storm drain inlets as required to conform to maximum allowable street ponding. All design submittals involving the utilization of public streets for the conveyance of storm flows shall be reviewed based upon the criteria in this Section.

## 1102 STREETS AS PART OF THE DRAINAGE SYSTEM

The primary function of public streets is the movement of traffic. Therefore, use of streets as part of the drainage system must be limited in order to minimize interference with traffic functions. Street inundation and ponding limits are specified in this Section in order to limit this potential interference. However, due to the nature of the storms typical to Mesa County, and the resulting ephemeral interference with traffic movement, these limits are not highly restrictive.

Streets typically convey runoff collected not only on the street surface itself, but often from some portion of the surrounding area as well. As an integral part of the drainage system, streets must be capable of conveying some portion of said runoff to a primary drainage conveyance facility such as a storm drain or open channel drainage system. The maximum allowable capacity of a street is based upon its cross-sectional configuration, longitudinal slope, and the maximum allowed ponding depth. Where the calculated minor storm event flow depth exceeds this maximum depth, inlets must be used to reduce street flow. During a major storm event, streets become emergency runoff channels, routing floodwaters away from structures as much as possible. During such an event, many streets will be inundated to a degree such that they are impassable to most vehicles.

## 1103 DRAINAGE IMPACTS ON TRAFFIC AND STREETS

A number of factors may affect traffic movement and the streets themselves when used for drainage purposes. Five of these factors are discussed in the following sections.

### 1103.1 Sheet Flow

Rainfall on the paved surface of a street must flow overland (sheet flow) until it reaches a channel. In this case, the channel will be either a curb-and-gutter or a roadside ditch. Ignoring the effects of street inundation due to upgradient runoff, the depth of sheet flow will be near zero at the street crown, increasing in the direction of the collection channel (i.e., the gutter).

Sheet flow can interfere with traffic movement by increasing the risk of hydroplaning or splashing. Hydroplaning is a phenomenon in which one or more of a vehicle's tires lose significant contact with the pavement and become supported by a thin layer of water. This water acts as a lubricant between the pavement and the vehicle, and can cause loss of vehicle control. The potential for hydroplaning increases with vehicle speed and with the depth of water on the road surface, so extra attention shall be paid to this issue during the design of arterial and highway-type roadways. Sheet flow depths can be decreased by increasing the street cross-slope.

Splashing is also dependent on vehicle speeds and water depth, and can interfere with traffic movement by reducing driver visibility. Again, increasing street cross-slope may help to reduce sheet flow depths and splashing potential. In general, a 2% cross slope is desirable to promote swift removal of runoff and reduce sheet flow depths while minimizing potential vehicle side-slippage from ice buildup during winter months.

### 1103.2 Gutter Flow

Where curb-and-gutter is utilized, all runoff tributary to a street will be directed to, and flow in, a gutter until it reaches a storm drain inlet or other primary conveyance. Generally, flow width will increase in the downstream direction due to an ever-increasing tributary area (and thus higher flows), eventually spreading into the traffic lane(s) and to the street crown. It is important to take into account the gutter flow width and its effect upon traffic movement. For streets where on-street parking is allowed (e.g., residential streets), gutter flow may spread at least to the inner edge of the parking lane without interfering with moving traffic. However, where no street parking is allowed, flow spreading beyond the actual gutter almost immediately reaches a travel lane. Drivers tend to avoid even partially inundated lanes if possible, causing traffic congestion and occasionally vehicular collisions. However, some interference with traffic movement is allowable and expected, even during the minor storm. The duration of such interference is to be minimized wherever possible. Emergency vehicles must be able to travel all streets without encountering extensive (potentially vehicle-disabling) ponding depths. These depths are defined in Section 1104 as maximum total gutter depths for the minor storm event.

### 1103.3 Temporary Ponding and Localized Cross Flow

Grade changes and street crowns at intersections can cause storm runoff to flow at depths greater than the intended design depth and to flow at greater depths for longer durations than anticipated. This localized, temporary ponding poses a high safety risk, especially on roads with higher speed limits (e.g., arterials). Because the ponding is localized, drivers may not be aware of the obstruction and vehicles may enter the ponded area at high speeds (see reference to hydroplaning, 1103.1). Even if traffic is moving at low speeds, vehicles encountering a deeper ponded area may stop entirely to avoid entering such an area or may experience engine stalling while attempting to cross such an area. Both of these scenarios have the effect of reducing or halting traffic movement. Therefore, street and drainage design must incorporate measures to control the depths at locations where temporary ponding may occur through the use of grading changes or additional inlets. For streets with high traffic volumes, it may be necessary to take steps to effectively eliminate any temporary ponding in order to avoid extensive interference with traffic movement.

Cross flow occurs as a result of a number of scenarios that cause significant flow depths other than that which is the result of sheet flow to travel across traffic lanes. The most common location to encounter this phenomenon is at intersections where gutter flow from the cross-street spills across the intersection. Where allowable, a concrete V-pan may be utilized to control these flows (see City of Grand Junction Standard Details for Construction of Streets, Storm Drains and Utilities, Revised July 2005, page C-12). Localized cross flow can have the same negative effects on vehicular safety and traffic movement as general temporary ponding. Therefore, careful consideration shall be given to potential cross flow locations during design to minimize its effects, especially for streets with higher speed limits and/or higher traffic volume.

### 1103.4 Pavement Deterioration Effects

Utilization of streets for drainage of storm runoff can have a significant effect on roadway maintenance and repair and can, in certain cases, contribute to the structural failure of the roadway pavement. The latter situation may occur if water is able to penetrate the pavement and reach the subgrade. Subgrade saturation and/or material washout will eventually cause subgrade failure, leading to pavement failure and an unsafe/unstable road surface. These locations of distress are caused by other factors such as weathering, overweight vehicles, temperature and temperature changes, heavy traffic, machinery, pavement quality, subgrade inconsistencies, and pavement age. Water flow across undamaged pavement will not typically penetrate to the subgrade. Therefore, consistent roadway maintenance not only affects the efficiency of the street as a drainage system component, but can also reduce the likelihood of roadway failure due to the effects of ponding water.

Although undamaged pavement surfaces for the most part keep water from penetrating through to the subgrade, it is possible for some water to seep through with time. Street designs shall incorporate measures to decrease the duration for which a given section of pavement is submerged to minimize this seepage.

A common practice used to reduce the problem of bituminous surface deterioration is to seal-coat or overlay the existing pavement surface. While this method does effectively reduce pavement deterioration, it reduces the available street flow area with each layer added to the surface. To minimize this effect, it is recommended that the surface be scarified whenever possible before new layers of pavement are added.

### 1103.5 Sedimentation and Debris

Sediment and debris buildup may occur on streets in any area where flow velocities tend to decrease such as near grade changes and inlets. Sediment and debris buildup can have a significant impact on the flow capacities of gutters and streets, causing increased flow width and thus increased interference with traffic movement.

Locations where significant deposits may occur shall be identified for maintenance purposes to include street sweeping and inlet cleanout as necessary. Inlets shall be designed to function properly based on expected sediment and debris clogging as specified later in this Section.

Localized sedimentation issues due to construction activities shall be controlled per the criteria presented in Section 1500 of this Manual.

## 1104 STREET CLASSIFICATION AND ALLOWABLE FLOW DEPTH AND VELOCITIES

Streets are classified according to estimated traffic volume and right-of-way width. The City of Grand Junction and Mesa County have adopted the Transportation, Engineering, and Design Standards (TEDS) manual, including standard drawings and details for the construction of streets and location for utilities. While these drawings and details were developed and are maintained by the Grand Junction Department of Public Works and Utilities (Engineering Division), they have been adopted by the Mesa County Board of Commission (TEDS Chapter 1, p.1). Eight street classifications are specified therein, but some of these have been grouped together in this Manual due to similar hydraulic characteristics. The Rural Roadway street type is not presented in this Section since it utilizes a roadside ditch and culverts instead of curb-and-gutter. See Section 800 of this Manual for open channel design criteria.

The local jurisdictions within Mesa County have adopted the policy that streets can be used to convey storm runoff subject to the limits established by the Manual. For the minor storm event:

1. Flow depths must not exceed 0.5 feet at the gutter flowline,
2. Must not exceed curb height (varies) AND
3. Flow velocities must not exceed 8.0 feet per second.

Street flow depth from the major storm event must not exceed 1.0 feet from the gutter flowline and the flow velocity must not exceed 8.0 feet per second.

**Figures 1101** and **1102** show typical half-street sections and 2-year storm event inundation limits for each street classification group. Note that the cross-sections shown include geometric assumptions made for hydraulic computation purposes and do not precisely reflect the approved Standard Street Details.

Calculations for flow capacity and velocity in a given street section are based upon these maximum depths and the assumption that any area not within the street right-of-way would not contribute to the capacity of the street system (called "ineffective flow area"). Therefore, for calculation purposes, it is assumed that an infinitely high vertical wall of zero roughness exists at the right-of-way boundary, and any flow area outside this boundary is not considered in analysis. Due to the potential for a single street cross-section to have different half-street cross-sections, all street capacity calculations are to be completed on a half-street basis. Therefore, the same vertical-wall assumption applies to the street centerline as to the right-of-way where the calculated flow width exceeds the half-street width.

At street sag locations, pipes and/or channels must be provided to facilitate compliance with maximum ponding depths for both minor and major storm events as described above. Maintenance access must be provided for these facilities, including permanent easements.

Not inclusive of median spill gutters, the Standard Concrete Details include two curb types, vertical and drive-over (mountable). The vertical curb configuration may be constructed monolithically with or without a sidewalk, and has a height of 6 inches. This configuration or an approved alternative design must be used for streets with an A.D.T. value of more than 1000 (Standard Street Details, Page ST-05). A cross-section detail of the vertical curb and gutter is found in **Figure 1103**. The drive-over, or mountable, curb configuration may only be used for residential streets with an A.D.T. value of less than 1000 (Standard Street Details, Page ST-05). The total vertical height from the gutter flowline to the "top" of the curb is 4.5 inches, resulting in a lower maximum minor storm event water surface elevation for streets utilizing this gutter type. It does, however, reduce the need for driveway "ramps", which are often a major contributor to the reduction of residential gutter capacity. **Figure 1104** is a detail of the standard mountable curb.

## 1105 STREET HYDRAULIC CAPACITY EVALUATION

Gutter and street flow can generally be assumed to be uniform for the purpose of hydraulic evaluation and design, but as street flow depth increases, flow width increases at a much faster rate. This wide, relatively shallow flow has the effect of decreasing the hydraulic radius, rendering the standard Manning's equation somewhat inaccurate. The Federal Highway Administration (FHWA) presents a modified form of the equation in Section 5.3.2 of *Introduction to Highway Hydraulics*, taken from HEC-12:

$$Q_S = \frac{K_u}{n} S_X^{5/3} S_L^{1/2} T_S^{8/3} \tag{1101}$$

Where:

$Q_S$ = Street Flow Capacity (not including gutter) (cfs)

$n$ = Manning's Roughness Coefficient

$K_u$ = 0.56 for English units

$K_u$ = 0.376 for S.I. units

$S_X$ = Street Cross Slope (ft/ft)

$S_L$ = Street Longitudinal Slope (ft/ft)

$T_S$ = Flow Top Width (not including gutter) (ft)

For streets with a single cross slope for the gutter and street section, the above Equation 1101 will suffice for determining total gutter/street flow capacity if the $T_S$ term is replaced by *T = Total Flow Top Width (including gutter, inside curb).* However, the Standard Concrete Details indicate the use of composite cross slopes for all streets – the gutter has a steeper cross slope than the street. In these cases, the above equation specifies capacity in the flow area between the edge of pavement (not including the gutter itself) and the edge of flow. Note that the result of the same equation as applied to any flow area beyond the street centerline must be subtracted from the total capacity (see **Figure 1101**). The total flow capacity in the street and the gutter is calculated as:

$$Q = \frac{Q_S}{1 - E_o} - Q_{S,\text{Outside Street Centerline}} \tag{1102}$$

$$E_o = 1/\left[1+\frac{S_W/\ S_X}{\left[1+\frac{S_W/\ S_X}{(T/W)-1}\right]^{8/3}-1}\right] \tag{1103}$$

Where:

$S_W$ = Gutter Cross Slope (ft/ft)

$S_X$ = Street Cross Slope (ft/ft)

T = Top Width (inside curb) (ft)

W = Gutter Width (ft)

**Figures 1105 through 1110** show calculated half-street flow capacities for each street classification group based on longitudinal street slope and flow depth. The HEC-12 equation (Equation 1101) was used to determine most flow capacities for these charts except in cases where the flow width does not exceed gutter width. Where gutter flow does not encroach on the street surface, it is assumed that the width to depth ratio is not large enough to warrant the utilization of the HEC-12 modified Manning's equation. Instead, the standard form is used in these cases:

$$Q = \frac{1.49}{n} A \cdot R^{2/3} \cdot S_L^{1/2} \tag{1104}$$

Where:

Q = Flow Capacity (cfs)

$S_L$ = Street Longitudinal Slope (ft/ft)

R = Hydraulic Radius (ft) = $A/P$

A = Cross – Sectional Flow Area (sf)

P = Wetted Perimeter (ft)

Major storm event flow depth may exceed curb height, thus the flow area behind the curb – between the curb and the right-of-way – must also be considered in the calculation of total street capacity. The flow in this area is found using Equation 1101, replacing $T_S$ with $T_B$ = Flow Top Width (behind curb). Note that the truncation procedure described above must also be applied here if the calculated top width extends beyond the right-of-way boundary. Therefore, the total flow between the curb and street centerline, $Q_B$, is:

$$Q_B = Q_{B,Gross} - Q_{B,Outside\ Street\ Right-of-Way} \tag{1105}$$

Total street capacity, then, is the sum of the resulting values from Equations 1102 and 1105. The HEC-12 procedure has certain limitations and includes certain assumptions. The most applicable of these are listed below.

1. One value for Manning's roughness n must be used for the entire cross section. This can be a composite value derived from multiple roughness segments in the cross section.

2. The HEC-12 method ignores any roughness characteristics of the vertical segment of the curb. The energy-dissipating effect of this portion is considered to be negligible when compared to the wide bottom sections.
3. The "vertical" curb shown in **Figure 1111** actually slopes back away from the street by 1 inch while rising 6 inches. This slope is ignored by the HEC-12 equations. The gutter width is assumed to include this extra 1 inch with a vertical segment of 6-inch height at this point. See **Figure 1111** for detail.
4. The Standard Concrete Details indicate that the edge of the roadway pavement shall be ¼ inch to ½ inch above the edge of the concrete gutter. The HEC-12 method is unable to account for this directly, and the hydraulic effects are assumed to be negligible, so this is ignored.

Two of the provided standard street capacity charts, **Figures 1105** and **1110**, were not prepared using the procedure described above. Both residential streets with mountable curbs (**Figure 1105**) and Principal Arterials (**Figure 1110**) have a more complex cross-section than the HEC-12 method can accurately model. The former is due to the alternate curb-and-gutter configuration, and the latter has a required median to which the flow can spread, adding another flow-control surface to the cross-section. Therefore, Haestad Methods' FlowMaster computer program was used to model these sections and prepare capacity charts. The program uses the Cox open-channel weighting method to determine composite roughness and utilizes the standard form of Manning's equation to compute discharge.

Certain assumptions were made in the creation of the street capacity charts to minimize the required number of figures and to simplify the design process. However, it is the responsibility of the designer to ensure that each assumption is valid for a specific design.

1. A Manning's roughness value of $n = 0.016$ was assumed for all flow surfaces encountered within the street right-of-way when using the HEC-12 method. Varying values of $n$ were used to construct the FlowMaster cross-sections, but composite roughness values were not significantly different from 0.016 for the two special cases.
2. A street cross slope of 2.0% was assumed for all streets.
3. A gutter cross slope of 8.33% was assumed for all gutters.
4. Velocity curves are provided on the capacity charts for reference only – the designer is responsible for the calculation of actual gutter velocities.

In cases where these assumptions may not be valid, such as designs incorporating the use of non-standard street sections, the designer shall utilize the equations presented above to determine allowable street capacity.

The maximum allowable gutter velocity is 8.0 feet per second. Velocities exceeding this value can create safety issues, cause erosive damage to the street and other surfaces, and reduce the effectiveness of storm drain inlets.

## 1106 STORM INLET SELECTION, SIZING, AND LOCATION

Wherever street and gutter capacity exceeds allowable values based on flow spread, velocity, or depth, some or all of the runoff must be intercepted and diverted to an alternate flow path such as a storm drain or designated runoff channel. The most common method of interception is by the utilization of storm drainage inlets, of which there are four major types:

1. Curb-Opening Inlets
2. Grate Inlets
3. Combination Inlets (Curb-Opening and Grate)
4. Slotted Drain Inlet

The slotted drain inlet is not recommended due to its high susceptibility to clogging, and may only be used where specifically approved by a local jurisdiction. All inlets must be labeled with "NO DUMPING – DRAINS TO RIVER". Isometric-view examples of the first three types are shown in **Figure 1112**.

Inlets may either be located on a continuous grade – where flow not intercepted by the inlet will pass to another location – or in a sag portion of a street's vertical alignment (or any other sump location). Computation of inlet capacity involves several factors including type of inlet, location (on-grade or in a sump), grate type (if applicable), inlet geometry, flow width and depth, and both longitudinal and cross slopes.

Methods and rationale utilized in this Section for the calculation of inlet capacities are based on those presented in HEC-12 (1984). However, HEC-12 contains a nomograph for finding grate splash-over velocity, and here we use an empirical formula for this value (Guo, *Storm Water System Design*, CE 5803, University of Colorado at Denver, 1999). Additionally, clogging must be considered in the selection and location of inlets, and will be addressed in Section 1106.3.

### 1106.1 Hydraulic Capacity of Inlets on a Continuous Grade

Gutter velocities exceeding a "splash-over" velocity and flow widths greater than grate widths typically result in interception efficiencies of less than 100% for an on-grade inlet. Inlet efficiency, E, is defined as:

$$E = Q_i/Q \qquad (1106)$$

Where:

$Q_i$ = Intercepted Flow Rate

$Q$ = Gutter Total Flow Rate

Flow which is not intercepted by the inlet is called bypass flow, $Q_b$:

$$Q_b = Q - Q_i \qquad (1107)$$

The interception efficiency of an inlet is affected by different factors for different inlet types. Grate inlets are most sensitive to the amount of water flowing directly over the grate (frontal flow) and the velocity of flow in the gutter. Curb-opening inlets vary primarily with inlet length, depth of flow, and both longitudinal and cross slopes of the gutter and street. Combination inlets where the curb opening and grate inlets are of similar length and are installed adjacently have been shown to have essentially the same interception capacity as the same grate configuration acting without a curb opening. However, curb-opening inlets have greater debris-handling capabilities than grates, and therefore combination inlets typically experience significantly less grate clogging than would be experienced if the curb-opening was excluded. A "sweeper inlet" is a type of combination inlet that utilizes a segment of curb opening upstream from the grated portion. This configuration has the highest debris-removal efficiency, and therefore lowest clogging rate, of all types listed here. Total capacity is the sum of the flow intercepted by the curb opening located upstream of the grate(s) and that intercepted by the grated portion of the combination inlet.

1106.1.1 Grate Inlets

Grand Junction Standard Storm Drain Details (Page D-05, 2005) lists storm drain grate types which have been approved for use. The four primary types for use on streets are Types R, L, V, and D in single, double, and triple-inlet configurations, and are included in the inlet capacity charts provided herein. Page D-05 also contains two important notes concerning the selection of grates:

1. Use Type R or Type D grate where inlet is located in sump condition.
2. Use Type V or Type L where gutter flow is from one direction only.

Type V and Type L grates are vane grates, which are more efficient for on-grade locations, but are very inefficient for sump conditions. Therefore, local jurisdictions do not allow the use of grate Types V and L for locations where sump conditions may exist. However, these types are recommended for on-grade locations where bicycle traffic is expected.

The FHWA and others have extensively researched the hydraulic characteristics of the seven grate types listed in **Table 1101**. The approved grate types were each matched to the FHWA grate type most closely matching in geometry and apparent hydraulic functionality. These assumed pairings are indicated in **Table 1101**.

The total rate of intercepted flow by a grate inlet (or standard combination inlet) is the sum of the intercepted frontal flow and the intercepted side flow. Frontal flow, $Q_w$ is that portion of the total gutter flow which has a flow width equal to or less than the grate width. Total frontal flow is found using the following HEC-12 equation:

$$Q_w = Q\left[1-\left(1-\left(W/T\right)\right)^{2.67}\right] \tag{1108}$$

Where:

$Q_w$ = Total Frontal Flow (cfs)

$Q$ = Total Gutter Flow (no behind – curb flow) (cfs)

$W$ = Width of Grate (ft)

$T$ = Total Top Width of Flow (ft)

Total side flow is that portion of total gutter flow which is between the inner edge of the grate(s) and the street centerline, and is the remainder of the total flow between the curb and the centerline:

$$Q_s = Q - Q_w \tag{1109}$$

Where: $Q_s$ = Total Side Flow

To find the ratio of intercepted frontal flow ($Q_{wi}$) to total frontal flow ($Q_w$), $R_f$, one must first find the "splash-over velocity", $V_o$, for the selected grate type. This function was developed empirically by Guo (1999):

$$V_o = \alpha + \beta L_e - \gamma L_e^2 + \eta L_e^3 \qquad (1110)$$

Where: $L_e$ = Effective Unit Grate Length (see Equation 1124)
$\alpha, \beta, \gamma, \eta$ = Splash Velocity Constants (see **Table 1101**)

**Table 1101 Splash Velocity Constants**

| Type of Grate | Assumed Equivalent | $\alpha$ | $\beta$ | $\gamma$ | $\eta$ |
|---|---|---|---|---|---|
| Bar P-1-7/8 | Type D | 2.22 | 4.03 | 0.65 | 0.06 |
| Bar P-1-1/8 | Not used | 1.76 | 3.12 | 0.45 | 0.03 |
| Vane Grate | Type L,V | 0.30 | 4.85 | 1.31 | 0.15 |
| 45-Degree Bar | Not used | 0.99 | 2.64 | 0.36 | 0.03 |
| Bar P-1-7/8-4 | Not used | 0.74 | 2.44 | 0.27 | 0.02 |
| 30-Degree Bar | Not used | 0.51 | 2.34 | 0.20 | 0.01 |
| Reticuline | Type R | 0.28 | 2.28 | 0.18 | 0.01 |

Adapted from Guo, *Storm Water System Design*, 1999.

Splash-over velocity is an experimentally-derived value at which frontal flow begins to bypass the grate essentially because water does not spend enough time over the inlet to allow it to fall through the grate. As previously mentioned, this value varies per grate type – some grates tend to better capture higher velocity flows. This allows the designer to calculate the HEC-12 frontal flow interception ratio:

$$R_f = Q_{wi}/Q_w = 1.0 - 0.09(V - V_o) \quad \text{where } V \geq V_o\text{, otherwise } R_f = 1.0 \qquad (1111)$$

Due to the primarily longitudinally-flowing nature of street and gutter flow, the interception ratio of side flow is typically minimal, and in some jurisdictions is ignored altogether. This ratio, $R_s$, is defined as:

$$R_s = Q_{si}/Q_s = \frac{1}{1 + \dfrac{0.15 \cdot V^{1.8}}{S_X \cdot L_e^{2.3}}} \qquad (1112)$$

Where:
$Q_{si}$ = Intercepted Side Flow (cfs)
$V$ = Gutter Flow Velocity (fps)
$S_X$ = Street Cross Slope
$L_e$ = Effective Length of Grate(s) (ft)
(see Equation 1124)

Total intercepted flow ($Q_i$), then, is found using Equation 1113:

$$Q_i = Q_{wi} + Q_{si} = R_f \cdot Q_w + R_s \cdot Q_s \tag{1113}$$

Total capture efficiency (*E*) for a grate inlet can be found using:

$$E = R_f(Q_w / Q) + R_s(Q_s / Q) \tag{1114}$$

Inlet clogging due to the buildup of debris and sediment can adversely affect the interception capacity of an inlet. A clogging factor must be applied to find the design value for $Q_i$. See Section 1106.3 for an explanation of the calculation of clogging factors.

Included in **Figures 1114** through **1116** are inlet capacity charts for all street types and curb-and-gutter and inlet configurations considered by this Manual. **Figure 1113** is a legend for these figures. The capacity curves are specific to grate type, inlet type, and whether the subject inlet is single, double, or triple length. All curves include appropriate clogging factors per Section 1106.3. Inlet capacity values calculated from the provided equations may be used if they do not exceed the values provided in the inlet capacity charts.

1106.1.2 Curb-Opening Inlets

The interception capacity of a curb-opening inlet on a continuous grade is dependent on inlet length, flow depth, and the longitudinal and cross slopes of the street. To determine the capture efficiency of a curb-opening inlet, one must first calculate the length that would be required for 100% interception of the gutter flow at that location ($L_T$):

$$L_T = 0.6Q^{0.42}S_L^{0.3}\left(\frac{1}{n \cdot S_e}\right)^{0.6} \tag{1115}$$

$$S_e = S_X + \left(a/W\right)E_o \tag{1116}$$

Where:

$Q$ = Total Gutter Flow (cfs)

$S_L$ = Longitudinal Street Slope

$n$ = Manning's Roughness Coefficient

$S_e$ = Equivalent Cross Slope

$S_X$ = Street Cross Slope

$a$ = Gutter Depression Below Street Slope (ft)

$W$ = Gutter Width (ft)

$E_o$ = Defined by Equation 1103

The efficiency of a curb-opening inlet is calculated using Equation 1117, where effective length $L_e$ is defined in Equation 1124 (Section 1106.3):

$$E = 1 - \left[1 - \left(L_e / L_T\right)\right]^{1.8} \quad \text{where } L_e < L_T\text{, otherwise } E = 1.0 \tag{1117}$$

Included in **Figures 1114** through **1116** are inlet capacity charts for all street types, grate types, inlet types and curb-and-gutter configurations considered by this Manual. Each figure is specific to the type of grate used. The capacity curves for all on-grade inlets are specific to a single inlet length of 5.0 feet for curb-opening-only inlets and 3.0 feet for both grate-only and combo inlets. **Figure 1114GJ** is the inlet capacity chart for the Grand Junction Drive-Over inlet with its own legend. It assumes the curb height is 4.5 inches, the flow line of the frame and grate is 1 inch below normal gutter flow line, and the vertical opening is set at 1 inch in height. Curves are provided for single, double, or triple length. All curves include appropriate clogging factors per Section 1106.3. Inlet capacity values calculated from the provided equations may be used if they do not exceed the values provided in the inlet capacity charts.

1106.1.3 Combination Inlets

Per the introduction to Section 1106.1 of this Manual, combination inlets are assumed to have the capacity of the same inlet with the grate(s) acting alone except in the case of "sweeper inlets". However, the designer shall note the reduced clogging susceptibility of the combination inlet when compared with a grate-only configuration noted in Section 1106.3.

1106.1.4 Slotted Drain Inlets

Slotted Drains are effective for intercepting flow over a wide section, such as sheet flow across the pavement of a street. However, these inlets are highly susceptible to clogging, and are only allowed as specifically approved by a local jurisdiction. Methods for the design of such inlets can be found in HEC-12 and HEC-22.

### 1106.2 Hydraulic Capacity of Inlets in Sump Conditions

A sag or sump condition occurs in a location where water that flows into the area must pond to some depth before any of the flow can escape the area via channel or overland flow. Unlike inlets on a continuous grade, those in a sump condition are not designed to bypass a portion of the flow incident to the inlet location. This means that these inlets must have the capacity to effectively capture all of the runoff that ponds in the sump and to maintain acceptable ponding depths.

These requirements, along with an increased potential for inlet clogging due to low flow velocities, necessitate special provisions for the design of sump inlets. A secondary flow path must be provided to maintain a reasonable ponding depth in the case of inlet failure (near-complete clogging, for instance). The preferred secondary flow path is a designated emergency overflow weir and channel, which must be located within an accessible drainage easement and must be protected from erosive effects as necessary by pavement or riprap. If no easement is available at the inlet location, flanker inlets must be installed in the same gutter on each side of the primary inlet. Flanker inlets are located upgradient 10 to 50 feet from the primary sump inlet. The two flanker inlets shall have a combined design capacity equal to or greater than that of the primary inlet.

Local jurisdictions recommend the use of combination inlets in sumps due to their higher capacity and lower clogging tendency. Curb-opening inlets are also allowable, but grate-only inlets and slotted-drain inlets are not allowed for use in sump conditions.

Per Section 1106.1.1, Type L and Type V grates are prohibited for use with inlets located in sumps. Mesa County and the City of Grand Junction have approved grate Types D and R for inlets in these locations.

The hydraulic capacity of an inlet in a sump condition is dependent on the configuration of the inlet and the depth of the ponded water. At small depths, the flow into the inlet is by weir flow, transitioning to orifice flow at increasing depths. These depths are defined in **Table 1102**.

The gross capacity of an inlet operating as a weir is defined by Equation 1118:

$$Q_i = C_W L_W d^{1.5} \qquad (1118)$$

Where:

$Q_i$ = Inlet Capacity (cfs)

$L_W$ = Weir Length (ft)

$d$ = Flow or Ponding Depth (ft)

$C_W$ = Weir Discharge Coefficient (see **Table 1102**)

The gross capacity of an inlet operating as an orifice is defined by Equation 1119:

$$Q_i = C_o A_o (2 \cdot g \cdot d_o)^{0.5} \qquad (1119)$$

Where:

$Q_i$ = Inlet Capacity (cfs)

$A_o$ = Orifice Open Area (sf)

$g = 32.2\ ft/s^2$

$d_o$ = Depth to Orifice Centroid (ft)

$C_o$ = Orifice Discharge Coefficient (see **Table 1102**)

**Table 1102 Discharge Coefficients and Variable Definitions for Inlets in a Sump Condition**

| Type of Inlet | $C_w$ | $C_o$ | Weir | Orifice | $L_w$ |
|---|---|---|---|---|---|
| Grate | 3.00 | 0.67 | $d < 1.79(A_o/L_w)$ | $d > 1.79(A_o/L_w)$ | 2w+L |
| Curb-Opening | 3.00 | 0.67 | $d < h$ | $d > 1.4h$ | L |
| Depressed Curb-Opening | 2.30 | 0.67 | $d < h + a$ | $d > 1.4h$ | 2w+L |

Where:

$A_o$ = Orifice Open Area (sf)

$L_w$ = Weir Length (ft)

d = Flow or Ponding Depth (ft)

w = Grate Width (ft)

L = Inlet Length (ft)

It is important to note that the capacity of a combination inlet in a sump is defined by the capacity of the grate portion only when operating as a weir (the curb opening is ineffectual and thus ignored), but is defined by the cumulative capacity of the grate and curb opening when operating as an orifice. Any curb opening length extending beyond the ends of the grates may be included in the weir length.

For any given inlet, a certain range of depths will result in "transitional flow", where neither the weir equation nor the orifice equation accurately models flow through the inlet. Linsley (1992) states that where transition conditions exist, "the capacity is intermediate between that of an orifice and a weir." For design purposes, the capacity for depths in the transitional range is based on the lesser of the results of Equations 1118 and 1119.

Local inlet depression increases the capacity of inlets, especially those in sumps, by increasing the depth over the inlet without increasing street flow depth. Local depression loses effectiveness for curb-opening inlets of 12-foot length or greater, so the "Curb-Opening" information from **Table 1102** shall be used for these. **Figure 1117** contains two tables with maximum inlet capacities for inlets with a 2-inch local depression and without depression (level with curb flow line). The first capacity table is for the standard 6-inch vertical curb configuration (applies to most streets in Mesa County) and the second table contains values for the 4.5-inch mountable (drive-over) curb. All capacities listed in **Figure 1117** include a clogging factor per Section 1106.3.

### 1106.3 Inlet Clogging Considerations

Inlets are always susceptible to clogging when water flows to them. Low flows can carry leaves and finer sediments and deposit them at the inlet, while higher flows can deposit larger debris. The latter is of most concern since it is often too large to be washed down the inlet, even with heavy storm flows. Proper inlet design and timely maintenance including street sweeping, visual inspection of inlets, and removal of large debris are essential to the proper operation of inlets. The clogging factors listed in **Table 1103** are based on the assumption that routine maintenance is performed on all drainage structures.

**Table 1103** contains the assumed clogging factors for different inlet types. A value of 0% indicates that no clogging is expected, and a value of 100% indicates that the inlet shall be considered completely clogged or is not allowed in the specified conditions.

**Table 1103 Clogging Factors for Single Inlets**

| Type of Inlet | On-Grade | Sag/Sump $d \le 0.5$ ft | Sag/Sump $d > 0.5$ ft |
|---|---|---|---|
| Grate | 50% | 100% | 100% |
| Curb-Opening | 20% | 20% | 20% |
| Combination (Grate Portion) | 0% | 0% | 50% |
| Combination (Curb-Opening Portion) | 100% | 100% | 0% |

In the case of multiple (double or triple) inlets, the continuous (linear) application of some of these factors would result in the inability to capture 100% of the flow incident to the subject inlet. Although on-grade inlets are rarely designed to capture 100% of gutter flow from the minor storm event, inlet design lengths may become unnecessarily long to achieve the desired flow interception rate. It is reasonable to assume that a majority of the sediment and debris is deposited at the inlet by the first few minutes of significant storm flow, possibly before the peak flow occurs at that inlet. Also, the majority of clogging tends to more greatly affect the upgradient segments of a multi-unit inlet. For instance, the first grate of a triple inlet may become 50% clogged, while the third grate experiences only 10% clogging. This phenomenon was generalized by Guo in *Design of Grate Inlets with a Clogging Factor* (2000) using a decay equation and empirically-derived decay factors. It may be applied to inlets on a continuous grade and inlets in sump conditions.

$$C = \frac{1}{N}\left(C_o + eC_o + e^2C_o + \ldots + e^{N-1}C_o\right) = \frac{C_o}{N}\sum_{i=1}^{i=N} e^{i-1} \qquad (1120)$$

Where:

$C =$ Multiple – Unit Clogging Factor (decimal)

$C_o =$ Single – Unit Clogging Factor (decimal)

$N =$ Number of Units

$e =$ Decay Ratio

$e =$ 0.5 for grates, 0.25 for curb – opening inlets

This adjusted clogging factor can be applied to the sump-inlet capacity equations (1118 and 1119) by replacing the variables for weir length $L_w$ with effective weir length ($L_{we}$) from Equation 1121 and orifice area $A_o$ with effective opening area ($A_{oe}$) from Equation 1122. The clogging factor may also be applied directly to the gross flow capacities from Equations 1118 and 1119 using Equation 1123.

$$L_{we} = (1 - C)L_w \qquad (1121)$$

$$A_{oe} = (1 - C)A_o \qquad (1122)$$

$$Q_i(\text{net}) = (1 - C)Q_i \qquad (1123)$$

To apply the clogging factor to an on-grade inlet, the designer must first find the effective length ($L_e$) – that portion of the inlet length which is considered unclogged.

$$L_e = (1 - C)L \qquad (1124)$$

This value is used in the computation of splash-over velocity in Equation 1110, side-flow interception ratio in Equation 1112, and curb-opening efficiency in Equation 1117.

### 1106.4 Grate Selection and Rating

Selection of grates for on-grade and sump inlets must be consistent with Section 1106.1.1 of this Manual and the page titled "Approved Storm Drain Inlets" (D-05) of the City of Grand Junction Standard Details. The designer is urged to obtain any grate rating data available from the manufacturer of the selected grate. This data may be used to cross-check the values obtained using the methods presented in this Manual and/or to obtain interception and capacity values. However, in the latter case, it is imperative that the designer still incorporate all clogging/safety factors that apply to the inlet design per local jurisdictions' requirements.

### 1106.5 Inlet Location and Spacing

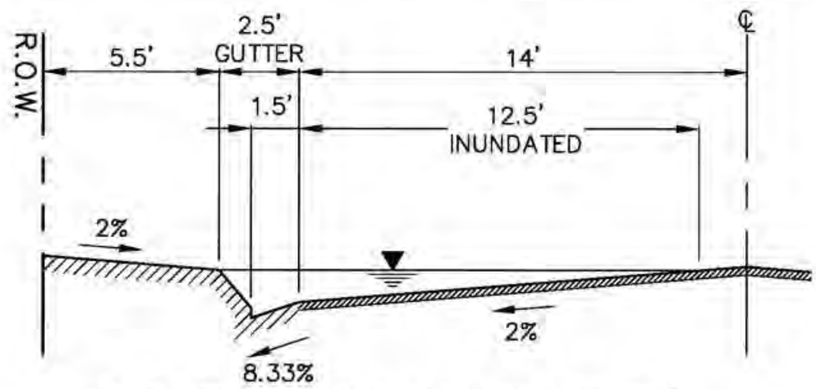
Inlets shall be placed at any location where ponding water may encroach on street traffic beyond the allowable limits. These limits are defined by gutter flow depth during the minor storm event (Section 1104). An inlet location is determined using an iterative process:

1. Determine a preliminary location for the inlet based on street configuration and estimated runoff to the gutter.
2. If the inlet is in a sump, location is essentially fixed during the remainder of the design process. The inlet shall be sized to maintain ponding depths smaller than those required by local jurisdictions. If the required inlet size becomes excessively large, the designer is urged to install additional inlets upgradient from the sump.
3. For inlets on a grade, the designer must find the flow characteristics at the selected preliminary inlet location to determine whether the inlet needs to be placed further upstream or may be moved downstream based on maximum flow depth for the minor storm event.
4. The designer shall take into account the change in tributary area to the inlet associated with any upstream or downstream movement.
5. A typical design interception efficiency of an on-grade inlet is 70 to 80 percent. As mentioned previously, on-grade inlets designed to capture 100 percent of the minor storm runoff tend to be significantly less effective both hydraulically and economically.
6. The designer shall include any carryover (bypass) flow from an upstream inlet when calculating the flow at a downstream inlet. Although the peak runoff to an inlet may not coincide with the peak carryover flow from an upstream inlet, these two peak flows shall be added to find the total peak flow to the downstream inlet.

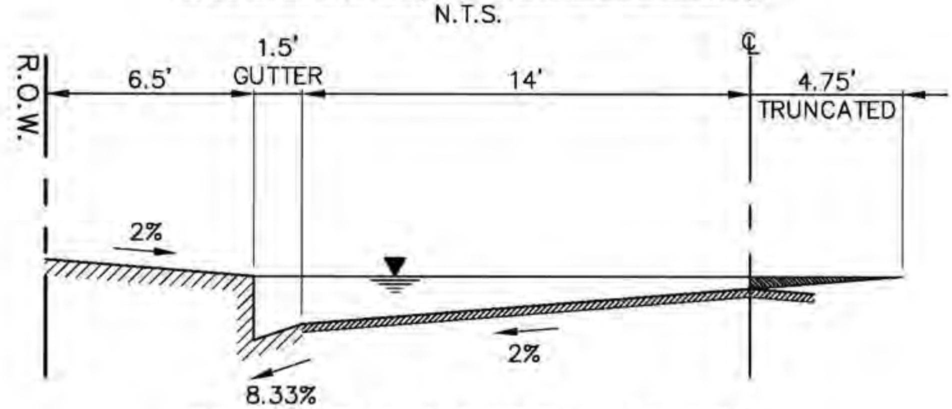
Maximizing the use of sump inlets tends to increase the overall efficiency of the inlet system, and inlets must be installed at all street sags (vertical curve low points) and at all sumps formed by intersections except where other drainage provisions have been made. Therefore, it is suggested that sump inlets are located prior to the placement of any on-grade inlets during the design process.

# STORMWATER MANAGEMENT MANUAL

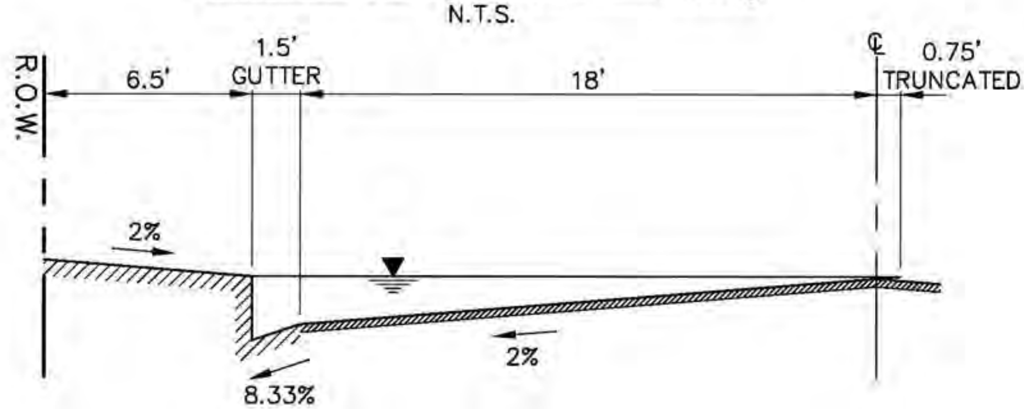
## STREET INUNDATION LIMITS FOR THE 2-YEAR STORM




RESIDENTIAL (MOUNTABLE CURB)
N.T.S.




RESIDENTIAL (VERTICAL CURB)
N.T.S.




RESIDENTIAL COLLECTOR/COMMERCIAL/INDUSTRIAL
N.T.S.

NOTE THAT IN NO CASE SHALL THE 100-YEAR FLOW DEPTH EXCEED 1.0 FEET AT THE GUTTER FLOWLINE.

| Revision | Date |
|---|---|
| ORIGINAL ISSUE | 3/27/06 |
| ADDED NOTE | 12/6/07 |
| | |

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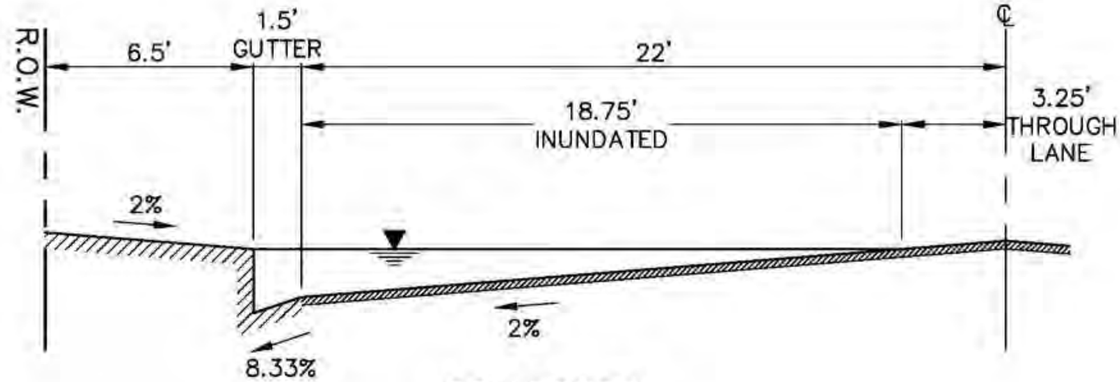
REFERENCE: GRAND JUNCTION STANDARD STREET DETAILS
GRAND JUNCTION STANDARD CONCRETE DETAILS

**FIGURE 1101**

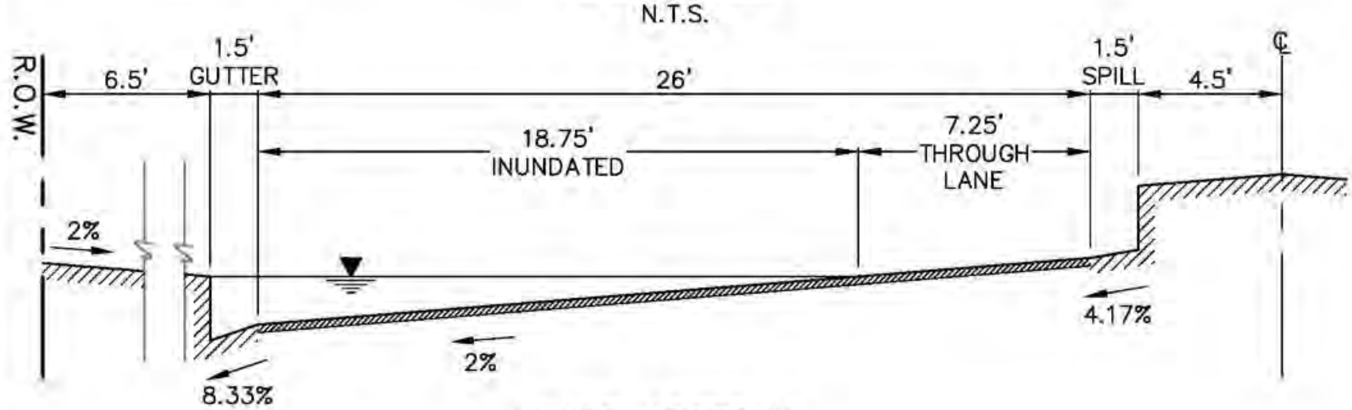
# STORMWATER MANAGEMENT MANUAL

## STREET INUNDATION LIMITS FOR THE 2-YEAR STORM



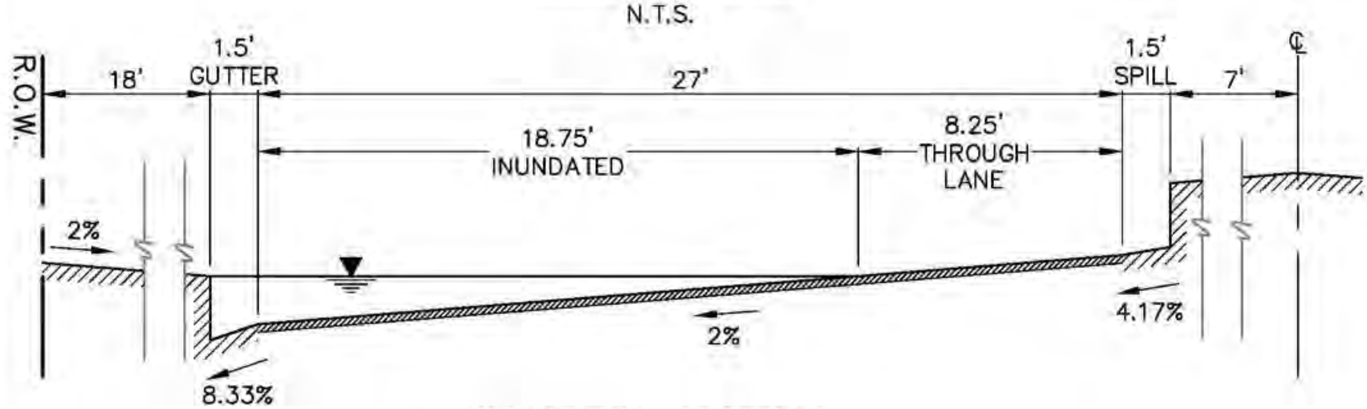

COLLECTOR

N.T.S.




MINOR ARTERIAL

N.T.S.




PRINCIPAL ARTERIAL

N.T.S.

NOTE THAT IN NO CASE SHALL THE 100-YEAR FLOW DEPTH EXCEED 1.0 FEET AT THE GUTTER FLOWLINE.

| Revision | Date |
|---|---|
| ORIGINAL ISSUE | 3/27/06 |
| ADDED NOTE | 12/6/07 |

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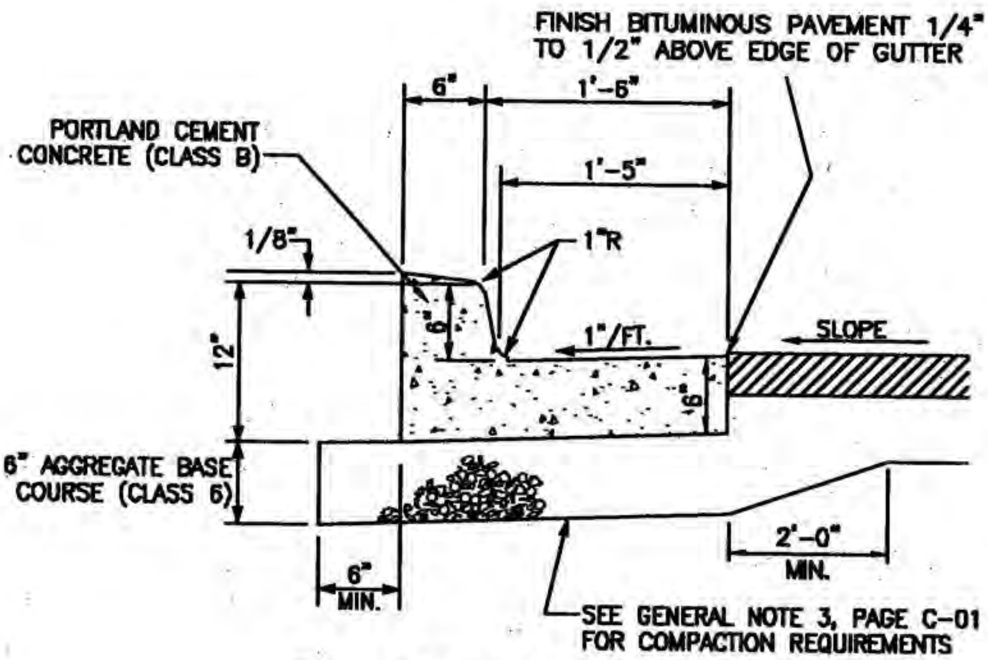
REFERENCE:
GRAND JUNCTION STANDARD STREET DETAILS
GRAND JUNCTION STANDARD CONCRETE DETAILS

**FIGURE 1102**

# STORMWATER MANAGEMENT MANUAL

## VERTICAL CURB/GUTTER DETAIL




STANDARD CURB AND GUTTER

| Revision | Date |
|---|---|
| ORIGINAL ISSUE | 3/27/06 |
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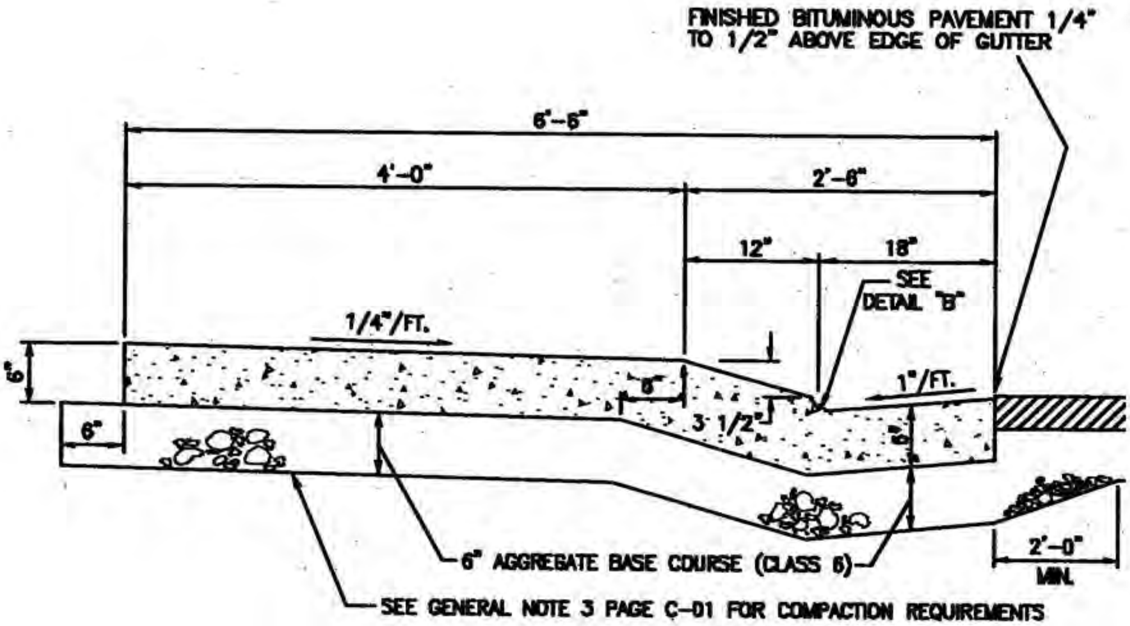
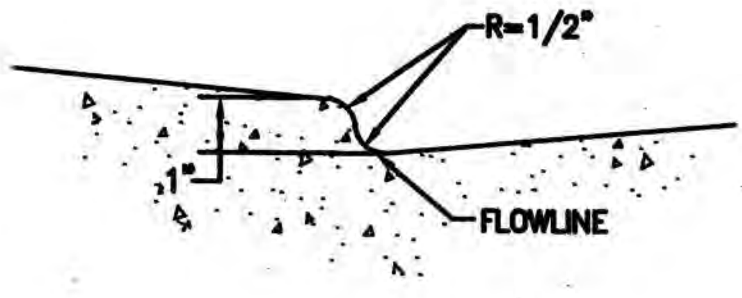
REFERENCE:
GRAND JUNCTION STANDARD CONCRETE DETAILS

**FIGURE 1103**

# STORMWATER MANAGEMENT MANUAL

## MOUNTABLE CURB/GUTTER DETAIL




DETAIL "B"

| Revision | Date |
|---|---|
| ORIGINAL ISSUE | 3/27/06 |
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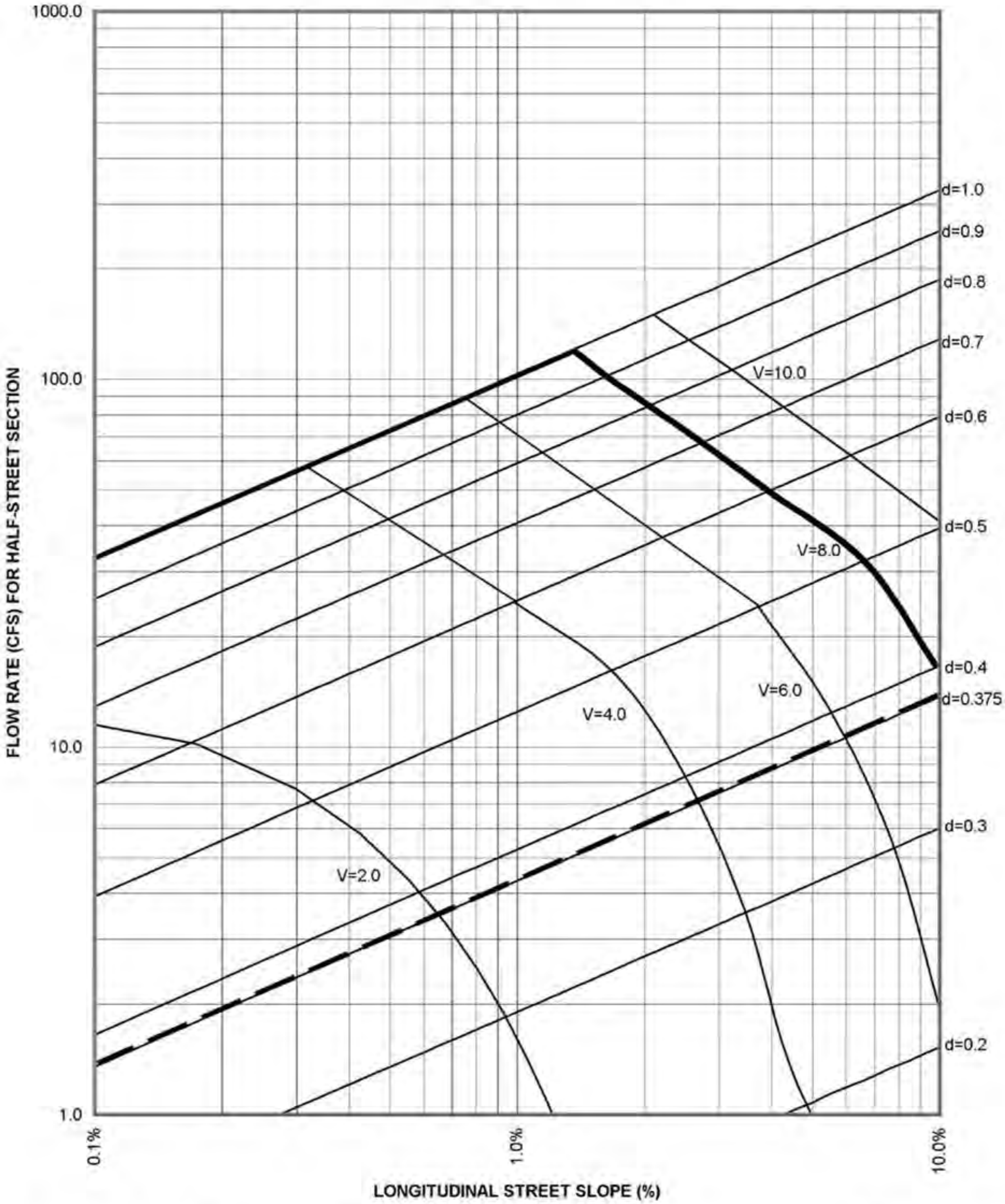
REFERENCE:

GRAND JUNCTION STANDARD CONCRETE DETAILS

**FIGURE 1104**

# STORMWATER MANAGEMENT MANUAL

**HALF-STREET FLOW CAPACITY**
**(RESIDENTIAL, MOUNTABLE CURB)**




| DESIGN LIMITS | |
|---|---|
| — — — | MINOR STORM |
| ——— | MAJOR STORM |

| Revision | Date |
|---|---|
| ORIGINAL ISSUE | 3/27/06 |
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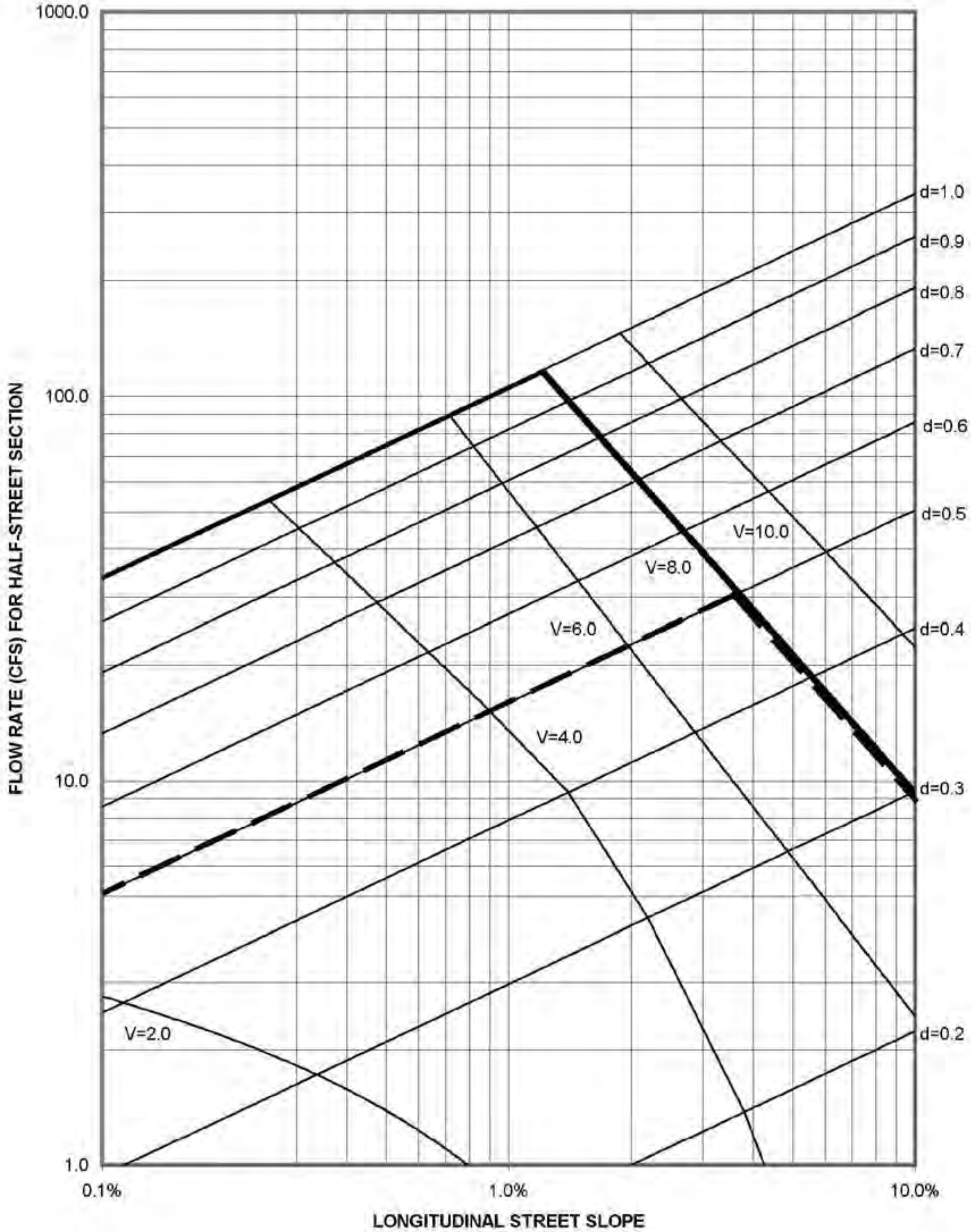
REFERENCE:

**FIGURE 1105**

# STORMWATER MANAGEMENT MANUAL

**HALF-STREET FLOW CAPACITY**
**(RESIDENTIAL, VERTICAL CURB)**




**DESIGN LIMITS**

— — — MINOR STORM

——— MAJOR STORM

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|---|---|
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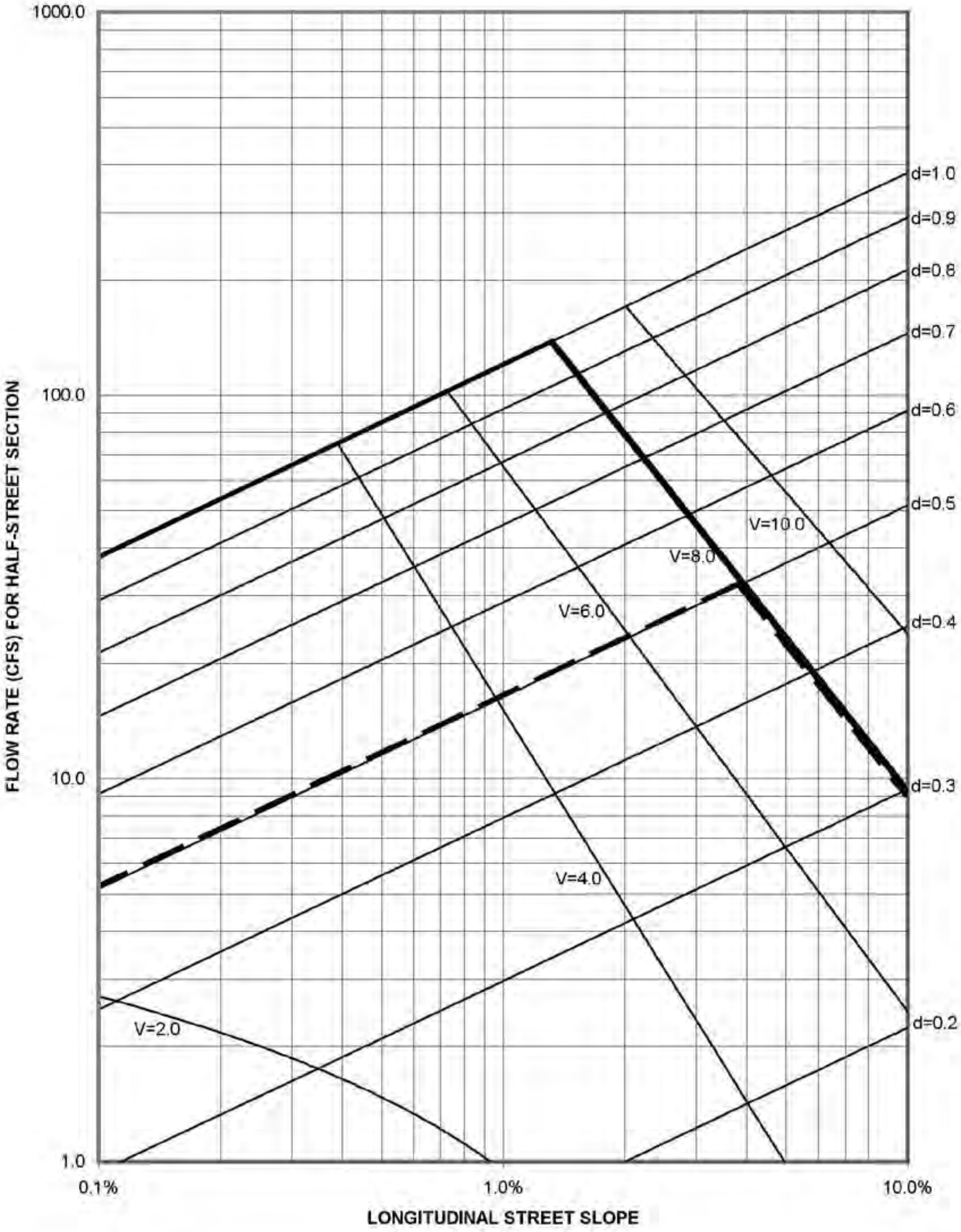
**FIGURE 1106**

## HALF-STREET FLOW CAPACITY
## (RESIDENTIAL COLLECTOR, COMMERCIAL)




**DESIGN LIMITS**

MINOR STORM

MAJOR STORM

| Revision | Date |
|---|---|
| ORIGINAL ISSUE | 3/27/06 |
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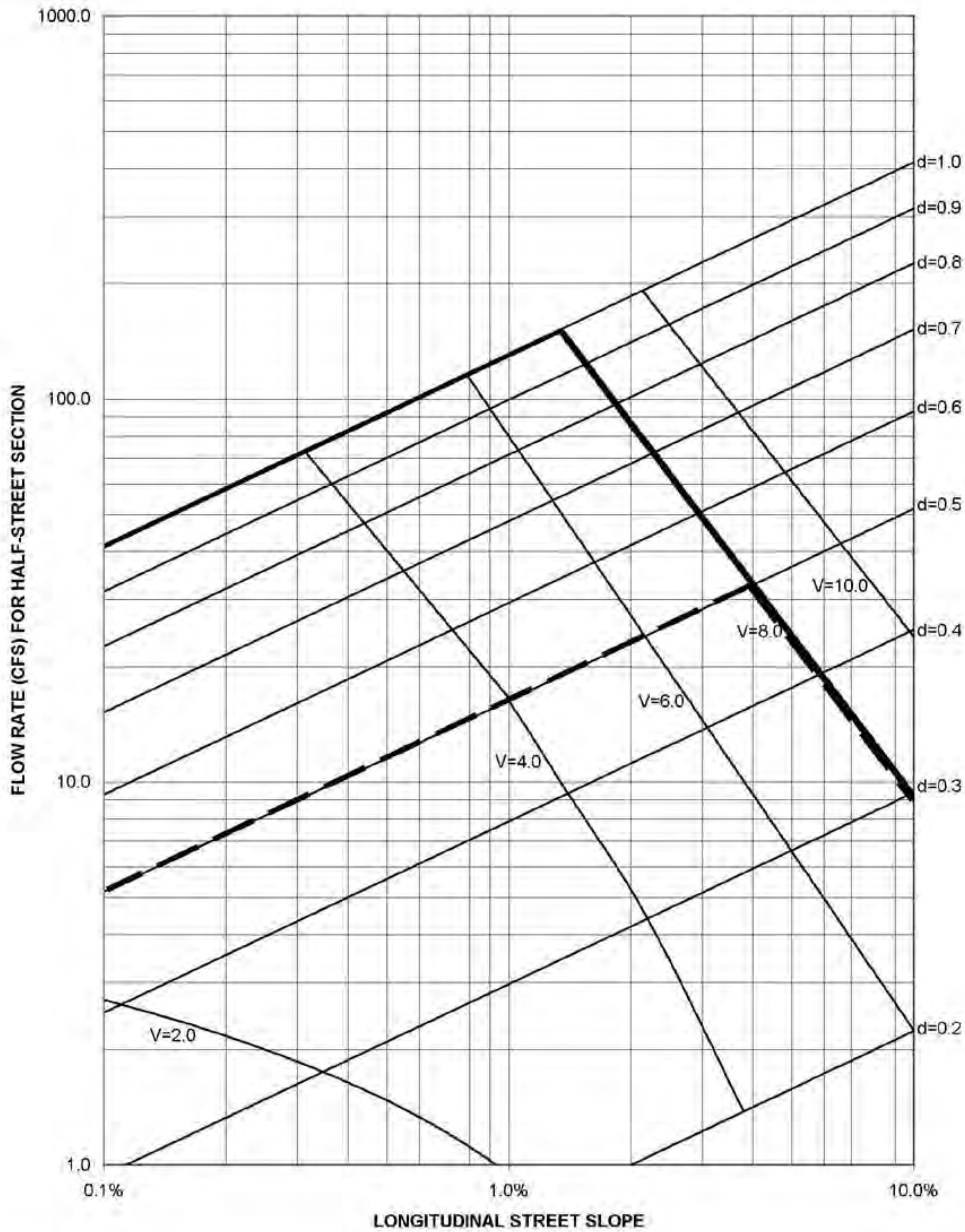
REFERENCE:

**FIGURE 1107**

**HALF-STREET FLOW CAPACITY**
**(COLLECTOR, VERTICAL CURB)**




**DESIGN LIMITS**

— — — MINOR STORM

——— MAJOR STORM

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|---|---|
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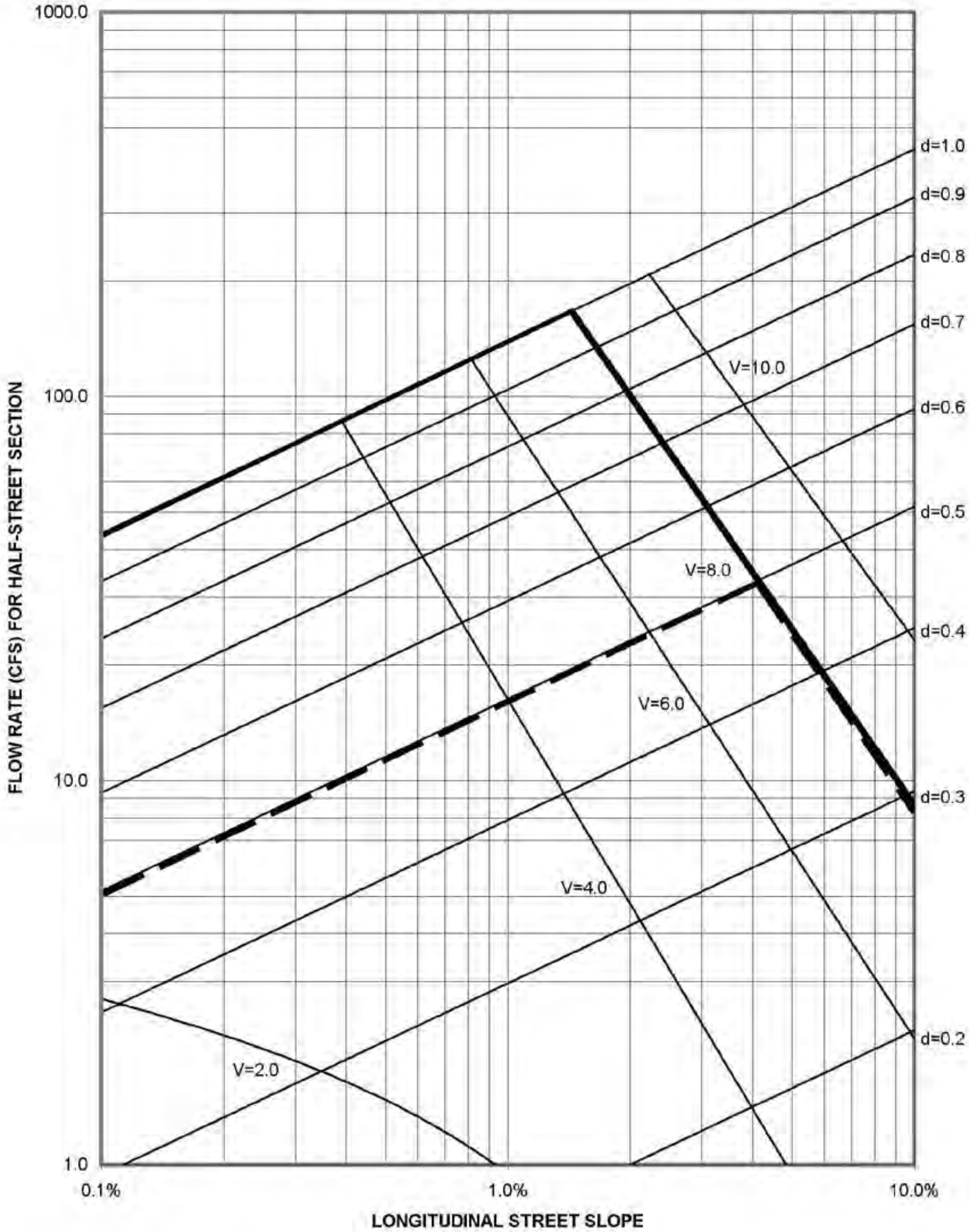
**FIGURE 1108**

## HALF-STREET FLOW CAPACITY
## (MINOR ARTERIAL)




**DESIGN LIMITS**

— — — MINOR STORM

——— MAJOR STORM

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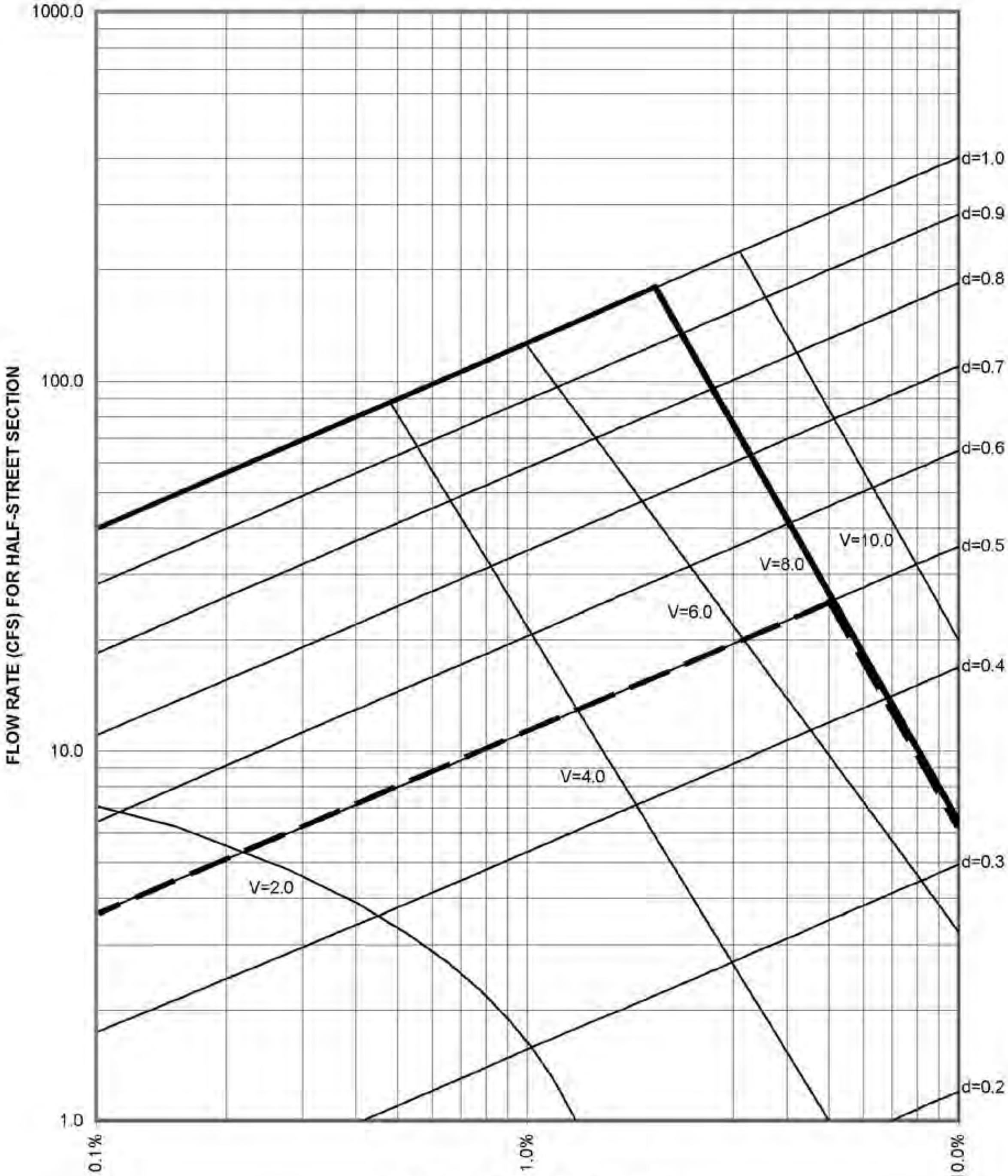
**FIGURE 1109**

# STORMWATER MANAGEMENT MANUAL

## HALF-STREET FLOW CAPACITY
## (PRINCIPAL ARTERIAL)




**DESIGN LIMITS**

— — — MINOR STORM

——— MAJOR STORM

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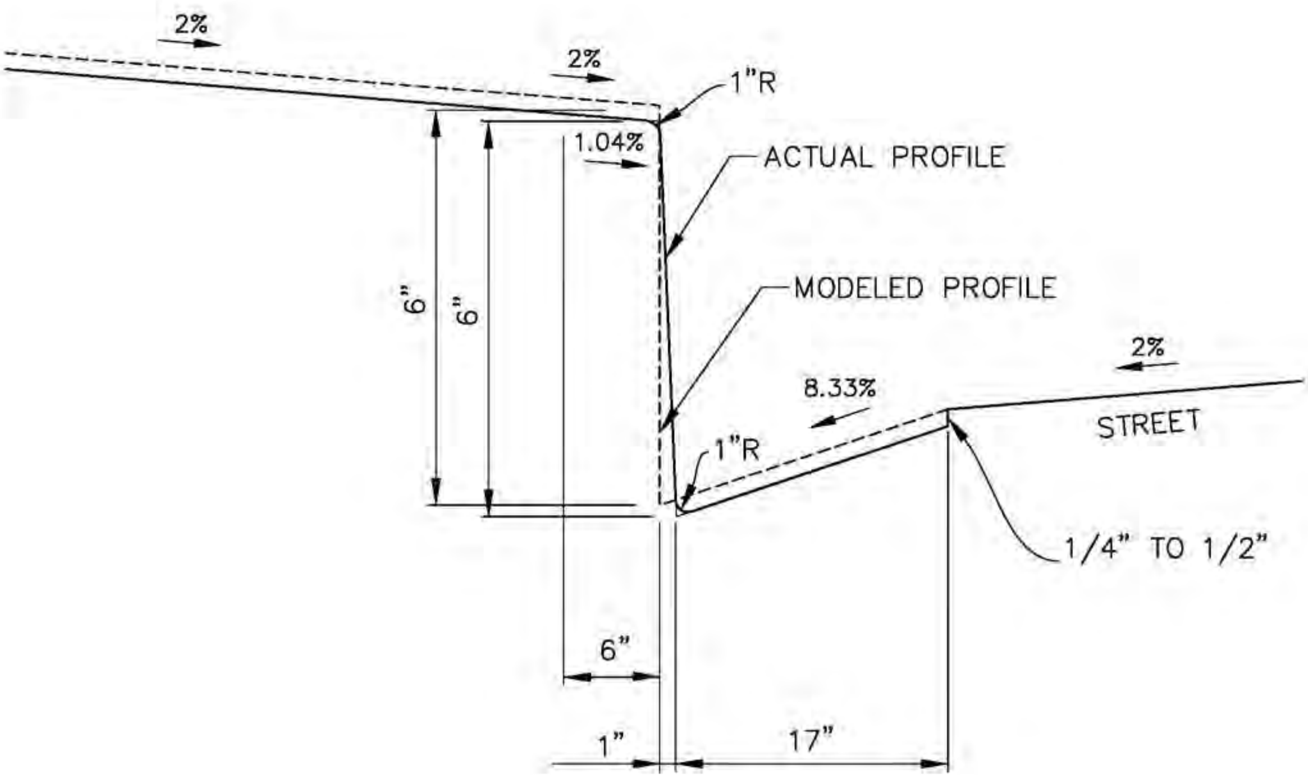
**FIGURE 1110**

# STORMWATER MANAGEMENT MANUAL

## ACTUAL VS. MODELED
## VERTICAL CURB DETAIL

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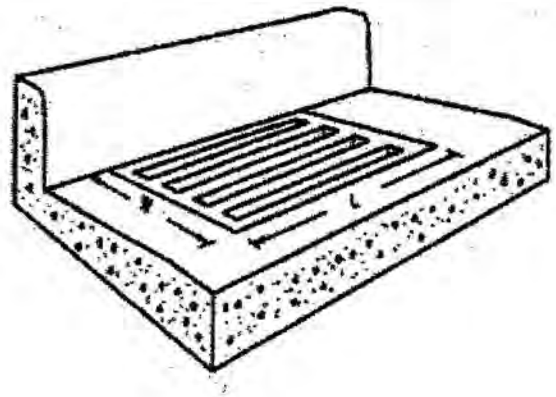
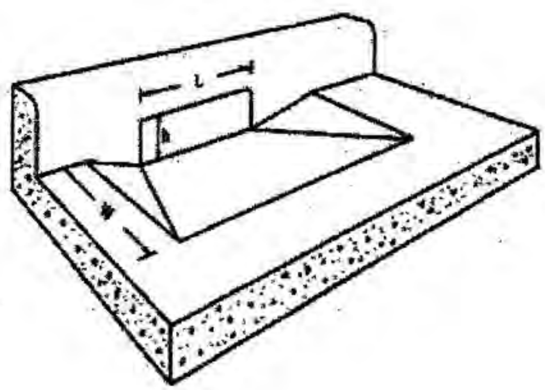
REFERENCE: GRAND JUNCTION STANDARD CONCRETE DETAILS
HEC-12, FHWA

**FIGURE 1111**

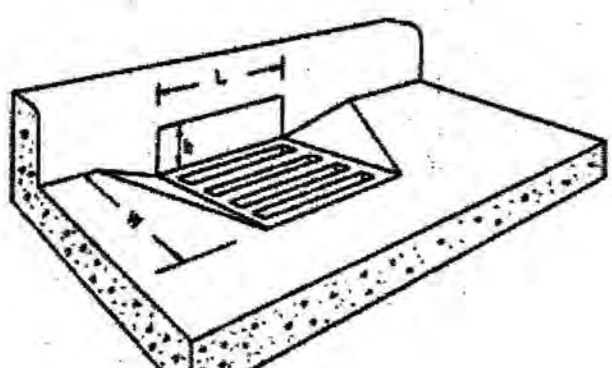
## INLET TYPES ISOMETRIC VIEWS




a. Grate




b. Curb-opening Inlet



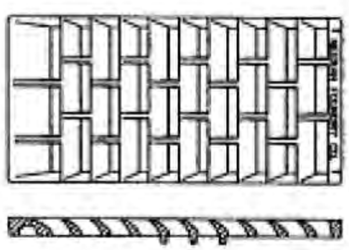

c. Combination Inlet

| Revision | Date |
|---|---|
| ORIGINAL ISSUE | 3/27/06 |
| | |
| | |

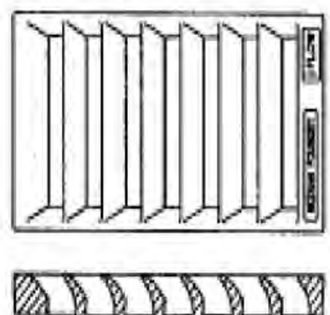
**Chart Legend**
**Capacities for On-Grade Inlets**

| | |
|---|---|
| ——— (light blue) | Grate-Only Inlets |
| ——— (dark blue) | Combination Inlets |
| ——— (red) | Curb-Opening Inlets |
| ——— | Single Inlets |
| — · · — · | Double Inlets |
| — — — | Triple Inlets |

Standard Single Grate-Only or Combination Inlet
Length = 3'
Standard Single Curb-Opening Length = 5'



**Grate Type L**



**Grate Type V**

**Grate Type D**

**Grate Type R w/Curb Opening**

| Revision | Date |
|---|---|
| ORIGINAL ISSUE | 3/27/06 |
| ADDED GRATE DIAGRAMS | 12/6/07 |
| FIGURE VARIABLES | 12/31/07 |

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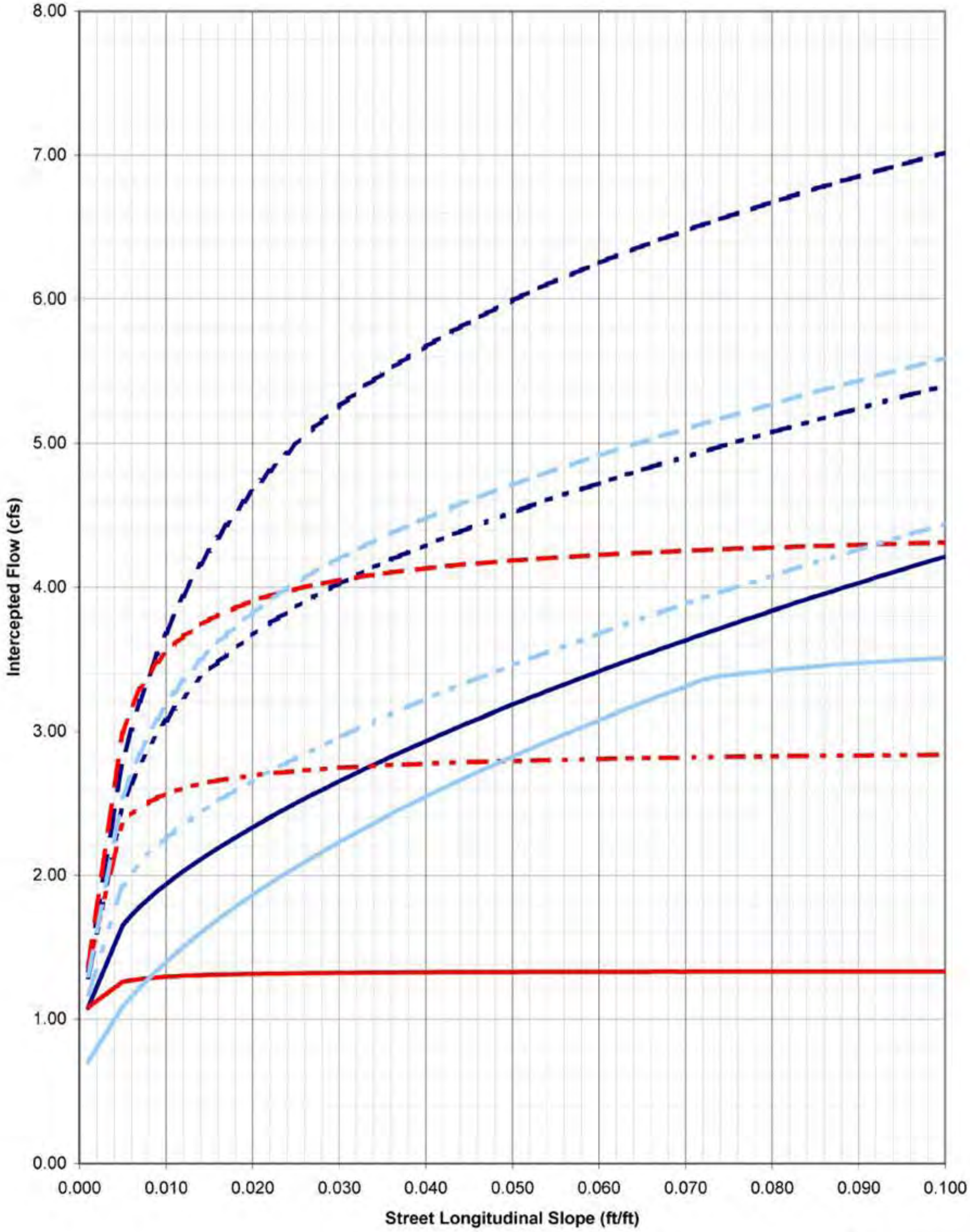
REFERENCE:

**FIGURE 1113**

# STORMWATER MANAGEMENT MANUAL

**Capacities for On-Grade Inlets (Grate Type D)**

**Residential Street, Mountable Curb**




| Revision | Date |
|---|---|
| ORIGINAL ISSUE | 3/27/06 |
| FIGURE VARIABLES | 12/31/07 |
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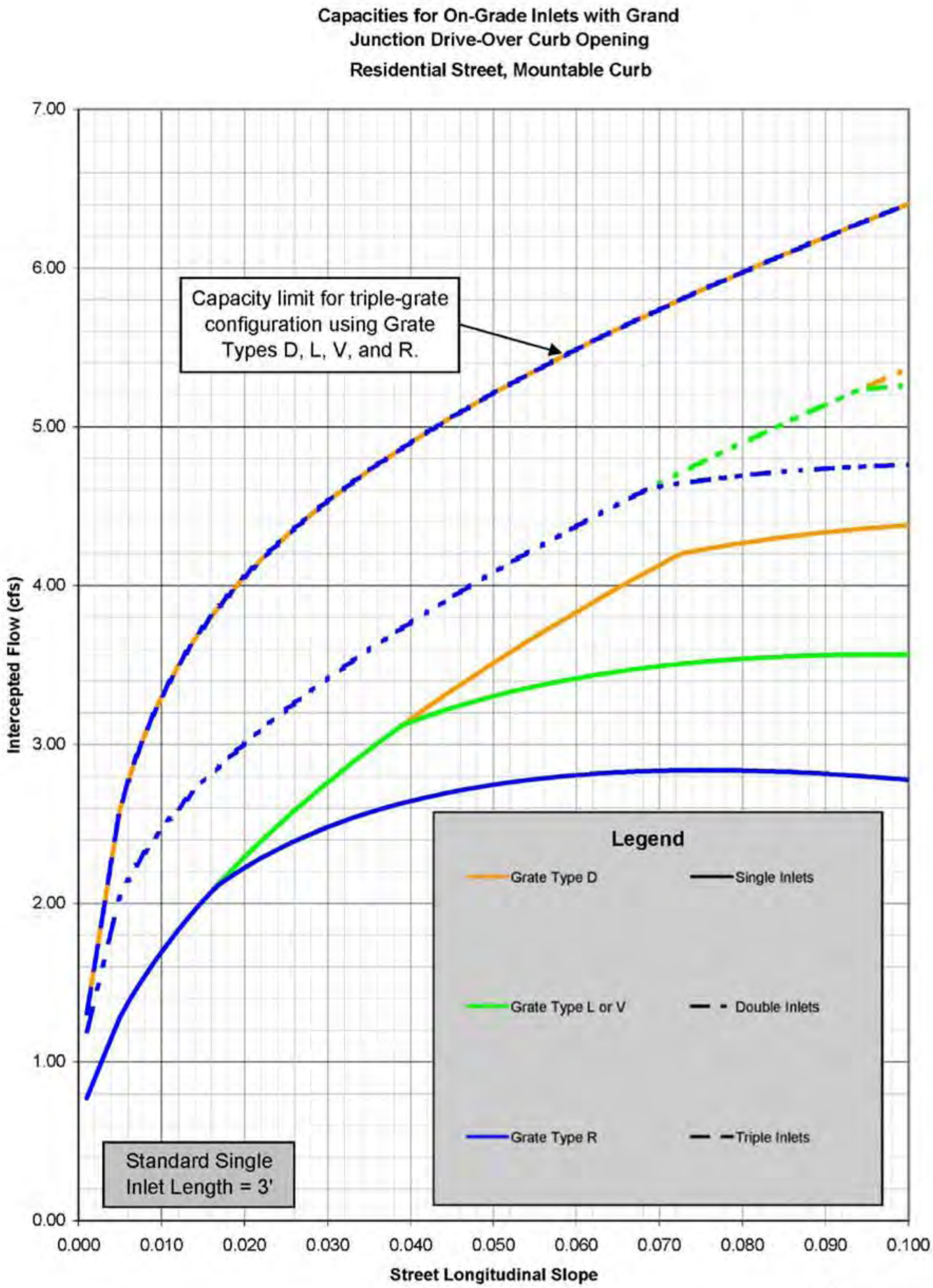
REFERENCE:

**FIGURE 1114D**

# STORMWATER MANAGEMENT MANUAL




| Revision | Date |
|---|---|
| ORIGINAL ISSUE | 12/31/07 |
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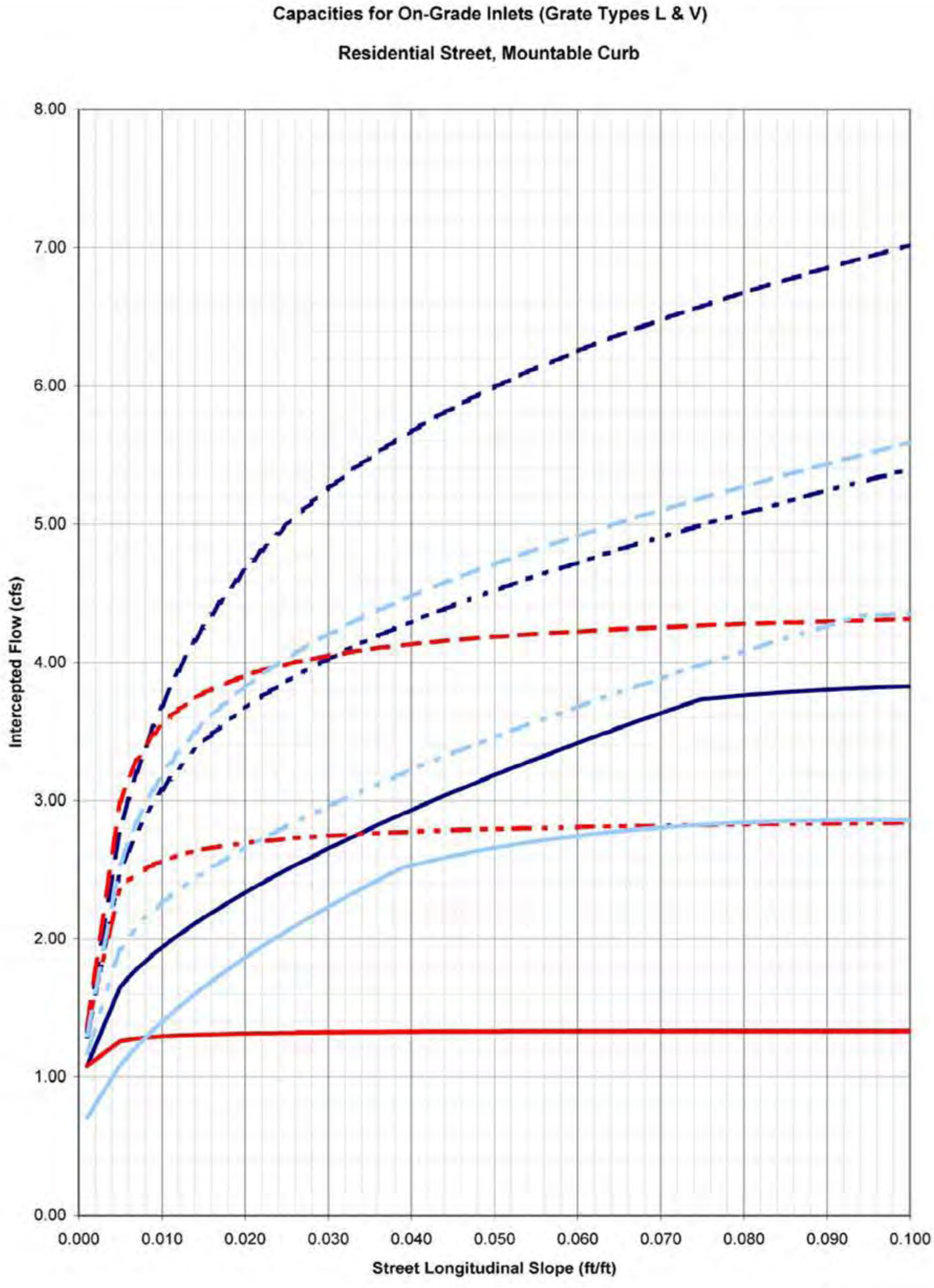
**FIGURE 1114GJ**




| Revision | Date |
|---|---|
| ORIGINAL ISSUE | 12/6/07 |
| FIGURE VARIABLES | 12/31/07 |
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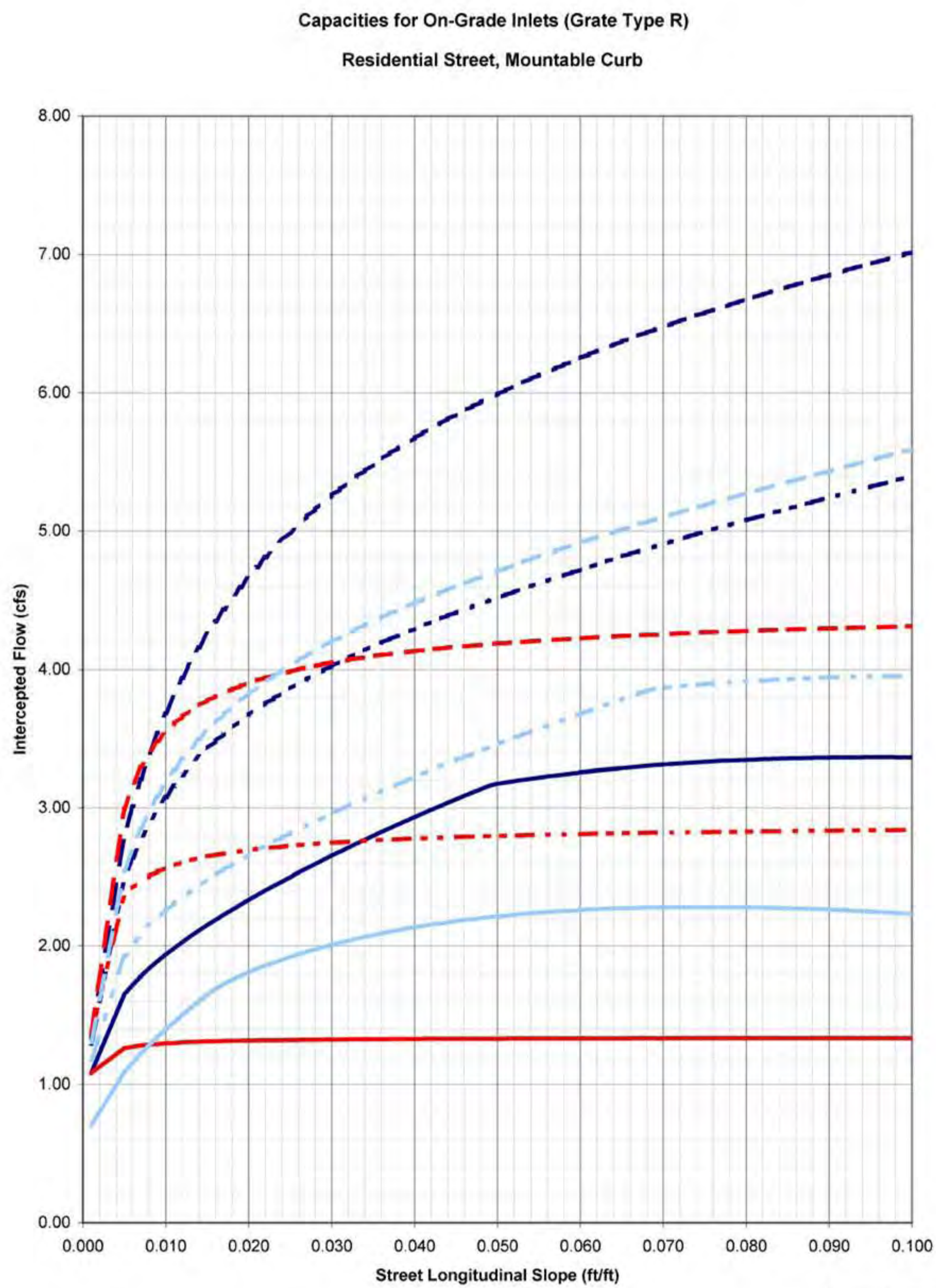
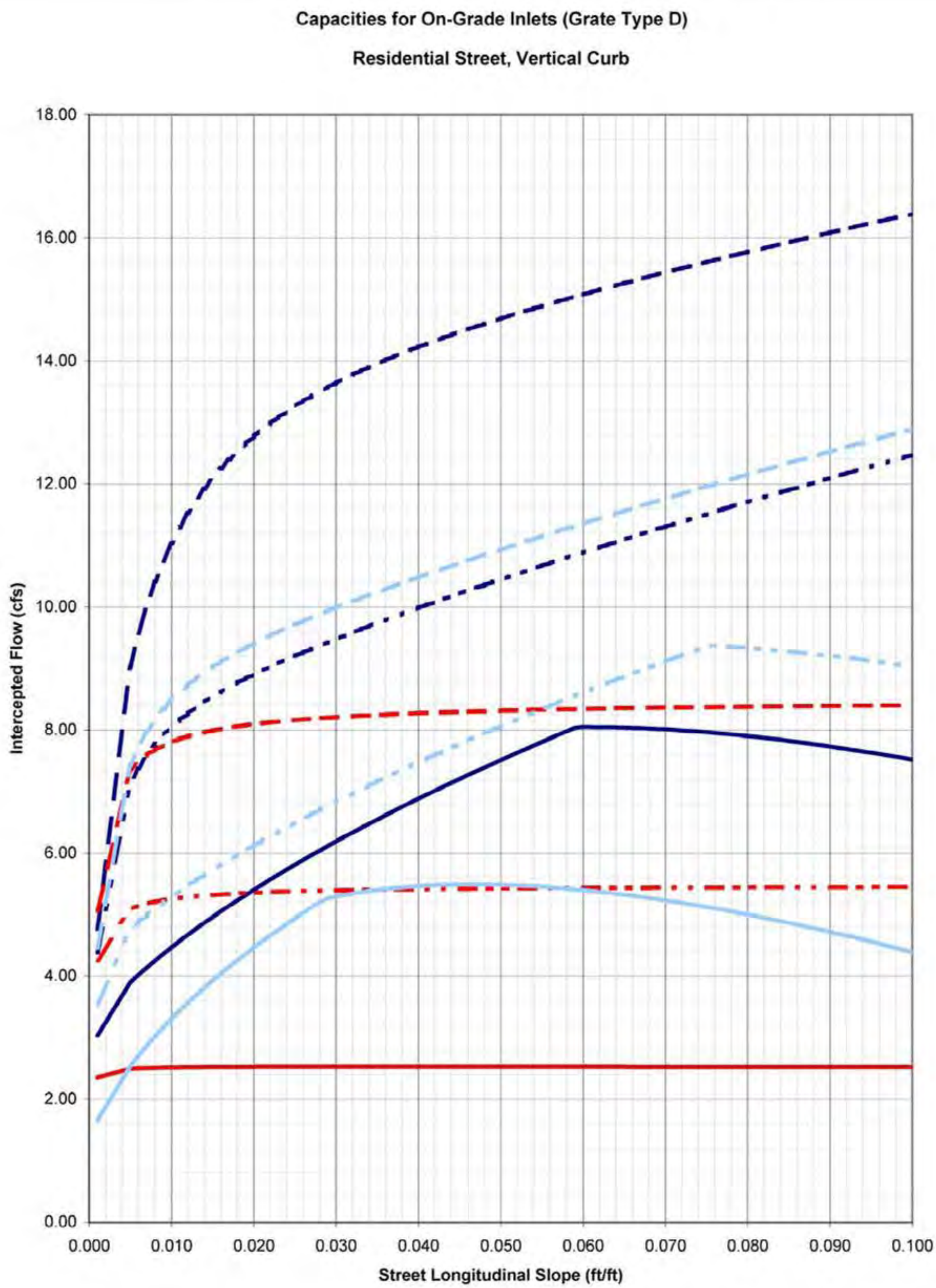
REFERENCE:

**FIGURE 1114LV**




| Revision | Date |
|---|---|
| ORIGINAL ISSUE | 3/27/06 |
| FIGURE VARIABLES | 12/31/07 |
| | |

# STORMWATER MANAGEMENT MANUAL




| Revision | Date |
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| ORIGINAL ISSUE | 3/27/06 |
| FIGURE VARIABLES | 12/31/07 |
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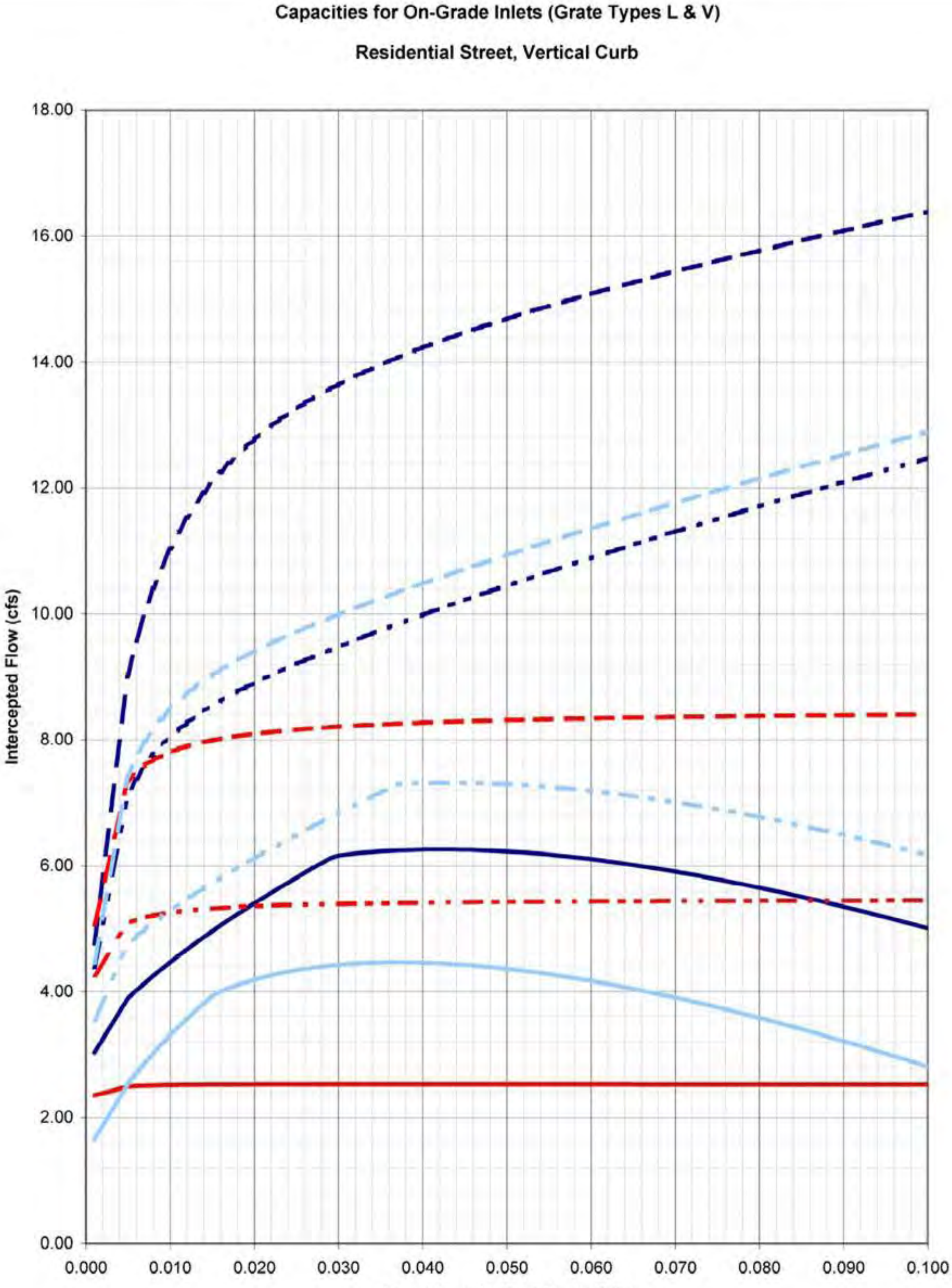
REFERENCE:

**FIGURE 1115D**




| Revision | Date |
|---|---|
| ORIGINAL ISSUE | 3/27/06 |
| FIGURE VARIABLES | 12/31/07 |
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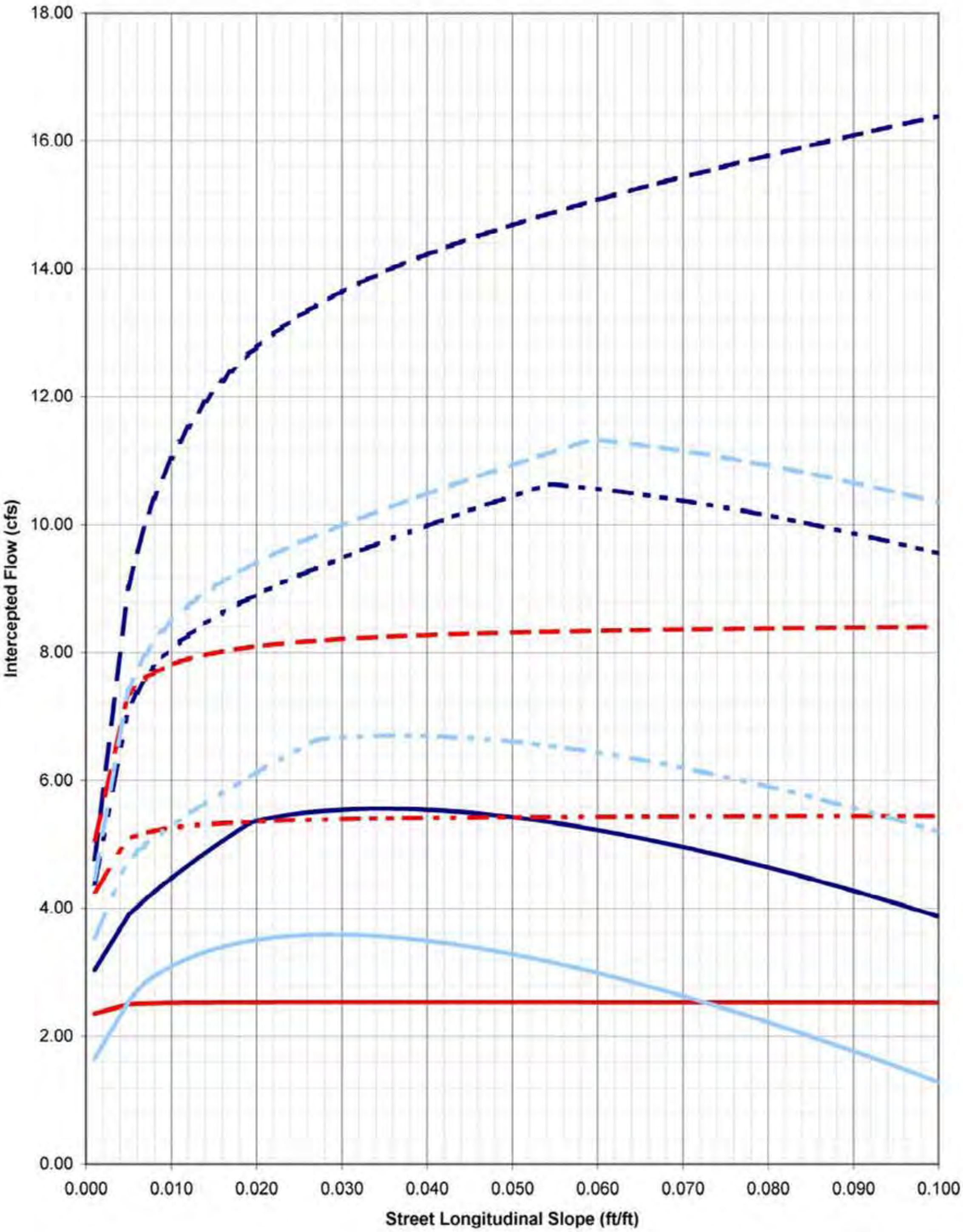
WRC ENGINEERING, INC.

REFERENCE:

FIGURE 1115LV

**Capacities for On-Grade Inlets (Grate Type R)**

**Residential Street, Vertical Curb**

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| ORIGINAL ISSUE | 3/27/06 |
| FIGURE VARIABLES | 12/31/07 |
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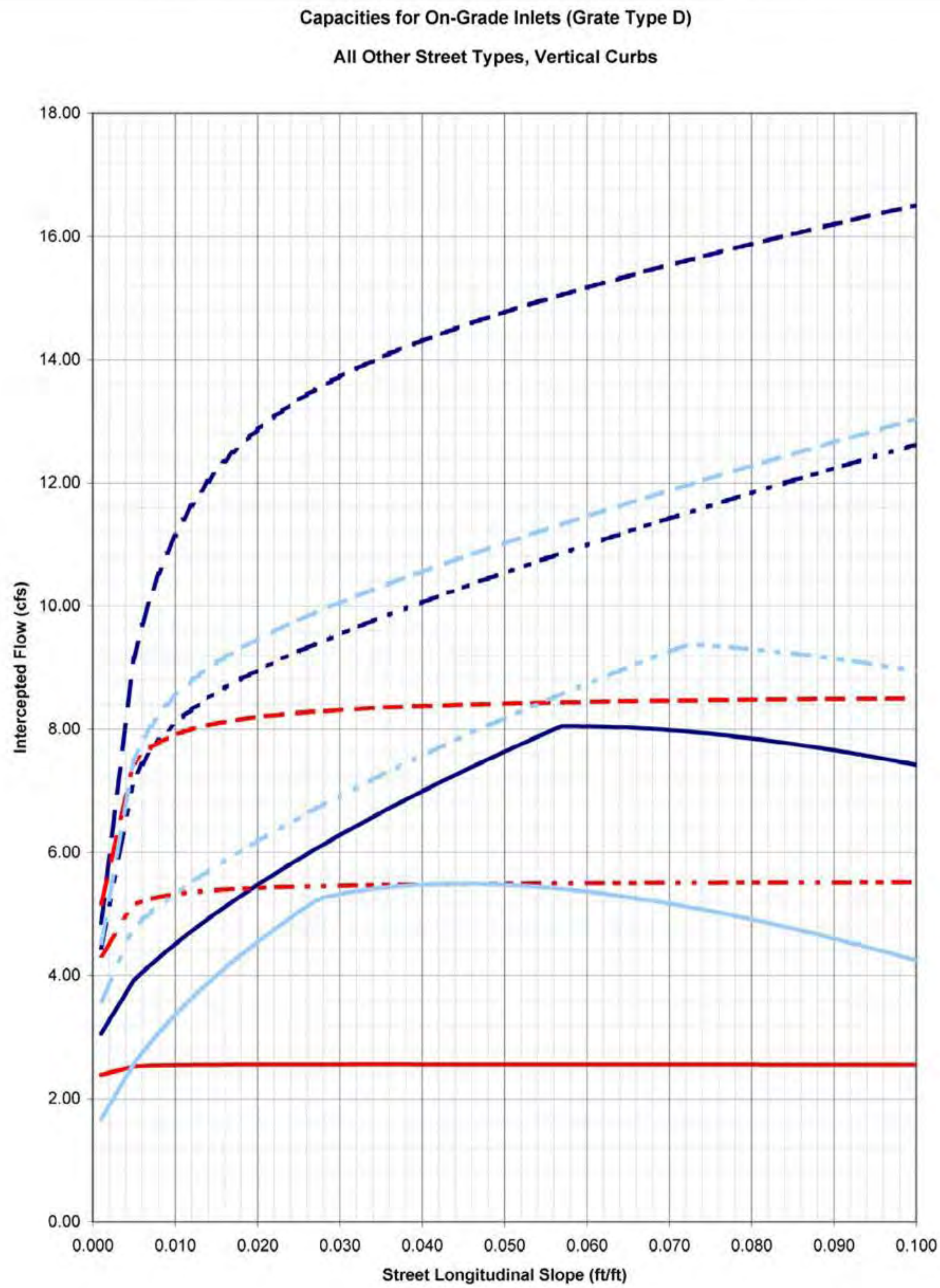
WRC ENGINEERING, INC.

REFERENCE:

**FIGURE 1115R**




| Revision | Date |
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| ORIGINAL ISSUE | 3/27/06 |
| FIGURE VARIABLES | 12/31/07 |
| | |

WRC ENGINEERING, INC.

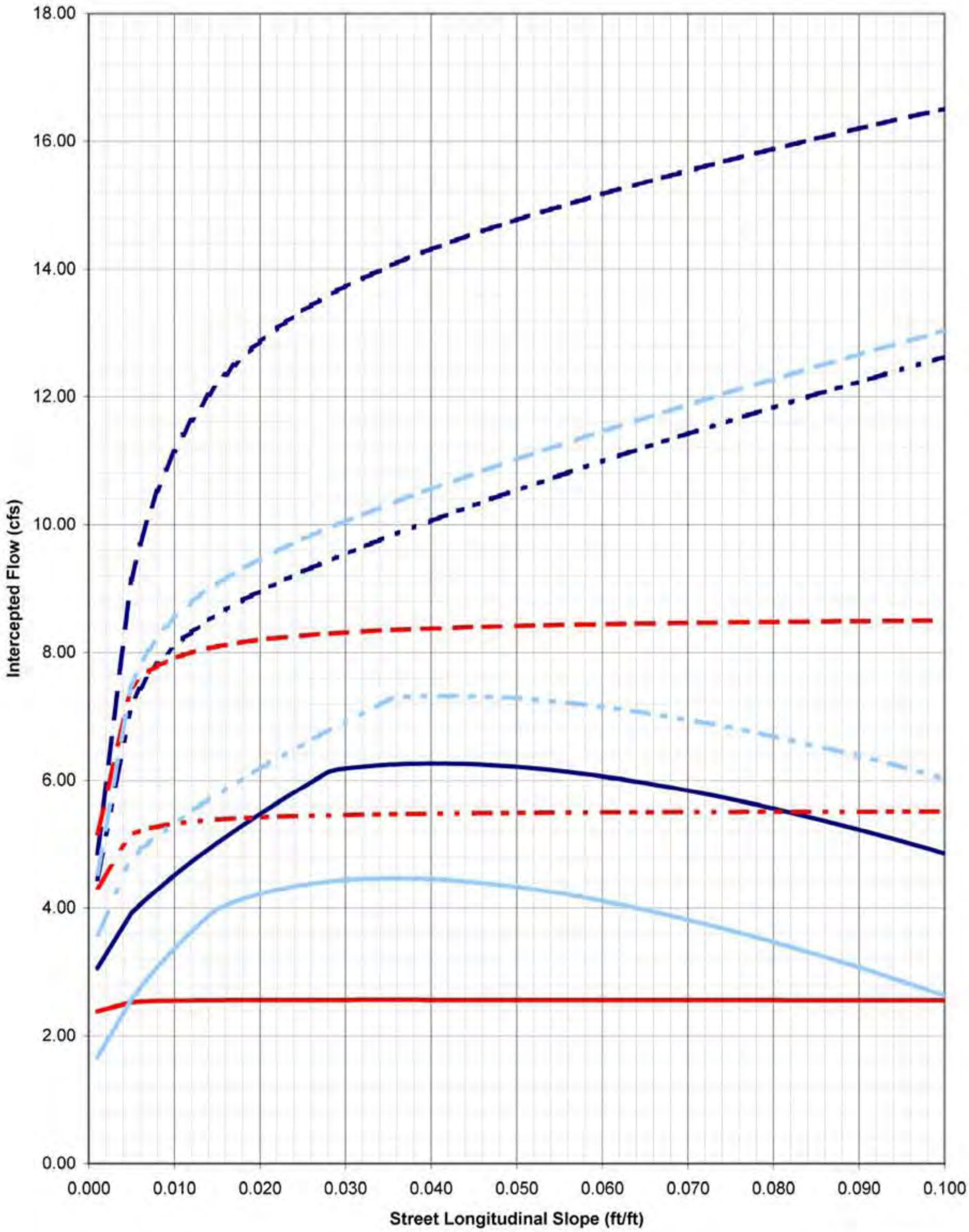
REFERENCE:

**FIGURE 1116D**

# STORMWATER MANAGEMENT MANUAL

**Capacities for On-Grade Inlets (Grate Types L & V)**

**All Other Street Types, Vertical Curbs**




| Revision | Date |
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| ORIGINAL ISSUE | 3/27/06 |
| FIGURE VARIABLES | 12/31/07 |
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WRC ENGINEERING, INC.

REFERENCE:

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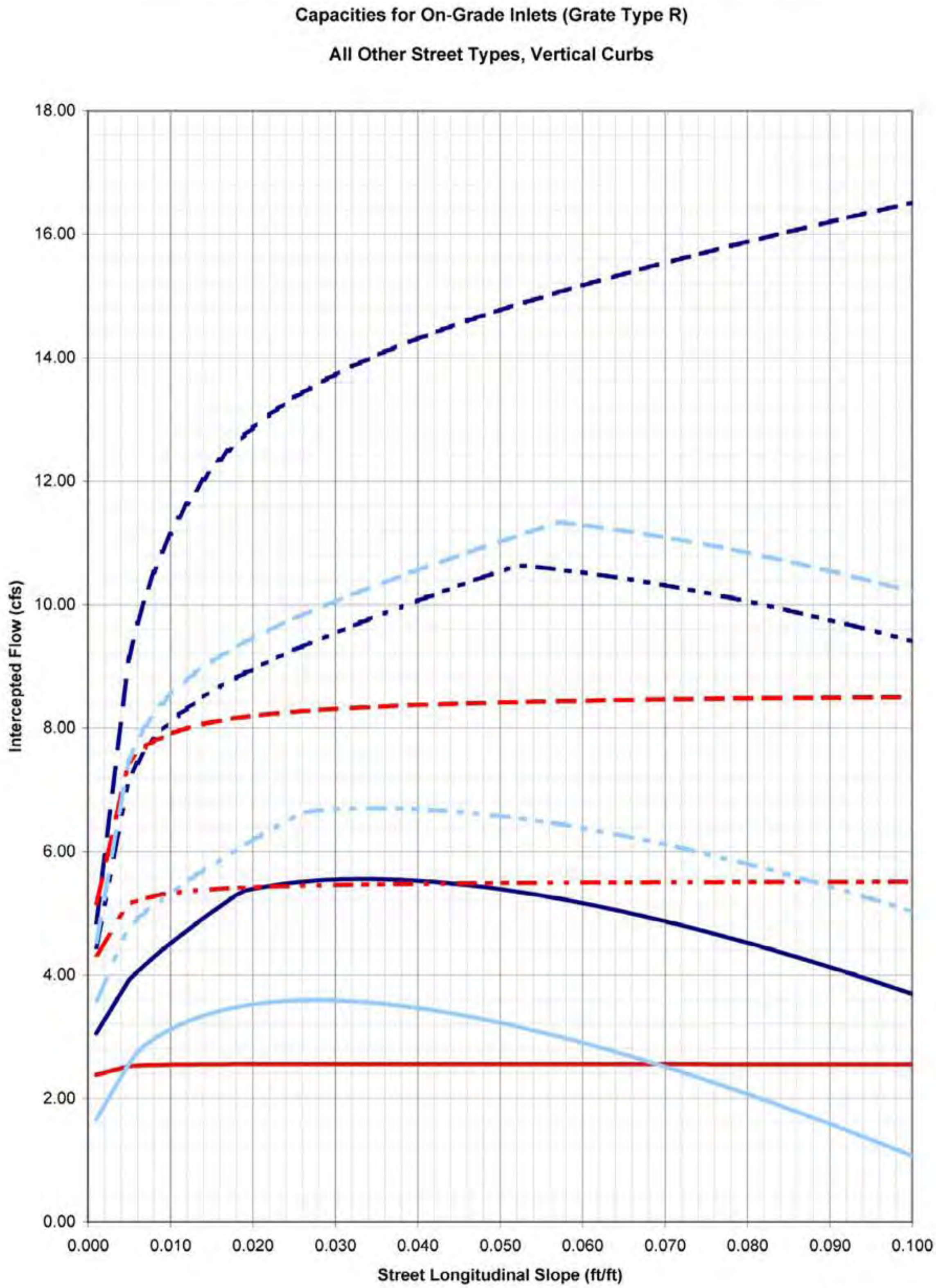
**FIGURE 1116LV**

# STORMWATER MANAGEMENT MANUAL

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| ORIGINAL ISSUE | 3/27/06 |
| FIGURE VARIABLES | 12/31/07 |
| | |

WRC ENGINEERING, INC.

REFERENCE:

**FIGURE 1116R**

**Maximum Inlet Capacities**
**Sump or Sag Condition**

| | INLET TYPE | 6-INCH VERTICAL CURB | | | | | |
|---|---|---|---|---|---|---|---|
| | | SINGLE | | DOUBLE | | TRIPLE | |
| | | 2-YR | 100-YR | 2-YR | 100-YR | 2-YR | 100-YR |
| 2-INCH CURB-OPENING DEPRESSION | COMBINATION INLET (TYPE D GRATES) | 9.8 | 12.4 | 14.7 | 20.1 | 19.6 | 27.8 |
| | COMBINATION INLET (TYPE R GRATES) | 9.8 | 11.1 | 14.7 | 18.8 | 19.6 | 26.5 |
| | CURB-OPENING INLET CAPACITY | 7.7 | 10.3 | 12.7 | 20.6 | 15.0 | 30.9 |
| NO CURB-OPENING DEPRESSION | COMBINATION INLET (TYPE D GRATES) | 6.4 | 9.3 | 9.5 | 14.2 | 12.7 | 19.1 |
| | COMBINATION INLET (TYPE R GRATES) | 5.1 | 8.1 | 9.5 | 13.0 | 12.7 | 17.9 |
| | CURB-OPENING INLET CAPACITY | 4.1 | 6.5 | 8.3 | 13.1 | 12.4 | 19.6 |

| | INLET TYPE | 4.5-INCH MOUNTABLE CURB | | | | | |
|---|---|---|---|---|---|---|---|
| | | SINGLE | | DOUBLE | | TRIPLE | |
| | | 2-YR | 100-YR | 2-YR | 100-YR | 2-YR | 100-YR |
| 2-INCH CURB-OPENING DEPRESSION | COMBINATION INLET (TYPE D GRATES) | 7.2 | 10.8 | 10.8 | 16.8 | 14.4 | 22.7 |
| | COMBINATION INLET (TYPE R GRATES) | 7.2 | 9.4 | 10.8 | 15.4 | 14.4 | 21.4 |
| | CURB-OPENING INLET CAPACITY | 5.6 | 8.0 | 9.3 | 16.0 | 11.0 | 23.9 |
| NO CURB-OPENING DEPRESSION | COMBINATION INLET (TYPE D GRATES) | 4.1 | 7.8 | 6.2 | 10.9 | 8.3 | 14.1 |
| | COMBINATION INLET (TYPE R GRATES) | 4.1 | 6.5 | 6.2 | 9.7 | 8.3 | 12.8 |
| | CURB-OPENING INLET CAPACITY | 2.3 | 4.2 | 4.7 | 8.5 | 7.0 | 12.7 |

**See Chart Legend (Figure 1113) for standard inlet lengths.**

Inlet capacities shown above are based upon the following:
1. Type D grate used for calculation is Neenah model R-3577.
2. Type R grate used for calculation is Neenah model R-3289-C.
3. Angled- and curved-vane grates are not allowed for sump or sag design conditions.
4. Capacities shown are based upon maximum ponding depths for the 2-year and 100-year storm events:
    a. 2-year event maximum ponding depth: curb height
    b. 100-year event maximum ponding depth: 1.0 foot
5. Combination inlets are preferred for sump or sag conditions. Curb-opening inlets without grates are allowed.
6. Grate-only inlets are not allowed for sump or sag conditions.

| Revision | Date |
|---|---|
| ORIGINAL ISSUE | 3/27/06 |
| REVISED CALCULATIONS | 12/20/07 |

WRC ENGINEERING, INC.

REFERENCE: HEC-12, FHWA
WATER-RESOURCES ENGINEERING, LINSLEY (1992)
GRAND JUNCTION STANDARD DETAILS

**FIGURE 1117**

# SECTION 1200

## CULVERTS AND BRIDGES

# SECTION 1200
# CULVERTS AND BRIDGES

## TABLE OF CONTENTS

## LIST OF TABLES



## LIST OF FIGURES

# SECTION 1200

## CULVERTS AND BRIDGES

### 1201 INTRODUCTION

The purpose of this Section is to provide information for the hydraulic design of culverts and bridges. The methodology of culvert and bridge design presented in this Section is intended for those with a good understanding of basic hydrologic and hydraulic methods and with experience in the design of hydraulic structures. The experienced designer is assumed to be able to understand the variety of flow conditions which are possible in these complex hydraulic structures and make appropriate adjustments.

Culverts and bridges are used to convey surface water through or beneath engineered structures such as highways, railroads, or other embankments. The size, alignment, and support structures of a culvert or bridge will directly affect the carrying capacity of the drainage system. Inadequate culvert or bridge capacity can force water out of the conveyance system, flood an alternate path, and cause damage away from the channel.

In addition to the hydraulic function, culverts and bridges must carry construction, highway, railroad, or other traffic and earth loads. Therefore, culvert and bridge design involves both hydraulic and structural design considerations. The hydraulic aspects of culvert and bridge design are covered in this Section.

A careful approach to culvert and bridge design is essential, both in new land development and retrofit situations, because culverts and bridges often significantly influence upstream and downstream flood risks, floodplain management, and public safety.

### 1202 CULVERT HYDRAULICS

This Section presents the general procedures for hydraulic design and evaluation of culverts. The user is assumed to posses a basic knowledge of culvert hydraulics and is encouraged to review the technical literature on the subject.

The information and references necessary to design culverts according to the procedure given in this Section can be found in the Federal Highway Administration (FHWA) *Hydraulic Design of Highway Culverts,* Hydraulic Design Series No. 5 (HDS-5). Research by the National Bureau of Standards sponsored and supported by the FHWA resulted in a series of reports providing a comprehensive analysis of culvert hydraulics under various flow conditions. These data were used to develop culvert design aids, called nomographs. These nomographs are the basis for the culvert design process. The more commonly used nomographs from HDS-5 are provided in this Section. HDS-5 is the required method for design of culverts in Mesa County and in the City of Grand Junction and shall be used for all situations not covered in this Section.

Inlet and outlet control are the two basic types of flow in culverts. 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 primarily by culvert slope, roughness, and tailwater elevation.

When designing a culvert, the designer must evaluate both inlet and outlet control conditions for the given design constraints (e.g. discharge, headwater, tailwater, outlet velocity, etc.). The control condition which produces the greater energy loss for the design condition determines the appropriate control to use for the culvert design. Culvert hydraulic calculations shall be performed using rating nomographs and/or culvert hydraulic analysis programs.

### 1202.1 Required Design Information

1202.1.1 Discharge

Culverts are required where natural or manmade channels are crossed by roads or streets. The amount of channel flow which crosses over the road shall 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 Mesa County and the City of Grand Junction shall be to require culvert crossings of streets within the following limitations:

| Right-Of-Way Width | Minimum Capacity (Recurrence Interval) |
|---|---|
| Major and minor arterial highways/greater than or equal to 80 feet | 100-year (No Overflow) |
| Collector and local streets/ less than 80 feet | 100-year (See Note) |

Note: A dipped overflow section may be allowed by the local jurisdiction if the maximum velocity does not exceed 6 feet per second and the maximum depth does not exceed 1.0 feet at the street crown.

All other culverts not crossing streets shall be designed to convey the 100-year flow unless otherwise approved by the local jurisdiction. In all cases 1.0 feet of freeboard must be maintained between the lowest point of the drive lane and the minor storm water surface elevation. See Section 404.2 for additional discussion.

1202.1.2 Headwater

Energy is required to force flow through a culvert. This energy takes the form of an increased water surface elevation on the upstream side of the culvert. The depth of the upstream water surface measured from the invert at the culvert entrance is referred to as headwater. The maximum headwater for the 100-year design flow shall be 1.5 times the culvert diameter or culvert rise dimension.

A considerable volume of water may be ponded upstream of a culvert installation under high fills or in areas with flat ground slopes. The pond which is created may attenuate flood peaks under such conditions. This peak discharge attenuation may justify a reduction in the required culvert size, provided it is in compliance with applicable floodplain regulations.

1202.1.3 Tailwater

Tailwater is defined as the depth of water downstream of the culvert measured from the outlet invert. Tailwater may be caused by an obstruction in the downstream channel or by the hydraulic resistance of the channel. Backwater calculations from the downstream control point are required to precisely define tailwater. When appropriate, normal depth approximations may be used instead of backwater calculations.

1202.1.4 Outlet Velocity

Since a culvert usually constricts the available channel area, flow velocities in the culvert are likely to be higher than in the channel. These increased velocities can cause streambed scour and bank erosion in the vicinity of the culvert outlet. Minor problems can occasionally be avoided by increasing the barrel roughness.

The design of culverts shall include an analysis of the channel stability at the outlet to be governed by a maximum permissible mean velocity. This design method assumes that a given channel section will remain stable up to a maximum permissible velocity. **Table 805** in Section 800 *Open Channels* presents the maximum permissible velocities for several types of natural, improved, unlined, and lined channels. In the case of unlined channels, velocities exceeding the values shown shall require outlet protection. Velocities shall not exceed the values given for lined channels. See Section 1203.9 for specific erosion protection requirements.

## 1202.2 Inlet Control

Inlet control occurs when the culvert barrel is capable of conveying more flow than the inlet will accept. This is controlled at the entrance by the depth of headwater, cross-sectional area, inlet edge configuration, and/or barrel shape. Under inlet control, the culvert barrel usually flows partially full. The control section of a culvert operating under inlet control is located just inside the entrance. Critical depth occurs at or near this location, and the flow regime immediately downstream is supercritical. Hydraulic characteristics downstream of the inlet control section do not affect the culvert capacity.

Inlet control for culverts may occur in two ways, unsubmerged or submerged. In an unsubmerged condition, the headwater is not sufficient to submerge the top of the culvert and the culvert slope is supercritical, shown in **Figure 1201** (Urban Drainage and Flood Control District). In this situation, the culvert inlet acts like a weir.

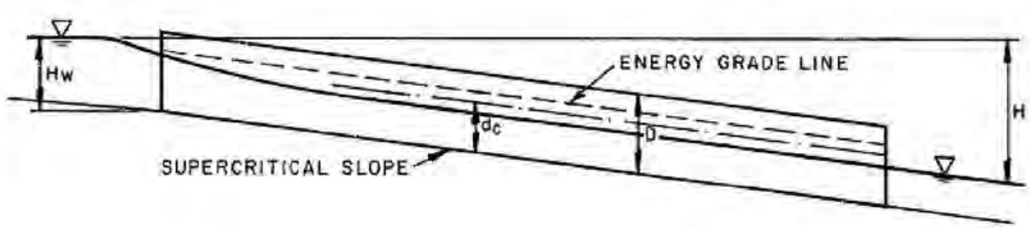


**Figure 1201** Inlet Control – Unsubmerged Inlet

In a submerged condition, the headwater submerges the top of the culvert but the pipe does not flow full, shown in **Figure 1202** (Urban Drainage and Flood Control District). In this situation, the culvert inlet acts like an orifice.

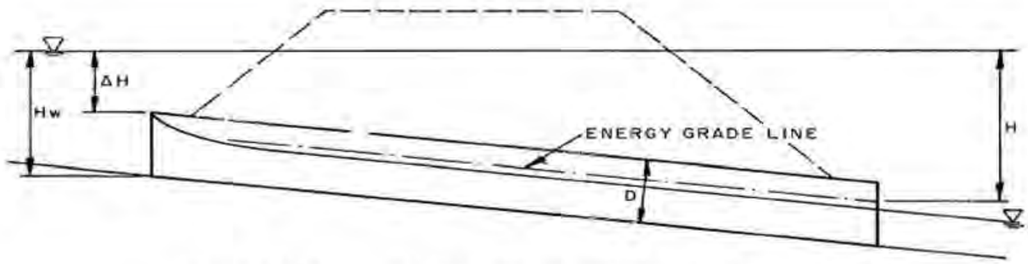


**Figure 1202** Inlet Control – Submerged Inlet

For a culvert operating with inlet control, the upstream water surface elevation and the inlet geometry represent the major flow controls. Roughness, slope, and length of the culvert barrel and outlet conditions, including tailwater, are not factors in determining culvert hydraulic performance.

### 1202.3 Outlet Control

Outlet control flow occurs when the culvert barrel is not capable of conveying as much flow as the inlet opening will accept. The control section for the outlet control flow in a culvert is located at the barrel exit or further downstream. Either subcritical or pressure flow exists in the culvert barrel under these conditions. All of the geometric and hydraulic characteristics of the culvert play a role in determining its capacity. These characteristics include all of the factors governing inlet control, the water surface elevation at the outlet, and the slope, length, and hydraulic roughness of the culvert barrel.

Outlet control will govern if the headwater and/or tailwater are/is high enough, the culvert slope is relatively flat, and the culvert relatively long. Outlet control will exist under two conditions. The first and least common is that where the headwater is insufficient to submerge the top of the culvert, and the culvert slope is subcritical, shown in **Figure 1203** (Urban Drainage and Flood Control District).

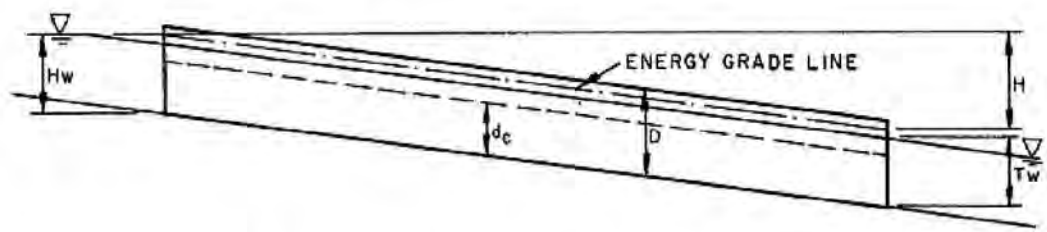


**Figure 1203** Partially Full Conduit

The most common condition exists when the culvert is flowing full, shown in **Figure 1204** (Urban Drainage and Flood Control District).

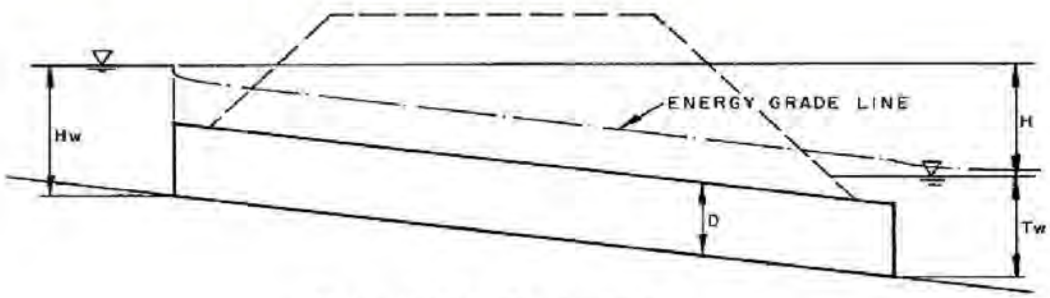


**Figure 1204** Full Conduit

Under outlet control, culverts may flow full or partly full depending on various combinations of the above factors. Performance of a culvert under outlet control may be affected by culvert length, roughness, and tailwater depth.

## 1203 CULVERT SIZING AND DESIGN

All culverts shall be designed and constructed using the following standards. The analysis and design shall consider design flow, culvert size and material, upstream channel and entrance configuration, downstream channel and outlet configuration, and erosion protection.

### 1203.1 Size and Material

Culverts shall be constructed from reinforced concrete, or acceptable polymers. For material other than concrete, documentation must be submitted for review which shows that the subject pipe material has a design equal to that of concrete and that the interior lining, if any, will maintain the design Manning's roughness coefficient, *n*, for the life of the pipe material. The minimum pipe size shall be a 12-inch diameter round pipe or shall have an equivalent 12-inch diameter round cross sectional area for other shapes. Corrosion resistance shall be evaluated and determined as specified in Section 624 of the Colorado Department of Transportation *Standard Specifications for Road and Bridge Construction*, 2005.

### 1203.2 Location

Culvert location is an integral part of the total design. The designer shall identify all live stream crossings, springs, low areas, gullies, and impoundment areas created by the new embankment for possible culvert locations. Culverts shall be located on existing stream alignments and aligned to give the stream a direct entrance and direct exit. Abrupt changes in direction at either end may retard the flow and make a larger structure necessary. If possible, a culvert shall have the same alignment as its channel. If this is not practical and the water must be turned into the culvert, headwalls, wingwalls, and aprons shall be used as protection against scour and to provide an efficient inlet.

Where the natural 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, an investigation into other options is recommended. Although the economic factors are important, the hydraulic effectiveness of the culvert must be given major consideration.

Roadway alignment also affects culvert design. The vertical alignment of roadways may define the maximum culvert diameter that can be used. Low vertical clearance may require the use of elliptical or arched culverts, or the use of a multiple-barrel culvert system. All culverts shall have a minimum of 1.5 feet of cover from top of asphalt (or gravel for gravel road) to outside top of pipe. Culverts for which less than 1.5 feet of cover is available will require additional structural analysis and other provisions (i.e. full depth concrete paving to compensate for the loss of proper cover).

### 1203.3 Culvert Design Form

The Culvert Design Form, shown in **Figure 1205**, has been formulated to guide the user through the design process. Summary blocks are provided at the top of the form for the project description and the designer's identification. Summaries of hydrologic data are also included. At the top right is a small sketch of the culvert with blanks for inserting important dimensions and elevations.

The central portion of the design form contains lines for inserting the trial culvert description and calculating the inlet and outlet control headwater elevations. Space is provided at the lower center for comments and at the lower right for a description of the culvert barrel selected.

The first step in the design process is to summarize all known data for the culvert at the top of the Culvert Design Form. This information will have been collected or calculated prior to performing the actual culvert design. The next step is to select a preliminary culvert material, shape, size, and entrance type. The user then enters the design flow rate and proceeds with the inlet control calculations.

The culvert design form is also included in Section 1700 as Standard Form 4.

### 1203.4 Inlet Control Calculation

The inlet control calculations determine the headwater elevation required to pass the design flow through the selected culvert configuration in inlet control. The approach velocity head may be included as part of the headwater, if desired. Inlet control nomographs are used in the design process. Inlet control nomographs for typical configurations are included at the end of this Section, **Figures 1219** through **1222**. For all other situations, refer to the FHWA's HDS-5. For the following discussion, refer to the schematic inlet control nomograph shown in **Figure 1206**.

1. Locate the selected culvert size (point 1) and flow rate (point 2) on the appropriate scales of the inlet control nomograph. For box culverts, the flow rate per foot of barrel width is used.

2. Using a straightedge, carefully extend a straight line from the culvert size (point 1) through the flow rate (point 2) and mark a point on the first headwater/culvert height (*HW/D*) scale (point 3). The first *HW/D* scale is also a turning line.

3. Each scale represents a different inlet type. If another *HW/D* scale is required, extend a horizontal line from the first *HW/D* scale to the desired scale and read the result.

4. Multiply *HW/D* by the culvert height, *D*, to obtain the required headwater depth, *HW*, from the invert of the control section to the energy grade line. If the approach velocity is neglected, *HW* equals the required headwater depth, $HW_i$. If the approach velocity is included in the calculations, deduct the approach velocity head from *HW* to determine $HW_i$.

5. Calculate the required depression (FALL) of the inlet control section below the stream bed as follows:

$$HW_d = EL_{hd} - EL_{sf} \tag{1201}$$

$$FALL = HW_i - HW_d \tag{1202}$$

Where:

$HW_d$ = design headwater depth (ft)
$EL_{hd}$ = design headwater elevation (ft)
$EL_{sf}$ = elevation of the stream bed at the face (ft)
$HW_i$ = required headwater depth (ft)

Possible results and consequences of this calculation are:

a. If the FALL is negative or zero, set FALL equal to zero and proceed to step 6.

b. If the FALL is positive, the inlet control section invert must be depressed below the streambed at the face by that amount. If the FALL is acceptable, proceed to step 6.

c. If the FALL is positive and greater than is judged to be acceptable, select another culvert configuration and begin again at step 1.

6. Calculate the inlet control section invert elevation as follows:

$$EL_i = EL_{sf} - FALL \tag{1203}$$

where $EL_i$ is the invert elevation at the face of a culvert or at the throat of a culvert with a tapered inlet.

### 1203.5 Outlet Control Calculation

The outlet control calculations result in the headwater elevation required to convey the design discharge through the selected culvert in outlet control. The approach and downstream velocities may be included in the design process, if desired. Critical depth charts and outlet control nomographs are used in the design process. Critical depth charts and outlet control nomographs for typical configurations are included at the end of this Section, **Figures 1223 through**

**1229**. For all other situations, refer to the FHWA's HDS-5. For illustration, refer to the schematic critical depth chart and outlet control nomograph shown in **Figures 1207 and 1208**, respectively.

1. Determine the tailwater depth, TW, above the outlet invert at the design flow rate. This is obtained from backwater or normal depth calculations, or from field observations.

2. Enter the appropriate critical depth chart, **Figure 1207**, with the flow rate and read the critical depth, $d_c$. The critical depth, $d_c$, cannot exceed the culvert diameter, D.

   The $d_c$ curves are truncated for convenience when they converge. If an accurate $d_c$ is required for $d_c > .9D$ consult the *Handbook of Hydraulics* or other hydraulic reference.

3. Calculate $(d_c + D)/2$

4. Determine the depth from the culvert outlet invert to the hydraulic grade line, $h_o$.

   $h_o$ = TW or $(d_c + D)/2$, whichever is larger

5. From **Table 1201**, obtain the appropriate entrance loss coefficient, $K_e$, for the culvert inlet configuration.

6. Determine the head losses through the culvert barrel, *H*, using the outlet control nomograph, **Figure 1208**.

   a. Required Manning's *n* values are presented in **Table 1002** of Section 1000 *Storm Sewer Systems*. If the Manning's *n* value given in the outlet control nomograph is different than the required Manning's *n* for the culvert, adjust the culvert length using the formula:

   $$L_1 = L(n_1/n)^2 \qquad (1204)$$

   Where:

   $L_1$ = adjusted culvert length (ft)
   L = actual culvert length (ft)
   $n_1$ = desired Manning's n value
   n = Manning's *n* value from the outlet control chart

   b. Using a straightedge, connect the culvert size (point 1) with the culvert length on the appropriate $k_e$ scale (point 2). This defines a point on the turning line (point 3)

   c. Again using the straightedge, extend a line from the discharge (point 4) through the point on the turning line (point 3) to the head loss, *H*, scale. *H* is the energy loss through the culvert, including entrance, friction, and outlet losses.

Careful alignment of the straightedge is necessary to obtain good results from the outlet control nomograph

7. Calculate the required outlet control headwater elevation, $EL_{ho}$.

$$EL_{ho} = EL_o + H + h_o \qquad (1205)$$

where $EL_o$ is the invert elevation at the outlet.

8. If the outlet control headwater elevation exceeds the design headwater elevation, a new culvert configuration must be selected and the process repeated. Generally, an enlarged barrel will be necessary since inlet improvements are of limited benefit in outlet control.

### 1203.6 Evaluation of Results

Repeat the design process until an acceptable culvert configuration is determined. If the culvert selected will not fit the site, return to the design process and select another culvert. Compare the headwater elevations calculated for inlet and outlet control. The higher of the two is designated the controlling headwater elevation. The culvert can be expected to operate with that higher headwater for at least part of the time.

If outlet control governs and the headwater depth is less than 1.2*D*, it is possible that the barrel flows partly full through its entire length. In this case, caution shall be used in applying the approximate method of setting the downstream elevation based on the greater of tailwater or $(d_c + D)/2$. If an accurate headwater is necessary, backwater calculations shall be used to check the result from the approximate method. If the headwater depth falls below 0.75*D*, the approximate method shall not be used.

Due to problems arising from topography and other considerations, the actual design of a culvert installation is more difficult than the simple process of sizing culverts. However, the process presented herein shall be followed to insure that some special problem is not overlooked. Several combinations of entrance types, invert elevations, and pipe diameters shall be tried to determine the most economic design that will meet the conditions imposed by topography and engineering.

### 1203.7 Outlet Velocity Calculation

The outlet velocity is calculated as follows:

1. If the controlling headwater is based on inlet control, determine the normal depth and velocity in the culvert barrel. The velocity at normal depth is assumed to be the outlet velocity.

2. If the controlling headwater is in outlet control, determine the area of flow at the outlet based on the barrel geometry and the following:

   a. Critical depth if the tailwater is below critical depth
   b. Tailwater depth if the tailwater is between critical depth and the top of the barrel

c. Height of the barrel if the tailwater is above the top of the barrel

### 1203.8 Computer Applications

Although the nomographs discussed in this Section are still used, engineers are increasingly designing culverts using computer applications. Among these applications are the FHWA's HY8 Culvert Analysis and numerous proprietary applications. If a computer application other than HY8 is to be used, the designer must first submit documentation of the program to the local jurisdiction for approval.

### 1203.9 Outlet Protection

**Table 805** in Section 800 *Open Channels* presents maximum permissible mean channel velocities. For unlined channels, erosion protection in the form of a horizontal riprap lined apron is required for all culvert outlets when the outlet velocity exceeds the values presented in **Table 805**. The length of the apron, $L_a$, is determined using the following empirical relationships that were developed for the U.S. Environmental Protection Agency:

$$L_a = \frac{1.8Q}{D_0^{3/2}} + 7D_0\text{, for } TW < \frac{D_0}{2} \qquad (1206)$$

and

$$L_a = \frac{3Q}{D_0^{3/2}} + 7D_0\text{, for } TW > \frac{D_0}{2} \qquad (1207)$$

Where:

$D_0$ = maximum inside culvert width (ft)

Q = pipe discharge (cfs)

TW = tailwater depth (ft)

Where there is no well-defined channel downstream of the apron, the width, *W*, of the apron shall be as follows, as shown in **Figure 1209**:

$$W = 3D_0 + 0.4L_a\text{, for } TW \geq \frac{D_0}{2} \qquad (1208)$$

and

$$W = 3D_0 + L_a\text{, for } TW < \frac{D_0}{2} \qquad (1209)$$

The width of the apron at the culvert outlet shall be at least 3 times the culvert width.

Where there is a well-defined channel downstream of the apron, the bottom width of the apron shall be at least equal to the bottom width of the channel and the lining shall extend at least 1.0 feet above the tailwater elevation and at least two thirds of the vertical conduit dimension above the invert.

The side slopes shall be 2:1 or flatter, the bottom grade shall be level, and there shall be overfall at the end of the apron or culvert.

The median stone diameter required, $d_{50}$, is determined from the following equation:

$$d_{50} = \frac{0.02(Q)^{4/3}}{TW(D_0)} \tag{1210}$$

Preformed scour holes may be used where flat aprons are impractical. **Figure 1210** shows a general design of a scour hole. The stone diameter is determined using the following equations:

$$d_{50} = \frac{0.0125(Q)^{4/3}}{TW(D_0)}, \text{ for } y = \frac{D_0}{2} \tag{1211}$$

and

$$d_{50} = \frac{0.0082(Q)^{4/3}}{TW(D_0)}, \text{ for } y = D_0 \tag{1212}$$

where y = depth of scour hole below culvert invert.

When riprap is not an option or is unavailable, the engineer has other natural and man-made materials available for use in erosion protection. Other options include gabions, concrete, recycled concrete, shotcrete, masonry, geotextiles, and woody plants. Outlet protection measures other than riprap may be used if first approved by the local jurisdiction.

### 1203.10 Trash/Safety Racks

The use of typical gratings at inlets to culverts and long underground pipes shall be considered on an individual basis. While there is a sound argument for the use of gratings for safety reasons, field experience has clearly shown that when the culvert is operating during heavy runoff, normal gratings often become clogged and the culvert is rendered ineffective. A general rule of thumb is that if it will be possible to 'see daylight' from one side of the culvert to the other, a trash/safety rack will not be needed. At entrances to longer culverts, a trash rack is necessary. Engineering judgment shall be used to determine if trash/safety racks will be used. Factors which may influence whether or not trash/safety racks will be used include the following:

1. Tributary land use (urban, rural, forest)
2. Location (urban/rural)
3. Design flow
4. Size of culvert
5. Anticipated debris loading
6. Performance of nearby existing structures

Trash/safety racks shall be required for all culverts located adjacent to schools, parks, playgrounds, and other recreational facilities where the pipe alignment or length does not allow for an unobstructed view through the culvert. Where it is found that trash/safety racks are needed, the open area through the grate at the design water surface shall be four times the design flow area of the culvert.

## 1204 CULVERT INLETS

One of the most important considerations in the design of a culvert is the inlet configuration. Since the natural channel is usually wider than the culvert barrel, the culvert inlet edge represents a flow contraction and may be the primary flow control. The provision of a more gradual flow transition will lessen the energy loss and thus create a more hydraulically efficient inlet condition. The inlet type can also increase the overall structural integrity by retaining fill slope and preventing inlet scour with subsequent undermining of the culvert. All culverts shall be designed either with headwalls and wingwalls or flared-end sections at the inlet and outlet.

A multitude of different inlet configurations are utilized on culvert barrels. These include both prefabricated and constructed-in-place installations. Commonly used inlet configurations include projecting culvert barrels, cast-in-place concrete headwalls, precast or prefabricated end sections, and culvert ends mitered to conform to the fill slope. Structural stability, aesthetics, erosion control, and fill retention are considerations in the selection of various inlet configurations.

### 1204.1 Projecting Inlets

Projecting inlets vary greatly in hydraulic efficiency and adaptability to requirements with the type of pipe material used. The primary advantage of projecting inlets is relatively low cost. Corrugated metal pipe projecting inlets have limitations which include low efficiency and susceptibility to damage. A projecting entrance of corrugated metal pipe is equivalent to a sharp-edged entrance with a thin wall and has an entrance coefficient of approximately 0.9, **Table 1201**. Bell-and-spigot concrete pipe or tongue-and-groove concrete pipe with the bell end or grooved end used as the inlet section are quite efficient hydraulically, having an entrance coefficient of approximately 0.2, **Table 1201**. For concrete pipe that has been cut, the entrance is square-edged, the entrance coefficient is approximately 0.5, **Table 1201**.

### 1204.2 Inlets with Headwalls

Headwalls may be used for a variety of reasons, including increasing the efficiency of the inlet, providing embankment stability, and providing embankment protection against erosion. The relative efficiency of the inlet varies with the pipe material used. Corrugated metal pipe in a headwall is essentially a squared-edged entrance with an entrance coefficient of approximately 0.5. The entrance losses may be reduced by rounding the entrance. For tongue-and-groove or bell-end concrete pipe, little increase in hydraulic efficiency is realized by adding a headwall. The primary reasons for using headwalls are embankment protection and ease of maintenance.

Wingwalls are used where the side slopes of the channel adjacent to the entrance are unstable and where the culvert is skewed to the normal channel flow. Little increase in hydraulic efficiency is realized with the use of wingwalls, regardless of the pipe material used and, therefore, the use shall be justified for reasons other than an increase in hydraulic efficiency.

### 1204.3 Tapered Inlets

Inlet configuration is one of the prime factors influencing the performance of a culvert operating under inlet control. Inlet edges can cause severe contraction of the flow, as in the case of a thin edge, projecting inlet. In a flow contraction, the effective cross-sectional area of the barrel may be reduced to about one half of the actual available barrel area. As the inlet configuration is improved, the flow contraction is reduced, thus improving the performance of the culvert.

A tapered inlet is a flared culvert inlet with an enlarged face section and a hydraulically efficient throat section. Tapered inlets improve culvert performance by providing a more efficient control section (the throat). However, tapered inlets are not recommended for use on culverts flowing under outlet control because the simple beveled edge is of equal benefit. The two most common improved inlets are the side-tapered inlet and the slope-tapered inlet. FHWA's HDS-5 provides guidance on the design of improved inlets.

## 1205 BRIDGE HYDRAULICS

Bridges are required across nearly all open channels sooner or later and, therefore, sizing the bridge openings is of great importance. Open channels with improperly designed bridges will either have excessive scour or deposition or not be able to carry the design flow. Confining flood waters by bridges can cause excessive backwater resulting in flooding of upstream property, backwater damage suits, overtopping of roadways, costly maintenance, or even loss of a bridge. Bridge openings shall be designed to have as little effect on the flow characteristics as reasonable, consistent with good design and economics.

### 1205.1 Hydraulic Analysis

The hydraulic analysis procedures described in this Section are suitable, although methods such as FHWA HY-4 or HEC-RAS are acceptable as well. If a computer application other than HY-4 or HEC-RAS is to be used, the designer must first submit documentation of the program to the local jurisdiction for approval. The preliminary assessment approach described below is presented in greater detail in the FHWA *Hydraulics of Bridge Waterways*.

### 1205.2 Expression of Backwater

A practical expression for backwater has been formulated by applying the principle of conservation of energy between the point of maximum backwater upstream from the bridge and a point downstream from the bridge at which normal stage has been reestablished, as shown in Section 4, of **Figure 1211**. The expression is reasonably valid if the channel in the vicinity of the bridge is reasonably uniform, the gradient of the bottom is approximately constant between Sections 1 and 4, there is no appreciable erosion of the bed in the constriction due to scour, and the flow is subcritical.

The expression for computation of backwater upstream from the bridge constricting the flow is as follows:

$$h_1^* = (K^*)\left(\frac{(V_{n2})^2}{2g}\right) + \propto 1\left[\left(\frac{A_{n2}}{A_4}\right)^2 - \left(\frac{A_{n2}}{A_1}\right)^2\right]\frac{V_{n2}{}^2}{2g} \qquad (1213)$$

Where:

$h_1^* =$ total backwater (ft)

$K^* =$ total backwater coefficient

$\propto 1 = \dfrac{qv^2}{QV_1{}^2} =$ kinetic energy coefficient

$A_{n2} =$ gross water area in constriction measured below normal stage (ft$^2$)

$V_{n2} =$ average velocity in constriction or $Q/A_{n2}$ (ft/s). The velocity $V_{n2}$ is not an actual measurable velocity but represents a reference velocity readily computed for both model and field structures

$A_4 =$ water area at Section 4 where normal stage is reestablished (ft$^2$)

$A_1 =$ total water area at Section 1 including backwater (ft$^2$)

$g =$ acceleration constant (32.2 ft/sec$^2$)

To compute backwater by Equation 1213, it is necessary to obtain the approximate value of $h_1^*$ by using the first part of the equation:

$$h_1^* = (K^*)\left(\frac{V_{n2}^2}{2g}\right) \qquad (1214)$$

The value of $A_1$ in the second part of Equation 1213, which depends on $h_1^*$, can then be determined. This part of the expression represents the difference in kinetic energy between Sections 4 and 1, expressed in terms of the velocity head $V_{n2}^2/2g$. Equation 1213 may appear cumbersome, but it was set up as shown to permit omission of the second part when the difference in kinetic energy between Sections 4 and 1 is small enough to be insignificant in the final result.

To permit the designer to readily recognize cases in which the kinetic energy term may be ignored, the following guides are provided:

$M > 0.7$, where $M =$ bridge opening ratio

$V_{n2} < 7$ ft/s

$(K^*)\left(\dfrac{V_{n2}^2}{2g}\right) < 0.5$ ft

If values meet all three conditions, the backwater obtained from Equation 1214 can be considered sufficiently accurate. Should one or more of the values not meet the conditions set forth, it is required to use Equation 1213 in its entirety. The use of the guides is further demonstrated in the examples given in FHWA *Hydraulics of Bridge Waterways* that shall be used in all bridge design work.

### 1205.3 Backwater Coefficient

The value of the overall backwater coefficient $K^*$, which was determined experimentally, varies with the following:

1. Stream constriction as measured by the bridge opening ratio, M
2. Type of bridge abutment: wingwall, spill through, etc.
3. Number, size, shape, and orientation of piers in the constriction
4. Eccentricity or asymmetric position of bridge with respect to the floodplain
5. Skew (bridge crosses floodplain at other than 90 degree angle)

The overall backwater coefficient $K^*$ consists of a base curve coefficient, $K_b$, to which are added incremental coefficients to account for the effect of piers, eccentricity, and skew. The value of $K^*$ is primarily dependent on the degree of constriction of the flow but also changes to a limited degree with the other factors.

### 1205.4 Effect of M and Abutment Shape (Base Curves)

**Figure 1212** shows the base curves for backwater coefficient, $K_b$, plotted with respect to the opening ratio, $M$, for several wingwall abutments and vertical wall type. Note how the coefficient $K_b$ increases with channel constriction. The several curves represent different angles of wingwalls as can be identified by the accompanying sketches. The lower curves represent the better hydraulic shapes.

**Figure 1213** shows the relation between the backwater coefficient, $K_b$, and $M$ for spill-through abutments for three embankment slopes. A comparison of the three curves indicates that the coefficient is little affected by embankment slope. **Figures 1212 and 1213** are 'base curves' and $K_b$ is referred to as the 'base curve coefficient.' The base curve coefficients apply to normal crossings for specific abutment shapes but do not include the effect of piers, eccentricity, or skew.

### 1205.5 Effect of Piers (Normal Crossings)

The effect on the backwater from introduction of piers in a bridge constriction has been treated as an incremental backwater coefficient designated $\Delta K_p$, which is added to the base curve coefficient when piers are a factor. The value of the incremental backwater coefficient, $\Delta K_p$, is dependent on the ratio that the area of the piers bears to the gross area of the bridge opening; the type of piers (or pilings in the case of pile bents); the value of the bridge opening ratio, $M$; and the angularity of the piers with the direction of flood flow. The ratio of the water area occupied by piers, $A_p$, to the gross water area of the constriction, $A_{n2}$, both based on the normal water surface, has been assigned the letter J. In computing the gross water area, $A_{n2}$, the presence of piers in the constriction is ignored. The incremental backwater coefficient for the more common types of piers and pile bents can be obtained from **Figure 1214**. The procedure is to enter Chart A, **Figure 1214**, with the proper value of J and read $\Delta K$ and obtain the correction factor $\sigma$ from Chart B, **Figure 1214**, for opening ratios other than unity. The incremental backwater coefficient is then:

$$\Delta K_p = \Delta K\sigma \qquad (1215)$$

The incremental backwater coefficients for piers can, for all practical purposes, be considered independent of diameter, width, or spacing but shall be increased if there are more than 5 piles in a bent. A bent with 10 piles shall be given a value of $\Delta K_p$ about 20 percent higher than those shown for bents with 5 piles. If there is a good possibility of trash collecting on the piers, it is advisable to use a value greater than the pier width to include the trash. For a normal crossing with piers, the total backwater coefficient becomes:

$$K^* = K_b\,(\textbf{Figures 1212} \text{ or } \textbf{1213}) + \Delta K_p\,(\textbf{Figure 1214}) \qquad (1216)$$

## 1206 BRIDGE SIZING AND DESIGN

The method of planning for bridge openings must include water surface profiles and hydraulic gradient analyses of the channel for the 100-year event. Once this hydraulic gradient is established without the bridge, the maximum reasonable effect on the channel flow by the bridge shall be determined.

### 1206.1 Bridge Design Standards

All bridges shall be designed to pass the 100-year design flow and meet all applicable floodplain regulations. The 100-year water surface elevation within the bridge shall be a minimum of 1.0 foot below the bridge low chord.

Where bridge abutments and foundations are located below the 100-year water surface elevation, concrete wingwalls at angles of 40 degrees to 60 degrees shall be tied to the existing side slopes to prevent erosion behind the abutments.

Where supercritical flow exists in a lined channel, the bridge shall have no influence on the flow. There shall be no encroachment into the 100-year water surface elevation.

The design and supporting calculations for both private bridges and low water crossings shall be prepared and certified by a Colorado Registered Professional Engineer.

### 1206.2 Design Procedure

The following is a brief step-by-step outline for determination of backwater produced by a bridge constriction:

1. Determine the magnitude and frequency of the discharge for which the bridge is to be designed.

2. Determine the stage of the stream at the bridge site for the design discharge.

3. Plot representative cross section of stream for design discharge at Section 1. If the stream is essentially straight and the cross section substantially uniform in the vicinity of the bridge, the natural cross section of the stream at the bridge site may be used for this purpose.

4. Subdivide the above cross section according to marked changes in depth of flow and roughness. Assign values of Manning's roughness coefficient, *n*, to each subsection. Typical roughness coefficients can be found in **Table 802** of Section 800 *Open Channels*. Careful judgment is necessary in selecting these values.

5. Compute conveyance and then discharge in each subsection.

6. Determine the value of the kinetic energy coefficient.

7. Plot the natural cross section under the proposed bridge based on normal water for design discharge and compute the gross water area, including area occupied by piers.

8. Compute the bridge opening ratio, *M*, observing modified procedure for skewed crossings.

9. Obtain the value of $K_b$ from the appropriate base curve.

10. If piers are involved, compute the value of J and obtain the incremental coefficient, $\Delta K_p$.

11. If eccentricity is severe, compute the value of eccentricity and obtain the incremental coefficient, $\Delta K_e$.

12. If a skewed crossing is involved, observe proper procedure in previous steps, then obtain the incremental coefficient, $\Delta K_s$, for proper abutment type.

13. Determine the total backwater coefficient, K*, by adding incremental coefficients to the base curve coefficient, $K_b$.

14. Compute the backwater by Equation 1213.

15. Determine the distance upstream to where the backwater effect is negligible.

Detailed steps illustrated by examples are presented in FHWA *Hydraulics of Bridge Waterways*.

## 1207 EXAMPLE APPLICATION

### 1207.1 Culvert Sizing

Problem: Determine the culvert size necessary to convey the 100-year discharge in Doe Creek beneath John Boulevard. The results of the analysis are shown in the Culvert Design Form of **Figure 1215**.

Top of road elevation = 4928 ft
Culvert inlet elevation = 4920 ft
Culvert outlet elevation = 4918 ft
Culvert length = 200 ft

Inlet - Groove end with headwall and wingwalls at 45°
Outlet - Groove end with headwall and wingwalls at 45°
Flow = 191 cfs
Tailwater Depth = 4 ft

Solution:

Step 1: Assume a reinforced concrete pipe diameter and determine the headwater to depth ratio for inlet control from **Figure 1219**. Assume a reinforced concrete pipe diameter of 5 feet. Given are a 100-year design flow of 191 cfs and a grooved end with headwall inlet configuration.

HW/D = 1.28 see **Figure 1216**

Step 2: Calculate the required headwater depth assuming inlet control conditions. Multiple the pipe diameter times the headwater to depth ratio.

Headwater = $HW_i$ = D*(HW/D) = 5.0*1.28 = 6.4 ft

Step 3: Calculate the required headwater elevation assuming inlet control conditions.

FALL = 0, since the culvert invert is at grade

$HW_i = HW_d$

$EL_i = EL_{sf}$

$EL_{hd} = EL_i + HW_i = 4920 + 6.4 = 4926.4$ ft

Step 4: Estimate the critical depth, $d_c$, in the culvert from **Figure 1223**.

$d_c$ = 3.9 ft (see **Figure 1217)**

Step 5: Since the tailwater depth is less than the culvert diameter, compute the estimated water depth at the culvert outlet assuming the tailwater does not control the outlet conditions.

Outlet Depth = $(d_c + D)/2 = (3.9+5.0)/2 = 4.5$ ft

Step 6: Determine the flow depth at the culvert outlet, $h_o$. The estimated depth is the maximum value of the tailwater depth and the water depth assuming no tailwater.

4.5 ft > 4.0 ft, therefore $h_o$ = 4.5 ft

Step 7: Estimate the head, H, for outlet control conditions from **Figure 1226**.

H = 2.6 ft (see **Figure 1218**)

Step 8: Calculate the required outlet control headwater elevation, $EL_{ho}$.

$EL_{ho} = EL_o + H + h_o = 4918 + 2.6 + 4.5 = 4925.1$ ft

Step 9: Determine if the culvert is under inlet control or outlet control and provide the resulting control headwater elevation and depth.

$EL_{hd} > EL_{ho}$ (4926.4 > 4925.1), the culvert is under inlet control

Control Headwater Elevation = 4926.4

HW = 6.4 ft

Step 10: Calculate the outlet velocity by an appropriate method and determine the amount of outlet protection needed, see Section 1203.9.

v = 10.0 ft/s

## ENTRANCE LOSS

| Type of Structure and Design of Entrance | Coefficient K |
|---|---|
| • **Pipe, Concrete** | |
| Projecting from fill, socket end (groove-end) | 0.2 |
| Projecting from fill, sq. cut end | 0.5 |
| Headwall or headwall and wingwalls | |
| Socket end of pipe (groove-end) | 0.2 |
| Square-edge | 0.5 |
| Rounded (radius = D/12) | 0.2 |
| Mitered to conform to fill slope | 0.7 |
| * End-Section conforming to fill slope | 0.5 |
| Beveled edges, 33.7° or 45° bevels | 0.2 |
| Side- or slope-tapered inlet | 0.2 |
| • **Pipe, or Pipe-Arch, Corrugated Metal** | |
| Projecting from fill (no headwall) | 0.9 |
| Headwall or headwall and wingwalls square-edge | 0.5 |
| Mitered to conform to fill slope, paved or unpaved slope | 0.7 |
| * End-Section conforming to fill slope | 0.5 |
| Beveled edges, 33.7° or 45° bevels | 0.2 |
| Side- or slope-tapered inlet | 0.2 |
| • **Box, Reinforced Concrete** | |
| Headwall parallel to embankment (no wingwalls) | |
| Square-edged on 3 edges | 0.5 |
| Rounded on 3 edges to radius of D/12 or B/12 or beveled edges on 3 sides | 0.2 |
| Wingwalls at 30° to 75° to barrel | |
| Square-edged at crown | 0.4 |
| Crown edge rounded to radius of D/12 or beveled top edge | 0.2 |
| Wingwall at 10° to 25° to barrel | |
| Square-edged at crown | 0.5 |
| Wingwalls parallel (extension of sides) | |
| Square-edged at crown | 0.7 |
| Side- or slope-tapered inlet | 0.2 |

*Note: "End Sections conforming to fill slope," made of either metal or concrete, are the sections commonly available from manufacturers. From limited hydraulic tests they are equivalent in operation to a headwall in both inlet and outlet control. Some end sections, incorporating a closed taper in their design have a superior hydraulic performance. These latter sections can be designed using the information given for the beveled inlet.

| Revision | Date |
|---|---|
| ORIGINAL ISSUE | 3/27/06 |
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| | |

| PROJECT: ________________ | STATION: ________________ | CULVERT DESIGN FORM |
|---|---|---|
| ________________ | SHEET _____ OF _____ | DESIGNER/DATE: ______ / ______<br>REVIEWER/DATE: ______ / ______ |

**HYDROLOGICAL DATA**

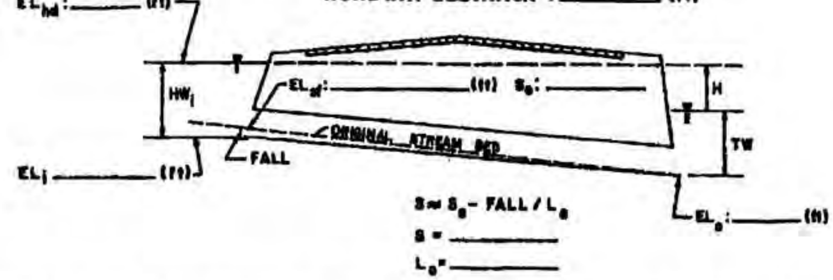
SEE ADD'L SHTS.

- ☐ METHOD: ________________
- ☐ DRAINAGE AREA: ________ ☐ STREAM SLOPE: ________
- ☐ CHANNEL SHAPE: ________________
- ☐ ROUTING: ________ ☐ OTHER: ________

**DESIGN FLOWS/TAILWATER**

| R.I. (YEARS) | FLOW (cfs) | TW (ft) |
|---|---|---|
| | | |
| | | |

ROADWAY ELEVATION: ________ (ft)




$S = S_o - FALL / L_a$

S = ________

$L_a$ = ________

| CULVERT DESCRIPTION: MATERIAL - SHAPE - SIZE - ENTRANCE | TOTAL FLOW Q (cfs) | FLOW PER BARREL Q/N (1) | HEADWATER CALCULATIONS: INLET CONTROL $HW_i/D$ (2) | $HW_i$ | FALL (3) | $EL_{hi}$ (4) | OUTLET CONTROL TW (5) | $d_c$ | $\frac{d_c+D}{2}$ | $h_o$ (6) | $k_e$ | H (7) | $EL_{ho}$ (8) | CONTROL HEADWATER ELEVATION | OUTLET VELOCITY | COMMENTS |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | |

**TECHNICAL FOOTNOTES:**

(1) USE Q/NB FOR BOX CULVERTS

(2) $HW_i/D = HW/D$ OR $HW_i/D$ FROM DESIGN CHARTS

(3) $FALL = HW_i - (EL_{hd} - EL_{sf})$; FALL IS ZERO FOR CULVERTS ON GRADE

(4) $EL_{hi} = HW_i + EL_i$ (INVERT OF INLET CONTROL SECTION)

(5) TW BASED ON DOWN STREAM CONTROL OR FLOW DEPTH IN CHANNEL.

(6) $h_o = TW$ or $(d_c+D)/2$ (WHICHEVER IS GREATER)

(7) $H = \left[1 + k_e + (29 n^2 L) / R^{1.33}\right] V^2/2g$

(8) $EL_{ho} = EL_o + H + h_o$

**SUBSCRIPT DEFINITIONS:**

a. APPROXIMATE
f. CULVERT FACE
hd. DESIGN HEADWATER
hi. HEADWATER IN INLET CONTROL
ho. HEADWATER IN OUTLET CONTROL
i. INLET CONTROL SECTION
o. OUTLET
sf. STREAMBED AT CULVERT FACE
tw. TAILWATER

**COMMENTS / DISCUSSION:**

**CULVERT BARREL SELECTED:**

SIZE: ________

SHAPE: ________

MATERIAL: ________ n ________

ENTRANCE: ________

| Revision | Date |
|---|---|
| ORIGINAL ISSUE | 3/27/06 |
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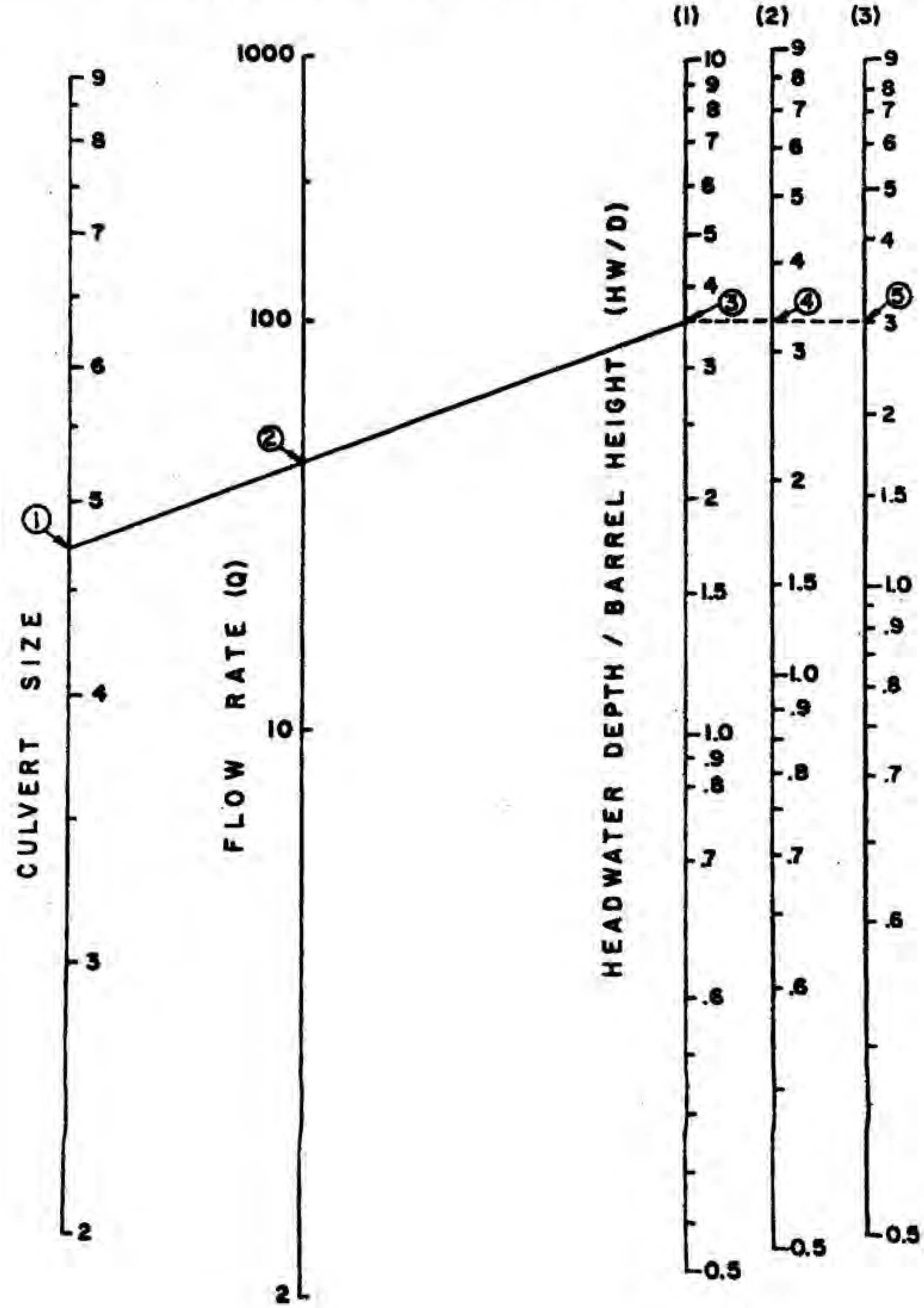
## INLET CONTROL NOMOGRAPH (SCHEMATIC)

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|---|---|
| ORIGINAL ISSUE | 3/27/06 |
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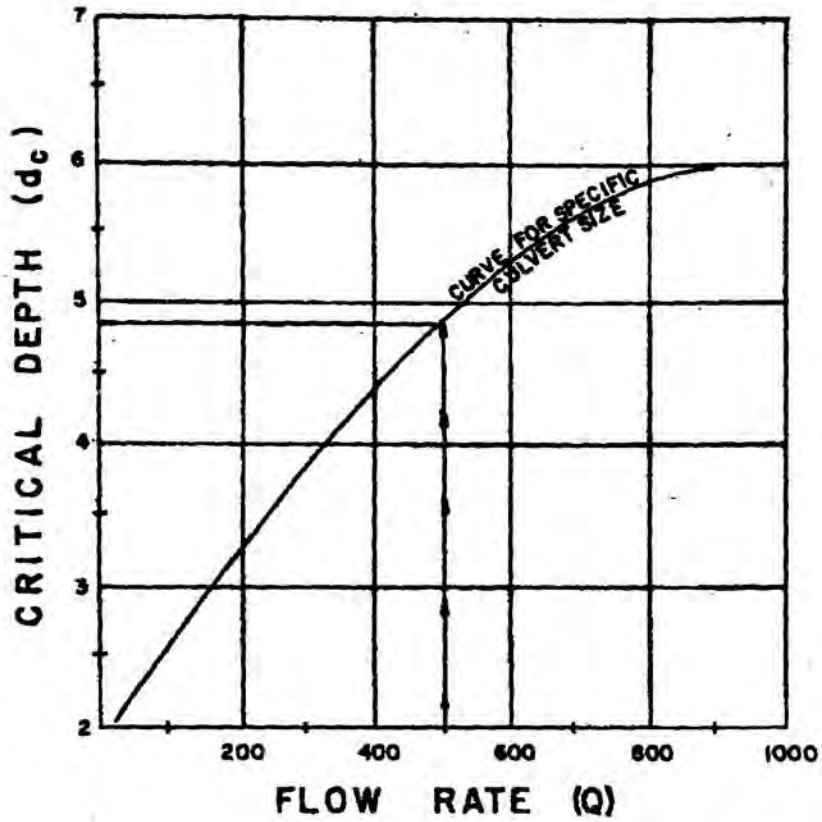
REFERENCE:

HDS-5. FHWA

**FIGURE 1206**

## CRITICAL DEPTH CHART (SCHEMATIC)

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| ORIGINAL ISSUE | 3/27/06 |
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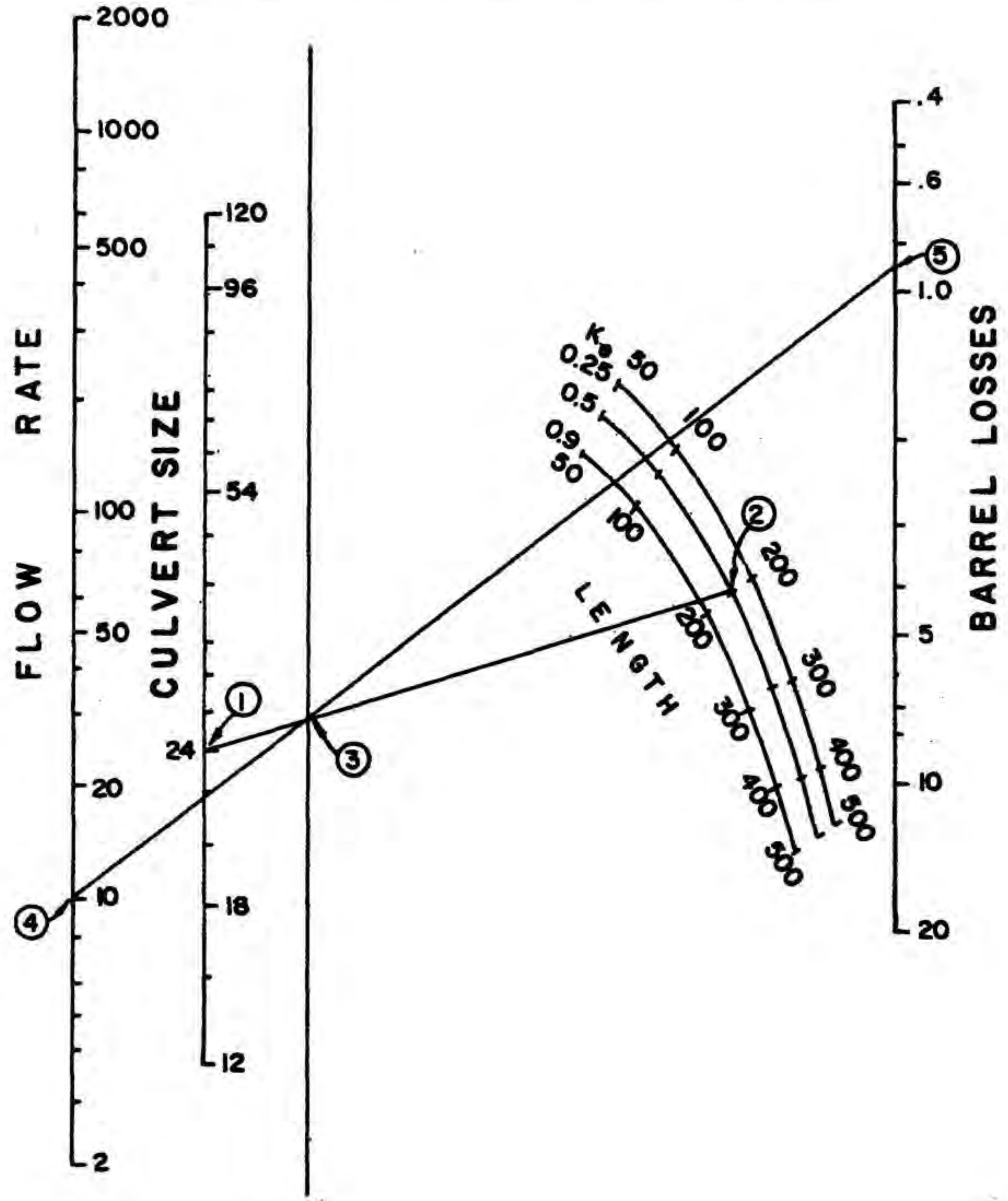
## OUTLET CONTROL NOMOGRAPH (SCHEMATIC)

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| ORIGINAL ISSUE | 3/27/06 |
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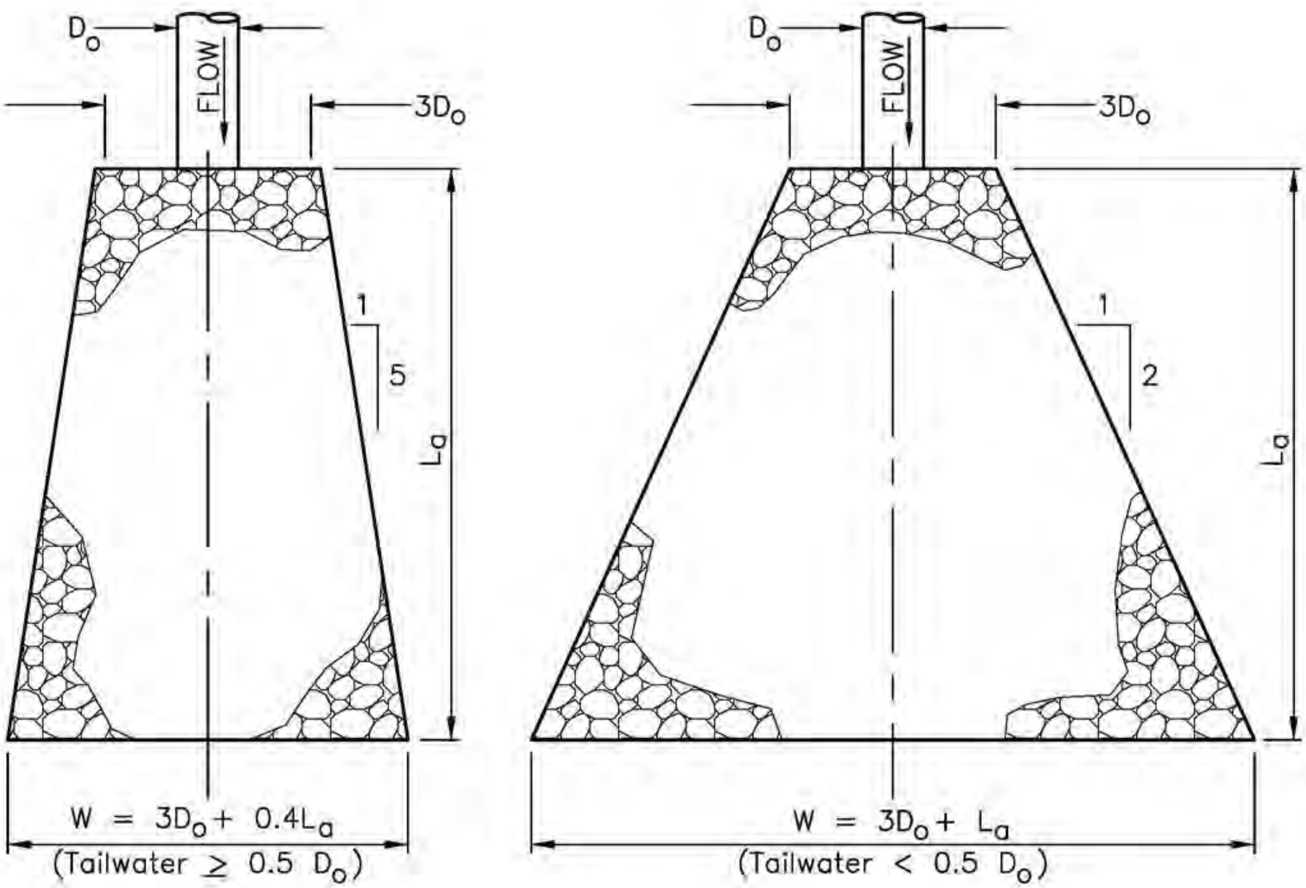
# STORMWATER MANAGEMENT MANUAL

## CONFIGURATION OF CULVERT OUTLET PROTECTION

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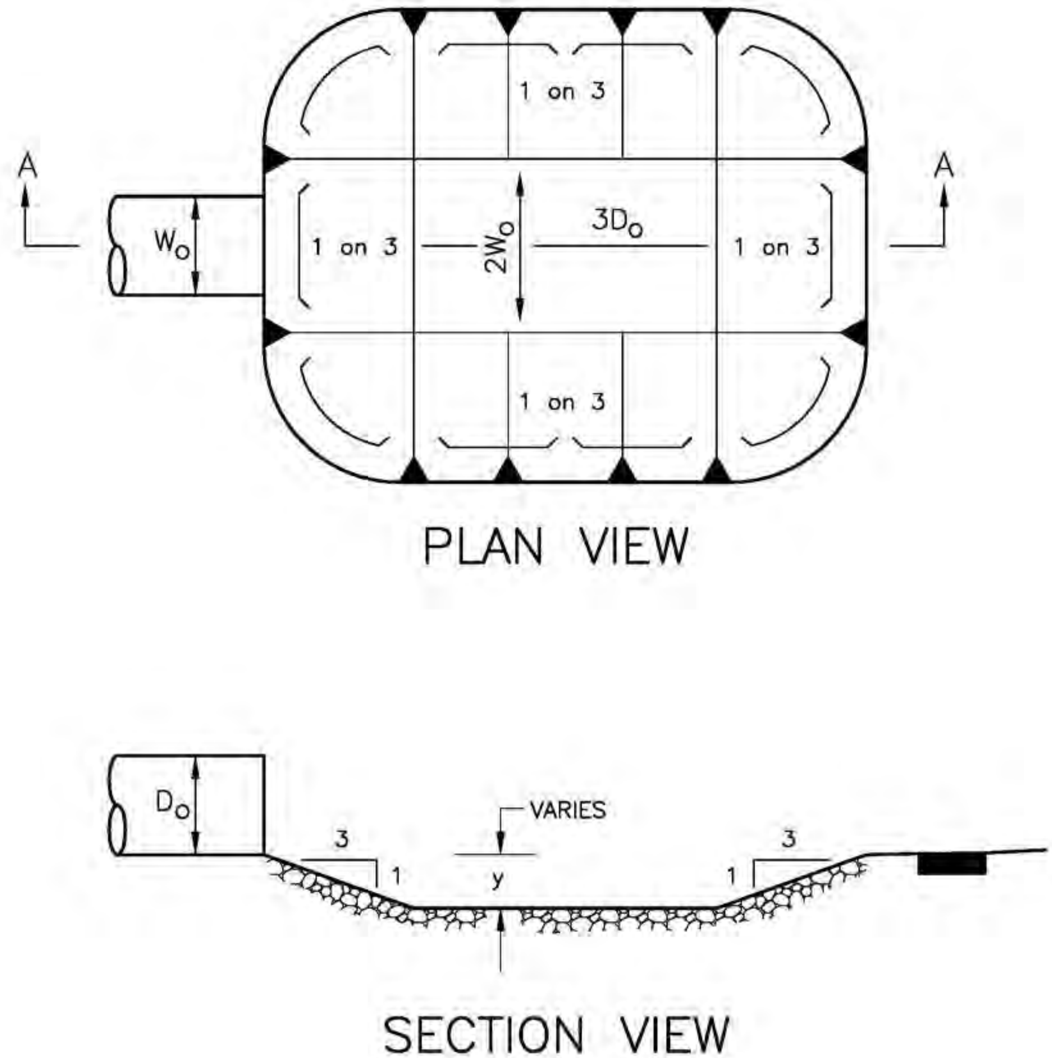
REFERENCE:

U.S. EPA

**FIGURE 1209**

# STORMWATER MANAGEMENT MANUAL

## CULVERT OUTLET PROTECTION AT PREFORMED SCOUR HOLE

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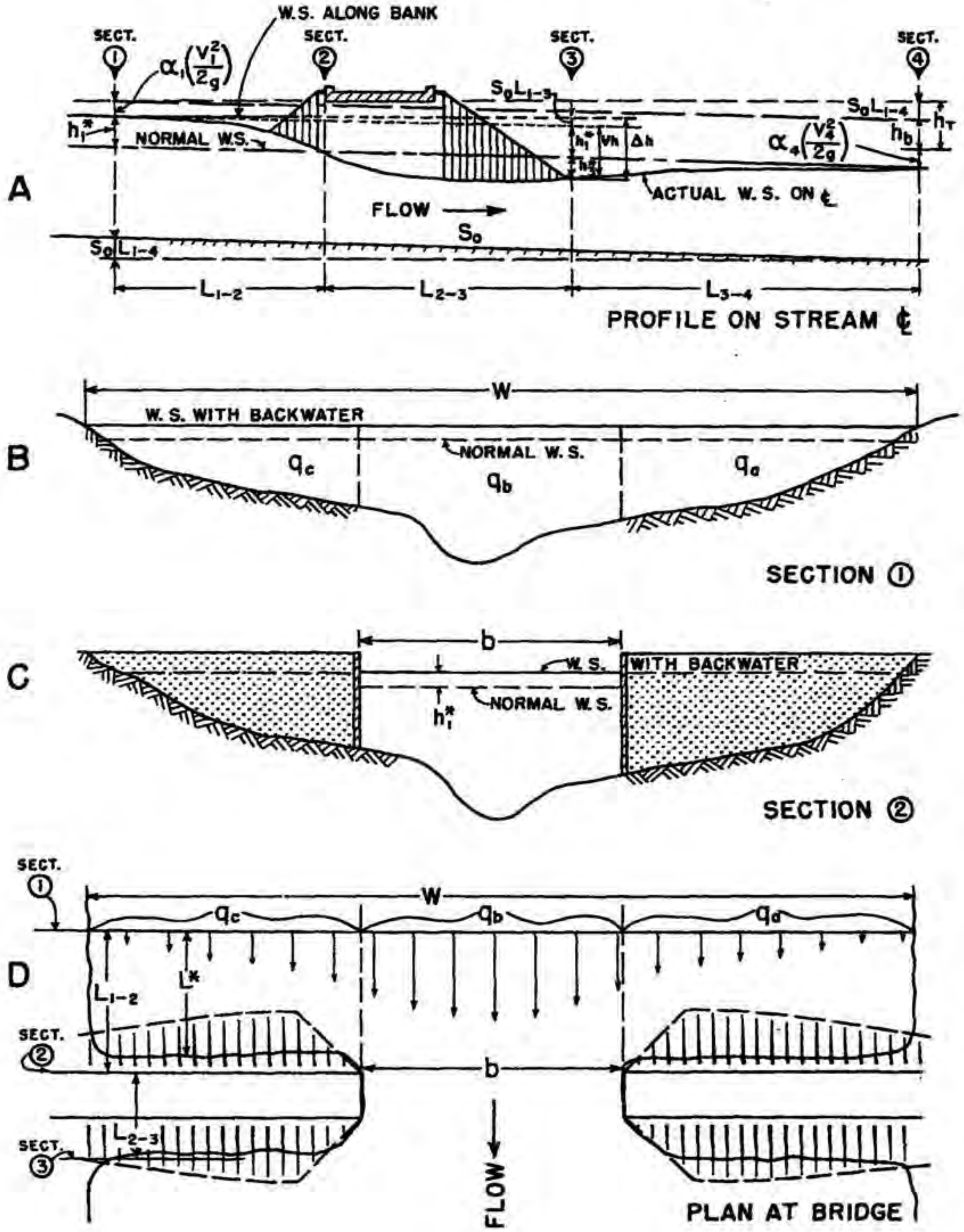
REFERENCE: DESIGN AND CONSTRUCTION OF URBAN STORMAWATER MANAGEMENT SYSTEMS, ASCE

**FIGURE 1210**

## NORMAL BRIDGE CROSSING DESIGNATION

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| ORIGINAL ISSUE | 3/27/06 |
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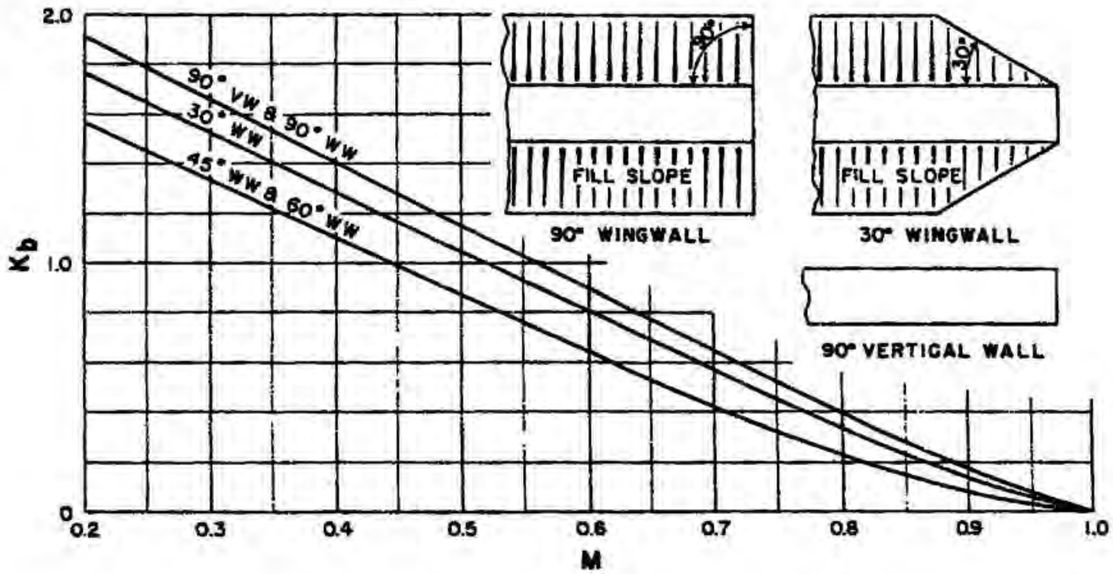
## BASE CURVES FOR WINGWALL ABUTMENTS

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|---|---|
| ORIGINAL ISSUE | 3/27/06 |
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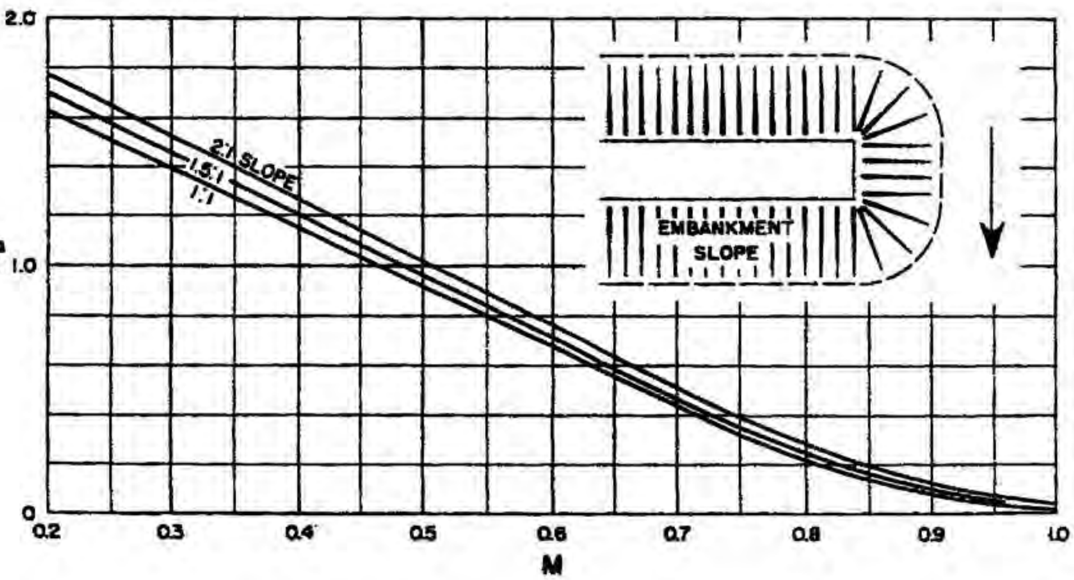
REFERENCE:

HDS-1, FHWA

**FIGURE 1212**

## BASE CURVES FOR SPILLTHROUGH ABUTMENTS

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| ORIGINAL ISSUE | 3/27/06 |
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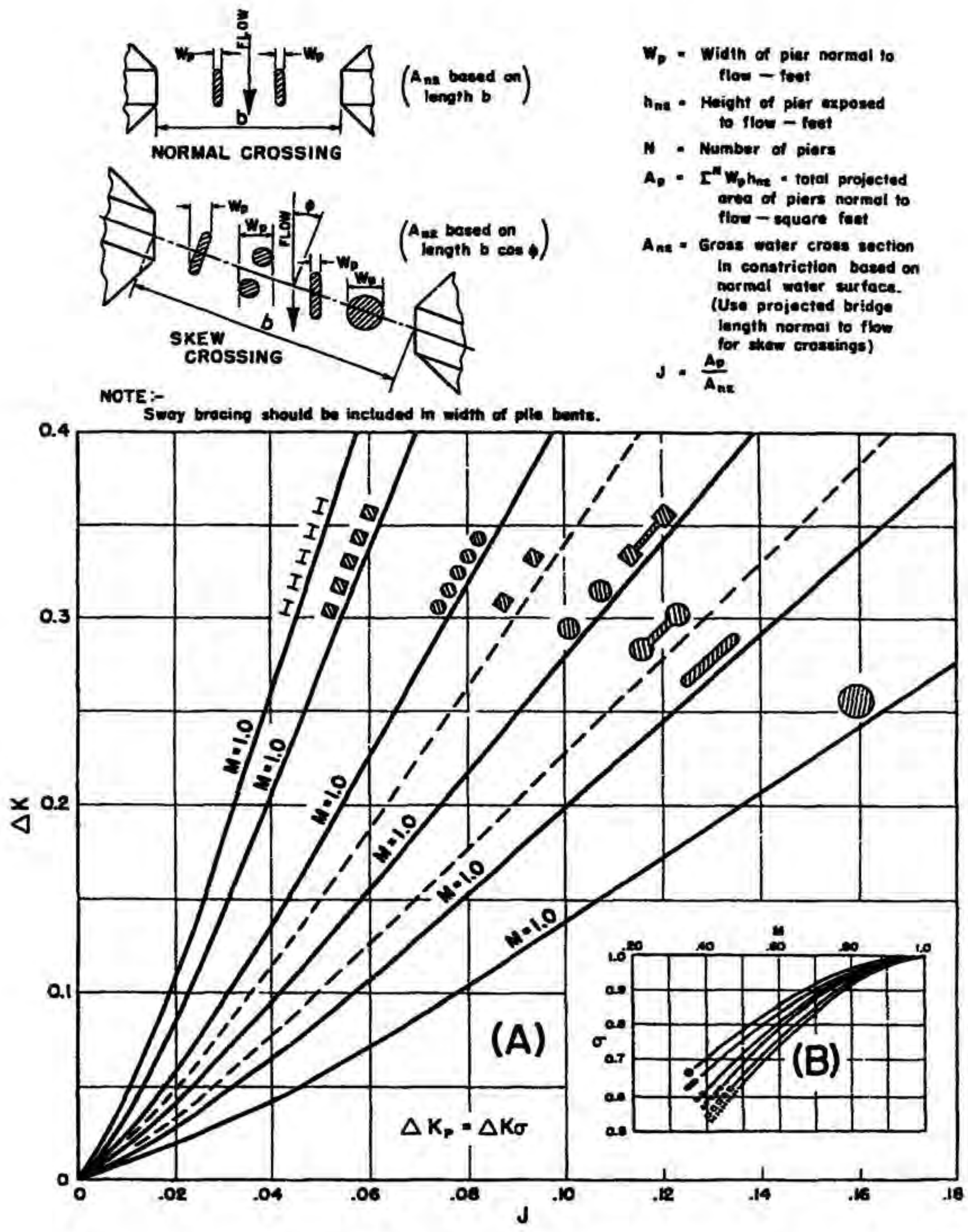
## INCREMENTAL BACKWATER COEFFICIENT FOR PIER



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|---|---|
| ORIGINAL ISSUE | 3/27/06 |
| | |
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REFERENCE:

HDS-1, FHWA

**FIGURE 1214**

## EXAMPLE - CULVERT DESIGN FORM

| | | |
|---|---|---|
| PROJECT: DOE CREEK<br>JOHN BOULEVARD | STATION: 1+00<br>SHEET 1 OF 1 | **CULVERT DESIGN FORM**<br>DESIGNER / DATE: ______ / ______<br>REVIEWER / DATE: ______ / ______ |

**HYDROLOGICAL DATA**

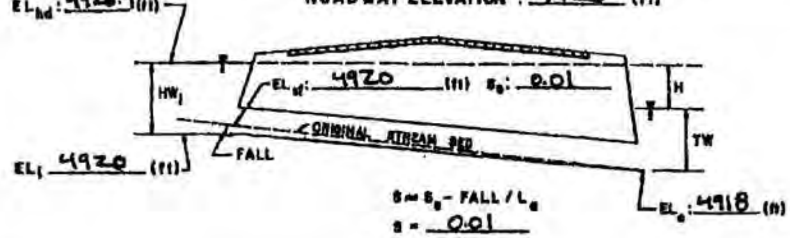
SEE ADD'L SHTS.

- ☐ METHOD: RAINFALL / RUNOFF
- ☐ DRAINAGE AREA: 100 ☐ STREAM SLOPE: 1.0%
- ☐ CHANNEL SHAPE: TRAPEZOIDAL
- ☐ ROUTING: N/A ☐ OTHER: ______

**DESIGN FLOWS/TAILWATER**

| R.I. (YEARS) | FLOW (cfs) | TW (ft) |
|---|---|---|
| 100 | 191 | 4.0 |
| | | |

ROADWAY ELEVATION: 4928 (ft)




$S = S_o - FALL / L_a$

$S =$ 0.01

$L_a =$ 200

| CULVERT DESCRIPTION: MATERIAL - SHAPE - SIZE - ENTRANCE | TOTAL FLOW Q (cfs) | FLOW PER BARREL Q/N (1) | HEADWATER CALCULATIONS: INLET CONTROL $HW_i/D$ (2) | $HW_i$ | FALL (3) | $EL_{hi}$ (4) | OUTLET CONTROL TW (5) | $d_c$ | $\frac{d_c+D}{2}$ | $h_o$ (6) | $h_e$ | H (7) | $EL_{ho}$ (8) | CONTROL HEADWATER ELEVATION | OUTLET VELOCITY | COMMENTS |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| CONCRETE - CIRCULAR - 60 in | 191 | 191 | 1.28 | 6.4 | 0 | 4926.4 | 4.0 | 3.9 | 4.5 | 4.5 | 0.20 | 2.6 | 4925.1 | 4926.4 | 10.0 | - |
| GROOVE END WITH HEADWALL | | | | | | | | | | | | | | | | |
| AND 45° WINGWALL | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | |

**TECHNICAL FOOTNOTES:**

(1) USE Q/NB FOR BOX CULVERTS

(2) $HW_i/D = HW/D$ OR $HW_i/D$ FROM DESIGN CHARTS

(3) $FALL = HW_i - (EL_{hd} - EL_{sf})$; FALL IS ZERO FOR CULVERTS ON GRADE

(4) $EL_{hi} = HW_i + EL_i$ (INVERT OF INLET CONTROL SECTION)

(5) TW BASED ON DOWN STREAM CONTROL OR FLOW DEPTH IN CHANNEL.

(6) $h_o = TW$ or $(d_c + D)/2$ (WHICHEVER IS GREATER)

(7) $H = \left[1 + k_e + (29 n^2 L) / R^{1.33}\right] V^2 / 2g$

(8) $EL_{ho} = EL_o + H + h_o$

**SUBSCRIPT DEFINITIONS:**

a. APPROXIMATE
f. CULVERT FACE
hd. DESIGN HEADWATER
hi. HEADWATER IN INLET CONTROL
ho. HEADWATER IN OUTLET CONTROL
i. INLET CONTROL SECTION
o. OUTLET
sf. STREAMBED AT CULVERT FACE
tw. TAILWATER

**COMMENTS / DISCUSSION:**

**CULVERT BARREL SELECTED:**

SIZE: 60 in

SHAPE: CIRCULAR

MATERIAL: CONCRETE n 0.012

ENTRANCE: GROOVE END

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|---|---|
| ORIGINAL ISSUE | 3/27/06 |
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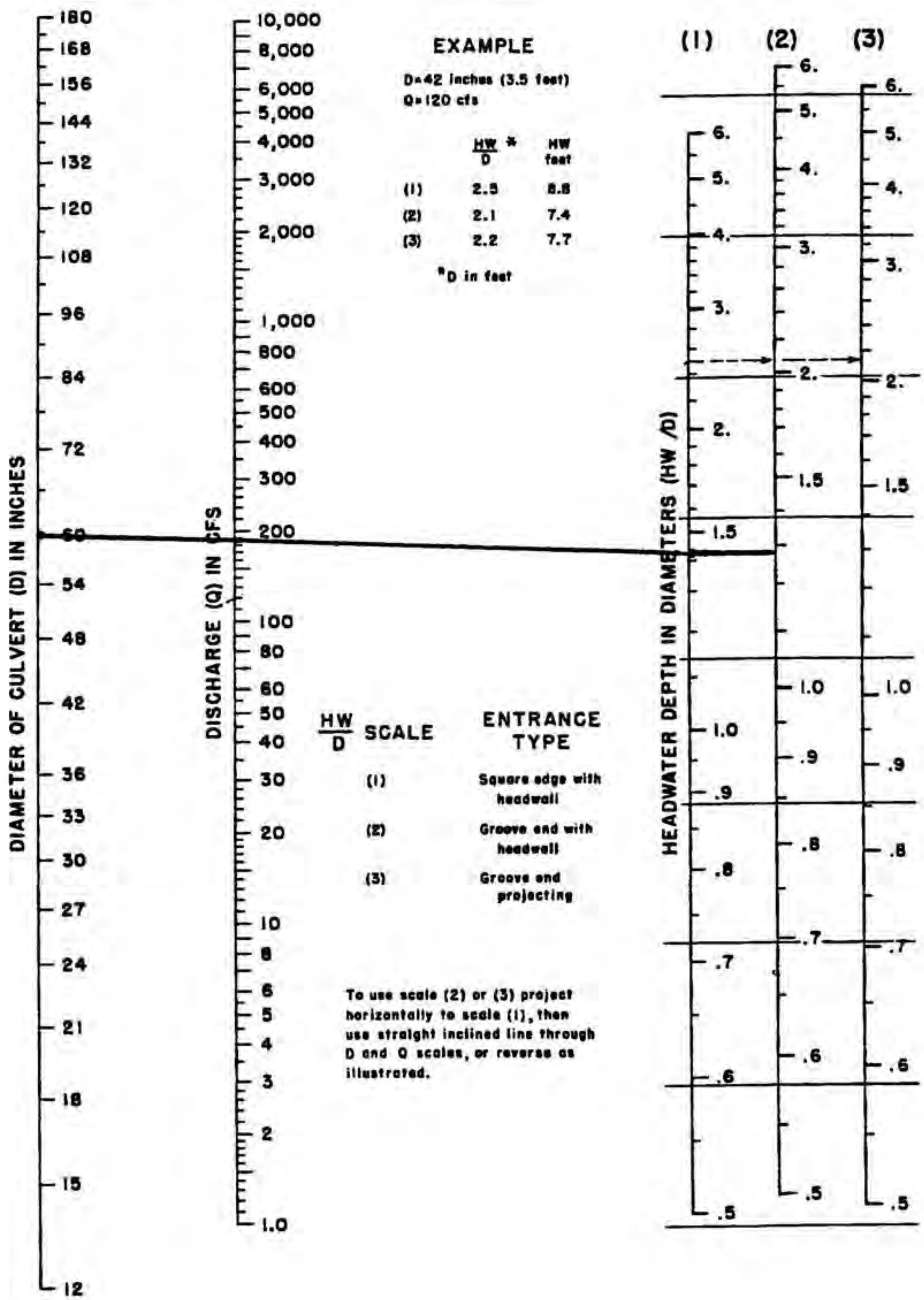
**FIGURE 1215**

# STORMWATER MANAGEMENT MANUAL

## EXAMPLE - INLET CONTROL NOMOGRAPH

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|---|---|
| ORIGINAL ISSUE | 3/27/06 |
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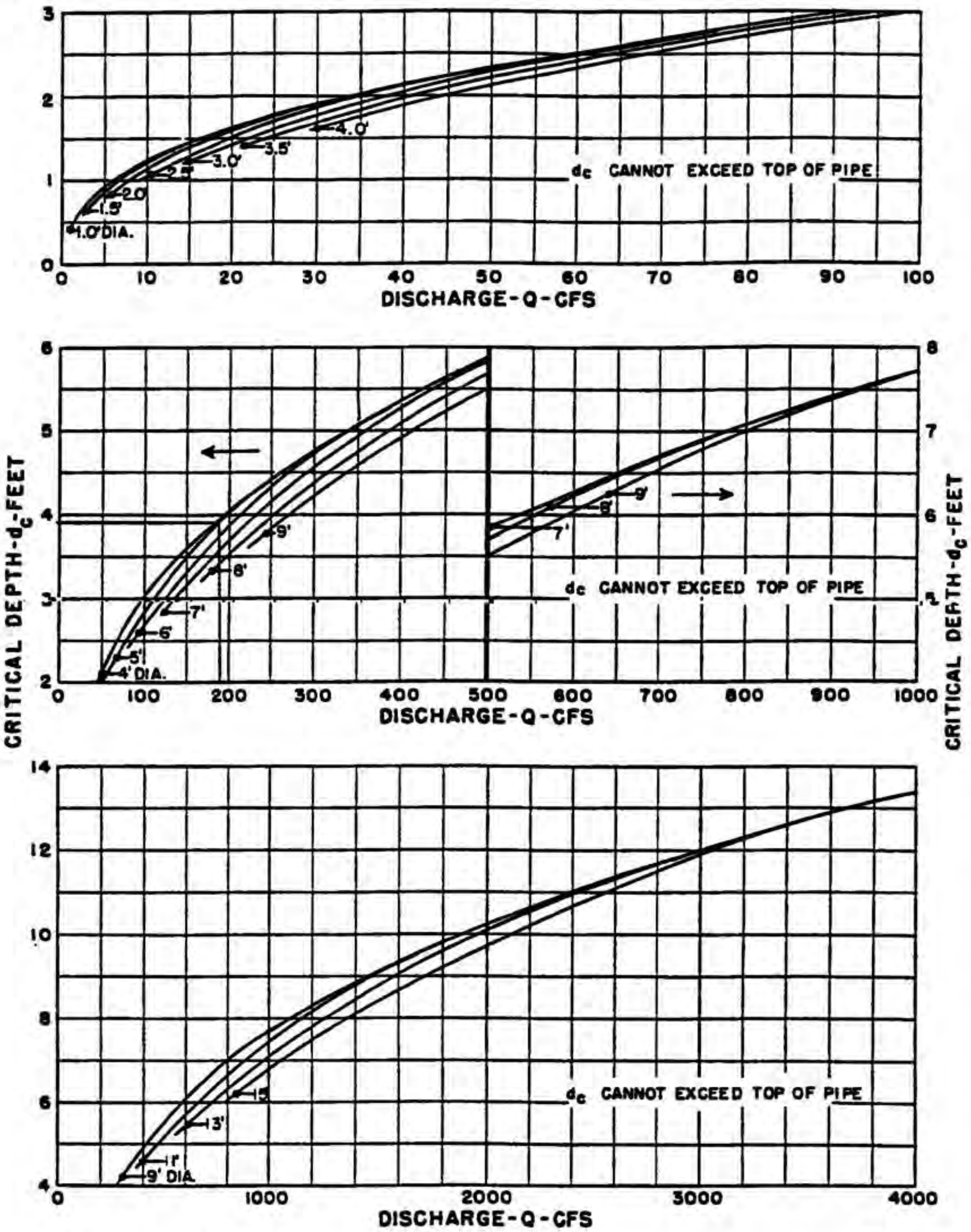
**FIGURE 1216**

**EXAMPLE - CRITICAL DEPTH**

## CHART 4

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| ORIGINAL ISSUE | 3/27/06 |
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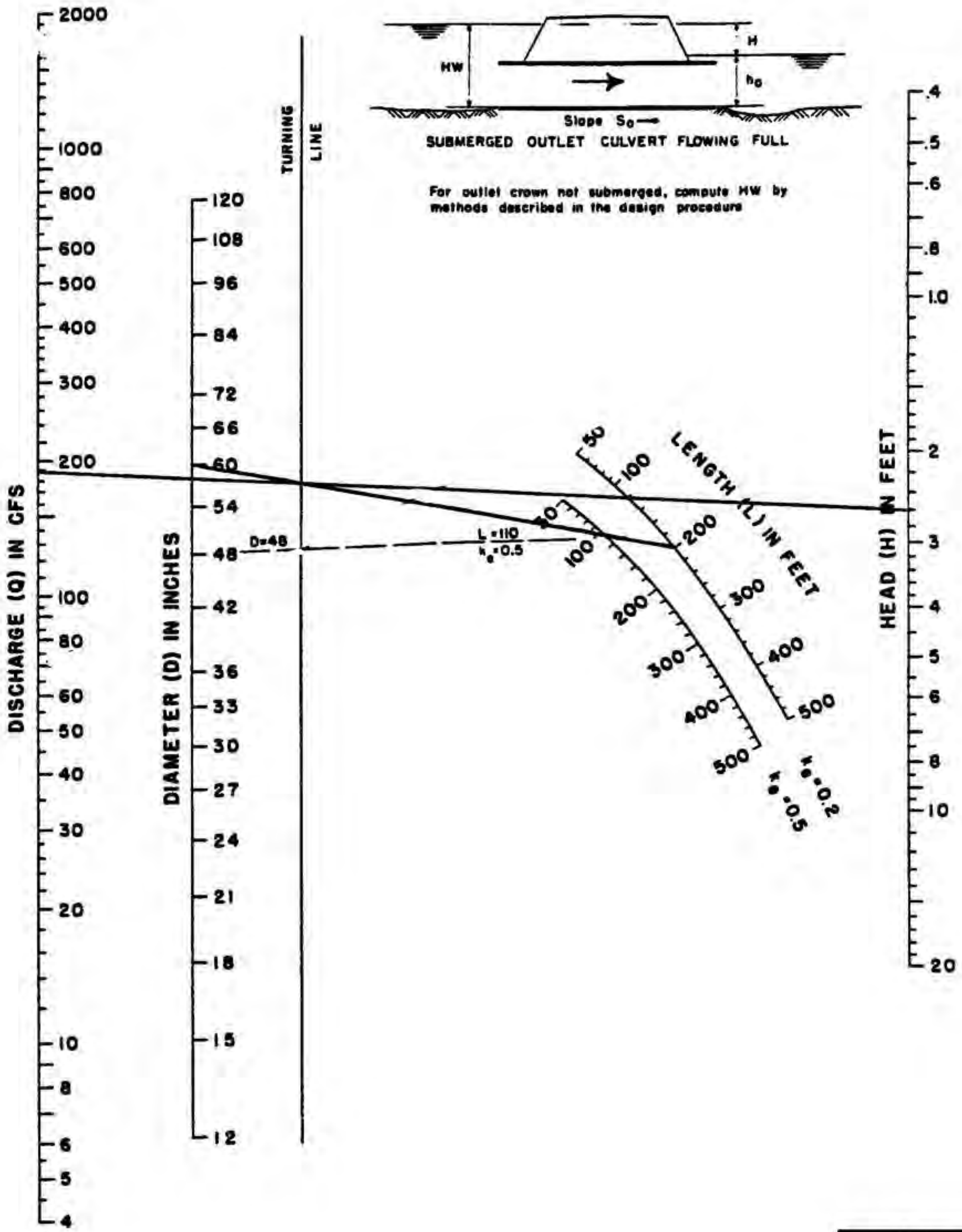
**FIGURE 1217**

## EXAMPLE - OUTLET CONTROL NOMOGRAPH

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|---|---|
| ORIGINAL ISSUE | 3/27/06 |
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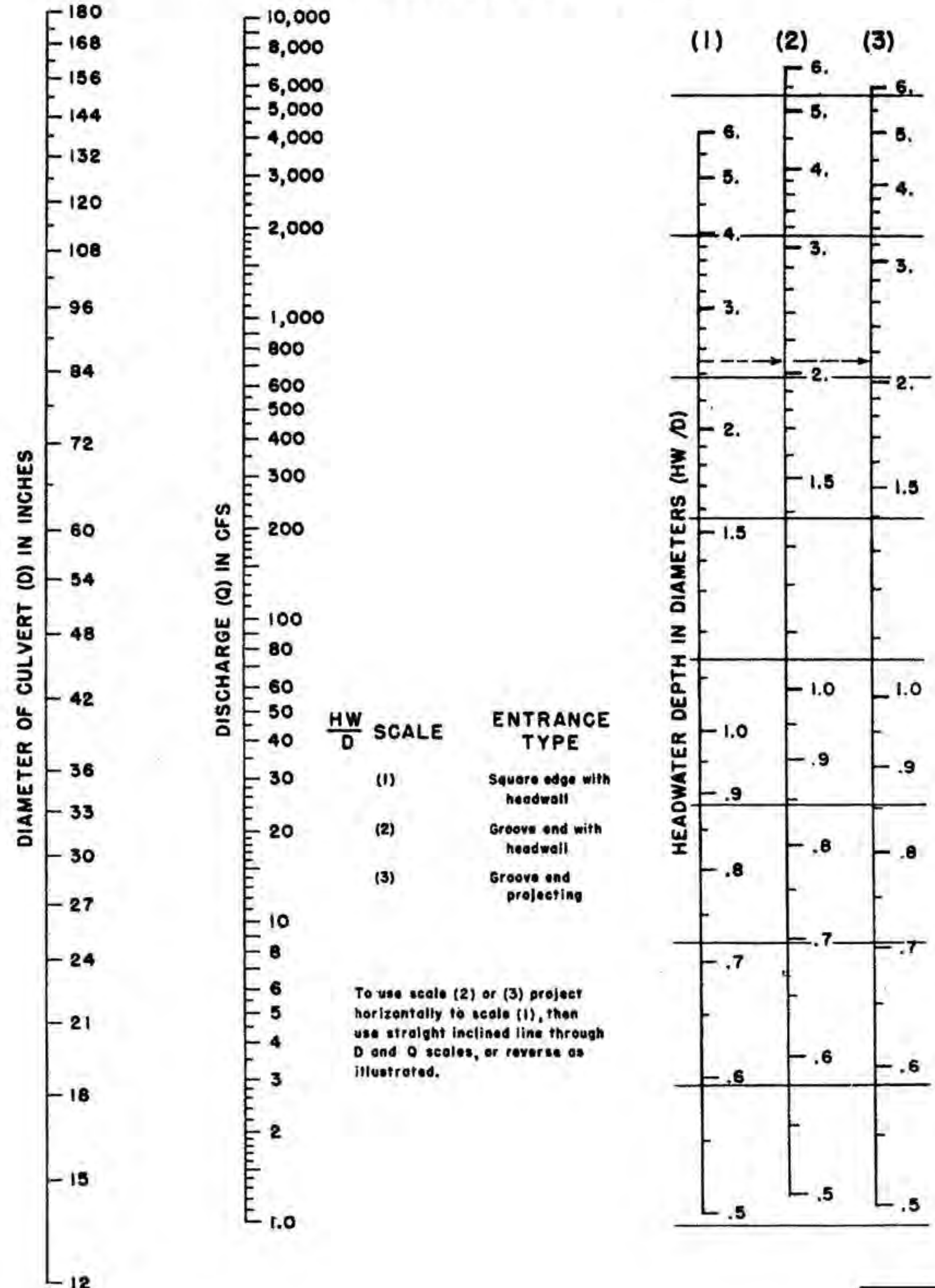
**FIGURE 1218**

## HEADWATER DEPTH FOR CONCRETE PIPE CULVERTS WITH INLET CONTROL

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|---|---|
| ORIGINAL ISSUE | 3/27/06 |
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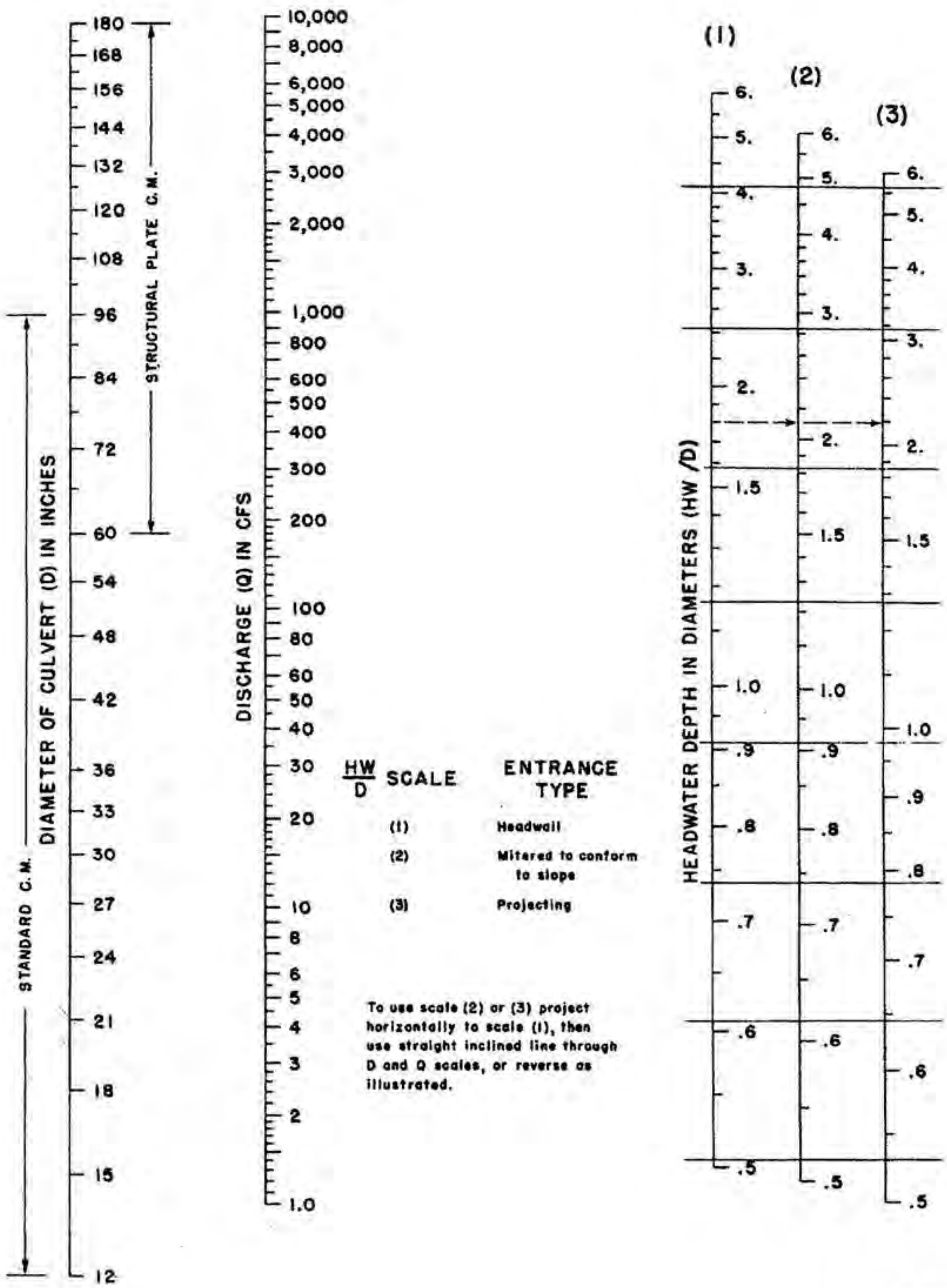
**FIGURE 1219**

## HEADWATER DEPTH FOR CORRUGATED METAL PIPE CULVERTS WITH INLET CONTROL




| $\frac{HW}{D}$ SCALE | ENTRANCE TYPE |
|---|---|
| (1) | Headwall |
| (2) | Mitered to conform to slope |
| (3) | Projecting |

To use scale (2) or (3) project horizontally to scale (1), then use straight inclined line through D and Q scales, or reverse as illustrated.

| Revision | Date |
|---|---|
| ORIGINAL ISSUE | 3/27/06 |
| | |
| | |

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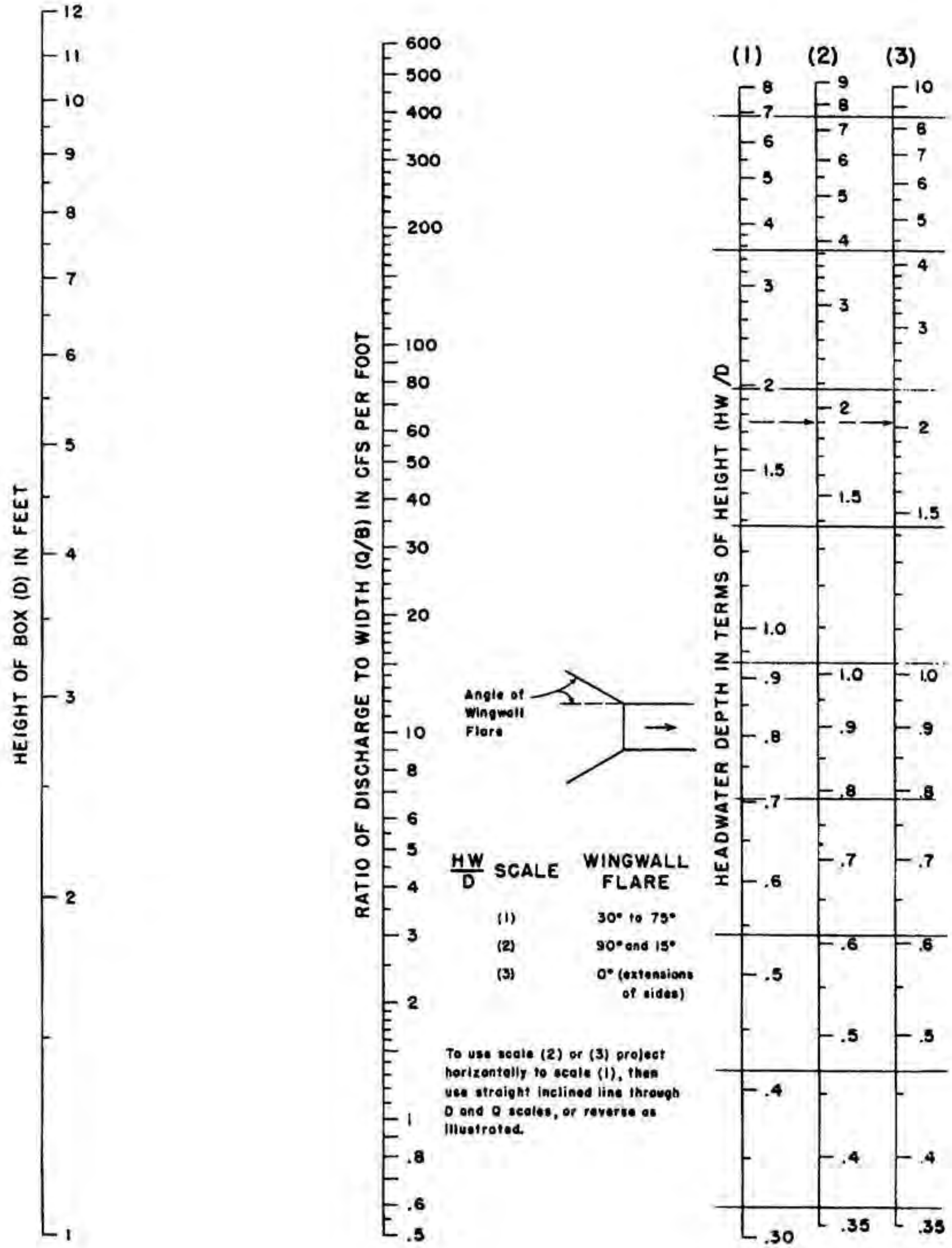
**FIGURE 1220**

## HEADWATER DEPTH FOR CONCRETE BOX CULVERTS WITH INLET CONTROL




| Revision | Date |
|---|---|
| ORIGINAL ISSUE | 3/27/06 |
| | |
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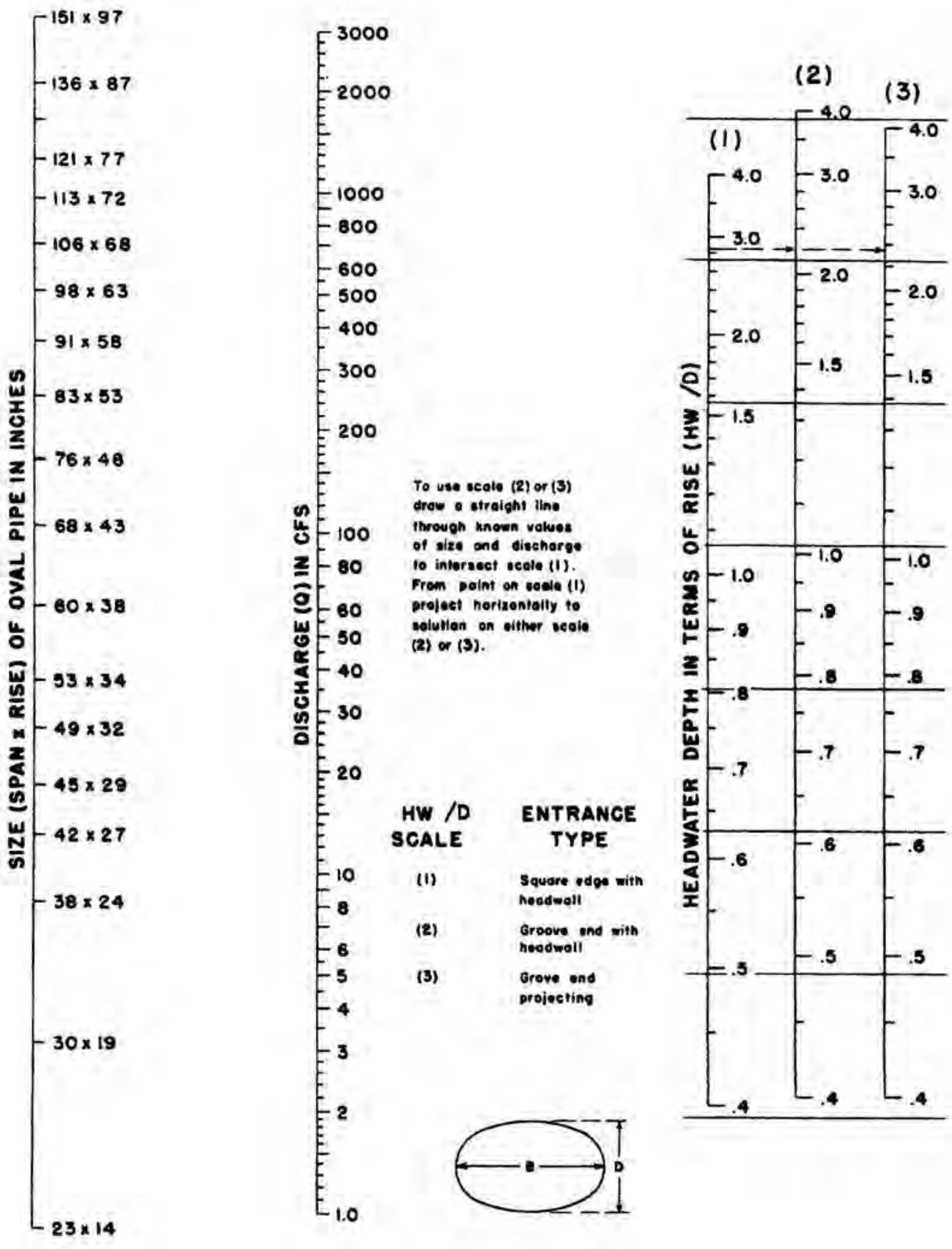
**FIGURE 1221**

## HEADWATER DEPTH FOR OVAL CONCRETE PIPE CULVERTS LONG-AXIS HORIZONTAL WITH INLET CONTROL

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|---|---|
| ORIGINAL ISSUE | 3/27/06 |
| | |
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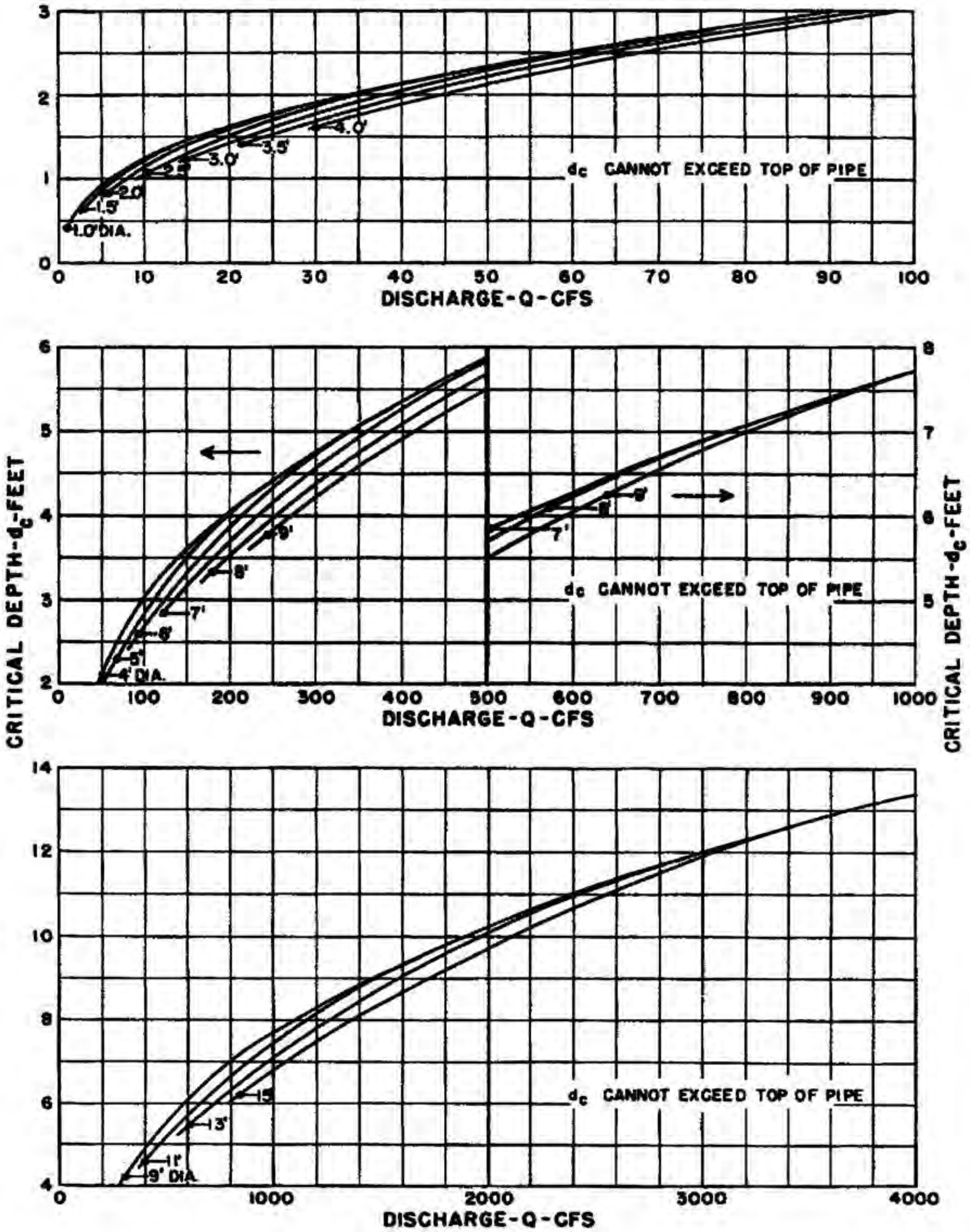
## CRITICAL DEPTH - CIRICULAR PIPE

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|---|---|
| ORIGINAL ISSUE | 3/27/06 |
| | |
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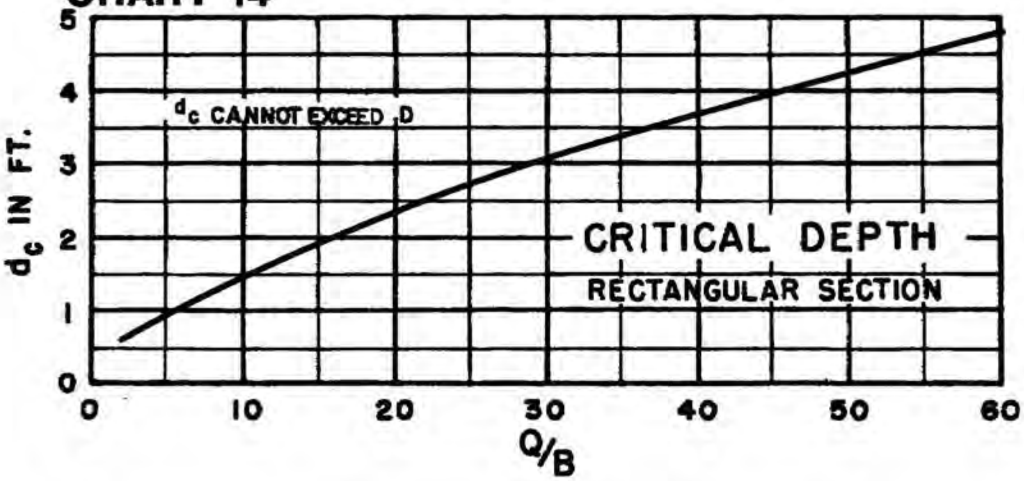
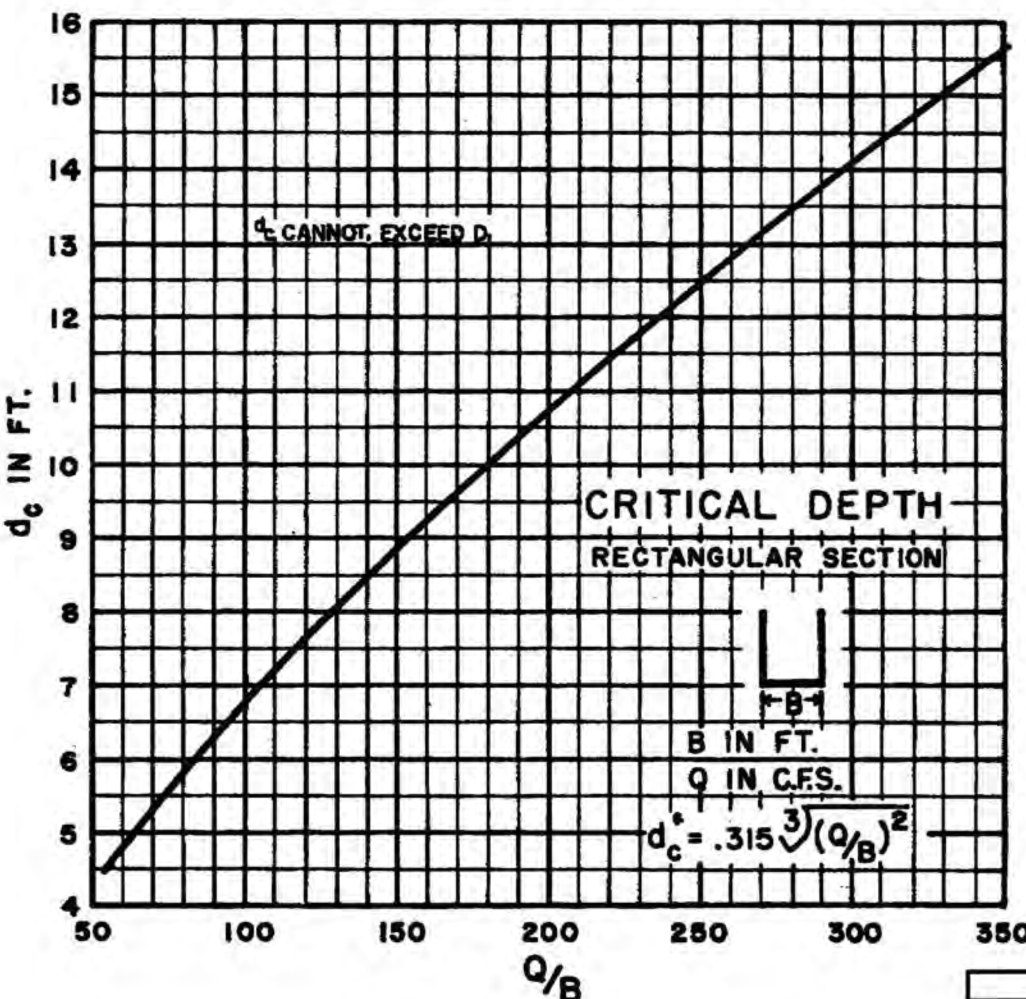
**FIGURE 1223**

## CRITICAL DEPTH - RECTANGULAR SECTION

### CHART 14

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|---|---|
| ORIGINAL ISSUE | 3/27/06 |
| | |
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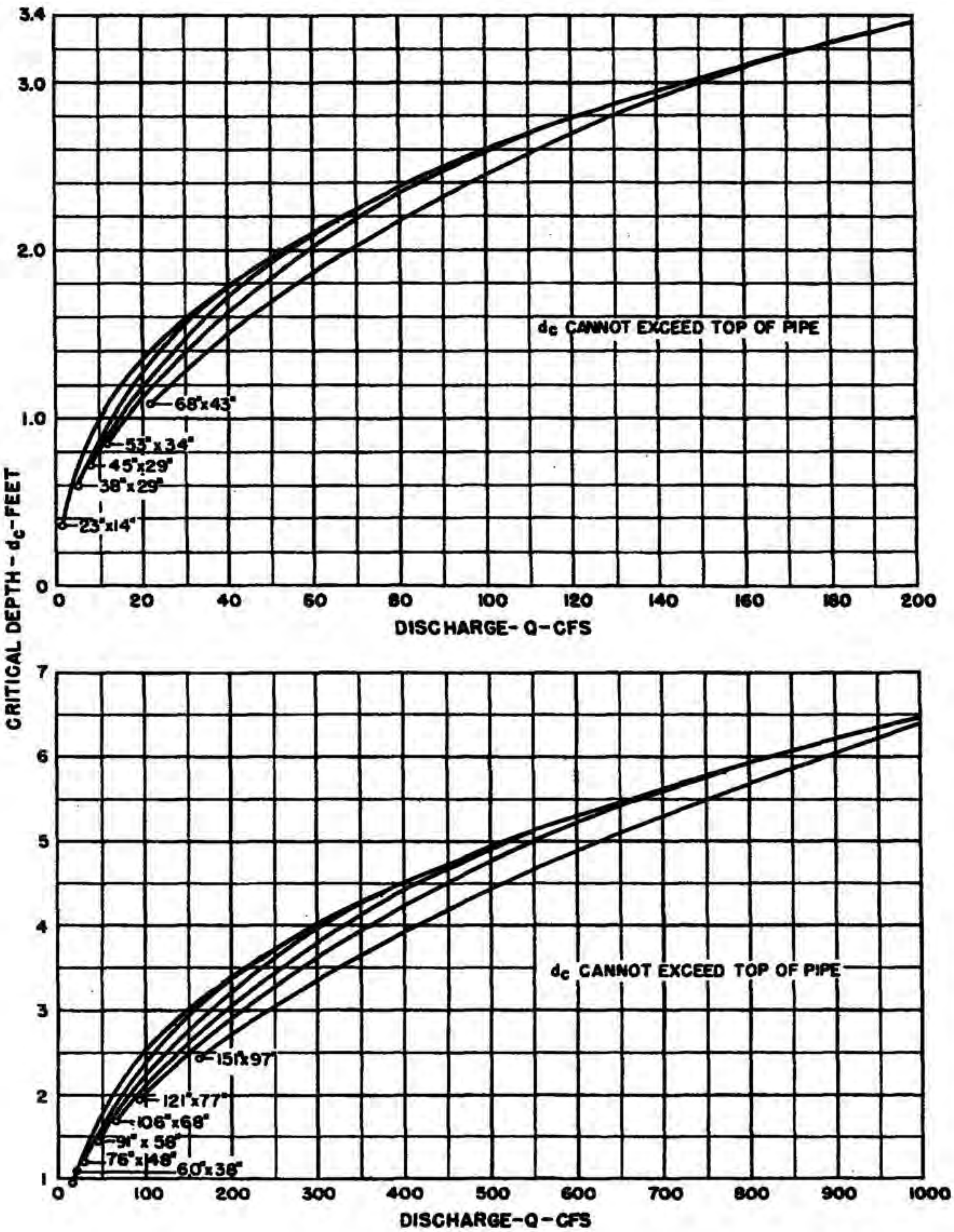
**FIGURE 1224**

# STORMWATER MANAGEMENT MANUAL

## CRITICAL DEPTH - OVAL CONCRETE PIPE
## LONG-AXIS HORIZONTAL

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|---|---|
| ORIGINAL ISSUE | 3/27/06 |
| | |
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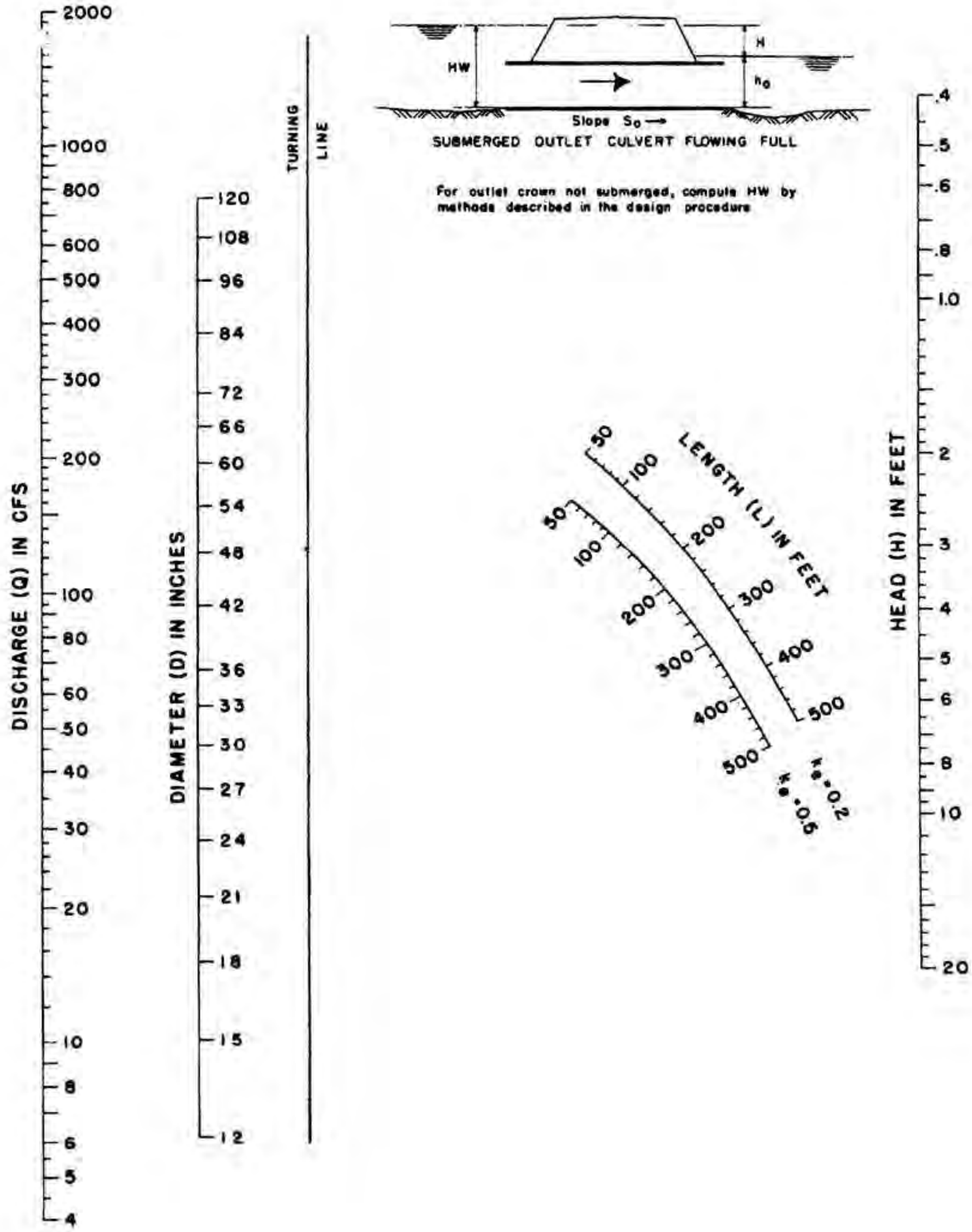
**FIGURE 1225**

## HEAD FOR CONCRETE PIPE CULVERTS WITH OUTLET CONTROL, n=0.012

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|---|---|
| ORIGINAL ISSUE | 3/27/06 |
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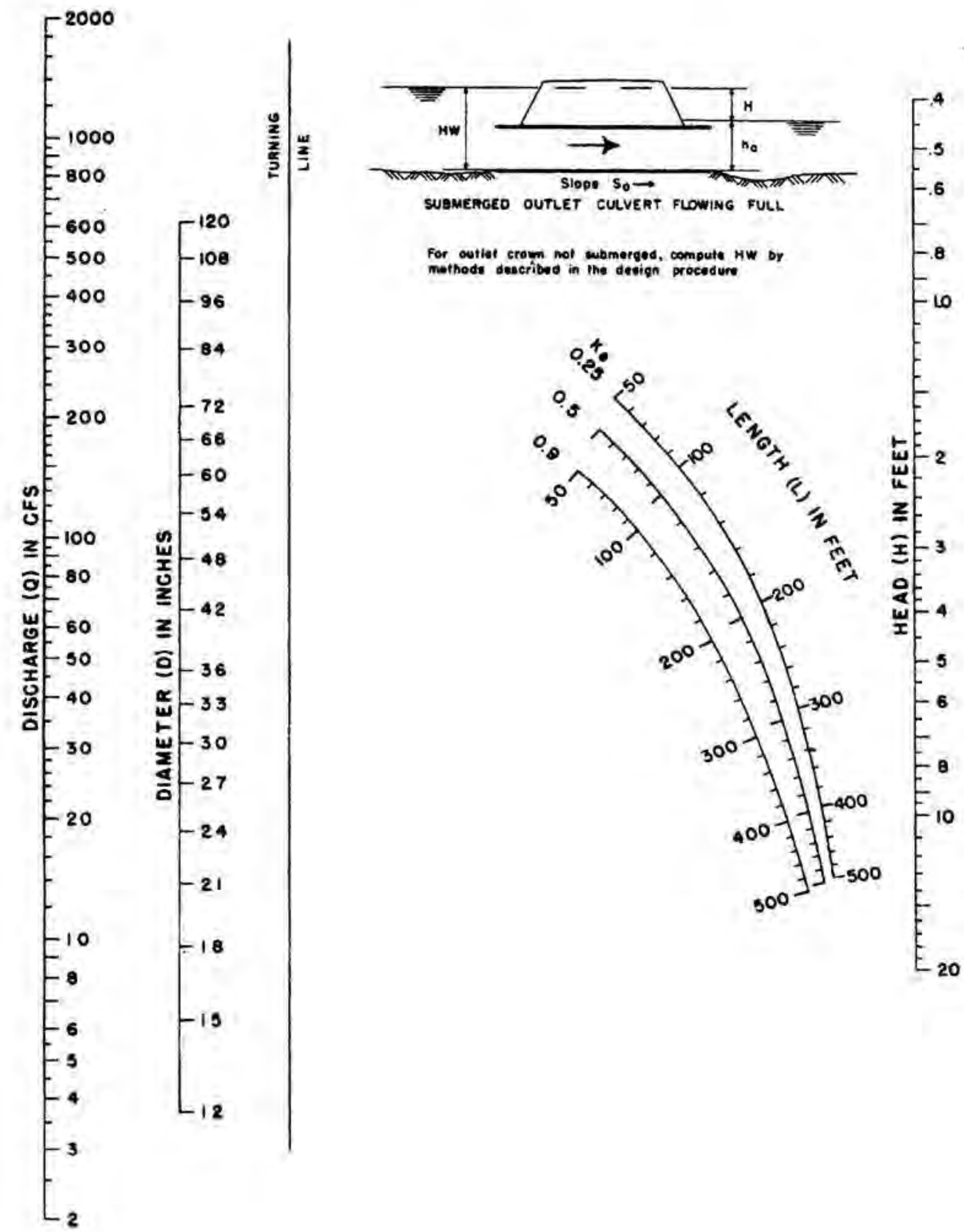
**FIGURE 1226**

## HEAD FOR CORRUGATED METAL PIPE CULVERTS WITH OUTLET CONTROL, n=0.024

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|---|---|
| ORIGINAL ISSUE | 3/27/06 |
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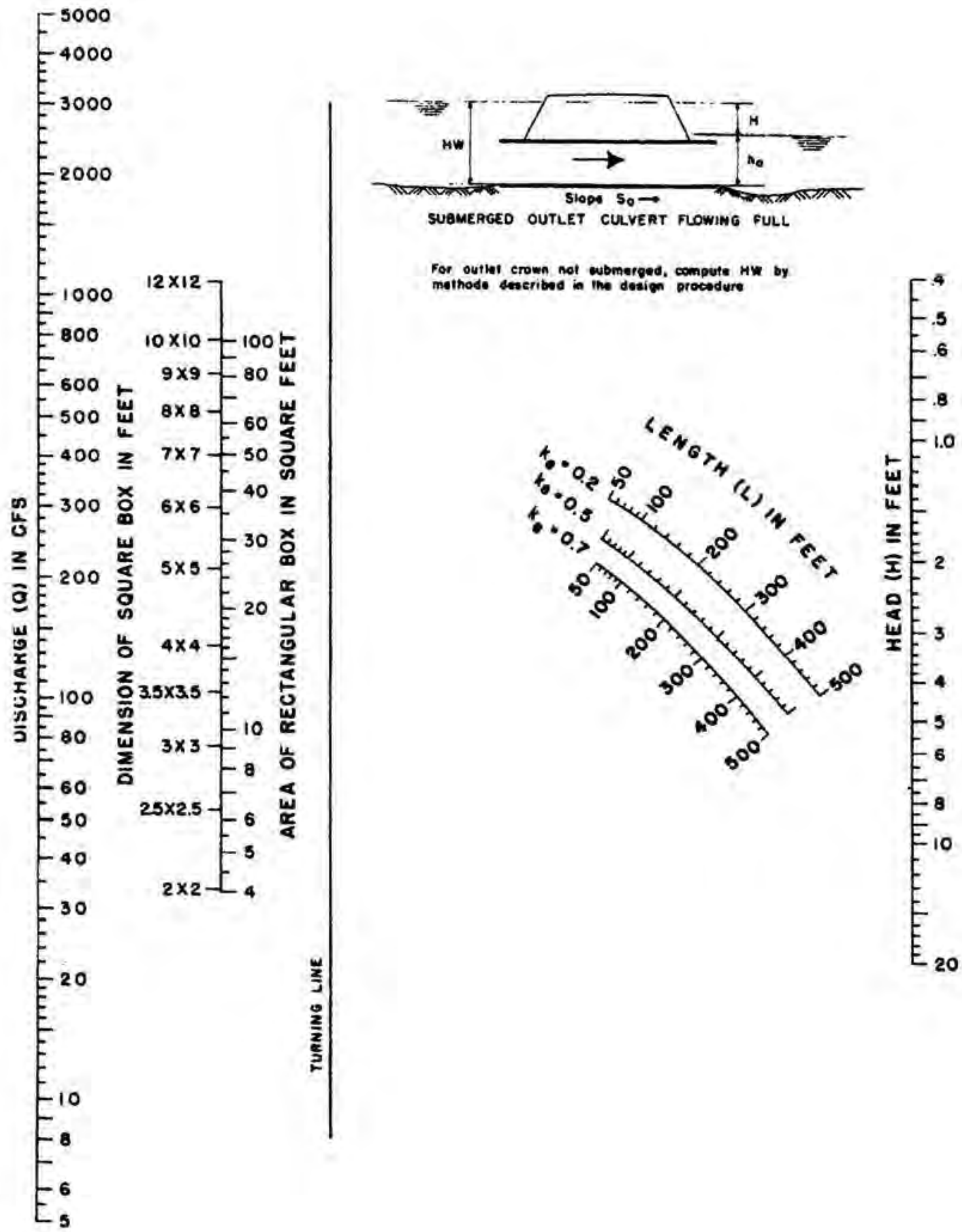
**FIGURE 1227**

## HEAD FOR CONCRETE BOX CULVERTS WITH OUTLET CONTROL, n=0.012

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| ORIGINAL ISSUE | 3/27/06 |
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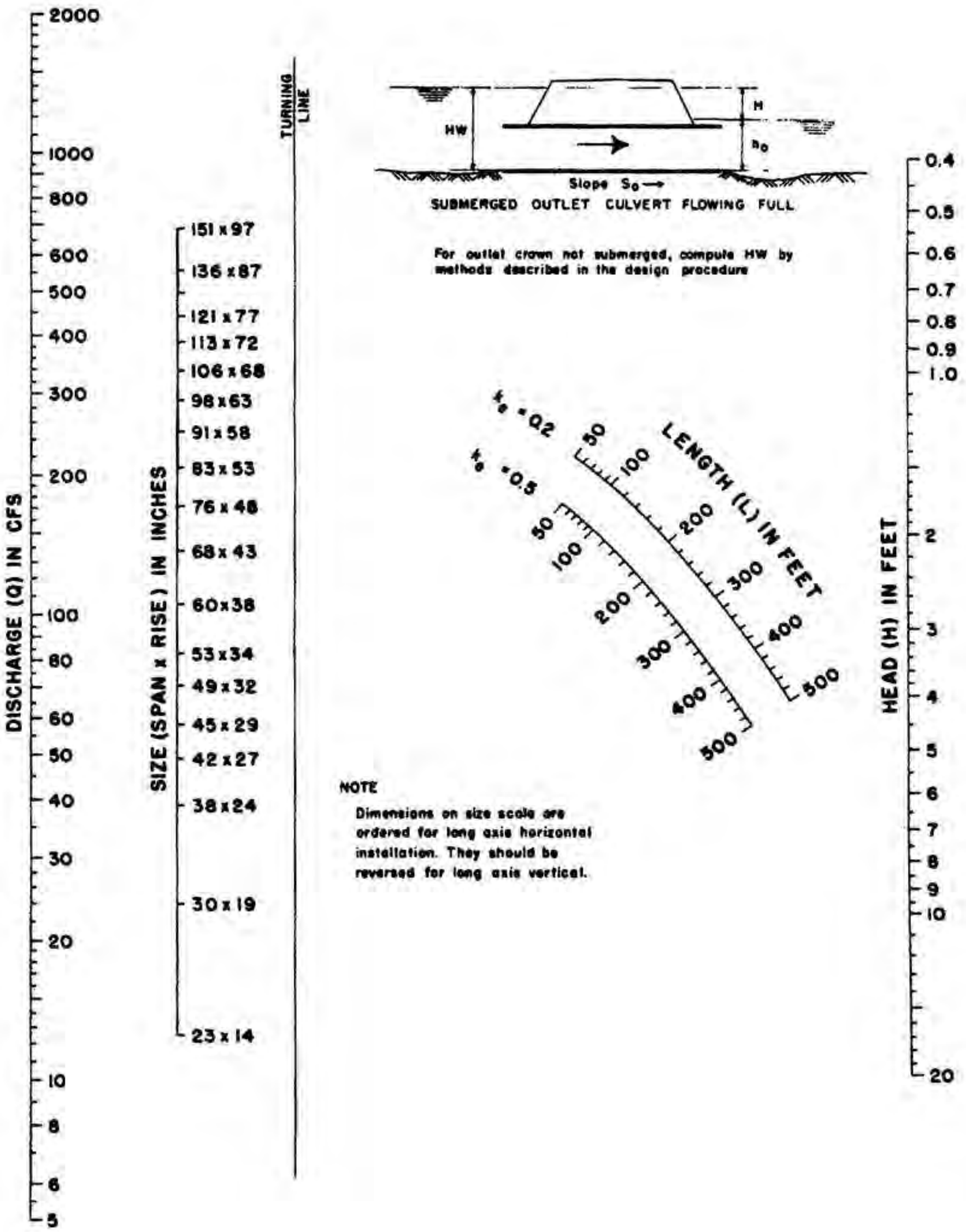
**FIGURE 1228**

## HEAD FOR OVAL CONCRETE PIPE CULVERTS LONG-AXIS HORIZONTAL OR VERTICAL WITH OUTLET CONTROL, n=0.012

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|---|---|
| ORIGINAL ISSUE | 3/27/06 |
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**FIGURE 1229**

# SECTION 1300

## IRRIGATION / DRAINAGE STRUCTURES

# SECTION 1300
## IRRIGATION / DRAINAGE STRUCTURES

## TABLE OF CONTENTS



## LIST OF TABLES



## LIST OF FIGURES

# SECTION 1300
# IRRIGATION / DRAINAGE STRUCTURES

## 1301 INTRODUCTION

A large number of agricultural irrigation facilities exist in Mesa County, and many have historically intercepted runoff from rural and agricultural areas with little consequence. However, the development (urbanization) of these areas results in storm runoff of much higher peak flows and larger total volumes. In addition, water quality of the runoff is often adversely impacted by this urbanization. As a result, the traditional practice of utilizing irrigation ditches, drains, and reservoirs for stormwater control must be reexamined on a case-by-case basis.

It is recommended that the designer/engineer, when faced with a specific irrigation / drainage structures interface, to review in detail Section 405 of this Manual. Only after a thorough review and understanding of this Section, and with coordination with the parties involved, shall the user proceed with the specific tasks that need to be performed. Further, the designer/engineer is cautioned to verify that damage to downstream properties will not occur by bypassing of storm runoff.

## 1302 IRRIGATION DITCH CROSSINGS (CROSS-DRAINAGE STRUCTURES)

It is common for a storm drainage system to encounter irrigation ditches, canals, or even conduits, especially in agricultural areas. Mesa County contains a large percentage of agricultural lands, thus the interaction of storm runoff systems and agricultural irrigation structures is common, especially for new developments. Storm drains are often buried with sufficient cover to completely avoid an interaction with existing irrigation structures. However, it may occasionally be necessary to install storm drain pipe by boring or jacking to avoid disruption of the irrigation flow. Where the invert of a drainage channel is low enough in relation to the irrigation structure, it may be possible to utilize a standard culvert design for the crossing (see Section 1200).

In locations where stormwater flow is in an open channel or relatively shallow pipe at an intersection with an irrigation structure, other options must be considered. In certain (rare) cases, it is allowable for stormwater flow to enter an irrigation canal and then be removed (see side-channel spillways in Section 904.1) at another location. Otherwise, stormwater flow shall be kept separate from irrigation conveyances as reasonably possible. Two methods for completing this task are presented in the following paragraphs: inverted siphons and overchutes. Overchutes include both flumes and pipe overchutes for the conveyance of stormwater over another channel.

### 1302.1 Inverted Siphons

An inverted siphon consists of a closed conduit used to convey water under an obstruction such as an irrigation structure or roadway where the use of a continuous-slope conduit would interfere with said obstruction. Sometimes called “sag pipes”, the conduit drops to an invert low enough to pass under the obstruction then rises to the channel invert at the downstream end. This infers pressure pipe flow, with successful operation being dependent on sufficient head at the upstream end to overcome the rise as well as pipe losses in the siphon section.

Transitions are recommended for the upstream and downstream ends of all siphons to reduce head losses and to prevent excessive erosion. Entrance head losses reduce the effective head on the inverted siphon, thereby requiring larger upstream depths to achieve the same flow through the conduit. Concrete inlet and outlet transitions are required for siphons which:

1. Cross railroads or state/federal highways.
2. Are 36 inches in diameter or larger and cross a road.
3. Are used with an unlined channel and the pipe velocity exceeds 3.5 feet per second.

In locations where the siphon may be affected by groundwater flow, it may be necessary to include pipe collars to reduce piping effects. Cutoff walls may also be necessary depending on site conditions.

It is recommended that the design of long inverted siphons include a blowoff structure at the low point of the alignment to allow for draining of the system (see Section 905.3.1). These can be designed for operation by pumping or gravity draining. Shorter siphons can usually be easily drained by pumping from either end of the structure.

Pipe used for inverted siphons shall be pressure-rated as required per design, shall utilize rubber gaskets (may use other joint connection devices in addition), and shall comply with the applicable pipe selection criteria set forth in Section 1004.1 of this Manual.

It is good design practice to include features in the design of an inverted siphon to minimize the risk of flooding due to the failure of the siphon to properly convey the channel flow. These features may include, but are not limited to:

1. Increased freeboard in the upstream channel in the vicinity of the siphon.
2. The use of multiple barrels to allow for at least partial operation if one barrel fails.
3. The installation of a wasteway (and associated side-channel spillway) to limit the depth of water in the upstream channel.

Inverted siphons pose a significant risk to human and animal safety. Specific features must be included in the design of these structures to help alleviate these risks. It is recommended that the location and safety features of any proposed inverted siphon be discussed with Mesa County and any local jurisdictions early in the design process. At some locations, a jurisdiction may disallow the use of these structures where they pose excessive or unwarranted risk to the public.

The design procedure for an inverted siphon is as follows (USBR 1974):

1. Determine an initial system layout with all known elevations and lengths. Pipe slopes between the inlet/outlet transitions and the main

section of the siphon are limited to a maximum slope of 2:1. All siphon pipes shall have a slope of at least 0.005.

2. Determine the type of inlet and outlet structures required (transitions, headwalls, etc.).
3. Determine the type of pipe to be used. This is typically pressure-rated reinforced concrete pipe.
4. Select initial pipe size based on the table in **Figure 1301**. This is based on design flow, transitions used, and the subjective length of the siphon. Presented with the table are maximum permissible pipe velocities for different siphon lengths and transition types. Siphons are considered to be relatively short if they are crossing under a road or a canal. Only flows of up to 99 cfs are included in **Figure 1301** since it is typically more economical to consider a bridge at flows of 100 cfs or higher. However, multiple barrels may be utilized to convey larger flows.
5. Using the design flow rate and the properties of the initially selected pipe, determine the velocity head in the pipe ($H_{vp}$) and the friction slope ($S_f$). Using the normal depth in the upstream channel, find the velocity head ($H_{v1}$).
6. Determine the additional freeboard required ($FB_{add}$) for the 50 feet of channel upstream of the structure:

$$FB_{add} = 0.5\,(\text{Normal Canal Bank Freeboard}) \qquad (1301)$$

7. Invert elevations of the transitions are set to allow for an adequate hydraulic seal at the inlet (to minimize hydraulic loss) and to avoid submergence at the outlet. Due to the sloping pipe inlet, the effective diameter is larger than the pipe diameter:

$$D_i = \frac{D}{\cos\alpha} \qquad (1302)$$

Where:

$D_i$ = Effective Inlet Diameter (feet)

$D$ = Siphon Pipe Diameter (feet)

$\alpha$ = Slope of Inlet ($\alpha_1$) or Outlet ($\alpha_2$) Pipe (degrees)

Required hydraulic seal is based on the difference in velocity heads between the upstream channel and the pipe:

$$H_{seal} = 1.5 \cdot \Delta H_v = 1.5\left(H_{vp} - H_{v1}\right) \qquad (1303)$$

Where:

$H_{seal}$ = Hydraulic Seal Required at Inlet, Min. 0.25' (feet)

$H_{vp}$ = Velocity Head in the Pipe $\left(\frac{V^2}{2g}\right)$ (feet)

$H_{v1}$ = Velocity Head in the Upstream Channel (feet)

Throughout the remainder of this process, the designer is referred to **Figure 1301** for the locations of Stations A through H and J. Note that the siphon in **Figure 1301** is crossing under a roadway. In this section, the focus is on irrigation canal crossings, so the cover requirements may differ from those in the Figure. Siphons crossing under a channel with flexible lining shall have a minimum of 2.0 feet of cover, and those crossing under a canal with concrete or other non-flexible lining shall have a minimum cover of 6 inches.

**Table 1301** presents equations for finding invert elevations at Stations A through H:

**Table 1301 Inverted Siphon Invert Elevation Equations**

| Station (see Figure 1301) | Invert Elevation |
|---|---|
| A | Channel IE @ 10' U/S from the U/S transition |
| C | NWS Elev. @ Sta. A – $H_{seal}$ + $D_i$ |
| B | Minimum: Channel IE @ U/S end of transition<br>Maximum: IE Sta. C + $p_{inlet}$ |
| G | Channel IE @ 10' D/S from the D/S transition |
| H | Channel IE @ 10' D/S from the D/S transition |
| F | IE @ Sta. G - $p_{outlet}$ |

Where:

$IE = \text{Invert Elevation (feet)}$

$NWS = \text{Normal Water Surface (Design Flow) (feet)}$

$p = \text{Difference in invert elevations between the ends of the transitions (Sta. B and C or F and G) (feet)}$

$p_{inlet} \le \frac{3}{4} D \text{(feet)}$

$p_{outlet} \le \frac{1}{2} D \text{(feet)}$

The invert elevations of Stations D, J, and E are determined by cover requirements and pipe slope.

8. Determine the total amount of hydraulic head available ($H_{profile}$) across the siphon profile:

$$H_{profile} = NWS @ Sta.\,A - NWS @ Sta.\,H \quad (1304)$$

9. Determine the approximate head loss across the preliminary profile, as well as a 10% factor of safety as included in Equation 1305:

$$H_L = 1.1(h_i + h_f + h_b + h_o) \quad (1305)$$

Where:

$H_L$ = Head loss across siphon profile (with 10% F.S.) (feet)

$h_i$ = Inlet transtion head loss = $0.4\Delta H_v$ (feet)

$h_f$ = Pipe friction losses = Pipe Length $\times S_f$ (feet)

$h_b$ = Bend losses (See Equation 1026) (feet)

$h_o$ = Outlet transition head loss = $0.7\Delta H_v$ (feet)

$\Delta H_v = (H_{vp} - H_{v1})$

For the siphon to function properly, the total head loss $H_L$ must not exceed the available head across the profile, $H_{profile}$.

10. Headwall height above the transition invert is dependent on backfill height and required freeboard. In many cases, the top elevation of the headwall is equal to the top of wall elevation at the cutoff, Station B. (The wall height there is determined by adding standard channel freeboard to the additional freeboard calculated in Equation 970.)

11. Transition dimensions C, B, and cutoff depth e, and wall thickness $t_w$ are defined by **Figure 1302** (USBR 1974). Transition length L shall be at least 3 times the pipe diameter D.

12. For most siphons, forces exerted on the pipe bends are not great enough to warrant additional structural considerations. However, for large pipes, high heads, poor foundation conditions, and large deflection angles, thrust blocks and other appurtenances shall be considered. (USBR 1974)

13. The locations of Stations C and F are determined by the width of the obstruction and the siphon pipe slope requirements. Using the new locations for Stations C and F, determine new locations for Stations A, B, G, and H, and recompute invert elevations as applicable.

14. Recompute total head loss (Equation 1305) and verify that the final siphon profile is viable.

15. Erosion protection may be required upstream and/or downstream from the siphon transitions. See Section 800 of this Manual. Pipe collars may be necessary to reduce the effects of piping due to percolation along the outer wall of the pipe. A blowoff (drain) valve may be warranted for longer pipes. See Section 906 for criteria regarding these appurtenances.

16. Air vents, pressure-release valves, or air jumper pipes may be necessary to relieve air pressure in the siphon pipe, especially under less-than-capacity flows. A hydraulic jump can occur in the pipe, causing blowback and significantly reducing the capacity of the siphon. The aforementioned appurtenances allow for the release of trapped air from the pipe. See Section 906 for criteria regarding these appurtenances.

## 1302.2 Overchutes (Flumes and Pipe Overchutes)

The term "overchutes" refers to cross-drainage structures that pass over the normal water surface elevation of the drainage being crossed. While typically used to convey stormwater (or other) across an irrigation canal or natural drainageway, other obstructions such as roads and railroad tracks occasionally require these structures. Where the obstruction being crossed conveys anything other than water (such as vehicles), clearance to the overchute must be considered. The designer shall ensure that an overchute installed at a waterway will not adversely impact the design capacity of that waterway. This includes a minimum clearance of 1.0 feet over the normal water surface of the waterway.

Overchutes can be open-channel structures (flumes) or pipes, depending on site conditions. Rectangular concrete flume sections are typically used for larger cross-drainage (e.g., stormwater) flows and for areas where debris is expected to prohibit the use of pipes. **Figure 1303** presents a typical plan and profile for a rectangular concrete flume overchute. For smaller cross-drainage flows, pipe overchutes are typically employed for economic reasons. See **Figure 1304** for typical plans and profiles for two pipe overchute types. Note that welded steel pipe is specified for the suspended section of each due to the additional forces and exposure to which those sections are subjected. Both Figures assume that the obstruction being crossed is an existing trapezoidal canal.

### 1302.2.1 Flume Overchutes

The design of a flume overchute ("chute") is as follows (USBR 1974):

1. The chute invert slope is dependent upon the selected method of energy dissipation at the chute outlet. Some, such as a stilling basin, best function with supercritical influent. Others, such as a baffled apron drop (Section 902.4), require subcritical flow at the entrance. Also, a wider, shallower flume section will require a smaller depth at the inlet pool and will more easily satisfy freeboard requirements. Therefore, the most economical flume section may not be the most efficient overall.

   Where supercritical flow is desired, the following equations are used to determine critical slope:

$$d_c = \frac{q^{2/3}}{g^{1/3}} \tag{1306}$$

$$A_c = b \cdot d_c \tag{1307}$$

$$V_c = \frac{Q}{A_c} \tag{1308}$$

$$H_{V_c} = \frac{V_c^2}{2g} \tag{1309}$$

$$E_{s_c} = d_c + H_{V_c} \tag{1310}$$

$$S_c = \left[\frac{nV_c}{1.49 \cdot R^{2/3}}\right]^2 \qquad (1311)$$

Where:

$d_c$ = Critical depth (feet)

$q$ = Flow per unit width (cfs/ft)

$A_c$ = Critical area (sf)

$V_c$ = Critical velocity (fps)

$H_{V_c}$ = Velocity head (critical flow) (feet)

$E_{S_c}$ = Specific energy (critical flow) (feet)

$S_c$ = Critical slope (ft/ft)

$n$ = Manning's roughness = 0.015

$R$ = Hydraulic radius (feet)

The chute invert slope shall be at least 20% greater than critical slope ($S_c$) for the selected cross section to avoid unstable flow that occurs around critical flow.

Where subcritical flow is desired, the slope shall be set appreciably less than critical slope as defined by Equation 1311.

Normal depth at design flow in the flume shall be a minimum of 1.0 feet below the top of the flume wall for all non-piped overchutes. Normal depth shall be based upon a Manning's roughness coefficient of 0.015 for the purpose of this criterion.

2. Determine the required pool depth at the inlet transition, $d_o$. Equation 1312 assumes that velocity in the inlet pool is zero:

$$d_o = d_c + H_{V_c} + 0.3 \cdot H_{V_c} \qquad (1312)$$

Provide 2.0 feet of freeboard above the pool depth in the channel upstream from the overchute, and 1.0 feet of freeboard above the pool depth in the flume from the inlet pool to the suspended portion.

3. The chute wall height across the suspended portion of the flume shall be equal to the maximum depth in the chute plus 1.0 feet of freeboard.

4. Determine the flow depth at the downstream end of the chute using the Bernoulli Equation iteratively on $d_2$:

$$E_{S_2} + h_f = E_{S_1} + S_o L \qquad (1313)$$

Where:

$E_{s_2}$ = D/S Specfic Energy = $d_2 + H_{v_2}$ (feet)

$h_f$ = Friction Loss (feet)

$E_{s_1}$ = U/S Specfic Energy = $d_1 + H_{v_1}$ (feet)

$S_oL$ = Vertical Drop across Chute (feet)

5. A stilling basin or other energy dissipation structure shall be included as necessary per Sections 902 and 903 of this Manual.

1302.2.2 Pipe Overchutes

The design of a pipe overchute is as follows (USBR 1974):

1. The invert of the inlet transition is limited by the following:
   a. At least 1.0 feet of clearance shall be provided between the irrigation canal water surface and the pipe, where applicable. Larger clearances may be required for other types of obstructions.
   b. 2.0 feet of bank freeboard shall be provided above the maximum water surface in the upstream channel.
   c. Inlet losses reduce the effective capacity of the pipe. Therefore, it is recommended that the inlet opening be submerged by a minimum of $1.5H_{v(pipe)}$ to offset this loss and maintain one full velocity head in the pipe.

2. Determine inlet/outlet control:

   For inlet control, the required depth at the inlet ($d_i$) for a discharge of Q through the pipe is determined using Equation 1314:

$$d_i = \frac{Q^2}{2gC^2A^2} \tag{1314}$$

Where:

C = Orifice discharge coefficient = 0.6

$d_i$ = depth(feet)

Q = Flow rate (cfs)

A = cross-sectional area of pipe (sf)

q = gravitational constant, $32.2 \frac{ft}{s^2}$

   Under outlet control, the required head for discharge Q is equal to the head losses through the pipe:

$$d_i = \sum H_L = h_i + h_f + h_o \tag{1315}$$

Where:

$H_L$ = Head loss across pipe (feet)

$h_i$ = Inlet head loss(feet)

$h_f$ = Pipe friction losses = Pipe Length $\times S_f$ (feet)

$h_o$ = Outlet transition head loss (feet)

3. Pipe overchutes shall be designed with a maximum full pipe velocity of 10 feet per second for concrete transition outlets or 12 feet per second for baffled outlets.

4. Pipe diameter is determined by:

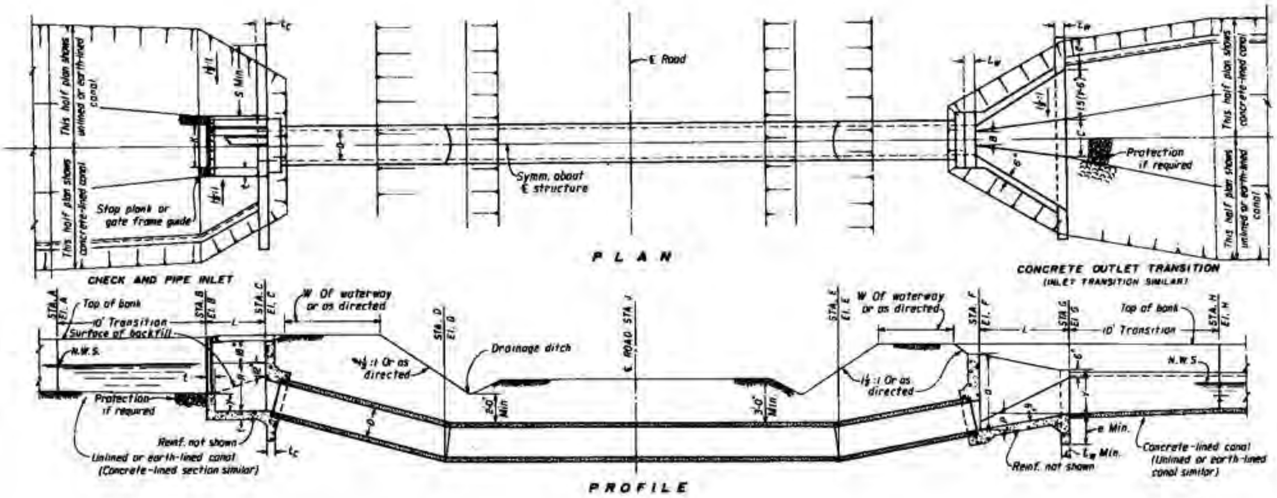
$$D = 1.13\sqrt{\frac{Q}{V}} \qquad (1316)$$

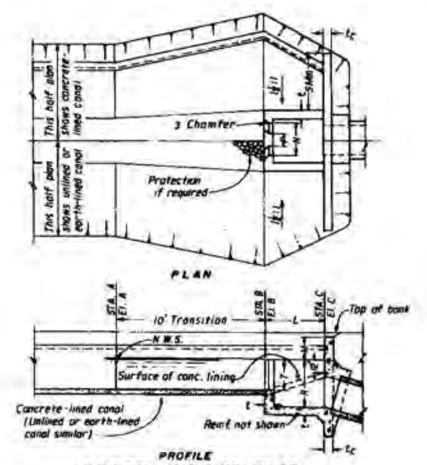
   The minimum recommended diameter for an overchute pipe for the purpose of conveying stormwater is 24 inches. Depending on expected sediment and debris loads, some sites may necessitate larger minimum diameters.

5. Structural and reinforcement requirements are not specified herein as they are outside the scope of this Manual. Support piers shall be located so as to minimize potential adverse effects on the operation of the obstruction being crossed.

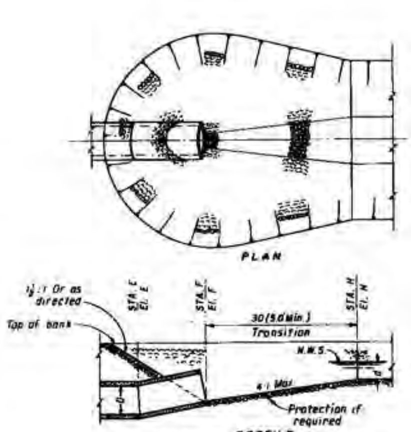
6. An energy dissipation structure and/or other erosion protection shall be installed downstream of the overchute as applicable per Sections 800, 902, 903, and 1200 of this Manual.

# INVERTED SIPHON




CONTROL AND PIPE INLET




EARTH OUTLET TRANSITION
(INLET TRANSITION SIMILAR)

**PIPE DIAMETER SELECTION DATA**

| MAX. V<3.5fps (EARTH TRANSITION) Q(cfs) | | MAX. V=5.0fps (CONC. TRANSITION) Q(cfs) | | MAX. V=10.0fps (CONC. TRANSITION) Q(cfs) | | PIPE | |
|---|---|---|---|---|---|---|---|
| FROM | INCLUDING | FROM | INCLUDING | FROM | INCLUDING | DIA. (INCHES) | AREA (SQ. FT.) |
| 0 | 2.7 | 0 | 3.9 | 0 | 7.9 | 12 | 0.785 |
| 2.7 | 4.3 | 3.9 | 6.1 | 7.9 | 12.3 | 15 | 1.227 |
| 4.3 | 6.2 | 6.1 | 8.8 | 12.3 | 17.7 | 18 | 1.767 |
| 6.2 | 8.4 | 8.8 | 12.0 | 17.7 | 24.1 | 21 | 2.405 |
| 8.4 | 11.0 | 12.0 | 15.7 | 24.1 | 31.4 | 24 | 3.142 |
| 11.0 | 13.9 | 15.7 | 19.9 | 31.4 | 39.8 | 27 | 3.976 |
| 13.9 | 17.2 | 19.9 | 24.5 | 39.8 | 49.1 | 30 | 4.909 |
| 17.2 | 20.8 | 24.5 | 29.7 | 49.1 | 59.4 | 33" | 5.940 |
| 20.8 | 24.7 | 29.7 | 35.3 | 59.4 | 70.7 | 36 | 7.069 |
| 24.7 | 29.0 | 35.3 | 41.5 | 70.7 | 83.0 | 39 | 8.296 |
| 29.0 | 33.7 | 41.5 | 48.1 | 83.0 | 96.2 | 42 | 9.621 |
| 33.7 | 38.7 | 48.1 | 55.2 | | | 45 | 11.045 |
| 38.7 | 44.0 | 55.2 | 62.8 | | | 48 | 12.566 |
| 44.0 | 49.7 | 62.8 | 70.9 | | | 51 | 14.186 |
| 49.7 | 55.7 | 70.9 | 79.5 | | | 54 | 15.904 |
| 55.7 | 62.0 | 79.5 | 88.6 | | | 57 | 17.721 |
| 62.0 | 68.7 | 88.6 | 98.2 | | | 60 | 19.635 |
| 68.7 | 75.8 | | | | | 63 | 21.648 |
| 75.8 | 83.2 | | | | | 66 | 23.758 |
| 83.2 | 90.9 | | | | | 69 | 25.967 |
| 90.9 | 99.0 | | | | | 72 | 28.274 |

**NOTES**

Locations of vertical P.I.'s at D and E are to be considered approximate only and may be adjusted to fit actual laying length of pipe.
Stations and elevations refer to invert unless otherwise shown.

| Revision | Date |
|---|---|
| ORIGINAL ISSUE | 3/27/06 |
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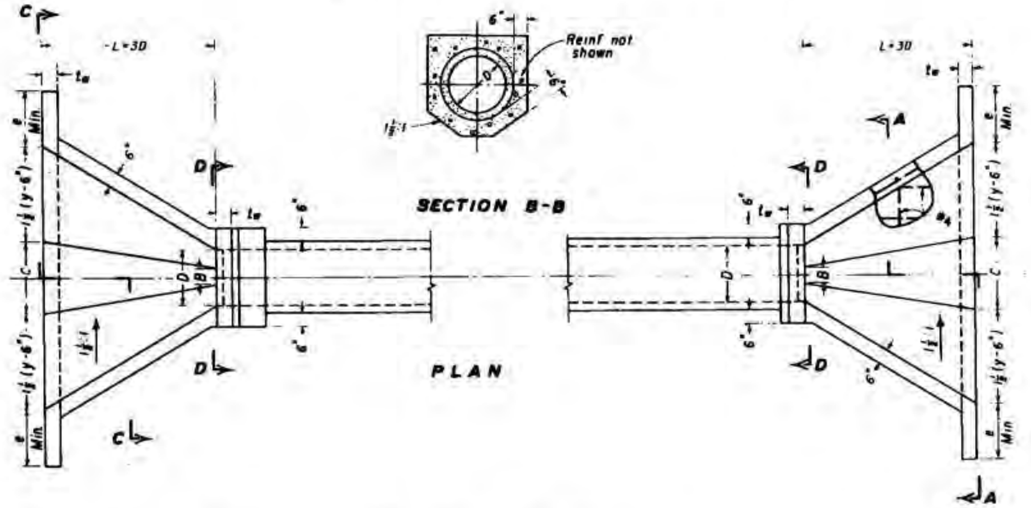
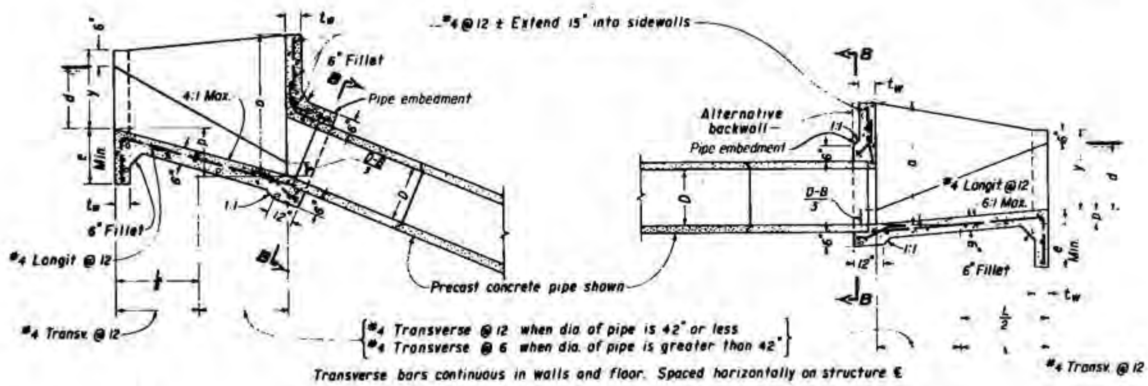
WRC ENGINEERING, INC.

REFERENCE: DESIGN OF SMALL CANAL STRUCTURES (USBR, 1974)

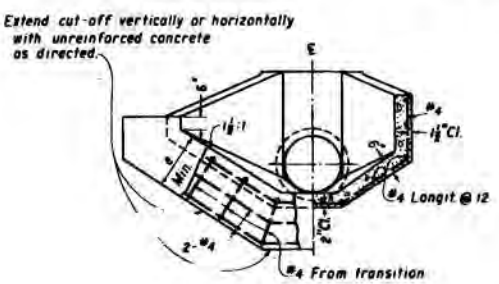
**FIGURE 1301**

# INVERTED SIPHON (TRANSITION DETAILS)

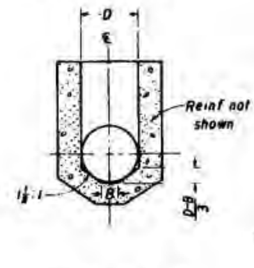



LONGITUDINAL SECTION




SECTION A-A
(SECTION C-C SIMILAR)




SECTION D-D

### DIMENSIONS e AND $t_w$

| DEPTH (d) OF WATER IN CANAL | e | $t_w$ |
|---|---|---|
| 0' to 3' | 24" | 6" |
| 3' to 6' | 2'-6" | 8" |
| Greater than 6' | 3'-0" | 8" |

### NOTES

The minimum freeboard at the transition cutoff of structures in earth lined or unlined canals should be 6 inches for depths of water at the cutoff of 0 to 1.25 feet, 9 inches for depths 1.26 to 2.00 feet, and 12 inches for depths 2.00 to 5.00 feet.
The freeboard at the transition cutoff of structures in hard surface lined canals should be the same as the freeboard of the lining.
The freeboard at transition headwalls should be equal to or greater than the freeboard at the cutoff for 24 inch pipe and smaller and the freeboard should increase as the size of the structure increases.
B = .303D
y = Depth (d) at cutoff plus freeboard at cutoff
The elevation of the invert at the headwall is determined from the required submergence at the headwall opening.

### DIMENSION C

| RELATIONSHIP OF PIPE DIAMETER (D) TO DEPTH (d) IN CANAL | C WITH WATER SURFACE ANGLE OF 22 ½° | C WITH WATER SURFACE ANGLE OF 25° | C WITH WATER SURFACE ANGLE OF 27 ½° |
|---|---|---|---|
| D = d | .5D | .8D | 1.1D |
| D = 1.25d | 1.1D | 1.4D | 1.7D |
| D = 1.5d | 1.5D | 1.8D | 2.1D |
| D = 2d | 2.0D | 2.3D | 2.6D |

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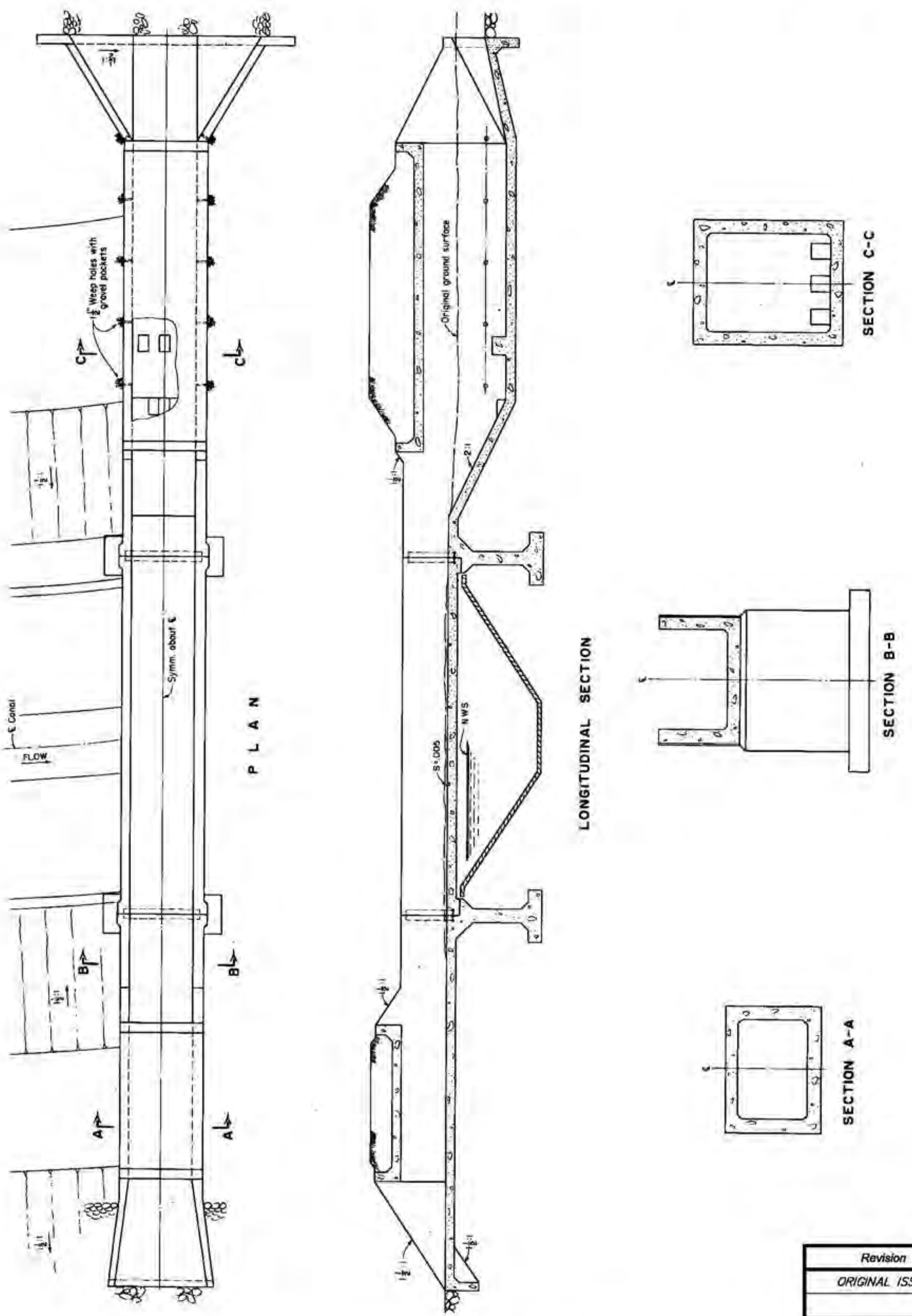
| ORIGINAL ISSUE | 3/27/06 |

STORMWATER MANAGEMENT MANUAL

# RECTANGULAR CONCRETE FLUME OVERCHUTE




| Revision | Date |
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| ORIGINAL ISSUE | 3/27/06 |
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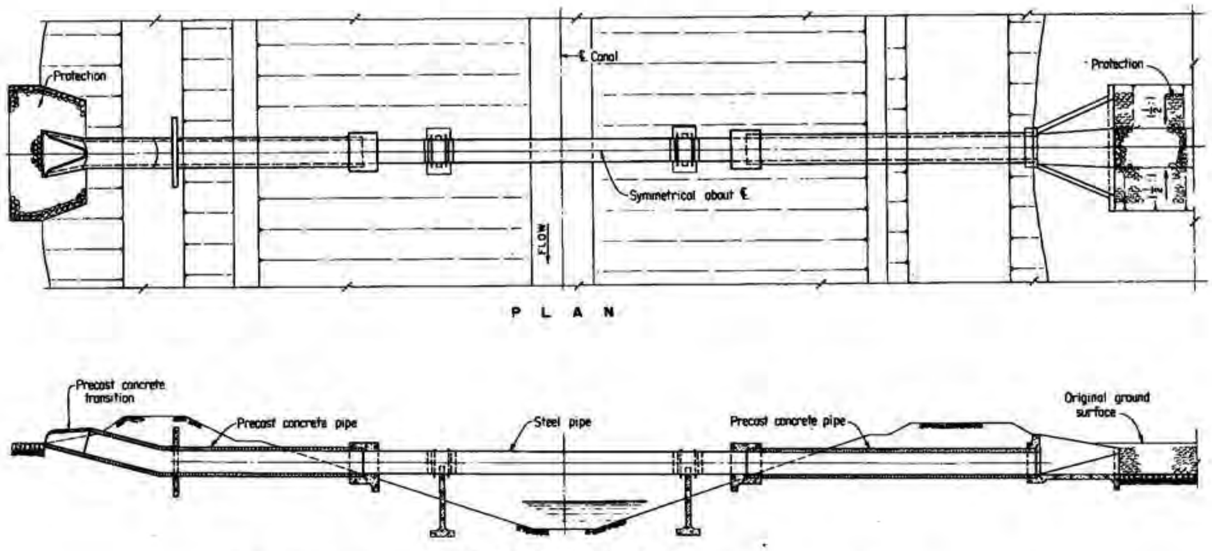
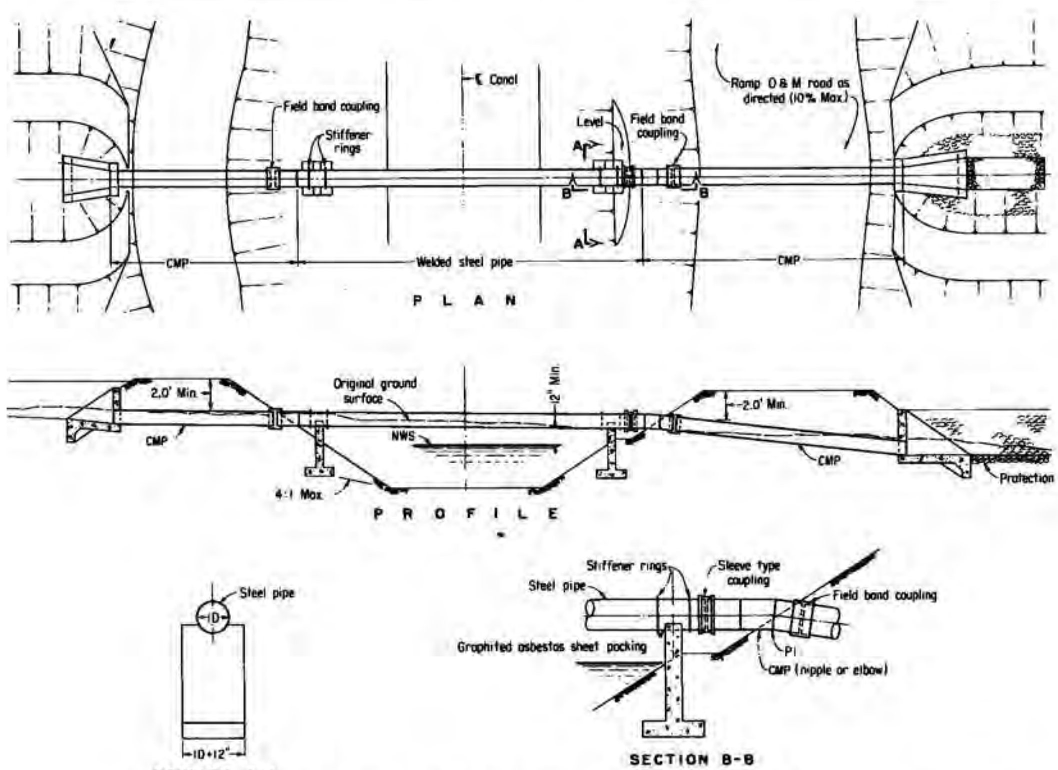
**FIGURE 1303**

# PIPE OVERCHUTES




WELDED STEEL PIPE AND PRECAST CONCRETE PIPE




WELDED STEEL PIPE AND CORRUGATED METAL PIPE

| Revision | Date |
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| ORIGINAL ISSUE | 3/27/06 |
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WRC ENGINEERING, INC.

REFERENCE: DESIGN OF SMALL CANAL STRUCTURES (USBR, 1974)

**FIGURE 1304**

# SECTION 1400

## DETENTION AND RETENTION

# SECTION 1400
## DETENTION AND RETENTION

## TABLE OF CONTENTS

## LIST OF TABLES



## LIST OF FIGURES

# SECTION 1400
# DETENTION AND RETENTION

## 1401 INTRODUCTION

The main purpose of a detention basin is to store runoff and reduce peak discharge by allowing flow to be discharged later at a controlled rate, and within a reasonable time. 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 predevelopment conditions (local facilities only). Detention has been shown to be very beneficial in controlling flood peaks in an urbanized area. Use of detention includes local detention such as in-channel or within a parking lot, and regional detention, such as a large recreational lake or reservoir and off-line detention facilities. Regional and local detention facilities are more fully discussed below. Policy regarding detention and extended detention (retention) is found in Sections 403.7 and 403.8 of this Manual.

It is important that any extended detention (retention) be supported by valid water rights that allows for retention without impacting any vested water rights on the stream system.

## 1402 DETENTION VERSUS RETENTION

Stormwater storage reservoir types are numerous, but they essentially fit into one of two categories: detention or retention. The words "pond" and "basin" are used interchangeably when used in connection with both detention and retention reservoirs. A detention basin or pond "detains" water temporarily, releasing water through a pipe or channel by means of a weir, orifice, or pump. Because of its ability to release flow during inflow, the overall volume of storage required for a given storm event is reduced. Another advantage of the detention basin is the positive means of outflow, resulting in fewer problems with long-term ponding. A retention basin or pond "retains" water without any initial release during inflow. Once the storm event is over, pond drainage may occur due to evaporation and percolation into the soil. In some instances, retention basins may also involve a gated pipe or pump which is closed or inoperative during the storm event. However, if a gated pipe or pump is an available or desirable option, it would normally be advantageous to release water during stormwater inflow, which would change the basin from a retention basin to a detention basin. The difference in detention and retention basins is depicted in **Figure 1401**.

The words "pond" and "basin" may be used to refer to reservoirs that remain dry outside of storm events, or store water for other purposes, e.g., irrigation, recreation, aesthetics, etc., in addition to receiving stormwater during storm events. Words "wet" and "dry" are used as prefixes to describe the condition of ponds and basins. However, a wet pond and dry basin each have a specific meaning.

Wet ponds may be desirable compared to dry basins in some circumstances. It may be that ample storage volume exists to provide an aesthetic, recreational, or irrigation pond below the required reservoir volume. Use of irrigation storage facilities for stormwater detention purposes must be reviewed by the appropriate jurisdiction(s) on a case-by-case basis. It is required that the stormwater reservoir volume provided must be in addition to the maximum expected base storage (irrigation or wet pond) volume. This is depicted in **Figure 1401**.

## 1403 LOCAL AND REGIONAL DETENTION FACILITIES

### 1403.1 Local Detention Facilities

Local detention facilities are usually designed by and financed by developers or local property owners. 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.

#### 1403.1.1 Local Minor Facilities

Local minor detention facilities are defined as serving a single development with a hydrologic basin smaller than or equal to 20 acres and are designed to mitigate the impact of increased runoff due to development. The outlet capacity is generally based on predevelopment hydrology, and the detention structures are generally small (0.01 to 1 acre-foot). Detention storage volume may be provided as small landscaped or turfed basins, parking lot storage, or a suitable combination of all.

#### 1403.1.2 Local Major Facilities

Local major detention facilities are defined as serving more than a single development or serving hydrologic basins greater than 20 acres in size. These facilities may serve dual functions. They typically reduce existing flooding problems to allow more development and/or control increased runoff caused by additional 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 larger size, these basins are designed much the same as regional detention facilities.

### 1403.2 Regional Detention Facilities

Regional detention facilities are those identified in the current Grand Valley Stormwater Master Management plan or as designated by local jurisdiction. Generally, these facilities control flow on major drainageways, are large in size, and are owned and maintained by public agencies. The purpose of these facilities is to significantly reduce downstream flows in order to maximize the capacity of existing systems and maintain flows at or below historic rates.

## 1404 DRAINAGE FEE IN LIEU OF DETENTION

The developer may be given the option of paying a drainage fee in lieu of providing adequate detention facilities. Such an option may be considered only if the developer completes an engineering analysis to show that all downstream facilities have adequate capacity to handle the un-detained flows from the proposed development. If such option is allowed, this does not waive the requirements for drainage planning submittal requirements as outlined in Section 300 of this Manual.

### 1404.1 Drainage Fee Basis

For stormwater management purposes, charges assessed to existing and new development to offset cost of providing drainage facilities and services are typically categorized as an impact fee or a utility fee.

An impact fee is based on the cost of upgrading drainage services within a watershed, shared among all new developments within the watershed. The basis for "sharing" costs can be as simple as the size of the water tap and as complex as calculating imperviousness of the property then making adjustments for various best management practices, such as detention. An impact fee is a one-time charge against new development.

A utility fee is based on the cost of upgrading and maintaining drainage services within a watershed to meet current standards or level-of-service. A utility fee is often assessed to all properties, developed or not and often is limited to providing maintenance of the system and minor capital improvements. Utility fees are normally charged on a monthly basis.

Since the local jurisdiction charge is a fee-in-lieu of detention for new development, it is considered a limited impact fee. Reasonableness tests applied to impact fees include:

- The system impact fee shall be related to the amount of drainage improvements needed to meet local standards.
- The impact fee shall only cover capital and related (i.e. engineering, construction administration, etc.) costs for improvements.

### 1404.2 Drainage Fee Calculation

The formula used by local jurisdictions to calculate drainage fee is of the form:

$$\text{Drainage Fee (\$)} = B(C_D - C_H)A^{0.7} \qquad (1401)$$

Where:

$B$ = Fee Constant (established annually by City Council of local jurisdiction)

$C_D$ = 100-Yr. runoff coefficient (expressed as a decimal) based on developed land use conditions

$C_H$ = 100-Yr. runoff coefficient (expressed as a decimal) based on pre-developed land use conditions

$A$ = Area of Development (acres)

Assigning B = \$14,000, A = 1 acre, $C_D$ = 0.60 and $C_H$ = 0.51, based on SCS Type C soils, and an imperviousness of 2% pre-developed and 50% developed, the fee-in-lieu of detention would be \$1,260.

The equation used in Mesa County and the other local jurisdictions takes into account key factors for a fee-in-lieu of detention, such as:

- The fee is based on area.
- The fee takes into account imperviousness by using the Rational Method C values.
- The fee takes into account the difference between pre- and post-development runoff rates, which is the main function of onsite detention.

The constant B needs to closely reflect the actual cost of providing detention, and must be changed with time as the need arises. Local jurisdictions will review this figure annually and make appropriate recommendations for its adjustment.

## 1405 DETENTION FOR MANAGEMENT OF STORMWATER QUALITY

In addition to the primary purpose of detention and retention facilities to reduce peak flows, they can be adapted to enhance stormwater quality.

The outlet for a detention basin can be designed to allow for a slower rate of release and thus provide time for particulate pollutants to settle out of the stormwater. Such basins are called extended detention basins (EDB). It is recommended that an EDB outlet be designed to completely drain a full basin in 48 hours, allowing for the removal of a significant portion of insoluble pollutants with proper operation and maintenance.

A retention pond (RP) is a sedimentation facility designed to have a permanent water pool. This pool of water is mixed with stormwater during storm runoff events from frequently occurring storms and allows for sedimentation and thus water quality enhancement to occur.

For frequently occurring storms, both EDBs and RPs capture the total runoff as a surcharge. However, in the case of RPs, the stormwater is allowed to mix with the permanent pool water as it rises above the permanent pool level. All surcharged water above the permanent pool level is to be released over 48 hours. Details related to the design of EDBs and RPs can be found in Section 1500 *Construction Site Stormwater Runoff Control* and 1600 *Post-Construction Stormwater Management* of this Manual.

For the 100-year detention, the water quality volume is considered to be a part of the volume required for detention purposes.

## 1406 GENERAL DETENTION AND RETENTION CRITERIA

### 1406.1 Design Storm Criteria

Peak runoff from a site may not be increased in the 10- and 100-year storms due to development. The site runoff may be a composite of detention/retention basin release/overflow and direct runoff, both of which must be considered. If direct runoff is allowed from the site, the sum of the direct runoff plus the release from the detention basin must not exceed the historic rate. This is depicted in **Figure 1402** and discussed further in Section 1407.3.

### 1406.2 Geometric Requirements

For proper function and safety considerations, geometric requirements shall be as shown on **Figure 1403**.

### 1406.3 Dry Basin Bottom Drainage

Most drainage conveyance systems are designed to divert even minor nuisance flows to stormwater storage facilities. For dry basins, this can present an aesthetic and maintenance problem. Conveyance facilities to a dry basin shall be capable of transporting flow to the outlet facility rather than causing a soggy bog condition that cannot be properly maintained. Facilities conveying trickle or nuisance flows, such as from irrigation sprinklers, shall be adequate to convey at least 0.5 cfs. Reference is made to **Figures 1404 and 1405**.

The outlet facility for a retention basin would be a dry well or riprap filled dissipation pit. For a detention basin, the nuisance flows shall be conveyed to the basin outlet.

### 1406.4 Accessibility and Maintenance

All reservoirs or ponds which serve more than a single lot or site must be provided with a detention/retention tract dedicated for maintenance access. Maintenance of required volume and inflow and outflow works is necessary for the facility to function as required. Dedicated rights-of-way and/or easements must be provided as required in Section 402.2 of this Manual.

### 1406.5 Calculating Storage Volume Available

Storage volume shall be calculated by the methods shown prescribed in **Figure 1406**.

### 1406.6 Groundcover and Landscaping

After final grading, the slopes and bottom of each detention and retention basin shall be protected from erosion by seeding and mulching, sodding or other approved groundcover and shall be in accordance with jurisdictional specifications.

The planting of trees and shrubs on the slopes of stormwater basins is also encouraged. Temporary and/or permanent irrigation systems shall be provided as required for the type of groundcover and landscape installed and approved.

### 1406.7 Freeboard Requirements

The minimum required freeboard for open space detention/retention facilities is 1.0 feet above the computed 100-year water surface elevation.

### 1406.8 State Engineer's Office

Any dam constructed for the purpose of storing water, with a surface area, volume, or dam height as specified in Colorado Revised Statues 37-87-105 as amended, shall require the approval of the plans by the State Engineer's Office. All detention and retention storage areas shall be designed and constructed in accordance with these criteria. Those facilities subject to state statutes shall be designed and constructed in accordance with the criteria of the state.

### 1406.9 Embankment Protection

Whenever a detention pond uses an embankment to contain water, the embankment shall be protected from catastrophic failure due to overtopping. Overtopping can occur when the pond outlets become obstructed or when a larger than 100-year storm occurs. Failure protection for the embankment may be provided in the form of a buried heavy riprap layer on the entire downstream face of the embankment or a separate emergency spillway having a minimum capacity of twice the maximum release rate for the 100-year storm. Structures shall not be permitted in the path of the emergency spillway or overflow. The invert of the emergency spillway shall be set equal to or above the 100-year water surface elevation.

### 1406.10 Inlet and Outlet Design

The outlet from the detention basin shall consist of a short (maximum 25 feet) length(s) of CAP or RCP with an 18-inch minimum diameter. Multiple pipe outlets may be required to control different design frequencies. The invert of the lowest outlet pipe shall be set at the lowest point in the detention pond or at the top of the minimum pool, if present. The outlet pipe(s) shall discharge into a standard manhole or into a drainageway with proper erosion protection for the proposed structure. If an orifice plate is required to control the release rates, the plate(s) shall be hinged to open into the detention pipes to facilitate back flushing of the outlet pipe(s).

Inlet to the detention pipes can be by way of surface inlets and/or by a local private storm sewer system.

### 1406.11 Off-Site Flows

It is rarely the case that a new development comprises an entire drainage basin. Instead, developments are most often located within a larger drainage basin. This means that unless the new development is located at the very upstream edge of a drainage basin, it will most likely receive sheet flow, if not concentrated flow, from off-site areas. Developers must look not only at the new development but also at the surrounding topography to determine just how much off-site area drains to the new development site.

For drainage design of new developments, there are two considerations for accommodating off-site storm runoff. These are peak runoff rates from off-site areas and routing of off-site storm runoff relative to the new development.

1406.11.1 Peak Storm Runoff Rates

<u>Undeveloped Off-Site Areas</u>: Runoff rates from off-site areas that are undeveloped but have a comprehensive development plan (comp plan) in place will be calculated using imperviousness values in **Table 701** for the land use or surface characteristic specified by the comp plan. Runoff rates from off-site areas that are undeveloped and do not have a comp plan in place will be calculated using an imperviousness of 45 percent as specified in **Table 701** (or as specified by the local jurisdiction).

Developed Off-Site Areas: Runoff rates from off-site areas that are developed should be determined by calculating the actual imperviousness of the area using approved drainage reports, aerial photography, and field investigations. If this information is not available, imperviousness shall be determined based on general land use or surface characteristic as shown in **Table 701**.

The reduction of peak runoff rates from detention shall *only* be assumed for developments that are confirmed to have detention facilities and shall be subject to approval by the local jurisdiction. The discharge rates from these detention facilities shall be assumed to be those rates approved in the final drainage report for each off-site upstream area having detention facilities.

1406.11.2 Routing of Off-Site Storm Runoff

There are two methods by which off-site flows can be routed relative to the new development. These are routing around the new development and routing through the new development

Routing Around the New Development: Ideally the new development will be graded so off-site storm runoff is routed around the development based on peak rates determined in the above section. If this method is chosen, off-site flows must be routed to their historic path immediately downstream of the new development. Concentrated flows draining to the development should be maintained as concentrated flows downstream of the development and sheet flows should be maintained as sheet flow.

Routing Through the New Development: If the site cannot be graded to accommodate rerouting off-site flows, the development's detention pond must be sized to account for any off-site flow that comes into the new development and drains to the new detention pond. It is possible that routing off-site runoff through the new detention pond will significantly alter the volume and outlet requirements of the new detention pond.

The effects of routing storm runoff from off-site areas through the new detention pond shall be determined by hydrograph routing methods. A hydrograph of the off-site area, assuming ultimate development conditions without detention, shall be generated to achieve a total volume of runoff. The developed condition hydrograph shall then be truncated at the allowable release rate and extended in time such that the total runoff volume is the same. This "modified" hydrograph is a reasonable representation of the ultimate development runoff that is routed through a detention pond within the offsite development.

The modified off-site hydrograph is then added to the developed conditions hydrograph for the new development (assuming no on-site detention). This composite hydrograph shall then be routed through the new detention pond to verify that the release rates from the new detention pond meet allowable release rates. In this instance, the allowable release rate is the sum of the allowable release rates from *both* the off-site area and the new development. The design of the detention pond may require iteration on the detention volume and outlet configuration to achieve the required results.

### 1406.12 Retention

Retention will only be considered on a case-by-case basis with the following minimum requirements:

Retention basins must drain within 48 hours *of all storm events up to* the 100-year storm event, beginning either at the start of the storm or when runoff first reaches the basin. Total runoff volume shall be determined by calculating imperviousness then converting to this to a curve number (CN) following the procedures in Section 700. Using the CN, excess precipitation for the design storm is determined using procedures described in Section 704.1. The design storm shall be the 24-hour precipitation amount of 2.01 inches multiplied by a factor of 1.5.

The design of the retention basin must be supported by vertical hydraulic conductivity data for subsurface soil and/or rock obtained via an appropriate engineering test such as a tri-axial cell test, a permeameter test, or a vertical conductivity aquifer test conducted by qualified geotechnical engineer familiar with the Mesa County soils and geology. Test data shall then be used in an appropriate calculation to determine the required size of the retention basin and the time required for complete infiltration of the 100-year storm event. One of the following models shall be used to demonstrate the design meets all criteria:

- Hydrus-2D (U.S. Dept. of Agriculture, Agricultural Research Service)
- SEEP/W (GeoSlope International, Inc.)
- Equivalent model (upon approval of local jurisdiction)

When using vertical hydraulic conductivity data in these calculations, a saturated K-value shall only be used when it has been demonstrated that the infiltration media over the entire length of the infiltration path are saturated. If the infiltration path is characterized by unsaturated conditions, a partially saturated K-value shall be used for infiltration calculations

Measures that minimize sediment from entering and clogging the pond surface shall be implemented. These include pre-sediment basins and/or frequent sediment removal. Depth of storage shall be minimized and surface area shall be maximized for infiltration purposes. The wetted sides of the pond shall not be included in the area used to calculate infiltration releases.

An emergency overflow from the retention area shall be provided with a minimum capacity of the 100-year peak inflow rate. The overflow path shall be carefully selected and analyzed for capacity and downstream impact. The overflow shall be located at the maximum retention volume water surface. A minimum of 1.0 feet of freeboard shall be provided above the maximum retention volume water surface.

An operation maintenance plan shall be developed and be part of the development agreement that includes monitoring and reporting requirements and penalties for non-compliance. This plan may include pumping to meet the drain time requirements and should consider that there may be power outages during a major storm.

## 1407 HYDROLOGIC DESIGN METHODS AND CRITERIA

The hydrologic design of detention facilities is based on the type of facility (regional vs. local) and the method used to estimate the runoff (HEC-1 vs. the Rational Method). If HEC-1 is used, a full hydrograph is available for traditional storage routing. If the Rational method is used, only a peak flow rate is available. The following sections discuss the procedures for these two methods. Note that the procedures used to calculate storm runoff are described in Section 700 of this Manual.

### 1407.1 HEC-1 Method

The HEC-1 Method may be used to develop inflow hydrographs for hydrologic basins of any size, and may be used for both local and regional facilities. The inflow hydrographs shall be based on ultimate development conditions.

This program can calculate a hydrograph for any location in the hydrologic basin. The data input file must be structured so that the proposed detention basin site is a hydrograph-routing or hydrograph-combining point.

#### 1407.1.1 Detention Basin Outflow Limitations

The controlled outlet capacity has direct influence on the required size of the basin. The outflow limitation can be based on either the existing undeveloped peak flow from the hydrologic basin or on limitations in the capacity of the downstream conveyance system (based on a hydrologic analysis of local conditions). The outflow limitation for local facilities is stated in Section 403.7 of this Manual. The design maximum outlet capacity of a regional facility must be coordinated with the local jurisdiction.

#### 1407.1.2 Hydrologic Calculation Method

After the inflow hydrograph has been calculated and the outflow limits have been determined, the required storage volume can be estimated. Separate methods for calculating required storage are used depending on the method used to estimate the inflow hydrograph.

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 vs. elevation data for the proposed basin site;
d) Proposed storage vs. 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. 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.

## 1407.2 Rational Method

The minimum required volume shall be determined using the HEC-1 method or the following equations. These empirical equations were developed as part of the UDFCD hydrology research program and modified to reflect rainfall conditions in Mesa County. The equations are based on a computer modeling study and represent average conditions.

One of the most difficult aspects of storm drainage is obtaining consistent results between various methods for estimating detention requirements. These equations provide consistent and more effective approaches to the sizing of on-site detention ponds. The equations presented in this section may be used for hydrologic basins with a total area of less than 160 acres, per the Rational Method restrictions set forth in Section 705.3 of this Manual. The use of these equations in the design of regional detention facilities must be approved by Mesa County or the applicable local jurisdiction.

1407.2.1 Minimum Detention Volumes

$$V = KA \qquad (1402)$$

For the 100-year,

$$K_{100} = \left(1.78P - 0.002P^2 - 3.56\right)\left(X_{100} / 900\right) \qquad (1403)$$

For the 10-year,

$$K_{10} = \left(0.95P - 1.90\right)\left(X_{10} / 1000\right) \qquad (1404)$$

Where:
V = required volume for the 100- or 10-year storm (acre-feet)
P = Developed basin imperviousness (%)
A = Tributary area (Acres)
X = Mesa County and the other local jurisdictions adjustment factor per **Table 1401**

**Table 1401 Detention Volume Adjustment Factor**

| ULTIMATE DEVELOPMENT PERCENT IMPERVIOUSNESS | $X_{100}$ | $X_{10}$ |
|---|---|---|
| < 50% | 0.42 | 0.26 |
| ≥ 50% | 0.48 | 0.38 |

1407.2.2 Allowable Release Rates

The maximum release rates at the ponding depths corresponding to the 10- and 100-year volumes are as follows:

**Table 1402 Allowable Release Rates for Detention Ponds (cfs/acre)**

| CONTROL FREQUENCY | SOIL GROUP | | |
|---|---|---|---|
| | A | B | C/D |
| 10-year | 0.05 | 0.09 | 0.12 |
| 100-year | 0.25 | 0.43 | 0.50 |

The predominant soil group for the total basin area tributary to the detention pond shall be used for determining the allowable release rate. Information on the soils in Mesa County can be found in published SCS Soil Surveys.

### 1407.3 Compensating Detention Analysis

If any storm runoff will be discharged from the property without first being routed through a detention pond, on-site detention facilities are to be designed using the "compensating detention procedure". The total of all un-detained area shall not exceed 5% or 5,000 square feet, which ever is less.

Compensating detention is based on the following assumptions:

a. The 10- and 100-year peak discharge from the property from detained and un-detained area when added together will be no greater than allowable discharge. Therefore, the more un-detained release of storm runoff from the site, the less the detention pond is permitted to release, which requires proportionally larger detention volume.

b. Regardless of the method used, the volume of the detention pond must be adjusted to result in reduced discharge rates. For the HEC-1 method, the detention volume is determined using actual runoff hydrographs and storage routing based on the outlet configuration. For the Rational Method, the increased in volume is accomplished by simply computing the volume based on the *entire* property area, not just the area tributary to the detention pond. This is a reasonable assumption given the 5% area or 5,000- square feet size limitation on un-detained area.

The compensating detention procedure is given in six steps below. The following procedure applies to both the HEC-1 and Rational Method methods for determining detention volumes, except as specifically noted otherwise.

**Step 1)** If the un-detained area is less than 5% of the total project area or 5,000-square feet, whichever is less, continue. If not, then the un-detained area must either be reduced in size or the site layout revised to be incompliance with the SWMM.

**Step 2)** Determine the allowable release rate for the 10- and 100-year flood events based on the pre-project site conditions using the *entire* site area. See Table 1402.

**Step 3)** Determine the post-project runoff rates for the 10- and 100-year floods for the un-detained area only.

**Step 4)** Determine the *adjusted* allowable release rates by subtracting the runoff rates from the post project, un-detained area from the allowable release rates in Step 2.

**Step 5)** Determine minimum required 100-year and 10-year storage volumes for the area tributary to the detention pond. If using the Rational Method, the storage volume is determined using the equations in Section 1407.2 based on the *entire* project area, not just the area tributary to the detention pond.

**Step 6)** Determine the final outlet configuration that results in the *adjusted* allowable release rates at the computed detention volumes for the entire site.

### 1407.4 Over-Detention Analysis

Over-detention is defined as detaining developed conditions peak flows to the point that release rates are lower than pre-developed conditions. Over-detention is sometimes required to meet capacity limitations of downstream drainage facilities. Specific over-detention requirements are identified in basin-wide or watershed master plans or by the local jurisdiction within which the detention pond is proposed.

Over-detention requirements are unique for each basin or watershed and are often based on a detailed hydrologic investigation using hydrograph methods to generate peak flows and route the flows through the drainage system. Simplified detention volume and release rate methods are not appropriate, and over-detention volume and release rate requirements shall be determined using a hydrograph analysis in accordance with the SWMM requirements.

## 1408 HYDRAULIC DESIGN OF OUTLET WORKS

Hydraulic design data for sizing of detention facilities outlet works is as follows:

### 1408.1 Weir Flow

The general form of the equation for horizontal crested weirs is:

$$Q = CL(H)^{3/2} \qquad (1405)$$

Where:

- Q = discharge (cfs)
- C = weir coefficient (see **Table 1403**)
- L = horizontal length (feet)
- H = total energy head (feet)

Another common weir is the v-notch, whose equation is as follows:

$$Q = 2.5 \tan(\Theta/2)H5/2 \qquad (1406)$$

Where Θ = angle of the notch at the apex (degrees)

When designing or evaluating weir flow, the effects of submergence must be considered. A single check on submergence can be made by comparing the tailwater to the headwater depth. The example calculation for a weir design on **Figure 1403** illustrates the submergence check.

### 1408.2 Orifice Flow

The equation governing the orifice opening and plate is the orifice flow equation:

$$Q = C_d A\,(2gh)^{1/2} \qquad (1407)$$

Where:

$Q$ = Flow (cfs)
$C_d$ = Orifice coefficient
$A$ = Area ($ft^2$)
$g$ = Gravitational constant = 32.2 $ft/sec^2$
$h$ = Head on orifice measured from center of the opening (ft)

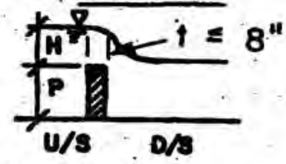
An orifice coefficient ($C_d$) value of 0.65 shall be used for sizing of squared edged orifice openings and plates.

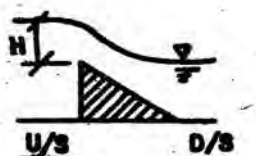
# STORMWATER MANAGEMENT MANUAL

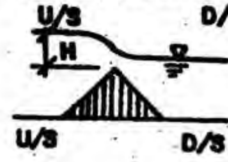
## WEIR FLOW COEFFICIENTS

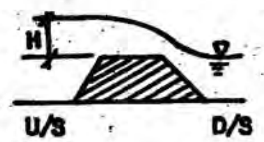
| SHAPE | COEFFICIENT | COMMENTS |
|---|---|---|
| Sharp Crested | - | |
| Projection Ratio (H/P = 0.4) | 3.4 | H< 1.0 |
| Projection Ratio (H/P = 2.0) | 4.0 | H> 1.0 |
| Broad Crested | - | |
| W/Sharp U/S Corner | 2.6 | Minimum Value |
| W/Rounded U/S Corner | 3.1 | Critical Depth |
| Triangular Section | - | |
| A) Vertical U/S Slope | - | |
| 1:1 D/S Slope | 3.8 | H > 0.7 |
| 4:1 D/S Slope | 3.2 | H > 0.7 |
| 10:1 D/S Slope | 2.9 | H > 0.7 |
| B) 1:1 U/S Slope | - | |
| 1:1 D/S Slope | 3.8 | H >0.5 |
| 3:1 D/S Slope | 3.5 | H >0.5 |
| Trapezoidal Section | | |
| 1:1 U/S Slope, 2:1 D/S Slope | 3.4 | H > 1.0 |
| 2:1 U/S Slope, 2:1 D/S Slope | 3.4 | H > 1.0 |
| Road Crossings | | |
| Gravel | 3.0 | H > 1.0 |
| Paved | 3.1 | H > 1.0 |

SCHEMATIC

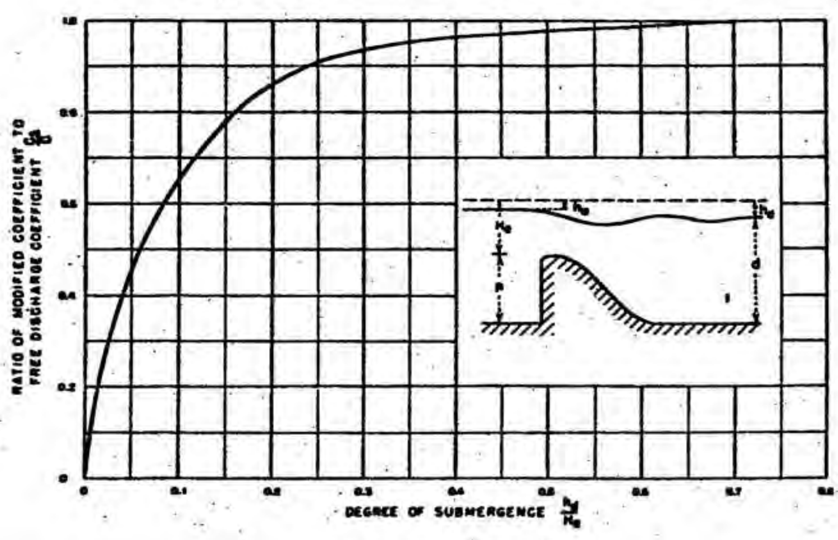



## ADJUSTMENTS FOR TAILWATER




| Revision | Date |
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| ORIGINAL ISSUE | 3/27/06 |
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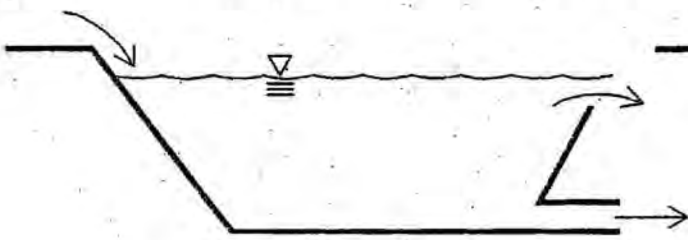
WRC ENGINEERING INC

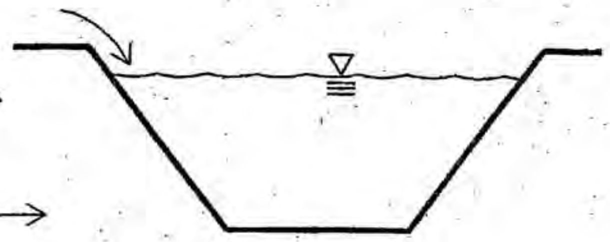
REFERENCE: KING & BRATER, HANDBOOK OF HYDRAULICS, 1963
DESIGN OF SMALL DAMS, USBR, 1977

**TABLE 1403**

## DETENTION AND RETENTION BASIN

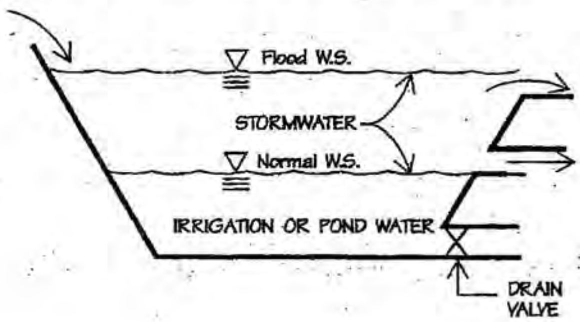


DETENTION BASIN

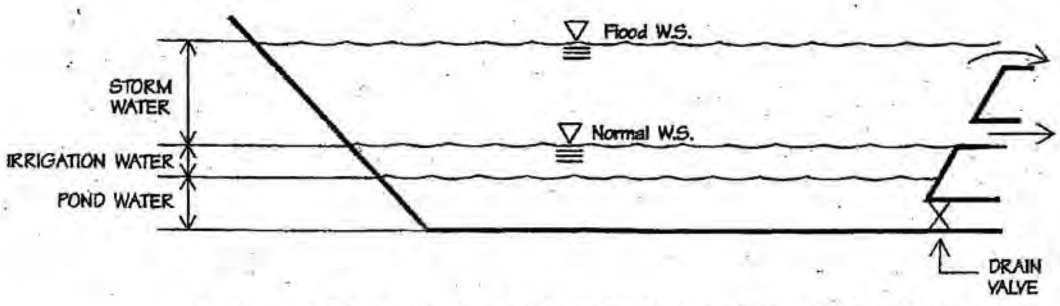


RETENTION BASIN

## WET POND COMBINATIONS




IRRIGATION OR POND AND STORMWATER DETENTION




POND, IRRIGATION, AND STORM WATER DETENTION POND

NOTE: Although the irrigation water will cycle between the pond normal surface and the maximum depth shown, all hydraulic analysis shall be based upon the full amount of irrigation water being present.

| Revision | Date |
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| ORIGINAL ISSUE | 3/27/06 |
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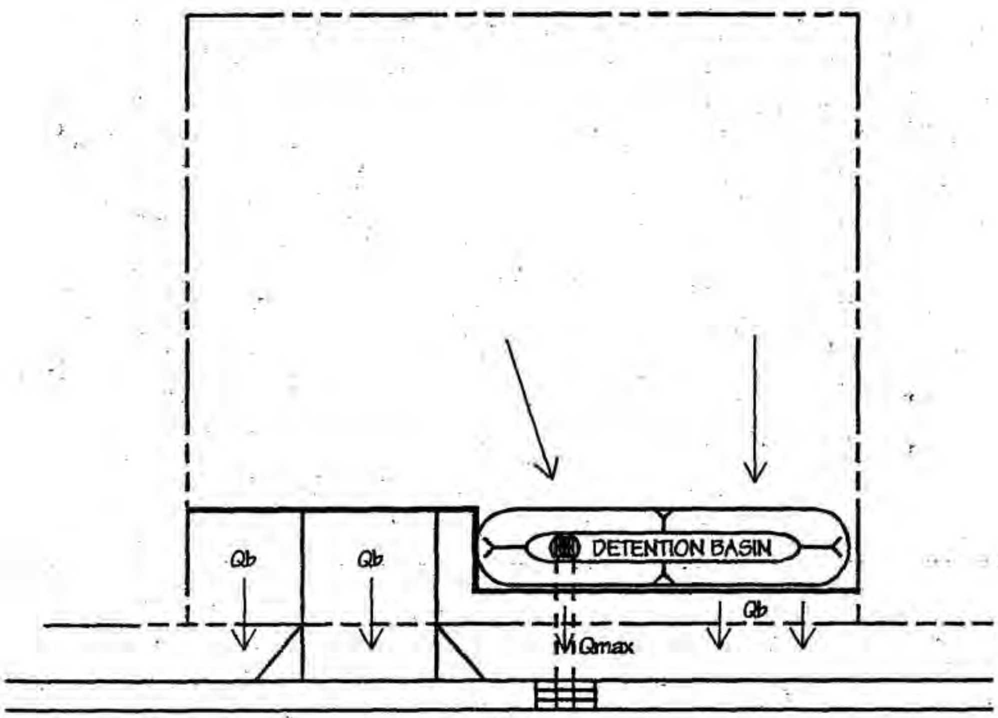
# STORMWATER MANAGEMENT MANUAL

## TOTAL SITE RUNOFF




Maximum release from detention pond Qmax ≤ Historic peak $Q_h$ minus direct bypass runoff Qb

$Qmax \le Q_h - Qb$

| Revision | Date |
|---|---|
| ORIGINAL ISSUE | 3/27/06 |

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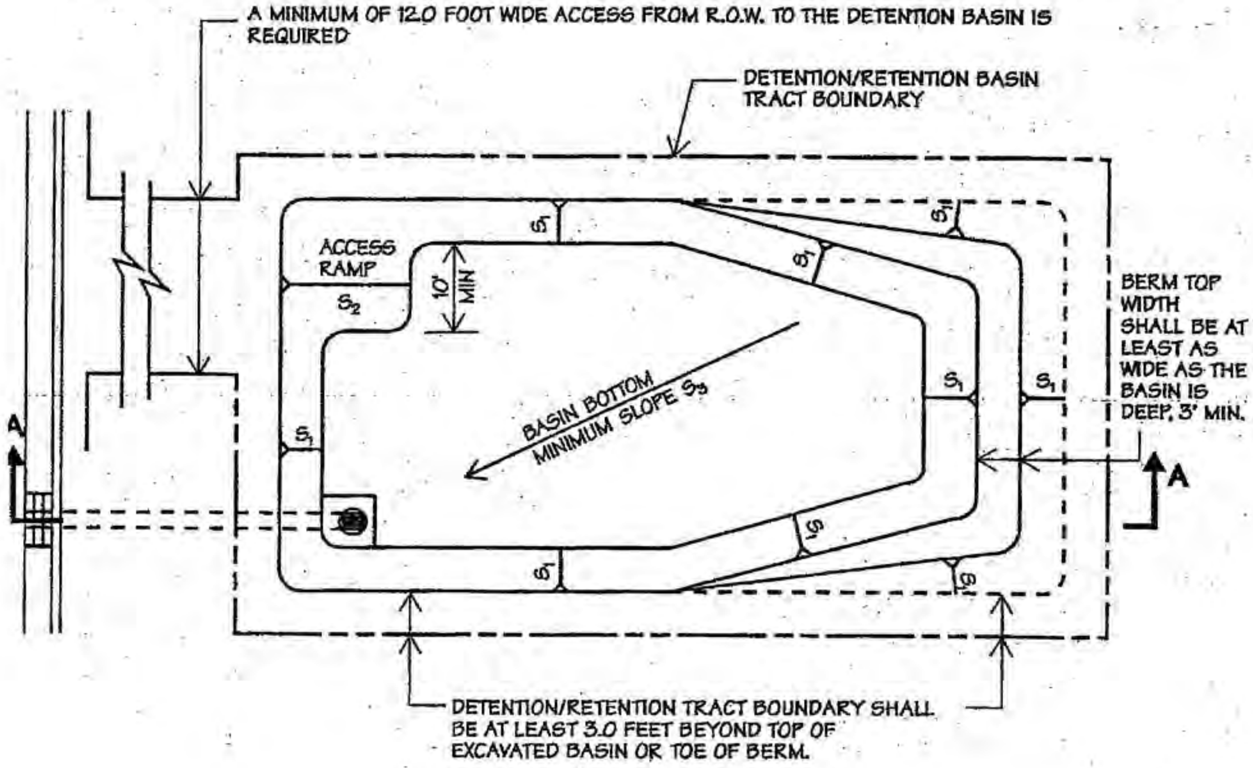
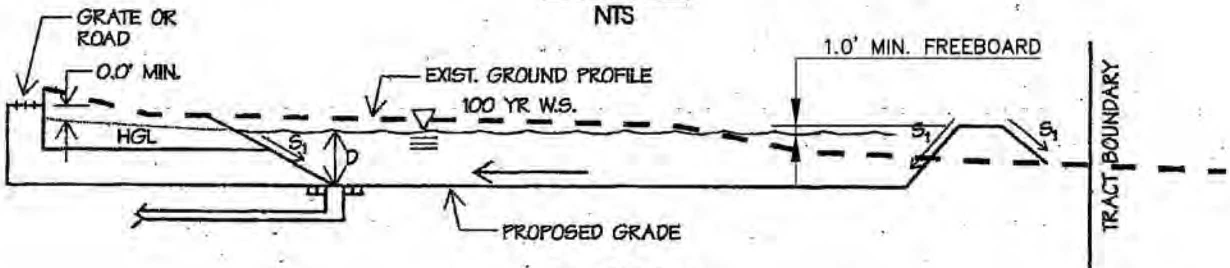
REFERENCE: MESA COUNTY SWMM DATED MAY 1996

**FIGURE 1402**

# DETENTION/RETENTION BASIN GEOMETRIC REQUIREMENTS




PLAN VIEW
NTS




SECTION A-A
NTS

STEEPEST $S_1$: 4H:1V FOR BASINS ON PUBLIC LANDS AND PARKS
3H:1V FOR SEEDED OR SODDED SLOPES
2H:1V FOR RIPRAP OR OTHER APPROVED SLOPE PROTECTION
VERTICAL WALLS WITH SAFETY RAILING LIMITED TO ONE SIDE ONLY WHERE APPROVED BY THE CITY ENGINEER OR COUNTY DEVELOPMENT ENGINEER.

STEEPEST $S_2$: 6H:1V FOR ACCESS RAMP, ALL SURFACES

MINIMUM $S_3$: 0.5% FOR CONCRETE CHANNEL
1.0% FOR ASPHALT (PARKING LOT)
2.0% FOR ALL OTHER SURFACES

MAXIMUM D: 4' RETENTION BASIN
8' WET OR DRY DETENTION FACILITY
>8' SPECIAL APPROVAL REQUIRED, BUT MAY BE ALLOWED FOR MULTIPLE USE PONDS OR FOR STEEP TERRAINS

MINIMUM D: 4' WET PONDS (SEE PAGE VIII-1)

| Revision | Date |
|---|---|
| ORIGINAL ISSUE | 3/27/06 |
| | |
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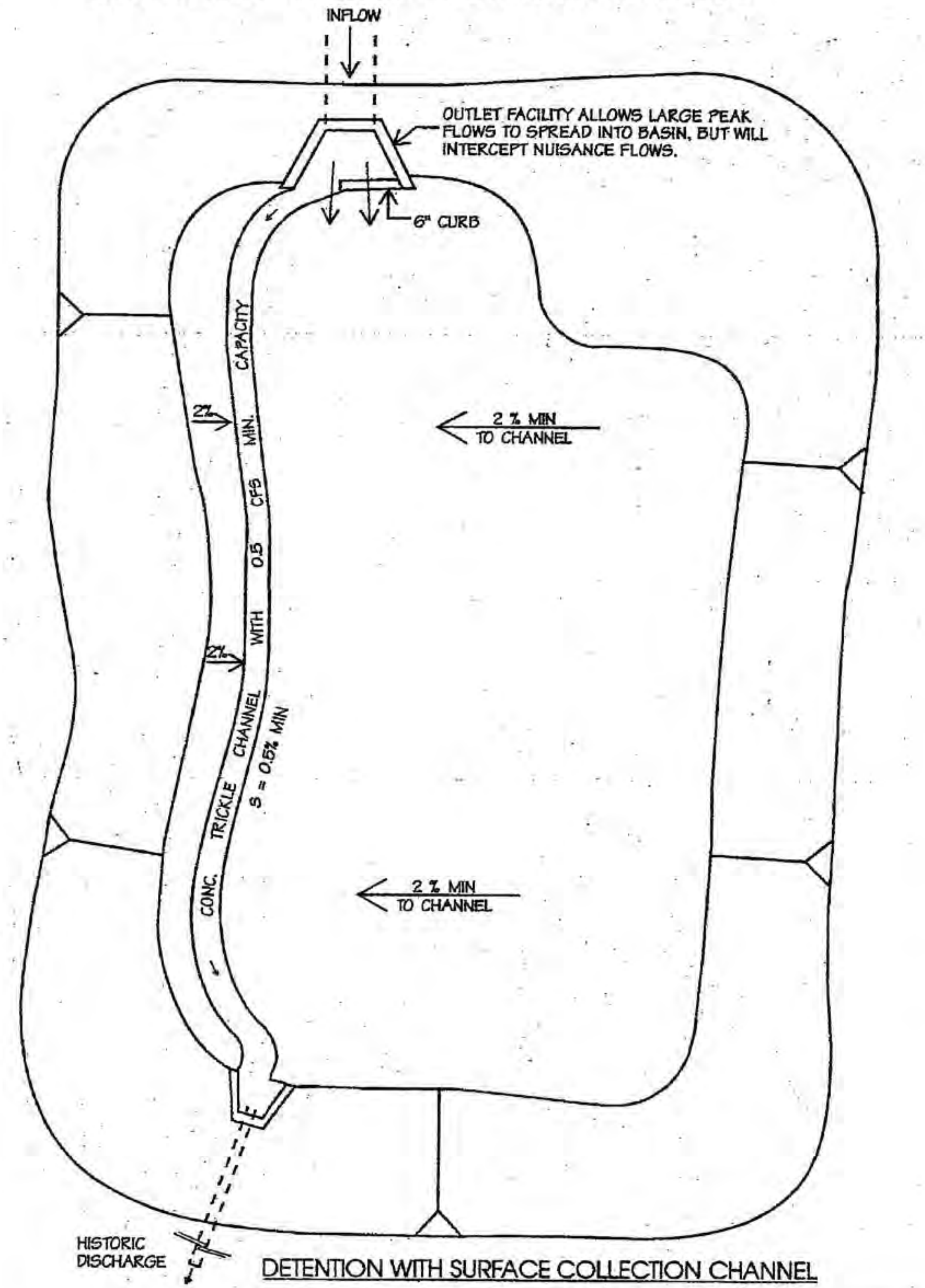
# DRY BASIN TRICKLE FLOW CONVEYANCE




DETENTION WITH SURFACE COLLECTION CHANNEL

| Revision | Date |
|---|---|
| ORIGINAL ISSUE | 3/27/06 |
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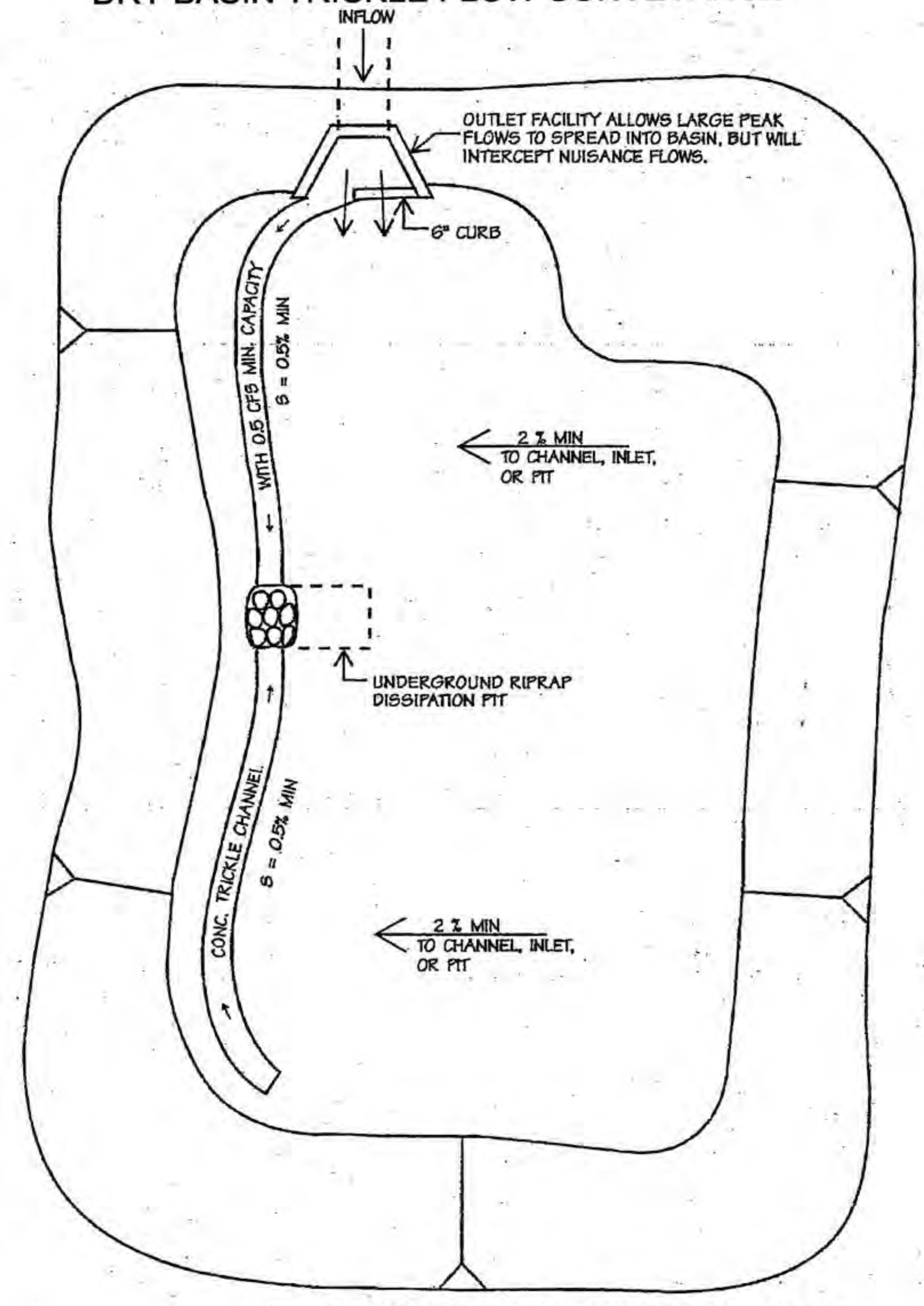
REFERENCE:

MESA COUNTY SWMM DATED MAY 1996

**FIGURE 1404**

# DRY BASIN TRICKLE FLOW CONVEYANCE




RETENTION WITH SURFACE COLLECTION CHANNEL
(THE SAME CONCEPT APPLIES TO AN UNDERGROUND SYSTEM)

| Revision | Date |
|---|---|
| ORIGINAL ISSUE | 3/27/06 |
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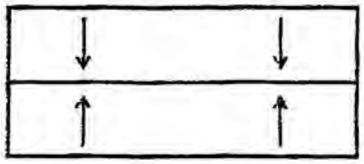
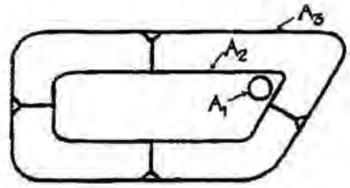
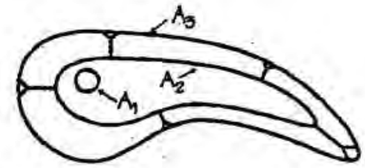
MESA COUNTY SWMM DATED MAY 1996

FIGURE 1405

# CALCULATING STORAGE VOLUME

| | | | | |
|---|---|---|---|---|
| BASIN SHAPE |  |  | |  |
| BASIN TYPE | VERTICAL WALLS AND/OR PRISMATIC BASINS | FAIRLY UNIFORM SHAPE AND SIDE SLOPES | OR | HIGHLY IRREGULAR SHAPE AND SIDE SLOPES |
| VOLUME CALCULATION METHOD | AVERAGE END AREA METHOD | CONIC METHOD | | |
| EQUATION | $V = \left(\frac{A_n + A_{n+1}}{2}\right)L$ | $V = \Sigma V_{n \text{ to } n+1}$<br>$V_{n \text{ to } n+1} = \left[A_n + A_{n+1} + (A_n A_{n+1})^5\right]\frac{h}{3}$ | | |

WHERE: $V$ = VOLUME (ft)
$A_n$ = HORIZONTAL AREA (ft²) AT ELEVATION "n"
$A_{n+1}$ = HORIZONTAL AREA (ft²) AT ELEVATION "n+1"
$h$ = VERTICAL HEIGHT (ft) BETWEEN ELEVATION "n" AND "n+1"
$V_{n \text{ to } n+1}$ = VOLUME BETWEEN ELEVATION "n" AND "n+1"
$L$ = LENGTH (ft) BETWEEN TWO ENDS

NOTE: THE ABOVE EQUATIONS MAY BE USED IN SUCCESSION FOR INCREMENTAL HEIGHTS WITHIN A BASIN. AN AREA SHOULD BE SELECTED AT ALL SIGNIFICANT CHANGES IN SHAPE OR SIDE SLOPE.

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| ORIGINAL ISSUE | 3/27/06 |
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REFERENCE: MESA COUNTY SWMM DATED MAY 1996

**FIGURE 1406**

# SECTION 1400

## DETENTION AND RETENTION

# SECTION 1400
## DETENTION AND RETENTION

## TABLE OF CONTENTS

## LIST OF TABLES



## LIST OF FIGURES

# SECTION 1400
# DETENTION AND RETENTION

## 1401 INTRODUCTION

The main purpose of a detention basin is to store runoff and reduce peak discharge by allowing flow to be discharged later at a controlled rate, and within a reasonable time. 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 predevelopment conditions (local facilities only). Detention has been shown to be very beneficial in controlling flood peaks in an urbanized area. Use of detention includes local detention such as in-channel or within a parking lot, and regional detention, such as a large recreational lake or reservoir and off-line detention facilities. Regional and local detention facilities are more fully discussed below. Policy regarding detention and extended detention (retention) is found in Sections 403.7 and 403.8 of this Manual.

It is important that any extended detention (retention) be supported by valid water rights that allows for retention without impacting any vested water rights on the stream system.

## 1402 DETENTION VERSUS RETENTION

Stormwater storage reservoir types are numerous, but they essentially fit into one of two categories: detention or retention. The words "pond" and "basin" are used interchangeably when used in connection with both detention and retention reservoirs. A detention basin or pond "detains" water temporarily, releasing water through a pipe or channel by means of a weir, orifice, or pump. Because of its ability to release flow during inflow, the overall volume of storage required for a given storm event is reduced. Another advantage of the detention basin is the positive means of outflow, resulting in fewer problems with long-term ponding. A retention basin or pond "retains" water without any initial release during inflow. Once the storm event is over, pond drainage may occur due to evaporation and percolation into the soil. In some instances, retention basins may also involve a gated pipe or pump which is closed or inoperative during the storm event. However, if a gated pipe or pump is an available or desirable option, it would normally be advantageous to release water during stormwater inflow, which would change the basin from a retention basin to a detention basin. The difference in detention and retention basins is depicted in **Figure 1401**.

The words "pond" and "basin" may be used to refer to reservoirs that remain dry outside of storm events, or store water for other purposes, e.g., irrigation, recreation, aesthetics, etc., in addition to receiving stormwater during storm events. Words "wet" and "dry" are used as prefixes to describe the condition of ponds and basins. However, a wet pond and dry basin each have a specific meaning.

Wet ponds may be desirable compared to dry basins in some circumstances. It may be that ample storage volume exists to provide an aesthetic, recreational, or irrigation pond below the required reservoir volume. Use of irrigation storage facilities for stormwater detention purposes must be reviewed by the appropriate jurisdiction(s) on a case-by-case basis. It is required that the stormwater reservoir volume provided must be in addition to the maximum expected base storage (irrigation or wet pond) volume. This is depicted in **Figure 1401**.

## 1403 LOCAL AND REGIONAL DETENTION FACILITIES

### 1403.1 Local Detention Facilities

Local detention facilities are usually designed by and financed by developers or local property owners. 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.

1403.1.1 Local Minor Facilities

Local minor detention facilities are defined as serving a single development with a hydrologic basin smaller than or equal to 20 acres and are designed to mitigate the impact of increased runoff due to development. The outlet capacity is generally based on predevelopment hydrology, and the detention structures are generally small (0.01 to 1 acre-foot). Detention storage volume may be provided as small landscaped or turfed basins, parking lot storage, or a suitable combination of all.

1403.1.2 Local Major Facilities

Local major detention facilities are defined as serving more than a single development or serving hydrologic basins greater than 20 acres in size. These facilities may serve dual functions. They typically reduce existing flooding problems to allow more development and/or control increased runoff caused by additional 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 larger size, these basins are designed much the same as regional detention facilities.

### 1403.2 Regional Detention Facilities

Regional detention facilities are those identified in the current Grand Valley Stormwater Master Management plan or as designated by local jurisdiction. Generally, these facilities control flow on major drainageways, are large in size, and are owned and maintained by public agencies. The purpose of these facilities is to significantly reduce downstream flows in order to maximize the capacity of existing systems and maintain flows at or below historic rates.

## 1404 DRAINAGE FEE IN LIEU OF DETENTION

The developer may be given the option of paying a drainage fee in lieu of providing adequate detention facilities. Such an option may be considered only if the developer completes an engineering analysis to show that all downstream facilities have adequate capacity to handle the un-detained flows from the proposed development. If such option is allowed, this does not waive the requirements for drainage planning submittal requirements as outlined in Section 300 of this Manual.

### 1404.1 Drainage Fee Basis

For stormwater management purposes, charges assessed to existing and new development to offset cost of providing drainage facilities and services are typically categorized as an impact fee or a utility fee.

An impact fee is based on the cost of upgrading drainage services within a watershed, shared among all new developments within the watershed. The basis for "sharing" costs can be as simple as the size of the water tap and as complex as calculating imperviousness of the property then making adjustments for various best management practices, such as detention. An impact fee is a one-time charge against new development.

A utility fee is based on the cost of upgrading and maintaining drainage services within a watershed to meet current standards or level-of-service. A utility fee is often assessed to all properties, developed or not and often is limited to providing maintenance of the system and minor capital improvements. Utility fees are normally charged on a monthly basis.

Since the local jurisdiction charge is a fee-in-lieu of detention for new development, it is considered a limited impact fee. Reasonableness tests applied to impact fees include:

- The system impact fee shall be related to the amount of drainage improvements needed to meet local standards.
- The impact fee shall only cover capital and related (i.e. engineering, construction administration, etc.) costs for improvements.

### 1404.2 Drainage Fee Calculation

The formula used by local jurisdictions to calculate drainage fee is of the form:

$$\text{Drainage Fee (\$)} = B(C_D - C_H)A^{0.7} \qquad (1401)$$

Where:

$B$ = Fee Constant (established annually by City Council of local jurisdiction)

$C_D$ = 100-Yr. runoff coefficient (expressed as a decimal) based on developed land use conditions

$C_H$ = 100-Yr. runoff coefficient (expressed as a decimal) based on pre-developed land use conditions

$A$ = Area of Development (acres)

Assigning B = \$14,000, A = 1 acre, $C_D$ = 0.60 and $C_H$ = 0.51, based on SCS Type C soils, and an imperviousness of 2% pre-developed and 50% developed, the fee-in-lieu of detention would be \$1,260.

The equation used in Mesa County and the other local jurisdictions takes into account key factors for a fee-in-lieu of detention, such as:

- The fee is based on area.
- The fee takes into account imperviousness by using the Rational Method C values.
- The fee takes into account the difference between pre- and post-development runoff rates, which is the main function of onsite detention.

The constant B needs to closely reflect the actual cost of providing detention, and must be changed with time as the need arises. Local jurisdictions will review this figure annually and make appropriate recommendations for its adjustment.

## 1405 DETENTION FOR MANAGEMENT OF STORMWATER QUALITY

In addition to the primary purpose of detention and retention facilities to reduce peak flows, they can be adapted to enhance stormwater quality.

The outlet for a detention basin can be designed to allow for a slower rate of release and thus provide time for particulate pollutants to settle out of the stormwater. Such basins are called extended detention basins (EDB). It is recommended that an EDB outlet be designed to completely drain a full basin in 48 hours, allowing for the removal of a significant portion of insoluble pollutants with proper operation and maintenance.

A retention pond (RP) is a sedimentation facility designed to have a permanent water pool. This pool of water is mixed with stormwater during storm runoff events from frequently occurring storms and allows for sedimentation and thus water quality enhancement to occur.

For frequently occurring storms, both EDBs and RPs capture the total runoff as a surcharge. However, in the case of RPs, the stormwater is allowed to mix with the permanent pool water as it rises above the permanent pool level. All surcharged water above the permanent pool level is to be released over 48 hours. Details related to the design of EDBs and RPs can be found in Section 1500 *Construction Site Stormwater Runoff Control* and 1600 *Post-Construction Stormwater Management* of this Manual.

For the 100-year detention, the water quality volume is considered to be a part of the volume required for detention purposes.

## 1406 GENERAL DETENTION AND RETENTION CRITERIA

### 1406.1 Design Storm Criteria

Peak runoff from a site may not be increased in the 10- and 100-year storms due to development. The site runoff may be a composite of detention/retention basin release/overflow and direct runoff, both of which must be considered. If direct runoff is allowed from the site, the sum of the direct runoff plus the release from the detention basin must not exceed the historic rate. This is depicted in **Figure 1402** and discussed further in Section 1407.3.

### 1406.2 Geometric Requirements

For proper function and safety considerations, geometric requirements shall be as shown on **Figure 1403**.

### 1406.3 Dry Basin Bottom Drainage

Most drainage conveyance systems are designed to divert even minor nuisance flows to stormwater storage facilities. For dry basins, this can present an aesthetic and maintenance problem. Conveyance facilities to a dry basin shall be capable of transporting flow to the outlet facility rather than causing a soggy bog condition that cannot be properly maintained. Facilities conveying trickle or nuisance flows, such as from irrigation sprinklers, shall be adequate to convey at least 0.5 cfs. Reference is made to **Figures 1404 and 1405**.

The outlet facility for a retention basin would be a dry well or riprap filled dissipation pit. For a detention basin, the nuisance flows shall be conveyed to the basin outlet.

### 1406.4 Accessibility and Maintenance

All reservoirs or ponds which serve more than a single lot or site must be provided with a detention/retention tract dedicated for maintenance access. Maintenance of required volume and inflow and outflow works is necessary for the facility to function as required. Dedicated rights-of-way and/or easements must be provided as required in Section 402.2 of this Manual.

### 1406.5 Calculating Storage Volume Available

Storage volume shall be calculated by the methods shown prescribed in **Figure 1406**.

### 1406.6 Groundcover and Landscaping

After final grading, the slopes and bottom of each detention and retention basin shall be protected from erosion by seeding and mulching, sodding or other approved groundcover and shall be in accordance with jurisdictional specifications.

The planting of trees and shrubs on the slopes of stormwater basins is also encouraged. Temporary and/or permanent irrigation systems shall be provided as required for the type of groundcover and landscape installed and approved.

### 1406.7 Freeboard Requirements

The minimum required freeboard for open space detention/retention facilities is 1.0 feet above the computed 100-year water surface elevation.

### 1406.8 State Engineer's Office

Any dam constructed for the purpose of storing water, with a surface area, volume, or dam height as specified in Colorado Revised Statues 37-87-105 as amended, shall require the approval of the plans by the State Engineer's Office. All detention and retention storage areas shall be designed and constructed in accordance with these criteria. Those facilities subject to state statutes shall be designed and constructed in accordance with the criteria of the state.

### 1406.9 Embankment Protection

Whenever a detention pond uses an embankment to contain water, the embankment shall be protected from catastrophic failure due to overtopping. Overtopping can occur when the pond outlets become obstructed or when a larger than 100-year storm occurs. Failure protection for the embankment may be provided in the form of a buried heavy riprap layer on the entire downstream face of the embankment or a separate emergency spillway having a minimum capacity of twice the maximum release rate for the 100-year storm. Structures shall not be permitted in the path of the emergency spillway or overflow. The invert of the emergency spillway shall be set equal to or above the 100-year water surface elevation.

### 1406.10 Inlet and Outlet Design

The outlet from the detention basin shall consist of a short (maximum 25 feet) length(s) of CAP or RCP with an 18-inch minimum diameter. Multiple pipe outlets may be required to control different design frequencies. The invert of the lowest outlet pipe shall be set at the lowest point in the detention pond or at the top of the minimum pool, if present. The outlet pipe(s) shall discharge into a standard manhole or into a drainageway with proper erosion protection for the proposed structure. If an orifice plate is required to control the release rates, the plate(s) shall be hinged to open into the detention pipes to facilitate back flushing of the outlet pipe(s).

Inlet to the detention pipes can be by way of surface inlets and/or by a local private storm sewer system.

### 1406.11 Off-Site Flows

It is rarely the case that a new development comprises an entire drainage basin. Instead, developments are most often located within a larger drainage basin. This means that unless the new development is located at the very upstream edge of a drainage basin, it will most likely receive sheet flow, if not concentrated flow, from off-site areas. Developers must look not only at the new development but also at the surrounding topography to determine just how much off-site area drains to the new development site.

For drainage design of new developments, there are two considerations for accommodating off-site storm runoff. These are peak runoff rates from off-site areas and routing of off-site storm runoff relative to the new development.

1406.11.1 Peak Storm Runoff Rates

<u>Undeveloped Off-Site Areas</u>: Runoff rates from off-site areas that are undeveloped but have a comprehensive development plan (comp plan) in place will be calculated using imperviousness values in **Table 701** for the land use or surface characteristic specified by the comp plan. Runoff rates from off-site areas that are undeveloped and do not have a comp plan in place will be calculated using an imperviousness of 45 percent as specified in **Table 701** (or as specified by the local jurisdiction).

Developed Off-Site Areas: Runoff rates from off-site areas that are developed should be determined by calculating the actual imperviousness of the area using approved drainage reports, aerial photography, and field investigations. If this information is not available, imperviousness shall be determined based on general land use or surface characteristic as shown in **Table 701**.

The reduction of peak runoff rates from detention shall *only* be assumed for developments that are confirmed to have detention facilities and shall be subject to approval by the local jurisdiction. The discharge rates from these detention facilities shall be assumed to be those rates approved in the final drainage report for each off-site upstream area having detention facilities.

1406.11.2 Routing of Off-Site Storm Runoff

There are two methods by which off-site flows can be routed relative to the new development. These are routing around the new development and routing through the new development

Routing Around the New Development: Ideally the new development will be graded so off-site storm runoff is routed around the development based on peak rates determined in the above section. If this method is chosen, off-site flows must be routed to their historic path immediately downstream of the new development. Concentrated flows draining to the development should be maintained as concentrated flows downstream of the development and sheet flows should be maintained as sheet flow.

Routing Through the New Development: If the site cannot be graded to accommodate rerouting off-site flows, the development's detention pond must be sized to account for any off-site flow that comes into the new development and drains to the new detention pond. It is possible that routing off-site runoff through the new detention pond will significantly alter the volume and outlet requirements of the new detention pond.

The effects of routing storm runoff from off-site areas through the new detention pond shall be determined by hydrograph routing methods. A hydrograph of the off-site area, assuming ultimate development conditions without detention, shall be generated to achieve a total volume of runoff. The developed condition hydrograph shall then be truncated at the allowable release rate and extended in time such that the total runoff volume is the same. This "modified" hydrograph is a reasonable representation of the ultimate development runoff that is routed through a detention pond within the offsite development.

The modified off-site hydrograph is then added to the developed conditions hydrograph for the new development (assuming no on-site detention). This composite hydrograph shall then be routed through the new detention pond to verify that the release rates from the new detention pond meet allowable release rates. In this instance, the allowable release rate is the sum of the allowable release rates from *both* the off-site area and the new development. The design of the detention pond may require iteration on the detention volume and outlet configuration to achieve the required results.

### 1406.12 Retention

Retention will only be considered on a case-by-case basis with the following minimum requirements:

Retention basins must drain within 48 hours *of all storm events up to* the 100-year storm event, beginning either at the start of the storm or when runoff first reaches the basin. Total runoff volume shall be determined by calculating imperviousness then converting to this to a curve number (CN) following the procedures in Section 700. Using the CN, excess precipitation for the design storm is determined using procedures described in Section 704.1. The design storm shall be the 24-hour precipitation amount of 2.01 inches multiplied by a factor of 1.5.

The design of the retention basin must be supported by vertical hydraulic conductivity data for subsurface soil and/or rock obtained via an appropriate engineering test such as a tri-axial cell test, a permeameter test, or a vertical conductivity aquifer test conducted by qualified geotechnical engineer familiar with the Mesa County soils and geology. Test data shall then be used in an appropriate calculation to determine the required size of the retention basin and the time required for complete infiltration of the 100-year storm event. One of the following models shall be used to demonstrate the design meets all criteria:

- Hydrus-2D (U.S. Dept. of Agriculture, Agricultural Research Service)
- SEEP/W (GeoSlope International, Inc.)
- Equivalent model (upon approval of local jurisdiction)

When using vertical hydraulic conductivity data in these calculations, a saturated K-value shall only be used when it has been demonstrated that the infiltration media over the entire length of the infiltration path are saturated. If the infiltration path is characterized by unsaturated conditions, a partially saturated K-value shall be used for infiltration calculations

Measures that minimize sediment from entering and clogging the pond surface shall be implemented. These include pre-sediment basins and/or frequent sediment removal. Depth of storage shall be minimized and surface area shall be maximized for infiltration purposes. The wetted sides of the pond shall not be included in the area used to calculate infiltration releases.

An emergency overflow from the retention area shall be provided with a minimum capacity of the 100-year peak inflow rate. The overflow path shall be carefully selected and analyzed for capacity and downstream impact. The overflow shall be located at the maximum retention volume water surface. A minimum of 1.0 feet of freeboard shall be provided above the maximum retention volume water surface.

An operation maintenance plan shall be developed and be part of the development agreement that includes monitoring and reporting requirements and penalties for non-compliance. This plan may include pumping to meet the drain time requirements and should consider that there may be power outages during a major storm.

## 1407 HYDROLOGIC DESIGN METHODS AND CRITERIA

The hydrologic design of detention facilities is based on the type of facility (regional vs. local) and the method used to estimate the runoff (HEC-1 vs. the Rational Method). If HEC-1 is used, a full hydrograph is available for traditional storage routing. If the Rational method is used, only a peak flow rate is available. The following sections discuss the procedures for these two methods. Note that the procedures used to calculate storm runoff are described in Section 700 of this Manual.

### 1407.1 HEC-1 Method

The HEC-1 Method may be used to develop inflow hydrographs for hydrologic basins of any size, and may be used for both local and regional facilities. The inflow hydrographs shall be based on ultimate development conditions.

This program can calculate a hydrograph for any location in the hydrologic basin. The data input file must be structured so that the proposed detention basin site is a hydrograph-routing or hydrograph-combining point.

1407.1.1 Detention Basin Outflow Limitations

The controlled outlet capacity has direct influence on the required size of the basin. The outflow limitation can be based on either the existing undeveloped peak flow from the hydrologic basin or on limitations in the capacity of the downstream conveyance system (based on a hydrologic analysis of local conditions). The outflow limitation for local facilities is stated in Section 403.7 of this Manual. The design maximum outlet capacity of a regional facility must be coordinated with the local jurisdiction.

1407.1.2 Hydrologic Calculation Method

After the inflow hydrograph has been calculated and the outflow limits have been determined, the required storage volume can be estimated. Separate methods for calculating required storage are used depending on the method used to estimate the inflow hydrograph.

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 vs. elevation data for the proposed basin site;
d) Proposed storage vs. 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. 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.

### 1407.2 Rational Method

The minimum required volume shall be determined using the HEC-1 method or the following equations. These empirical equations were developed as part of the UDFCD hydrology research program and modified to reflect rainfall conditions in Mesa County. The equations are based on a computer modeling study and represent average conditions.

One of the most difficult aspects of storm drainage is obtaining consistent results between various methods for estimating detention requirements. These equations provide consistent and more effective approaches to the sizing of on-site detention ponds. The equations presented in this section may be used for hydrologic basins with a total area of less than 160 acres, per the Rational Method restrictions set forth in Section 705.3 of this Manual. The use of these equations in the design of regional detention facilities must be approved by Mesa County or the applicable local jurisdiction.

1407.2.1 Minimum Detention Volumes

$$V = KA \tag{1402}$$

For the 100-year,

$$K_{100} = \left(1.78P - 0.002P^2 - 3.56\right)\left(X_{100} / 900\right) \tag{1403}$$

For the 10-year,

$$K_{10} = \left(0.95P - 1.90\right)\left(X_{10} / 1000\right) \tag{1404}$$

Where:
V = required volume for the 100- or 10-year storm (acre-feet)
P = Developed basin imperviousness (%)
A = Tributary area (Acres)
X = Mesa County and the other local jurisdictions adjustment factor per **Table 1401**

**Table 1401 Detention Volume Adjustment Factor**

| ULTIMATE DEVELOPMENT PERCENT IMPERVIOUSNESS | $X_{100}$ | $X_{10}$ |
|---|---|---|
| < 50% | 0.42 | 0.26 |
| ≥ 50% | 0.48 | 0.38 |

1407.2.2 Allowable Release Rates

The maximum release rates at the ponding depths corresponding to the 10- and 100-year volumes are as follows:

**Table 1402 Allowable Release Rates for Detention Ponds (cfs/acre)**

| CONTROL FREQUENCY | SOIL GROUP | | |
|---|---|---|---|
| | A | B | C/D |
| 10-year | 0.05 | 0.09 | 0.12 |
| 100-year | 0.25 | 0.43 | 0.50 |

The predominant soil group for the total basin area tributary to the detention pond shall be used for determining the allowable release rate. Information on the soils in Mesa County can be found in published SCS Soil Surveys.

### 1407.3 Compensating Detention Analysis

If any storm runoff will be discharged from the property without first being routed through a detention pond, on-site detention facilities are to be designed using the "compensating detention procedure". The total of all un-detained area shall not exceed 5% or 5,000 square feet, which ever is less.

Compensating detention is based on the following assumptions:

a. The 10- and 100-year peak discharge from the property from detained and un-detained area when added together will be no greater than allowable discharge. Therefore, the more un-detained release of storm runoff from the site, the less the detention pond is permitted to release, which requires proportionally larger detention volume.

b. Regardless of the method used, the volume of the detention pond must be adjusted to result in reduced discharge rates. For the HEC-1 method, the detention volume is determined using actual runoff hydrographs and storage routing based on the outlet configuration. For the Rational Method, the increased in volume is accomplished by simply computing the volume based on the *entire* property area, not just the area tributary to the detention pond. This is a reasonable assumption given the 5% area or 5,000- square feet size limitation on un-detained area.

The compensating detention procedure is given in six steps below. The following procedure applies to both the HEC-1 and Rational Method methods for determining detention volumes, except as specifically noted otherwise.

**Step 1)** If the un-detained area is less than 5% of the total project area or 5,000-square feet, whichever is less, continue. If not, then the un-detained area must either be reduced in size or the site layout revised to be incompliance with the SWMM.

**Step 2)** Determine the allowable release rate for the 10- and 100-year flood events based on the pre-project site conditions using the *entire* site area. See Table 1402.

**Step 3)** Determine the post-project runoff rates for the 10- and 100-year floods for the un-detained area only.

**Step 4)** Determine the *adjusted* allowable release rates by subtracting the runoff rates from the post project, un-detained area from the allowable release rates in Step 2.

**Step 5)** Determine minimum required 100-year and 10-year storage volumes for the area tributary to the detention pond. If using the Rational Method, the storage volume is determined using the equations in Section 1407.2 based on the *entire* project area, not just the area tributary to the detention pond.

**Step 6)** Determine the final outlet configuration that results in the *adjusted* allowable release rates at the computed detention volumes for the entire site.

### 1407.4 Over-Detention Analysis

Over-detention is defined as detaining developed conditions peak flows to the point that release rates are lower than pre-developed conditions. Over-detention is sometimes required to meet capacity limitations of downstream drainage facilities. Specific over-detention requirements are identified in basin-wide or watershed master plans or by the local jurisdiction within which the detention pond is proposed.

Over-detention requirements are unique for each basin or watershed and are often based on a detailed hydrologic investigation using hydrograph methods to generate peak flows and route the flows through the drainage system. Simplified detention volume and release rate methods are not appropriate, and over-detention volume and release rate requirements shall be determined using a hydrograph analysis in accordance with the SWMM requirements.

## 1408 HYDRAULIC DESIGN OF OUTLET WORKS

Hydraulic design data for sizing of detention facilities outlet works is as follows:

### 1408.1 Weir Flow

The general form of the equation for horizontal crested weirs is:

$$Q = CL(H)^{3/2} \qquad (1405)$$

Where:
- Q = discharge (cfs)
- C = weir coefficient (see **Table 1403**)
- L = horizontal length (feet)
- H = total energy head (feet)

Another common weir is the v-notch, whose equation is as follows:

$$Q = 2.5 \tan(\Theta/2)H5/2 \qquad (1406)$$

Where Θ = angle of the notch at the apex (degrees)

When designing or evaluating weir flow, the effects of submergence must be considered. A single check on submergence can be made by comparing the tailwater to the headwater depth. The example calculation for a weir design on **Figure 1403** illustrates the submergence check.

### 1408.2 Orifice Flow

The equation governing the orifice opening and plate is the orifice flow equation:

$$Q = C_d A\,(2gh)^{1/2} \qquad (1407)$$

Where:

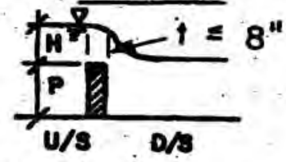
- $Q$ = Flow (cfs)
- $C_d$ = Orifice coefficient
- $A$ = Area ($ft^2$)
- $g$ = Gravitational constant = 32.2 $ft/sec^2$
- $h$ = Head on orifice measured from center of the opening (ft)

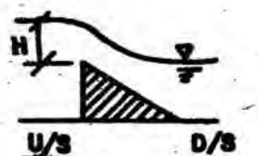
An orifice coefficient ($C_d$) value of 0.65 shall be used for sizing of squared edged orifice openings and plates.

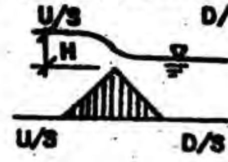
# WEIR FLOW COEFFICIENTS

| SHAPE | COEFFICIENT | COMMENTS |
|---|---|---|
| **Sharp Crested** | - | |
| Projection Ratio (H/P = 0.4) | 3.4 | H< 1.0 |
| Projection Ratio (H/P = 2.0) | 4.0 | H> 1.0 |
| **Broad Crested** | - | |
| W/Sharp U/S Corner | 2.6 | Minimum Value |
| W/Rounded U/S Corner | 3.1 | Critical Depth |
| **Triangular Section** | - | |
| A) Vertical U/S Slope | - | |
| 1:1 D/S Slope | 3.8 | H > 0.7 |
| 4:1 D/S Slope | 3.2 | H > 0.7 |
| 10:1 D/S Slope | 2.9 | H > 0.7 |
| B) 1:1 U/S Slope | - | |
| 1:1 D/S Slope | 3.8 | H >0.5 |
| 3:1 D/S Slope | 3.5 | H >0.5 |
| **Trapezoidal Section** | | |
| 1:1 U/S Slope, 2:1 D/S Slope | 3.4 | H > 1.0 |
| 2:1 U/S Slope, 2:1 D/S Slope | 3.4 | H > 1.0 |
| **Road Crossings** | | |
| Gravel | 3.0 | H > 1.0 |
| Paved | 3.1 | H > 1.0 |

SCHEMATIC

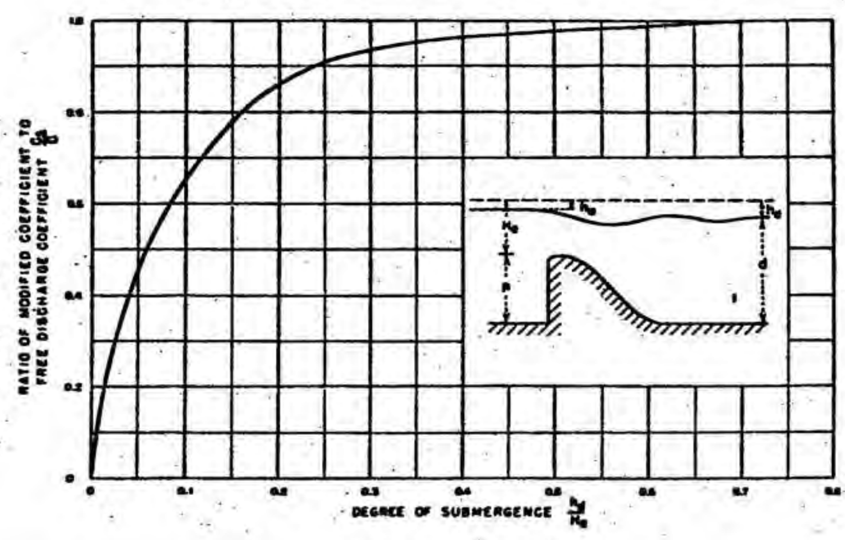




## ADJUSTMENTS FOR TAILWATER

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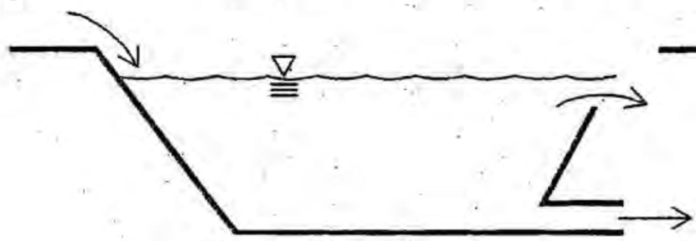
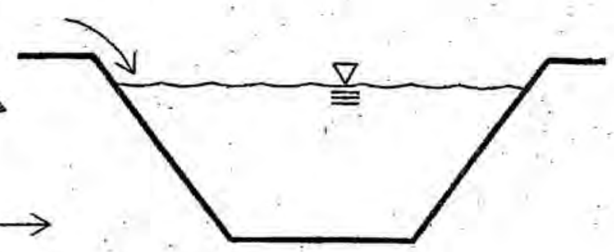
# DETENTION AND RETENTION BASIN

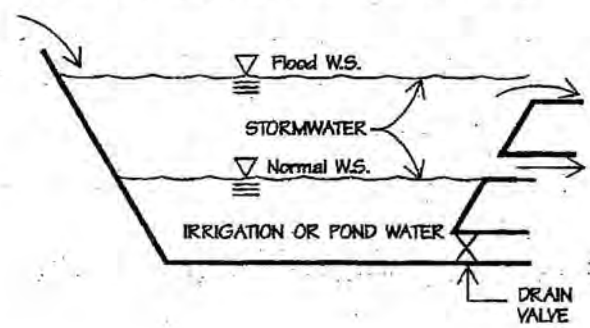


DETENTION BASIN

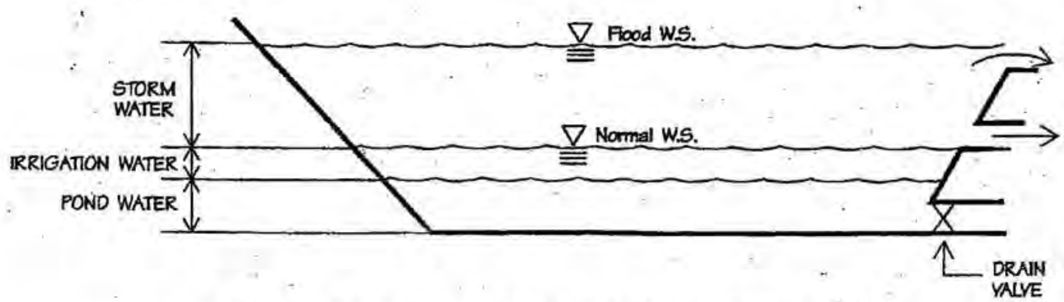


RETENTION BASIN

# WET POND COMBINATIONS




IRRIGATION OR POND AND STORMWATER DETENTION




POND, IRRIGATION, AND STORM WATER DETENTION POND

NOTE: Although the irrigation water will cycle between the pond normal surface and the maximum depth shown, all hydraulic analysis shall be based upon the full amount of irrigation water being present.

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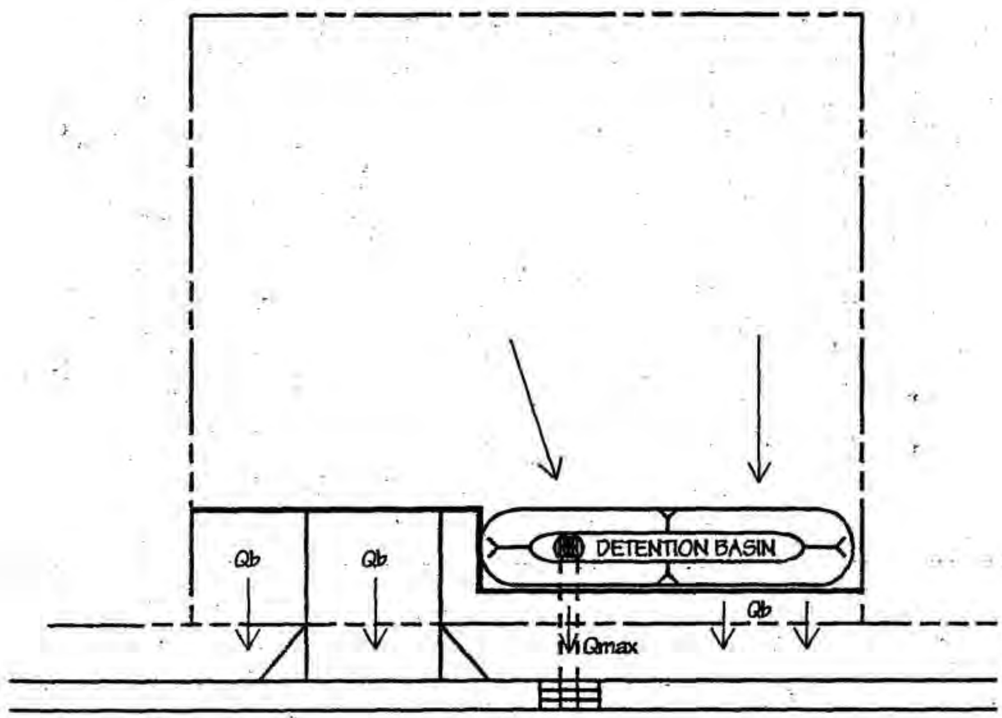
REFERENCE:

MESA COUNTY SWMM DATED MAY 1996

FIGURE 1401

## TOTAL SITE RUNOFF




Maximum release from detention pond Qmax ≤ Historic peak $Q_h$ minus direct bypass runoff Qb

$Qmax \leq Q_h - Qb$

| Revision | Date |
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| ORIGINAL ISSUE | 3/27/06 |
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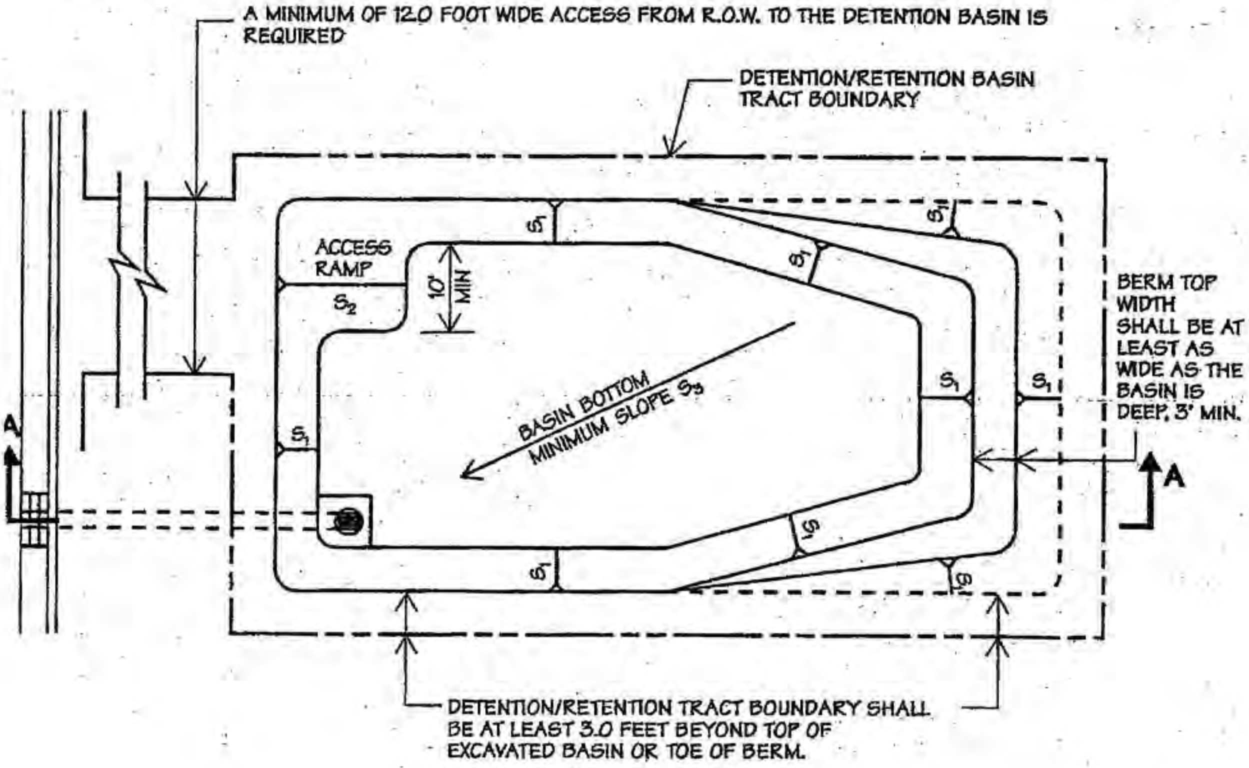
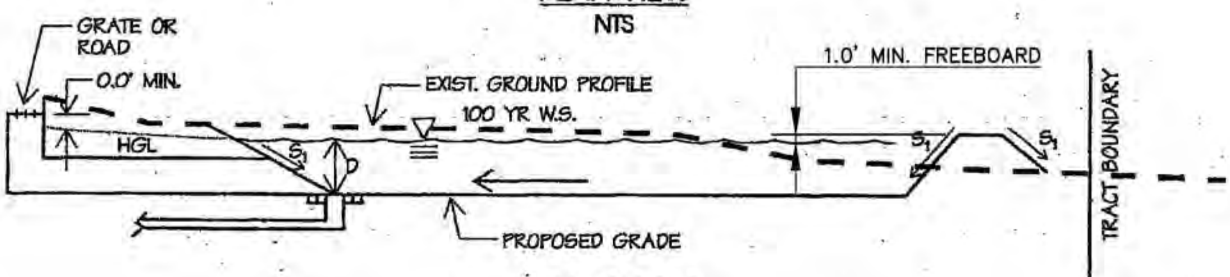
**FIGURE 1402**

# DETENTION/RETENTION BASIN GEOMETRIC REQUIREMENTS




PLAN VIEW
NTS




SECTION A-A
NTS

STEEPEST $S_1$: 4H:1V FOR BASINS ON PUBLIC LANDS AND PARKS
3H:1V FOR SEEDED OR SODDED SLOPES
2H:1V FOR RIPRAP OR OTHER APPROVED SLOPE PROTECTION
VERTICAL WALLS WITH SAFETY RAILING LIMITED TO ONE SIDE ONLY WHERE APPROVED BY THE CITY ENGINEER OR COUNTY DEVELOPMENT ENGINEER.

STEEPEST $S_2$: 6H:1V FOR ACCESS RAMP, ALL SURFACES

MINIMUM $S_3$: 0.5% FOR CONCRETE CHANNEL
1.0% FOR ASPHALT (PARKING LOT)
2.0% FOR ALL OTHER SURFACES

MAXIMUM D: 4' RETENTION BASIN
8' WET OR DRY DETENTION FACILITY
>8' SPECIAL APPROVAL REQUIRED, BUT MAY BE ALLOWED FOR MULTIPLE USE PONDS OR FOR STEEP TERRAINS

MINIMUM D: 4' WET PONDS (SEE PAGE VIII-1)

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| ORIGINAL ISSUE | 3/27/06 |
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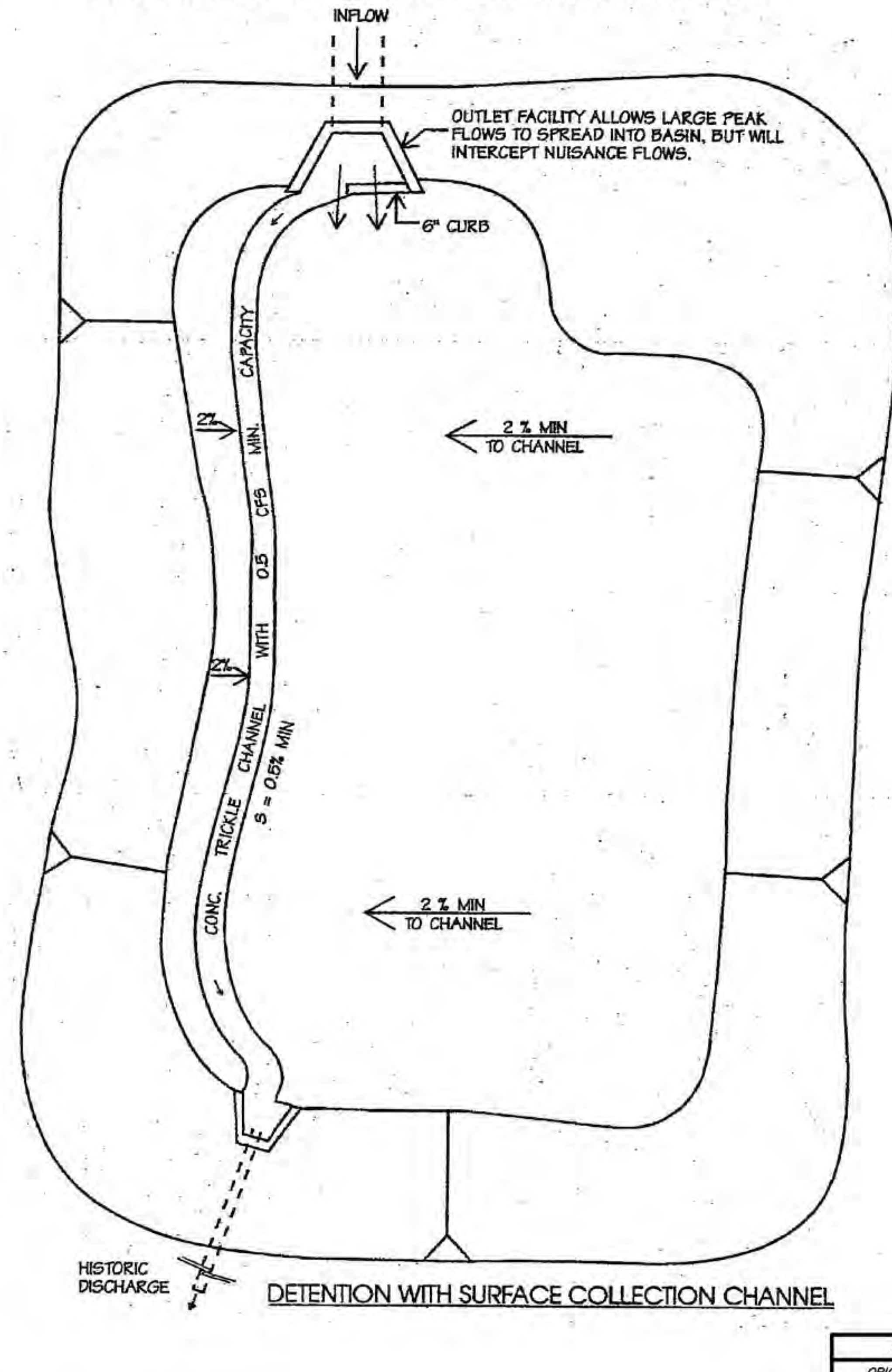
# DRY BASIN TRICKLE FLOW CONVEYANCE




DETENTION WITH SURFACE COLLECTION CHANNEL

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| ORIGINAL ISSUE | 3/27/06 |
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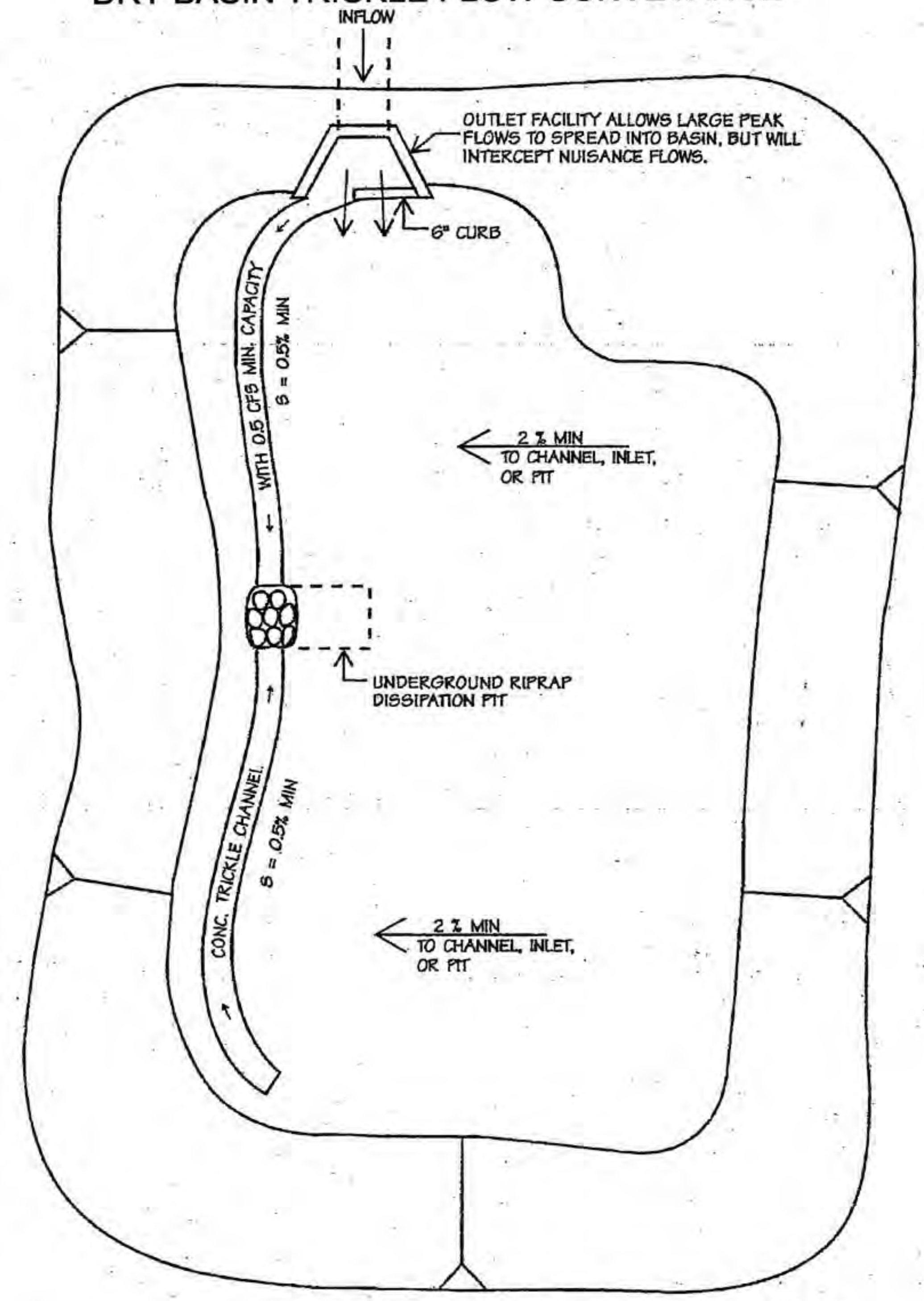
MESA COUNTY SWMM DATED MAY 1996

**FIGURE 1404**

# STORMWATER MANAGEMENT MANUAL

## DRY BASIN TRICKLE FLOW CONVEYANCE




RETENTION WITH SURFACE COLLECTION CHANNEL
(THE SAME CONCEPT APPLIES TO AN UNDERGROUND SYSTEM)

| Revision | Date |
|---|---|
| ORIGINAL ISSUE | 3/27/06 |
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| | |

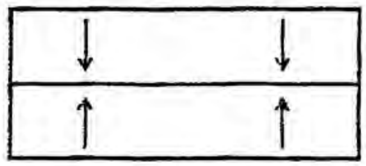
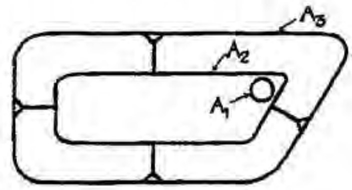
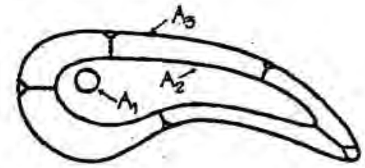
WRC ENGINEERING, INC

REFERENCE:

MESA COUNTY SWMM DATED MAY 1996

**FIGURE 1405**

## CALCULATING STORAGE VOLUME

| | | |
|---|---|---|
| BASIN SHAPE |  |   |
| BASIN TYPE | VERTICAL WALLS AND/OR PRISMATIC BASINS | FAIRLY UNIFORM SHAPE AND SIDE SLOPES OR HIGHLY IRREGULAR SHAPE AND SIDE SLOPES |
| VOLUME CALCULATION METHOD | AVERAGE END AREA METHOD | CONIC METHOD |
| EQUATION | $V = \left(\frac{A_n + A_{n+1}}{2}\right)L$ | $V = \Sigma V_{n \text{ to } n+1}$<br><br>$V_{n \text{ to } n+1} = \left[A_n + A_{n+1} + (A_n A_{n+1})^5\right]\frac{h}{3}$ |

WHERE: $V$ = VOLUME (ft)
$A_n$ = HORIZONTAL AREA ($ft^2$) AT ELEVATION "n"
$A_{n+1}$ = HORIZONTAL AREA ($ft^2$) AT ELEVATION "n+1"
$h$ = VERTICAL HEIGHT (ft) BETWEEN ELEVATION "n" AND "n+1"
$V_{n \text{ to } n+1}$ = VOLUME BETWEEN ELEVATION "n" AND "n+1"
$L$ = LENGTH (ft) BETWEEN TWO ENDS

NOTE: THE ABOVE EQUATIONS MAY BE USED IN SUCCESSION FOR INCREMENTAL HEIGHTS WITHIN A BASIN. AN AREA SHOULD BE SELECTED AT ALL SIGNIFICANT CHANGES IN SHAPE OR SIDE SLOPE.

| Revision | Date |
|---|---|
| ORIGINAL ISSUE | 3/27/06 |
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# SECTION 1500

## CONSTRUCTION SITE STORMWATER RUNOFF CONTROL

# SECTION 1500
## CONSTRUCTION SITE STORMWATER RUNOFF CONTROL

## TABLE OF CONTENTS

## LIST OF TABLES



## LIST OF FIGURES

# SECTION 1500
# CONSTRUCTION SITE STORMWATER RUNOFF CONTROL

## 1501 INTRODUCTION

All persons engaged in land disturbance for new development, disturbing 1.0 acres or more, or development disturbing less than 1.0 acres that is part of a larger common plan of development or sale, shall implement the following measures to prevent the discharge of pollutants to the stormwater system to the maximum extent practicable:

a. Obtain a permit from the Water Quality Control Division (WQCD) for Stormwater Discharges Associated with Construction Activity, called "Stormwater Construction Permit". A condition of the permit requires the preparation and implementation of a Stormwater Management Plan (SWMP) to control the discharge of pollutants resulting from construction activity.

   Information required by the WQCD for a Stormwater Management Plan is included in this Section and shall be referred to as the "Construction SWMP". The Construction SWMP must include appendices with the WQCD permit application, text and graphics identifying erosion control measures to be implemented during construction, and be certified by a qualified erosion control specialist.

   Exemptions to the requirement for a SWMP for projects that disturb less than 5-acres, may be granted by the WQCD when the Rainfall Erosivity Factor (R-Factor) is less than a value of 5 ( see website at http://www.cdphe.state.co.us/wq/PermitsUnit/wqcdpmt.html). However, the exemptions do not relieve the owner/operator from meeting discharge limitations described in Section 1503.1.

b. For development within the designated urbanized area of unincorporated Mesa County (see **Figure 401**), obtain a Mesa County Stormwater Construction Permit. Application for the Mesa County Stormwater Construction Permit may be submitted concurrent with the application for the WQCD Stormwater Construction Permit, but the WQCD permit must be received by the County before approval of the County permit. Within the City of Fruita and the Town of Palisade, contact the Engineering or Planning Department for requirements.

c. For development within the City of Grand Junction, obtain a Grand Junction Stormwater Construction Permit. Application for the Grand Junction Stormwater Construction Permit may be submitted concurrent with the application for the WQCD Stormwater Construction Permit, but the WQCD permit must be received by the City of Grand Junction before approval of the Grand Junction permit. Development within the City of Grand Junction **does not** need a Mesa County permit.

d. Provide measures to control erosion and sedimentation within the onsite major drainageway(s), in accordance with Section 800 *Open Channels*.

e. Implement post-construction BMPs to control the discharge of pollutants after construction is completed, in accordance with Section 1600 *Post-Construction Stormwater Management*.

## 1502 COLORADO DISCHARGE PERMIT

### 1502.1 Stormwater Discharge Permit

The Water Quality Control Division (WQCD) of the Colorado Department of Public Health and Environment (CDPHE) has direction and authority to administer the Federal stormwater permit program (i.e.: NPDES Phase I and Phase II) as part of the Colorado Water Quality Control Act (CRS 25-8-101 et seq.). The terms and conditions of the permit have set forth minimum requirements for stormwater management programs for Municipalities that address water quality from significant development or redevelopment, from construction sites and from other activities. Programs include:

a. Public Education and Outreach on Stormwater Impacts. Prepare and distribute education materials to the community or conduct outreach activities about impacts of stormwater discharges on water bodies.

b. Public Involvement/Participation. Conduct and document public meetings related to implementation of the Stormwater Management Programs.

c. Illicit Discharge Detection and Elimination. Detect and prohibit illicit discharges through regulatory measures.

d. Construction Sites Stormwater Runoff Control. Develop, implement, and enforce a program to reduce pollutants in any stormwater runoff to the MS4 from construction activities that result in a land disturbance of greater than or equal to 1.0 acres. This permit condition is the basis for requirements identified in this Section of the Manual.

e. Post-Construction Stormwater Management in New Development and Redevelopment. Develop, implement, and enforce a program to reduce pollutants in any stormwater runoff to the MS4 from all new development and significant redevelopment. This permit condition is the basis for requirements identified in Section 1600.

f. Pollution Prevention/Good Housekeeping for Municipal Operations. Develop and implement an operation and maintenance program that includes an employee training component and has the ultimate goal of preventing or reducing pollutant runoff from municipal operations.

Additional information can be found at the CDPHE website address
http://www.cdphe.state.co.us/cdphehom.asp

### 1502.2 Mesa County Permittees

Mesa County, the City of Grand Junction, the Grand Junction Drainage District, and the Town of Palisade, who are members of the 5-2-1 Drainage Authority, each have obtained a permit to discharge stormwater under the Colorado Discharge Permit System (CDPS, COR-090031, COR-090077, COR-090006, and COR-090005, respectively). Other jurisdictions within Mesa County may be subject to these regulations in the future and the Manual user is encouraged to contact their local jurisdiction for more information.

### 1502.3 Stormwater Management Manual

Information presented in this Manual meets, in part, requirements for the following programs: Illicit Discharge Detection and Elimination, Construction Sites Stormwater Runoff Control, and Post-Construction Stormwater Management in New Development and Redevelopment. The CDPS Phase II Stormwater Discharge Permit includes requirements for MS4's to control erosion and sedimentation from construction activities and has identified:

a. Procedures for site planning

b. Requirements for selection, implementation, installation and maintenance of best management practices (BMPs)

c. Procedures for site inspection and enforcement.

Requirements and procedures for these items are addressed by this Manual.

## 1503 STANDARDS AND REQUIREMENTS

### 1503.1 Stormwater Discharge Limitations

All stormwater discharges from construction sites disturbing 1.0 acres or more shall meet the following standards:

a. Stormwater discharges from construction activities shall not cause, have the reasonable potential to cause, or measurably contribute to an exceedance of any water quality standard, including narrative standards for water quality.

b. Concrete wash water shall not be discharged to state waters or to storm sewer systems. On-site permanent disposal of concrete washout waste is not authorized. Discharge to the ground of concrete washout waste that will subsequently be disposed of off-site is authorized.

   Discharges to the ground of concrete washout water from washing of tools and concrete mixer chutes may be authorized, provided that:
   1. The source is identified in the Construction SWMP;
   2. BMPs are included in the Construction SWMP to prevent pollution of groundwater; and
   3. These discharges do not leave the site as surface runoff or to surface waters.

c. Bulk storage structures for petroleum products and other chemicals shall have adequate protection so as to contain all spills and prevent any spilled material from entering State waters.

d. All site wastes must be removed from the site for disposal in licensed disposal facilities. No building material wastes or unused building materials shall be buried, dumped, or discharged at the site. All site wastes must be properly managed to prevent potential pollution of State Waters.

e. Off-site vehicle tracking of sediments shall be minimized.

f. Land disturbances shall be conducted in a manner to effectively reduce accelerated soil erosion and sedimentation.

g. Discharges to the ground of water from construction dewatering activities may be authorized provided that:
   1. The source is groundwater and/or groundwater combined with stormwater that does not contain pollutants in concentrations exceeding the State groundwater standards in Regulations 5 CCR 1002-41 and 42;
   2. The source is identified in the Construction SWMP;
   3. BMPs are included in the Construction SWMP; and
   4. These discharges do not leave the site as surface runoff or to surface waters.

### 1503.2 Minimum BMP Requirements

The Stormwater Management Plan (SWMP) prepared for the WQCD as a condition of the Stormwater Construction Permit shall include as a minimum both structural and non-structural measures. Facilities must select, install, implement, and maintain appropriate BMPs following good engineering, hydrologic and pollution control practices. BMPs implemented at the site must be adequately designed to provide control for all potential pollutant sources associated with construction activity to prevent pollution or degradation of State waters. The primary goal of the Construction SWMP is to reduce erosion at the source, followed by trapping eroded materials before they leave the site. The Construction SWMP shall include, as a minimum the following (See Section 1507 for detailed Construction SWMP requirements):

a. BMPs are required to control erosion at the source, such as those that stabilize earth disturbances with vegetation or mulch after grading is substantially complete on any portion of the site not otherwise permanently stabilized. Typical examples include surface roughening (Symbol SR, **Figure 1502**), mulching (Symbol MU, **Figure 1503**) and vehicle tracking control (Symbol VTC, **Figure 1504**). Other control methods not shown include installation of blankets, straw wattles, tackifiers, netting, and matting.

b. BMPs are required to trap sediment before it leaves the site or enters the municipal storm sewer system, which ever comes first. Such BMPs shall be installed prior to initiating earth disturbances. Typical examples include check dams (Symbol CD, **Figure 1505**), inlet protection (Symbol IP, **Figure 1506**), sediment basin (Symbol SB, **Figure 1507**), and silt fence (Symbol SF, **Figure 1508**).

c. BMPs are required to prevent spills of petroleum products and other chemicals and contain storm runoff from construction wastes to a designated area, if applicable. See Section 1607 *BMPs for Industrial and Commercial Activities* for examples.

d. Construction sequencing for all BMPs is required to reduce the duration a disturbed area is exposed. Temporary disturbed areas shall be exposed no longer than 30-days. Disturbed areas that are to be permanently stabilized shall be exposed no longer than 7-days.

e. BMPs are required to capture and retain runoff from equipment washing operations, such as cleaning of concrete trucks. See Section 1607 *BMPs for Industrial and Commercial Activities* for examples.

f. Program and schedule for regular inspection and maintenance of BMPs.

### 1503.3 Acceptable BMPs

Criteria for selection and design of BMPs can be found in the following documents:

a. Colorado Department of Transportation 2002. Erosion Control and Stormwater Quality Guide. This document is available in pdf format at *http://www.dot.state.co.us/environmental/envWaterQual/wqms4.asp* CAD drawings for BMPs are also available at CDOT website.

b. *Urban Drainage & Flood Control District (UDFCD) 1999.* Urban Storm Drainage Criteria Manual, Volume 3 - Best Management Practices. This can be found at *http://www.udfcd.org/downloads/down_critmanual.htm*

c. *EPA 2002. Consideration in the Design of Treatment Best Management Practices (BMP) to Improve Water Quality. EPA 600 R-03/103.* This document is available in pdf form at *the following website http://www.epa.gov/ORD/NRMRL/pubs/600r03103/600r03103chp5.pdf*

d. City of Grand Junction August 2005. Standard Contract Documents for Capital Improvement Projects. *This document is available in pdf form at http://www.gjcity.org/citydeptwebpages/publicworksandutilities/engineering/standardcontractdocuments.htm.*

The BMPs presented in the documents referenced above shall be used in the preparation of the Construction SWMP (See Section 1507), except as modified as follows. Use of alternate BMP's not specified above is subject to approval by the local jurisdiction.

a. Where stormwater is within a swale and a silt fence is used to trap sediment, the silt fence is to be reinforced such as with straw bales, or a double silt fence arrangement.

b. Straw bales to trap sediment shall not be used when stormwater is concentrated in a swale, or a channel (see Section 800).

See **Figure 1501a & b** for list of construction BMPs and map symbols for the erosion control plan.

## 1504 STORMWATER PERMIT

### 1504.1 Mesa County

The Construction SWMP shall be reviewed in accordance with this Section of the Manual and a Stormwater Construction Permit will be issued after:

a. The WQCD issues a permit for Stormwater Discharges Associated with Construction Activity. Note that the application to the WQCD may occur before the application to the County, but the WQCD permit must be received by the County before the County will approve the Construction SWMP and issues a Construction Stormwater Permit. The County permit is not needed within the City of Grand Junction limits.

b. Approval of the Construction Stormwater Management Plan, which includes written, and graphical descriptions of construction erosion control measures, and a certification (see Section 1507).

c. Submittal of proper collateral, if required.

d. Written notice from Construction Stormwater Permit holder that initial construction BMPs will be installed prior to any land disturbance.

### 1504.2 City of Grand Junction

The Construction SWMP shall be reviewed in accordance with this Section of the Manual and a Grand Junction Construction Stormwater Permit will be issued after:

a. The WQCD issues a permit for Stormwater Discharges Associated with Construction Activity. Note that the application to the WQCD may occur before the application to the City, but the WQCD permit must be received by the City before the City will approve the Construction SWMP and issues a Construction Stormwater Permit.

b. Approval of the Construction Stormwater Management Plan, which includes written, and graphical descriptions of construction erosion control measures, and a certification (see Section 1507).

c. Submittal of proper collateral, if required.

d. Written notice from Construction Stormwater Permit holder that initial construction BMPs will be installed prior to any land disturbance.

e. In addition, the City may require a Construction Stormwater Permit if the development disturbs less than 1.0 acres and if:

   i. The City determines that the development is on steep slopes, may contribute to a violation of water quality standards, or may be a significant source of pollutants, or

   ii. The development is in close proximity to a wetland, water body or waterway.

### 1504.3 Other Jurisdictions

The City of Fruita, the Town of Palisade, and the Grand Junction Drainage District do not require a separate construction related permit, but the owner/operator must obtain a Stormwater Construction Permit from the WQCD, which requires preparation of a Construction Stormwater Management Plan. A copy of the WQCD stormwater permit shall be provided to the local jurisdiction.

## 1505 STORMWATER PERMIT VARIANCE/EXCEPTION

Variances/exceptions from Stormwater Permitting may be granted for the following conditions (check with local jurisdictions). The variance/exception from permitting does not relieve the owner/operator from meeting discharge limitations described in Section 1503.1, *Stormwater Discharge Limitations*, or from those in City of Grand Junction Ordinance Number 3824 within the City's jurisdiction.

a. Land disturbance activities in areas where the topography would prohibit runoff from leaving the site or entering a waterway.

b. Agricultural and silviculture activities such as home gardening and tilling a field for weed control.

c. Maintenance activities, such as regrading a dirt road, relandscaping a lawn, cleaning out roadside ditches, and other land disturbances that do not alter original line and grade, hydraulic capacity or original purpose.

d. Repaving a roadway, providing that underlying and/or surrounding soil is not cleared, graded, excavated or otherwise disturbed.

e. For small construction activity (i.e.: from 1 to 5 acres of earth disturbance) based on the Rainfall Erosivity Factor (i.e.: the R Factor) (see Colorado Discharge Permits System Regulation No. 61 @ 61.3(2)(f)(ii)(B)) and duration of the exposed disturbance. To obtain a waiver from the CDPS Construction General Permit, the applicant must submit calculations to the WQCD (see "Policy on the State Approved Method for Calculating the Rainfall Erosivity Factor" which can be obtained in pdf form by searching for "erosivity factor" in the website: http://www.cdphe.state.co.us/index.html

f. The following activities provided the land disturbance is less than 1-acre and are not part of a Larger Common Plan of Development or Sale:

   i. Grading or an excavation below finished grade for basements, footings, retaining wall, or other structures on plots unless otherwise required by the Uniform Building Code.

   ii. A sidewalk or driveway authorized by a valid permit.

   iii. Gravel, sand, or soil removal as authorized.

# 1506 INSPECTION AND ENFORCEMENT

## 1506.1 Mesa County

Upon approval of the Construction SWMP (See Section 1504 *Stormwater Permit*), a Stormwater Permit will be issued to the applicant, which requires:

a. **Inspector**. The owner/operator shall retain a qualified erosion control specialist to perform inspections of the BMPs in the Construction SWMP and determine if the construction is in accordance with the Construction SWMP.

b. **Inspections**. The site perimeter, all disturbed areas, material and/or waste storage areas that are exposed to precipitation, discharge locations, and locations where vehicles access the site shall be inspected for evidence of, or the potential for, pollutants leaving the construction site boundaries, entering the stormwater drainage system, or discharging to state waters. All erosion and sediment control practices identified in the Construction SWMP shall be evaluated to ensure that they were properly installed and are being maintained and operated correctly.

c. **Inspection Frequency**. The inspector shall perform inspections at least once every 14 days and within 24-hours after the end of a precipitation or snowmelt event that causes surface erosion on the site.

d. **Inspection Reporting**. The inspector shall prepare inspection reports and submit a copy of all inspection reports and any changes to the Construction SWMP to Mesa County within five working days of the inspection. Inspection results shall be logged on forms provided, site conditions noted, and any changes to the Construction SWMP shall be documented.

   The permittee shall keep a record of inspections. Inspection reports must identify any incidents of non-compliance with the terms and conditions of this permit. Inspection records must be retained for three years from expiration or inactivation of permit coverage. At a minimum, the inspection report must include:

   1. The inspection date;
   2. Name(s) and title(s) of personnel making the inspection;
   3. Location(s) of discharges of sediment or other pollutants from the site;
   4. Location(s) of BMPs that need to be maintained;
   5. Location(s) of BMPs that failed to operate as designed or proved inadequate for a particular location;
   6. Location(s) where additional BMPs are needed that were not in place at the time of inspection;
   7. Deviations from the minimum inspection schedule as provided in item C above.
   8. Description of corrective action for items iii, iv, v, and vi, above, dates corrective action(s) taken, and measures taken to prevent future violations, including requisite changes to the Construction SWMP, as necessary; and
   9. After adequate corrective action(s) has been taken, or where a report does not identify any incidents requiring corrective action, the report shall

contain a signed statement indicating the site is in compliance with the permit to the best of the signer's knowledge and belief.

e. **Changes to Construction SWMP**. The permittee shall amend the Construction SWMP:

1. When there is a change in design, construction, operation, or maintenance of the site, which would require the implementation of new or revised BMPs; or
2. If the Construction SWMP proves to be ineffective in achieving the general objectives of controlling pollutants in stormwater discharges associated with construction activity; or
3. When BMPs are no longer necessary and are removed.

Construction SWMP changes shall be made prior to changes in the site conditions, except as allowed for in the paragraph below. Construction SWMP revisions may include, but are not limited to: potential pollutant source identification; selection of appropriate BMPs for site conditions; BMP maintenance procedures; and interim and final stabilization practices. The Construction SWMP changes may include a schedule for further BMP design and implementation, provided that, if any interim BMPs are needed to comply with the permit, they are also included in the Construction SWMP and implemented during the interim period.

Construction SWMP changes addressing BMP installation and/or implementation are often required to be made in response to changing conditions, or when current BMPs are determined ineffective. The majority of Construction SWMP revisions to address these changes can be made immediately with quick in-the-field revisions to the Construction SWMP. In the less common scenario where more complex development of materials to modify the Construction SWMP is necessary, Construction SWMP revisions shall be made in accordance with the following requirements:

1. The Construction SWMP shall be revised as soon as practicable, but in no case more than 72 hours after the change(s) in BMP installation and/or implementation occur at the site, and
2. A notation must be included in the Construction SWMP prior to the site change(s) that includes the time and date of the change(s) in the field, an identification of the BMP(s) removed or added, and the location(s) of those BMP(s)

f. **Substantial Completion.** For construction sites that are substantially complete but where final stabilization is not complete, additional inspections shall be conducted at no less than once a month until final stabilization is achieved. These Inspections shall include the perimeter of the site, disturbed areas, and material storage areas exposed to precipitation for evidence of, or potential for, the discharge of pollutants to the drainage system.

g. **Final Completion**. For construction sites where final stabilization has been achieved, the inspector shall:

i. Provide certification from the Engineer of Record that the stormwater volume capacity of any detention site has been restored to the design condition prior to turning over maintenance responsibility to an owners association.

ii. Submit to the County the final inspection report, final Construction SWMP, and certification that the project is completed in accordance with the Construction SWMP.

The Construction Stormwater Permit will be released after final stabilization has taken place, as certified by the qualified erosion control specialist.

h. **Enforcement.** Mesa County will audit 10 percent of all construction sites to verify that inspections have been conducted, that the Construction SWMP is current and is addressing erosion and sedimentation concerns.

If Mesa County determines that inspections are not being conducted in accordance with the Manual, they will notify the inspector and the permit holder by mail, and will require the deficiencies be corrected within the stated time frame. If deficiencies are not corrected with 30-days of the first notice of a deficiency, enforcement measures for the Stormwater Permit, such as required changes to the Construction SWMP, stop work-orders, and fines, will be implemented, in accordance with requirements of the Land Development Code.

### 1506.2 City of Grand Junction

Inspection and enforcement of construction BMPs will be performed by the City in accordance with Ordinance No. 3824 (Article VII, Chapter 16 of the Code of Ordinances) and as amended.

Upon approval of the Construction SWMP (See Section 1504 *Stormwater Permit*), a Stormwater Permit will be issued to the applicant, which requires:

a. **Inspection Frequency**. The owner/operator shall inspect all BMPs at least once every fourteen days, and after any precipitation or snowmelt event that causes surface erosion. The owner/operator must provide consent to the City for the City to inspect any BMP without advance notice or permission from the owner/operator.

b. **Inspections**. Inspections shall include proper installation and maintenance of all BMPs, the perimeter of the site, disturbed areas, and material storage areas exposed to precipitation and shall be conducted for evidence of, or potential for, the discharge of pollutants to the drainage system.

c. **Inspection Reporting**. Inspection results shall be logged on forms provided, site conditions noted, and any changes to the Construction SWMP shall be documented. Based upon inspections performed by the owner/operator or by authorized City personnel, modifications to the Construction SWMP shall be necessary if at any time the specified BMPs do not meet the objectives of Code of Ordinances, Article VII Chapter 16 and as amended.

The permittee shall keep a record of inspections. Inspection reports must identify any incidents of non-compliance with the terms and conditions of this permit. Inspection records must be retained for three years from expiration or inactivation of permit coverage. At a minimum, the inspection report must include:

i. The inspection date;
ii. Name(s) and title(s) of personnel making the inspection;
iii. Location(s) of discharges of sediment or other pollutants from the site;
iv. Location(s) of BMPs that need to be maintained;
v. Location(s) of BMPs that failed to operate as designed or proved inadequate for a particular location;
vi. Location(s) where additional BMPs are needed that were not in place at the time of inspection;
vii. Deviations from the minimum inspection schedule as provided in item C above.
viii. Description of corrective action for items iii, iv, v, and vi, above, dates corrective action(s) taken, and measures taken to prevent future violations, including requisite changes to the Construction SWMP, as necessary; and
ix. After adequate corrective action(s) has been taken, or where a report does not identify any incidents requiring corrective action, the report shall contain a signed statement indicating the site is in compliance with the permit to the best of the signer's knowledge and belief.

d. **Changes to Construction SWMP**. If major modification to the Construction SWMP is required, such as addition or deletion of a sediment basin, the owner/operator shall meet and confer with authorized City personnel to determine the nature and extent of modification(s). Minor modifications necessary to meet the objectives of Code of Ordinances, Article VII Chapter 16 and as amended may be performed without City authorization. All approved modifications(s) shall be completed in a timely manner. All modification(s) shall be recorded on the owner/operator's copy of the Construction SWMP. In the case of an emergency, the contractor shall implement conservative BMPs and follow up with City personnel the next working day.

   The permittee shall amend the Construction SWMP:

   1. When there is a change in design, construction, operation, or maintenance of the site, which would require the implementation of new or revised BMPs; or
   2. If the Construction SWMP proves to be ineffective in achieving the general objectives of controlling pollutants in stormwater discharges associated with construction activity; or
   3. When BMPs are no longer necessary and are removed.

   Construction SWMP changes shall be made prior to changes in the site conditions, except as allowed for in the paragraph below. Construction SWMP revisions may include, but are not limited to: potential pollutant source identification; selection of appropriate BMPs for site conditions; BMP maintenance procedures; and interim and final stabilization practices. The Construction SWMP changes may include a schedule for further BMP design and implementation, provided that, if any interim BMPs are needed to comply with the permit, they are also included in the Construction SWMP and implemented during the interim period.

Construction SWMP changes addressing BMP installation and/or implementation are often required to be made in response to changing conditions, or when current BMPs are determined ineffective. The majority of Construction SWMP revisions to address these changes can be made immediately with quick in-the-field revisions to the Construction SWMP. In the less common scenario where more complex development of materials to modify the Construction SWMP is necessary, Construction SWMP revisions shall be made in accordance with the following requirements:

1. The Construction SWMP shall be revised as soon as practicable, but in no case more than 72 hours after the change(s) in BMP installation and/or implementation occur at the site, and
2. A notation must be included in the Construction SWMP prior to the site change(s) that includes the time and date of the change(s) in the field, an identification of the BMP(s) removed or added, and the location(s) of those BMP(s).

e. **Substantial Completion**. For construction sites that are substantially complete but where final stabilization is not complete, additional inspections shall be conducted at no less than once a month until final stabilization is achieved. These Inspections shall include the perimeter of the site, disturbed areas, and material storage areas exposed to precipitation for evidence of, or potential for, the discharge of pollutants to the drainage system.

f. **Final Completion/Inactivation**. For construction sites where final stabilization has been achieved, the inspector shall:

   i. Provide certification from the Engineer of Record that the stormwater volume capacity of any detention site has been restored to the design condition prior to turning over maintenance responsibility to an owners association.

   ii. Submit to the City the final inspection report, final Construction SWMP, and certification that the project is completed in accordance with the Construction SWMP.

   iii. Provide a copy of the CDPHE Inactivation Notice for Construction Stormwater Discharge General Permit Certification.

The Construction Stormwater Permit will be released after final stabilization has taken place, as certified by the qualified erosion control specialist.

g. **Enforcement.** The City of Grand Junction has the right to enter the premises at any time to investigate if the discharger is complying with all the requirements of the Construction SWMP, in accordance with City Code of Ordinances Section 16-144 and as amended.

h. **Upset Condition**. An upset condition determination constitutes an affirmative defense to an action brought for noncompliance when the terms of the Construction SWMP are met. An owner/operator who whishes to establish the affirmative defense of upset must demonstrate, through properly signed, contemporaneous operating logs, or other relevant evidence that:

i. An upset occurred and that the cause(s) of the upset can be identified,
ii. The facility or operation was at the time being properly operated,
iii. Notice of the upset was submitted as required in City Code of Ordinances § 16-42(D) and as amended, and
iv. Remedial measures were complied with as required.

In any enforcement proceeding, the one seeking to establish the occurrence of an upset has the burden of proof.

### 1506.3 Other Municipalities

Contact the City of Fruita, the Town of Palisade, and the Grand Junction Drainage District to determine inspection and enforcement requirements.

## 1507 CONSTRUCTION SWMP

The Construction SWMP shall be prepared by a qualified erosion control specialist in accordance with good hydrologic and pollution control practices. A current copy of the Construction SWMP must be on the site at all times.

The Construction SWMP shall identify potential sources of pollution (including sediment) which may reasonably be expected to affect the quality of stormwater discharges associated with construction activity. The Construction SWMP shall also describe and ensure the implementation of BMPs which will be used to reduce the pollutants in stormwater discharges associated with construction activity. Construction operations must implement the provisions of the Construction SWMP.

A checklist of items required for the Construction SWMP is provided in **Table 1501a & b**. The Construction SWMP shall include the following items, at a minimum:

### 1507.1 Site Description.

Each plan shall provide a description of the following:

a. A description of the nature of the construction activity.
b. The proposed sequence for major activities.
c. Estimates of the total area of the site, and the area of the site that is expected to undergo clearing, excavation or grading.
d. A summary of any existing data used in the development of the site construction plans or Construction SWMP that describe the soil or existing potential for soil erosion.
e. A description of the existing vegetation at the site and an estimate of the percent vegetative ground cover. Photos of the existing vegetation may be useful to determine if final stabilization has occurred.
f. The location and description of all potential pollution sources, including ground surface disturbing activities, vehicle fueling, storage of fertilizers or chemicals, etc.
g. The location and description of any anticipated non-stormwater components of the discharge, such as uncontaminated springs, landscape irrigation return flow, and construction dewatering.

h. The name of the receiving water(s) and the size, type and location of any outfall(s). If the discharge is to a municipal separate storm sewer, the name of that system, the location of the storm sewer discharge, and the ultimate receiving water(s).

### 1507.2 Site Map

Each plan shall provide a generalized site map or maps which indicate:

a. Construction site boundaries
b. All areas of soil disturbance
c. Areas of cut and fill
d. Areas used for storage of building materials, soils or wastes
e. Location of any dedicated asphalt or concrete batch plants
f. Location of BMPs needed to address pollutant sources identified in Section 1507.3 (b) below
g. Location of springs, streams, wetlands and other surface waters
h. Stormwater discharge locations

### 1507.3 Stormwater Management Controls

The Construction SWMP must include a description of all stormwater management controls that will be implemented as part of the construction activity to control pollutants in stormwater discharges. The appropriateness and priorities of stormwater management controls in the Construction SWMP shall reflect the potential pollutants sources identified at the facility.

The description of stormwater management controls shall address the following components, at a minimum:

a) **Construction SWMP Administrator** – The Construction SWMP shall identify a specific individual(s), position or title responsible for developing, implementing, maintaining, and revising the Construction SWMP. The activities and responsibilities of the administrator shall address all aspects of the facility's Construction SWMP.

b) **Identification of Potential Pollutant Sources** - All potential pollutant sources, including materials and activities, at a site must be evaluated for the potential to contribute pollutants to stormwater discharges. The Construction SWMP shall identify and describe those sources determined to have the potential to contribute pollutants to stormwater discharges, and the sources must be controlled through BMP selection and implementation, as required in paragraph (c), below.

   At a minimum, each of the following sources and activities shall be addressed in the Construction SWMP. If the source or activity does not have the potential to contribute pollutant it must be indicated in the Construction SWMP:

   1. all disturbed and stored soils;
   2. vehicle tracking of sediments;
   3. management of contaminated soils;
   4. loading and unloading operations;

5. outdoor storage activities (building materials, fertilizers, chemicals, etc.);
6. vehicle and equipment maintenance and fueling;
7. significant dust or particulate generating processes;
8. routine maintenance activities involving fertilizers, pesticides, detergents, fuels, solvents, oils, etc.;
9. onsite waste management practices (waste piles, liquid wastes, dumpsters, etc.);
10. concrete truck/equipment washing, including the concrete truck chute and associated fixtures and equipment;
11. dedicated asphalt and concrete batch plants;
12. non-industrial waste sources such as worker trash and portable toilets; and
13. other areas or procedures where potential spills can occur.

c) **Best Management Practices (BMPs) for Stormwater Pollution Prevention** - The Construction SWMP shall identify and describe appropriate BMPs, including, but not limited to, those required by paragraphs 1 through 8 below, that will be implemented at the facility to reduce the potential of the sources identified in Section 1507.3 b to contribute pollutants to stormwater discharges. The Construction SWMP shall clearly describe the installation and implementation specifications for each BMP identified in the Construction SWMP to ensure proper implementation, operation and maintenance of the BMP.

1. Structural Practices for Erosion and Sediment Control. The Construction SWMP shall clearly describe and locate all structural practices implemented at the site to minimize erosion and sediment transport. Practices may include, but are not limited to: straw bales, wattles/sediment control logs, silt fences, earth dikes, drainage swales, sediment traps, subsurface drains, pipe slope drains, inlet protection, outlet protection, gabions, and temporary or permanent sediment basins.

2. Non-Structural Practices for Erosion and Sediment Control. The Construction SWMP shall clearly describe and locate, as applicable, all non-structural practices implemented at the site to minimize erosion and sediment transport. Description must include interim and permanent stabilization practices, and site-specific scheduling for implementation of the practices. The Construction SWMP should include practices to ensure that existing vegetation is preserved where possible. Non-structural practices may include, but are not limited to: temporary vegetation, permanent vegetation, mulching, geotextiles, sod stabilization, slope roughening, vegetative buffer strips, protection of trees, and preservation of mature vegetation.

3. Phased BMP Implementation. The Construction SWMP shall clearly describe the relationship between the phases of construction, and the implementation and maintenance of both structural and non-structural stormwater management controls. The Construction SWMP must identify the stormwater management controls to be implemented during the project phases, which can include, but are not limited to, clearing and grubbing; road construction; utility and infrastructure installation; vertical construction; final grading; and final stabilization.

4. Materials Handling and Spill Prevention. The Construction SWMP shall clearly describe and locate all practices implemented at the site to minimize impacts from procedures or significant materials that could contribute pollutants to runoff. Such procedures or significant materials could include: exposed storage of building materials; paints and solvents; fertilizers or chemicals; waste material; and equipment maintenance or fueling procedures. Areas or procedures where potential spills can occur must have spill prevention and response procedures identified in the Construction SWMP.

5. Dedicated Concrete or Asphalt Batch Plants. The Construction SWMP shall clearly describe and locate all practices implemented at the site to control stormwater pollution from dedicated concrete batch plants or dedicated asphalt batch plants covered by this certification.

6. Vehicle Tracking Control. The Construction SWMP shall clearly describe and locate all practices implemented at the site to control potential sediment discharges from vehicle tracking. Practices must be implemented for all areas of potential vehicle tracking and can include: minimizing site access, street sweeping or scraping, tracking pads, graveled parking areas, requiring that vehicles stay on paved areas on-site, wash racks, contractor education, and/or sediment control BMPs, etc.

7. Waste Management and Disposal, Including Concrete Washout.

i. The Construction SWMP shall clearly describe and locate the practices implemented at the site to control stormwater pollution from all construction site wastes (liquid and solid), including concrete washout activities. Designated concrete washouts shall be identified in the field with signage.

ii. The practices used for concrete washout must ensure that these activities do not result in the contribution of pollutants associated with the washing activity to stormwater runoff.

iii. Section 1503.1 (b) authorizes the conditional discharge of concrete washout water to the ground. The Construction SWMP shall clearly describe and locate the practices to be used that will ensure that no washout water from concrete washout activities is discharged from the site as surface runoff or to surface waters.

iv. Because pH is a pollutant of concern for washout activities, the soil must have adequate buffering capacity to result in protection of the groundwater standard, or a liner/containment must be used. The following management practices are recommended to prevent an impact from unlined pits to groundwater:

a. The use of the washout site should be temporary (less than 1 year), and
b. The washout site should not be located in an area where shallow groundwater may be present, such as near natural drainages, springs, or wetlands.

8. Groundwater and Stormwater Dewatering.

   i. The Construction SWMP shall clearly describe and locate the practices implemented at the site to control stormwater pollution from the dewatering of groundwater or stormwater from excavations, wells, etc.

   ii. Section 1503.1 authorizes the conditional discharge of construction dewatering to the ground. For any construction dewatering of groundwater not authorized under a separate CDPS discharge permit, the Construction SWMP shall clearly describe and locate the practices to be used that will ensure that no groundwater from construction dewatering is discharged from the site as surface runoff or to surface waters.

### 1507.4 Final Stabilization and Long-Term Stormwater Management

The Construction SWMP shall clearly describe the practices used to achieve final stabilization of all disturbed areas at the site, and any planned practices to control pollutants in stormwater discharges that will occur after construction operations have been completed at the site. The designs of post-construction BMPs, which are described in Chapter 1600 - Post Construction Stormwater Management, are included with the Final Drainage Report (see Section 303 *Final Drainage Report)*.

Final stabilization practices for obtaining a vegetative cover shall include, as appropriate: seed mix selection and application methods; soil preparation and amendments; soil stabilization practices (e.g., crimped straw, hydro mulch or rolled erosion control products); and appropriate sediment control BMPs as needed until final stabilization is achieved; etc.

Final stabilization is reached when all ground surface disturbing activities at the site have been completed, and uniform vegetative cover has been established on all remaining pervious soil surfaces with an individual plant density of at least 70 percent of pre-disturbance levels and equivalent permanent, physical erosion reduction methods (buildings, asphalt, concrete, etc.) have been employed. This does not mean that 70 percent of total surface area of the site has been converted to impervious surfaces.

### 1507.5 Inspection and Maintenance

The plan shall include a description of procedures to inspect and maintain in good and effective operating condition the vegetation, erosion and sediment control measures and other protective measures identified in the Construction SWMP. Section 1506 describes inspection requirements.

### 1507.6 Certification

The Construction Stormwater Management Plan shall include the following certification.

"I hereby certify that the Construction Stormwater Management Plan for the construction of (name of Development) was prepared by me (or under my direct supervision) in accordance with the provisions of the *Stormwater Management*

*Manual* for the owner/operators of (name of Development). I understand that the (Identify permitting agency) does not and will not assume liability for facilities designed by others."

______________________________ ____________________

Erosion Control Specialist Date

### 1507.7 Estimated BMP Costs

The plan shall include the estimated total cost (purchase, installation and maintenance) of the required temporary soil erosion and sediment control measures to determine surety or bonding requirements for the proposed plan. Provide quantities and unit costs.

## 1508 PERMIT TRANSFER REQUIREMENTS

When a portion of a permitted site is sold to a new owner or operator, a permit certification must be in place that is held by the new owner/operator of the sold area. This may be accomplished in one of the following ways:

a. **Coverage Under the Existing Certification**

Activities at the sold area may be covered under an existing permit for the project if the current permittee has contractual responsibility and operational control to address the impacts that construction activities at the sold area may have on stormwater runoff (including implementation of the Construction SWMP for the sold area). Therefore, a legally binding agreement must exist assigning this responsibility to the permit holder on behalf of the new owner/operator for the sold area. It is necessary to notify the local jurisdiction in such case. Documentation of the agreement must be available upon request and the Construction SWMP must be maintained to include all activities covered by the permit.

Example: Developer Dan sells a lot to Builder Bob. Developer Dan is currently covered under a permit that covers a larger area, which includes the sold lot. Developer Dan and Builder Bob may enter into a contract that assigns the responsibility for permit coverage and stormwater management to Developer Dan for Builder Bob's lot. Developer Dan is also responsible for making sure his Construction SWMP includes the activities on the lot. Developer Dan's permit will continue to cover construction activities on Builder Bob's lot.

b. **New Certification Issued – Reassignment**

A new permit may be issued to the new owner/operator of the sold area. The existing permittee and the new owner/operator must complete the reassignment form to remove the sold area from the existing permit and cover it under a new permit issued to the new owner/operator of the sold area. Both entities must have their Construction SWMP in place that accurately reflects their current covered areas and activities.

Example: Developer Dan sells a lot to Builder Bob. Developer Dan is currently covered under a permit that covers a larger area, which includes the sold lot. For this example, Developer Dan and Builder Bob must jointly submit the reassignment form. Builder Bob will be issued a new permit for his lot and the lot will be removed from Developer Dan's permit coverage. Prior to submittal of the reassignment form, Developer Dan must revise his Construction SWMP to reflect the changes in his covered area and activities, and Builder Bob must develop his own Construction SWMP to cover the area and activities for which he will obtain coverage. The reassignment form and associated instructions can be found in Section 1700 *Standard Forms*. The form and instructions can also be found at the CDPHE website http://www.cdphe.state.co.us/wq/PermitsUnit/stormwater/construction.html under the "construction reassignment form" link.

c. **Amend Existing Permit Certifications**

In some cases, both parties (the original owner/operator and the new owner/operator of an area undergoing transfer of ownership) will already be permit holders for their portions of the overall project (i.e., at least two permits are issued for the project and cover both the party wishing to reassign coverage and the party wishing to accept coverage). When an additional area is transferred between the two parties, the permittees may amend their permits instead of completing the reassignment form. Both parties must separately provide a letter to the local jurisdiction that includes the revised information, removing the applicable area(s) from the original owner/operator's permit certification, and adding the area(s) to the new owner/operator's permit certification. The requests must cite both permit numbers. (Note: this request may be submitted jointly if it is signed by both entities.) This option will likely be used in cases where a developer and an owner/operator have already submitted a reassignment form, as discussed in part b. above, where an initial transfer of lots has occurred, and then additional lots are transferred at a later date. Both entities must have a Construction SWMP in place that accurately reflects their current covered areas and activities.

Example: Developer Dan sells a lot to Builder Bob. Developer Dan is currently covered under a permit that covers a larger area, which includes the sold lot. In addition, Builder Bob also holds a permit for other portions of the development which he already owns, and Builder Bob wishes to cover his new lot under this permit. Developer Dan submits a request to remove the lot from his permit and provides Builder Bob's permit number that the lot will now be under which that lot will be covered. Builder Bob also submits a request to modify his permit to add the lot, and provides Developer Dan's permit number under which the lot was previously covered. Developer Dan and Builder Bob must revise their Construction SWMP to reflect the changes in their covered area and activities.

d. **Sale of Residence to Homeowner**

For residential construction only, when a residential lot has been conveyed to a homeowner and all criteria in paragraphs 1 through 5 below are met, coverage under this permit is no longer required and the conveyed lot may be removed from coverage under the permittee's certification. At such time, the permittee is no longer responsible for meeting the terms and conditions of this permit for the conveyed lot, including the requirement to transfer or reassign permit coverage. The permittee remains responsible for inactivation of the original certification.

1. The lot has been sold to the homeowner(s) for private residential use;
2. The lot is less than 1.0 acres of disturbed area;
3. All construction activity conducted by the permittee on the lot is completed;
4. A certificate of occupancy (or equivalent) has been awarded to the home owner, and
5. The Construction SWMP has been amended to indicate the lot is no longer covered by permit.

Lots not meeting all of the above criteria require continued permit coverage. However, this permit coverage may be transferred or reassigned to a new owner or operator as described in Paragraphs A through C above.

## 1509 CHECKLIST

A checklist for the Construction SWMP is provided in **Table 1501a & b**. This checklist contains recommended report outline and contents for the Construction SWMP. A copy of the completed checklist shall be bound with the Construction SWMP.

Applicant is to identify with a checkmark ("✔") if information is provided with the appropriate submittal. If applicant believes information is not required, indicate with "n/a". The submittal will be reviewed to determine if information is required and whether information must be submitted.

## CONSTRUCTION STORMWATER MANAGEMENT PLAN CHECKLIST

Instructions: Applicant to identify with a "check-mark" if information is provided. If applicant believes information is not required, indicate with "n/a" and provide a brief explanation. The local jurisdiction will determine if information labeled "n/a" is required and whether information must be submitted.

1) A site description including:

__ A description of the nature of the construction activity.
__ The proposed sequence for major activities.
__ Estimates of the total area of the site and the area of the site that is expected to undergo clearing, excavation or grading.
__ A summary of any existing data used in the development of the site construction plans or Construction SWMP that describe the soil or existing potential for soil erosion.
__ A description of the existing vegetation at the site and an estimate of the percent vegetative ground cover. Photos of the existing vegetation may be useful to determine if final stabilization has occurred.
__ The location and description of all potential pollution sources, including ground surface disturbing activities, vehicle fueling, storage of fertilizers or chemicals, etc.
__ The location and description of any anticipated non-stormwater components of the discharge, such as uncontaminated springs, landscape irrigation return flow, and construction dewatering.
__ The name of the receiving water(s) and the size, type and location of any outfall(s). If the discharge is to a municipal separate storm sewer, the name of that system, the location of the storm sewer discharge, and the ultimate receiving water(s).

2) A site map with:

__ Construction site boundaries
__ All areas of soil disturbance
__ Areas of cut and fill
__ Areas used for storage of building materials, soils or wastes
__ Location of any dedicated asphalt or concrete batch plants
__ Location of BMPs needed to address pollutant sources identified in Section 1507.3 (b)
__ Location of springs, streams, wetlands and other surface waters
__ Stormwater discharge locations

3) A description of all control measures that reflect the potential pollutants sources identified at the facility. The description of stormwater management controls shall address the following components, at a minimum:

__ Construction SWMP Administrator
Identification of Potential Pollutant Sources including:
- __ all disturbed and stored soils
- __ vehicle tracking of sediments
- __ management of contaminated soils
- __ loading and unloading operations
- __ outdoor storage activities (building materials, fertilizers, chemicals, etc.)
- __ vehicle and equipment maintenance and fueling
- __ significant dust or particulate generating processes
- __ routine maintenance activities involving fertilizers, pesticides, detergents, fuels, solvents, oils, etc.
- __ on-site waste management practices (waste piles, liquid wastes, dumpsters, etc.)
- __ concrete truck/equipment washing, including the concrete truck chute and associated fixtures and equipment
- __ dedicated asphalt and concrete batch plants
- __ non-industrial waste sources such as worker trash and portable toilets
- __ other areas or procedures where potential spills can occur

| Revision | Date |
|---|---|
| ORIGINAL ISSUE | 3/27/06 |
| REVISED | 2/20/08 |
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# STORMWATER MANAGEMENT MANUAL

## CONSTRUCTION STORMWATER MANAGEMENT PLAN CHECKLIST
(continued)

Instructions: Applicant to identify with a "check-mark" if information is provided. If applicant believes information is not required, indicate with "n/a" and provide a brief explanation. The local jurisdiction will determine if information labeled "n/a" is required and whether information must be submitted.

4) A description of all Best Management Practices (BMPs) for Stormwater Pollution Prevention including:

__ Structural Practices for Erosion and Sediment Control
__ Non-Structural Practices for Erosion and Sediment Control
__ Phased BMP Implementation
__ Materials Handling and Spill Prevention
__ Dedicated Concrete or Asphalt Batch Plants
__ Vehicle Tracking Control
__ Waste Management and Disposal, Including Concrete Washout
__ Groundwater and Stormwater Dewatering
__ Construction SWMP Administrator
__ Construction SWMP Administrator
__ Construction SWMP Administrator

5) The Construction SWMP shall clearly describe the practices used to achieve final stabilization of all disturbed areas at the site

6) The plan shall include a description of procedures to inspect and maintain in good and effective operating condition the vegetation, erosion and sediment control measures.

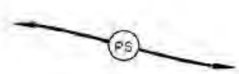
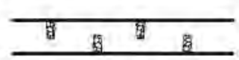
7) The Construction Stormwater Management Plan shall be certified.

8) The plan shall include the estimated total cost (purchase, installation and maintenance) of the required temporary soil erosion and sediment control measures to determine surety or bonding requirements for the proposed plan.

| Revision | Date |
|---|---|
| ORIGINAL ISSUE | 3/27/06 |
| REVISED | 2/20/08 |
| | |

# STORMWATER MANAGEMENT MANUAL

## EROSION CONTROL PLAN SYMBOLS

| TITLE | KEY | SYMBOL |
|---|---|---|
| CHECK DAM | CD |  |
| CONSTRUCTION ROAD STABILIZATION | CRS |  |
| CURB SOCK INLET PROTECTION | CS |  |
| TEMPORARY DIVERSION DIKE | DD |  |
| TEMPORARY CHANNEL DIVERSION | DV |  |
| STORM DRAIN INLET PROTECTION | IP |  |
| MULCHING | MU |  |
| OUTLET PROTECTION | OP |  |
| PAVED FLUME | PF |  |
| PERMANENT SEEDING | PS |  |
| ROUGH CUT STREET CONTROL | RCS |  |

| Revision | Date |
|---|---|
| ORIGINAL ISSUE | 3/27/06 |
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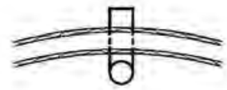
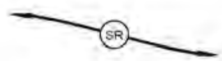
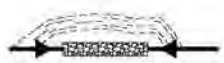
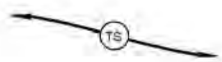
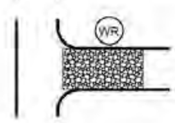
REFERENCE:
*UDFCD. URBAN STORM DRAINAGE CRITERIA MANUAL*
*VOLUME 3 BEST MANAGEMENT PRACTICES*

**FIGURE 1501a**

# EROSION CONTROL PLAN SYMBOLS (cont'd)

| TITLE | KEY | SYMBOL |
|---|---|---|
| SEDIMENT BASIN | SB |  |
| TEMPORARY STREAM CROSSING | SC |  |
| SILT FENCE | SF |  |
| SURFACE ROUGHENING | SR |  |
| SEDIMENT TRAP | ST |  |
| STRAW BALE BARRIER | STB |  |
| TEMPORARY SEEDING | TS |  |
| TEMPORARY SLOPE DRAIN | TSD |  |
| VEHICLE TRACKING CONTROL | VTC |  |
| VEHICLE TRACKING CONTROL WITH WASH RACK | WR |  |

| Revision | Date |
|---|---|
| ORIGINAL ISSUE | 3/27/06 |
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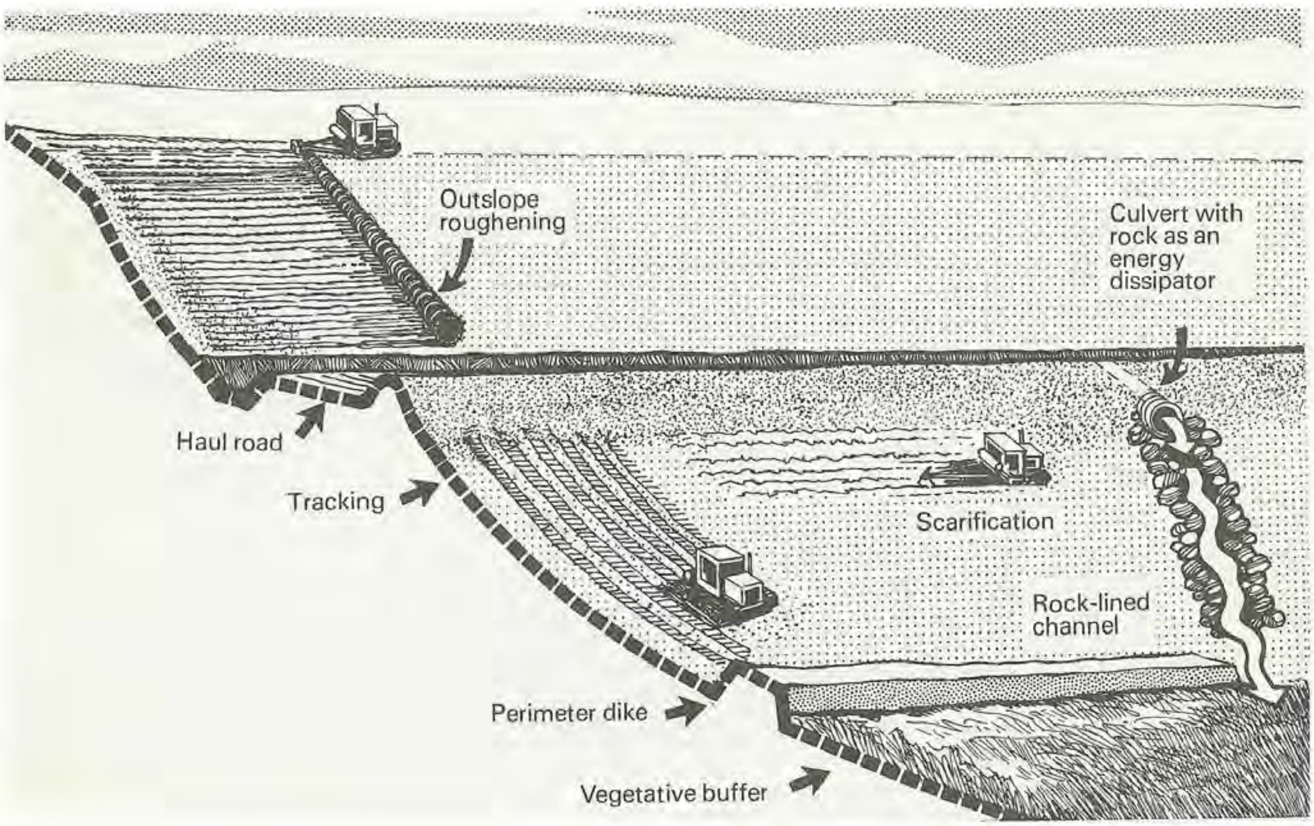
**FIGURE 1501b**

# STORMWATER MANAGEMENT MANUAL

## SURFACE ROUGHENING




Notes:

1. Roughen soil surface with depressions from 2- to 4-inches deep and spaced 4- to 6-inches apart' using chisel or ripping equipment. For slopes 2:1 or greater, use track dozer working perpendicular to slope.
2. Roughen slope after final grading. Roughening of ridges and depressions shall follow along slope contours.
3. Additional treatments may be necessary.

| Revision | Date |
|---|---|
| ORIGINAL ISSUE | 3/27/06 |
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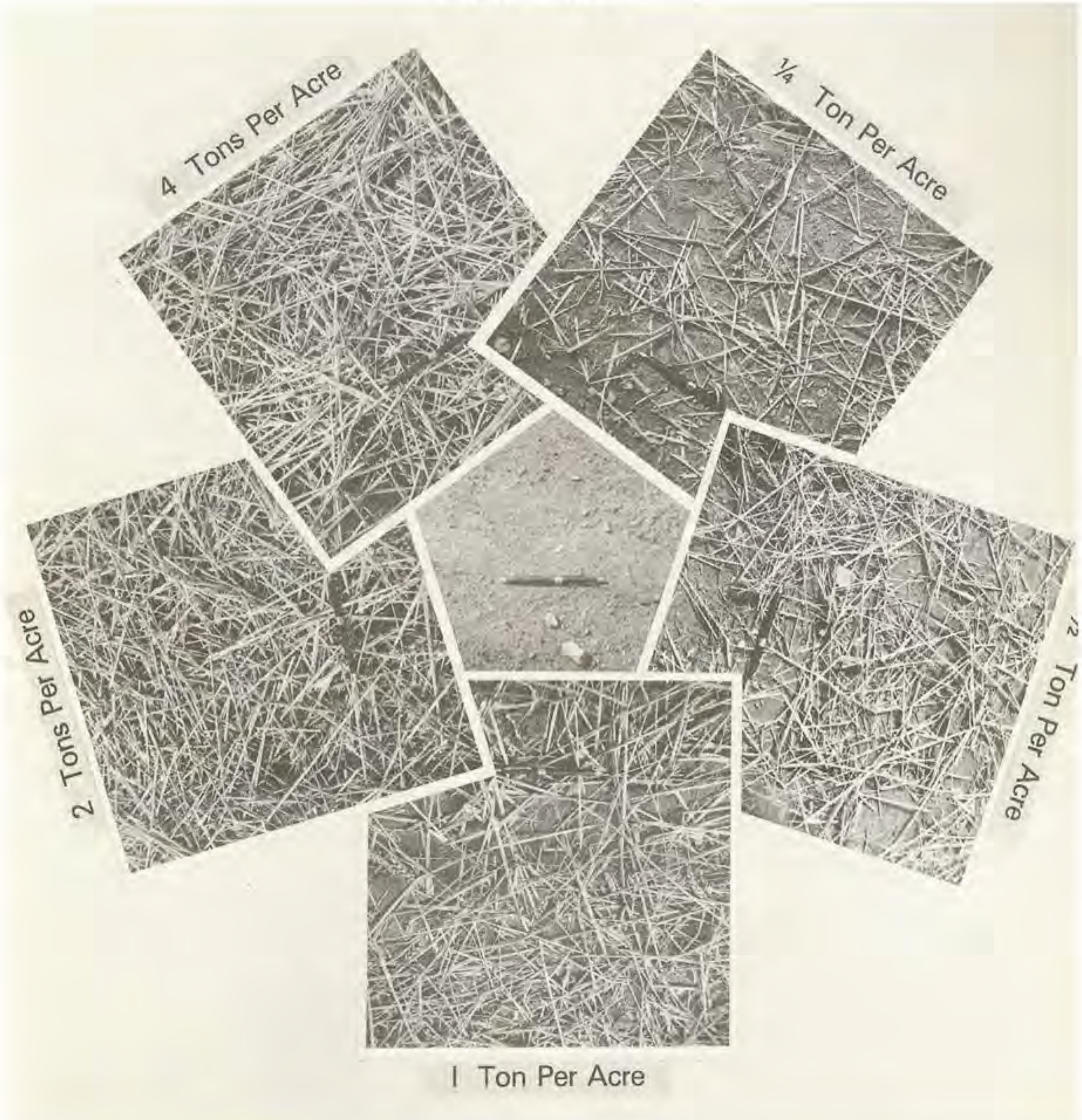
REFERENCE: *EPA 1976. EROSION AND SEDIMENT CONTROL SURFACE MINING IN THE EASTERN U.S. PLANNING*

**FIGURE 1502**

# STORMWATER MANAGEMENT MANUAL

## MULCHING




1. All disturbed areas must be mulched or seeded and mulched within 14–days after final grade is reached on any portion of the site not otherwise permanently stabilized. All mulching and seeding must meet local jurisdiction requirements.
2. Mulch shall be clean, weed– and seed–free, long stemmed grass hay or cereal grain straw with 50% or more by weight being ten inches or greater.
3. Mulch shall be anchored by mechanical crimping, with tackifier, or with nets.
4. Hydraulic mulching limited to areas where slopes are steeper than 3:1 or access is too difficult.
5. Mulch shall be applied at a 1–ton per acre or greater rate.

| Revision | Date |
|---|---|
| ORIGINAL ISSUE | 3/27/06 |
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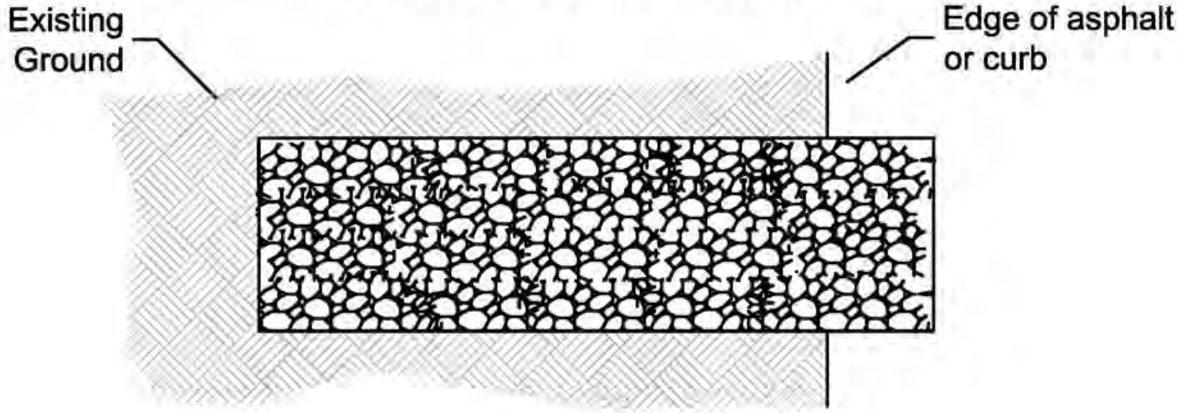
REFERENCE: *EPA 1976. EROSION AND SEDIMENT CONTROL SURFACE MINING IN THE EASTERN U.S. PLANNING*

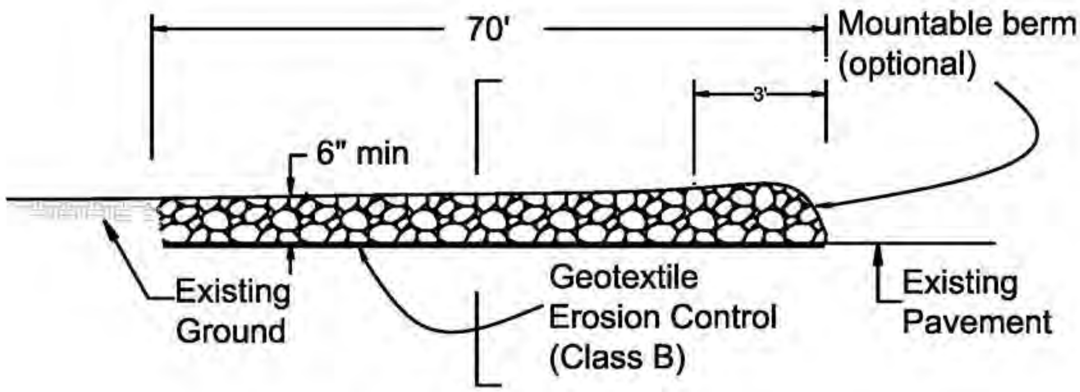
**FIGURE 1503**

# STORMWATER MANAGEMENT MANUAL

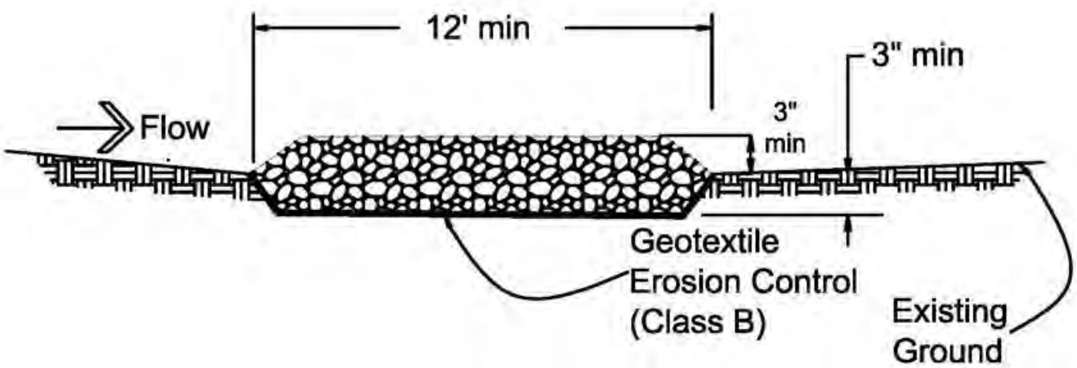
## VEHICLE TRACKING CONTROL




PLAN




SIDE ELEVATION




SECTION

NOTES:

1. VEHICLE TRACKING REQUIRED WHEN LAND DISTURBANCE 2-ACRES ORE GREATER.
2. USE 2- TO 3-INCH COARSE AGGREGATE MATERIAL.
3. GEOTEXTILE SHALL CONFORM TO CDOT 712.08, CLASS B.
4. SEDIMENT, DIRT OR MUD TRACKED ONTO PUBLIC STREETS SHALL BE REMOVED MECHANICALLY BY SWEEPING, SCOOPING AND SHOVELING.

| Revision | Date |
|---|---|
| ORIGINAL ISSUE | 3/27/06 |
| | |
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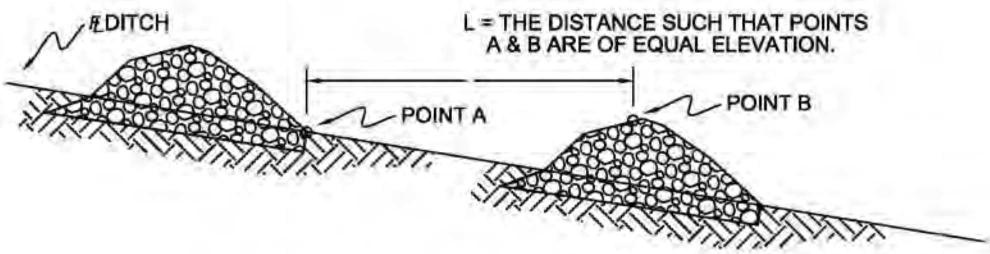
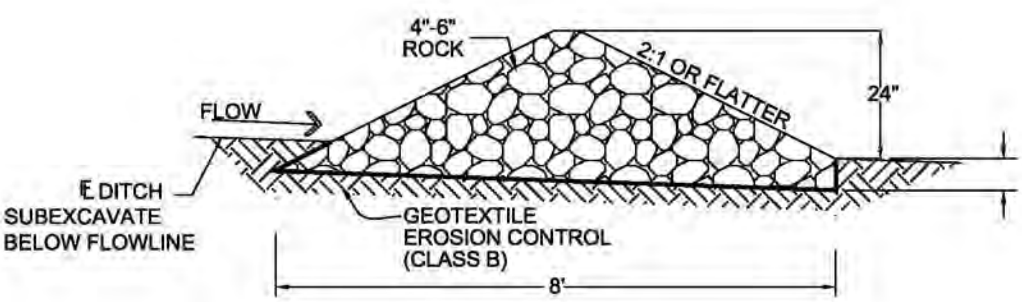
REFERENCE: *CDOT 2000. STANDARD PLANS M & S STANDARDS M-208-1, SHEET 5 OF 7*

**FIGURE 1504**

# CHECK DAM (RIPRAP)




PROFILE VIEW ALONG DITCH FLOWLINE




PROFILE

RIPRAP CHECK DAM

| Revision | Date |
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| ORIGINAL ISSUE | 3/27/06 |
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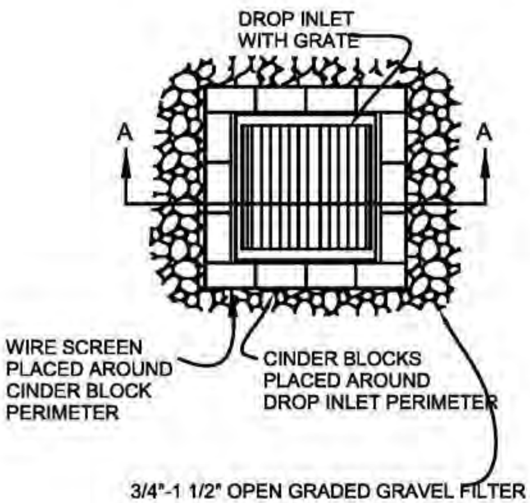
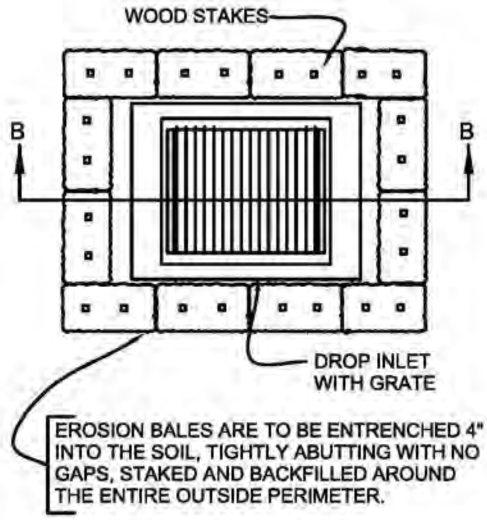
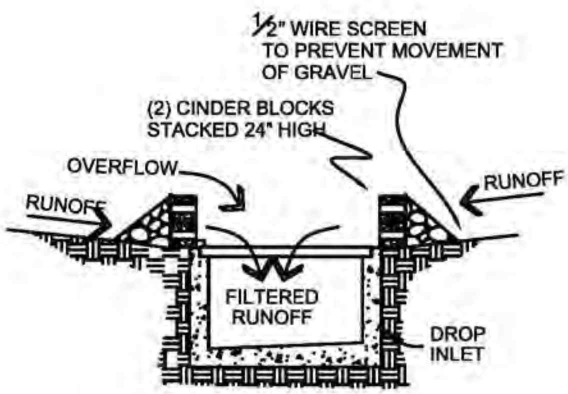
# INLET PROTECTION




PLAN VIEW

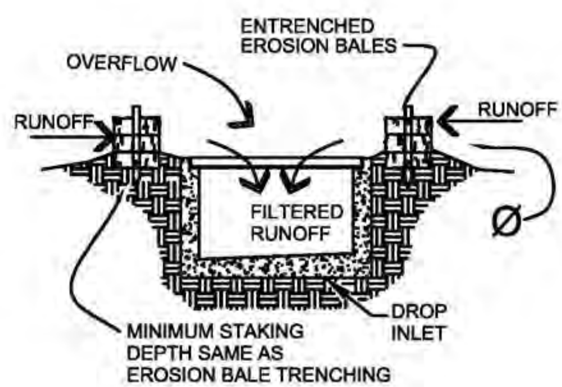



PLAN VIEW




CROSS SECTION A-A

DROP INLET BLOCK AND GRAVEL FILTER




CROSS SECTION B-B

DROP INLET EROSION BALE FILTER

Ø Sediment removal shall be performed continuously for proper function

NOTE:
VISUAL WARNING SHALL BE PROVIDED WHERE INLET IS OBSTRUCTING TRAFFIC (VEHICLE, PEDESTRIAN, BICYCLE, ETC.)

| Revision | Date |
|---|---|
| ORIGINAL ISSUE | 3/27/06 |
| | |
| | |

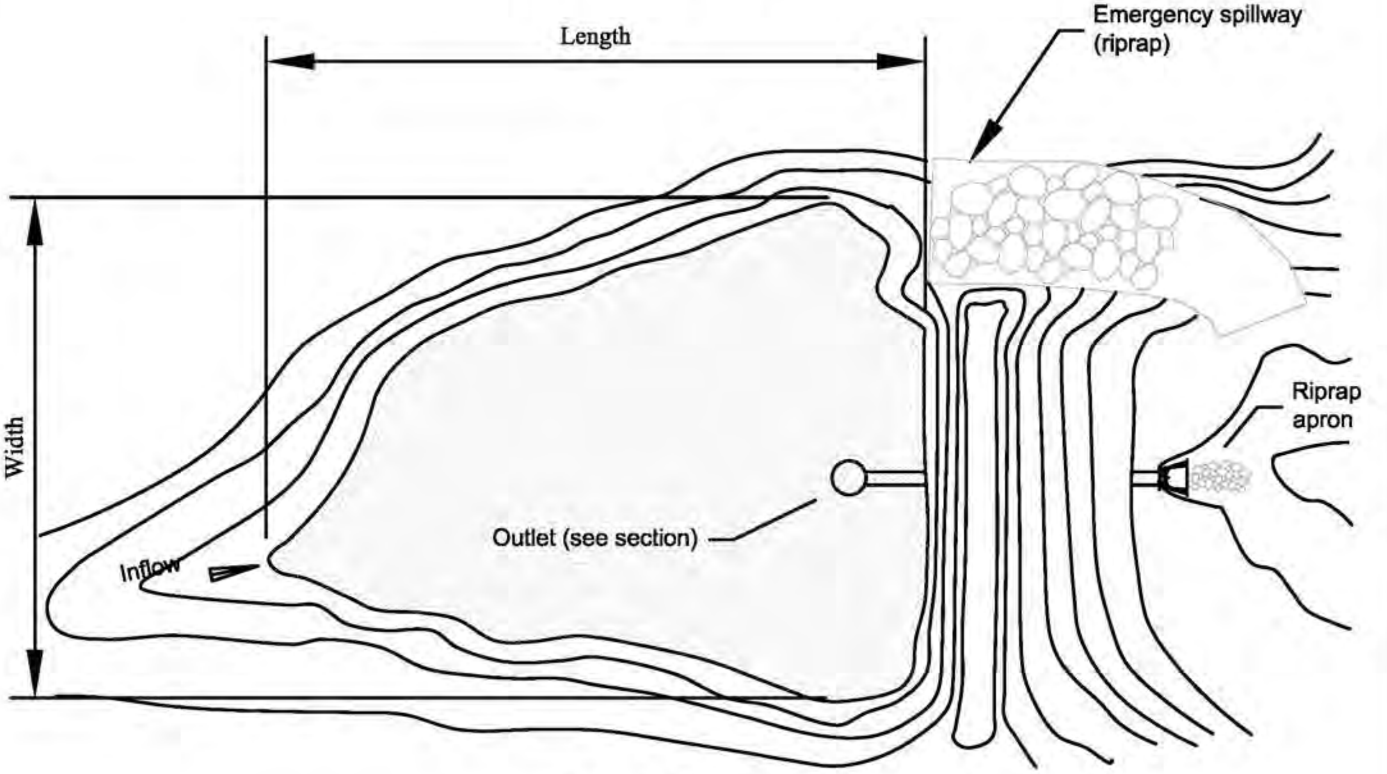
WRC ENGINEERING, INC.

REFERENCE: *CDOT 2000. STANDARD PLANS M & S STANDARDS M-208-1, SHEET 3 OF 7*

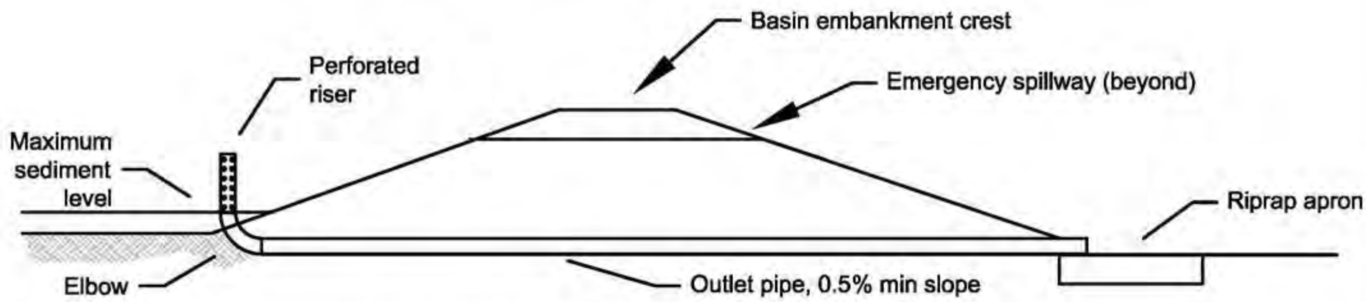
**FIGURE 1506**

# STORMWATER MANAGEMENT MANUAL

## SEDIMENT BASIN




PLAN




SECTION THROUGH OUTLET

Notes:

1. Required volume to crest of emergency spillway is 1,800 cubic feet per acre of drainage area.
2. Emergency spillway to pass 100-year or greater flood peak.
3. Emergency spillway must not be constructed over fill material. Protect spillway with riprap.
4. Basin length to width ratio, L/W to be greater than 2.0
5. Remove sediment when level reaches the invert of the lowest orifice at 50% of the storage volume
6. Outlet to be minimum 8" diameter PVC. Riser to include perforations to drain volume below emergency spillway in 40-hours.

| Revision | Date |
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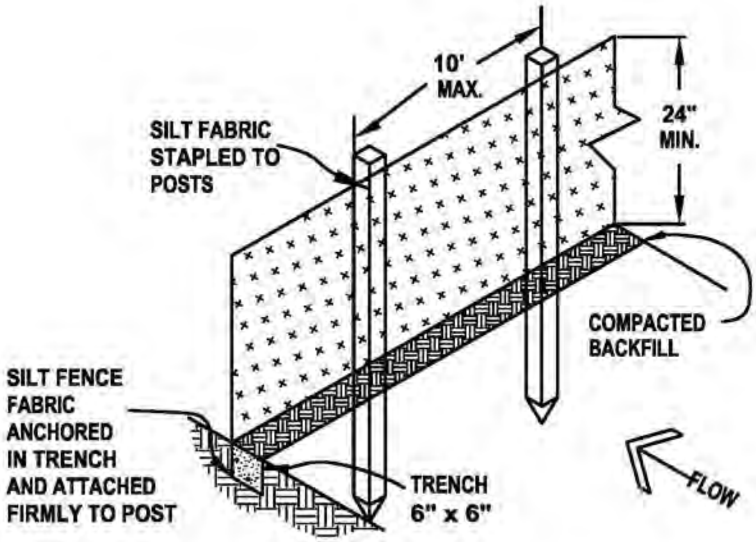
| ORIGINAL ISSUE | 3/27/06 |
| | |
| | |

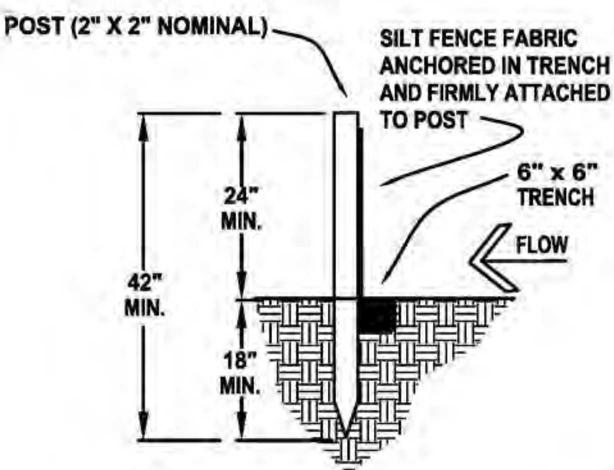
WRC ENGINEERING, INC

REFERENCE:
*UDFCD. URBAN STORM DRAINAGE CRITERIA MANUAL VOLUME 3 BEST MANAGEMENT PRACTICES*

**FIGURE 1507**

# SILT FENCE




Ø SEDIMENT REMOVAL SHALL BE PERFORMED CONTINUOUSLY FOR PROPER FUNCTION.

Notes:
1. Silt fence should not be used in areas of concentrated flow.
2. Where stormwater is within a swale and a silt fence is used to trap sediment, the silt fence is to be reinforced such as with straw bales, or a double silt fence arrangement.

| Revision | Date |
|---|---|
| ORIGINAL ISSUE | 3/27/06 |
| | |
| | |

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REFERENCE: CDOT 2000. STANDARD PLANS M & S STANDARDS M-208-1, SHEET 2 OF 7

**FIGURE 1508**

# SECTION 1600

## POST-CONSTRUCTION STORMWATER MANAGEMENT

# SECTION 1600
## POST-CONSTRUCTION STORMWATER MANAGEMENT

## TABLE OF CONTENTS

# LIST OF FIGURES

# SECTION 1600
# POST-CONSTRUCTION STORMWATER MANAGMENT

## 1601 INTRODUCTION

All persons engaged in new development and redevelopment of property that disturbs 1.0 acres or more shall implement post-construction BMPs to control the discharge of pollutants after construction is completed, in accordance with the provisions of the Manual. Post-construction BMPs, which includes both structural and non-structural measures, shall be identified in the Final Drainage Report (see Section 304 *Final Drainage Report*). For requirements during construction, see Section 1500, *Construction Site Stormwater Runoff Control.*

## 1602 STANDARDS FOR BEST MANAGEMENT PRACTICES (BMPs)

### 1602.1 Minimum Structural BMPs Requirements

BMPs for significant development and redevelopment capture and detain stormwater in best management practices, called volume capture devices. These BMPs are designed to capture and temporarily detain a large percentage of runoff events and runoff volumes then release the runoff over a 6- to 40-hour period, which allows sufficient time for settling of pollutants. Volume capture BMPs are widely used by communities throughout the United States and are considered a standard of practice.

The following BMPs meet the minimum water quality capture volume (WQCV) requirements for onsite or regional stormwater quality facilities.

| | |
|---|---|
| **Figure 1601** | Porous Pavement Detention (with under-drain only) |
| **Figure 1602** | Porous Landscape Detention (with under-drain only) |
| **Figure 1603** | Extended Detention Basin |
| **Figure 1604** | Sand Filter Extended Detention Basin |
| **Figure 1605** | Constructed Wetland Basin |
| **Figure 1606** | Retention Basin |

Examples of these BMPs can be found in:

a. *Urban Drainage & Flood Control District (UDFCD) 1999.* Urban Storm Drainage Criteria Manual (USDCM), Volume 3 - Best Management Practices. *This document is available at: http://www.udfcd.org/usdcm/vol3.htm.*

b. *Water Environment Federation (WEF) and American Society of Civil Engineers (ASCE) 1998.* Urban Runoff Quality Management. *WEF Manual No. 23, ASCE Manual No. 87.*

c. *EPA 2002. Consideration in the Design of Treatment Best Management Practices (BMP) to Improve Water Quality. EPA 600 R-03/103. This document is available in pdf form at the following website http://www.epa.gov/ORD/NRMRL/pubs/600r03103/600r03103chp5.pdf*

Portions of the USDCM Volume 3 that are required to design an extended detention basin to include water quality requirements are in Appendix B to this manual. Each designer should note that all UDFCD documents are updated continuously, and it is the designer's responsibility to check for these updates to ensure use of the most up-to-date data. Each designer should also note that some equations in this Manual have been modified from those in Volume 3 to be specific to Mesa County. Equations in this Manual shall govern.

Use of alternate BMPs not specified above is subject to approval by the local jurisdiction. **Figure 1611** offers a schematic for determining the best BMP options for individual sites.

When new development is served by a regional stormwater quality facility designed in accordance with this Section *and* stormwater from the development is discharged to an open conveyance system before reaching the regional facility, onsite BMPs must also be provided. If the new development discharges to the regional stormwater quality facility through a storm drain, then this additional requirement does not apply.

### 1602.2 Aesthetic Requirements

When storm runoff will be treated by an extended detention basin (dry), a retention pond with a permanent pool (wet), or constructed wetlands, the facility shall be designed with the intent of improving appearance in accordance with requirements under Section 1400 *Stormwater Detention*.

### 1602.3 Proprietary BMPs

Proprietary systems which meet the requirements of this Section and have continued inspection/operation and maintenance guarantees in place are allowed provided:

a. Clear evidence must be provided why WQCV cannot be provided on the ground surface using a volume-capture BMPs, and why the use of a subsurface proprietary device is the best choice for the site, considering factors such as initial installation, maintenance, and ability to assure long-term function.

b. Proprietary BMPs are limited to commercial, new development and redevelopment sites with an area less than 5-acres tributary to the proprietary BMP. If greater than 5 acres, a variance or deviation may be requested as described in Section 207.

c. The proprietary device(s) must provide volume control based on a drain time of no less than six hours or be a supplement to other BMPs which meet these requirements.

d. A binding, long-term inspection/operation and maintenance plan, including monitoring and reporting, and demonstration of adequate funding for such maintenance, must be provided.

### 1602.4 Considerations in BMPs Selection

Because of unique conditions in Mesa County, there are important considerations in the selection and design of BMPs. For example:

a. **Vegetation**: Vegetation for stabilization is preferred over rock or other "hard-scape". Native vegetation and grasses that require little water will satisfy this requirement. However, BMPs that rely on vegetation that requires more water than nature provides must include a supplemental irrigation source to maintain adequate plant growth. BMPs such as grass swales and buffers can be supplemented through potable sources and landscape irrigation systems. However, wetlands, wetland ponds, and extended detention basins also require supplemental water sources that include watershed runoff and/or larger-scale, irrigation-water sources to maintain vegetation.

b. **Infiltration**: Due to the nature of the soils in Mesa County, BMPs that rely specifically on infiltration into the groundwater are discouraged, since some pollutants in the soils can also be leached into the groundwater and eventually to surface waters. However, specific requirements for some BMPs, such as wet-ponds, wetlands, and filtration systems which may result in infiltration, have been modified to require a liner and collection system to minimize or prevent infiltration.

c. **Maintenance Agreement and Guarantee**: All drainage and water quality facilities must be designed to facilitate maintenance (see Section 403.10 *Drainage Facilities Maintenance*) and maintenance agreements shall be entered into between a landowner and the local jurisdiction.

   Additional guidance on design, site selection, maintenance and other considerations can be found in:

   i. *Urban Drainage & Flood Control District (UDFCD) 1999.* Urban Storm Drainage Criteria Manual, Volume 3 - Best Management Practices. This document is available in pdf form at the following website: *http://www.udfcd.org/usdcm/vol3.htm*.

   ii. City and County of Denver 2004. *Water Quality Management Plan 2004.* This document is available in pdf form at the following website: http://www.denvergov.org/WMDWaterQuality/77516706template3jump.asp See Chapter 6 Stormwater Quality BMP Implementation Guidelines, Part 4 - BMP Fact sheets.

### 1602.5 Local Modifications to Minimum Structural BMPs

Structural BMPs which typically employ micro-pools, namely extended detention basins, may be modified to omit the micro-pool should the local jurisdiction be concerned about negative side-effects. Micro-pools were initially added to the standard design for extended detention basins because 1) they seemed to develop whether or not they were intended and 2) some data suggested pollutant removal, particularly of nutrients, would improve.

However, the West Nile Virus outbreak in 2003, several outbreaks of algae growth during the drought of 2002, and the increase operation and maintenance requirements that come with micro-pools prompted many communities to forego their use completely. Given the very dry climate of Mesa County, micro-pools may be considered an optional feature of the extended detention basin (EDB) without compromising the standard of practice for stormwater quality control.

The use of a micro-pool may be considered more appropriate when the:

i. EDB has a more reliable source of water than solely stormwater runoff
ii. EDB serves an area greater than 160 acres and
iii. Permanent pool can be maintained as an aesthetic feature rather than a nuisance.

## 1603 FINAL DRAINAGE REPORT

A Final Drainage Report shall be prepared which includes a narrative, calculations and drawings showing location and sizes of permanent BMPs used to control the discharge of pollutants to the MS4 system after construction. Requirements for a Final Drainage Report are described in Section 300 *Drainage Planning Submittal Requirements*.

## 1604 WATER QUALITY CAPTURE VOLUME

Water quality capture volume (WQCV) requirements for structural BMPs can be determined using the following sources. For this Manual, equations developed by the UDFCD were used and adjusted for the Mesa County area.

a. Urban Drainage & Flood Control District (UDFCD) 2002. *Urban Storm Drainage Criteria Manual, Volume 3 - Best Management Practices*. This document can be obtained in pdf form at the following website http://www.udfcd.org/usdcm/vol3.htm.

*b.* EPA 2002. *Consideration in the Design of Treatment Best Management Practices (BMP) to Improve Water Quality. EPA 600 R-03/103.* This document can be obtained in pdf form at the following website *http://www.epa.gov/ORD/NRMRL/pubs/600r03103/600r03103chp5.pdf*

*c.* Water Environment Federation (WEF) and American Society of Civil Engineers (ASCE) 1998. *Urban Runoff Quality Management.* WEF Manual No. 23, ASCE Manual No. 87.

### 1604.1 Directly-Connected Impervious Area

The procedures described in Section 600 *Rainfall* and in Section 700 *Runoff* are intended for the design of drainages and flood control facilities that prevent damage to property and help protect human life. These procedures show that the depth of rainfall produced from a design storm can vary with location and that runoff is a function of total imperviousness.

Post-construction BMPs focus more on the smaller events that deliver frequent flow pulses and pollutant loads to receiving waters. The runoff volume for smaller events is especially sensitive to the impervious area that is hydraulically connected to the stormwater runoff system. The impervious portion of a watershed determines the runoff volume that needs to be used for the design of water quality facilities, and the percentage of impervious surface therefore becomes important in the design of structural BMPs.

The methodology for calculating basin imperviousness is presented in Section 700 *Runoff*. This procedure needs to be modified, however, when using the practice of minimizing directly-connected impervious areas in combination with extended detention basins, retention ponds, wetlands, and other practices depended on a design water quality capture volume. See UDFCD Urban Storm Drainage Criteria Manual (USDCM) Volume 3 Best Management Practices for calculation details.

### 1604.2 Water Quality Capture Volume

All structural BMPs recommended in this Manual are based on the 80th percentile storm event. Specific guidance for finding the needed WQCV for each BMPs types' design section can be found in the USDCM Volume 3 chapter on Structural BMP. This WQCV varies with the type of BMPs used and is based on the time it takes to fully drain the brim-full WQCV, which varies from 6-hour to 40-hours, depending on the BMPs. WQCV can be determined using the following relationships:

$$WQCV = K\{a(0.91*I^3 - 1.19*I^2 + 0.78*I)\}$$

Where:

WQCV is in watershed inches

K = Adjustment to equation for Mesa County area = $d_6/0.43$

a = Adjustment for BMPs drain time

| BMP Drain Time (hrs) | Value of "a" |
|---|---|
| 6 | 0.7 |
| 12 | 0.8 |
| 24 | 0.9 |
| 40 | 1.0 |

I = watershed impervious as a decimal

$d_6$ = Depth of average runoff producing storm

The average runoff producing storm value, $d_6$ = 0.28 inches, was obtained from Driscoll, et al. 1989. *Analysis of Storm Event Characteristics for Selected Rainfall Gages Throughout the United States.* The value 0.43 is for the Denver Metropolitan area, for which the original WQCV equation was derived (UDFCD 1999). Therefore, K = 0.28/0.43 = 0.65, which indicates that the Mesa County area requires approximately 35% less WQCV than along the Front Range.

Determine the required storage volume in acre-feet as follows:

Required storage = [WQCV/12]* Area

Where:

Required storage = Required storage volume, in acre-feet

Area = Tributary watershed area, in acres

Required storage volume shall be based on 120% of calculated WQCV to allow for sediment accumulation.

The local jurisdictions recognize that some sites cannot be graded in a reasonable manner so that the entire site drains to the detention basin. If at least 95% of the site drains to the detention pond, then local jurisdiction may accept the design, provided the WQCV is based on 100% of the site draining into the detention pond.

## 1605 NON-STRUCTURAL BMPs

### 1605.1 General Information

For existing and new development areas, nonstructural BMPs shall be applied where considered appropriate by the local jurisdiction based on applicable regulations or industry standards. Also, the requirement to prepare and implement permanent BMPs described in the Final Drainage Report in accordance with good drainage policy and criteria (i.e.: This Manual) constitute non-structural BMP's. Examples of non-structural BMPs, include the following controls or management of activities:

- Disposal of Household Waste and Toxics
- Pesticides, Herbicides and Fertilizer Use
- Illicit Discharge
- Spill Prevention and Response
- Preventative Maintenance
- Painting Operations
- Outside Materials Storage
- Outside Manufacturing
- Stormwater Pollution Prevention Education
- Vehicle Washing
- Above Ground Storage Tanks
- Good Housekeeping
- Loading and Unloading
- Fueling
- Exposure Minimization
- Storm Drain Stenciling

Management and control of these activities will occur over time as part of the various program requirements under the CDPS permit.

### 1605.2 Spill Prevention and Response

Presented below is a list of steps to minimize spill occurrence and response measures to address spills that do occur.

a. Develop procedures to prevent/mitigate spills to storm drain systems. Standardize reporting procedures, containment, storage, and disposal activities, documentation, and follow-up procedures.

b. Post "No Dumping" signs with a phone number for reporting illegal dumping and disposal.

c. Conduct routine cleaning, inspections, and maintenance

   i. Sweep and clean storage areas consistently at a designated frequency (e.g. weekly, monthly). DO NOT hose down areas to storm drains.
   ii. Place drip pans or absorbent materials beneath all mounted taps, and at all potential drip and spill locations during filling and unloading of tanks. Reuse, recycle, or properly dispose of any collected liquids or soiled absorbent materials.

iii. Check tanks (and any containment sumps) frequently for leaks and spills. Replace tanks that are leaking, corroded, or otherwise deteriorating with tanks in good condition. Collect all spilled liquids and properly dispose of them.
iv. Check for external corrosion of material containers, structural failures, spills and overfills due to operator error, failure of piping system, etc.
v. Inspect tank foundations, connections, coatings, and tank walls and piping system.

d. Properly store and handle chemical materials.

i. Designate a secure material storage area that is paved with Portland cement concrete, free of cracks and gaps, and impervious in order to contain leaks and spills.
ii. Do not store chemicals, drums, or bagged materials directly on the ground. Place these items in secondary containers.
iii. Keep chemicals in their original containers, if feasible.
iv. Keep containers well labeled according to their contents (e.g., solvent, gasoline).
v. Label hazardous substances regarding the potential hazard (corrosive, radioactive, flammable, explosive, and poisonous).
vi. Prominently display required labels on transported hazardous and toxic materials (per US DOT regulations).

e. Utilize secondary containment systems for liquid materials.

i. Surround storage tanks with a berm or other secondary containment system.
ii. Slope the area inside the berm to a drain.
iii. Drain liquids to the sanitary sewer if available. DO NOT discharge wash water to sanitary sewer until contacting the local authority to find out if pretreatment is required
iv. Pass accumulated stormwater in petroleum storage areas through an oil/water separator.
v. Use catch basin filtration inserts.

f. Protect materials stored outside from stormwater. Construct a berm around the perimeter of the material storage area to prevent uncontaminated stormwater from adjacent areas from coming in contact with runoff from the stored material.

g. Secure drums stored in an area where unauthorized persons may gain access to prevent accidental spillage, pilferage, or any unauthorized use.

h. Identify key spill response personnel.

i. Adopt a hazardous materials response plan, which includes a set of planned responses to hazardous materials emergencies that include:

i. Description of the facility, owner and address, activities and chemicals present
ii. Facility map
iii. Notification and evacuation procedures

iv. Cleanup instructions
v. Identification of responsible departments

j. Clean up leaks and spills immediately.

i. Place a stockpile of spill cleanup materials where they will be readily accessible (e.g. near storage and maintenance areas).
ii. Utilize dry cleaning methods to clean up spills to minimize the use of water. Use a rag for small spills, a damp mop for general cleanup, and absorbent material for larger spills. If the spilled material is hazardous, then used cleanup materials are also hazardous and must be sent to a certified laundry (rags) or disposed of as hazardous waste. Physical methods for the cleanup of dry chemicals include the use brooms, shovels, sweepers, or plows.
iii. Do not hose down or bury dry material spills. Sweep up the material and dispose of properly.
iv. Clean up chemical materials with absorbents, gels, and foams. Use adsorbent materials on small spills rather than hosing down the spill. Remove and dispose the adsorbent materials promptly.
v. For larger spills, a private spill cleanup company or hazmat team may be necessary.

k. Reporting

i. Report spills that pose an immediate threat to human health or the environment to local agencies
ii. Establish a system for tracking incidents. The system shall be designed to identify: types and quantities (in some cases) of wastes, patterns in time of occurrence (time of day/night, month, or year), mode of dumping (abandoned containers, "midnight dumping" from moving vehicles, direct dumping of materials, accidents/spills), and responsible parties

l. Training. *Educate employees about spill prevention and cleanup.*

i. Establish training that provides employees with the proper tools and knowledge to immediately begin cleaning up a spill.
ii. Educate employees on aboveground storage tank requirements.
iii. Train all employees upon hiring and on an annual basis.
iv. Train employees responsible for aboveground storage tanks and liquid transfers on adopted spill plan.

m. Stencil storm drains. Storm drain system signs act as highly visible source controls that are typically stenciled directly adjacent to storm drain inlets. Stencils shall read "No Dumping: Drains to River".

## 1606 <u>INSPECTION AND ENFORCEMENT</u>

### 1606.1 Mesa County

Post-construction BMPs shall be inspected by the engineer of record during construction, and Record Drawings of completed drainage facilities shall be submitted to the County along with certification that the facilities were constructed in accordance with approved plans (see Section 304 *Record Drawings and Acceptance*). The County will audit a portion of the Record Drawings to verify completeness.

Maintenance of all drainage facilities shall be performed by the owner, in accordance with Section 403.10, *Drainage Facilities Maintenance,* and required by maintenance agreement, Section 1602.4(c). The County requires that all facilities be inspected annually by a qualified erosion control specialist to verify maintenance activities and that inspection reports be provided to the County, who will audit 10-percent of all inspections. The County will provide necessary forms for inspection. Failure to properly maintain drainage facilities may result in enforcement actions, in accordance with the Land Development Code.

### 1606.2 City of Grand Junction

Inspection and enforcement of permanent BMPs will be performed in accordance with the stormwater pollution prevention Ordinance No. 3824, as amended. (Article VII, Chapter 16 of the Code of Ordinances).

a. Structural BMPs located on property shall be owned, operated, inspected and maintained by the owner(s) of the property and those persons responsible for the property on which the BMPs are located. The legal responsibility to maintain the BMPs shall be included in property owners' association (POA) declarations, incorporation articles, bylaws and/or development agreements for commercial sites. As a condition of approval of the BMPs, the owner and those persons responsible for the property shall also agree to maintain the BMPs to their design capacity unless or until the City shall relieve the property owner of that responsibility in writing. Drainage easements for inspection, installation, operation, repair and maintenance with required to access and inspect the BMP(s) and to perform routine maintenance as necessary to ensure proper functioning of the stormwater BMPs shall be dedicated to the City. The building of any structures on such easements is prohibited. Any agreement arising out of or under Article VII, Chapter 16 may be recorded as directed by the City Manager in the office of the Grand Junction City Clerk and/or the Mesa County land records.

b. The City may issue annual notices to POAs as well as to commercial and industrial sites to ensure inspections and maintenance of permanent BMPs are performed properly.

### 1606.3 Other Municipalities

Contact the City of Fruita, the Town of Palisade, and the Grand Junction Drainage District to determine inspection and enforcement requirements.

## 1607 BMPs FOR INDUSTRIAL AND COMMERCIAL ACTIVITIES

This section contains guidance for BMPs that control illicit discharges from industrial and commercial activities that come in contact with precipitation and result in contaminated stormwater discharges to the MS4. Owners of industrial or commercial activities that may come in contact with precipitation shall prepare a stormwater management plan (SWMP) approved by the local jurisdiction and implement best management practices, as described herein. Owners shall also be required to obtain a discharge permit from the Industrial Permits Unit of the CDPHE prior to receiving plan approval.

### 1607.1 Industrial and Commercial Activities

Federal regulations define an illicit discharge as "*...any discharge to an MS4 that is not composed entirely of stormwater...*" with some exceptions. These exceptions include discharges from CDPS-permitted industrial sources and discharges from fire-fighting activities and certain municipal activities. Illicit discharges are considered "illicit" because MS4s are not designed, or allowed by law, to accept, process, or discharge such non-stormwater wastes.

Typical illicit stormwater discharges may contain fuel, oil, solvents, degreasing agents, waste automotive fluids, acids, lubricants, caustic fluids, paint, paint thinners and solvents, dust from sanding and grinding, sand blasting residue, metal and wood shavings, industrial chemicals, herbicides, lawn products, hazardous wastes, sediment, dirt, toxic substances, trash and other pollutants. Example commercial and industrial activities where these illicit discharges may occur, *if allowed to come in contact with precipitation*, include:

- Manufacturing
- Material handling, including loading and unloading
- Vehicle and equipment maintenance, storage, parking, and washing.
- Waste containment.
- Painting.
- Material and product storage.
- Fueling areas

### 1607.2 Best Management Practices

BMPs to control illicit discharges from commercial and industrial activities can be grouped into source controls and treatment practices. Source controls are measures that prevent an illicit discharge from occurring (i.e.: spill prevention), while treatment practices remove pollutants from stormwater prior to leaving the site, which is the classical definition of structural practices.

#### 1607.2.1 Source Controls

Source controls include:

- Prevention - Practices that eliminate or reduce the amount and the chance of pollutants being generated on the site. This category includes good house keeping practices, such as employee training, signage, and labeling.

- Exposure Minimization - Practices that eliminate or minimize the chances of stormwater runoff coming in contact with pollutants. Simply covering the activity greatly reduces, if not eliminates the possibility of an illicit discharge.

Source control BMPs for industrial and commercial activities are listed on **Figure 1608**. This information can be found at the following webpage:

http://www.pweng.slco.org/pdf/imatrix.pdf. Then by clicking on the source control BMPs, a one-page fact sheet can be obtained which describes the application, the criteria, limitations, and targeted pollutants.

For additional information regarding industrial stormwater discharges, refer to UDFCD Urban Drainage Criteria Manual, Volume 3 Best Management Practices (USDCM). This document is available in pdf form at http://www.udfcd.org/usdcm/vol3.htm.

**1607.2.2 Treatment Practices**

Treatment practices involve cleaning up or recovering a substance after it has been released or spilled, which is the last-line of defense after the pollutant has been released on site. Treatment practices are considered a last resort and shall be used only when source control options are not practical. Section 1602 *Standards for Best Management Practices* describes the preferred treatment options.

For small sites where oil spills are probable, such as parking lots, industrial yards, etc., a treatment device referred to as an oil/water separator may be required. General information on oil/water separators can by found at the following: http://www.pweng.slco.org/pdf/imatrix.pdf, then click on "*OWS*" in the "*Treatment Control BMPs to Consider*" table.

Where oil/water separators are being considered, the following limitations are recommended:

a. The tributary area must be small (i.e. less than 1-acre) and essentially 100% impervious.

b. The site grading and drainage must be designed to minimize the area to the maximum extent practicable.

c. Oil/Water separators shall precede all other stormwater treatment BMPs and be located off-line from the stormwater conveyance or detention system.

d. Oil/Water separators shall be either the coalescing plate/media type, filter-type, or other type system with the following sizing requirements:

    i. Peak flow rate is based on the mean storm intensity of 0.044 in/hr with provisions for diversion of larger storm events.

    ii. Include a forebay to collect floatable and larger settleable solids with a surface area of not less than 20 square feet per 10,000 square feet of tributary area.

    iii. Remove droplets 90 microns and larger with a rise-rate of 0.033-feet per minute. Limited data from petroleum-storage-terminal discharges suggest that 80% of droplets are greater than 90-microns.

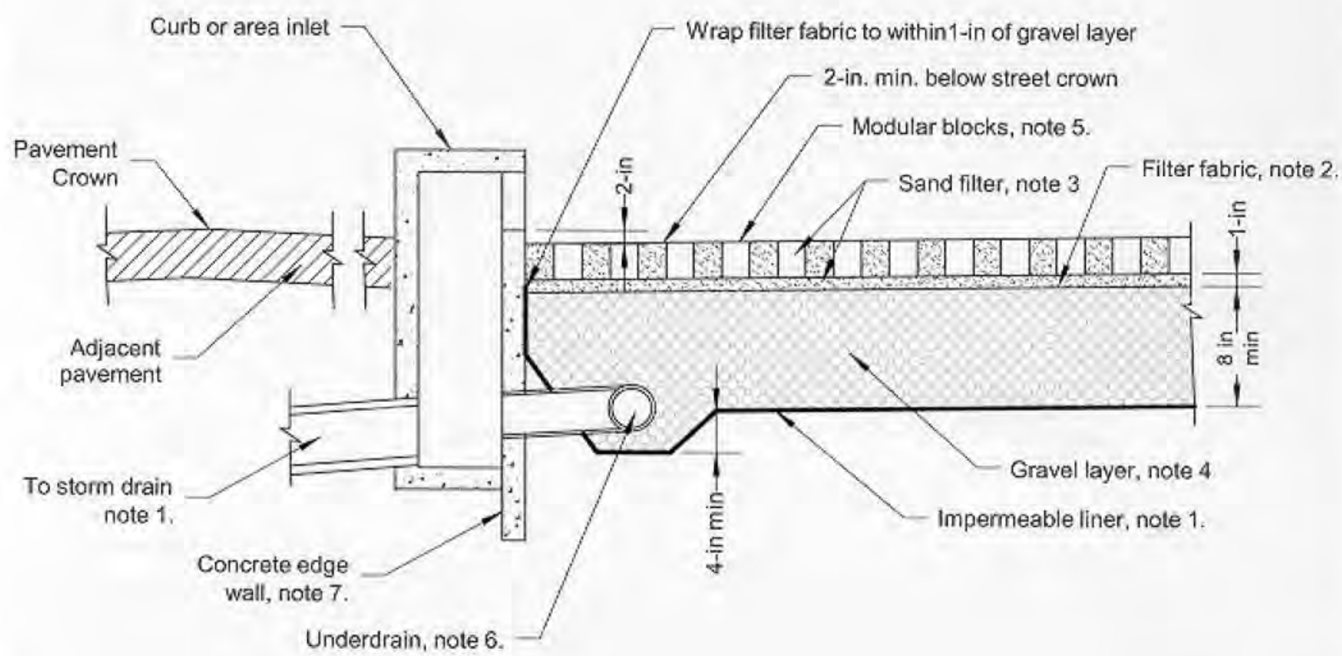
iv. If coalescing plates are used, they shall be separated by no less than ¾ inches and be angled from 45 to 60 degrees from the horizontal.

e. Provide access to all chambers for monitoring and maintenance.

f. Requires a formal operations and maintenance agreement between the local jurisdiction and the property owner be in force.

Two examples of proprietary products that can separate oil and water are shown on **Figures 1609** and **1610**. Reference to any proprietary product does not imply endorsement of the product by the local jurisdictions, but is meant to demonstrate that different products are available. **Figure 1609** shows an example of a precast concrete vault which includes a coalescing media. The media attracts oil droplets and increases the oil droplet size allowing more oil to rise to the surface than a system that relies on gravity alone. **Figure 1610** shows an example of a system that relies on the filtering mechanism to remove oil droplets. Both systems also capture coarse sediment, floatables and trash.

## POROUS PAVEMENT DETENTION (PPD)




NOTES:

1. PPD MUST INCLUDE AN UNDER–DRAIN SYSTEM THAT DISCHARGES TO A SURFACE DRAINAGE AND A 16 MIL IMPERMEABLE LINER. INFILTRATION NOT PERMITTED.
2. FILTER FABRIC(GEO–TEXTILE) SHALL MEET THE FOLLOWING: ASTM D–4751–AOS US STD SIEVE #50 TO #70; ASTM D–4632 – MIN. GRAB STRENGTH OF 100–LBS; ASTM D–4491 – MIN. PERMITTIVITY OF 1.8/SEC.
3. SAND FILTER TO BE ASTM C33 SAND.
4. GRAVEL LAYER TO BE AASHTO #4 WITH ALL FRACTURED FACES(CDOT SECTION 703, #4 COURSE AGGREGATE).
5. MODULAR BLOCK WITH OPEN SURFACE AREA OF 40% OR GREATER FOR VOID SPACE. SURFACE OF BLOCKS TO BE PLACED AT 0% GRADE IN ALL DIRECTIONS.
6. UNDER–DRAIN TO BE 4" MIN. DIAMETER, WITH 0.2% MIN. SLOPE, AT 20–FT MAXIMUM, ON CENTERS.
7. CONSTRUCT CONCRETE EDGE–WALL AROUND PERIMETER OF PPD, MIN. 6–IN THICK AND 18–IN DEEP.

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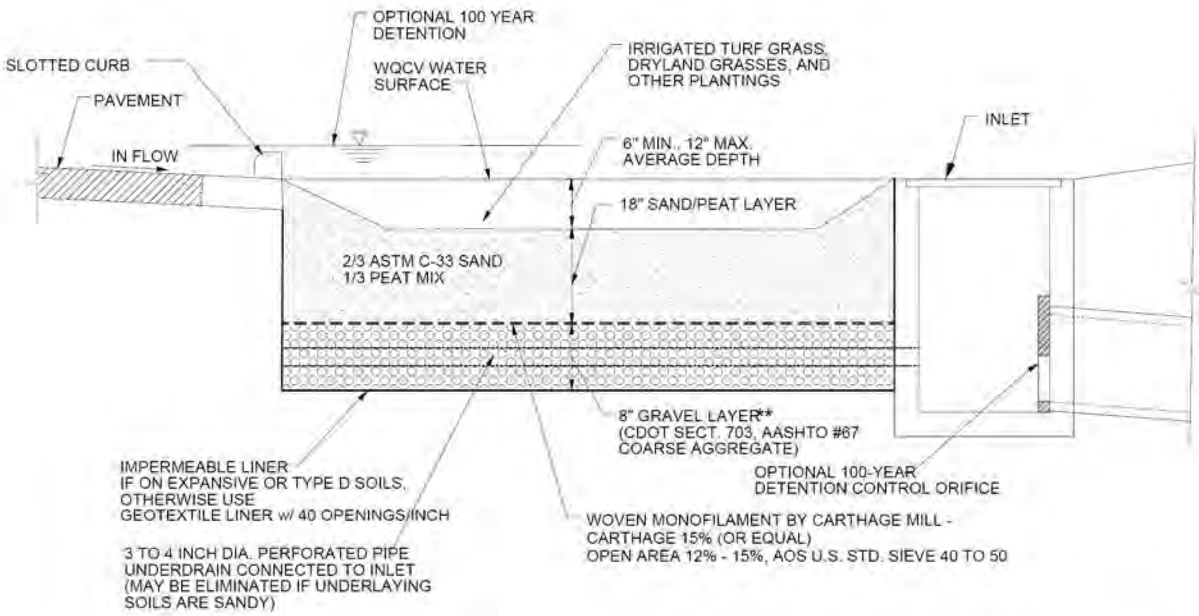
REFERENCE: *UDFCD. URBAN DRAINAGE CRITERIA MANUAL VOLUME 3 BEST MANAGEMENT PRACTICES*

**FIGURE 1601**

# STORMWATER MANAGEMENT MANUAL

## POROUS LANDSCAPE DETENTION (PLD)




** SUBSTITUTE AN 18" LAYER OF SAND/PEAT MIX FOR THE 8" GRAVEL LAYER WHEN NO UNDERDRAIN IS USED IN NRCS TYPE D SOILS OR IN EXPANSIVE SOILS. USE IMPERMEABLE LINER UNDER AND ON SIDES OF BASIN.

**NOTES:**

1. **PLD MUST INCLUDE AN UNDERDRAIN SYSTEM THAT DISCHARGES TO A SUFACE DRAINAGE. NO INFILTRATION PERMITTED.**

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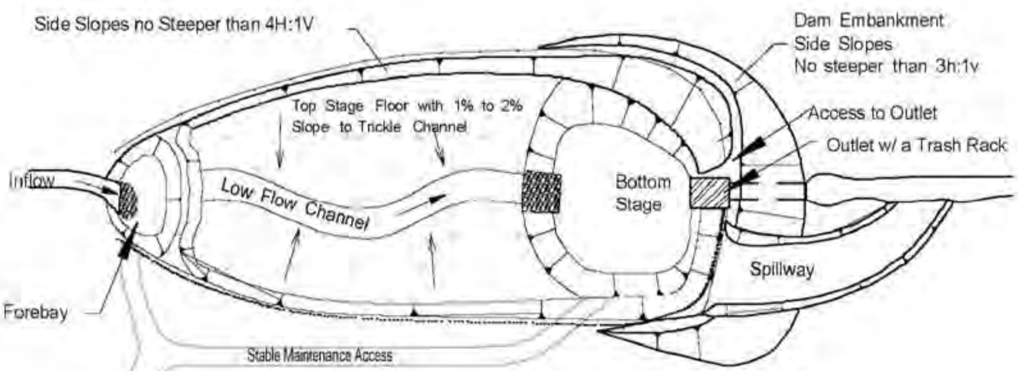
**FIGURE 1602**

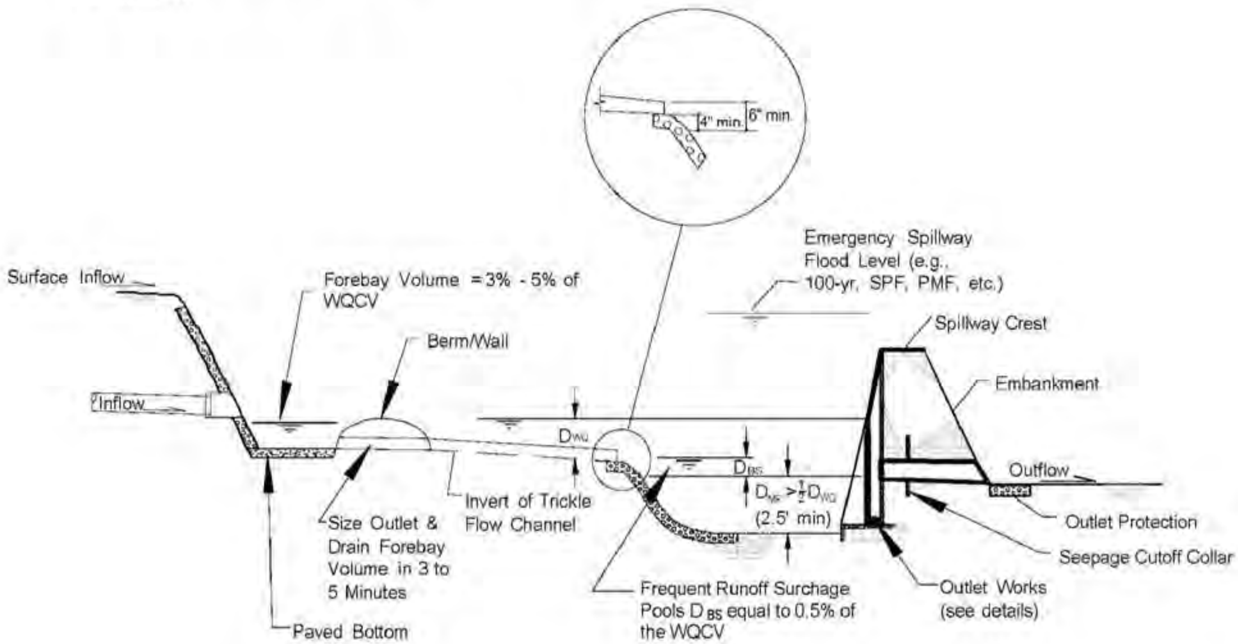
# STORMWATER MANAGEMENT MANUAL

## EXTENDED DETENTION BASIN (EDB)




Note: Provide energy dissipating inlet such as impact basin for pipes and GSB drop or baffle chute for channel/swales.

PLAN
Not to scale




PROFILE
Not to scale

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|---|---|
| ORIGINAL ISSUE | 3/27/06 |
| UDFCD UPDATE | 12/6/07 |

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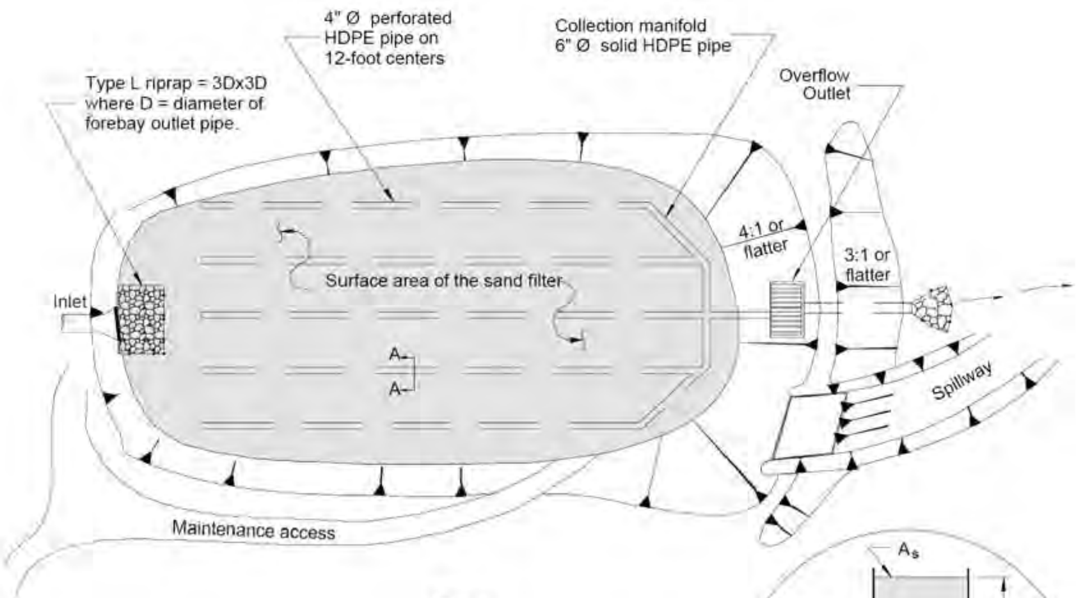
REFERENCE:

*UDFCD. URBAN DRAINAGE CRITERIA MANUAL*
*VOLUME 3 BEST MANAGEMENT PRACTICES*

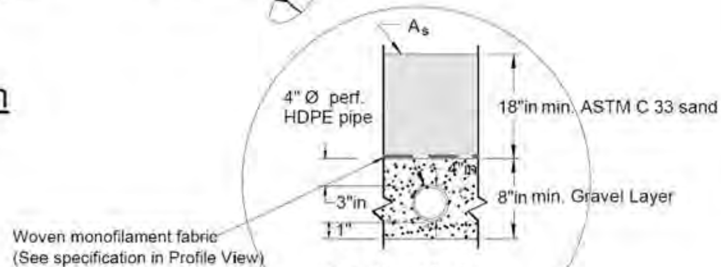
**FIGURE 1603**

# STORMWATER MANAGEMENT MANUAL

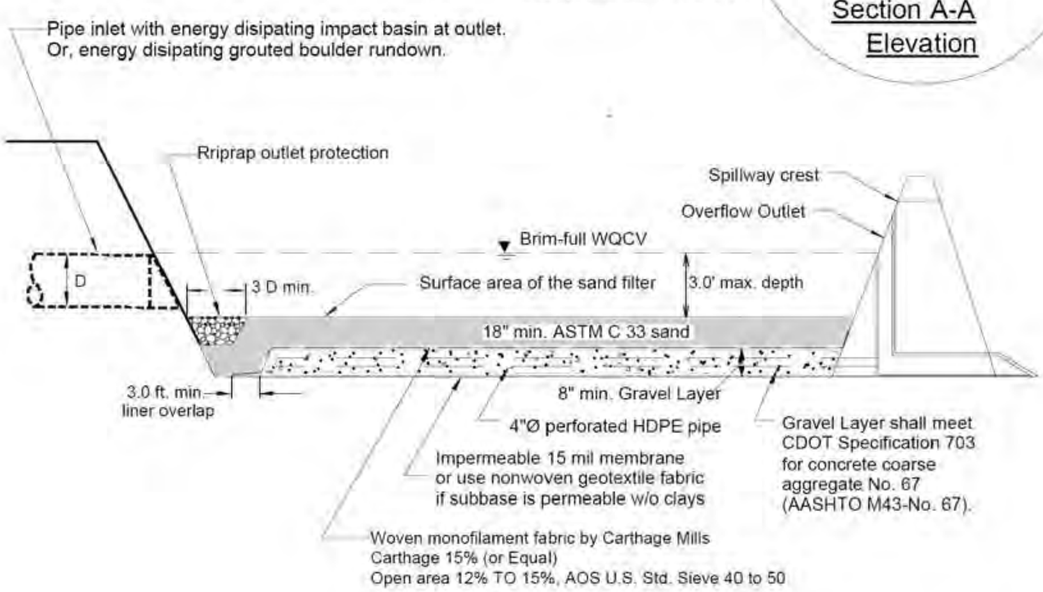
## SAND FILTER EXTENDED DETENTION BASIN (SFB)




Plan




Section A-A
Elevation




Profile

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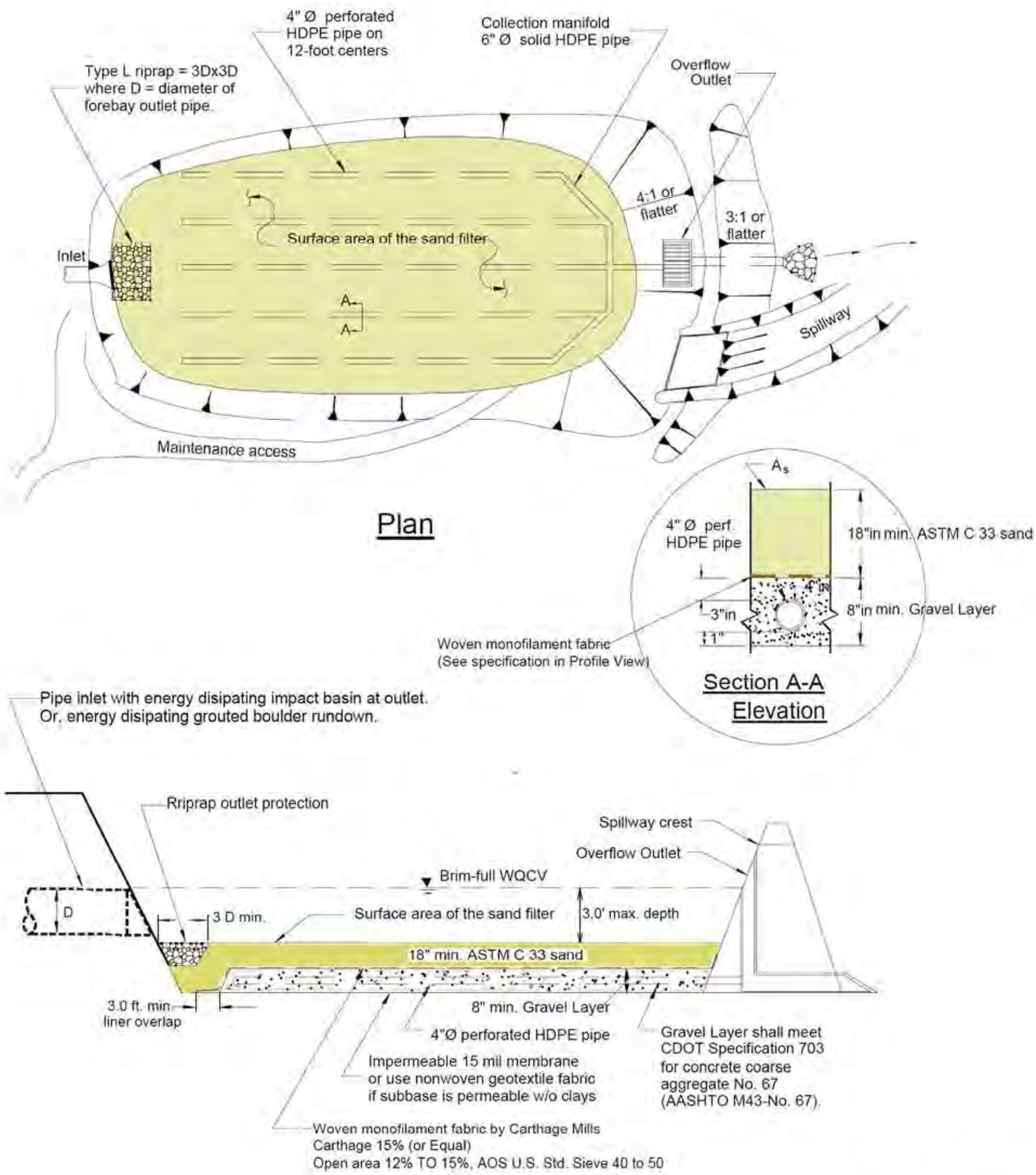
**FIGURE 1604**

# STORMWATER MANAGEMENT MANUAL

## SAND FILTER EXTENDED DETENTION BASIN (SFB)




Profile

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**FIGURE 1604**

# CONSTRUCTED WETLAND BASIN (CWB)

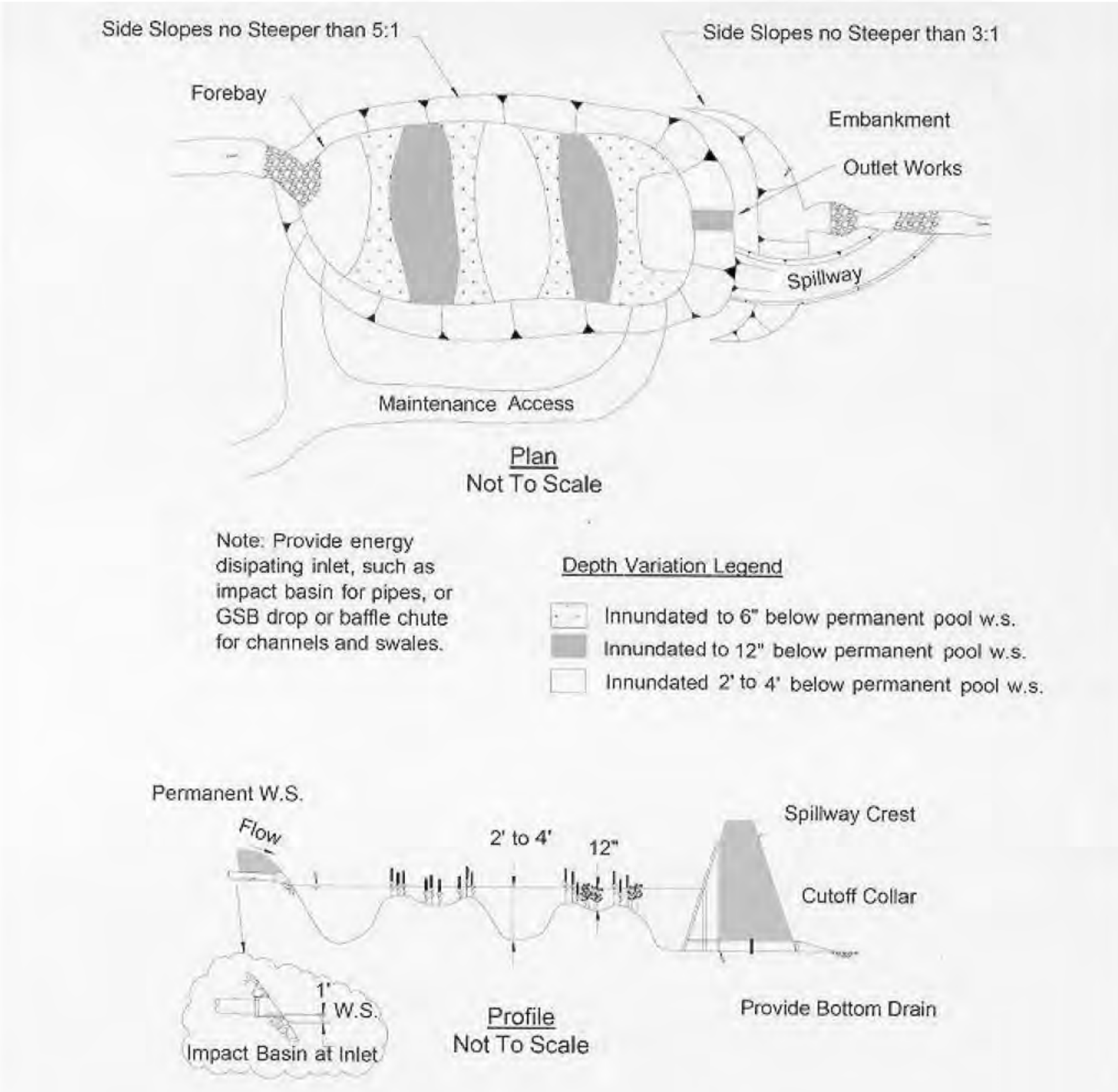


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**FIGURE 1605**

# STORMWATER MANAGEMENT MANUAL

## RETENTION BASIN (RB)

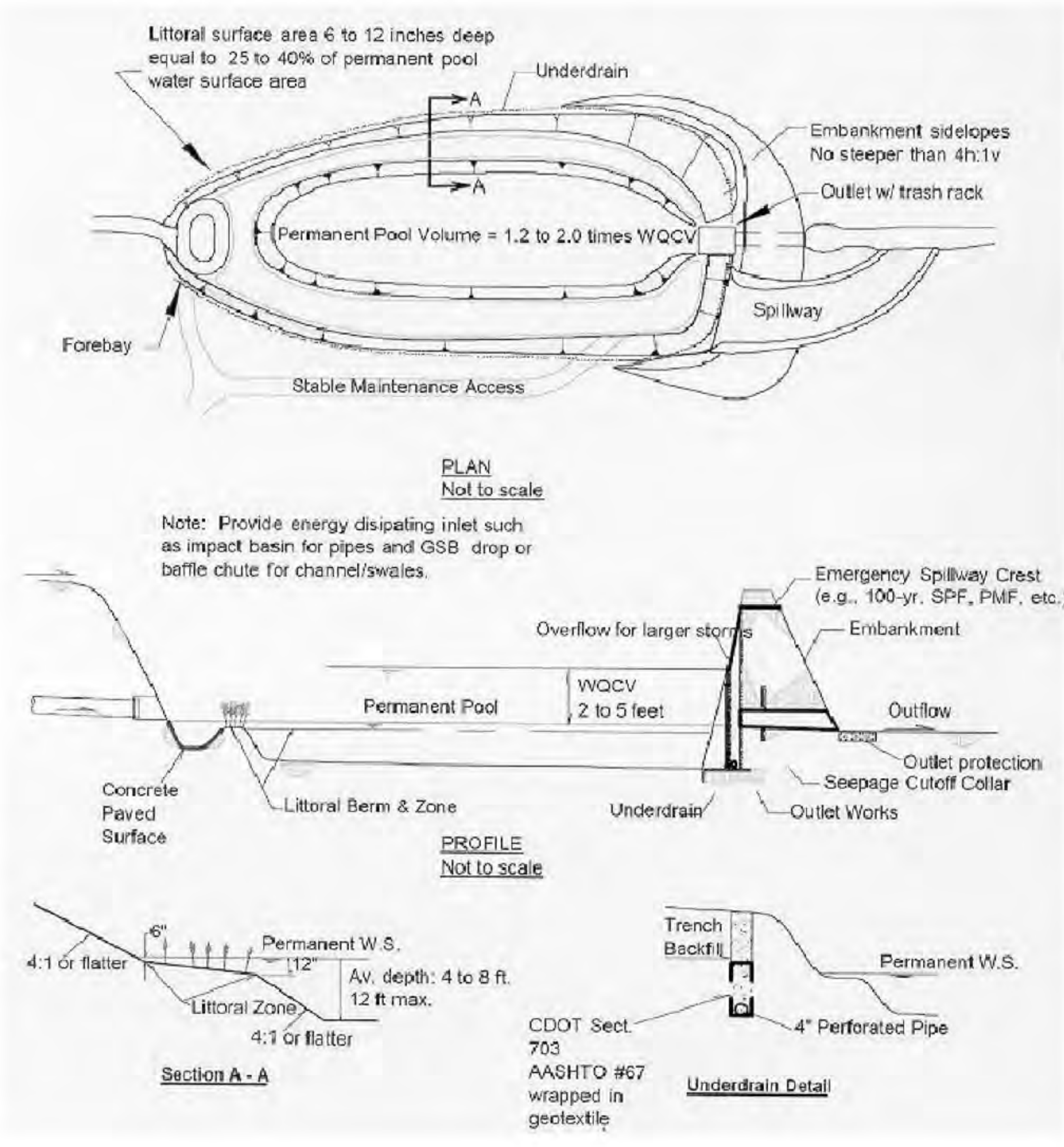


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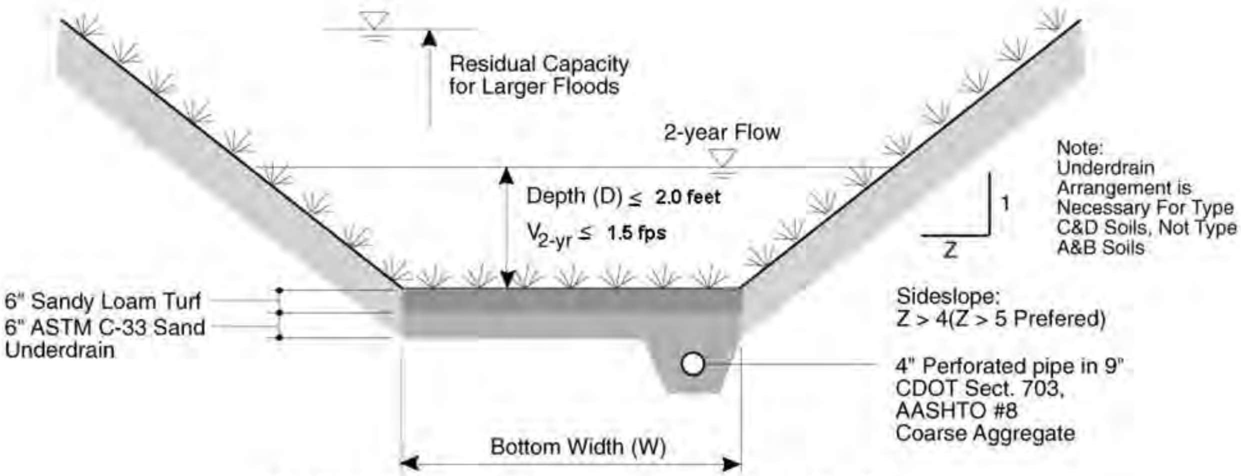
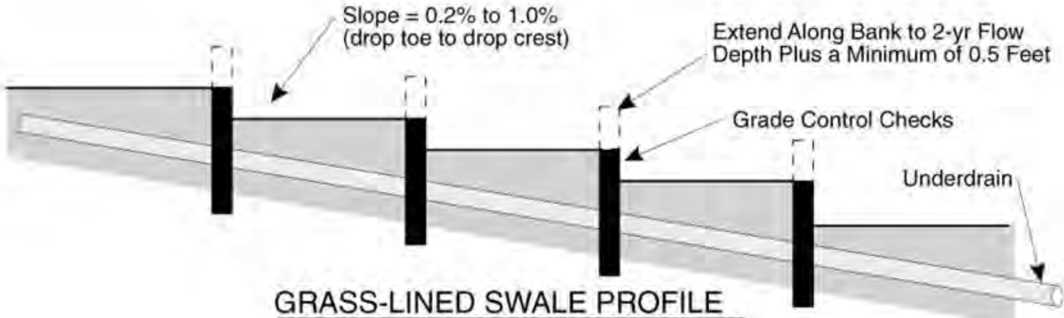
**FIGURE 1606**



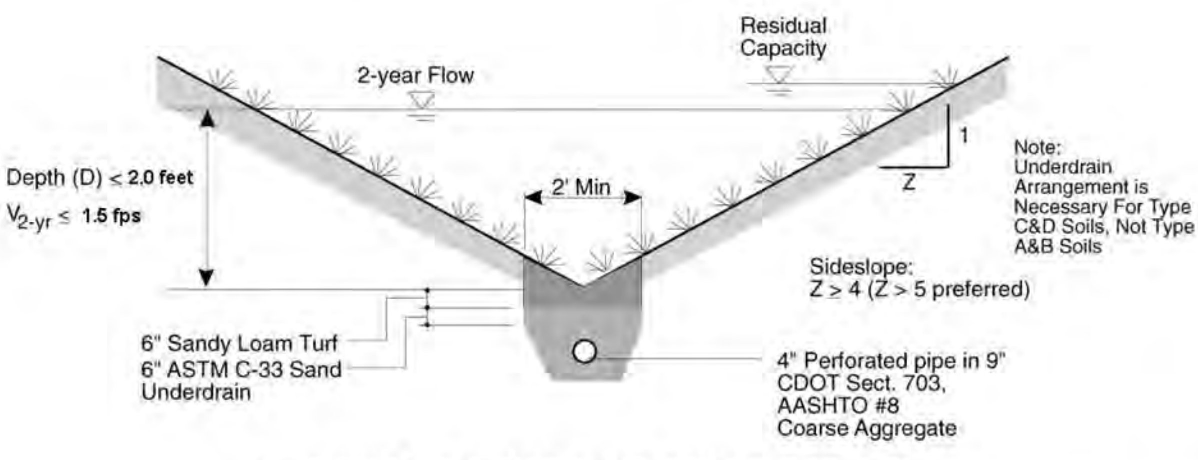

TRAPEZOIDAL GRASS-LINED SWALE SECTION

NOT TO SCALE




GRASS-LINED SWALE PROFILE

NOT TO SCALE




TRIANGULAR GRASS-LINED SWALE SECTION

NOT TO SCALE

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*VOLUME 3 BEST MANAGEMENT PRACTICES*

**FIGURE 1607**

| Source Control BMPs to Consider | | BMP Criteria | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| Abbreviation | Title | Manufacturing | Material Handling | Vehicle Maintenance | Construction | Commercial Activity | Roadways | Waste Containment | Housekeeping Practices |
| ACP | Area Control Procedures | X | | | | X | | | X |
| ATL | Aboveground Tank Leak and Spill Control | X | X | | | X | X | X | |
| BGM | Buildings and Grounds Maintenance | | | | | X | X | | X |
| BRRC | Building Repair, Remodeling and Construction | | | | X | X | | | X |
| CD | Containment Dikes | X | | | | | | X | |
| CESA | Contaminated or Erodible Surface Areas | X | | | X | | X | | |
| CO | Covering | X | X | X | X | X | | X | X |
| CU | Curbing | | X | | X | X | | X | X |
| DCUS | De-Icing Chemical Use and Storage | X | X | | | | X | X | X |
| DP | Drip Pans | X | | | | | | X | |
| ET | Employee Training | X | X | X | X | X | X | X | X |
| HWM | Hazardous Waste Management | X | X | X | X | | | X | X |
| NSWD | Non-Storm Water Dischrages to Drains | | | X | | X | | X | X |
| OCSL | Outdoor Container Storage of Liquids | | X | | X | X | | X | X |
| OLUM | Outdoor Loading/Unloading of Materials | X | X | | X | X | | | X |
| OPE | Outdoor Process Equipment Operations | X | | X | X | X | | | X |
| OSRM | Outdoor Storage of Raw Materials | X | X | | X | X | | | X |
| SL | Signs and Labels | | | | | X | | | X |
| VEC | Vehicle and Equipment Cleaning | | | X | X | X | | | X |
| VEF | Vehicle and Equipment Fueling | | X | X | X | X | | | X |
| VEMR | Vehicle and Equipment Maintenance & Repair | | X | X | X | X | | | X |
| WHD | Waste Handling and Disposal | | X | | X | X | | X | X |

Note:
Go to the following internet link: http://www.pweng.slco.org/pdf/imatrix.pdf. Then by clicking on the source control BMP, a one-page fact sheet can be obtained which describes the application, the criteria, limitations, and targeted pollutants.

| Revision | Date |
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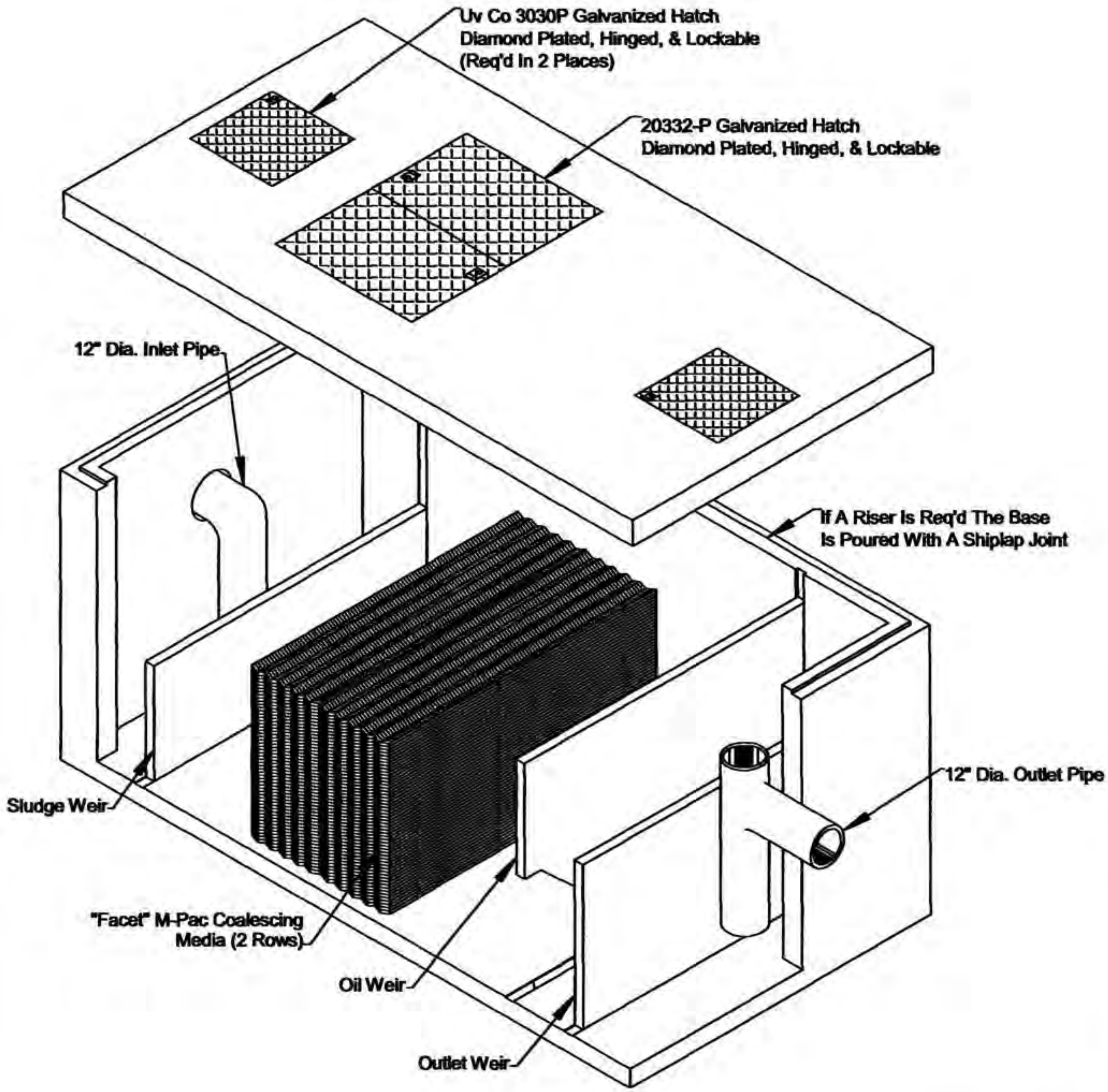
| ORIGINAL ISSUE | 3/27/06 |
| CORRECTED TITLE | 12/6/07 |

# OIL/WATER SEPARATOR - COALESCING MEDIA

(Note: Reference to any proprietary product does not imply endorsement by the County or other local jurisdiction)

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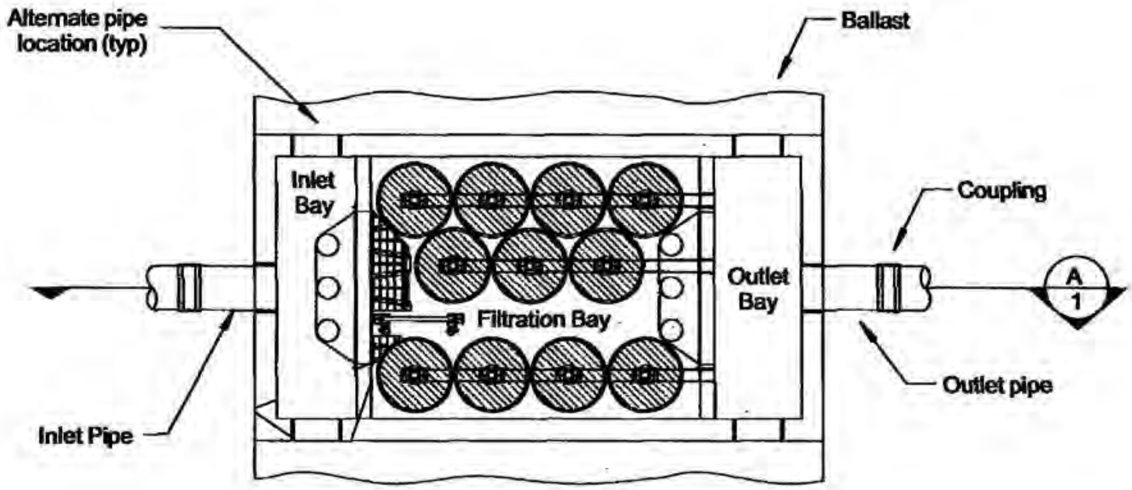
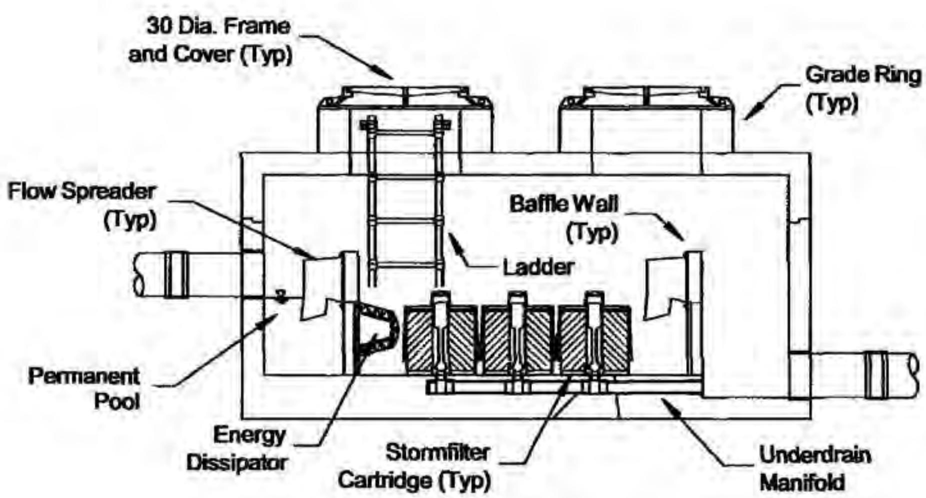
# OIL/WATER SEPARATOR - FILTER MEDIA

(Note: Reference to any proprietary product does not imply endorsement by the County or other local jurisdiction)




6' x 12' STORMFILTER - PLAN VIEW $\frac{1}{1}$




6' x 12' STORMFILTER - SECTION VIEW $\frac{A}{1}$

THE STORMWATER MANAGEMENT StormFilter® U.S. PATENT No. 5,322,629, No. 5,707,527, No. 6,027,639 No. 6,649,048, No. 5,624,576, AND OTHER U.S. AND FOREIGN PATENTS PENDING



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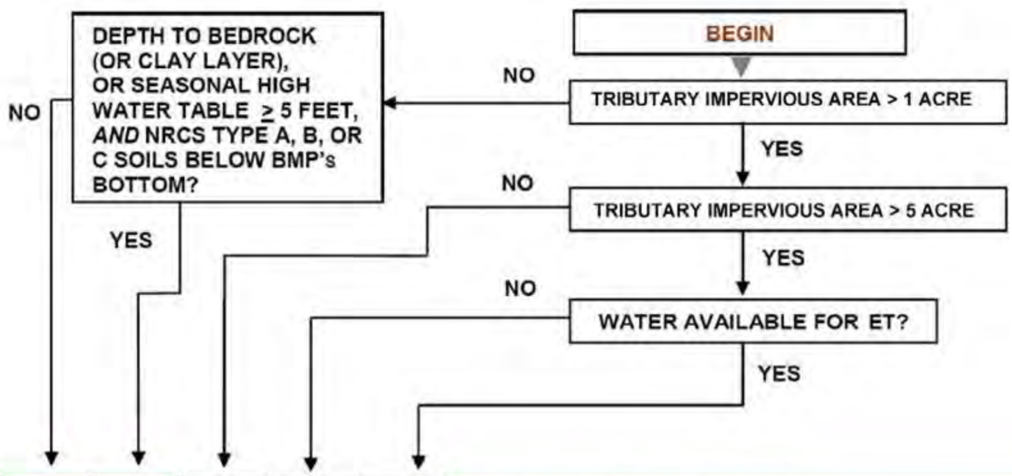
WRC ENGINEERING INC

REFERENCE: *THE STORMWATER MANAGEMENT, STORMFILTER 2005 STORMWATER 360, WWW.STORMWATER360.COM*

**FIGURE 1610**

# DECISION TREE FOR IDENTIFYING POTENTIAL INDIVIDUAL SITE BMPS




| | | | | | |
|---|---|---|---|---|---|
| ■ | ■ | | | | GREEN ROOF WITH IRRIGATION |
| ■ | ■ | | | | GRASS BUFFER (GB) |
| ■ | ■ | ■ | ■ | ■ | GRASS SWALE (GS) |
| | ■ | | | | POROUS PAVEMENT (PP) WITH INFILTRATION |
| ■ | | | | | POROUS PAVEMENT (PP) WITH UNDERDRAINS |
| | ◆ | | | | POROUS PAVEMENT DETENTION (PPD) WITH INFILTRATION |
| ◆ | | | | | POROUS PAVEMENT DETENTION (PPD) WITH UNDERDRAINS |
| | ◆ | | | | POROUS LANDSCAPE DETENTION (PLD) WITH INFILTRATION |
| ◆ | | | | | POROUS LANDSCAPE DETENTION (PLD) WITH UNDERDRAINS |
| | | ● | ● | ● | EXTENDED DETENTION BASIN (EDB) *** |
| ◆ | ◆ | ◆ | ◆ | ◆ | SAND FILTER DETENTION BASIN (EDB) |
| | | | | ● | CONSTRUCTED WETLAND BASIN (CWB) |
| | | | | ● | RETENTION POND (RP) |
| ■ | ■ | ■ | ■ | ■ | CONSTRUCTED WETLAND CHANNEL (CWC) |

Legend:
- ■ Provides for reductions in runoff volume, along with some reduction in pollutant EMCs
- ◆ Provides for reductions in runoff volume and the recommended *WQCV*
- ● Provides recommended *WQCV*
- *** EDBs are not very suitable for catchments with less than 5 acres of impervious tributary area. The International BMP Database reports EDBs provide as much as 30% reductions in annual runoff volume.

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|---|---|
| ORIGINAL ISSUE | 12/6/07 |
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**FIGURE 1611**

# SECTION 1700

## STANDARD FORMS

# SECTION 1700
## STANDARD FORMS

### LIST OF FORMS

**Hydrologic and Hydraulic Design Forms**

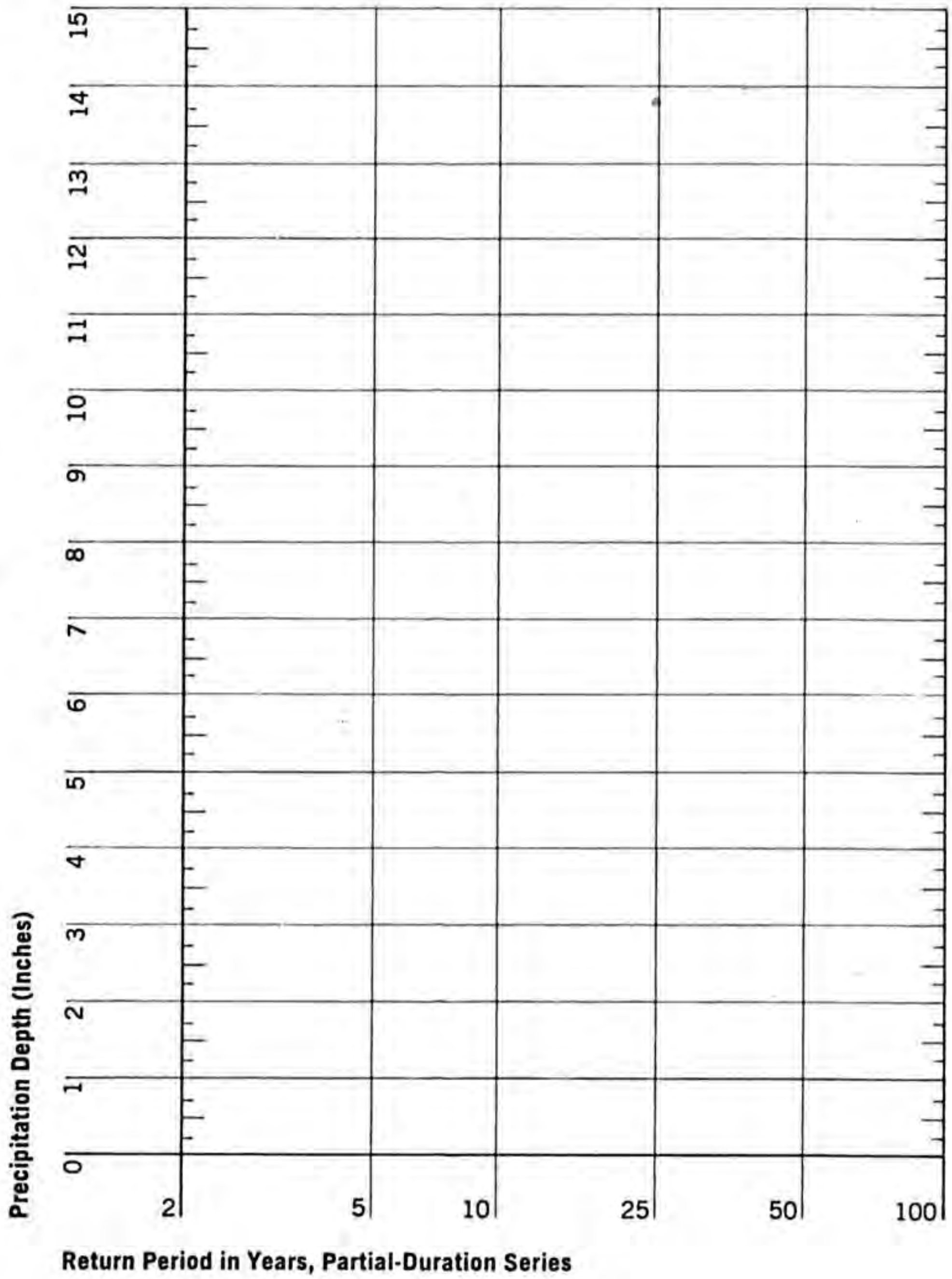
| | |
|---|---|
| Standard Form 1 | Precipitation Depth vs. Return Period |
| Standard Form 2 | Time of Concentration |
| Standard Form 3 | Storm Drain Hydraulic Calculations |
| Standard Form 4 | Culvert Design |

**Stormwater Permit Applications and Report Forms**

1 Mesa County Engineering Division Stormwater Construction Permit
2 Mesa County Stormwater Construction Permit Field Inspection Report
3 Mesa County Post-Construction Stormwater Controls and Best Management Practices Operations and Maintenance Agreement
4 City of Grand Junction Storm Water Construction Permit Application
5 Reassignment of Permit Coverage
6 Grand Junction Post-Construction Stormwater Control Operations and Maintenance Agreement
7 City of Grand Junction Stormwater Construction Permit Field Inspection Report

# STORMWATER MANAGEMENT MANUAL

## PRECIPITATION DEPTH VS. RETURN PERIOD

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REFERENCE:

NOAA ATLAS 2, VOLUME III COLORADO, 1973

**STANDARD FORM 1**

# STORMWATER MANAGEMENT MANUAL

## TIME OF CONCENTRATION

DEVELOPMENT: ______________________________

CALCULATED BY: ______________________ DATE: __________

| SUB-BASIN DATA | | | INITIAL/OVERLAND TIME ($t_i$) | | | TRAVEL TIME ($t_t$) | | | | $t_c$ ($t_i + t_t$) | $t_{c\,(MAXIMUM)}$ (URBANIZED BASINS) | | $t_c$ FINAL | NOTES |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| NAME (1) | "K" TABLE 702 (2) | AREA (AC) (3) | LENGTH (FT) (4) | SLOPE (%) (5) | $t_i$ (MIN) (6) | LENGTH (FT) (7) | SLOPE (%) (8) | VELOCITY (FPS) (9) | $t_t$ (MIN) (10) | $t_c$ (MIN) (11) | TOTAL LENGTH (FT) (12) | $t_{c\,(MAX)} = (L/180)+10$ (MIN) (13) | (MIN) (14) | |
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(2) FROM TABLE 702
(3) tl = 1.8 (1.1 - K) Lo1/2/S1/3 (EQUATION 702)
(14) MINIMUM tc: URBAN = 5 MINUTES, NON-URBAN = 10 MINUTES

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REFERENCE:

**STANDARD FORM 2**

# STORMWATER MANAGEMENT MANUAL

## STORM DRAIN HYDRAULIC CALCULATIONS

| 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 11 | 12 |
|---|---|---|---|---|---|---|---|---|---|---|---|
| CONDUIT DATA | | | | | | | FLOW DATA | | | | |
| STATION | | SIZE / TYPE | INVERTS | | LENGTH | SLOPE | A (full) ($FT^2$) | Q (avg) (CFS) | V (avg) (FPS) | $H_V = V^2/2G$ (FT) | $S_f$ (avg) (FT/FT) |
| FROM | TO | | D/S ELEV. | U/S ELEV. | | | | | | | |
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| MANHOLE HYDRAULICS | | | | | |
|---|---|---|---|---|---|
| Manhole at Design Point | Initial $h_L$ coefficient $K_o$ | $C_D$ or $C_d$ | $C_Q$ or $C_p$ | $C_B$ | Adjusted $h_L$ coefficient K |
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| 1 | 2 | 13 | 14 | 15 | 16 | 17 | 18 | 19 | 20 | 21 | 22 | 23 | 24 | 25 | 26 | 27 | 28 | 29 | 30 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | ENERGY LOSSES | | | | | | | | | | ENERGY / HYDRAULIC GRADE LINE ELEVATIONS | | | | | NOTES | | |
| STATION | | PIPE | MINOR PIPE LOSSES | | | | | | $\Sigma h_L$ (PIPE) (FT) | U/S MH | | PIPE | | | MANHOLE (U/S) | | PRESSURE FLOW? (Y/N) | MH RIM/INLET GRATE ELEV. | COMMENTS |
| FROM | TO | $h_f$ (FT) | $h_e$ (FT) | $h_c$ (FT) | $h_b$ (FT) | $h_j$ (FT) | $h_t$ (FT) | $h_o$ (FT) | | $h_{mh}$ (THIS LINE) (FT) | NEXT U/S STATION | D/S EGL ELEV. | U/S EGL ELEV. | U/S HGL ELEV. | U/S EGL ELEV. | U/S HGL ELEV. | | | |
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REFERENCE:

**STANDARD FORM 3**

## CULVERT DESIGN

**PROJECT :** ______________________

**STATION :** ______________

**SHEET** ______ **OF** ______

**CULVERT DESIGN FORM**

**DESIGNER / DATE :** ________ / ________

**REVIEWER / DATE :** ________ / ________

**HYDROLOGICAL DATA**

SEE ADD'L SHTS.

☐ METHOD: ______________________

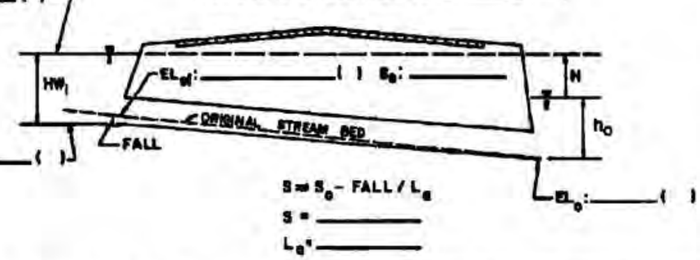
☐ DRAINAGE AREA: ________ ☐ STREAM SLOPE: ________

☐ CHANNEL SHAPE: ______________________

☐ ROUTING: ________ ☐ OTHER: ________

**DESIGN FLOWS/TAILWATER**

| R.I. (YEARS) | FLOW( ) | TW( ) |
|---|---|---|
| | | |
| | | |




| CULVERT DESCRIPTION: MATERIAL - SHAPE - SIZE - ENTRANCE | TOTAL FLOW | FLOW PER BARREL | HEADWATER CALCULATIONS | | | | | | | | | | | CONTROL HEADWATER ELEVATION | OUTLET VELOCITY | COMMENTS |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | INLET CONTROL | | | | OUTLET CONTROL | | | | | | | | | |
| | Q ( ) | Q/N (1) | $HW_i/D$ (2) | $HW_i$ | FALL (3) | $EL_{hi}$ (4) | TW (5) | $d_c$ | $\frac{d_c+D}{2}$ | $h_o$ (6) | $k_e$ | H (7) | $EL_{ho}$ (8) | | | |
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| | | | | | | | | | | | | | | | | |

**TECHNICAL FOOTNOTES:**

(1) USE Q/NB FOR BOX CULVERTS

(2) $HW_i/D = HW/D$ OR $HW_i/D$ FROM DESIGN CHARTS

(3) $FALL = HW_i - (EL_{hd} - EL_{sf})$; FALL IS ZERO FOR CULVERTS ON GRADE

(4) $EL_{hi} = HW_i + EL_i$ (INVERT OF INLET CONTROL SECTION)

(5) TW BASED ON DOWN STREAM CONTROL OR FLOW DEPTH IN CHANNEL.

(6) $h_o = TW$ or $(d_c + D)/2$ (WHICHEVER IS GREATER)

(7) $H = \left[1 + k_e + (K_u n^2 L)/R^{1.33}\right] v^2/2g$ WHERE Ku = 19.63 (29 IN ENGLISH UNITS)

(8) $EL_{ho} = EL_o + H + h_o$

**SUBSCRIPT DEFINITIONS :**

a. APPROXIMATE
f. CULVERT FACE
hd. DESIGN HEADWATER
hi. HEADWATER IN INLET CONTROL
ho. HEADWATER IN OUTLET CONTROL
i. INLET CONTROL SECTION
o. OUTLET
sf. STREAMBED AT CULVERT FACE
tw. TAILWATER

**COMMENTS / DISCUSSION :**

**CULVERT BARREL SELECTED :**

SIZE: ______________

SHAPE: ______________

MATERIAL: ________ n ______

ENTRANCE: ______________

| Revision | Date |
|---|---|
| ORIGINAL ISSUE | 3/27/06 |
| | |
| | |

Permit No. ____________________

# Mesa County
# Engineering Division
# Stormwater Construction Permit

Please print or type. All items must be completed accurately and in their entirety or the application will be deemed incomplete and the application returned. Processing of the application will not begin until all information is received. **Review of a complete permit application will be completed within twenty calendar days.** Original signature for Part 7 is required.

A Stormwater Construction Permit is required, within the urbanized area, for all land disturbance activities equal to or greater than 1 one (1) acre or for land disturbance activities less than one (1) acre that are part of a larger common plan of development or sale that would disturb one (1) acre or more.

## 1. Name and address of the Permittee:

Property Owner: ____________________________________________________
Mailing Address: ___________________________________________________
City, State and Zip Code: ____________________________________________
Phone Number ( ) _____________________________________________
Legally Responsible Person (application signer) E-mail Address: _______________

Project Site Contact (familiar with facility): _____________________________
Title _______________ Phone Number ( )_________________________
Site Contact E-mail Address: _________________________________________

2. List, in addition to the permittee, any site personnel along with their contact information, who will have the authority to make modifications to the Construction Stormwater Management Plan.

Name: ______________________________Phone Number ( )_______________
Name: ______________________________Phone Number ( )_______________
Name: ______________________________Phone Number ( )_______________
Name: ______________________________Phone Number ( )_______________

## 3. Location of the project site:

Street Address (or cross streets): ______________________________________
Name of plan, project, or development: __________________________________
Current Mesa County Assessor's Tax parcel number: ________________________

## 4. Area of the project site:

Total area of project site (acres):______________________________
Area of project site to undergo disturbance (acres):___________________________
Total disturbed area of Larger Common Plan of Development or Sale, if applicable (i.e., total including all phases, filings, lots, and infrastructure not covered by this application.): __________________________________________

Permit No. ____________________

## 5. Nature of the construction activities:

Check the appropriate box(es) or provide a brief description that indicates the general nature of the construction activities. (The full description of activities must be included in the Stormwater Management Plan.)

☐Single Family Residential Development
☐Multi-Family Residential Development
☐Commercial Development
☐Highway/Road Development (not including roadways associated with commercial or residential development)
☐Utility
☐Other Describe: ____________________________________________________
____________________________________________________

## 6. Anticipated construction schedule:

Construction Start Date: _____/____/_____ (month / day / year) Final Stabilization Date: ____/____/_____ (month / day / year)

## 7. Acknowledgement Certificate:

By signing below, the Permittee hereby applies for a Mesa County Stormwater Construction Permit for the aforementioned property and certifies as follows:

a) To the best of my knowledge, the information provided herein is correct; and is consistent with the development plans approved by the planning and/or building departments.
b) A Construction Stormwater Management Plan for the disturbed area on the project site was prepared and submitted in accordance with the Stormwater Management Manual (SWMM) and the applicable Mesa County Land Development Code.
c) I certify that I am legally authorized to sign on behalf of and bind the above-listed entity.

The Stormwater Construction Permit is granted with the explicit understanding that it is the Permittee's responsibility to:

a) Comply with all requirements of the SWMM and applicable Mesa County Land Development Code.
b) Allow Mesa County or it's designee reasonable access to the project site to conduct regular site inspections.
c) Immediately cease land-disturbing and all other construction activities upon receipt of a written Stop Work Order from an authorized representative of Mesa County.
d) Comply with any and all directives of Mesa County contained in any Notice of Violation pertaining to the project site issued to the Permittee by Mesa County.

Permit No. ____________________

By signing below the Permittee agrees as follows:

a) If the Construction Stormwater Management Plan is submitted lacking the required information outlined in the SWMM, the Construction Stormwater Management Plan will be returned to the Permittee for revisions.
b) If the Permittee becomes insolvent or declares bankruptcy, such that the requirements of this Permit are not being met, Mesa County may perform any and all necessary corrective actions, at the sole and absolute discretion of Mesa County, to provide sufficient remedies to correct project site deficiencies.
c) Mesa County may issue a Stop Work Order and/or a Notice of Violation and/or may revoke this Permit or pursue any other legal remedies available if Mesa County determines the Permittee is not in compliance with the SWMM, Stormwater Construction Permit, and applicable Mesa County Land Development Code, or the Permittee fails to take all corrective action requested within the time specified on any Stop Work Order or Notice of Violation issued to the Permitte by Mesa County.
d) That in addition to other remedies, a violation of the Permit shall constitute a violation of the Land Development Code; and
e) Any approval obtained from Mesa County does not absolve the need to comply with requirements of Sections 7 and 9 of the Endangered Species Act of 1973, 16 U.S.C. 11531, et seq., as amended, or with any other applicable federal, state, or local laws or regulations.

_______________________________________________________________________

Signature of Legally Responsible Person (submission must include original signature) Date Signed

_______________________________________________________________________

Name (printed) Title

## 7. Approval

Mesa County Approval:____________________________________

Date: __________________________________________________

Permit No. ____________________

Instructions

1. **Name and address of the Permittee:** The Permittee must be the current property owner of the project. An authorized agent may submit a permit application if, the agent is legally authorized to sign on behalf of and bind the Permittee. A project site contact is necessary to provide Mesa County a person to contact on the site in regards to inspections and compliance issues.
2. The Construction Stormwater Management Plan is a working document and field changes are expected. List the names of site personnel along with their contact information, who will have the authority to make modifications to the Construction Stormwater Management Plan.
3. **Location of Project Site:** Indicate the street address or cross streets, if a street address is not available, name of plan, project, or development, and current Mesa County Assessor's Tax Parcel number.
4. **Area of Project Site:** Indicate the total area of the project site in acres, and the area of project site to undergo disturbance. Disturbance includes any activity that disturbs the soil on the site, including: grading, clearing, excavation activities, areas receiving overburden (e.g. stockpiles), demolition areas, and areas with equipment/vehicle traffic and storage that disturb vegetative cover.
5. **Nature of Construction Activities:** Either check the appropriate box or boxes, or if the given descriptions do not fit the project, provide a brief description that indicates the general nature of the construction activities for which permit coverage is being requested. A more detailed description of the project must be included in the Construction Stormwater Management Plan.
6. **Anticipated Construction Schedule:** Provide the current estimated start and final stabilization dates for the construction project as follows:
   a) Construction Start Date - This is the day you expect to begin disturbing soils, including grubbing, stockpiling, excavating, demolition, and grading activities.
   b) Final Stabilization Date - in terms of permit coverage, this is when the site is finally stabilized. This means that all disturbed areas have been either built on, paved, or a uniform vegetative cover has been established. **Permit coverage must be maintained until that time.** Even if the Permittee is only doing one part of the project, the estimated completion date must be for the overall project. If permit coverage is still required once the Permittee's part is completed, the permit certification may be transferred or reassigned to a new responsible entity(s).
7. Acknowledgement Certificate: The Permittee must be the property owner of the project site. The application must be signed by the Permittee to be considered complete. In all cases, it shall be signed as follows:
   a) In the case of corporations, by a principal executive officer of at least the level of vice-president or his or her duly authorized

representative, if such representative is responsible for the overall operation of the facility from which the discharge described in the application originates.

b) In the case of a partnership, by a general partner.
c) In the case of a sole proprietorship, by the proprietor.
d) In the case of a municipal, state, or other public facility, by either a principal executive officer, ranking elected official, or other duly authorized employee.

# DRAFT MESA COUNTY STORMWATER CONSTRUCTION PERMIT FIELD INSPECTION REPORT

(1) Date of Inspection

(2) Property Owner

(3) Site Contact Name (print)

(4) Inspector Name (print)

(5) Project Name

(6) Permit No.

(7) Reason for Inspection :

- ❑ Required Maximum 14 Calendar Day Inspection
- ❑ Required 30 Calendar Day Inspection for Completed Projects
- ❑ Required Storm Event Inspection
- ❑ Complaint:______________________
- ❑ Other:______________________

## (8) CONSTRUCTION SITE ASSESSMENT

- ❑ Construction site perimeter contained. Offsite tracking minimized.
- ❑ Disturbed areas contained.
- ❑ Estimate disturbed area at the time of the inspection. ______ Acres
- ❑ Areas used for material and waste storage and fueling contained.

## (9) CSWMP MANAGEMENT

- ❑ Does site have current CSWMP Yes / No
- ❑ Contents of CSWMP:
  - ❑ Site Description Yes / No
  - ❑ Site Map Yes / No
  - ❑ Description of BMPs during construction activities Yes / No
  - ❑ Materials Handling & Spill Prevention Yes / No
- ❑ Changes noted & dated on the plans? Yes / No
- ❑ Contents of CSWMP (Cont.)
  - ❑ Final Stabilization Yes / No
  - ❑ Other Controls Yes / No
  - ❑ Inspection & Maintenance Yes / No
  - ❑ Certification Yes / No

## BEST MANAGEMENT PRACTICES (BMPs)

| (10) **BMP Type** | (11) **Practice Req/Used** | | (12) **Reason** | (13) **Maintenance/ Sediment Removal Required Y/N** | (14) **Course of Action** | (15) **Date for Action to be Completed** |
|---|---|---|---|---|---|---|
| | | | | **EROSION CONTROL** | | |
| Seeding | | | | | | |
| Mulching | | | | | | |
| Blankets | | | | | | |
| Check Dams | | | | | | |
| Earth Berms | | | | | | |
| Diversion | | | | | | |
| Embankment Protector | | | | | | |
| Outlet Protection | | | | | | |
| Surface Roughening | | | | | | |
| Other: | | | | | | |
| Other: | | | | | | |

| (10) **BMP Type** | (11) **Practice Req/Used** | | (12 ) **Reason** | (13) **Maintenance/ Sediment Removal Required Y/N** | (14) **Course of Action** | (15 ) **Date for Action to be Completed** |
|---|---|---|---|---|---|---|
| **SEDIMENT CONTROL** | | | | | | |
| Inlet Protection | | | | | | |
| Erosion Bales | | | | | | |
| Silt Fence | | | | | | |
| Sediment Trap/Basin | | | | | | |
| Stabilized Construction Entrance | | | | | | |
| Dewatering Structure | | | | | | |
| Other: | | | | | | |
| **MATERIALS HANDLING AND SPILL PREVENTION, WASTE MANAGEMENT AND GENERAL POLLUTION PREVENTION** | | | | | | |
| Stockpile Management | | | | | | |
| Materials Delivery and Storage | | | | | | |
| Spill Prevention and Control | | | | | | |
| Concrete Washout | | | | | | |
| Concrete Saw Water Containment | | | | | | |
| Solid Waste | | | | | | |
| Sanitary Waste | | | | | | |
| Maintenance and Fueling | | | | | | |
| Street Sweeping Vacuuming | | | | | | |
| Other: | | | | | | |
| Other: | | | | | | |

Comments:

| (16) **INSPECTIONS AND MAINTENANCE PROGRAM** | |
|---|---|
| ❑ Inspection occurring at least every 14 calendar days. | Course of Action: |
| ❑ Inspections occurring after storm events that result in runoff. | Course of Action: |
| ❑ Inspections occurring at least every 30 calendar days since project completion. | Course of Action: |
| ❑ Inspection reports retained at the construction project site. | Course of Action: |
| ❑ Corrective measures completed within 7 calendar days of inspection. | Course of Action: |

| **CERTIFICATION**<br>**I certify this Stormwater Construction Permit Field Inspection Report is complete and accurate.** | |
|---|---|
| Inspector Signature | Date |

# Stormwater Management Plan Field Inspection Report
## Instructions

1. **Date of Inspection:** Indicate the date the inspection was completed on.
2. **Property Owner:** Indicate the name of the property owner.
3. **Site Contact:** Indicate the main contact for the site.
4. **Inspector Name**: Indicate the name of the Inspector completing the inspection.
5. **Project Name**: Indicate the name of the project for which the report is being completed.
6. **Permit No.**: Indicate the permit number issued with the approval of the permit.
7. **Reason for Inspection**: Indicate the purpose for the inspection. The types of inspections include the following:
   - "Required 14 Calendar Day Inspection". These inspections are required at least every 14 calendar days during the life of the construction project.
   - "Required 30 Day Inspection for Completed Projects". These inspections are required at least every 30 calendar days following the completion of the construction project where final stabilization has not been achieved.
   - "Required Storm Event Inspection". These inspections are required after a storm event that results in runoff.
   - Inspection as a response to a complaint.
   - Inspection for any other reason.

   The first three types of inspections are required to comply with Mesa County Construction Stormwater Permit and the Colorado Discharge Permit System General Permit for Stormwater Discharges Associated with Construction Activity (CDPS General Permit).
8. **Construction Site Assessment**: Inspect the noted areas of the construction site and indicate with a "✓" the items which apply.
   - "Construction site perimeter contained". Are the appropriate BMPs in place and offsite sediment tracking minimized? Is there any evidence of pollutants entering a storm drainage system?
   - "Disturbed areas contained". Are the appropriate BMPs implemented to minimize erosion or sediment tracking from the disturbed areas? Is there any evidence of pollutants entering a storm drainage system? Provide an estimate of the disturbed area at the time of the inspection.
   - "Areas used for material and waste storage and fueling contained". Are the appropriate BMPs implemented to prevent and contain spills? Are wastes removed from the site and disposed of properly? Are the storage areas located at least 50 feet from a watercourse? Is there any evidence of pollutants entering a storm drainage system?
9. **CSWMP Management**:
   - Indicate if the Construction site has a current and update CSWMP, by circling yes or no.
   - Indicate if the required contents of the CSWMP are included, by circling yes or no on each line.
   - Indicate whether changes have been made to the CSWMP during construction and whether the changes have been documented and dated.
10. **BMP**: The BMPs shown may not be a complete list of what is required by the CSWMP. Cross out the BMPs not required by the CSWMP and add the BMPs that are required. Additional sheets can be inserted to show all the BMPs required by the CSWMP.
11. **Practice Req/Used**: This column can be used as follows:
    - If the BMP is required by the CSWMP and implemented, indicate by placing a "✓ " in both the "Req" and "Used" columns.
    - If the BMP is required by the CSWMP, but not implemented, indicate by placing a "✓ " in the "Req" column. Indicate the reason for the change in column (14), "Reason".
    - If the BMP has been added to the CSWMP, indicate with a "✓ " in the "Used " column. Indicate the reason for the change in column (14), "Reason".
12. **Reason**: Indicate the reason(s) for the deletion, addition, and modification of BMP(s) to the SWMP.
13. **Maintenance/Sediment Removal Required**: Indicate whether maintenance and sediment removal are required with a Yes or No. If maintenance and sediment removal are required, indicate what the action plan is in column (15), "Course of Action".
14. **Course of Action**: If maintenance and/or sediment removal is required, describe the action plan.
15. **Date for Action to be Completed**: Indicate the date for which the course of action will be completed. The course of action must be completed in a timely manner, but in no case more than 7 days after the inspection.
16. **Inspections and Maintenance**: Evaluate the inspection and maintenance aspect of the construction project and check all that applies with an " ✓". To comply with Mesa County Construction Stormwater Permit and the CDPS General Permit, all of the items identified must be adhered to.

**MESA COUNTY**
**POST-CONSTRUCTION STORMWATER CONTROLS AND**
**BEST MANAGEMENT PRACTICES**
**OPERATIONS AND MAINTENANCE AGREEMENT**

**THIS AGREEMENT**, made and entered into this ____________ day of _________, 200__, by and between ______________________________________, (hereinafter the "Landowner"), and Mesa County, Colorado (hereinafter "County");
**WITNESSETH**

**RECITALS**

The Landowner is the owner of certain real property described by a recorded deed in the land records of Mesa County, Colorado, Deed Book ___________ at Page ______, the "Property";

The Landowner is developing the Property;

The County, and the Landowner, his successors and assigns, agree that the health, safety, and welfare of the residents of the County and the protection and maintenance of water quality require that on-site stormwater Best Management Practices be constructed and maintained on the Property.

For the purposes of this agreement, the following definition shall apply:

BMP – "Best Management Practice;" activities, facilities, designs, measures or procedures used to manage stormwater impacts from land development, to protect and maintain water quality and groundwater recharge and to otherwise meet the purposes of the Municipal Stormwater Pollution Prevention Ordinance, including but not limited to infiltration trenches, seepage pits, filter strips, bioretention, wet ponds, permeable paving, grassed swales, forested buffers, sand filters and detention basins.

State of Colorado BMP definition: schedules of activities, prohibitions of practices, maintenance procedures, and other management practices to prevent or reduce pollution of waters of the State. BMPs also include treatment requirements, operating procedures, and practices to control site runoff, spillage or leaks, waste disposal, or drainage from material storage. Another definition from the State of Colorado: physical, structural, and/or managerial practices that, when used singly or in combination, prevent or reduce pollution of stormwater.

The County requires, through the implementation of this Agreement, that Post-Construction Storm Water Management BMPs as required by the Final Drainage Report, Stormwater Management Manual, and the Land Development Code be constructed and adequately operated and maintained by the Landowner, his successors and assigns;

NOW, THEREFORE, in consideration of the foregoing promises, the mutual covenants contained herein and the following terms and conditions, the parties hereto agree as follows:

1. The BMPs shall be constructed by the Landowner in accordance with the plans and specifications shown and described in the Final Drainage Report.

2. The Landowner shall operate and maintain the BMP(s) as shown and described on the Final Drainage Report in good working order acceptable to the County and in accordance with the specific maintenance requirements noted on the Final Drainage Report. Inspections of the BMP(s) shall be performed on an annual basis to ensure good working order of the BMP(s) with the inspection results sent to the County. Inspections must be performed by a qualified person, either a professional engineer or a person with training in the inspections of these BMP(s).

3. The Landowner hereby grants permission to the County, its authorized agents and employees, to enter upon the property, at reasonable times and upon presentation of proper identification to the Land Owner if requested, to inspect the BMP(s) whenever it deems necessary. The County shall notify the Landowner prior to entering the property.

4. In the event the Landowner fails to operate and maintain the BMP(s) as shown and described on the Final Drainage Report in good working order acceptable to the County, the County or its representatives may enter upon the Property and take whatever action is deemed necessary to maintain and/or reconstruct said BMP(s). This provision shall not be construed to allow the County to erect any permanent structure on the land of the Landowner. It is expressly understood and agreed that the County is under no obligation to maintain or repair said facilities, and in no event shall this Agreement be construed to impose any such obligation on the County.

5. In the event the County, pursuant to this Agreement, performs work of any nature, or expends any funds in performance of said work for labor, use of equipment, supplies, materials, and the like, the Landowner shall reimburse the County for all expenses (direct and indirect) incurred within 10 days of receipt of invoice from the County.

6. The intent and purpose of this Agreement is to ensure the proper maintenance of the onsite BMP(s) by the Landowner; provided, however, that this Agreement shall not be deemed to create or effect any additional liability of any party for damage alleged to result from or be caused by stormwater runoff.

7. The Landowner, its administrators, assigns, and other successors in interests, shall release the County's employees and designated representatives from all damages, accidents, casualties, occurrences or claims which might arise or be asserted against said employees and representatives from the construction, presence, existence, or maintenance of the BMP(s) by the Landowner or County. In the event that a claim is asserted against the County, its designated representatives or employees, the County shall promptly notify the Landowner and the Landowner shall defend, at his own expense, any suit based on the claim. If any judgment or claims against the County's employees or designated representatives shall be

allowed, the Landowner shall pay all costs and expenses regarding said judgment or claim.

This Agreement shall be recorded in the Mesa County, Colorado land records and shall constitute a covenant running with the Property and/or equitable servitude, and shall be binding on the Landowner, his administrators, executors, assigns, heirs and any other successors in interests, in perpetuity.

ATTEST:

***This section will be different based on whether the owner is an individual or an entity!***

WITNESS the following signatures and seals:

(SEAL) For Mesa County:

____________________________________

(SEAL) For the Landowner:

____________________________________

ATTEST:

______________________________ (Mesa County)

County of Mesa, Colorado

I, ________________________________________, a Notary Public in and for the County and State aforesaid, whose commission expires on the __________ day of __________________, 20__, do hereby certify that ________________________________________ whose name(s) is/are signed to the foregoing Agreement bearing date of the __________ day of __________________, 20__, has acknowledged the same before me in my said County and State.

**GIVEN UNDER MY HAND THIS** ____________ day of __________, 200_.

_________________________________ _____________________________________

**NOTARY PUBLIC** **(SEAL)**

| Grand Junction COLORADO | # Storm Water Construction Permit Application | Permit Number ______<br>Date Submitted______<br>Final Stabilization Date<br>__________ |
|---|---|---|

**COMMUNITY DEVELOPMENT DEPARTMENT**

250 N. 5TH STREET GRAND JUNCTION, COLORADO 81501

GENERAL INFORMATION (970) 244-1430 (FAX) 970-256-4031 www.gjcity.org

A Storm Water Construction Permit is required for all land disturbance activities equal to or greater than one (1) acre OR for land disturbance activities less than one (1) acre that are part of a larger common plan of development or sale that would disturb one (1) acre or more.

**NOTE: This permit is separate from any permits required by the Colorado Department of Public Health and Environment (CDPHE), Water Quality Control Division under the Colorado Discharge Permit System.**

| **(1) Project Information:** | | |
|---|---|---|
| Project Name: | City Project Number: | |
| Address/Location: | Current Mesa County Assessor's Tax Parcel Number: | |
| | | |
| Existing Land Use: | Proposed Land Use: | |
| **(2) Contact Information:** | | |
| **(a) Project Owner:** | | |
| Contact Person: | Company: | |
| Address: | | |
| Phone: | Email: | |
| Fax: | Mobile: | Pager: |
| **(b) Contractor (Site contact familiar with project)** | | |
| Contact Person: | Company: | |
| Address: | | |
| Phone: | Email: | |
| Fax: | Mobile: | Pager: |
| **(c) Other:** | | |
| Contact Person: | Company: | |
| Address: | | |
| Phone: | Email: | |
| Fax: | Mobile: | Pager: |
| **(3) Construction Site Operator (Permittee):** | | |
| Name:<br>(Print) | Check One:<br>☐ Phone; ☐ Mobile; ☐ Pager: | Number: __________ |

## (4) List, in addition to the permittee, any site personnel along with their contact information, who will have authority to make modifications to the Construction Stormwater Management Plan (CSWMP):

| | | |
|---|---|---|
| Name: | Check One: ☐ Phone; ☐ Mobile; ☐ Pager: | Number: ____________ |
| Name: | Check One: ☐ Phone; ☐ Mobile; ☐ Pager: | Number: ____________ |
| Name: | Check One: ☐ Phone; ☐ Mobile; ☐ Pager: | Number: ____________ |

## (5) Project Information:

| | | |
|---|---|---|
| Total area of project (acres): ____________ | Area of project to undergo disturbance (acres): ____________ | Total disturbed area of larger common plan or development or sale if applicable (acres): ____________ |

## (6) Nature of Construction Activities:

Check the appropriate box(es) or provide a brief description that indicates the general nature of the construction activities. (The full description of activities and construction phases must be included in the Construction Storm Water Management Plan.) ☐ Single Family Residential ☐ Multi-Family Residential ☐ Commercial / Industrial

☐ Utility ☐ Highway / Road ☐ Other (describe): ______________________________

Brief Description of Project:

## (7) Project Schedule:

| | | |
|---|---|---|
| Expected project start date: ____________ | Expected project completion date: ____________ | Expected final stabilization date: ____________ |

## (8) Acknowledgement Certificate:

By signing below, I hereby apply for a City of Grand Junction Storm Water Construction Permit for the aforementioned property and certify that I agree to follow this permit including but not limited to applicable conditions in the Storm Water Management Manual (SWMM) and the project-specific Construction Storm Water Management Plan (CSWMP).

______________________________ ____________

Signature of Legally Responsible Person (submission must include original signature) Date Signed

______________________________ ____________

Name (printed) Title

## *** FOR OFFICE USE ONLY ***

| | |
|---|---|
| CDPHE Stormwater Discharge Permit Received: ☐ No<br>☐ Yes, ____________ date received; Permit No. ____________ | City Approval: ____________ Date: ____________ |
| Stormwater Management Plan (CSWMP) and drawings received:<br>☐ No ☐ Yes, ____________ date received | Total Performance Security collected: $ ____________<br>Date Paid: ____________<br>Method of Payment: ____________ |
| Sediment/Erosion Control Plan Sheet Drawings Received:<br>☐ No ☐ Yes, ____________ date received; | File Number: ____________<br>Security date of expiration: ____________<br>DIA period of performance: ____________ |

STATE OF COLORADO

# NOTICE OF <u>REASSIGNMENT</u> OF PERMIT COVERAGE

## FOR A PORTION OF A PERMITTED AREA AND GENERAL PERMIT APPLICATION

## GENERAL PERMIT FOR STORMWATER DISCHARGES ASSOCIATED WITH CONSTRUCTION ACTIVITY

This form is to be used when a permittee under the State of Colorado General Permit for Stormwater Discharges Associated with Construction Activity (the Construction General Permit) no longer has control of a <u>specific portion</u> of a permitted site through either ownership or contract, and wishes to transfer coverage of that portion of the site to a second party that does not currently have coverage under the Construction General Permit. **If both parties currently have permit coverage, refer to the Alternative Options section for additional guidance.** This application covers Large Construction Sites (disturbing 5 or more acres) and Small Construction Sites (disturbing at least 1 but less than 5 acres). Sites that are part of a larger common plan of development or sale that will disturb at least 1 acre are also included. A "common plan of development or sale" is a site where multiple separate and distinct construction activities may be taking place at different times on different schedules, but still under a single plan. This includes phased projects, projects with multiple filings or lots, and projects in a contiguous area that may be unrelated but still under the same contract

The following actions will be triggered by completion and submittal of this form:

- The Permit certification in Item II.A will be amended to no longer include the area described in Items I.B and I.C. **The current permittee will <u>not</u> receive a revised certification**. The corrected information will be placed in the permit file. If the current permittee requires notification of the Water Quality Control Division's (the Division's) receipt of this information, it is up to the permittee to request verification of delivery from the carrier (i.e., by sending certified mail).
- A new permit certification under the General Permit for Stormwater Discharges Associated with Construction Activity will be issued to the *new permittee* listed in Part I. A new permit certification and permit materials will be sent to the attention of the legally responsible person for the new permittee (Item I.i. on the form).

**Alternative Options:** To transfer the <u>entire permit certification</u>, the Notice of Transfer of Permit Coverage form must be used instead. Also, if both parties are currently permit holders for portions of the overall project (i.e., at least two permit certifications are issued for the project and cover both the party wishing to reassign coverage and the party wishing to accept coverage), and it is not desired that a new permit certification be issued, the permittees may <u>amend their permit certifications</u> instead of completing this form. For additional instructions, please refer to the guidance on "Amending Your Permit Certification" in the Stormwater Fact Sheet for Construction, available from the Division's web site at www.cdphe.state.co.us/wq/PermitsUnit.

**Applicant Liability:** For the area described for which permit coverage is to be reassigned (Part I.b of the form), permit coverage and liability will be assigned to the new permit applicant, regardless of ownership or contract, until the certification is transferred, amended, inactivated, or expired. <u>(Note that even though the new permittee will be liable for permit compliance for the reassigned area, if the new permittee does not meet the definition of owner or operator as discussed in the "Who May Apply" section below, the entity(s) that does could be held liable for operating without the necessary permit coverage.)</u> The permittee may amend their application in writing to cease coverage for a portion of a site that the permittee no longer either owns or is under contract for. Information on the procedures that must to be used for such amendments will be supplied with the permit certification.

A more detailed explanation is available in the Stormwater Fact Sheet for Construction, available from the Division's web site at www.cdphe.state.co.us/wq/PermitsUnit. **The Division <u>strongly recommends</u> that entities involved with sites where multiple owners/operators exist review this information.**

**Who May Apply For and Maintain Permit Coverage:** The applicant must be a legal entity that meets the definition of <u>either the owner and/or operator</u> of the construction site, in order for this application to <u>legally cover the activities occurring at the site</u>. The applicant must have day-to-day supervision and control over activities at the site and implementation of the Stormwater Management Plan (SWMP) discussed in Item I.h of the instructions. Although it is acceptable for the applicant to meet this requirement through the actions of a contractor, as discussed in the examples below, the applicant remains liable for violations resulting from the actions of their contractor. Examples of acceptable applicants include:

- **Owner or Developer**- An owner or developer who is operating as the site manager or otherwise has supervision and control over the site, either directly or through contract with an entity such as those listed below.
- **General Contractor or Subcontractor** - A contractor with contractual responsibility and operational control (including SWMP implementation) to address the impacts construction activities may have on stormwater quality.
- **Other Delegated Agents/Contractors** - Other agents, such as a consultant acting as construction manager under contract with the owner or developer, with contractual responsibility and operational control to address the impacts construction activities may have on stormwater quality (including SWMP implementation).

An entity engaged in construction activities may be held liable for operating without the necessary permit coverage if a site does not have a permit certification in place that is issued to either an owner and/or operator. For example, if a site, or portion of a site, is sold or contractors change, so that the site's permit certification is then held by a permittee that is no longer either the owner or operator (such as the previous owner or contractor), that permit certification will no longer cover the new operator's activities, and a new certification must be issued, or the current certification transferred.

A separate permit certification is not needed for subcontractors, such as utility service line installers, whose activities result in earth disturbance, but where the permittee or their contractor is identified as having the operational control to address the impacts their activities may have on stormwater quality.

**Stormwater Management Plan (SWMP):** Both the current permittee and the new applicant must develop and maintain SWMPs that accurately reflect the activities and BMPs for the areas for which they will have permit coverage. The **current permittee** must already have a SWMP in place as required by their permit coverage, but must revise this plan to account for the changes in their covered area and activities. As discussed in Item I.h. of the instructions, the **new applicant** must also develop a SWMP for their covered area, prior to submittal of this form. Although the new applicant may utilize the current permittee's SWMP, if available, in development of their own SWMP, it must be modified and maintained to accurately reflect the new applicant's activities. Appendix A of the General Permit Application and Stormwater Management Plan Guidance for Stormwater Discharges Associated with Construction Activity (available from the Division's web site at www.cdphe.state.co.us/wq/PermitsUnit) contains the requirements for the SWMP. Failure by either entity to maintain a SWMP in accordance with this guidance is a violation of the permit. Additional guidance for multi owner/operator development is also available in the Stormwater Fact Sheet for Construction, also available from the Division's web site.

**Application Due Dates:** At least **ten days** prior to the requested effective date for the coverage reassignment, the owner or operators of the construction activities shall submit this form to the Water Quality Control Division (the "Division"). This form may be reproduced, and is also available from the Division's web site at www.cdphe.state.co.us/wq/PermitsUnit.

**Permit Fee:** The new permittee does not send any payment with this form. They will be billed once they are covered under a permit. Current permit fees can be obtained from the Division's web site at www.cdphe.state.co.us/wq/PermitsUnit.

**Application Completeness:** All items of the form must be completed accurately and in their entirety or the application will be deemed incomplete, and processing of the form will not begin until all information is received. (Do not include a copy of the Stormwater Management Plans for either the current or new permittee, unless requested by the Division.) One original copy of the completed form (**no faxes or e-mails**), signed by both the new permittee and the current permittee, shall be submitted, only to:

Colorado Department of Public Health and Environment
Water Quality Control Division
WQCD-Permits
4300 Cherry Creek Drive South
Denver, Colorado 80246-1530

**Note: Partial applications will not be accepted.**

If you have questions on completing this application, you may contact the Division at cdphe.wqstorm@state.co.us or (303) 692-3517.

# REASSIGNMENT FORM

## INSTRUCTIONS

**PART I - (To be completed by the New permit applicant.)**

Part I of the application is to be filled out by the new permit applicant who will be assuming permitting liability for the reassigned portion of the original applicant's site. The **effective date** when the area addressed in the form will be reassigned must be provided at the beginning of Part I. The effective date must be at least ten days following the receipt of this form by the Division. It will be the effective date of the new permit certification issued to the new applicant in Part I.

**Item I.a.**: Provide the name and address of the new permit applicant, including the company name, local contact, phone number, and mailing address. Indicate whether the applicant is the owner, developer or contractor.

**Item I.b.**:. Provide the street address of the site that will be covered under the new certification and removed from the current certification. If an exact address is not available you may use an approximate address, the nearest intersection or boundary streets including directional identifiers (e.g., "S. of Park St. between 5th Av and 10th Av.", or "W. side of C.R. 21, 3.25 miles N. of Hwy 10") or other identifying information. For the approximate center point of the property, the longitude/latitude, to the nearest 15 seconds, **must** be included. The latitude and longitude must be provided as either degrees, minutes, and seconds, or in decimal degrees with three decimal places. This information may be obtained from a variety of sources, including:

- Surveyors or engineers for the project should have, or be able calculate, this information.
- EPA maintains a web-based siting tool as part of their Toxic Release Inventory program that uses interactive maps and aerial photography to help users get latitude and longitude. The siting tool can be accessed at: www.epa.gov/tri/report/siting_tool/index.htm
- U.S. Geological Survey topographical map, available at area map stores.
- Using a Global Positioning System (GPS) unit to obtain a direct reading.

**Item I.c.**: One of these two items **must** be provided:

- Legal Description of the entire site covered by the application that must include subdivision(s), block(s), and lot(s) (providing the metes and bounds or just the township/section/range, is not adequate). This information should be available for subdivided properties from documents submitted to or maintained by the city or county, such as the subdivision plat or deed. If this information is not available, a map must be submitted. **– or –**
- Site Map that defines the boundaries of the site being applied for. The level of detail that must be provided will depend on the nature of the project and must be adequate so that it can be determined during a field audit what construction activities are covered under the issued certification. For typical developments within a specific surveyed property, a map clearly showing the property boundaries should be obtainable. For projects located in areas with adjacent construction areas that will not be covered by the application (such as multi-lot developments with multiple owners/operators), this detail is essential. However, for projects such as road or utility projects, where providing this detail may not be feasible or necessary to distinguish the project from adjacent activities, a less detailed map showing the approximate area is adequate. Maps must have a minimum scale of 1:24000 (the scale of a USGS 7.5 minute map). Maps must be folded to 8½ x 11 inches. Do not submit grading plans or other blueprints as the site map.

**Item I.d.**: Provide both the total area of the construction site that will be covered under the new certification, and the area that will undergo disturbance, in acres. **Note:** aside from clearing, grading and excavation activities, disturbed areas also include areas receiving overburden (e.g., stockpiles), demolition areas, and areas with heavy equipment/vehicle traffic and storage that disturb existing vegetative cover.

**Item I.e.**: Either check the appropriate box or boxes, or if the given descriptions do not fit the project, provide a brief description that indicates the general nature of the construction activities for which permit coverage is being requested. A more detailed description of the project must be included in the Stormwater Management Plan (see Item I.i.).

**Item I.f.**: Provide the current estimated completion date for the construction project to be covered under the new certification. In terms of permit coverage, the completion date is when the site is finally stabilized. This means that all disturbed areas have been either built on, paved, or a uniform vegetative cover has been established. **Permit coverage must be maintained until that time.** If permit coverage is still required once your part is completed, the permit must be transferred to a new responsible entity.

**Item I.g.**: Identify the receiving water. Receiving waters are any waters of the State of Colorado. These include any and all surface waters that are contained in or flow in or through the State of Colorado (except for water withdrawn for use until use and treatment have been completed). This definition includes all water courses, even if they are usually dry. If stormwater from the construction site enters a ditch or storm sewer system, identify that system and indicate the ultimate receiving water for the ditch or storm sewer. **Note:** a stormwater discharge permit does not allow a discharge into a ditch or municipal storm sewer system without the approval of the owner/operator of that system if such approval is otherwise needed.

**Item I.h.**: The certification of completion of a Stormwater Management Plan (SWMP) **must** be signed by the applicant or their authorized agent. Appendix A of the General Permit Application and Stormwater Management Plan Guidance for Stormwater Discharges Associated with Construction Activity (available form www.cdphe.state.co.us/wq/PermitsUnit) contains the requirements for the SWMP during the period of construction (as listed in the general permit). Submittal of the SWMP is **not** required, however it must be developed and implemented and kept at the construction site. The Division reserves the right to request the SWMP at any time.

**Item I.i.**: The applicant must be either the owner and/or operator of the construction site. Refer to page i of the instructions for additional information. The application **must be signed** to be considered complete. In all cases, it shall be signed as follows:

a) In the case of corporations, by a principal executive officer of at least the level of vice-president or his or her duly authorized representative, if such representative is responsible for the overall operation of the facility from which the discharge described in the application originates.
b) In the case of a partnership, by a general partner.
c) In the case of a sole proprietorship, by the proprietor.
d) In the case of a municipal, state, or other public facility, by either a principal executive officer, ranking elected official, or other duly authorized employee.

**This certification includes an acknowledgment that the applicant understands that the permit coverage, and therefore the applicant's liability, will be for the entirety of the portion of the construction project described and applied for, until such time as the application is amended or the certification is transferred, inactivated, or expired.**

**PART II - (To be completed by the Current permittee.)**

Part II of the application, starting on page 3 of the form, is to be completed by the **current** permittee. The information provided in Part II.b should be revised from the original application to take into account the changes in the site based on reassignment of the applied-for area. Refer to the instructions for I.c and I.d above for information on providing location and construction activity description information.

NOTICE OF
**REASSIGNMENT OF PERMIT COVERAGE**
and GENERAL PERMIT APPLICATION

STORMWATER DISCHARGES ASSOCIATED WITH
**CONSTRUCTION ACTIVITY** (Permit No. COR-030000)

For Agency Use Only **C O R - 0 3** ____ ____ ____ ____

Date Received: ___ ___ / ___ ___ / ___ ___ ___ ___
Day Month Year

**9A**

**Please print or type**. All items must be completed accurately and in their entirety or the application will be deemed incomplete and processing of the permit/reassignment will not begin until all information is received. Please refer to the instructions for information about the required items. Original signatures of the new permit applicant and the current permittee are **required.**

**Part I** of the application beginning below is to be filled out by the new permit applicant that will be assuming permitting liability for the reassigned portion of the original applicant's site. **Part II** of the application, starting on page 3 of the form, is to be completed by the current permittee. **Both Parts I (pages 1 and 2) and II (page 3) must be completed**.

**PART I - To be completed by the New permit applicant:**

I hereby accept the reassignment of permit coverage for the area described in this application. I have reviewed the terms and conditions of this permit and the Stormwater Management Plan and accept full responsibility, coverage and liability.

**This reassignment will be effective on:** __ __ /__ __ / __ __ __ __
month day year

**I.a. Name and address of the new permit applicant**:

Company Name ______________________________

Mailing Address ______________________________

City, State and Zip Code ______________________________

Phone No. (_____) ____________ Who is applying? Owner Developer Contractor

Local Contact (familiar with facility) ______________________________

Title ____________________ Phone Number (_____) ____________

Local Contact E-mail Address (if available) ______________________________

Legally Responsible Person (application signer) E-mail Address ______________________________

**I.b. Location of the area to be covered under the new permit certification:**

Street Address (or Cross Streets) ______________________________

City (if unincorporated, so indicate) ____________________ County ____________________

Name of plan, project, or development ______________________________

Latitude/Longitude – use one of the following formats:

Latitude ___ ___ / ___ ___ / ___ ___ Longitude ___ ___ ___ / ___ ___ / ___ ___ (e.g., 39°42'11'', 104°55'57'')
degrees minutes seconds degrees minutes seconds

-or-

Latitude ___ ___ . ___ ___ ___ Longitude ___ ___ ___ . ___ ___ ___ (e.g., 39.703°, 104.933°')
degrees (to 3 decimal places) degrees (to 3 decimal places)

**I.c. Legal Description (subdivision, block, and lot) or Map Indicating Site Location/Boundaries:**

If a map is attached to provide this information, this must be indicated below. Maps must be folded to 8½ x 11 inches.

Map Attached? Yes, skip to item 4 No; include legal description **per Instructions** (use separate sheet if needed):

Subdivision(s), Lot(s), Block(s): ______________________________


**Form Continued on Next Page**

**I.d.** **Area of the construction site**:

Total area of project site (acres) ______________________________

Area of project site to undergo disturbance (acres) ______________________________

**I.e.** **Nature of the construction activity**:

Check the appropriate box(s) or provide a brief description that indicates the general nature of the construction activities. (The full description of activities must be included in the Stormwater Management Plan.)

Single Family Residential Development

Multi-Family Residential Development

Commercial Development

Other, Describe:______________________________

______________________________

**I.f.** **Anticipated Final Stabilization Date**: __ __ /__ __ / __ __ __ __
month day year

**I.g.** **The name of the receiving stream(s).** (If discharge is to a ditch or storm sewer, also include the name of the ultimate receiving water): ______________________________

| **STOP!** A Stormwater Management Plan must be completed prior to signing the following certifications! |
|---|

**I.h.** **Stormwater Management Plan Certification (new applicant)**:

"I certify under penalty of law that a **complete Stormwater Management Plan, as described in Appendix A of the General Permit Application and Stormwater Management Plan Guidance for Stormwater Discharges Associated with Construction Activity, has been prepared for my activity**. Based on my inquiry of the person or persons who manage the system, or those persons directly responsible for gathering the information, the Stormwater Management Plan is, to the best of my knowledge and belief, true, accurate, and complete. I am aware that there are significant penalties for falsely certifying the completion of said SWMP, including the possibility of fine and imprisonment for knowing violations."

______________________________

Signature of Legally Responsible Person (submission must include original signature) Date Signed

______________________________

Name (printed) Title

**I.i** **Signature of Applicant (legally responsible person - new applicant)**

"I certify under penalty of law that I have personally examined and am familiar with the information submitted in this application and all attachments and that, based on my inquiry of those individuals immediately responsible for obtaining the information, I believe that the information is true, accurate and complete. I am aware that there are significant penalties for submitting false information, including the possibility of fine or imprisonment.

"I understand that submittal of this application is for coverage under the State of Colorado General Permit for Stormwater Discharges Associated with Construction Activity **for the entirety of the construction site/project described and applied for, until such time as the application is amended or the certification is transferred, inactivated, or expired**."

______________________________

Signature of Legally Responsible Person (submission must include original signature) Date Signed

______________________________

Name (printed) Title

 **Form Continued on Next Page**

**PART II – Amendment to the current permit certification**
**To be completed by the Current permittee:**

**II.a. Current Permit Certification Information**

Certification Number: **COR-03** ___ ___ ___ ___

Check if this is a new name, address, etc.

Permittee (Company) Name: ______________________________

Permittee Address: ______________________________

______________________________

Local Contact: ______________________________

Phone No. (_____) ______________________________

E-mail Address (if available) ______________________________

Name of plan, project, or development ______________________________

**II.b. Revised Site Information**

Legal Description or Map Indicating Site Boundaries: (Subdivision, block, and lot, or similar information clearly identifying the site area that retains coverage under the current certification. If a map is attached to provide this information, this must be indicated below. Maps must be folded to 8½ x 11 inches.)

Map Attached? Yes No; include legal description **per Instructions** (use separate sheet if needed):

Subdivision(s), Lot(s), Block(s): ______________________________

______________________________

Check the appropriate box(s) or provide a brief description that indicates the general nature of the construction activities. (The full description of activities must be included in the Stormwater Management Plan.)

Single Family Residential Development

Multi-Family Residential Development

Commercial Development

Other, Describe: ______________________________

______________________________

**II.c. Certification for Reassignment**

"I certify under penalty of law that I have personally examined and am familiar with the information submitted in Part II of this application and all attachments in reference to Part II and that, based on my inquiry of those individuals immediately responsible for obtaining the information, I believe that the information is true, accurate and complete. I am aware that there are significant penalties for submitting false information, including the possibility of fine or imprisonment.

"As the permittee currently covered by the above-referenced certification, I hereby agree to reassign the permit coverage for the area and activity described in Items I.b. and I.c., and all responsibilities thereof, from the above-referenced permit certification to the new permittee listed in Part I of this form."

______________________________

Signature of Legally Responsible Person (submission must include original signature) Date Signed

______________________________

Name (printed) Title Title

**POST-CONSTRUCTION STORMWATER CONTROL**
**OPERATIONS AND MAINTENANCE AGREEMENT**

**THIS AGREEMENT** is made and entered into this ____________ day of _________, 20____, by and between _____________________________________________, (hereinafter the "Landowner"), and the City of Grand Junction, Colorado (hereinafter "Municipality");

**RECITALS**

The Landowner is the owner of the following real property:

hereinafter referred to as "the Property."

The Landowner is developing the Property;

The Municipality and the Landowner, on behalf of all successors and assigns, agree that the health, safety, and welfare of the residents of the Municipality and the protection and maintenance of water quality require that on-site stormwater Best Management Practices be constructed and maintained on the Property.

For the purposes of this agreement, the following definitions shall apply:

BMP – "Best Management Practice;" activities, facilities, designs, measures or procedures used to manage stormwater impacts from land development, to protect and maintain water quality and groundwater recharge and to otherwise meet the purposes of the Municipal Stormwater Pollution Prevention Ordinance, including but not limited to infiltration trenches, seepage pits, filter strips, bioretention, wet ponds, permeable paving, grassed swales, forested buffers, sand filters and detention basins.

As part of the approval of the development of the property, the Municipality will review and approve a Final Drainage Report which will implement Post-Construction Storm Water Management BMPs required by the Municipal Stormwater Pollution Prevention Ordinance.

The purpose of this Agreement is to insure the adequate maintenance, operation and repair of the storm water management facilities, in perpetuity, by the owners of the property served by these facilities.

The Landowner desires to subject and place upon the Property the covenants and servitudes set forth herein which shall run with the Property and be binding on all parties having any right, title, or interests in the Property or any part thereof, their heirs, personal representatives, successors and assigns, and shall inure to the benefit of each owner thereof.

NOW, THEREFORE, in consideration of the foregoing promises, the mutual covenants contained herein and the following terms and conditions, the parties hereto agree as follows:

1. The BMPs shall be constructed by the Landowner in accordance with the plans and specifications shown and described in the Final Drainage Report.

2. The Landowner shall operate and maintain in perpetuity the BMP(s) as shown and described on the Final Drainage Report in good working order as reasonably determined by the Municipality and in accordance with the specific maintenance requirements noted on the Final Drainage Report. The Landowner shall cause the BMP(s) to be inspected annually by a Qualified Erosion Control Specialist to ensure good working order and shall send a report from said inspection to the Municipality annually, on or before December 31st of each year.

3. The Landowner hereby grants a perpetual easement to the Municipality, its authorized agents and employees, to enter upon the Property, at reasonable times and upon presentation of proper identification, to inspect the BMP(s) whenever it deems necessary. The Municipality shall make reasonable efforts to notify the Landowner prior to entering the Property.

4. In the event the Landowner fails to operate and maintain the BMP(s) as shown and described on the Final Drainage Report in good working order as reasonably determined by the Municipality, the Municipality or its representatives may enter upon the Property and take action to maintain and/or repair and/or reconstruct said BMP(s). It is expressly understood and agreed that the Municipality is under no obligation to maintain or repair said facilities, and in no event shall this Agreement be construed to impose any such obligation on the Municipality.

5. In the event the Municipality, pursuant to this Agreement, performs work of any nature, or expends any funds in performance of said work for labor, use of equipment, supplies, materials, and the like, the Landowner shall reimburse the Municipality for all reasonable expenses (direct and indirect) incurred within 10 days of receipt of invoice from the Municipality.

6. The intent and purpose of this Agreement is to ensure the proper maintenance of the onsite BMP(s) by the Landowner. This Agreement shall not be deemed to create or effect any additional liability of any party for damage alleged to result from or be caused by stormwater runoff.

7. The Municipality may conduct routine inspections of the BMP(s) to verify their continued adequate functioning. The Municipality may also inspect the BMP(s) in the event of reported or suspected failure to function adequately. These inspection activities shall not absolve the Landowner of its obligation to maintain the BMPs in perpetuity or to provide the Municipality with the required Landowner inspection report.

8. This Agreement shall not be interpreted or deemed to limit the authority, privilege or right of the Municipality pursuant to any duly enacted ordinance of the Municipality, charter provision, statute or any duly granted federal or state water discharge permit.

9. Notifications and reports made under this Agreement shall be provided to the City at:

Mr. Chris Spears
Street Systems Supervisor
250 N 5th Street-
Grand Junction, CO 81501

and to the Landowner at:

This Agreement shall be recorded in the Mesa County, Colorado land records and shall, once recorded, constitute a covenant running with the Property and shall be an equitable servitude binding on present and subsequent owners of the Property in whole or in part, and their administrators, executors, assigns, heirs and successors in interest, in perpetuity.

ATTEST:

WITNESS the following signatures and seals:

(SEAL) For the City of Grand Junction:

______________________________________________

(SEAL) For the Landowner:

______________________________________________

ATTEST:

______________________________ (City of Grand Junction)

County of Mesa, Colorado

I, ________________________________________, a Notary Public in and for the County and State aforesaid, whose commission expires on the __________ day of __________________, 20__, do hereby certify that ________________________________________ whose name(s) is/are signed to the foregoing Agreement bearing date of the ___________ day of ___________________, 20__, has acknowledged the same before me in my said County and State.

**GIVEN UNDER MY HAND THIS** _____________ day of ___________, 200_.

_________________________________ _____________________________________

**NOTARY PUBLIC** **(SEAL)**

# CITY OF GRAND JUNCTION
# STORMWATER CONSTRUCTION PERMIT FIELD INSPECTION REPORT

(1) Date of Inspection

(2) Property Owner

(3) Site Contact Name (print)

(4) Inspector Name (print)

(5) Project Name

(6) Permit No.

(7) Reason for Inspection :

- ☐ Required Maximum 14 Calendar Day Inspection
- ☐ Required 30 Calendar Day Inspection for Completed Projects
- ☐ Required Storm Event Inspection
- ☐ Complaint:____________________
- ☐ Other:____________________

## (8) CONSTRUCTION SITE ASSESSMENT

- ☐ Construction site perimeter contained. Offsite tracking minimized.
- ☐ Disturbed areas contained.
- ☐ Estimate disturbed area at the time of the inspection. ______ Acres
- ☐ Areas used for material and waste storage and fueling contained.

## (9) CSWMP MANAGEMENT

- ☐ Does site have current CSWMP Yes / No
- ☐ Changes noted & dated on the plans? Yes / No
- ☐ Contents of CSWMP:
  - ☐ Site Description Yes / No
  - ☐ Site Map Yes / No
  - ☐ Description of BMPs during construction activities Yes / No
  - ☐ Materials Handling & Spill Prevention Yes / No
- ☐ Contents of CSWMP (Cont.)
  - ☐ Final Stabilization Yes / No
  - ☐ Other Controls Yes / No
  - ☐ Inspection & Maintenance Yes / No
  - ☐ Certification Yes / No

## BEST MANAGEMENT PRACTICES (BMPs)

| (10) BMP Type | (11) Practice Req/Used | | (12 ) Reason | (13) Maintenance/ Sediment Removal Required Y/N | (14) Course of Action | (15 ) Date for Action to be Completed |
|---|---|---|---|---|---|---|
| **EROSION CONTROL** | | | | | | |
| Seeding | | | | | | |
| Mulching | | | | | | |
| Blankets | | | | | | |
| Check Dams | | | | | | |
| Earth Berms | | | | | | |
| Diversion | | | | | | |
| Embankment Protector | | | | | | |
| Outlet Protection | | | | | | |
| Surface Roughening | | | | | | |
| Other: | | | | | | |
| Other: | | | | | | |

| (10) **BMP Type** | (11) **Practice Req/Used** | | (12) **Reason** | (13) **Maintenance/ Sediment Removal Required Y/N** | (14) **Course of Action** | (15) **Date for Action to be Completed** |
|---|---|---|---|---|---|---|
| **SEDIMENT CONTROL** | | | | | | |
| Inlet Protection | | | | | | |
| Erosion Bales | | | | | | |
| Silt Fence | | | | | | |
| Sediment Trap/Basin | | | | | | |
| Stabilized Construction Entrance | | | | | | |
| Dewatering Structure | | | | | | |
| Other: | | | | | | |
| **MATERIALS HANDLING AND SPILL PREVENTION, WASTE MANAGEMENT AND GENERAL POLLUTION PREVENTION** | | | | | | |
| Stockpile Management | | | | | | |
| Materials Delivery and Storage | | | | | | |
| Spill Prevention and Control | | | | | | |
| Concrete Washout | | | | | | |
| Concrete Saw Water Containment | | | | | | |
| Solid Waste | | | | | | |
| Sanitary Waste | | | | | | |
| Maintenance and Fueling | | | | | | |
| Street Sweeping Vacuuming | | | | | | |
| Other: | | | | | | |
| Other: | | | | | | |

Comments: __________________________________________

| (16) **INSPECTIONS AND MAINTENANCE PROGRAM** | |
|---|---|
| ❑ Inspection occurring at least every 14 calendar days. | Course of Action: |
| ❑ Inspections occurring after storm events that result in runoff. | Course of Action: |
| ❑ Inspections occurring at least every 30 calendar days since project completion. | Course of Action: |
| ❑ Inspection reports retained at the construction project site. | Course of Action: |
| ❑ Corrective measures completed within 7 calendar days of inspection. | Course of Action: |

| **CERTIFICATION**<br>**I certify this Stormwater Construction Permit Field Inspection Report is complete and accurate.** | |
|---|---|
| Inspector Signature | Date |

# Stormwater Management Plan Field Inspection Report
# Instructions

1. **Date of Inspection:** Indicate the date the inspection was completed on.
2. **Property Owner:** Indicate the name of the property owner.
3. **Site Contact:** Indicate the main contact for the site.
4. **Inspector Name**: Indicate the name of the Inspector completing the inspection.
5. **Project Name**: Indicate the name of the project for which the report is being completed.
6. **Permit No.**: Indicate the permit number issued with the approval of the permit.
7. **Reason for Inspection**: Indicate the purpose for the inspection. The types of inspections include the following:
   - "Required 14 Calendar Day Inspection". These inspections are required at least every 14 calendar days during the life of the construction project.
   - "Required 30 Day Inspection for Completed Projects". These inspections are required at least every 30 calendar days following the completion of the construction project where final stabilization has not been achieved.
   - "Required Storm Event Inspection". These inspections are required after a storm event that results in runoff.
   - Inspection as a response to a complaint.
   - Inspection for any other reason.

   The first three types of inspections are required to comply with Mesa County Construction Stormwater Permit and the Colorado Discharge Permit System General Permit for Stormwater Discharges Associated with Construction Activity (CDPS General Permit).
8. **Construction Site Assessment**: Inspect the noted areas of the construction site and indicate with a "✓" the items which apply.
   - "Construction site perimeter contained". Are the appropriate BMPs in place and offsite sediment tracking minimized? Is there any evidence of pollutants entering a storm drainage system?
   - "Disturbed areas contained". Are the appropriate BMPs implemented to minimize erosion or sediment tracking from the disturbed areas? Is there any evidence of pollutants entering a storm drainage system? Provide an estimate of the disturbed area at the time of the inspection.
   - "Areas used for material and waste storage and fueling contained". Are the appropriate BMPs implemented to prevent and contain spills? Are wastes removed from the site and disposed of properly? Are the storage areas located at least 50 feet from a watercourse? Is there any evidence of pollutants entering a storm drainage system?
9. **CSWMP Management**:
   - Indicate if the Construction site has a current and update CSWMP, by circling yes or no.
   - Indicate if the required contents of the CSWMP are included, by circling yes or no on each line.
   - Indicate whether changes have been made to the CSWMP during construction and whether the changes have been documented and dated.
10. **BMP**: The BMPs shown may not be a complete list of what is required by the CSWMP. Cross out the BMPs not required by the CSWMP and add the BMPs that are required. Additional sheets can be inserted to show all the BMPs required by the CSWMP.
11. **Practice Req/Used**: This column can be used as follows:
    - If the BMP is required by the CSWMP and implemented, indicate by placing a "✓ " in both the "Req" and "Used" columns.
    - If the BMP is required by the CSWMP, but not implemented, indicate by placing a "✓ " in the "Req" column. Indicate the reason for the change in column (14), "Reason".
    - If the BMP has been added to the CSWMP, indicate with a "✓ " in the "Used " column. Indicate the reason for the change in column (14), "Reason".
12. **Reason**: Indicate the reason(s) for the deletion, addition, and modification of BMP(s) to the SWMP.
13. **Maintenance/Sediment Removal Required**: Indicate whether maintenance and sediment removal are required with a Yes or No. If maintenance and sediment removal are required, indicate what the action plan is in column (15), "Course of Action".
14. **Course of Action**: If maintenance and/or sediment removal is required, describe the action plan.
15. **Date for Action to be Completed**: Indicate the date for which the course of action will be completed. The course of action must be completed in a timely manner, but in no case more than 7 days after the inspection.
16. **Inspections and Maintenance**: Evaluate the inspection and maintenance aspect of the construction project and check all that applies with an " ✓". To comply with Mesa County Construction Stormwater Permit and the CDPS General Permit, all of the items identified must be adhered to.

# SECTION 1800

## REFERENCES

# SECTION 1800
## REFERENCES

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# APPENDIX A

## DRAINAGE LAW

# APPENDIX A
## DRAINAGE LAW

## TABLE OF CONTENTS

# APPENDIX A
# DRAINAGE LAW

## A101 INTRODUCTION

The materials contained in this appendix are not intended to be an exhaustive presentation of each area of law which is discussed. The purpose is to familiarize the design professionals with these areas to enable them to better perform engineering duties and tasks contained in this Manual. 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, is that stormwater does not respect arbitrary jurisdictional boundaries. Stormwater does not respect the various rights and liabilities of adjacent land owners as it flows through depressions, gullies, and washes seeking refuge. 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 appendix includes a brief description and citing of applicable state and local laws. The contents relies heavily on similar section included in the Colorado Floodplain and Stormwater Criteria Manual prepared for the Colorado Water Conservation Board. The reader is encouraged to read the Drainage Law section of the UD&FCD's Drainage Criteria Manual, which represents the geneses for the update by the Colorado Water Conservation Board.

The user of this Manual is encouraged to check the applicability of the laws and court cases included herein at the time of its use. Applicable statues, ordinances, court cases and other local laws change with time. The materials contained herein should be reviewed periodically and updated as deemed appropriate.

## A102 HISTORICAL EVOLUTION OF SURFACE WATER DRAINAGE 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.

### A102.1 The Common Enemy Doctrine

The common enemy doctrine is a harsh rule which is still followed in some states. Stated in its extreme form, the common enemy doctrine provides that as an incident to property use each landowner has an unqualified right, by operations on the land, to fight off surface waters as necessary 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.

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.

### A102.2 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 its 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 associated with 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".

### A102.3 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 its 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 his 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 his 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?

## A103 LEGAL PRINCIPLES

This section has been reproduced from the Colorado Floodplain and Stormwater Criteria Manual prepared for the Colorado Water Conservation Board.

a. The owner of upstream property possesses a natural easement on land downstream for drainage of surface water flowing in its natural course. The upstream property owner may alter drainage conditions so long as the water is not sent down in a manner or quantity to do more harm to the downstream land than formerly. Bittersweet Farms, Inc. v. Zimbelman, 976 P.2d 326 (Colo. App. 1998).

b. For purposes of determining liability in a negligence action, the duty of a public entity shall be determined in the same manner as if it were a private party. Leake v. Cain, 720 P.2d 152 (Colo. 1986).

c. A natural watercourse may be used as a conduit or outlet for the drainage of lands, at least where the augmented flow will not tax the stream beyond its capacity and cause flooding of adjacent lands. Ambrosio v. Pearl-Mack Construction Co., 351 P.2d 803 (Colo. 1960).

d. Ditch corporations that own ditches owe a duty to those property owners through which their ditches pass to maintain their ditches using ordinary care so as to prevent damage to adjoining real property. Oliver v. Amity Mut. Irrigation Co., 994 P.2d 495 (Colo. App. 1999).

e. Construction or enlargement of jurisdictional dams or reservoirs is subject to approval by the Colorado State Engineer, which includes consideration of requiring their spillways to be capable of passing the inflow design flood generated by 100 percent of the probable maximum precipitation. A "jurisdictional dam" is defined as a dam that impounds water above the elevation of the natural surface of the ground creating a reservoir with a capacity of more than 100 acre-feet or creating a reservoir with a surface area exceeding 20 acres at the high waterline or exceeding 10 feet in height measured vertically from the elevation of the lowest point of the natural surface of the ground where that point occurs along the longitudinal centerline of the dam up to the flow line crest of the emergency spillway of the dam. Rules 4 & 5 of the Department of Natural Resources, Division of Water Resources, Office of the State Engineer, Rules and Regulations for Dam Safety and Dam Construction.

f. The boundaries of the floodplain should be accurately determined and based on a reasonable standard. Mallett v. Mamarooneck, 125 N.E. 2d 875 (N.Y. 1955).

g. Adoption of a floodplain regulation to regulate flood-prone areas is a valid exercise of police power and is not a taking as long as the regulation does not go beyond protection of the public's health, safety, morals, and welfare. Hermanson v. Board of County Commissioners of Fremont, 595 P.2d 694 (Colo. App. 1979).

h. The adoption by a municipality of floodplain ordinances to regulate flood-prone areas is a valid exercise of police power and is not a taking. Morrison v. City of Aurora, 745 P.2d 1042 (Colo. App. 1987).

i. A zoning ordinance is not unconstitutional because it prohibits a landowner from using or developing his land in the most profitable manner. It is not required that a landowner be permitted to make the best, maximum or most profitable use of his property. Baum v. City and County of Denver, 363 P.2d 688 (Colo. 1961) and Sundheim v. Board of County Commissioners of Douglas County, 904 P.2d 1337 (Colo. App. 1995).

j. The safest approach to avoiding liability in regard to drainage and flood control improvements is to assume that the defense of a design error will not protect a governmental entity from a lawsuit and liability for injury to property or person. Scott v. City of Greeley, 931 P.2d 525 (Colo. App. 1996) and 24-10-106 (1)(e) and (f) C.R.S.

k. A "dangerous condition" constitutes an unreasonable risk to the health or safety of the public, which is known to exist or which in the exercise of reasonable care should have been known to exist and which condition is proximately caused by the negligent act or omission of the public entity in constructing or maintaining such facility. 24-10-103 C.R.S.

l. Under the Colorado Governmental Immunity Act (CGIA), a drainage and flood control facility is considered to be a "sanitation facility" and thus not protected by the defense that the facility caused damage solely because the design of the facility was inadequate. 24-10-106 (f) and 24-10-103 C.R.S. and Burnworth v. Adams County, 826 P.2d 368 (Colo. App. 1991).

m. Under the CGIA, a governmental entity will be liable for the negligent operation and maintenance of any drainage and flood control facility. 24-10-106 (f) and 24-10-103 C.R.S. and Burnworth v. Adams County, 826 P.2d 368 (Colo. App. 1991).

n. Under the CGIA, a governmental entity will not be liable for its failure to upgrade, modernize, modify, or improve the design or construction of a drainage or flood control facility. 24-10-103 (1) C.R.S.

o. In imposing conditions upon the granting of land-use approvals, no local government shall require an owner of private property to dedicate real property to the public or pay money to a public entity in an amount that is determined on an individual and discretionary basis, unless there is an essential nexus between the dedication or payment and a legitimate local government interest and the dedication or payment is roughly proportional both in nature and extent to the impact of the proposed use or development of such property. This law does not apply to any legislatively formulated assessment, fee, or charge that is imposed on a broad class of property owners by a local government. 29-20-203 C.R.S.

p. Public entities that own dams or reservoirs are not subject to strict liability for damages caused by water escaping from their dams or reservoirs. Further, those public entities have no duty to ensure that waters released from an upstream reservoir because of a dam failure would be contained by their facilities or would bypass those facilities without augmentation. Kane v. Town of Estes Park, 786 P.2d 412 (Colo. 1990).

q. A professional engineer is required not only to serve the interests of his or her employer/client but is also required, as his or her primary obligation, to protect the safety, health, property, and welfare of the public. Rule I 2. of The Colorado Rules of Professional Conduct of the State Board of Registration for Professional Engineers and Professional Land Surveyors.

r. Where a municipality imposes a special fee upon owners of property for purposes of providing a service and where the fee is reasonably designed to defray the cost of the service provided by the municipality, such a fee is a valid form of governmental charge within the legislative authority of the municipality. Bloom v. City of Fort Collins, 784 P.2d 304 (Colo. 1989).

## A104 LOCAL GOVERNMENTS DRAINAGE RESPONSIBILITIES AND POWERS

Local governments bear the greatest responsibility for stormwater management. They can best determine the community needs and approaches through local regulation. Local governments, however, are constrained by their resources and the powers permitted them.

Various legal methods for managing stormwaters are authorized by enabling legislation. Zoning ordinances and subdivision regulations are the most important methods available to local governments, followed by building regulations and building codes. Stormwater management may also be carried out by drainage districts, local governments having "home rule" powers, such as the Cities of Grand Junction, and Fruita, and by government agencies having authority to regulate floodplains.

The inherent police powers of a municipality enables it to enact ordinances that provide for the protection of public's health, safety, morals, or general welfare. These powers are exercised when specific ordinances are enacted to address drainage issues.

Statutory Powers also grants municipalities, counties, and state governments powers to provide directly or through special districts and authorities to construct, operate, and maintain a variety of public improvements including streets and sidewalks, water and sewage, storm drainage, and other facilities affecting stormwater quality and development in flood prone areas.

## A105 APPLICABLE DRAINAGE LAWS

### A105.1 Municipalities

- 31-23-301 C.R.S. / Authorizes municipalities to adopt regulations to promote public health, safety and general welfare of its citizenry.
- 31-15-701, 31-15-714 C.R.S. / Grants powers to municipalities to engage in public improvements.
- 31-15-711(1)(a) C.R.S / Allows municipalities to engage in activities to alter or change the natural channel of water courses.
- 31-25-501, 31-25-508 C.R.S. / Authorizes public improvements – Special Improvement Districts in Municipalities.
- 31-25-601, 31-25-604 C.R.S. / Allows municipalities to set up improvement districts with taxing powers for the purpose of constructing public improvements.
- 31-35-401, 31-35-417 C.R.S., 31-35-401(6) C.R.S. / allows municipalities to operate, maintain and finance facilities to include waters from storm, flood, or surface drainage.
- City of Fruita Land Use Code (Title 17) Section 17.45 / Floodways, Floodplains, Drainage and Erosion.

### A105.2 Counties

- 30-20-401, 30-20-402 C.R.S. / Authorizes County to construct water and sewerage facilities for County's own use or private and public users.
- 30-20-501, 30-20-504 C.R.S. / Authorizes creation of public improvement districts within any county as taxing units and for the purpose of implementing public improvements.
- 30-20-601, 30-20-603 C.R.S. / Authorizes a county by resolution to construct local improvements.
- 30-30-101, 30-28-105 C.R.S. / Authorizes the board of county commissioners of each county for flood control purposes only.
- 37-20-101, 37-33-109 C.R.S. / Authorizes owner of agricultural lands subject to drainage problems from the same general system to petition the board of county commissions to set up a drainage district.
- Mesa County Land Development Code:
  - § 7.7 Drainage
  - § 7.12 Irrigation Canals and Laterals

§ 7.13 Floodplain Regulations

### A105.3 State

- 24-65-101, 24-65-105 C.R.S. / Creates the Colorado land use commission with authority to assist counties and municipalities in developing guidelines for developing land uses and construction control within designated floodways.
- 29-1-204.2 C.R.S. / Allows for establishment of a drainage authority by any combination of municipalities, special districts, or other political subdivisions by entering into a contract with each other.
- 37-60-101, 37-60-106 C.R.S. / Creates Colorado Water Conservation Board for the purpose of water conservation and flood prevention. The board has the duty to "designate and approve storm or floodwater runoff channels or basins, and to make such designations available to legislative bodies of cities and incorporated town, ... and counties of the state" 30-60-123 C.R.S.
- 30-28-111 and 31-23-301 C.R.S. (24-65.1-403 C.R.S.) / Provides that no floodplains shall be designed by any local government until such description has been first approved by the Colorado Water Conservation Board.
- 29-1-201 C.R.S. / In 1974, Section 2 of Article VI of the State Constitution was amended to permit and encourage improvements to make the most efficient and effective use of their powers and responsibilities by cooperating and contracting with other governments.
- Water Quality Control Commission Regulations 93 (5CCR 1002—93, last date May 31. 2004) and 94 (SCR 1002-94, last date May 31, 2004).

### A105.4 Federal (NPDES)

On November 16, 1990, the EPA issued regulations on the control of stormwater from municipal and industrial stormwater discharges. The National Pollutant Discharge Elimination System (NPDES) includes stormwater management and discharge requirements and regulations, and is a part of the Federal Clean Water Act. The Stormwater Management regulation was developed to reduce the amount of pollutants entering streams, lakes and rivers as a result of runoff from residential, commercial and industrial areas. Regulations are found in 40 CFR 122.26, and are industry specific. Mesa County, City of Grand Junction and Grand Junction Drainage District have obtained permits to discharge stormwater under the Colorado Discharge Permit System (COR–090031). The County's CDPS Phase II Discharge Permit includes requirements to control erosion and sedimentation from construction activities.

## A106 SPECIAL MATTERS

### A106.1 Irrigation Ditches

In situations in which an irrigation ditch intersects a drainage basin, the irrigation ditch does not have to take underground waters diverted by a tile drain. However, the surface drainage must be accepted if the irrigation ditch is constructed in such a way that surface water would naturally flow into it. Clark v. Beauprez, 151 Colo. 119, 377 P.2d 105 (1962) (between private parties, the owner of an irrigation ditch can prevent an upstream landowner from diverting waters from their natural course into the irrigation ditch); City of Boulder v. Boulder and White Rock Ditch & Reservoir Company, 73 Colo. 426, 216 P. 553

(1923) (where an irrigation ditch was constructed in a natural drainageway into which surface water would naturally flow, the ditch owners could not complain merely on the ground that the city, in building storm sewers, collected the surface water and accelerated its flow and precipitated or discharged it at some particular point in the line of the ditch instead of spreading it out at different places of entrance).

In urbanizing areas, the conflict between the natural flow of surface water and irrigation ditches which bisect many drainage basins continues to be a difficult condition to resolve, taking into consideration the rights and liabilities of upstream property owners and irrigation ditch owners. Innumerable natural drainageways have been blocked by irrigation ditches, although they were constructed long before the basin became urbanized. This special area of urban drainage points to the need for good land use requirements, as well as identification of potential problem areas.

7-42-108 C.R.S. provides in part that:

> Every ditch corporation organized under the provisions of law shall be required to keep its ditch in good condition so that the water shall not be allowed to escape from the same to the injury of any mining claim, road, ditch, or other property.

This provision of Colorado law was recently interpreted in the case of Oliver v. Amity Mut. Irrigation Co., 994 P.2d 495 (Colo. App. 1999). In this case, the ditch company was being sued for damages to property resulting from a break in the bank of the ditch company's ditch. The court held that the statute imposed a duty of ordinary care, such as a person of average prudence and intelligence would use, under like circumstances to protect his or her own property. The court went on to state that, in order for the ditch company to fulfill its statutory duty, it had to prevent erosion of the ditch bank, keep the ditch free of sediment and debris, and control the amount of water flowing through its ditch, among other things, keeping the spillway at the intersection of its ditch and another free of obstructions. Finally, the court concluded that, although a ditch company is not liable for damages caused solely by an act of God, the company may not escape liability if its negligence contributed to or cooperated with an act of God to cause the damage.

In conclusion, those that own ditches owe a duty to those property owners through which their ditches pass to maintain their ditches, using ordinary care so as to prevent damage to the adjoining real property.

### A106.2 Ditch Owners Duty to Maintain, Use and Manage Their Ditches

In the Colorado Supreme Court case of Roaring Fork Club, L.P. v. St. Jude Company 36 P. 3d 1229 (Colo. 2001) the Court recognized that as early as Colorado's territorial legislatures, legislators recognized that our arid climate required the creation of a right to appropriate and convey water across the land of another so that lands not immediately proximate to water could be used and developed. Because of this importance of ditches, the holder of ditch easements has the right to inspect, operate, maintain, and repair the ditch. In addition, the owners of land upon which these ditches are located cannot damage the ditch or unreasonably inhibit the owner's ability to maintain the ditch. Thus, the owner of a ditch may go on the land of another for the purpose of cleaning out the waterway and making repairs. Additionally, the Colorado legislature has required ditch owners to undertake a host of duties in relation to ditch upkeep. The rights of ditch owners are so dominate that the Court held that the owner of property burdened by a ditch easement has not right to move or alter the easement without consent of the benefited owner unless he first obtains a declaration of a court that such alterations will cause no damage to the benefited owner.

### A106.3 Water Quality

Stormwater runoff is a major non-point source of water pollution. In urbanizing areas, where land-disturbing activities are numerous, stormwater washes soil and sediment into surface waters causing increased levels of turbidity and eutrophication, threatening fish and wildlife, and blocking drainage. In developed areas, runoff carries with it the pollutants from surfaces over which it runs, including, oil, litter, chemicals, nutrients and biological wastes, together with soils eroded from downstream channels of the flow.

It is reasoned that water quality control should be an integral part of any drainage or stormwater management program, since stormwater management techniques are often consistent with water quality objectives. However, this special area, as related to urban drainage, has not been researched adequately enough so as to provide the facts upon which a cost-effective approach could integrate water quality objectives with plans for surface drainage improvements. See City of Boulder v. Boulder and White Rock Ditch & Reservoir Company, 73 Colo. 426, 216 P. 553, 555 (1923).

### A106.4 Economic Loss Rule / Duty of Care

In the case of BRW, Inc. *et.al* v. Dufficy & Sons, Inc. 99 P. 3d 66 (Colo. 2004), the Colorado Supreme Court addressed the "economic loss rule" as it applies to contractual relationships. It holds that courts must focus on the contractual relationship between and among the parties when there is a claim of economic loss as a result of a construction contract. Thus it is not enough to simply allege negligence in a construction claims case. There must be a contractual relationship between the parties in order to sustain a claim based upon an economic loss. The Court reasoned that the "economic loss rule" applies in construction cases in order to permit the court to enforce expectancy interests of the parties so that they can reliably allocate risks and costs during their bargaining and to encourage the parties to build the cost considerations into the contract because they will not be able to recover economic damages in tort. Therefore, it should be assumed that in order to successfully pursue a claim in "economic loss" in a construction contract there must be a contractual relationship, either express or implied, between the parties and the provisions of that contract must address the obligation that is being alleged was breached.

### A106.5 Water Rights and Drainage Improvements

Although infrequently raised, the issue of the impact of drainage improvements on existing water rights in Colorado should be considered, evaluated and addressed as part of any drainage improvement planning. The Water Right Determination and Administration Act of 1969 provides remedies for water right owners who are impacted by the action of others.

In the case of The Board of County Commissioners of the County of Arapahoe *et.al.* v. Crystal Creek Homeowners' Association *et.al.* 14 P. 3d 325 (Colo. 2000) the Colorado Supreme Court affirmed its earlier holding in the case of Pueblo West Metro. Dist v. Southeastern Colo. Water Conservation Dist. 689 P. 2d 594 that the capture and storage of flood waters may be a "beneficial use" underlying an appropriation of water. Therefore, these cases confirm that the capture and storage of flood water is a permitted use under the statutory water rights scheme in Colorado thus establishing the need to obtain a recognized water right if a drainage or flood control facility will impact the availability of water and thus other water rights holders.

### A106.6 Governmental Entity Not Liable For Refusal to Issue Certificate of Occupancy of Residence Constructed in a Flood Plain

In the case of *Patzer v. City of Loveland* 2003 Colo. App. LEXIS 1506 (Colo. App. 2003), the City of Loveland was sued by a construction company who had received a building permit for a residence based upon its engineer's report. After the residence was completed, the City refused to issue a certificate of occupancy due to the fact that the City's engineering report, completed after the building permit was issued, showed that the residence encroached into the 100-year flood plain. Although the City eventually issued the certificate of occupancy, the Court held that the issuance of a building permit is an exercise of the City's police powers which include the regulation of flood control. Further, that a building permit contains no agreement, consideration, or promise that a certificate of occupancy would be issued. Therefore, the City could not be held liable for breach of contract. Finally, the Court went on to hold that the Governmental Immunity Act protected the City from a claim of negligent misrepresentation. Thus, the construction company received no relief from the Court.

### A106.7 Any Legislatively Formulated Assessment, Fee or Charge that is Imposed on a Broad Class of Property Owners by a Local Government is not Considered Taking of Property

In the case of *Marshall B. Krupp, et.al. v. The Breckenridge Sanitation District, et.al.* 19 P. 3d 687 (Colo. 2001), the Court was asked to address Colorado's regulatory takings statute and the statute's explicit declination to apply the Nolan/Dolan tests to "any legislatively formulated assessment, fee, or charge that is imposed on a broad class of property owners by a local government." The Krupp case arose when The Breckenridge Sanitation District legislatively assessed a fee on all building projects within the District. The Krupps challenged the assessment of the fee on their new residential townhouse project on the basis that it amounted to an unconstitutional taking of property. The Colorado Supreme Court held that a legislatively created, generally applicable service fee, is not subject to a takings analysis under Nollan/Dolan. Therefore, once a fee such as that in this case is assessed by way of a legislative act of the governmental entity it virtually cannot be challenged as being unconstitutional.

### A106.8 Governmental Immunity in Regard to Irrigation and Drainage Ditches Used as Part of Storm Water Drainage System

In the Colorado Supreme Court cases of *City of Colorado Springs v. Powell* 48 P. 3d 561 (Colo. 2002) and in the companion case of *City of Longmont v. Henry-Hobbs* 50 P. 3d 906 (Colo. 2002) the Court held that irrigation and drainage ditches used as part of a storm water drainage system are considered "sanitation facilities" under the Colorado Governmental Immunity Act. Since those ditches are covered under the CGIA, a governmental entity that uses those ditches for drainage or flood control will be held legally responsible, within the limits of the CGIA, for their negligent design or negligent maintenance. In a final holding of the Court, the Court clearly stated that it was not holding that all irrigation ditches are sanitation facilities.

In the 2003 session of the Colorado General Assembly, House Bill 03-1288 was passed and signed by the Governor. That Act specifically addressed the *City of Colorado Springs* and *City of Longmont* cases and noted that those cases may have significantly expanded the potential liability of governmental entities providing utility services to the public. The Act specifically redefined the word "maintenance" to mean "the act or omission of a public entity or public employee in keeping a facility in the same general state of repair or efficiency as initially constructed or in preserving a facility from decline or failure. 'Maintenance' does not include any duty to upgrade. Modernize, modify, or improve the design or construction of a facility." The purpose of this section of the Act was to clarify that governmental entities do not have an affirmative duty to improve the design or construction of a facility.

The Act went on to redefine what a "public sanitation facility" is and is not. In describing what a "public sanitation facility" is not, the Act reads as follows: "'Public sanitation facility' does not include: a public water facility; a natural watercourse even if dammed, channelized, or containing storm water runoff, discharge from a storm sewer, or discharge from a sewage treatment plant outfall; a drainage, borrow, or irrigation ditch even if the ditch contains stormwater runoff or discharge from storm sewers; a curb and gutter system; or other drainage, flood control, and stormwater facilities." Therefore, after this Act became effective on July 1, 2003, governmental entities were again protected from liability under the CGIA for negligent design and maintenance of a drainage facility which includes an irrigation ditch.

### A106.9 Concerning Interference with the Flow of Water in Ditch

37-89-101 C.R.S. was amended in the 2001 session of the Colorado General Assembly and provides that anyone who "interferes with the flow of water in any drainage ditch" shall be legally responsible for full restitution for the actual damages that were sustained as a result of that interference.

# APPENDIX B

## EXCERPTS FROM UDSCD VOLUME 3

# 6.0 EXTENDED DETENTION BASIN (EDB)—SEDIMENTATION FACILITY

## 6.1 Description

An extended detention basin (EDB) is a sedimentation basin designed to totally drain dry sometime after stormwater runoff ends. It is an adaptation of a detention basin used for flood control. The primary difference is in the outlet design. The EDB uses a much smaller outlet that extends the emptying time of the more frequently occurring runoff events to facilitate pollutant removal. The EDB's drain time for the brim-full water quality capture volume (i.e., time to fully evacuate the design capture volume) of 40 hours is recommended to remove a significant portion of fine particulate pollutants found in urban stormwater runoff. Soluble pollutant removal can be somewhat enhanced by providing a small wetland marsh or ponding area in the basin's bottom to promote biological uptake. The basins are considered to be "dry" because they are designed not to have a significant permanent pool of water remaining between storm runoff events. However, EDB may develop wetland vegetation and sometimes shallow pools in the bottom portions of the facilities.

## 6.2 General Application

An EDB can be used to enhance stormwater runoff quality and reduce peak stormwater runoff rates. If these basins are constructed early in the development cycle, they can also be used to trap sediment from construction activities within the tributary drainage area. The accumulated sediment, however, will need to be removed after upstream land disturbances cease and before the basin is placed into final long-term use. Also, an EDB can sometimes be retrofitted into existing flood control detention basins.

EDBs can be used to improve the quality of urban runoff coming from roads, parking lots, residential neighborhoods, commercial areas, and industrial sites and are generally used for regional or follow-up treatment. Refering to Figure ND-8 in the earlier chapter, EDBs are most applicable for catchments with a tributary impervious area of 10 acres of more. They can be used as an onsite BMP that works well with other BMPs, such as upstream onsite source controls and downstream infiltration/filtration basins or wetland channels. If desired, a flood routing detention volume can be provided above the water quality capture volume (*WQCV*) of the basin.

## 6.3 Advantages/Disadvantages

### 6.3.1 General

An EDB can be designed to provide other benefits such as recreation and open space opportunities in addition to reducing peak runoff rates and improving water quality. They are effective in removing particulate matter and the associate heavy metals and other pollutants. As with other BMPs, safety issues need to be addressed through proper design.

### 6.3.2 Physical Site Suitability

Normally, the land required for an EDB is approximately 0.5 to 2.0 percent of the total tributary development area. In high groundwater areas, consider the use of retention ponds (RP) instead in order to avoid many of the problems that can occur when the EDB's bottom is located below the seasonal high water table. Soil maps should be consulted, and soil borings may be needed to establish design geotechnical parameters.

### 6.3.3 Pollutant Removal

The pollutant removal range of an EDB was presented in Table SQ-6 in the STORMWATER QUALITY MANAGEMENT chapter of this volume. Removal of suspended solids and metals can be moderate to high, and removal of nutrients is low to moderate. The removal of nutrients can be improved when a small shallow pool or wetland is included as part of the basin's bottom or the basin is followed by BMPs more efficient at removing soluble pollutants, such as a filtration system, constructed wetlands or wetland channels.

The major factor controlling the degree of pollutant removal is the emptying time provided by the outlet. The rate and degree of removal will also depend on influent particle sizes. Metals, oil and grease, and some nutrients have a close affinity for suspended sediment and will be removed partially through sedimentation.

### 6.3.4 Aesthetics and Multiple Uses

Since an EDB is designed to drain very slowly, its bottom and lower portions will be inundated frequently for extended periods of time. Grasses in this frequently inundated zone will tend to be stressed, with only the species that can survive the specific environment at each site eventually prevailing. In addition, the bottom will be the depository of all the sediment that settles out in the basin. As a result, the bottom can be muddy and may have an undesirable appearance to some. To reduce this problem and to improve the basin's availability for other uses (such as open space habitat and passive recreation), it is suggested that the designer provide a lower-stage basin as suggested in the Two Stage Design procedure. As an alternative, a retention pond (RP) could be used, in which the settling occurs primarily within the permanent pool.

## 6.4 Design Considerations

Whenever desirable and feasible, incorporate the EDB within a larger flood control basin or as a part of a full-spectrum detention facility. Also, whenever possible try to accommodate within the basin other urban uses such as passive recreation and wildlife habitat. If multiple uses are being contemplated, consider using a multiple-stage detention basin to limit inundation of passive recreational areas to one or two occurrences a year. Generally, the area within the *WQCV* is not well suited for active recreation facilities such as ballparks, playing fields, and picnic areas. These are best located above the *WQCV* pool level.

Figure EDB-1 shows a representative layout of an EDB. Although flood control storage can be accomplished by providing a storage volume above the water quality storage, how best to accomplish this is not included in this discussion. Whether or not flood storage is provided, all embankments should be protected from catastrophic failure when runoff exceeds the design event. The State Engineer's regulatory requirements for larger dam embankments and storage volumes must be followed whenever regulatory height and/or volume thresholds are exceeded. Below those thresholds, the engineer should design the embankment-spillway-outlet system so that catastrophic failure will not occur.

Perforated outlet and trash rack configurations are illustrated in the TYPICAL STRUCTURAL BMP DETAINS AND SPECIFICATIONS chapter. Figure EDB-3 equates the *WQCV* that needs to be emptied over 40 hours, to the total required area of perforations per row for the standard configurations shown in that section. The chart is based on the rows being equally spaced vertically at 4-inch centers. This total area of perforations per row is then used to determine the number of uniformly sized holes per row (see detail in the TYPICAL STRUCTURAL BMP DETAINS AND SPECIFICATIONS chapter). One or more perforated columns on a perforated orifice plate integrated into the front of the outlet can be used; however, the fewer the number of columns, the better, maximizing the size of the orifice. Using least number of columns and the largest possible orifice, reduces clogging possibilities. Other types of outlets may also be used, provided they control the release of the *WQCV* in a manner consistent with the drain time requirements and are approved in advance by the District.

Although the soil types beneath the pond seldom prevent the use of this BMP, they should be considered during design. Any potential exfiltration capacity should be considered a short-term characteristic and ignored in the design of the *WQCV* because exfiltration will decrease over time as the soils clog with fine sediment and as the groundwater beneath the basin develops a mound that surfaces into the basin. At the same time, the findings by the International BMP Database team suggest that an EDP can reduce annual runoff volume by about 30% and this finding can help justify the annual pollutant load reductions when assessing the performance of this facility.

High groundwater should not preclude the use of an EDB. Groundwater, however, should to be considered during design and construction, and the outlet design must account for any upstream base flows that enter the basin or that may result from groundwater surfacing within the basin itself.

Stable, all weather access to critical elements of the pond, such as the inlet, outlet, spillway, and sediment collection areas must be provided for maintenance purposes.

### 6.5 Design Procedure and Criteria

The following steps outline the design procedure and criteria for an EDB.

1. Basin Storage Volume

Provide a storage volume equal to 120 percent of the *WQCV* based on a 40-hour drain time, above the lowest outlet (i.e., perforation) in the basin. The additional 20 percent of storage volume provides for sediment accumulation and the resultant loss in storage volume.

A. Determine the *WQCV* tributary catchment's percent imperviousness. Account for the effects of DCIA, if any, on Effective Imperviousness. Using runoff volume reduction practices in the tributary catchment and Figure ND-1, determine the reduction in impervious area to use with *WQCV* calculations.

B. Find the required storage volume (watershed inches of runoff):

Determine the Required *WQCV* (watershed inches of runoff) using Figure EDB-2, based on the EDB's 40-hour drain time.

Calculate the Design Volume in acre-feet as follows:

$$Design\ Volume = \left(\frac{WQCV}{12}\right) * Area * 1.2$$

In which:

| | | |
|---|---|---|
| *Area* | = | The watershed area tributary to the extended detention pond |
| *1.2 factor* | = | Multiplier of 1.2 to account for the additional 20% of required storage for sediment accumulation |

2. Outlet Works

The Outlet Works are to be designed to release the *WQCV* (i.e., not the "Design Volume") over a 40-hour period. Refer to the TYPICAL STRUCTURAL BMP DETAINS AND SPECIFICATIONS chapter for schematics pertaining to structure geometry; grates, trash racks, and screens; outlet type: orifice plate or perforated riser pipe; cutoff collar size and location; and all other necessary components.

For a perforated outlet, use Figure EDB-3 to calculate the required area per row based on *WQCV* and the depth of perforations at the outlet. See the TYPICAL STRUCTURAL BMP DETAINS AND SPECIFICATIONS chapter to determine the appropriate perforation geometry and number of rows. The lowest perforations should be set at the water surface elevation of the outlet micro-pool. The total outlet area is calculated by multiplying the area per row by the number of rows.

**Minimized the number of columns and maximize the perforation hole diameter when designing outlets to reduce chances of clogging by**

**accepting the orifice site that will empty the *WQCV* in 36 to 44 hours.**

3. Trash Rack

Provide a trash rack of sufficient size to prevent clogging of the primary water quality outlet. Size the rack so as not to interfere with the hydraulic capacity of the outlet. Using the total outlet area and the selected perforation diameter (or height), Figures 6, 6a or 7 in the TYPICAL STRUCTURAL BMP DETAINS AND SPECIFICATIONS chapter will help to determine the minimum open area required for the trash rack.

Use one-half of the perforated plate's total outlet area to calculate the trash rack's size. This accounts for the variable inundation of the outlet orifices. Figures 6 and 6a were developed as suggested standardized outlet designs for smaller sites.

4. Basin Shape

Shape the pond whenever possible with a gradual expansion from the inlet and a gradual contraction toward the outlet, thereby minimizing short circuiting. It is best to have a basin length to width ratio between 2:1 to 3:1. To achieve this, it may be necessary to modify the inlet and outlet points through the use of pipes, swales or channels to accomplish this.

Always maximize the distance between the inlet and the outlet.

5. Two-Stage Design

A two-stage design with a pool that fills often with frequently occurring runoff minimizes standing water and sediment deposition in the remainder of the basin. The two stages are as follows:

A. Top Stage: The top stage should be one or more feet deep with its bottom sloped at 1 to 2 percent toward the trickle flow channel.

B. Bottom Stage: The dry weather water surface of the active surcharge volume of the bottom stage should be 0.5 feet or more below the bottom of the top stage, but no less than 4-inches below the invert of the upstream trickle channel, and store no less than 0.5 percent of the *WQCV*.

   Provide a permanent micro-pool below the active storage volume of the lower stage in front of the outlet. The pool should be ½ the depth of the top stage depth described above, or 2.5 feet, whichever results in the larger depth.

6. Low-Flow Channel

Conveys low flows from the forebay to the bottom stage. To provide a maintainable trickle-flow channel, lining its bottom with concrete is recommended. Otherwise line its sides with buried Type VL soil riprap and bottom with concrete. Make it at least 4-inches deep if concrete lined sides and 8-inches if buried riprap sides are used. At a minimum provide capacity equal to twice the release capacity at the upstream forebay outlet.

7. Basin Side Slopes

Basin side slopes should be stable and gentle to facilitate maintenance and access. Side slopes should be no steeper than 4:1 and the use of flatter slopes is recommended; the flatter, the better and safer.

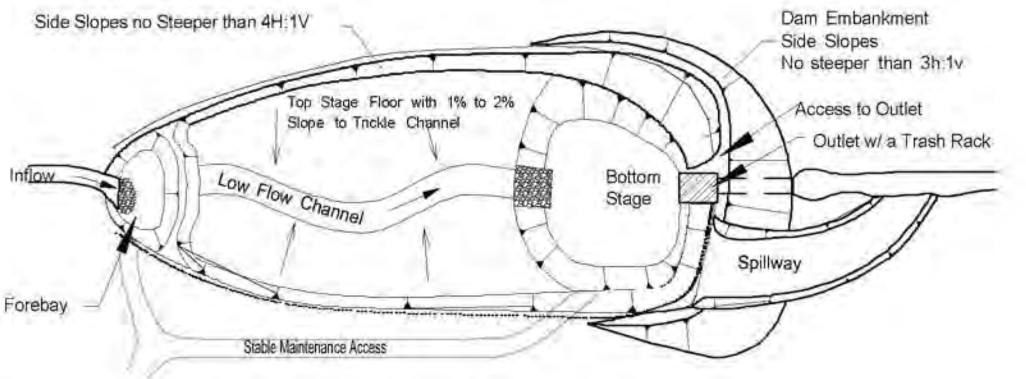
8. Dam Embankment

Design the embankment not to fail during a 100-year and larger storms. Embankment slopes should be no steeper than 3:1, preferably 4:1 or flatter, and planted with turf forming grasses. Poorly compacted native soils should be excavated and replaced. Embankment soils should be compacted to at least 95 percent of their maximum density according to

| | |
|---|---|
| | ASTM D 698-70 (Modified Proctor). Spillway structures and overflows should be designed in accordance with local drainage criteria and should consider UDFCD drop-structure design guidelines. |
| 9. Vegetation | Bottom vegetation provides erosion control and sediment entrapment. Pond bottom, berms, and side sloping areas may be planted with native grasses or with irrigated turf, depending on the local setting and needs. |
| 10. Access | All weather stable access to the bottom, forebay, and outlet works area shall be provided for maintenance vehicles. Grades should not exceed 10 percent, and a solid driving surface of gravel, rock, concrete, gravel-stabilized turf, or Type VL soil riprap should be provided. |
| 11. Inlet | Dissipate flow energy at pond's inflow point(s) to limit erosion and promote particle sedimentation. Inlets should be designed in accordance with UDFCD drop structure criteria, impact basin outlet details, or other types of energy dissipating structures. |
| 12. Forebay Design | Provides an opportunity for larger particles to settle out in the inlet in an area that has a solid surface bottom to facilitate mechanical sediment removal. A rock berm or concrete-wall should be constructed between the forebay and the main EDB. The forebay volume of the permanent pool should be about 3 to 5 percent of the design *WQCV*. A pipe throughout the berm to convey water the main body of the EDB should be offset from the inflow streamline to prevent short circuiting and should be sized to drain the forebay volume in 3 to 5 minutes, respectively. The floor of the forebay should be concrete or grouted boulder lined to define sediment removal limits. |
| 13. Flood Storage | Combining the water quality facility with a flood control facility is recommended. The 5-year, 10-year, 100-year, or other floods may be detained above the *WQCV*. See Section 1.5.5 of the BMP PLANNING FOR NEW DEVELOPMENT AND SIGNIFICANT REDEVELOPMENT chapter of this volume for further guidance. |
| 14. Multiple Uses | When desirable and feasible, incorporate the EDB within a larger flood control basin. Also, whenever possible, try to provide for other urban uses such as active or passive recreation, and wildlife habitat. If multiple uses are being contemplated, use the multiple-stage detention basin design approach to limit inundation of passive recreational areas to one or two occurrences a year. **The area within the *WQCV* is not suited for active recreation activities such as ballparks, playing fields, and picnic areas. These are best located above the *WQCV* level.** |

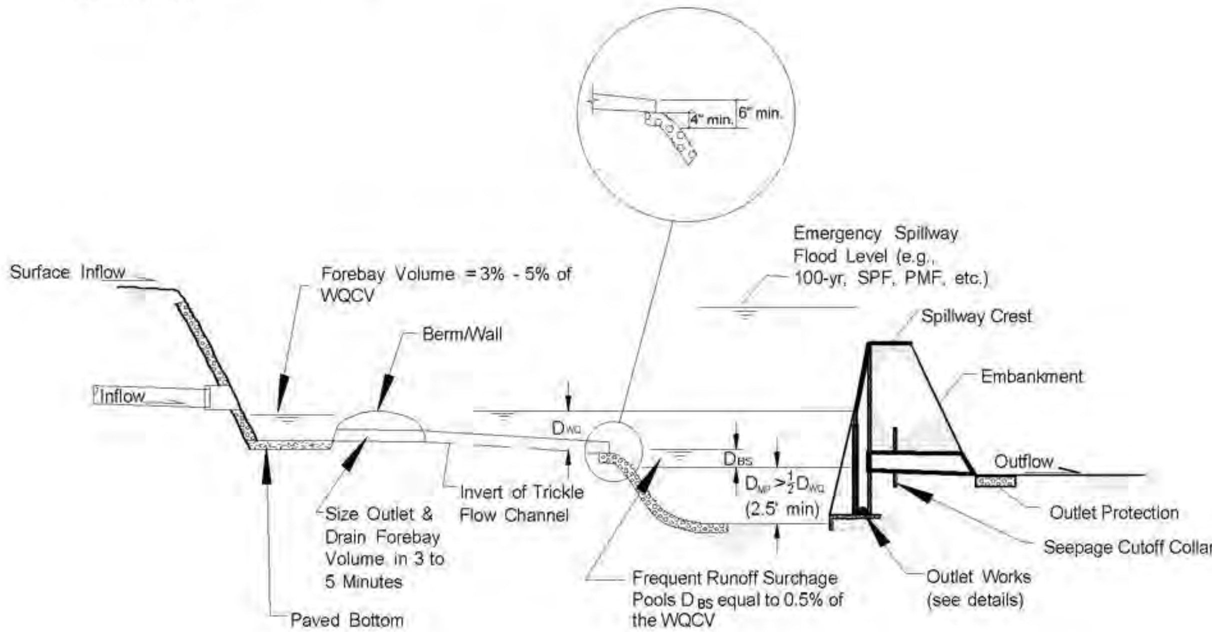
## 6.6 Design Example

Design forms that provide a means of documenting the design procedure are included in the DESIGN FORMS chapter. A completed form follows as a design example.




Note: Provide energy disipating inlet such as impact basin for pipes and GSB drop or baffle chute for channel/swales.

PLAN
Not to scale




PROFILE
Not to scale

**Figure EDB-1 –Extended Detention Basin (EDB) – Plan and Sections**

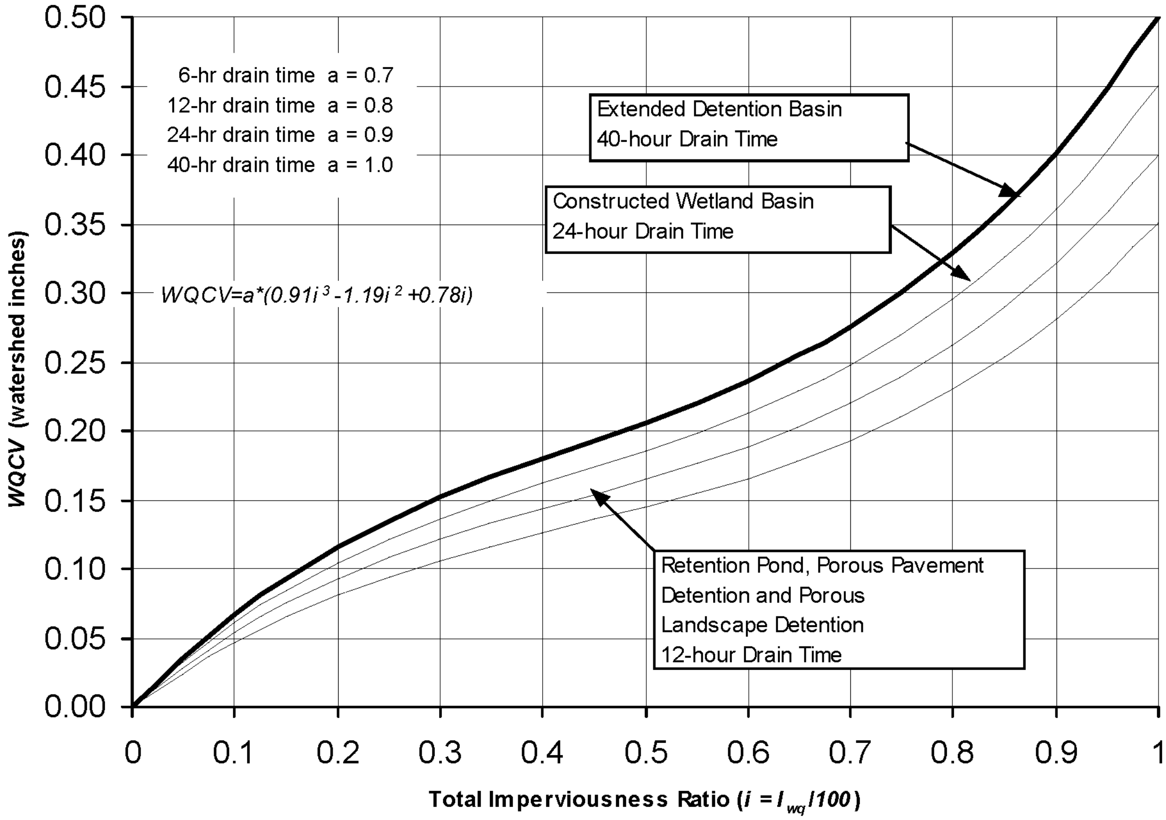



**Figure EDB-2—Water Quality Capture Volume (*WQCV*), 80th Percentile Runoff Event**

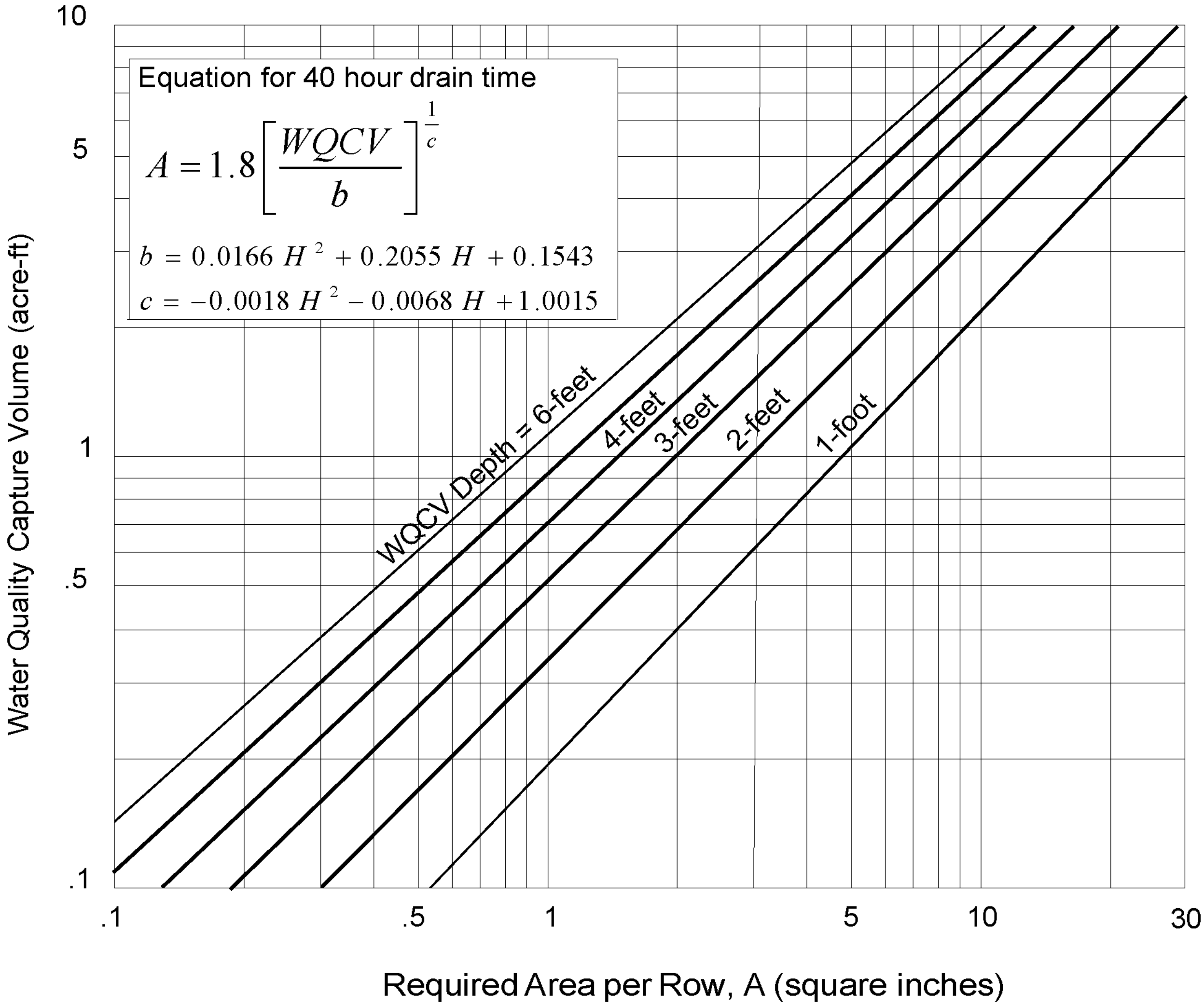



**Figure EDB-3—Water Quality Outlet Sizing: Extended Detention Basin (Dry) With 40-hour Drain Time for Capture Volume**

(Equation on this figure valid only for WQCV Depth H ≤ 6 feet)

## Design Procedure Form: Extended Detention Basin (EDB) - Sedimentation Facility

Sheet 1 of 3

| | |
|---|---|
| **Designer:** | **Joseph Stormwater** |
| **Company:** | **BLWH, Inc.** |
| **Date:** | **October 1, 2005** |
| **Project:** | **Sleepy Acres Residential and Commenrcial Getaway** |
| **Location:** | **40th Latitude, Colorado.** |

| | |
|---|---|
| 1. Basin Storage Volume | |
| A) Tributary Area's Imperviousness Ratio ($i = I_a / 100$) | $I_a$ = 60.00 % <br> i = 0.60 |
| B) Contributing Watershed Area (Area) | Area = 45.000 acres |
| C) Water Quality Capture Volume (WQCV) <br> ($WQCV = 1.0 * (0.91 * I^3 - 1.19 * I^2 + 0.78 * I)$) | WQCV = 0.24 watershed inches |
| D) Design Volume: Vol = (WQCV / 12) * Area * 1.2 | Vol = 1.0627 acre-feet |
| 2. Outlet Works | |
| A) Outlet Type (Check One) | X Orifice Plate <br> Perforated Riser Pipe <br> Other: |
| B) Depth at Outlet Above Lowest Perforation (H) | H = 2.50 feet |
| C) Recommended Maximum Outlet Area per Row, ($A_o$) | $A_o$ = 1.7 square inches |
| D) Perforation Dimensions: <br> i) Circular Perforation Diameter or <br> ii) Width of 2" High Rectangular Perforations | D = 1.500 inches <br> W = inches |
| E) Number of Columns (nc, See Table 6a-1 For Maximum) | nc = 1 number |
| F) Actual Design Outlet Area per Row ($A_o$) | $A_o$ = 1.8 square inches |
| G) Number of Rows (nr) | nr = 8 number |
| H) Total Outlet Area ($A_{ot}$) | $A_{ot}$ = 13.3 square inches |
| 3. Trash Rack | |
| A) Needed Open Area: $A_t = 0.5 * (\text{Figure 7 Value}) * A_{ot}$ | $A_t$ = 424 square inches |
| B) Type of Outlet Opening (Check One) | X ≤ 2" Diameter **Round** <br> 2" High **Rectangular** <br> Other: |
| C) For 2", or Smaller, **Round Opening** (Ref.: Figure 6a): | |
| i) Width of Trash Rack and Concrete Opening ($W_{conc}$) from Table 6a-1 | $W_{conc}$ = 15 inches |
| ii) Height of Trash Rack Screen ($H_{TR}$) | $H_{TR}$ = 60 inches |

## Design Procedure Form: Extended Detention Basin (EDB) - Sedimentation Facility

Sheet 2 of 3

| | |
|---|---|
| Designer: | Joseph Stormwater |
| Company: | BLWH, Inc. |
| Date: | October 1, 2005 |
| Project: | Sleepy Acres Residential and Commenrcial Getaway |
| Location: | 40th Latitude, Colorado. |

| | |
|---|---|
| iii) Type of Screen (Based on Depth H), Describe if "Other" | X S.S. #93 VEE Wire (US Filter)<br>Other: |
| iv) Screen Opening Slot Dimension, Describe if "Other" | X 0.139" (US Filter)<br>Other: |
| v) Spacing of Support Rod (O.C.)<br>Type and Size of Support Rod (Ref.: Table 6a-2) | 1.00 inches<br>**TE 0.074 in. x 0.50 in.** |
| vi) Type and Size of Holding Frame (Ref.: Table 6a-2) | **0.75 in. x 1.00 in. angle** |
| D) For 2" High **Rectangular Opening** (Refer to Figure 6b): | |
| i) Width of Rectangular Opening (W) | W = ____ inches |
| ii) Width of Perforated Plate Opening ($W_{conc}$ = W + 12") | $W_{conc}$ = ____ inches |
| iii) Width of Trashrack Opening ($W_{opening}$) from Table 6b-1 | $W_{opening}$ = ____ inches |
| iv) Height of Trash Rack Screen ($H_{TR}$) | $H_{TR}$ = ____ inches |
| v) Type of Screen (based on depth H) (Describe if "Other") | Klemp™ KPP Series Aluminum<br>Other: |
| vi) Cross-bar Spacing (Based on Table 6b-1, Klemp™ KPP Grating). Describe if "Other" | ____ inches<br>Other: |
| vii) Minimum Bearing Bar Size (Klemp™ Series, Table 6b-2)<br>(Based on depth of WQCV surcharge) | |
| 4. Detention Basin length to width ratio | 1.20 (L/W) |
| 5. Pre-sedimentation Forebay Basin - Enter design values | |
| A) Volume (3% to 5% of Design Volume from 1D)<br>(3% - 5% of Design Volume (0.0319 - 0.0531 acre-feet.) | 0.0400 acre-feet |
| B) Surface Area | 0.025 acres |
| C) Connector Pipe Diameter<br>(Size to drain this volume in 5-minutes under inlet control) | 4 inches |
| D) Paved/Hard Bottom and Sides | X yes/no |

**Design Procedure Form: Extended Detention Basin (EDB) - Sedimentation Facility**



| | |
|---|---|
| **Designer:** | **Joseph Stormwater** |
| **Company:** | **BLWH, Inc.** |
| **Date:** | **October 1, 2005** |
| **Project:** | **Sleepy Acres Residential and Commenrcial Getaway** |
| **Location:** | **40th Latitude, Colorado.** |

| | |
|---|---|
| 6. Two-Stage Design - See Figure EDB-1 | |
| A) Top Stage (Depth $D_{WQ}$ = 2' Minimum)<br>Top Stage Storage: no less than 95.74% of Design Volume (1.0174 acre-feet.) | $D_{WQ}$ = 2.00 feet<br>Storage= 1.1000 acre-feet |
| B) Bottom Stage Depth ($D_{BS}$ = 0.33' Minimum Below Trickle Channel Invert)<br>Bottom Stage Storage: no less than 0.5% of Design Volume (0.0053 acre-feet.)<br>Storage = A * Depth Above WS To Bottom Of Top Stage | $D_{BS}$ = 0.33 feet<br>Storage= 0.0055 acre-feet<br>Surf. Area= 0.017 acres |
| C) Micro Pool (Minimum Depth = the Larger of<br>0.50 * Top Stage Depth (1 ') or 2.5 ' | Depth= 2.60 feet |
| D) Total Volume: $Vol_{tot}$ = Storage from 5A + 6A + 6B<br>(Must be > Design Volume in 1D, or 1.0627 acre-feet.) | $Vol_{tot}$ = 1.1455 acre-feet |
| 7. Basin Side Slopes (Z, horizontal distance per unit vertical)<br>Minimum Z = 4, Flatter Preferred | Z = 5.00 (horizontal/vertical) |
| 8. Dam Embankment Side Slopes (Z, horizontal distance)<br>per unit vertical) Minimum Z = 3, Flatter Preferred | Z = 4.00 (horizontal/vertical) |
| 9. Vegetation (Check the method or describe "Other") | ____ Native Grass<br>__X__ Irrigated Turf Grass<br>____ Other: ________ |

Notes: ____________________

Extended Detention Basin (EDB)

## Design Procedure Form: Extended Detention Basin (EDB) - Sedimentation Facility

**Sheet 1 of 3**

**Designer:** ______________________
**Company:** ______________________
**Date:** ______________________
**Project:** ______________________
**Location:** ______________________

| | |
|---|---|
| 1. Basin Storage Volume | |
| A) Tributary Area's Imperviousness Ratio ($i = I_a / 100$ ) | $I_a$ = ______ % <br> $i$ = ______ |
| B) Contributing Watershed Area (Area) | Area = ______ acres |
| C) Water Quality Capture Volume (WQCV) <br> $(WQCV = 1.0 * (0.91 * i^3 - 1.19 * i^2 + 0.78 * i))$ | WQCV = ______ watershed inches |
| D) Design Volume: Vol = (WQCV / 12) * Area * 1.2 | Vol = ______ acre-feet |
| 2. Outlet Works | |
| A) Outlet Type (Check One) | ______ Orifice Plate <br> ______ Perforated Riser Pipe <br> ______ Other: ______ |
| B) Depth at Outlet Above Lowest Perforation (H) | H = ______ feet |
| C) Recommended Maximum Outlet Area per Row, ($A_o$) | $A_o$ = ______ square inches |
| D) Perforation Dimensions: <br> i) Circular Perforation Diameter or <br> ii) Width of 2" High Rectangular Perforations | D = ______ inches <br> W = ______ inches |
| E) Number of Columns (nc, See Table 6a-1 For Maximum) | nc = ______ number |
| F) Actual Design Outlet Area per Row ($A_o$) | $A_o$ = ______ square inches |
| G) Number of Rows (nr) | nr = ______ number |
| H) Total Outlet Area ($A_{ot}$) | $A_{ot}$ = ______ square inches |
| 3. Trash Rack | |
| A) Needed Open Area: $A_t = 0.5 * (\text{Figure 7 Value}) * A_{ot}$ | $A_t$ = ______ square inches |
| B) Type of Outlet Opening (Check One) | ______ ≤ 2" Diameter **Round** <br> ______ 2" High **Rectangular** <br> ______ Other: ______ |
| C) For 2", or Smaller, **Round Opening** (Ref.: Figure 6a): | |
| i) Width of Trash Rack and Concrete Opening ($W_{conc}$) from Table 6a-1 | $W_{conc}$ = ______ inches |
| ii) Height of Trash Rack Screen ($H_{TR}$) | $H_{TR}$ = ______ inches |

**Design Procedure Form: Extended Detention Basin (EDB) - Sedimentation Facility**

Sheet 2 of 3

**Designer:** ______
**Company:** ______
**Date:** ______
**Project:** ______
**Location:** ______

| | |
|---|---|
| iii) Type of Screen (Based on Depth H), Describe if "Other" | ______ S.S. #93 VEE Wire (US Filter)<br>______ Other: ______ |
| iv) Screen Opening Slot Dimension, Describe if "Other" | ______ 0.139" (US Filter)<br>______ Other: ______ |
| v) Spacing of Support Rod (O.C.)<br>Type and Size of Support Rod (Ref.: Table 6a-2) | ______ inches<br>______ |
| vi) Type and Size of Holding Frame (Ref.: Table 6a-2) | ______ |
| D) For 2" High **Rectangular Opening** (Refer to Figure 6b): | |
| i) Width of Rectangular Opening (W) | $W$ = ______ inches |
| ii) Width of Perforated Plate Opening ($W_{conc} = W + 12"$) | $W_{conc}$ = ______ inches |
| iii) Width of Trashrack Opening ($W_{opening}$) from Table 6b-1 | $W_{opening}$ = ______ inches |
| iv) Height of Trash Rack Screen ($H_{TR}$) | $H_{TR}$ = ______ inches |
| v) Type of Screen (based on depth H) (Describe if "Other") | ______ Klemp™ KPP Series Aluminum<br>______ Other: ______ |
| vi) Cross-bar Spacing (Based on Table 6b-1, Klemp™ KPP Grating). Describe if "Other" | ______ inches<br>______ Other: ______ |
| vii) Minimum Bearing Bar Size (Klemp™ Series, Table 6b-2)<br>(Based on depth of WQCV surcharge) | ______ |
| 4. Detention Basin length to width ratio | ______ (L/W) |
| 5. Pre-sedimentation Forebay Basin - Enter design values | |
| A) Volume (3% to 5% of Design Volume from 1D) | ______ acre-feet |
| B) Surface Area | ______ acres |
| C) Connector Pipe Diameter<br>(Size to drain this volume in 5-minutes under inlet control) | ______ inches |
| D) Paved/Hard Bottom and Sides | ______ yes/no |

**Design Procedure Form: Extended Detention Basin (EDB) - Sedimentation Facility**

**Sheet 3 of 3**

**Designer:** ______________________
**Company:** ______________________
**Date:** ______________________
**Project:** ______________________
**Location:** ______________________

| | |
|---|---|
| 6. Two-Stage Design - See Figure EDB-1 | |
| A) Top Stage (Depth $D_{WQ}$ = 2' Minimum)<br>Top Stage Storage (no less than 94% of Design Volume) | $D_{WQ}$ = ______ feet<br>Storage= ______ acre-feet |
| B) Bottom Stage Depth ($D_{BS}$ = 0.33' Minimum Below Trickle Channel Invert)<br>Bottom Stage Storage (no less than 0.5% of Design Volume from 1D)<br>Storage = A * Depth Above WS To Bottom Of Top Stage | $D_{BS}$ = ______ feet<br>Storage= ______ acre-feet<br>Surf. Area= ______ acres |
| C) Micro Pool (Minimum Depth = the Larger of<br>0.50 * Top Stage Depth or 2.5 Feet) | Depth= ______ feet |
| D) Total Volume: $Vol_{tot}$ = Storage from 5A + 6A + 6B<br>(Must be greater than the Design Volume in 1D.) | $Vol_{tot}$ = ______ acre-feet |
| 7. Basin Side Slopes (Z, horizontal distance per unit vertical)<br>Minimum Z = 4, Flatter Preferred | Z = ______ (horizontal/vertical) |
| 8. Dam Embankment Side Slopes (Z, horizontal distance)<br>per unit vertical) Minimum Z = 3, Flatter Preferred | Z = ______ (horizontal/vertical) |
| 9. Vegetation (Check the method or describe "Other") | ______ Native Grass<br>______ Irrigated Turf Grass<br>______ Other: ______ |

Notes: ______________________

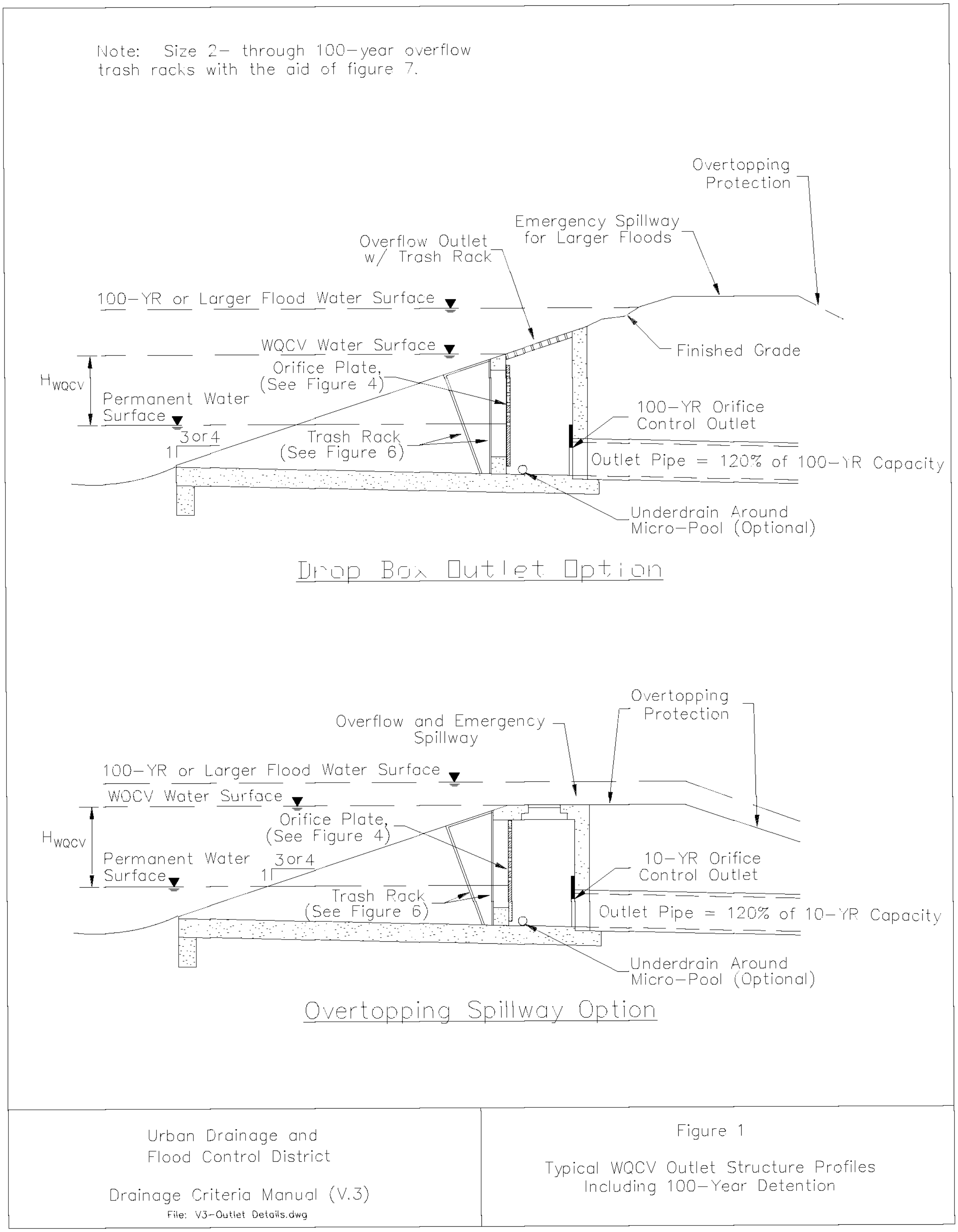
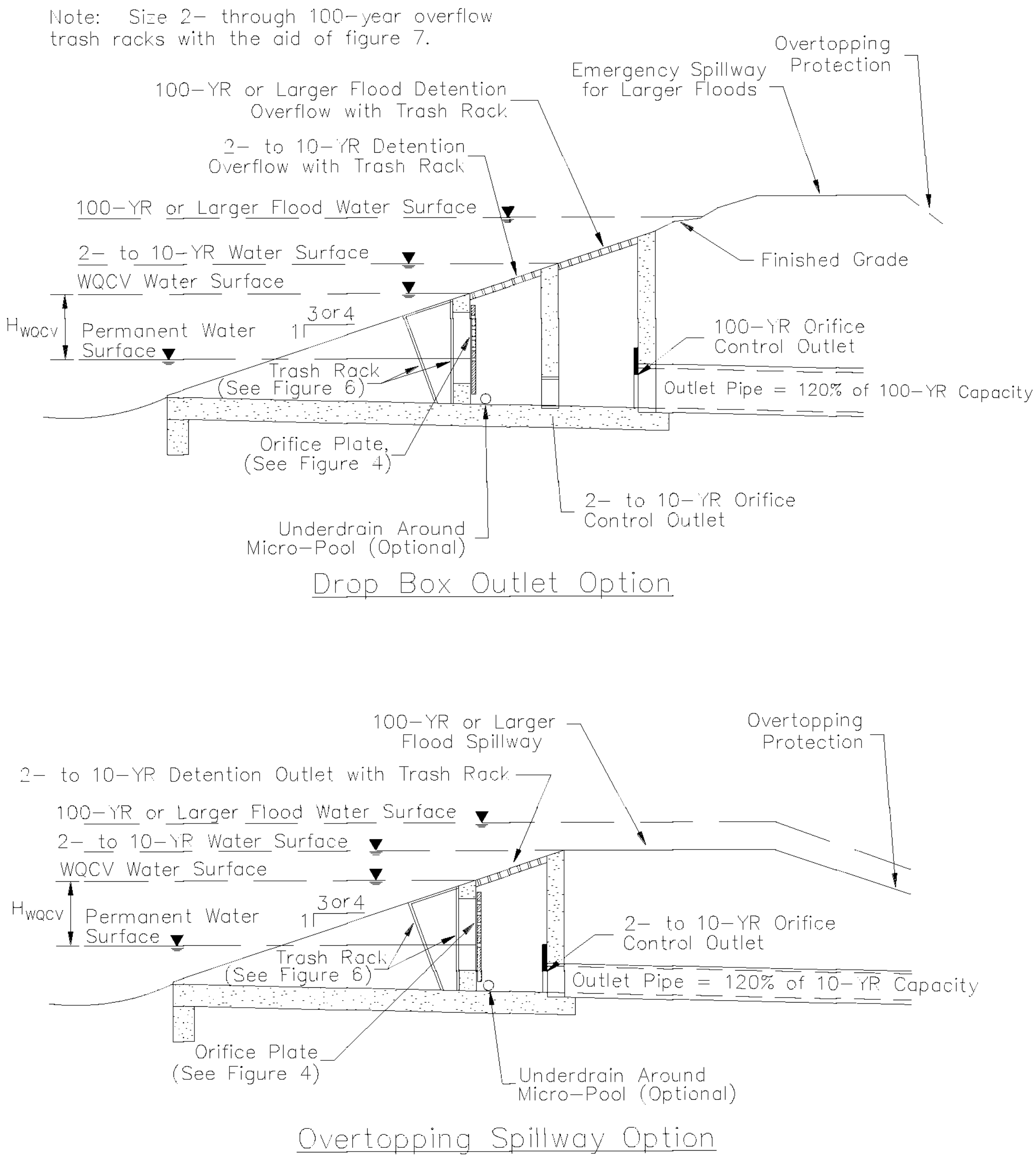



**Figure 1—Typical WQCV Outlet Structure Profile, Including 100-year Detention.**




| | |
|---|---|
| Urban Drainage and Flood Control District<br><br>Drainage Criteria Manual (V.3)<br>File: V3-Outlet Details.dwg | Figure 2<br><br>Typical WQCV Outlet Structure Profiles Including 2- to 10-Year and 100-Year Detention |

**Figure 2—Typical WQCV Outlet Structure Profile, Including 2- to 10-year and 100-year Detention.**

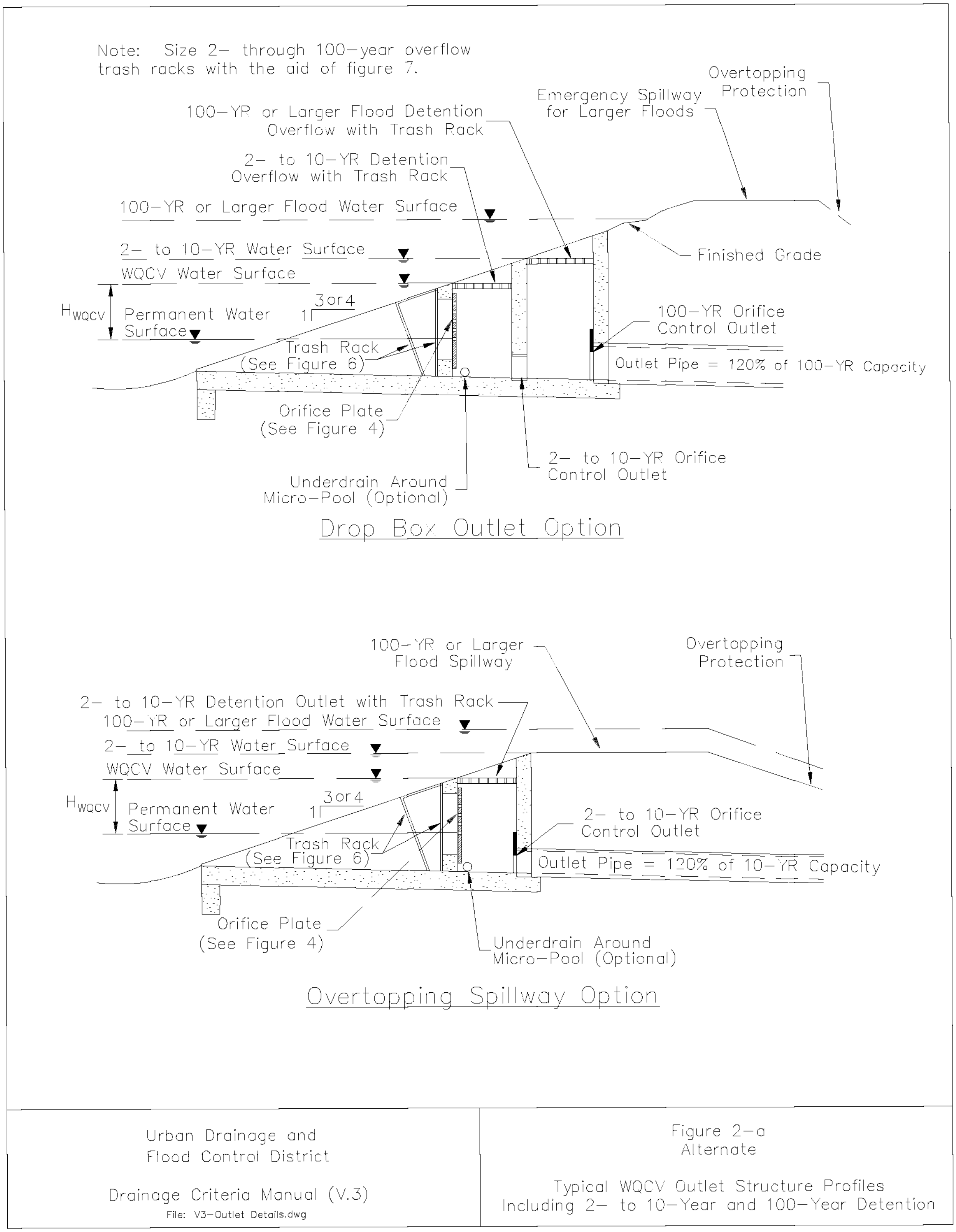



**Figure 2-a—Typical WQCV Outlet Structure Profile, Including 2- to 10-year and 100-year Detention.**

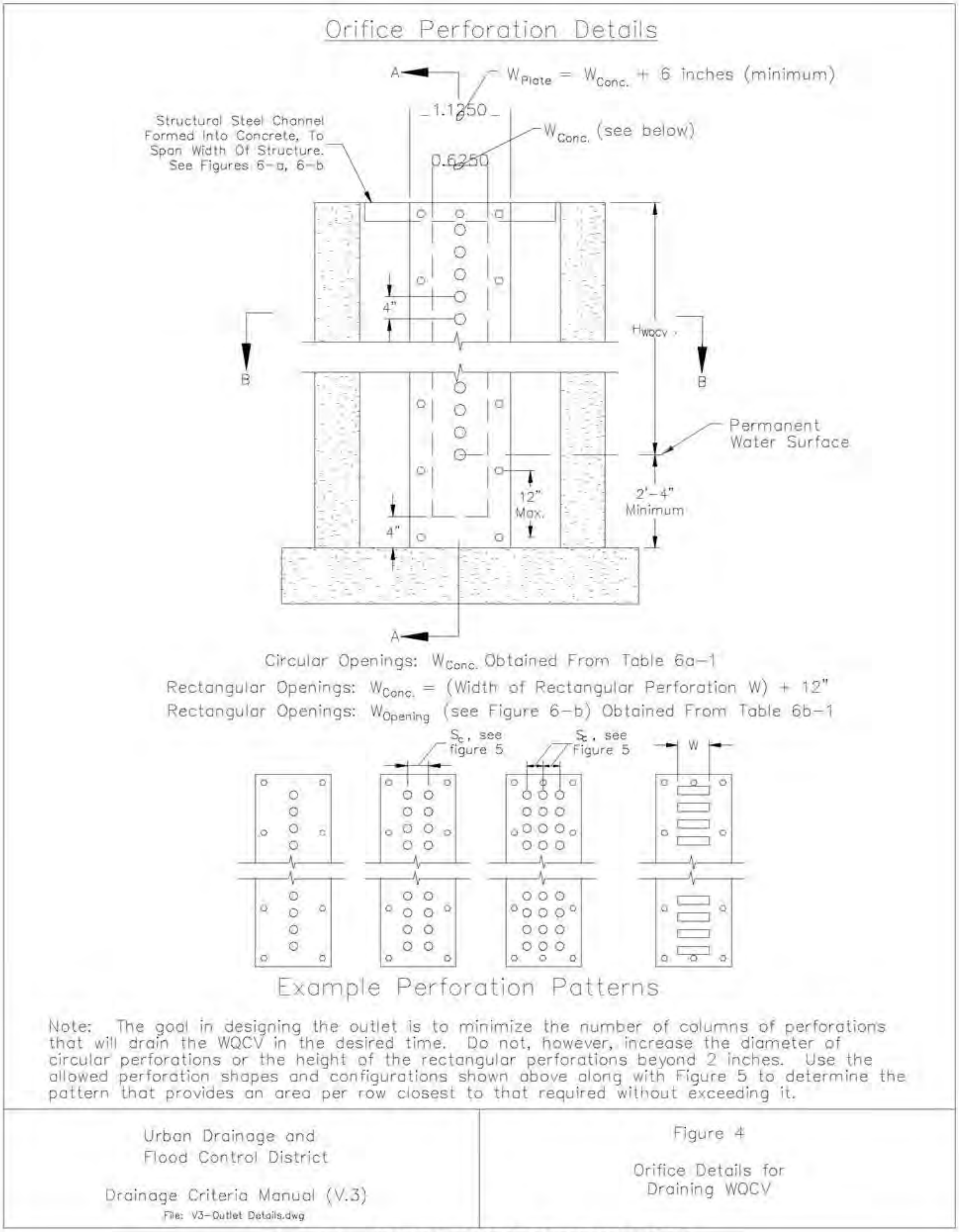



**Figure 4—Orifice Details for Draining WQCV.**

# Orifice Plate Perforation Sizing

## Circular Perforation Sizing

Chart may be applied to orifice plate or vertical pipe outlet.

| Hole Dia (in) * | Hole Dia (in) | Min. $S_c$ (in) | Area per Row (sq in) | | |
|---|---|---|---|---|---|
| | | | n=1 | n=2 | n=3 |
| 1/4 | 0.250 | 1 | 0.05 | 0.10 | 0.15 |
| 5/16 | 0.313 | 2 | 0.08 | 0.15 | 0.23 |
| 3/8 | 0.375 | 2 | 0.11 | 0.22 | 0.33 |
| 7/16 | 0.438 | 2 | 0.15 | 0.30 | 0.45 |
| 1/2 | 0.500 | 2 | 0.20 | 0.39 | 0.59 |
| 9/16 | 0.563 | 3 | 0.25 | 0.50 | 0.75 |
| 5/8 | 0.625 | 3 | 0.31 | 0.61 | 0.92 |
| 11/16 | 0.688 | 3 | 0.37 | 0.74 | 1.11 |
| 3/4 | 0.750 | 3 | 0.44 | 0.88 | 1.33 |
| 13/16 | 0.813 | 3 | 0.52 | 1.04 | 1.56 |
| 7/8 | 0.875 | 3 | 0.60 | 1.20 | 1.80 |
| 15/16 | 0.938 | 3 | 0.69 | 1.38 | 2.07 |
| 1 | 1.000 | 4 | 0.79 | 1.57 | 2.36 |
| 1 1/16 | 1.063 | 4 | 0.89 | 1.77 | 2.66 |
| 1 1/8 | 1.125 | 4 | 0.99 | 1.99 | 2.98 |
| 1 3/16 | 1.188 | 4 | 1.11 | 2.22 | 3.32 |
| 1 1/4 | 1.250 | 4 | 1.23 | 2.45 | 3.68 |
| 1 5/16 | 1.313 | 4 | 1.35 | 2.71 | 4.06 |
| 1 3/8 | 1.375 | 4 | 1.48 | 2.97 | 4.45 |
| 1 7/16 | 1.438 | 4 | 1.62 | 3.25 | 4.87 |
| 1 1/2 | 1.500 | 4 | 1.77 | 3.53 | 5.30 |
| 1 9/16 | 1.563 | 4 | 1.92 | 3.83 | 5.75 |
| 1 5/8 | 1.625 | 4 | 2.07 | 4.15 | 6.22 |
| 1 11/16 | 1.688 | 4 | 2.24 | 4.47 | 6.71 |
| 1 3/4 | 1.750 | 4 | 2.41 | 4.81 | 7.22 |
| 1 13/16 | 1.813 | 4 | 2.58 | 5.16 | 7.74 |
| 1 7/8 | 1.875 | 4 | 2.76 | 5.52 | 8.28 |
| 1 15/16 | 1.938 | 4 | 2.95 | 5.90 | 8.84 |
| 2 | 2.000 | 4 | 3.14 | 6.28 | 9.42 |
| n = Number of columns of perforations | | | | | |
| Minimum steel plate thickness | | | 1/4 " | 5/16 " | 3/8 " |

* Designer may interpolate to the nearest 32nd inch to better match the required area, if desired.

## Rectangular Perforation Sizing

Only one column of rectangular perforations allowed.

Rectangular Height = 2 inches

$$\text{Rectangular Width (inches)} = \frac{\text{Required Area per Row (sq in)}}{2''}$$

| Rectangular Hole Width | Min. Steel Thickness |
|---|---|
| 5" | 1/4 " |
| 6" | 1/4 " |
| 7" | 5/32 " |
| 8" | 5/16 " |
| 9" | 11/32 " |
| 10" | 3/8 " |
| >10" | 1/2 " |

Urban Drainage and Flood Control District

Drainage Criteria Manual (V.3)

File: V3-Outlet Details.dwg

Figure 5

WQCV Outlet Orifice Perforation Sizing

**Figure 5—WQCV Outlets Orifice Perforation Sizing.**

Orifice Plate Perforation Sizing

Circular Perforation Sizing

This table may be used to size perforation in a vertical plate of riser pipe.

| Hole Dia. (in.) * | Hole Dia. (in.) | Min. $S_c$ (in.) | Area per Row (sq. in.) | | |
|---|---|---|---|---|---|
| | | | n = 1 | n = 2 | n = 3 |
| 1/4 | 0.250 | 1 | 0.05 | 0.10 | 0.15 |
| 5/16 | 0.313 | 2 | 0.08 | 0.16 | 0.24 |
| 3/8 | 0.375 | 2 | 0.11 | 0.22 | 0.33 |
| 7/16 | 0.438 | 2 | 0.15 | 0.30 | 0.45 |
| 1/2 | 0.500 | 2 | 0.20 | 0.40 | 0.60 |
| 9/16 | 0.563 | 3 | 0.25 | 0.50 | 0.75 |
| 5/8 | 0.625 | 3 | 0.31 | 0.62 | 0.93 |
| 11/16 | 0.688 | 3 | 0.37 | 0.74 | 1.11 |
| 3/4 | 0.750 | 3 | 0.44 | 0.88 | 1.32 |
| 13/16 | 0.813 | 3 | 0.52 | 1.04 | 1.56 |
| 7/8 | 0.875 | 3 | 0.60 | 1.20 | 1.80 |
| 15/16 | 0.938 | 3 | 0.69 | 1.38 | 2.07 |
| 1 | 1.000 | 4 | 0.79 | 1.58 | 2.37 |
| 1 1/16 | 1.063 | 4 | 0.89 | 1.78 | 2.67 |
| 1 1/8 | 1.125 | 4 | 0.99 | 1.98 | 2.97 |
| 1 3/16 | 1.188 | 4 | 1.11 | 2.22 | 3.33 |
| 1 1/4 | 1.250 | 4 | 1.23 | 2.46 | 3.69 |
| 1 5/16 | 1.313 | 4 | 1.35 | 2.70 | 4.05 |
| 1 3/8 | 1.375 | 4 | 1.48 | 2.96 | 4.44 |
| 1 7/16 | 1.438 | 4 | 1.62 | 3.24 | 4.86 |
| 1 1/2 | 1.500 | 4 | 1.77 | 3.54 | 5.31 |
| 1 9/16 | 1.563 | 4 | 1.92 | 3.84 | 5.76 |
| 1 5/8 | 1.625 | 4 | 2.07 | 4.14 | 6.21 |
| 1 11/16 | 1.688 | 4 | 2.24 | 4.48 | 6.72 |
| 1 3/4 | 1.750 | 4 | 2.41 | 4.82 | 7.23 |
| 1 13/16 | 1.813 | 4 | 2.58 | 5.16 | 7.74 |
| 1 7/8 | 1.875 | 4 | 2.76 | 5.52 | 8.28 |
| 1 15/16 | 1.938 | 4 | 2.95 | 5.90 | 8.85 |
| 2 | 2.000 | 4 | 3.14 | 6.28 | 9.42 |
| n = Number of columns of perforations | | | | | |
| Minimum steel plate thickness | | | 1/4” | 5/16” | 3/8” |
| * Designer may interfere to the nearest 32$^{nd}$ inch to better match the needed area if desired. | | | | | |

Rectangular Perforation sizing

Use only one rectangular column whenever two 2;inch diameter circular perforations cannot provide needed outlet area.

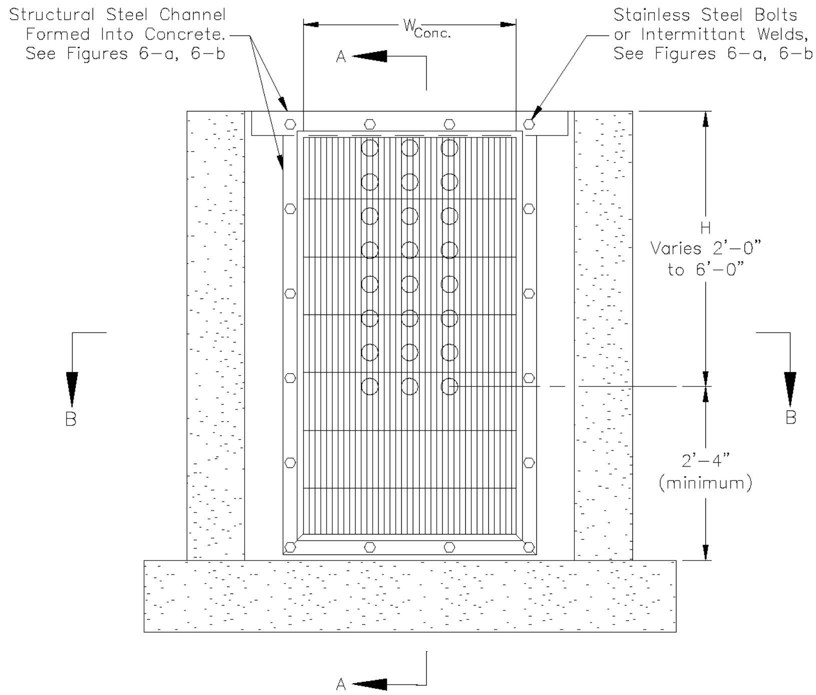
Rectangular Height = 2-inches

Rectangular Width = Required Area per Row / 2”

| Rectangular hole Width | Min. Steel Thickness |
|---|---|
| 5” | 1/4 " |
| 6” | 1/4 “ |
| 7” | 5/32 “ |
| 8” | 5/16 “ |
| 9” | 11/32 “ |
| 10” | 3/8 “ |
| > 10” | 1/2 “ |

**Figure 5—WQCV Outlets Orifice Perforation Sizing.**

Note: Vertical WQCV Trash Racks are shown in Figures 6, 6–a, and 6–b for suggested standardized outlet design. Adverse–Slope Trash Rack design may be used for non–standardized designs, but must meet minimum design criteria.




Elevation

WQCV Trash Racks:

1. Well–screen trash racks shall be stainless steel and shall be attached by intermittant welds along the edge of the mounting frame.
2. Bar grate trash racks shall be aluminum and shall be bolted using stainless steel hardware.
3. Trash Rack widths are for specified trash rack material. Finer well–screen or mesh size than specified is acceptable, however, trash rack dimensions need to be adjusted for materials having a different open area/gross area ratio (R value)
4. Structural design of trash rack shall be based on full hydrostatic head with zero head downstream of the rack.

Overflow Trash Racks:

1. All trash racks shall be mounted using stainless steel hardware and provided with hinged and lockable or boltable access panels.
2. Trash racks shall be stainless steel, aluminum, or steel. Steel trash racks shall be hot dip galvanized and may be hot powder painted after galvanizing.
3. Trash Racks shall be designed such that the diagonal dimension of each opening is smaller than the diameter of the outlet pipe.
4. Structural design of trash rack shall be based on full hydrostatic head with zero head downstream of the rack.

| | |
|---|---|
| Urban Drainage and Flood Control District<br><br>Drainage Criteria Manual (V.3)<br>File: V3–Outlet Details.dwg | Figure 6<br><br>Suggested WQCV Outlet Standardized Trash Rack Design |

**Figure 6—Suggested WQCV Outlets Standardized Trash Rack Design.**

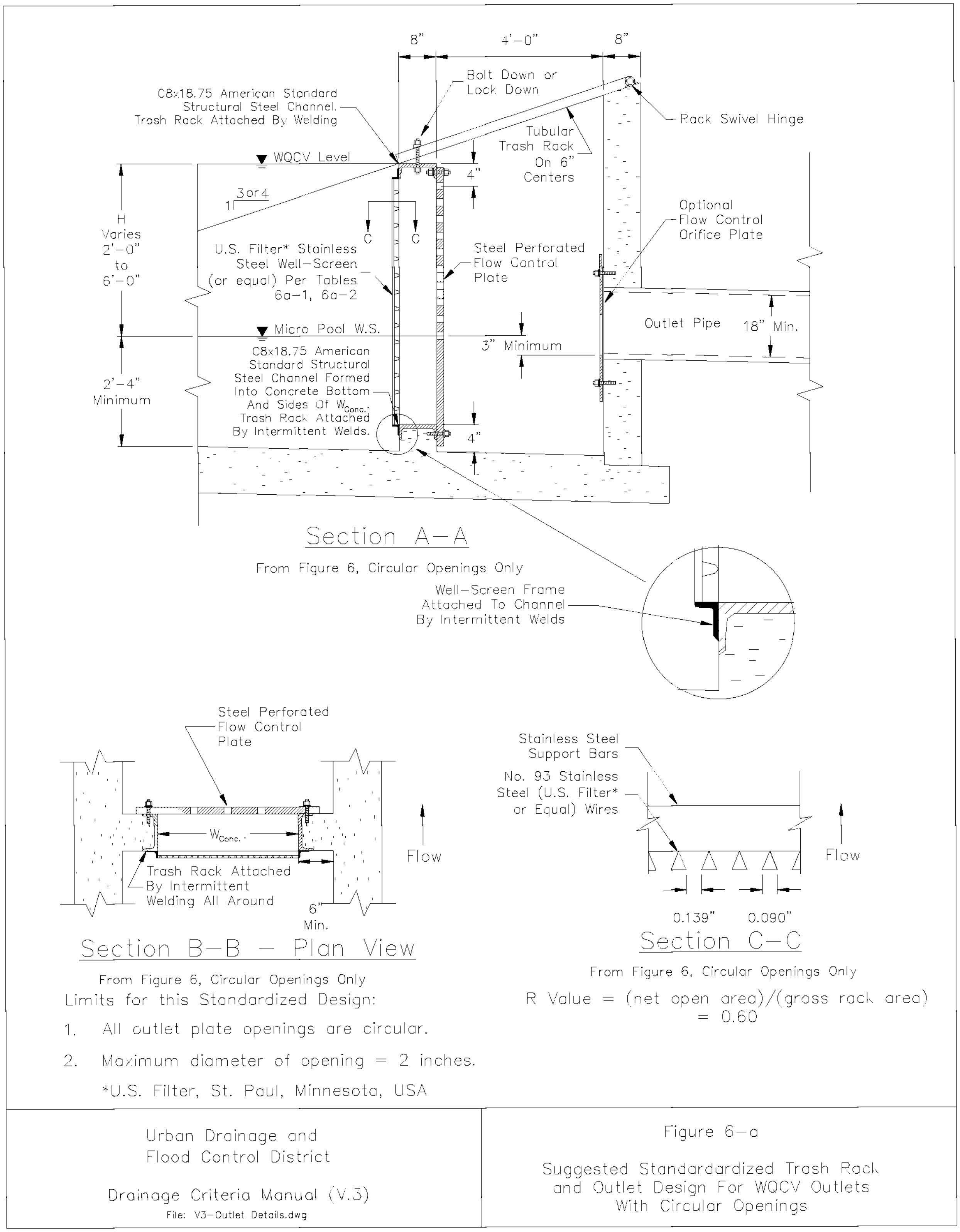



**Figure 6-a—Suggested Standardized Trash Rack and Outlet Design for WQCV Outlets With Circular Openings.**

**Table 6a-1—Trash Rack Mounting Opening Widths per Column of Circular Perforations**

(2” diameter maximum)

Minimum Width ($W_{opening}$) of Opening for a VEE Wire™ Screen Trash Rack.

Requires a minimum water depth below the lowest perforation of 2’-6”.

See Figure 6-a for Explanation of Terms.

| Maximum Dia. of Circular Opening (inches) | Width of Trash Rack Opening ($W_{conc.}$) Per Column of Holes as a Function of Water Depth H Above Lowest Perforation | | | | | |
|---|---|---|---|---|---|---|
| | H=2.0’ | H=3.0’ | H=4.0’ | H=5.0’ | H=6.0’ | Maximum Number of Columns |
| $\leq$ 0.25 | 3 in. | 3 in. | 3 in. | 3 in. | 3 in. | 14 |
| $\leq$ 0.50 | 3 in. | 3 in. | 3 in. | 3 in. | 3 in. | 14 |
| $\leq$ 0.75 | 3 in. | 6 in. | 6 in. | 6 in. | 6 in. | 7 |
| $\leq$ 1.00 | 6 in. | 9 in. | 9 in. | 9 in. | 9 in. | 4 |
| $\leq$ 1.25 | 9 in. | 12 in. | 12 in. | 12 in. | 15 in. | 2 |
| $\leq$ 1.50 | 12 in. | 15 in. | 18 in. | 18 in. | 18 in. | 2 |
| $\leq$ 1.75 | 18 in. | 21 in. | 21 in. | 24 in. | 24 in. | 1 |
| $\leq$ 2.00 | 21 in. | 24 in. | 27 in. | 30 in. | 30 in. | 1 |
| NOTE: Minimum width of opening in the concrete shall not be less than 9 inches whenever above table indicates otherwise. Always minimize number of columns of perforations used. | | | | | | |

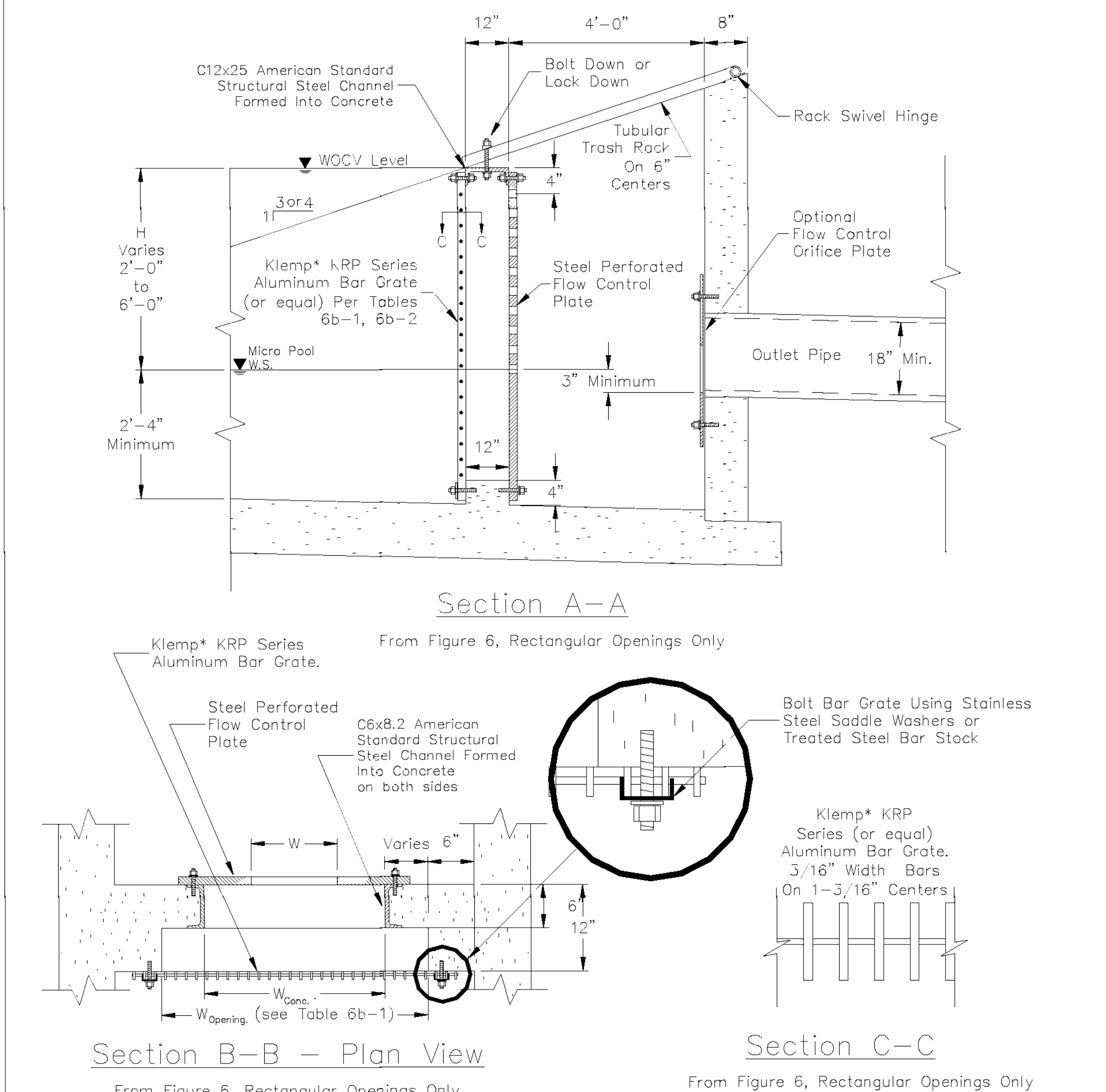
**Table 6a-2—Standardized WQCV Outlet Design Using Circular Openings**

(2” diameter maximum).

Johnson VEE Wire™ Stainless Steel Screen[1] (or equal) Trash Rack Design Specifications.

| Max. Width of Opening | Screen #93 VEE Wire Slot Opening | Support Rod Type | Support Rod, On-Center, Spacing | Total Screen Thickness | Carbon Steel Frame Type |
|---|---|---|---|---|---|
| 9” | 0.139” | #156 VEE | ¾” | 0.31” | 3/8”x1.0” flat bar |
| 18” | 0.139” | TE .074”x.50” | 1” | 0.655” | ¾” x 1.0 angle |
| 24” | 0.139” | TE .074”x.75” | 1” | 1.03” | 1.0” x 1½” angle |
| 27” | 0.139” | TE .074”x.75” | 1” | 1.03” | 1.0” x 1½” angle |
| 30” | 0.139” | TE .074”x1.0” | 1” | 1.155” | 1 1/4“x 1½” angle |
| 36” | 0.139” | TE .074”x1.0” | 1” | 1.155” | 1 1/4“x 1½” angle |
| 42” | 0.139” | TE .105”x1.0” | 1” | 1.155” | 1 1/4“x 1½” angle |

[1] Johnson Screens, St. Paul, Minnesota, USA (1-800-833-9473)




Limits for this Standardized Design:

1. All outlet plate openings are rectangular.
2. Height of all rectangular openings = 2 inches.
3. For trash rack opening width ($W_{Opening}$), see Table 6b-1
4. Concrete opening for outlet plate ($W_{Conc.}$) = W + 12 inches

R Value = (net open area)/(gross rack area)
= 0.71 for cross rods on 2" centers
= 0.77 for cross rods on 4" centers

*Klemp Corporation, Orem, Utah, USA

| | |
|---|---|
| Urban Drainage and Flood Control District<br><br>Drainage Criteria Manual (V.3)<br>File: V3-Outlet Details.dwg | Figure 6-b<br><br>Suggested Standardized Trash Rack and Outlet Design For WQCV Outlets With Rectangular Openings |

**Figure 6-b—Suggested Standardized Trash Rack and Outlet Design for WQCV Outlets With Rectangular Openings.**

**Table 6b-1—Standardized WQCV Outlet Design Using 2" High Rectangular Openings.**

Minimum Width ($W_{opening}$) of Opening for an Aluminum Bar Grate Trash Rack.

Requires a minimum water depth below the lowest perforation of 2'-4".

See Figure 6-b for Explanation of Terms.

| Maximum Width W of 2" Height Rectangular Opening (inches) | Minimum Width of Trash Rack Opening ($W_{opening}$) as a Function of Water Depth H Above Lowest Perforation | | | | | |
|---|---|---|---|---|---|---|
| | **H=2.0 ft.** | **H=3.0 ft.** | **H=4.0 ft.** | **H=5.0 ft.** | **H=6.0 ft.** | **Spacing of Bearing Bars, Cross Rods** |
| 2.0 | 2.0 ft. | 2.5 ft. | 2.5 ft. | 2.5 ft. | 3.0 ft. | 1-3/16", 2" |
| ≤ 2.5 | 2.5 ft. | 3.0 ft. | 3.0 ft. | 3.5 ft. | 3.5 ft. | 1-3/16", 2" |
| ≤ 3.0 | 3.0 ft. | 3.5 ft. | 3.5 ft. | 4.0 ft. | 4.0 ft. | 1-3/16", 2" |
| ≤ 3.5 | 3.5 ft. | 4.0 ft. | 4.5 ft. | 4.5 ft. | 5.0 ft. | 1-3/16", 2" |
| ≤ 4.0 | 3.5 ft. | 4.5 ft. | 5.0 ft. | 5.0 ft. | 5.5 ft. | 1-3/16", 2" |
| ≤ 4.5 | 4.0 ft. | 4.5 ft. | 5.0 ft. | 5.5 ft. | 5.5 ft. | 1-3/16", 4" |
| ≤ 5.0 | 4.0 ft. | 5.0 ft. | 5.5 ft. | 6.0 ft. | 6.0 ft. | 1-3/16", 4" |
| ≤ 5.5 | 4.5 ft. | 5.5 ft. | 6.0 ft. | 6.5 ft. | 7.0 ft. | 1-3/16", 4" |
| ≤ 6.0 | 5.0 ft. | 6.0 ft. | 6.5 ft. | 7.0 ft. | 7.5 ft. | 1-3/16", 4" |
| ≤ 6.5 | 5.5 ft. | 6.5 ft. | 7.0 ft. | 7.5 ft. | 8.0 ft. | 1-3/16", 4" |
| ≤ 7.0 | 6.0 ft. | 7.0 ft. | 7.5 ft. | 8.5 ft. | 8.5 ft. | 1-3/16", 4" |
| ≤ 7.5 | 6.0 ft. | 7.5 ft. | 8.5 ft. | 9.0 ft. | 9.5 ft. | 1-3/16", 4" |
| ≤ 8.0 | 6.5 ft. | 8.0 ft. | 9.0 ft. | 9.5 ft. | 10.0 ft. | 1-3/16", 4" |
| ≤ 8.5 | 7.0 ft. | 8.5 ft. | 9.5 ft. | 10.0 ft. | N/A | 1-3/16", 4" |
| ≤ 9.0 | 7.5 ft. | 9.0 ft. | 10.0 ft. | N/A | N/A | 1-3/16", 4" |
| ≤ 9.5 | 8.0 ft. | 9.5 ft. | N/A | N/A | N/A | 1-3/16", 4" |
| ≤ 10.0 | 8.5 ft. | 10.0 ft. | N/A | N/A | N/A | 1-3/16", 4" |
| ≤ 10.5 | 8.5 ft. | N/A | N/A | N/A | N/A | 1-3/16", 4" |
| ≤ 11.0 | 9.0 ft. | N/A | N/A | N/A | N/A | 1-3/16", 4" |
| ≤ 11.5 | 9.5 ft. | N/A | N/A | N/A | N/A | 1-3/16", 4" |
| ≤ 12.0 | 10.0 ft. | N/A | N/A | N/A | N/A | 1-3/16", 4" |

**Table 6b-2—Standardized WQCV Outlet Design Using 2" High Rectangular Openings.**

Klemp™ KRP Series Aluminum Bar Grate[1] (or equal) Trash Rack Design Specifications.

| Water Depth Above Lowest Opening, H | Minimum Bearing Bar Size, Bearing Bars Aligned Vertically |
|---|---|
| 2.0 ft. | 1" x 3/16" |
| 3.0 ft. | 1-1/4" x 3/16" |
| 4.0 ft. | 1-3/4" x 3/16" |
| 5.0 ft. | 2" x 3/16" |
| 6.0 ft. | 2-1/4" x 3/16" |

[1] Klemp Corporation, Orem, Utah, USA

DESIGN EXAMPLE:

Given: A WQCV outlet plate with one column of 2 inch high by 6.5 inch wide rectangular openings. Water Depth H = 4.5 feet above the lowest opening of and 2.5 feet below the lowest perforation.

Find: The dimensions for an aluminum bar grate trash rack.

Solution: Using Table 6b-1for openings having a width of 6.5 inches and Water Depth H = 5 feet (i.e., rounded up from 4.5 feet). The minimum width of the concrete opening at the trash rack, $W_{opening}$, is 7'-6".

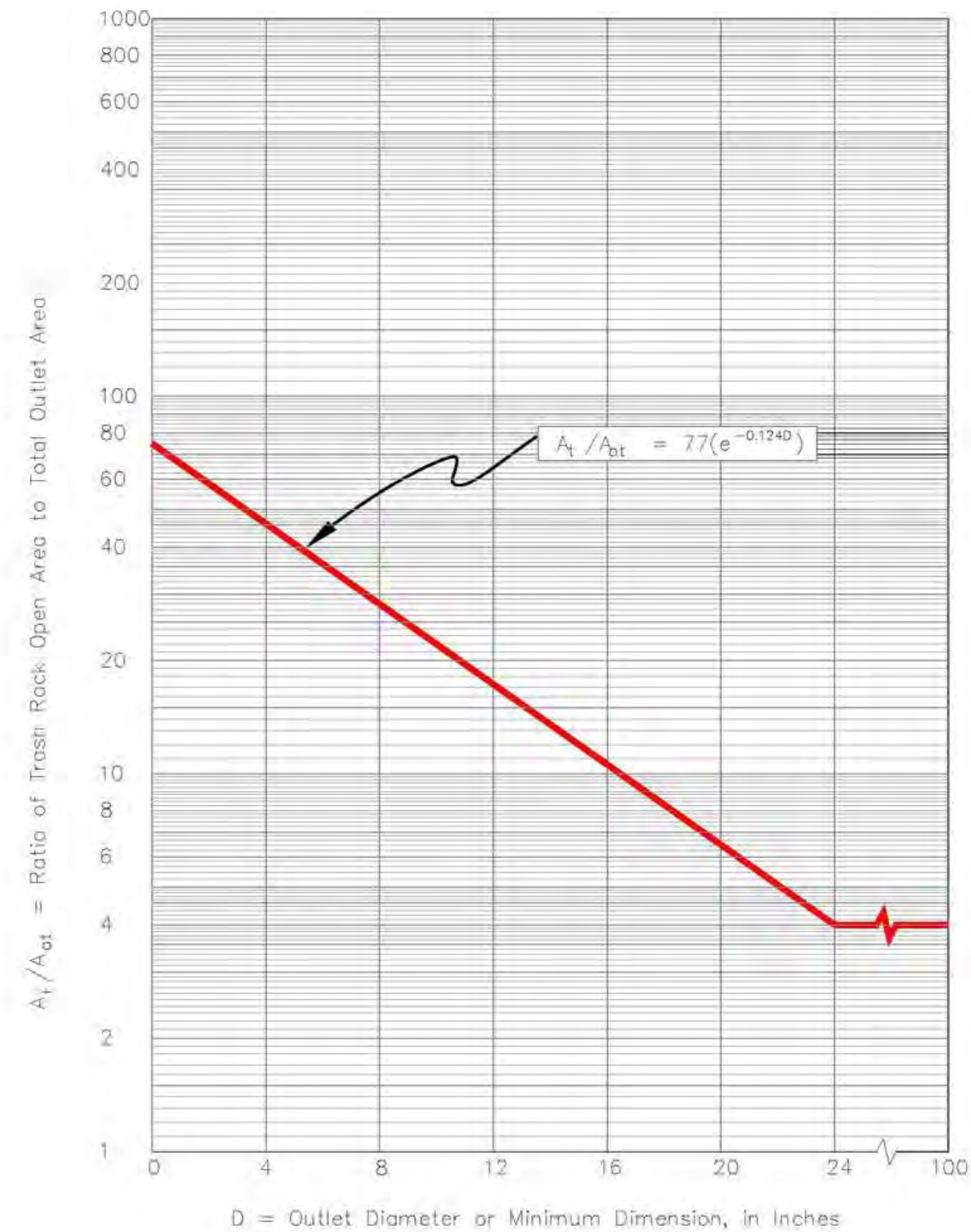
From Fig's 4 and 5, the total width of concrete opening at the orifice plate, $W_{conc.}$, is 6.5" + 12" = 18.5 inches (say 19 inches). Suggest setting the total width of the orifice plate at 26 inches.

Total minimum height of the rack structure (adding the 2-feet below the lowest row of openings to the water depth of 4.5 feet and adding 3 inches for all support channel flanges around the perimeter of the grate gives us the following total dimensions for the trash rack:

Width = 8'-0"

Height = 7'-0"

From Tables 6b-1 and 6b-2, select the ordering specifications for W = 6.5" and H = 5.0 feet or less, a 8'-0" wide by 7'-0" high trash rack using Klemp Corporation aluminum bar grate (or equal) with 2"x3/16" bearing bars spaced on 1-3/16" centers and cross rods spaced on 4"centers. **Bearing bars are to be aligned vertically.**




| | |
|---|---|
| Urban Drainage and Flood Control District<br>Drainage Criteria Manual (V.3)<br>File: V3-Trash Rack Sizing Graph.dwg | Figure 7<br>Minimum Trash Rack Open Area – Extended Range |

**Figure 7—Minimum Trash Rack Open Area – Extended Range.**

## SUGGESTED TECHNICAL SPECIFICATIONS

This section contains suggested technical specifications for some of the designs contained in this *Manual*. These specifications may not contain many of the provisions typically seen in construction specifications such as pay item definitions and bid items. The intent that these suggested technical specifications is for the engineer to incorporate the technical portions into specific project specifications after reviewing and, if the site conditions dictate, modify them to best fit the specific site conditions of the project. When modifying these suggested specifications the engineer should maintain the general technical requirements recommended in these technical specifications.

**Special Note and Disclaimer:**

*This information is intended for use by the design professional competent to evaluate its significance and limitation and who will accept the responsibility for its proper application. The Urban Drainage and Flood Control District and its employees disclaim any and all responsibility for any use of the information supplied herein and it is the responsibility of the user to apply and modify these guidelines as needed to meet specific site and design needs.*

December 2004

**Table 1—Typical Notes For EDB, RP and CWB Outlet Structures**

| | |
|---|---|
| 1. **The details shown herein** are conceptual design in nature. Preparation of final design plans that address details of structural adequacy, excavation, foundation preparation, concrete work, reinforcing steel, backfill, metalwork, and appurtenances, including preparation of technical specifications, are the responsibility of the design engineer in charge of the project. | |
| 2. **Alternate designs** to the typical outlet structures shown herein may be considered; however, alternate designs must address the hydraulic and trash handling functional features and intent for the structures shown in this *Manual.* | |
| 3. **Wingwalls** shown herein are designed to have the structure to be backfilled to be flush with the side slopes of the basin. The use of this geometry is recommended and permits the structure to blend into the landscape most aesthetically. Other geometries may be considered, however they need to be developed with full consideration of public safety, aesthetics, maintainability, and function. The superiority of these design should be demonstrated to be equal to or better than the design concepts shown in this *Manual.* | |
| 4. **Permanent Water Surface** refers to the water surface of the micro-pool for Extended Detention Basins and the permanent pool for Constructed Wetland Basins and Retention Ponds. | |
| 5. **Perforated orifice plate** shown herein is used to provide the specified the drain time of the WQCV. To reduce clogging potential, it is recommended that the largest possible circular opening be selected to minimize the number of columns. The intent is to have an outlet that empties the WQCV in the time specified (e.g., 12-, 24- or 40-hours), and being within –3% to 5% of this time is considered acceptable. See Figure 4 for orifice design information. | |
| 6. **Vertical Trash Rack** option is preferred; however, an Adverse-Slope Trash Rack is also acceptable. Both help to shed the accumulated trash as the water level after the storm recedes. The use of a Continuous-Slope Trash Rack for WQCV outlets is not recommended. See Figure 6 for trash rack design information. | |
| 7. **Drainage or flood control detention** above the WQCV may be sized for any storm event specified by local stormwater criteria and not only to the 2- or 10-year events shown herein. | |
| 8. **Underdrains** along the perimeter of the permanent pool, including a shutoff valve, are recommended for Constructed Wetland Basins and a Retention Ponds to help dewater the pool for rehabilitative maintenance. | |
| 9. **When the outlet designs differ from those shown herein**: | |
| a) Provide needed orifices that are distributed over the vertical height of the WQCV, with the invert of the lowest orifice located at 2'-6", or more above the bottom of the micro-pool and above the bottom of Retention Ponds and Constructed Wetland Basins. | |
| b) Provide full hydraulic calculations demonstrating that the outlet will provide the minimum required drain time of the Water Quality Capture Volume for the BMP type being used. | |
| c) Outlet openings (orifices) shall be protected by trash racks having a minimum net open area specified in Fig 7. All opening dimensions shall be less than any dimension of outlet openings. | |
| d) Trash racks shall be manufactured from stainless steel or aluminum alloy structurally designed to not fail under a full hydrostatic load on the upstream side and assuming zero backwater depth on the downstream side. | |
| **Urban Drainage and Flood Control District Drainage Criteria Manual (V.3)** | **Typical Outlet Structure General Notes for Extended Detention Basin, Retention Pond and Constructed Wetland Basin Outlets** |