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Hydrologic Criteria and Drainage Design Manual

DRAFT

This document is available for $__.00 from the Resource Planning & Management Division, Washoe County Department of Water Resources.

FIRST PRINTING, DECEMBER 1996
SECTION 100
INTRODUCTION

GENERAL INDEX

SECTION 100 - INTRODUCTION
SECTION 200 - GENERAL PROVISIONS
SECTION 300 - DRAINAGE POLICY
SECTION 400 - DRAINAGE LAW
SECTION 500 - DRAINAGE PLANNING AND SUBMITTAL
SECTION 600 - RAINFALL
SECTION 700 - STORM RUNOFF
SECTION 800 - OPEN CHANNELS
SECTION 900 - STORM SEWER SYSTEMS
SECTION 1000 - STREETS
SECTION 1100 - CULVERTS AND BRIDGES
SECTION 1200 - ADDITIONAL HYDRAULIC STRUCTURES
SECTION 1300 - DETENTION
SECTION 1400 - EROSION AND SEDIMENTATION
SECTION 1500 - STANDARD FORMS
SECTION 1600 - REFERENCES
AMENDMENTS AND REVISIONS

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

Users of this MANUAL are encouraged by Washoe County to submit their comments so that changes can be made. This information should be addressed to:

Mr. David T. Price, P.E.
Washoe County Engineer
Washoe County Public Works Department, Engineering Division
P. O. Box 11130
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Reno, Colorado 89520-0027

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

Return to: Mr. Leonard Crowe, P.E.
Water Resources Planning Manager
Washoe County Department of Water Resources
P. O. Box 11130
1001 East Ninth Street
Reno, Nevada 89520-0027

RE: Hydrologic Criteria and Drainage Design Manual

NAME:__________________________

COMPANY:__________________________

MAILING ADDRESS:__________________________

DATE MANUAL RECEIVED:__________________________
ACKNOWLEDGEMENTS

Washoe County and WRC Engineering, Inc. wish to acknowledge and thank all individuals and committee members who provided technical review of the draft standards and criteria during the preparation of this MANUAL. We wish to specifically acknowledge the contributions of the following members of the Technical Advisory Committee who provided specific input to the MANUAL:

Mr. David T. Price; Mr. Leonard Crowe; Ms. Kris Klein - Washoe County
Mr. Peter Etchart - SEA Engineering Representing Consulting Engineers Council of Nevada
Mr. Bob Gottsacker; Mr. Glen Daily - City of Reno
Mr. Kirk Nichols - W.C. Engineering
Mr. John McClung - Natural Resources Conservation Service (NRCS)
Mr. Mark Forest - Harding Lawson Alpha Representing American Society of Civil Engineers
Mr. John Fordham - Desert Research Institute
Mr. Amir Soltani - Nevada Department of Transportation
Mr. Jay Aldean; Mr. Tim Homann - Carson City

The following individuals on the staff of WRC Engineering, Inc. have contributed to the preparation and completion of this MANUAL:

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Mrs. Janet L. Brockman - Word Processing

The following individuals prepared selected sections of the MANUAL for WRC Engineering, Inc.:

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Mr. Michael K. Mansfield - Attorney-at-Law
# SECTION 200
## GENERAL PROVISIONS

## TABLE OF CONTENTS

| 201 | TITLE | 201 |
| 202 | ADOPTION AUTHORITY | 201 |
| 203 | JURISDICTION | 201 |
| 204 | PURPOSE | 201 |
| 205 | ENFORCEMENT RESPONSIBILITY | 201 |
| 206 | VARIANCE PROCEDURES | 201 |
| 207 | INTERPRETATION | 202 |
| 208 | REVIEW AND APPROVAL | 202 |
| 209 | IMPLEMENTATION | 202 |
| 209.1 | Development of the Manual | 202 |
| 209.2 | Updates | 203 |
| 209.3 | Adoption | 203 |
| 209.4 | Reconciliation of Pre- and Post-Manual Studies | 203 |
| 210 | ACRONYMS | 204 |
SECTION 200
GENERAL PROVISIONS

201 TITLE

These criteria and design standards with all future amendments and revisions shall be known as the "WASHOE COUNTY HYDROLOGIC CRITERIA AND DRAINAGE DESIGN MANUAL" (herein referred to as the MANUAL).

202 ADOPTION AUTHORITY

NRS 278.326 authorizes the adoption of ordinances that specify improvements that must be made (Articles 416 and 420 of Washoe County Development Code).

Articles 416 (Flood Hazards) and 420 (Storm Drainage Standards) of Washoe County Development Code establishes guidelines and requirements for development of properties within areas subject to flooding, and sets standards for development of drainage and flood control facilities within Washoe County.

203 JURISDICTION

These criteria and design standards shall apply to all areas within the boundaries of Washoe County excluding the Cities of Reno and Sparks, the Lake Tahoe Basin, the Pyramid Lake Indian Reservation, and the Reno Indian Colony.

204 PURPOSE

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

205 ENFORCEMENT RESPONSIBILITY

Washoe County is charged with enforcement of the MANUAL for all Flood Control Facilities. Each, entity adopting this MANUAL, is responsible for its enforcement within its jurisdictional boundaries for Flood Control Facilities.

206 VARIANCE PROCEDURES

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

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

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

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

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

207 INTERPRETATION

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

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

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

208 REVIEW AND APPROVAL

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

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

209 IMPLEMENTATION

209.1 DEVELOPMENT OF THE MANUAL

Washoe County has developed this MANUAL for use by the Adopting Entities, and consulting engineers. This MANUAL shall be used for the development and design of all Flood Control Facilities and any other facilities dedicated to a public entity for ownership and maintenance.
Respective Entities have been afforded the opportunity to participate in the development of this MANUAL and will be given the opportunity to also participate in subsequent updates.

209.2 UPDATE

The MANUAL will be updated from time to time as determined to be necessary by the County Engineer. The process by which these updates will be accomplished will be dependant upon the nature of the update and will be determined by the County Engineer.

209.3 ADOPTION

The Board of County Commissioners of Washoe County should adopt this MANUAL and all subsequent updates thereto. The Adopting Entities should also consider doing the same.

209.4 RECONCILIATION OF PRE- AND POST-MANUAL STUDIES

1. Developments for which the technical drainage reports or construction drawings have been approved are exempt from the provisions of this MANUAL.

2. Developments for which a conceptual drainage report has been approved are exempt from the provisions of this MANUAL if a technical drainage report and/or analysis is submitted for review within 180 days of the initial adoption of this MANUAL.

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

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

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

(b) Facilities planned but not under construction at the time of initial MANUAL adoption shall also be analyzed as discussed in 209.4.4(a) above. However, if the facility does not have adequate capacity including freeboard, the facility shall be redesigned in accordance with the requirements of this MANUAL.

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

The following acronyms are used within the contents of this MANUAL.

<table>
<thead>
<tr>
<th>Acronym</th>
<th>Description</th>
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<tbody>
<tr>
<td>CAP</td>
<td>Corrugated Aluminum Pipe</td>
</tr>
<tr>
<td>CAPA</td>
<td>Corrugated Aluminum Pipe Arch</td>
</tr>
<tr>
<td>CEC</td>
<td>Consulting Engineers Council</td>
</tr>
<tr>
<td>CMP</td>
<td>Corrugated Metal Pipe</td>
</tr>
<tr>
<td>CMPA</td>
<td>Corrugated Metal Pipe Arch</td>
</tr>
<tr>
<td>CSP</td>
<td>Corrugated Steel Pipe</td>
</tr>
<tr>
<td>CSPA</td>
<td>Corrugated Steel Pipe Arch</td>
</tr>
<tr>
<td>EGL</td>
<td>Energy Grade Line</td>
</tr>
<tr>
<td>FEMA</td>
<td>Federal Emergency Management Agency</td>
</tr>
<tr>
<td>HDS</td>
<td>Hydraulic Design Series</td>
</tr>
<tr>
<td>HEC</td>
<td>Hydraulic Engineering Circular</td>
</tr>
<tr>
<td>HERCP</td>
<td>Horizontal Elliptical Reinforced Concrete Pipe</td>
</tr>
<tr>
<td>HGL</td>
<td>Hydraulic Grade Line</td>
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<tr>
<td>NDOT</td>
<td>Nevada Department of Transportation</td>
</tr>
<tr>
<td>NFIP</td>
<td>National Flood Insurance Program</td>
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<tr>
<td>NOAA</td>
<td>National Oceanic and Atmospheric Administration</td>
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<tr>
<td>NRS</td>
<td>Nevada Revised Statues</td>
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<tr>
<td>NWS</td>
<td>National Weather Service</td>
</tr>
<tr>
<td>PE</td>
<td>Professional Engineer Licensed by the State of Nevada</td>
</tr>
<tr>
<td>PF</td>
<td>Probable Maximum Flood</td>
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<tr>
<td>RCBC</td>
<td>Reinforced Concrete Box Culvert</td>
</tr>
<tr>
<td>RCP</td>
<td>Reinforced Concrete Pipe</td>
</tr>
<tr>
<td>ROW</td>
<td>Right-of-Way</td>
</tr>
<tr>
<td>Acronym</td>
<td>Description</td>
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<td>---------</td>
<td>--------------------------------------------------</td>
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<tr>
<td>SCS</td>
<td>Soil Conservation Service</td>
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<tr>
<td>SPP</td>
<td>Structural Plate Pipe</td>
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<tr>
<td>TAC</td>
<td>Technical Advisory Committee</td>
</tr>
<tr>
<td>TRC</td>
<td>Technical Review Committee</td>
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<tr>
<td>USACE</td>
<td>United States Army Corps of Engineers</td>
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<tr>
<td>USBR</td>
<td>United States Bureau of Reclamation</td>
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<tr>
<td>USGS</td>
<td>United States Geological Survey</td>
</tr>
</tbody>
</table>
### SECTION 300

**DRAINAGE POLICY**

#### TABLE OF CONTENTS

<table>
<thead>
<tr>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>301</td>
</tr>
</tbody>
</table>

**301 STATUTORY AUTHORITY**

**302 BASIC PRINCIPLES**

- Drainage Planning and Required Space
- Multi-Purpose Resource
- Water Rights
- Jurisdictional Cooperation

**303 REGIONAL AND LOCAL PLANNING**

- Reasonable Use Rule
- Regional Master Planning
- Local Master Planning
- Drainage Improvements
- Drainage Planning Submittal and Review
- Floodplain Management
- Storm Runoff Detection
- Storm Runoff Retention
- Water Quality and Construction Activities
- Drainage Facilities Maintenance

**304 TECHNICAL CRITERIA**

- Stormwater Management Technology
- Design Storm Events
- Storm Runoff Determination
- Streets
- Culverts and Bridges
- Floodproofing
- Alluvial Fans

**305 IRRIGATION FACILITIES**

- Drainage Interaction
- Irrigation Ditches

**306 PRESERVATION OF NATURAL DRAINAGEWAYS**

December 2, 1996

Drainage Policy
SECTION 300

DRAINAGE POLICY

301 STATUTORY AUTHORITY

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

NRS 278.026 to 278.029, inclusive, have allowed in Washoe County the creation of the Truckee Meadows Regional Planning Commission and a Regional Governing Board. The regional planning commission shall develop a comprehensive regional plan that includes, among other things, flood control facilities. The regional planning commission may designate certain areas in the comprehensive regional plan as joint planning areas, thus allowing the County and the affected cities to jointly adopt a master plan for the areas so designated. The Truckee Meadows Regional Planning Commission has developed the Truckee Meadows Regional Plan. This plan has been adopted by the Regional Planning Governing Board. NRS 278.326 authorized the adoption of ordinances that specify improvements that must be made (Articles 416 and 420 of Washoe County Development Code).

NRS 487 creates the Regional Water Planning and Advisory Board of Washoe County (adopted name - Regional Water Management Agency/RWMA). Washoe County, as the Regional Water Management Agency (RWMA), has been delegated the authority to prepare the Regional Water Service and Facilities Plan. The Service and Facilities Plan, including storm runoff, shall be consistent with the Regional Plan. This Manual will be available to RWMA at such time when a district is formed as per requirements of NRS 543, and is intended to apply to all areas in Washoe County outside the Cities of Reno and Sparks, the Lake Tahoe Basin, the Pyramid Lake Indian Reservation, and the Reno Indian Colony. Articles 416 (Flood Hazards) and 420 (Storm Drainage Standards) of Washoe County Development Code establishes guidelines and requirements for development of properties within areas subject to flooding, and sets standards for development of drainage and flood control facilities within Washoe County.

302 BASIC PRINCIPLES

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

302.1 DRAINAGE PLANNING AND REQUIRED SPACE

The Stormwater Drainage System is an integral part of the total urbanization process. The planning of drainage facilities must be included in the urbanization process. The first step is to include drainage planning with all regional and local development master plans.
Drainage systems require space to accommodate their conveyance and storage functions. When the space requirements are considered, the provision for adequate drainage becomes a competing use for space along with other land uses. If adequate provision is not made in a land use plan for the drainage requirements, stormwater runoff will conflict with other land uses and will result in water damages, and will impair or even disrupt the functioning of other urban systems.

THE POLICY OF WASHOE COUNTY SHALL BE TO CONSIDER STORMWATER DRAINAGE AN INTEGRAL PART OF THE OVERALL URBAN SYSTEM, AND REQUIRE THAT ALL DEVELOPMENTS PROVIDE STORM DRAINAGE PLANNING THAT INCLUDES THE ALLOCATION OF SPACE FOR DRAINAGE FACILITY CONSTRUCTION AND MAINTENANCE, WHICH MAY ENTAIL THE DEDICATION OF RIGHT-OF-WAY AND/OR EASEMENTS.

302.2 MULTI-PURPOSE RESOURCE

Stormwater runoff is an integral part of Washoe County's surface and groundwater resources. This resource has the potential of being utilized for different beneficial uses. These uses, however, must be compatible with adjacent land uses and applicable State Water Laws.

THE POLICY OF WASHOE COUNTY SHALL BE TO CONSIDER STORM WATER RUNOFF AS AN INTEGRAL PART OF AREA'S SURFACE AND GROUNDWATER RESOURCES AND RECOGNIZE ITS POTENTIAL FOR OTHER USES.

302.3 WATER RIGHTS

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

THE POLICY OF WASHOE COUNTY SHALL BE TO RECOGNIZE THE EXISTENCE OF VESTED WATER RIGHTS AND TO ABIDE BY ANY AGREEMENTS IN WHICH THE COUNTY HAS RELINQUISHED ITS RIGHTS TO APPROPRIATE UNAPPROPRIATED WATER IN THE TRUCKEE RIVER BASIN.

302.4 JURISDICTIONAL COOPERATION

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

THE POLICY OF WASHOE COUNTY SHALL BE TO PURSUE A JURISDICTIONALLY UNIFIED DRAINAGE EFFORT TO PROMOTE AN INTEGRATED COMPREHENSIVE REGIONAL DRAINAGE PLAN.
303 REGIONAL AND LOCAL PLANNING

303.1 REASONABLE USE RULE

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

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.

THE POLICY OF WASHOE COUNTY REGARDING THE "REASONABLE USE RULE" IS TO:

1. LIMIT THE RATE OF FLOW FROM DEVELOPING PROPERTIES TO THEIR PREDEVELOPMENT CONDITION FLOW RATES. THE COUNTY WOULD CONSIDER PLANS TO ACCOMMODATE THIS LIMITATION BY CONSTRUCTION OF LOCAL ON-SITE FACILITIES OR REGIONAL FACILITIES.

2. TRANSITION FLOWS FROM DEVELOPING PROPERTIES TO THEIR PREDEVELOPMENT PATHS ON DOWNSTREAM PROPERTIES.

3. MAINTAIN FLOWS IN THEIR NATURAL DRAINAGE BASINS.

303.2 REGIONAL MASTER PLANNING

Washoe County has prepared a conceptual level Flood Control Master Plan. The next phase of the flood control master planning is to develop a detailed regional master plan, which will be adopted by the County and affected local entities. All regional facilities, with or without a regional master plan, must be so designated by Washoe County and the local entities.

THE POLICY OF WASHOE COUNTY SHALL BE TO DEVELOP AND ADOPT A REGIONAL FLOOD CONTROL MASTER PLAN, AND REGULATE IN A MANNER CONSISTENT WITH SUCH PLAN. THE REGIONAL PLAN SHALL INCLUDE PLANNING COMPLETED OR UNDERTAKEN BY THE CITIES OF RENO AND SPARKS, AND DEVELOPERS. THE COUNTY RECOGNIZES THE NEED TO REVIEW THE PLAN ANNUALLY AND UPDATE IT NOT LESS THAN EVERY 5 YEARS (NRS 278.0272).

303.3 LOCAL MASTER PLANNING

Local Flood Control Facilities, as planned by Washoe County, local entities, and developers, are an integral part of the total drainage system required to preserve and promote the general health, welfare, and economic well being of the area. Any facility that generates benefits exclusively to the benefit of the local entity, or is not designated as regional facility, shall be considered a local facility.
THE POLICY OF WASHOE COUNTY SHALL BE TO DEVELOP FLOOD CONTROL FACILITIES WHICH ARE COMPATIBLE WITH THE WASHOE COUNTY REGIONAL FLOOD CONTROL MASTER PLAN.

303.4 DRAINAGE IMPROVEMENTS

Drainage improvements include those in the Washoe County Regional Flood Control Master Plan, new development drainage plans, and basin management plans. The Local Flood Control Facilities consist of curb and gutter, inlets and storm sewers, culverts, bridges, swales, ditches, channels, detention areas, and other drainage facilities required to convey the minor and major storm runoff to its ultimate major drainageway. These Local Flood Control Facilities are further defined as on-site or off-site (private) facilities and off-site (public) facilities. The on-site and off-site (private) facilities serve a specific development and are privately owned and maintained. The off-site (public) facilities are facilities which are dedicated to the public and are publicly maintained. These off-site (public) facilities may actually be constructed within the specific development to pass through flow from upstream properties. The major drainageway Flood Control Facilities consist of channels, storm drains, bridges, detention areas, and other facilities which carry runoff from on-site and off-site facilities to an ultimate outfall location. The management of all privately maintained facilities must be acceptable to Washoe County.

When capital improvement plans identify that drainage improvements are justified, NRS 278B provides the mechanism for funding the required improvements. The funding for public improvements which serve only a single development shall be obtained from that development. This funding is provided by having these public improvements designed and constructed by the subject development.

THE POLICY OF WASHOE COUNTY SHALL BE THAT ALL NEW DEVELOPMENT PLAN, DESIGN, CONSTRUCT, AND MAINTAIN THE REQUIRED DRAINAGE IMPROVEMENTS IN ACCORDANCE WITH THE FOLLOWING:

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

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

3. Regional Flood Control Facilities passing through or directly adjacent to the subject development. WASHOE COUNTY may participate in funding of these regional improvements if the improvements are designed, constructed and implemented by the County, and in accordance with the Regional Master Plan and this MANUAL.

4. Regional facilities be designed to accommodate multi-purpose uses.

5. Maintenance shall be in accordance with Section 303.10 of this MANUAL.
303.5 DRAINAGE PLANNING SUBMITTAL AND REVIEW

Review and acceptance of drainage plans, studies, and construction drawings and specifications by WASHOE COUNTY is required to obtain a final drainage system which is consistent and integrated in analysis, design, and level of protection. The degree of review depends on the complexity of the drainage improvement under consideration.

THE POLICY OF WASHOE COUNTY IS TO REQUIRE THAT ALL DRAINAGE PLANS, STUDIES, AND CONSTRUCTION DOCUMENTS BE SUBMITTED FOR REVIEW AND ACCEPTANCE BY THE PUBLIC WORKS DEPARTMENT AND BE CONSISTENT WITH AN APPLICABLE BASIN MANAGEMENT PLAN AND REGIONAL MASTER PLAN.

State Agencies shall consider and, when applicable, comply with WASHOE COUNTY’s Master Plan when planning and designing their flood control facilities.

303.6 FLOODPLAIN MANAGEMENT

Article 416 of Washoe County Development Code establishes guidelines and requirements for development of properties within areas subject to flooding. The purpose of floodplain management is to provide the guidance, conditions, and restrictions for development in floodplain areas while protecting the public’s health, safety, welfare, and property from danger and damage.

To provide impetus for proper floodplain management, the United States government, acting through the Federal Emergency Management Agency’s (FEMA) National Flood Insurance Program (NFIP), has established regulations for development in floodplain areas. Compliance with these regulations allows property owners to obtain lower cost flood insurance premiums and/or eliminates the requirement for the owner to obtain flood insurance as a condition for obtaining government supported loans. Therefore, there is a benefit to WASHOE COUNTY population for remaining in compliance with the NFIP’s regulations, and further allows Washoe County to maintain eligibility for federal disaster relief funds.

THE POLICY OF WASHOE COUNTY SHALL BE TO REGULATE FLOODPLAINS IN ACCORDANCE WITH THE PROVISIONS OF COUNTY’S DEVELOPMENT CODE AND THE REGULATIONS OF THE NATIONAL FLOOD INSURANCE PROGRAM (NFIP).

303.7 STORM RUNOFF DETENTION

Detention is considered a viable method to reduce urban drainage costs. Temporarily detaining storm runoff can significantly reduce downstream flood hazards as well as reduce pipe and channel sizes in urban areas. 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 in their original or previously improved condition are adequate in capacity to carry flows from fully developed upstream areas without negatively impacting downstream properties.

THE POLICY OF WASHOE COUNTY SHALL BE TO REQUIRE LOCAL DETENTION STORAGE FOR NEW DEVELOPMENTS TO LIMIT FLOWS FROM A 5-YEAR STORM (Q₅) TO THEIR PRE-DEVELOPMENT CONDITIONS. ADDITIONALLY WASHOE COUNTY REQUIRES DETENTION OF OTHER STORMS UP TO AND INCLUDING THE 100-YEAR...

December 2, 1996

Drainage Policy
STORM \( (Q_{100}) \) IF, IN THE OPINION OF THE COUNTY ENGINEER, THE CAPACITY OF THE DOWNSTREAM STORM DRAINAGE FACILITIES WILL BE EXCEEDED.

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

EXEMPTIONS TO THE DETENTION POLICY MAY BE GRANTED BY WASHOE COUNTY FOR THE FOLLOWING:

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

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

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

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

5. UPGRAADING OF DOWNSTREAM FACILITIES TO ACCOMMODATE THE INCREASED FLOW RATE.

6. WHERE THE DOWNSTREAM CAPACITIES ARE ADEQUATE TO CARRY UP TO 100-YEAR FLOWS.

ALL EXEMPTIONS ARE SUBJECT TO APPROVAL OF WASHOE COUNTY.

303.8 STORM RUNOFF RETENTION

Storm Runoff retention has been used to eliminate the need for constructing outlet structures and for ease of construction. However, problems with past retention basins including soil expansion, siltation, and lack of infiltration capacity have created a nuisance to the general public. Further, runoff retention has the potential of depriving downstream water rights of their legal right to the retained water. Each potential site will have different site constraints which will require individual evaluation of suitability for retention purposes at said site.

THE POLICY OF WASHOE COUNTY SHALL BE TO MINIMIZE THE USE OF RETENTION FACILITIES, EXCEPT WHERE SIGNIFICANT ENVIRONMENTAL, RECREATION OR
RECHARGE BENEFITS ARE APPARENT. STANDARDS FOR DESIGN OF SUCH FACILITIES WILL BE ESTABLISHED BY WASHOE COUNTY ON A SITE BY SITE BASIS.

303.9 WATER QUALITY AND CONSTRUCTION ACTIVITIES

A number of studies by the Environmental Protection Agency (EPA) and others have shown that site disturbances due to construction and resulting urbanization decreases the quality of runoff from the natural conditions. WASHOE COUNTY recognizes that drainage facilities which enhance water quality may be needed in the future; and measures, methods of operation or construction practices are needed to control degradation of water quality. The County has jurisdiction over construction project storm water pollution prevention plans for sites that range in size between 1 and 5 acres, while the State Division of Environmental Protection has jurisdiction over sites that are 5 acres and larger.

THE POLICY OF WASHOE COUNTY SHALL BE TO ENCOURAGE THE DESIGN OF DRAINAGE FACILITIES AND OTHER MEASURES THAT ENHANCE THE QUALITY OF STORM RUNOFF. THE COUNTY AND STATE NPDES STORMWATER DISCHARGE PERMIT PROGRAMS REQUIRE THAT STORM WATER POLLUTION PREVENTION PLANS (SWPPP) FOR CONSTRUCTION ACTIVITIES BE PREPARED AND IMPLEMENTED. THESE PLANS WILL BE PREPARED USING THE LATEST EDITIONS OF THE COUNTY'S OR STATE'S BEST MANAGEMENT PRACTICE HANDBOOK DEPENDING UPON JURISDICTION.

303.10 DRAINAGE FACILITIES MAINTENANCE

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

THE POLICY OF WASHOE COUNTY SHALL BE TO REQUIRE THAT ALL DRAINAGE FACILITIES BE DESIGNED TO MINIMIZE FACILITY MAINTENANCE AS WELL AS TO PROVIDE EASE OF MAINTENANCE AND INCLUDE MAINTENANCE ACCESS TO THE ENTIRE DRAINAGE FACILITY. A MINIMUM 15 FOOT WIDE DRAINAGE EASEMENT SHALL BE PROVIDED FOR ALL PUBLICLY AND PRIVATELY MAINTAINED DRAINAGE FACILITIES.

THE POLICY OF WASHOE COUNTY IS TO REQUIRE THE PROPERTY OWNER OR DEVELOPER TO PROVIDE AN ACCEPTABLE AND PERPETUAL FUNDING AND MAINTENANCE OF ALL PRIVATELY OWNED OR OTHER NON-COUNTY MAINTAINED ON-SITE DETENTION BASINS AS WELL AS OFF-SITE DRAINAGE FACILITIES INCLUDING, BUT NOT LIMITED TO, INLETS, PIPES, CHANNELS, AND DETENTION BASINS, UNLESS MODIFIED BY SEPARATE AGREEMENT. SHOULD THE PROPERTY OWNER OR DEVELOPER FAIL TO ADEQUATELY MAINTAIN SAID FACILITIES, THE GOVERNING ENTITY SHALL BE GIVEN THE RIGHT TO ENTER SAID PROPERTY, UPON PROPER NOTICE, FOR THE PURPOSES OF MAINTENANCE. ALL SUCH MAINTENANCE
COSTS, NOT COVERED BY THE FUNDING PROVIDED, SHALL BE ASSESSED AGAINST THE OWNER. THE COUNTY ENGINEER SHALL APPROVE THE TYPE OF FUNDING, MAINTENANCE AND THE SCHEDULES ASSOCIATED WITH SUCH WORK.

THE POLICY OF WASHOE COUNTY SHALL BE TO INSURE THAT ALL PUBLIC FLOOD CONTROL FACILITIES ARE PROPERLY MAINTAINED AND TO FUND THE MAINTENANCE OF PUBLIC FLOOD CONTROL FACILITIES. BY DEFINITION PUBLIC FLOOD CONTROL FACILITIES ARE EITHER PIPED OR CONCRETE LINED FACILITIES, OR ARE FACILITIES THAT ARE FOUND BY THE COUNTY ENGINEER TO PROVIDE SIGNIFICANT FLOOD CONTROL BENEFITS TO DOWNSTREAM PROPERTIES OR THAT REDUCE AN EXISTING DOWNSTREAM FLOOD HAZARD. THE COUNTY WILL REQUIRE MAINTENANCE FUNDING SIMILAR TO PRIVATE FACILITIES FOR RIPRAP LINED CHANNELS.

304 TECHNICAL CRITERIA

304.1 STORMWATER MANAGEMENT TECHNOLOGY

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

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

304.2 DESIGN STORM EVENTS

WASHOE COUNTY has determined that drainage facilities should, as a minimum, be designed based on runoff from the MINOR storm event (5-year) and a MAJOR storm event (100-year).

THE POLICY OF WASHOE COUNTY SHALL BE TO REQUIRE THAT ALL NEW DEVELOPMENT INCLUDE THE PLANNING, DESIGN, AND CONSTRUCTION OF DRAINAGE FACILITIES FOR BOTH THE MINOR AND MAJOR 24-HOUR DESIGN STORM EVENTS. WHEN REQUIRED BY THE COUNTY ENGINEER, THESE FACILITIES SHALL INCLUDE EMERGENCY FLOW PATHS FOR FLOWS EXCEEDING THE MAJOR STORM. THE MINOR STORM EVENT SHALL HAVE A RECURRENCE INTERVAL OF 5 YEARS, WHILE THE MAJOR STORM EVENT SHALL HAVE A RECURRENCE INTERVAL OF 100 YEARS.

304.3 STORM RUNOFF DETERMINATION

The storm runoff peak, volume, and timing provide the basis for all planning, design, and construction of drainage facilities. The best method for determining storm runoff is to measure the runoff from a flood with a known intensity and recurrence interval. Since this approach is not practical in Washoe County due to lack of availability of long term rainfall/runoff data, various analytical methods have
been developed which predict the storm runoff from preselected hydrologic conditions (independent of chance). These methods are referred to as deterministic models.

THE POLICY OF WASHOE COUNTY SHALL BE TO REQUIRE THE DETERMINATION OF STORM RUNOFF (RATES AND VOLUMES) IN ACCORDANCE WITH THE FOLLOWING:

<table>
<thead>
<tr>
<th>CONTRIBUTING BASIN AREA</th>
<th>COMPUTATION PROCEDURE</th>
</tr>
</thead>
<tbody>
<tr>
<td>$A \leq 100$ ACRES</td>
<td>RATIONAL FORMULA, SCS TR-20, OR HEC-1 (SCS UNIT HYDROGRAPH OR KINEMATIC WAVE)</td>
</tr>
<tr>
<td>$10 \text{ S.M.} &gt; A &gt; 100$ ACRES</td>
<td>SCS TR-20 OR HEC-1 (SCS UNIT HYDROGRAPH OR KINEMATIC WAVE)</td>
</tr>
<tr>
<td>$A &gt; 10 \text{ S.M.}$</td>
<td>SCS TR-20 OR HEC-1 WITH COMPARISON TO PEAK FLOWS GENERATED BY A STATISTICAL ANALYSIS OF RUNOFF RECORDS WITHIN THE SAME OR ADJACENT DRAINAGE BASIN.</td>
</tr>
</tbody>
</table>

ALL STORM DRAIN PIPE SYSTEMS WITH A CONTRIBUTING AREA OF $\leq 100$ ACRES SHALL BE DESIGNED USING THE RATIONAL FORMULA.

ANY EXCEPTIONS TO THESE PROCEDURES MUST BE APPROVED BY THE COUNTY ENGINEER PRIOR TO THEIR SUBMITTAL TO WASHOE COUNTY.

304.4 STREETs

The use of streets to convey storm runoff, although naturally occurring, interferes with the primary function of the street for transportation purposes. Streets are, however, an important component in the storm drainage system due to their large storm runoff carrying capacity obtained for little or no drainage related costs. In order to balance these two competing street uses, limits on the street carrying capacity are required based on the classification of the street related to emergency usage during flood events.

THE POLICY OF WASHOE COUNTY SHALL BE TO LIMIT FLOODING OF STREETS TO THE FOLLOWING.

1. MINOR ON-SITE STORM EVENT ($Q_s$)
   A. Maximum velocity will be 6 feet per second.
   B. Runoff in excess of street capacity shall be piped.
C. Maximum limits of street inundation:

- Local: 12 foot width dry centered
- Collector: 18 foot width dry centered
- Arterial: 48 foot width dry centered

2. MAJOR ON-SITE STORM EVENT ($Q_{25a}$)

A. Contained within street R/W.
B. Maximum velocity will be 6 feet per second.
C. Maximum depth will be 1 foot at the gutter flowline.
D. Maximum limits of street inundation:

- Local: Street flooded
- Collector: 1 lane (12 feet) dry 6 foot width dry centered
- Arterial: 1 lane (12 feet) dry each direction 24 foot width dry centered

3. OFF-SITE MINOR AND MAJOR STORM EVENTS

A. Diverted around or piped/channeled through development.
B. The construction in special flood hazard areas and areas of interim delineation shall be completed in accordance with Article 416 of Washoe County Development Code.
C. Flows must return to the natural drainage path after exiting development.

4. Streets which intersect State Highways where local, collector, or arterial streets intersect State Highways, the criteria of the Nevada Department of Transportation shall be followed for design of storm drains and inlets at said intersections.

304.5 CULVERTS AND BRIDGES

Culverts and bridges are required where natural or manmade channels are crossed by roads and streets. The amount of channel flow which crosses over the road should be minimized to protect the road embankment and pavement from erosion damage as well as to protect vehicles and pedestrians from dangerous flow depths and velocities.
THE POLICY OF WASHOE COUNTY SHALL BE TO REQUIRE CULVERT/BRIDGE CROSSINGS OF STREETS WITHIN THE FOLLOWING LIMITATIONS:

<table>
<thead>
<tr>
<th>Right-Of-Way Width</th>
<th>Minimum Capacity (Recurrence Interval)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Major and Minor Arterial, Highways/Greater than or equal to 80 feet</td>
<td>100-year (No Overflow)</td>
</tr>
<tr>
<td>Collector and Local Streets/Less than 80 feet</td>
<td>100-Year (See Note)</td>
</tr>
</tbody>
</table>

Note: A dipped overflow section may be allowed by the County Engineer if the maximum velocity does not exceed 6 feet per second and the maximum depth does not exceed 0.5 feet at the street crown. As a minimum, where the existing channel is incapable of passing the 100-year flow, the culvert or bridge shall pass the existing channel capacity.

304.6 FLOODPROOFING

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

THE POLICY OF WASHOE COUNTY SHALL BE TO ALLOW THE FLOOD-PROOFING OF EXISTING COMMERCIAL STRUCTURES LOCATED WITHIN A DESIGNATED FLOODPLAIN AREA WHICH ARE NOT BUILT IN CONFORMANCE TO THE ADOPTED FLOODPLAIN REGULATIONS. ALL SUCH FLOOD PROOFING SHALL COMPLY WITH PROVISIONS OF ARTICLE 416 OF WASHOE COUNTY DEVELOPMENT CODE.

304.7 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 and require additional analysis and design to provide safe and effective facilities to accommodate these hazards.

THE POLICY OF WASHOE COUNTY SHALL BE TO REQUIRE DEVELOPMENT ON ACTIVE ALLUVIAL FANS TO COMPLY WITH ARTICLE 416 OF WASHOE COUNTY DEVELOPMENT CODE.

305 IRRIGATION FACILITIES

305.1 Drainage Interaction

There are a number of irrigation ditches and reservoirs in the Washoe County area. These ditches and reservoirs have historically intercepted the storm runoff from the rural and agricultural type basins, generally without major problems. With urbanization of the basins, however, the storm runoff has increased in rate, quantity and frequency, as well as changing in water quality. In urbanized areas, the
irrigation facilities can no longer be utilized indiscriminately to convey storm runoff, and therefore policies must be established to achieve compatibility between urbanization and the irrigation facilities.

In evaluating the interaction of irrigation ditches with a major drainageway for the purpose of basin delineation, the ditch should not be utilized as a basin boundary due to the limited flow capacity of the ditch. The ditches will generally be flowing full or near full during major storms and, therefore, the tributary basin runoff would flow across the ditch.

**THE POLICY OF WASHOE COUNTY SHALL BE TO REQUIRE DRAINAGE ANALYSIS TO ASSUME THAT AN IRRIGATION DITCH DOES NOT INTERCEPT THE STORM RUNOFF FROM THE UPPER BASIN AND THAT THE UPPER BASIN IS TRIBUTARY TO THE BASIN AREA DOWNSTREAM OF THE DITCH.**

### 305.2 Irrigation Ditches

Irrigation ditches are designed with flat slopes and limited carrying capacity, which decreases in the downstream direction. As a general rule, irrigation ditches cannot be used as an outfall point for the storm drainage system because of these physical limitations. In addition, certain ditches are abandoned after urbanization and therefore could not be successfully utilized for storm drainage.

In certain instances, however, irrigation ditches have been successfully utilized as outfall points for the initial drainage system, but only after a thorough hydrological and hydraulic analysis is completed and the irrigation ditches owners consent is secured. Since the owner's liability from ditch failure increases with the acceptance of storm runoff, the responsibility must be clearly defined before a combined system is approved.

**THE POLICY OF WASHOE COUNTY IS TO PROHIBIT THE USE OF IRRIGATION DITCHES AS STORM DRAINAGE FACILITIES UNLESS SPECIFICALLY APPROVED BY COUNTY ENGINEER AND DITCH COMPANY FOLLOWING PRESENTATION OF A DETAILED HYDROLOGIC AND HYDRAULIC ANALYSES.**

### 306 Preservation of Natural Drainageways

Natural drainageways are considered an important element that contributes to the image and livability in an urban environment. Their value extends beyond that of conveying flood water, to their use as trail and open space corridors, and to maintain natural vegetation and wildlife habitat to the greatest degree possible.

**THE POLICY OF WASHOE COUNTY IS TO INSURE THAT DEVELOPMENT OF PROPERTY SHALL NOT ADVERSELY AFFECT ANY NATURAL DRAINAGE FACILITY OR NATURAL WATER COURSE, AND SHALL BE SUBJECT TO THE FOLLOWING PROVISIONS:***

1. Natural drainageways shall remain in as near a natural state as is practicable, with any modification proposed, including any erosion mitigating measures, addressed in the Drainage Report and drainage plans; and

2. When the flows, velocity or side slope as determined by the Drainage Report indicates a hazard, the applicant shall provide fencing in accordance with County standards.
# SECTION 400
## DRAINAGE LAW

## TABLE OF CONTENTS

<table>
<thead>
<tr>
<th>Section</th>
<th>Title</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>401</td>
<td>INTRODUCTION</td>
<td>401</td>
</tr>
<tr>
<td>402</td>
<td>HISTORICAL EVOLUTION OF SURFACE WATER DRAINAGE LAW</td>
<td>401</td>
</tr>
<tr>
<td>402.1</td>
<td>- The Common Enemy Doctrine</td>
<td>401</td>
</tr>
<tr>
<td>402.2</td>
<td>- Civil Law Rule</td>
<td>402</td>
</tr>
<tr>
<td>402.3</td>
<td>- Reasonable Use Rule</td>
<td>402</td>
</tr>
<tr>
<td>403</td>
<td>NEVADA DRAINAGE LAW</td>
<td>403</td>
</tr>
<tr>
<td>403.1</td>
<td>- Civil Law Rule</td>
<td>403</td>
</tr>
<tr>
<td>403.2</td>
<td>- Reasonable Use Rule</td>
<td>404</td>
</tr>
<tr>
<td>403.3</td>
<td>- Surface Waters - Private Development</td>
<td>406</td>
</tr>
<tr>
<td>403.3.1</td>
<td>- Negligence</td>
<td>406</td>
</tr>
<tr>
<td>403.3.2</td>
<td>- Breach of Express/Implied Warranty</td>
<td>407</td>
</tr>
<tr>
<td>403.3.3</td>
<td>- Fraud/Misrepresentation</td>
<td>407</td>
</tr>
<tr>
<td>403.3.4</td>
<td>- Trespass</td>
<td>408</td>
</tr>
<tr>
<td>403.3.5</td>
<td>- Nuisance</td>
<td>409</td>
</tr>
<tr>
<td>403.3.6</td>
<td>- Strict Liability</td>
<td>409</td>
</tr>
<tr>
<td>403.3.7</td>
<td>- Punitive Damages</td>
<td>410</td>
</tr>
<tr>
<td>404</td>
<td>SURFACE WATERS - GOVERNMENTAL ENTITY LIABILITY</td>
<td>411</td>
</tr>
<tr>
<td>404.1</td>
<td>- Sovereign Immunity</td>
<td>411</td>
</tr>
<tr>
<td>404.2</td>
<td>- NRS 41.032 - Discretionary Immunity</td>
<td>412</td>
</tr>
<tr>
<td>404.3</td>
<td>- NRS 41.033 - Failure to Inspect</td>
<td>413</td>
</tr>
<tr>
<td>404.4</td>
<td>- Limitation of Tort Damage Awards</td>
<td>414</td>
</tr>
<tr>
<td>404.5</td>
<td>- Inverse Condemnation - Eminent Domain</td>
<td>414</td>
</tr>
</tbody>
</table>
SECTION 400
DRAINAGE LAW

401 INTRODUCTION

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

An important lesson which has been learned in Southern Nevada is that water does not respect arbitrary jurisdictional boundaries. Water does not respect the various rights and liabilities of adjacent land owners as it flows through depressions, 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 chapter discusses the historical evolution of water drainage law in Nevada. Unlike other states such as California and Colorado, there is not a great body of Nevada case law which discusses every identifiable issue with respect to water drainage law. There are many gray areas in Nevada law, but the engineer can avoid major legal obstacles by being more familiar with those cases which have been expressly decided by the Nevada Supreme Court. Relevant statutes will also be discussed.

402 HISTORICAL EVOLUTION OF SURFACE WATER DRAINAGE LAW

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

402.1 THE COMMON ENEMY DOCTRINE

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

Stated in its extreme form, the common enemy doctrine provides that as an incident to 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 (See, 93 ALR 3d 1193).

Surface water was thus regarded as a common enemy which each property owner could fight off or control by any means such as retention, diversion, repulsion or altered conveyance. Thus, there was no cause of action even if some injury occurred to the adjoining parcel.
All jurisdictions originally following this harsh rule have either modified the rule or adopted the civil law rule or reasonable use. Rule 5 Water and Water Rights, Sections 450.6.451 (RE Clark Ed. 1972).

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

402.2 CIVIL LAW RULE

Courts later recognized the rule of water drainage law which is basically diametrically opposed to the common enemy doctrine. The civil law rule recognizes a natural servitude for natural drainage between adjoining lands, so that the lower owner must accept the surface water which naturally drains onto 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".

The Nevada Supreme Court specifically recognized the civil law rule as early as 1885 in the case of Boyton v. Longley, 19 Nev. 69, 6 Pac. 437 (1885). This case will be discussed in detail in the analysis of Nevada drainage law.

402.3 REASONABLE USE RULE

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

Under the reasonable use rule, a property owner can legally make reasonable use of 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? See, Restatement Torts, 822-831, 833 (1939).

The Nevada Supreme Court has recognized consideration of at least five factors (please see discussion on factors on page 406) in determining whether a property owner's conduct was reasonable. As one can see from this analysis, it becomes very difficult to predict how a jury would rule in relation to any
particular set of facts because the standard for determination is reasonableness, and each jury will have its own standard for determining reasonable conduct.

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

403 NEVADA DRAINAGE LAW

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

403.1 CIVIL LAW RULE

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

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

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

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

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

19 Nev. at 69, 72-73.

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

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

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

403.2 REASONABLE USE RULE

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

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

The Nevada Supreme Court found that during the 1950's and early 1960's, the plaintiffs had acquired their parcels and developed them for residential use. Prior to major development in the area, the land immediately west of the two plaintiffs' parcels was sufficiently porous to absorb and dissipate most rain waters. Heavy rains, however, would collect in the low surrounding areas and would follow the natural terrain entering the plaintiffs' properties at the approximate border between the two properties. These waters would flow, if at all, at a slow velocity and would be naturally dissipated and absorbed. Flooding was rare. The "ephemeral stream" on the plaintiffs' property paralleled a wash which ran to the south of the plaintiffs' parcel.
The Court found that starting in 1967 the development of the lands west of the plaintiffs' parcel resulted in the alteration, diversion, channeling and acceleration of rain, nuisance, and flood waters on to respondents' properties. The court found the County had actively participated in the development of these lands, both by its own planning, design, engineering, and construction activities and by its adoption of the similar activities of various private developers as part of the County's Master Plan for the drainage and flood control of the area.

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

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

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

1) the injurious flow of water is reasonably necessary for drainage;
2) reasonable care is taken to avoid unnecessary injury;
3) the benefit to the drained land outweighs the gravity of harm inflicted upon the flooded land;
4) the drainage is accompanied, where practicable, by the reasonable improvement and aiding of normal and natural systems of drainage in accordance with their reasonable carrying capacity; and
5) Where no natural systems of drainage are available, the drainage is accomplished by the use of a reasonable, artificial system of drainage.

The reasonable use rule was adopted by the court because it felt that the economic costs incident to the expulsion of surface waters in the transformation of rural and semi-rural areas into urban and suburban communities should not be borne solely by adjoining landowners. Rather, landowners, developers, and local officials, should take into account the costs of development of the community prior to the implementation of their plans. The court found that absent such prior planning, the reasonable use rule allows for a more equitable allocation of the incidental economic costs than did the civil law rule.
The County also argued it had statutory immunity for damages which were caused by "urbanization". The Nevada Supreme Court rejected the concept of limited sovereign immunity, and held as follows:

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

96 Nev. a 505.

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

Although the Powers case changed Nevada law, it leaves many questions unanswered such as: (1) Is the governmental entity liable for mere approval of a private development; (2) What constitutes "substantial involvement" in the development of private land? (3) Is a governmental entity liable if it fails to detect design or construction deficiencies in a private drainage control design? and (4) Is a governmental entity liable for privately designed flood control improvements which are later dedicated to the entity?

The Powers case is the controlling legal precedent in this state. Engineers should be aware of the balancing test set forth in the decision, as well as the underlying factual situation. The balancing test should be considered when an engineer is designing or approving alternate methods of handling water drainage. As previously mentioned, one engineer's "reasonable" drainage approach could be a juror's "unreasonable" diversion.

403.3 SURFACE WATERS - PRIVATE DEVELOPMENT

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

403.3.1 NEGLIGENCE

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

Placing the negligence definition into an engineering context, the reasonable and prudent man standard becomes a reasonable professional and prudent professional standard. Thus, a professional engineer who fails to act within the standard of care of his engineering profession may be held liable for negligence. The applicable standard of care is established in court by expert testimony.
The concept of negligence is composed of the traditional elements of duty, breach of the duty, the breach resulting in the proximate cause of damage, or injury. Prosser, Torts 143 (4th Ed. 1971). Nevada courts and courts across the nation have broadly interpreted the duty element as a duty being owed to all persons who may foreseeably be affected by the work being performed.

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

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

Unfortunately, an engineer cannot always be guaranteed that by merely following the computer models and design procedures contained in this manual he will be insulated from negligence liability. It is possible that in a particular area of design the engineer's standard of care could require a higher standard of engineering practice. However, following the requirements of this manual will help to substantially establish that an engineer has met the accepted standard of care.

**403.3.2 BREACH OF EXPRESS/IMPLIED WARRANTY**

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

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

**403.3.3 FRAUD/MISREPRESENTATION**

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

Fraud in the general sense is deemed to be any conduct which is calculated to deceive another person or entity and results in damage.
The essential elements required to sustain a fraud action are the representations made as a statement of fact (non genuine) which was untrue and known to be untrue by the party making it, or else recklessly made; that the statement was made with intent to deceive and for the purpose of inducing the other party to act upon it, and the person did in fact rely on it and was induced to act to his detriment. Am.Jur. 2d, Fraud & Deceit, Sections 2, 12.

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

403.3.4 TRESPASS

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

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

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

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


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

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

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

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

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

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

403.6 STRICT LIABILITY

Nevada has recognized that an end user of a "product" has established a cause of action in strict liability against a manufacturer or distributor when "his injury is caused by a defect in the product, and [the user proves] that such defect existed when the product left the hands of the defendant". Shoshone Coca-Cola Bottling v. Dolinski, 82 Nev. 439, 443, 420 P.2d 855, 858 (1966).

California has applied the strict liability theory to the sale of homes and defective lots. The Nevada Supreme Court noted in Elvey v. Steven, 104 Nev. Adv. Op. 62, N.2 (1988) that courts are divided about whether a home is a product under strict liability theory. In that case the Nevada Supreme Court was presented with this issue but was able to decide the case without ruling on the applicability of the strict liability theory. As the law currently stands in Nevada,
the strict liability theory does not apply to homes. However, this theory could be extended to a situation where a "product" is sold by someone in the regular course of its business.

403.3.7

PUNITIVE DAMAGES

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

NRS 42.010 provides as follows:

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

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

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

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

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

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

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

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

404 SURFACE WATERS - GOVERNMENTAL ENTITY LIABILITY

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

404.1 SOVEREIGN IMMUNITY

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

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

In close cases where the issue of whether the allegations of conduct fall within the parameters of a waiver of sovereign immunity, courts must favor a waiver of immunity; only when it is concluded that a discretionary act alone is involved will the court find immunity. Hagblom v. State Director of
NRS 41.031 initially provided for a special claims procedure when a person wanted to sue the state. However, the Nevada Supreme Court eliminated this requirement in 1973.

404.2 NRS 41.032 - DISCRETIONARY IMMUNITY

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

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

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

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

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

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

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

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

In Esmeralda County v. Grogan, 94 Nev. 923 (1978) the Nevada Supreme Court held that the decision to grant, revoke, or withhold a liquor license is a discretionary act.

404.3 NRS 41.033 - FAILURE TO INSPECT

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

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

and

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

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

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

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

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

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

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

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

404.5 INVERSE CONDEMNATION - EMINENT DOMAIN

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

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

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

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

Chapters 340 and 342 of the Nevada Revised Statutes also contain additional information regarding eminent domain procedures and acquisition of real property. Of particular interest is NRS 342.280 which provides that no public body shall intentionally make it necessary for an owner to institute legal proceedings to prove the fact of the taking of his real property.
The Courts have generally upheld the concept that drainage improvements are public purposes. A public drainage ditch has been held to be for a public purpose under eminent domain, and therefore required compensation for private property taken or damaged in the construction thereof. Eminent Domain, 26 Am.Jur.2d Section 44. The courts quite generally have come to consider drainage district acts with favor as being for public purpose, whether exercised for the benefit of public health or for the reclamation or utilization of lands for agricultural purposes.

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

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

Thus, private property which is subject to intermittent but inevitable flooding can be "taken" as a result of governmental flood control projects. However, each of the cases is highly dependent upon its factual situation. Inverse condemnation liability extends to "just compensation" for the highest and best use of the property. The previously mentioned $50,000 damage limitation applies only to tort actions and does not apply to inverse condemnation actions. Additionally, the sovereign immunity defenses such as discretionary immunity, and failure to inspect immunity, are not available to the governmental entity because the right to just compensation for private property taken for a public use cannot be abridged or impaired by statute. Alper v. Clark County, 93 Nev. 569, 571 P.2d 810 (1977) cert. denied, 436 U.S. 905, 98 S.Ct. 2235, 56 L.Ed. 2d 402 (1978).
SECTION 500
DRAINAGE PLANNING AND SUBMITTAL

TABLE OF CONTENTS

501 PURPOSE

502 REQUISITE DRAINAGE REPORT SUBMITTALS
   502.1 - General
   502.2 - Submittals
   502.3 - Exemptions
   502.4 - Drainage Master Plan
   502.5 - Engineer’s Statement

503 CONCEPTUAL DRAINAGE REPORT
   503.1 - Report Contents
   503.2 - Drainage Plan

504 TECHNICAL DRAINAGE REPORT
   504.1 - Report Contents
   504.2 - Drainage Plan
   504.3 - Calculations Exemption
   504.4 - Improvement Plans

505 TECHNICAL DRAINAGE REPORT ADDENDUM

506 IMPROVEMENT PLANS

507 REQUIREMENTS FOR A FLOODPLAIN STUDY
   507.1 - Outline for a Floodplain Study
   507.2 - FEMA Designated Floodplains

LIST OF TABLES

501 REQUIRED DRAINAGE REPORT SUBMITTALS
SECTION 500
DRAINAGE PLANNING AND SUBMITTAL

501 PURPOSE

The purpose of a Drainage Report is to present conceptual or technical information to demonstrate that proposed drainage designs will adequately detain and convey design-storm runoff in accordance with the policies and standards set forth in this Manual. The goal of Drainage Report Submittal Standards is to obtain consistency in information, analysis content, and presentation to minimize time and effort needed to prepare and review the proposed drainage design.

Drainage Reports will be reviewed for compliance and consistency with the drainage policies and design standards established in this Manual. These policies and standards establish the minimum requirements for drainage analysis and design. The design engineer is ultimately responsible for the drainage facilities design and determining if drainage facilities which exceed the standards established herein are necessary to fully protect the proposed project and citizens from drainage and flood hazards.

By reviewing and accepting drainage designs for given developments, Washoe County will not assume liability for improper drainage design nor guarantee that drainage design reviews will absolve the developer or designer of future liability for improper design.

502 REQUIRED DRAINAGE REPORT SUBMITTALS

502.1 GENERAL

Washoe County utilizes two types of Drainage Reports for the drainage submittal and review process: the Conceptual Drainage Report and the Technical Drainage Report.

The Conceptual Drainage Report is a condensed report which conceptually addresses drainage problems and proposed solutions. This conceptual report provides Washoe County and other reviewing agencies information needed to enable a general review of drainage conditions at a site, and an evaluation of the feasibility and adequacy of the storm drainage systems proposed in concept. The Conceptual Drainage Report is also used to identify drainage easements and, at a conceptual level, evaluate their adequacy for proposed uses. For large projects that will be constructed in phases, the Conceptual Drainage Report can act as a local master plan and provide a basis for the design of future phased development. With respect to larger projects for which final designs will be immediately pursued, the Conceptual Drainage Report can provide a means for the submittal of conceptual designs for approval prior to the development of the more detailed Technical Drainage Report typically required for such projects. In certain cases, the Conceptual Drainage Report may be used to demonstrate that a proposed project will have little or no impact on downstream properties and drainage structures, and that further drainage analysis is not warranted.

The Technical Drainage Report provides detailed hydrologic and hydraulic analyses used for the design of proposed drainage facilities per the guidelines and standards set forth in this Manual. This report also presents drawings depicting drainage easement boundaries, layout of drainage facilities, grading
plans and special details required for proposed drainage structures. The information presented in the Technical Drainage Report should be of sufficient detail and comprehensiveness to enable the reviewing agencies to determine that all storm drainage designs proposed in the report will perform the intended purposes adequately and in conformance with the applicable design criteria.

In addition to the Technical Drainage Report, an Addendum to the report will be required if, during final design or development of the construction drawings, design changes require modification of the hydrologic or hydraulic analyses presented in the Technical Drainage Report. The Addendum should present a discussion of the cause for changes, a description of the changes, and complete computations and other backup data used for developing the design modifications. Revised designs at a level of detail compatible with the Technical Drainage Report previously submitted and approved should also be included in the Addendum.

502.2 SUBMITTALS

In order to standardize the drainage submittal and review process, the Drainage Report submittal requirements for all land development processes, land disturbance projects and improvement projects are presented in Table 501.

The submittal requirements are tailored to provide the minimum amount of information necessary for each development process and size of development, or type and size of land disturbance or improvement project, in order to minimize the cost of drainage report preparation as well as to minimize the time necessary for local entity review. The approval of a drainage report and accompanying proposed designs does not, however, relieve the design engineer of the responsibility to provide a correct and safe drainage design nor the developer to properly construct the designed drainage facilities.

All submitted reports should be clearly and cleanly reproduced. Photostatic copies of charts, tables, nomographs, calculations, or any other referenced material should be legible. Washed out, blurred, or unreadable portions of the report are unacceptable and will warrant resubmittal of the report. Drainage Plans submitted with reports should be clearly legible with appropriate symbolage used to identify all relevant drainage structures.

A checklist of required items for each drainage report type is presented on the "Drainage Submittal Checklist" (Standard Form 1/Section 1500). This checklist will be used by Washoe County to initially determine if the minimum submittal requirements have been met. If the submittal does not meet the minimum requirements, the submittal will be returned to the submitting party with the deficiencies noted. These deficiencies must be corrected and resubmitted before the submittal will be accepted for review.

502.3 EXEMPTIONS

Exemptions to the Drainage Report Submittal Requirements may be granted by Washoe County for just cause. Those processes/projects which can clearly demonstrate, without detailed analysis, that no adverse impacts will result to the on-site and downstream drainage systems may be exempted from submitting a Conceptual or Technical Drainage Report. Certain items of the submittal requirements may be waived if it is clearly demonstrated to and agreed by Washoe County, prior to submittal of the report for review, that the subject information is not needed to fulfill the intent of the report. Requests to Washoe County for such exemption or waiver shall be made in writing. For approved requests, Washoe County shall provide to the requesting party a written confirmation of the exemption or waiver.

December 2, 1996

Drainage Planning and Submittal
granted. If such exemption or waiver is so granted, the project developer will be required to submit a statement by a Nevada Registered Civil Engineer stating that the proposed project fully meets the policy, analysis and design requirements of this Manual.

502.4 DRAINAGE MASTER PLANS

Submittal requirements for drainage master plans may require Conceptual Drainage Reports, Technical Drainage Reports, or both. It is recommended that, during the conceptual development of a drainage master plan, a meeting should be held between the developer, Washoe County, and, if applicable, any regional agencies to identify major issues that may affect proposed master-planned drainage facilities.

Discussion with Washoe County should be held to determine what level of analysis will be appropriate for the proposed drainage master plan. Particular attention will need to be given to project phasing, compatibility with adjacent existing or proposed local and regional drainage systems, and compatibility with existing master plan analyses. The master plan drainage report submittal should provide a comprehensive discussion of all relevant issues that affect the design and implementation of the subject local and regional drainage facilities.

502.5 ENGINEER'S STATEMENT

All Conceptual and Technical Drainage Reports and Addenda shall include the following statement bearing the seal and signature of the Nevada Registered Civil Engineer that has prepared the report:

"I hereby affirm that this report (plan) for the (type or phase design) of (Name of Development) was prepared by me (or under my direct supervision) for the owners thereof in accordance with the appropriate level of engineering care and after taking into account the provisions of the Washoe County Hydrologic Criteria and Drainage Design Manual and approved variances and exceptions thereto.

Nevada Registered Civil Engineer Nº.  
(Affix Seal)

503 CONCEPTUAL DRAINAGE REPORT

A Conceptual Drainage Report is a condensed report which conceptually describes existing and proposed drainage conditions and facilities. The purpose of the Conceptual Drainage Report is described above in Section 502. The Conceptual Drainage Report shall contain a brief narrative letter, a calculation appendix (if applicable), and a drainage plan in accordance with the following outline. The report should include the information described in the outline as appropriate.

503.1 REPORT CONTENTS

I. Title Page

   A. Project Name and Type of Study
   B. Preparer's Name, Firm, and Date
WASHOE COUNTY
HYDROLOGIC CRITERIA AND DRAINAGE DESIGN MANUAL

II. Introduction

A. Location of project by ¼ Section, Section, Township and Range
B. Description of Project
C. Existing Site Conditions
D. Description of previous studies and relevant Master Plans including full references (if applicable)

III. Existing and Proposed Hydrology

A. Discuss existing and proposed drainage basin boundaries
B. Present existing and proposed minor and major storm flow calculations
C. Discuss existing drainage problems (if applicable)

IV. Proposed Drainage Facilities

A. Discuss routing of flow in and/or around site and location of drainage facilities
B. Discuss detention requirements per Section 303.7 of this Manual
C. Discuss proposed floodplain modifications (if applicable)
D. Discuss outfall system and anticipated phasing for relevant future facilities proposed by others (if applicable)

V. Conclusions

A. Compliance with all MANUAL policies and requirements
B. Requested MANUAL exemptions
C. Ability to provide emergency all weather access
D. Compliance with floodplain/flood hazard regulations
E. Discuss effect of development on off-site flow rates and patterns and impact to adjacent properties
F. Engineer's Statement, Seal & Signature

VI. Exhibits and Figures

A. General Location Map (8½" X 11" is suggested)
B. Off-site Basin Map (with topographic information at suitable scale)
C. Drainage Plan (see Section 503.2)

VII. Calculations Appendix

A. Runoff calculations
B. Street and drainage facility capacity calculations
C. Detention calculations (if applicable)
D. Copies of all tables, charts, etc. used for calculations with source noted

December 2, 1996
Drainage Planning and Submittal 504
503.2 DRAINAGE PLAN

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

1. Locate and label development boundary
2. Locate and label adjacent streets
3. Locate and label known 100-year floodplains
4. Locate and label existing and planned regional and local off-site drainage facilities
5. Locate and label proposed on-site drainage facilities
6. Show flow paths
7. Identify design inflow points and design outflow points and corresponding minor and major storm flow rates
8. Show drainage basin boundaries and basin labels
9. Provide adequate information to identify proposed land cover types within project area
10. Identify drainage basin areas, runoff coefficients and curve numbers as applicable
11. Identify all drainage easements
12. Show existing and proposed grading topographic contours 100' past the property line. (Spot elevations in lieu of topography past the property line may be used only upon prior approval of Washoe County.)

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

504 TECHNICAL DRAINAGE REPORT

The Technical Drainage Report discusses at a detailed level the existing site hydrologic conditions and the proposed drainage design to accommodate or modify these site drainage conditions in the final development plan for the site. The Technical Drainage Report addresses both on-site and off-site drainage analyses and improvements required for compliance with the policies and criteria set forth in this Manual.

The Technical Drainage Report shall be in accordance with the following outline and contain the applicable information listed.

504.1 REPORT CONTENTS

I. Title Page. The title page of the Drainage Report shall contain the following:

A. Project Name.
B. Preparer’s Name, Firm and Date.

II. Introduction. The introduction of the Drainage Report shall include, at a minimum, the following:

A. Identification of street location, Assessor’s parcel number(s), section reference, and adjacent developments.
B. General description of existing site conditions.
C. General description of proposed project.

III. Previous Studies

This section should provide a description of all previous studies relevant to the proposed project. Include all drainage reports, master plans and flood hazard studies, and discuss their relevance to the project. Provide full reference for all information sources.

IV. Hydrologic and Hydraulic Analysis

A. Describe method used for runoff computations.

B. Discuss design storm intensities or depths for 5- and 100-year storms.

C. Describe method used to determine hydrologic parameters, and include reference to the appendix containing hydrologic computations. Include appropriate geotechnical information if used to support the hydraulic design (i.e. natural channels).

D. Describe methods used for hydraulic computations. Discuss typical parameters used, and include reference to the appendix containing hydraulic computations. Include appropriate geotechnical information if used to support the hydraulic design (i.e. natural channels).

E. All computer programs used for hydrologic or hydraulic modelling should be identified, including version/release number. Include printout of input and output files in appendix.

V. Historic Drainage System. The Drainage Report shall provide sufficient information, including text and maps where possible, to describe the historic drainage system. This information shall include:

A. Major basins (100 acres or more), including relationship to major drainage facilities, and major basin drainage characteristics (topography, runoff, cover, use, erosion).

B. Sub-basin and site drainage, including 5-year and 100-year storm flows for each sub-basin affecting the site, existing drainage patterns, channeled or overland flow, points of entrance and discharge, flood hazard areas, and other drainage related features. A map showing off-site basins shall also be included. All items listed in Subsections A and B may be presented on a map or drawing.

C. Discussion of hydrologic and hydraulic analysis results for existing drainage conditions and facilities, including those items on drawings and maps not discussed elsewhere. Discussion shall include flow rates and paths, drainage facilities, irrigation ditches, impact of site runoff on adjacent properties, and any other relevant information.

VI. Proposed Drainage System. As a minimum, the following information regarding the proposed drainage system shall be provided in the Drainage Report. Maps shall be used to complement and clarify the description where appropriate.
WASHOE COUNTY
HYDROLOGIC CRITERIA AND DRAINAGE DESIGN MANUAL

A. Description of major basins and tributary sub-basins. Refer reader to appropriate figures or drawings.

B. Results of hydrologic and hydraulic analyses. Include summary tables if needed to facilitate discussion of results. Refer reader to the appropriate appendix, if applicable.

C. Description of the proposed storm drainage system to manage design storm runoff. Discussion should include management of off-site runoff tributary to the project site, on-site flows, and anticipated phasing of future downstream facilities that comprise the off-site drainage system for the proposed project. Identify any interim facilities that need to be constructed, and the authority under which such facilities will be constructed, until permanent off-site drainage facilities are in place. The intent of the off-site facilities information is to permit the reviewing agencies to determine the impacts of proposed development on off-site facilities and property prior to the construction of permanent off-site local or regional drainage systems.

D. Discussion of potential for and risk of sediment inflow and debris flow into the proposed drainage facilities.

E. Discussion of detention requirements for the 5- and 100-year storms per Section 303.7 of this Manual. For proposed detention facilities the following information should be provided:

1. Volume required and provided for zero increase in peak flows.
2. Release rates and methods.
3. Passage of runoff from storms exceeding the 5-year up to the 100-year storm.
4. Emergency overflow provisions which will not cause a direct impact to neighboring sites.
5. A detailed description of any downstream constraints and mitigation recommendations.
6. Description of detention areas and proposed multiple uses, as applicable. Identify maximum ponding depths for design storms. Refer reader to appropriate calculations appendix.

F. Discussion of compatibility of proposed design with previous studies. Provide justification of deviation from any design constraints recommended or imposed by previous studies or master plans.

G. Discussion of any drainage easements or rights-of-way relevant to the project. For drainage easements dedicated as part of the project, the report should identify the parties subject to the agreements and the form of conveyance of said easements (i.e., final plat, separate deed, etc.).

H. Identification of parties and/or entities responsible for maintenance of the private and/or public drainage systems constructed as part of the proposed project. The report should identify any agreements that define such maintenance responsibilities.

VII. Areas Within Flood Hazard Zone. Where the proposed development is located within a flood hazard area or limited flooding area as defined in Article 416, Flood Hazards, of the Washoe County Development Code, sufficient information shall be provided for the following:
A. Evaluation of the impacts of proposed development on the flood hazard area within the project area and with respect to adjacent properties. If specific analysis was performed for flood hazard area consideration, include description of analysis and pertinent backup data and calculations as applicable.

B. Identification of floodproofing or other protective measures for improvements to be constructed in the flood hazard area.

C. Description of impact of the floodplain on the proposed storm drainage system(s).

D. Discussion of compliance with FEMA requirements for CLOMR/LOMR submittal, if applicable. Include reference to all CLOMR/LOMR's submitted to FEMA for this project.

E. Discuss compliance with Article 416, Flood Hazards, of the Washoe County Development Code.

VIII. Conclusions. This section shall discuss the impacts of the proposed drainage system improvements, including:

A. Compliance with all MANUAL policies and requirements

B. Requested MANUAL exemptions

C. Compliance with State and Federal Regulations

D. Compliance with floodplain/flood hazard regulations

E. Benefits provided by the proposed facilities to off-site systems.

F. Adverse effects to off-site systems and mitigation measures for these effects.

G. Engineer's Statement, Seal and Signature.

H. Ability to provide emergency all weather access

IX. References. Include references for all sources of information used in report.

X. Drainage Report Appendices. The Drainage Report shall include the following information in the Appendices.

A. Site Location Map. Site location may be on a USGS map, at a scale appropriate to show relation of site to major drainage basins and sub-basins; flood hazard areas and 100-year floodplains, if applicable; and off-site flows through project.

B. Computations. Hydrologic and hydraulic computations including:

1. Hydrologic and Hydraulic parameter determination.
2. Off-site and on-site historic runoff.
3. Off-site and on-site proposed-development runoff.
4. Detention volumes and release rates for the design storms.
5. Hydraulic grade line (HGL) for 5- and 100-year storms.
6. Copies of all tables, figures, charts, etc. used for the analyses (with references).
7. Basin schematic showing connectivity between sub-basins, flow conveyance elements, and other pertinent modelling nodes.
8. Capacity analysis of off-site facilities.
9. Geotechnical information (as needed to support hydraulic design assumptions)

C. Drawings and Figures. Include Drainage Plan per Section 504.2 and any other figures developed for the report.

504.2 DRAINAGE PLAN

A detailed drainage plan(s) for the subject site shall be submitted with the Technical Drainage Report. The plan(s) shall be on a 24" x 36" drawing at an appropriate legible and microfilmable scale (a scale of 1" = 50' to 1" = 500' is recommended). The following information shall be shown on this drawing, except that the off-site drainage basin boundaries may be shown at an appropriate legible scale on an exhibit.

1. Property lines and streets (roads) including right-of-way widths within 100 feet of the property.
2. Existing contours and proposed elevations sufficient to analyze drainage patterns extending 100 feet outside of property lines.
3. Existing drainage facilities and structures, including ditches, storm sewers, channels, street flow directions, and culverts. All pertinent information such as material, size, shape, slope, and location shall also be included.
4. Limits of existing floodplains based on Flood Insurance Rate Maps, if available. Also, existing floodplains based on best available data (existing floodplain studies) should be shown where available.
5. Proposed on-site drainage basin boundaries with appropriate basin labels. Include off-site basin boundary intersections with on-site basin boundaries if not shown elsewhere. Label all design points.
6. Proposed future on-site and off-site flow directions and paths.
7. Proposed street and ditch flow paths and slopes.
8. Proposed storm sewer locations, type, size, and slope. Include inlet types, sizes and locations, and manhole locations.
9. Proposed channel alignment with typical cross section. Include major storm flow limits.
10. Proposed culvert locations, type, size, and slope.
11. Proposed On-site Drainage System outlet(s) to the Off-site Drainage System.
12. Detention/retention facilities and related structures. Indicate extent and depth of ponding for design storms.

13. Miscellaneous proposed drainage facilities (e.g. hydraulic structures, erosion protection, etc.)

14. Details for special structures (e.g. detention pond outlets, overflow spillways, erosion protection, etc.).

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

16. Drainage and maintenance easement widths and boundaries.

17. Labels of all inlets and manholes to correspond to tabular number system.

18. Legend for all symbols used on drawing.


20. Scale, North Arrow, Title Block, Professional Engineers Signature, Seal, Date.

504.3 CALCULATIONS EXEMPTION

The report requirements for a Technical Drainage Report may be reduced at the request of the applicant or local entity, pending approval of Washoe County, if there is uncertainty over the final characteristics of the proposed drainage facilities. In such cases, the Technical Drainage Report shall identify all areas where the uncertainty exists and explain why final characteristics can not be presently determined. Hydrologic and hydraulic calculations based upon assumptions may then be provided with less detail. Washoe County may tentatively approve such interim reports based on design assumptions. The relevant analyses and designs shall be completed in the appropriate detail as part of the Technical Drainage Report Addendum which will be required to be submitted at the time of or prior to submittal of the final Improvement Plans. The Technical Drainage Report and Addendum will need to have final approval from Washoe County before the Improvement Plans can be approved.

504.4 IMPROVEMENT PLANS

Improvement plans are desired but not required to be submitted with the technical drainage report. However, profiles of storm sewers with HGL's and EGL's may be required for adequate review if required by Washoe County.

505 TECHNICAL DRAINAGE REPORT ADDENDUM

The purpose of the Technical Drainage Report Addendum is to provide all detailed hydrologic and hydraulic calculations which were not mentioned in the Technical Drainage Report requirements, or to present analysis and design changes made subsequent to the submittal and approval of a Technical Drainage Report. This addendum shall be prepared in accordance with the following outline and contain the applicable information listed:

1. Title Page
   A. Project Name, Type of Study, Study Date
B. Preparer's Name, Firm and Date
C. Engineer's Statement, Seal and Signature

II. Introduction. Discussion of the reason for submitting the Addendum and overview of the Addendum contents.

III. Hydrologic and Hydraulic Analysis. Provide revised or additional analysis per the Technical Drainage Report requirements.

IV. Conclusion. Present a summary of the information or modifications presented in this Addendum.

V. References. Provide a list of all references used for the Addendum.

VI. Appendices.
   A. Include all relevant calculations.
   B. Include a revised Drainage Plan per the requirements of the Technical Drainage Report. The revised plan should contain all needed information to provide a replacement for the plan submitted with the original Technical Drainage Report.

506 IMPROVEMENT PLANS

Where drainage improvements are to be constructed, the final construction plans shall be submitted for approval. Approval of the final construction plans by Washoe County is a condition of issuing construction permits. The plans for the drainage improvements will include:

1. Storm sewers, inlets, outlets and manholes with pertinent elevations, dimensions, type, and horizontal control indicated.
2. Culverts, end sections, and inlet/outlet protection with dimensions, type, elevations, and horizontal control indicated.
3. Channels, ditches, and swales (including side/rear yard swales) with lengths, widths, cross-sections, grades and erosion control (i.e. riprap, concrete, grout) indicated.
4. Checks, channel drops, erosion control facilities.
5. Detention pond grading, trickle channels, outlets, and landscaping.
6. Other drainage related structures and facilities (including underdrains and sump pump lines).
7. EGL's and HGL's for minor (storm sewer) and major (channels) storm runoff.
8. Maintenance access considerations.
9. Overlot grading and erosion and sedimentation control facilities.
10. Drainage easements and rights-of-way with horizontal distance to improvements.
11. Drainage plan attached for information only.

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

507 REQUIREMENTS FOR A FLOODPLAIN STUDY

A Floodplain Study is required for designating a floodplain for drainageways where one has not been established or for modification of a floodplain that is delineated in a floodplain delineation study or on a FEMA Flood Insurance Rate Map (FIRM). A study is required to ensure that property being developed is actually outside of a 100-year floodplain or is to be removed from the floodplain, that other properties that share frontage along the floodplain will not be adversely impacted, that the channel alignment will be stable and will not be subject to erosion which may threaten property, that sufficient conveyance capacity is maintained, and that Washoe County complies with the requirements of FEMA for administering a floodplain management program.

The effort necessary for a floodplain study is dependent upon the amount of information previously generated, the potential for impact on adjacent properties, the magnitude of flow in the channel, the size of the area affected, the need for channel stabilization, and the sediment transport and fluvial morphological aspects of the stream. Floodplain Studies are required for the following activities:

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

2. To support a zoning case for establishment of a floodplain area for drainages that have not had the 100-year floodplain delineated.

3. To support a zoning case where a zone will be modified from an existing floodplain designation.

4. With a Conceptual and/or Technical Drainage Report where floodplain modifications are proposed.

5. For other agencies constructing highways, bridges, or other improvements which affect a FEMA designated floodplain.

507.1 OUTLINE FOR A FLOODPLAIN STUDY

A Floodplain Study must address the following points through actual analysis or through reference to adopted drainage master plans:

1. A description of the floodplain area (i.e. vegetation, condition, slopes constrictions).

2. A description of the contributing drainage basin(s).
3. Identification of applicable floodplain studies and or Flood Insurance Studies with analysis of the applicability of data to the subject area.

4. Hydrologic analysis.

5. Characteristics of the proposed channel including but not limited to slope, roughness, depth, velocity, Froude Number, centerline alignment and stationing, and cross sections. Existing topographic maps may be utilized if they are field verified to determine if changes have occurred. The profile and plan shall be given for existing conditions and for the proposed channel alignment including the cross section locations.


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

8. Discussion of the results and conclusions of the hydraulic analysis. This shall include a narrative summary of the results as well as comprehensive output data.

9. The delineation of the existing and proposed 100-year floodplain and water surface profile. Include cross-section locations.

10. A description of impacts on other property owners along the floodplain.

11. A conceptual design for the channel including embankment protection, drop structures, culverts, bridges, and the hardened trickle channel or low flow channel.

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

507.2 **FEMA DESIGNATED FLOODPLAINS**

In order for Washoe County to participate in the National Flood Insurance Program that is administered by FEMA, the County must conduct a floodplain management program that complies with FEMA requirements. Thus all Floodplain Studies that propose to change a FEMA designated floodplain must address compliance with the FEMA requirements for the project. This includes federal regulations published in 44CFR Part 65 on technical and submittal requirements for a Letter of Map Revision.

Washoe County is required to make submittals for map revisions to FEMA for projects which propose floodway revisions and for developments which desire a change in the floodplain boundaries. For these cases, the applicant shall prepare the FEMA submittal packages and provide the FEMA review fee. The Floodplain Study shall include a proposed schedule for obtaining a Letter of Map Revision for the project. The schedule shall include anticipated dates for:

a. Submittal of the Conditional Letter of Map Revision FEMA submittal package to Washoe County for review by Washoe County.

b. Submittal of the Conditional Letter of Map Revision submittal package to FEMA from Washoe County. A minimum 30-day review time is required for Washoe County review of the submittal.
c. Issuance by FEMA of a Conditional Letter of Map Revision. FEMA approval is required before Washoe County issues permits for construction for areas within a floodplain.

d. Submittal of a draft FEMA submittal package including as-built data for the Letter of Map Revision for review by Washoe County.

e. The Submittal of the Letter of Map Revision submittal package to FEMA by Washoe County. A minimum 30-day review time is required for Washoe County review of the submittal.

f. Issuance by FEMA of a Letter of Map Revision. FEMA approval is required before Washoe County grants final acceptance of public improvements in a subdivision.

The report should be prepared using the drawing size, map scale, and engineer certification requirements that are given for the Technical Drainage Report.

1. A pre-application consultation with Washoe County is suggested of all applicants for these processes. Information gained at said consultations may allow Washoe County to focus the applicant on those areas of significant drainage concerns, thus potentially lessening the applicants time for submittal and subsequent revisions.

2. For all applications which include proposed modifications to areas within a designated 100-year floodplain, a Floodplain Report shall also be submitted or incorporated into the Conceptual and/or Technical Drainage Report.

3. For all applications which include proposed modifications to a natural drainageway for which a 100-year floodplain has not been designated, a Floodplain Report shall also be submitted or incorporated into the Conceptual and/or Technical Drainage Report.

4. If drainage improvements are required to be constructed as part of this project, a Technical Drainage Report and Improvement Plans will be required to be submitted with the Tentative Map process for said drainage improvements.

5. Comments received on the Conceptual Drainage Report during the Tentative review process shall be addressed in the Technical Drainage Report.

6. If not waived per the result of a pre-application consultation, a Conceptual Drainage Report will be required if:
   a) Amendment area is within a 100-year floodplain, or
   b) Amendment area housing/development density will increase over that already included in the plan.

7. No submittals required if lot is part of an approved subdivision, parcel map, or map of division into large parcels. A Technical Drainage Report is required if a grading permit is required.

8. A Conceptual Drainage Report may be allowed for areas with minimal drainage impacts upon prior approval of Washoe County.
## REQUIRED DRAINAGE REPORT SUBMITTALS

<table>
<thead>
<tr>
<th>TYPE OF PROCESS/PROJECT</th>
<th>REQUIRED DRAINAGE REPORT</th>
<th>COMMENTS</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rezoning</td>
<td>May be required in special cases.</td>
<td>1,2,3</td>
</tr>
<tr>
<td>Tentative Map of Division into Large Parcels</td>
<td>Conceptual</td>
<td>1,2,3,4</td>
</tr>
<tr>
<td>Tentative Parcel Map</td>
<td>Technical</td>
<td>1,2,3,8</td>
</tr>
<tr>
<td>Subdivision Map</td>
<td></td>
<td></td>
</tr>
<tr>
<td>• Tentative</td>
<td>Conceptual</td>
<td>1,2,3</td>
</tr>
<tr>
<td>• Final</td>
<td>Technical</td>
<td>5</td>
</tr>
<tr>
<td>Grading Permit</td>
<td>Technical</td>
<td>1,2,3</td>
</tr>
<tr>
<td>• For grading of subdivisions and parcel map areas prior to final approvals and for all other grading activities.</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Public Works Projects</td>
<td>Technical</td>
<td>Submit with Improvement Plans</td>
</tr>
<tr>
<td>Special Use Permit</td>
<td>Conceptual</td>
<td>1,2,3,4</td>
</tr>
<tr>
<td>Plan Amendment</td>
<td>See Comments</td>
<td>1,6</td>
</tr>
<tr>
<td>Site Plan Review</td>
<td></td>
<td></td>
</tr>
<tr>
<td>• Single Family</td>
<td>See Comments</td>
<td>7</td>
</tr>
<tr>
<td>• All Other</td>
<td>Technical</td>
<td>1,2,3</td>
</tr>
</tbody>
</table>
SECTION 600
RAINFALL

TABLE OF CONTENTS

601 INTRODUCTION

602 RAINFALL DISTRIBUTION FOR SCS UNIT HYDROGRAPH METHOD
   602.1 - Rainfall Depth-Duration-Frequency Maps
   602.2 - Rainfall Depths for Durations from 5 minutes to 24 hours
   602.3 - Depth-Area Reduction Factors

603 RAINFALL DISTRIBUTION FOR RATIONAL METHOD
   603.1 - Rainfall Zones For Rational Method
   603.2 - Time-Intensity-Frequency Curves in Zone I
   603.3 - Time-Intensity-Frequency Curves in Zone II
   603.4 - Time-Intensity-Frequency Curves in Zone III

604 EXAMPLES
   604.1 - Introduction
   604.2 - Example: Rainfall Distribution Calculations for HEC-1 Input
   604.3 - Example: Time-Intensity-Frequency Curve Generation For Zone III

LIST OF TABLES

601 REGIONAL GROWTH FACTORS
602 CONVERSION RATIOS FOR DURATIONS OF LESS THAN ONE HOUR
603 TIME-INTENSITY-FREQUENCY DATA FOR RATIONAL METHOD RAINFALL ZONES
LIST OF FIGURES

601A  RAINFALL DEPTH 2-YEAR, 1-HOUR: SOUTHERN WASHOE COUNTY
601B  RAINFALL DEPTH 2-YEAR 1-HOUR: NORTHERN WASHOE COUNTY
602A  RAINFALL DEPTH 2-YEAR, 6-HOUR: SOUTHERN WASHOE COUNTY
602B  RAINFALL DEPTH 2-YEAR, 6-HOUR: NORTHERN WASHOE COUNTY
603A  RAINFALL DEPTH 2-YEAR, 24-HOUR: SOUTHERN WASHOE COUNTY
603B  RAINFALL DEPTH 2-YEAR, 24-HOUR: NORTHERN WASHOE COUNTY
604   DEPTH-AREA REDUCTION CURVE
605   TIME-INTENSITY-FREQUENCY CURVES FOR ZONE I
606   TIME-INTENSITY-FREQUENCY CURVES FOR ZONE II
607   EXAMPLE DRAINAGE BASIN
608   EXAMPLE TIME-INTENSITY-FREQUENCY CURVES
SECTION 600
RAINFALL

601 INTRODUCTION

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

The methodology used to generate the rainfall data will depend on the size of the drainage basin to be studied. The Rational Method for determining runoff is widely accepted as providing a sufficient level of detail for generating runoff from relatively small basins (area ≤ 100 acres). The Rational Method utilizes rainfall data in the form of time-intensity-frequency curves.

Since the assumptions used in the Rational Method become less valid over larger areas, larger basins (area ≥ 100 acres) require a more rigorous analysis to generate runoff data. The HEC-1 computer model developed by the U.S. Army Corps of Engineers is a commonly used model that generates storm runoff (USACE, 1990A). The rainfall data used in this model will be a centrally distributed storm event with depths at time intervals of 5 minutes, 15 minutes, 60 minutes, 2 hours, 3 hours, 6 hours, 12 hours, and 24 hours.

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

602 RAINFALL DISTRIBUTION FOR SCS UNIT HYDROGRAPH METHOD

602.1 RAINFALL DEPTH - DURATION - FREQUENCY MAPS

The National Weather Service’s Southwest Semiarid Precipitation Frequency Study (SSPFS, 1997) has developed three (3) rainfall depth maps for the 1-, 6-, and 24-hour storm durations for the 2-year recurrence frequency. These maps are shown in Figures 601 to 603. For the 2-year return periods, the rainfall depths for durations of 1 hour, 6 hours, and 24 hours can be estimated from the maps.

602.2 RAINFALL DEPTHS FOR DURATIONS FROM 5 MINUTES TO 24 HOURS

For return periods other than the 2-year event, the rainfall depths for durations of 1 hour, 6 hours, and 24 hours can be estimated using Table 601 and rainfall depth estimates for the 2-year event from Section 602.1. Table 601 supplies regional growth factors calculated by the SSPFS to estimate the 1-hour, 6-hour, and 24-hour storm events for a given return period from the 2-year values (Tarleton Julian, 1996). Regional Growth Factors (RGF’s) are used by the SSPFS to generate rainfall values from a base set of rainfall data. In this manner, rainfall maps for storm frequencies other than the 2-year base map are not necessary. The use of these factors are as follows:

\[ D_{x,1} = D_{2,1} \times RGF1 \]  

(601)
where \( D_{X,1} = "X" \)-year, 1-hour rainfall depth (Inches)
\( D_{2,1} = 2\)-year, 1-hour rainfall depth (Inches)

\( RGF1 = \text{Regional Growth Factor for an "X"-year, 1-hour event (Inch/Inch)} \)
\[ D_{X,6} = D_{2,6} \times RGF6 \]  \hspace{1cm} \text{(602)}

where \( D_{X,6} = "X" \)-year, 6-hour rainfall depth (Inches)
\( D_{2,6} = 2\)-year, 6-hour rainfall depth (Inches)
\( RGF6 = \text{Regional Growth Factor for an "X"-year, 6-hour event (Inch/Inch)} \)
\[ D_{X,24} = D_{2,24} \times RGF24 \]  \hspace{1cm} \text{(603)}

where \( D_{X,24} = "X" \)-year, 24-hour rainfall depth (Inches)
\( D_{2,24} = 2\)-year, 24-hour rainfall depth (Inches)
\( RGF24 = \text{Regional Growth Factor for an "X"-year, 24-hour event (Inch/Inch)} \)

Rainfall depths of 5 minutes and 15 minute durations can be estimated using ratios supplied in Table 602 and the previous calculation for the "X"-year, 1-hour rainfall depth (Tarleton Julian, 1996).

\[ D_{X,5} = D_{X,1} \times \text{RATIO5} \]  \hspace{1cm} \text{(604)}

where \( D_{X,5} = "X" \)-year, 5-minute rainfall depth (Inches)
\( D_{X,1} = "X" \)-year, 1-hour rainfall depth (Inches)
\( \text{RATIO5} = \text{Ratio to convert "X"-year, 1-hour rainfall depth to the "X"-year, 5-minute depth (Inch/Inch)} \)

\[ D_{X,15} = D_{X,1} \times \text{RATIO15} \]  \hspace{1cm} \text{(605)}

where \( D_{X,15} = "X" \)-year, 15-minute rainfall depth (Inches)
\( D_{X,1} = "X" \)-year, 1-hour rainfall depth (Inches)
\( \text{RATIO15} = \text{Ratio to convert "X"-year, 1-hour rainfall depth to the "X"-year, 15-minute depth (Inch/Inch)} \)

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

\[ D_{X,2} = 0.299 \times D_{X,6} + 0.701 \times D_{X,1} \]  \hspace{1cm} \text{(606)}

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

\[ D_{X,3} = 0.526 \times D_{X,6} + 0.474 \times D_{X,1} \]  \hspace{1cm} \text{(607)}

where \( D_{X,3} = "X" \)-year, 3-hour rainfall depth (Inches)
\( D_{X,1} = "X" \)-year, 1-hour rainfall depth (Inches)
\( D_{X,6} = "X" \)-year, 6-hour rainfall depth (Inches)
Based on Figure 17A in the NOAA Atlas 2, the 12-hour duration storm event for the desired recurrence frequency is essentially the average of the 6-hour and 24-hour storm events (NOAA, 1973).

\[ D_{X,12} = \frac{D_{X,6} + D_{X,24}}{2} \]  \hspace{1cm} (608)

where

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

The preceding analysis provides the rainfall distribution for a 24-hour storm at time intervals of 5 minutes, 15 minutes, 1 hour, 2 hours, 3 hours, 6 hours, 12 hours, and 24 hours for the desired recurrence frequency. The rainfall distribution is centered around the midpoint of the design storm (time = 12 hours). These rainfall values are input to the HEC-1 program using the PH record. When using the PH record, a value of 0.001 should be input into Field 2 to prevent the program from using an internal point rainfall reduction adjustment (See Section 602.3).

### 602.3 DEPTH-AREA REDUCTION FACTORS

The SSPFS precipitation depths are related to rainfall frequency at an isolated point. Storms, however, cause rainfall to occur over extensive areas simultaneously, with more intense rainfall typically occurring near the center of the storm. Standard precipitation analysis methods require adjusting point precipitation depths downward in order to estimate the average depth of rainfall over the entire storm area. This is normally performed using depth-area reduction curves relating point precipitation reduction factor to drainage basin area and duration.

Figure 604 provides the depth-area reduction curve for the 24-hour storm event (NOAA, 1973). Depth-area values are input to the HEC-1 program using the JR record. The peak flow value at a given point should be determined using the depth-area value for the total watershed area tributary to the subject point of interest.

The ability of the thunderstorm generating mechanisms (i.e., available moisture, strong convective currents, etc.) to sustain a thunderstorm greater than 200 square miles 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. Thus, in order to obtain a consistent method of analysis for drainage basins larger than 200 square miles, the designer shall consult the local entity to determine the appropriate method of analysis for the specific location and basin under consideration.

### 603 RAINFALL DISTRIBUTION FOR RATIONAL METHOD

#### 603.1 RAINFALL ZONES FOR RATIONAL METHOD

A review of the isopluvial maps generated by the SSPFS indicates that, for Rational Method analysis, Washoe County can be divided into three rainfall zones. Within each zone, the precipitation values were similar for the various return periods and duration storms. These zones are shown on Figure 601 thru 603.
If more than 50 percent of the basin lies in a given zone, the rainfall data for that zone shall be used. Basin area refers to the actual basin or sub-basin for which storm runoff information is being calculated and not necessarily the entire watershed area.

603.2 TIME-INTENSITY-FREQUENCY CURVES IN ZONE I

Within Zone I, the rainfall time-intensity-frequency curves used in the rational method are assumed to be identical throughout the zone. The curves are shown in Figure 605, and tabulated in Table 603.

603.3 TIME-INTENSITY-FREQUENCY CURVES IN ZONE II

Within Zone II, the rainfall time-intensity-frequency curves used in the rational method are assumed to be identical throughout the zone. The curves are shown in Figure 606, and tabulated on Table 603.

603.4 TIME-INTENSITY-FREQUENCY CURVES IN ZONE III

Due to the variability of the rainfall for a given recurrence frequency and duration within Zone III, the time-intensity-frequency curves must be generated for the desired location. First, the 1-hour rainfall point depth for desired return period is determined as described in Section 602.2. Second, Table 602 is then used to estimate the depth of the 5 minute, 10 minute, 15 minute, 30 minute, and 60 minute storm durations. The third step is to convert these point depth values to intensities by dividing the depth of precipitation by the duration of the storm. Finally, a time-intensity curve for a particular recurrence interval is developed by plotting the intensity values versus their corresponding storm duration values. An example showing the development of a time-intensity-frequency curve is given in Section 604.

604 EXAMPLES

604.1 INTRODUCTION

The following examples are the first of a series of example applications pertaining to the use of this manual. The series is set up to lead the reader through the MANUAL's design/evaluation procedures by analyzing a hypothetical drainage basin within Washoe County.

The example basin is introduced in Figure 607. Basin features and modifications are presented in each Section to emphasize the procedure to be modeled.

This basin is a hypothetical basin and is not intended to represent an actual location within Washoe County.

604.2 EXAMPLE: RAINFALL DISTRIBUTION CALCULATION FOR HEC-1 INPUT

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

Solution:

Step 1: Determine the average 2-year, 1-hour rainfall depth, the average 2-year, 6-hour rainfall depth, and the average 2-year, 24-hour rainfall depth in Basin A from Figures 601, 602, and 603, respectively. The location of the basin is not shown on these maps to prevent this hypothetical example problem from being construed to represent an actual location. (Basins that have highly variable rainfall depths for a given frequency and duration may need to be subdivided
into areas of common rainfall depth. A weighted average of the rainfall depth can then be calculated using the areas and rainfall depths of the sub-basins).

\[ D_{2,1} = 0.4 \text{ inches (Assumed for example purposes only)} \]
\[ D_{2,6} = 1.0 \text{ inches (Assumed for example purposes only)} \]
\[ D_{2,24} = 1.6 \text{ inches (Assumed for example purposes only)} \]

Step 2: Calculate the average 100-year, 1-hour rainfall event, 100-year, 6-hour rainfall event, and 100-year, 24-hour rainfall depth. The regional growth factors (RGF) found in Table 601 are multiplied by the 2-year rainfall depths to obtain the 100-year depths.

\[ \text{RGF (100,1)} = 3.62 \]
\[ \text{RGF (100,6)} = 2.26 \]
\[ \text{RGF (100,24)} = 2.22 \]

\[ D_{100,1} = D_{2,1} \times \text{RGF(100,1)} = 0.4 \times 3.62 = 1.45 \text{ inches} \]
\[ D_{100,6} = D_{2,6} \times \text{RGF(100,6)} = 1.0 \times 2.26 = 2.26 \text{ inches} \]
\[ D_{100,24} = D_{2,24} \times \text{RGF(100,24)} = 1.6 \times 2.22 = 3.55 \text{ inches} \]

Step 3: Determine the 100-year, 5 minute rainfall depth and the 100-year, 15-minute rainfall value. The conversion ratios provided in Table 602 are multiplied by 1-hour rainfall depth.

\[ \text{RATIO5} = 0.33 \]
\[ \text{RATIO15} = 0.60 \]

\[ D_{100,5} = \text{RATIO5} \times D_{100,1} = 0.33 \times 1.45 = 0.48 \text{ inches} \]
\[ D_{100,15} = \text{RATIO15} \times D_{100,1} = 0.60 \times 1.45 = 0.87 \text{ inches} \]

Step 4: Compute the 100-year, 2-hour rainfall event, 100-year, 3-hour rainfall event, and the 100-year, 12-hour rain values using Equations 606, 607, and 608, respectively.

\[ D_{100,2} = 0.299 \times D_{100,6} + 0.701 \times D_{100,1} = 0.299 \times 2.26 + 0.701 \times 1.45 = 1.69 \text{ inches} \]
\[ D_{100,3} = 0.526 \times D_{100,6} + 0.474 \times D_{100,1} = 0.526 \times 2.26 + 0.474 \times 1.45 = 1.88 \text{ inches} \]
\[ D_{100,12} = 0.5 \times D_{100,6} + 0.5 \times D_{100,24} = 0.5 \times 2.26 + 0.5 \times 3.55 = 2.91 \text{ inches} \]

Step 5: Estimate the depth-area reduction factor from Figure 604.

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

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

604.3 **EXAMPLE: TIME-INTENSITY-FREQUENCY CURVE GENERATION FOR ZONE III**

**Problem:** Develop the 5-year and 100-year time-intensity-frequency curves for Rose Subdivision located on the eastern edge of Drainage Basin A (See Figure 601).

**Solution:**

**Step 1:** Estimate the 1-hour rainfall depth for the 2-year recurrence interval for Rose Subdivision from Figure 601. The location of Rose Subdivision is not shown on Figure 601 to prevent this hypothetical example problem from being construed to represent an actual location.

\[ D_{2,1} = 0.4 \text{ inches (Assumed for example purposes only)} \]

**Step 2:** Determine the 1-hour rainfall depth for the 5-year and 100-year recurrence intervals. The regional growth factors found in Table 601 are used to obtain the 1-hour depths.

\[ \text{RGF (5,1)} = 1.36 \]
\[ \text{RGF (100,1)} = 3.62 \]

\[ D_{5,1} = \text{RGF(5,1)} \times D_{2,1} = 1.36 \times 0.4 = 0.54 \text{ inches} \]
\[ D_{100,1} = \text{RGF(100,1)} \times D_{2,1} = 3.62 \times 0.4 = 1.45 \text{ inches} \]

**Step 3:** Generate 5-year rainfall depths for storm durations of 5 minutes, 10 minutes, 15 minutes, and 30 minutes. The conversion ratios in Table 602 are used to obtain these values.

\[ D_{5,5} = \text{RATIO5*} D_{5,1} = 0.33 \times 0.54 = 0.18 \text{ inches} \]
\[ D_{5,10} = \text{RATIO10*} D_{5,1} = 0.49 \times 0.54 = 0.26 \text{ inches} \]
\[ D_{5,15} = \text{RATIO15*} D_{5,1} = 0.60 \times 0.54 = 0.32 \text{ inches} \]
\[ D_{5,30} = \text{RATIO30*} D_{5,1} = 0.82 \times 0.54 = 0.44 \text{ inches} \]

**Step 4:** Generate the 100-year rainfall depths for storm durations of 5 minutes, 10 minutes, 15 minutes and 30 minutes.

\[ D_{100,5} = \text{RATIO5*} D_{100,1} = 0.33 \times 1.45 = 0.48 \text{ inches} \]
\[ D_{100,10} = \text{RATIO10*} D_{100,1} = 0.49 \times 1.45 = 0.71 \text{ inches} \]
\[ D_{100,15} = \text{RATIO15*} D_{100,1} = 0.60 \times 1.45 = 0.87 \text{ inches} \]
\[ D_{100,30} = \text{RATIO30*} D_{100,1} = 0.82 \times 1.45 = 1.19 \text{ inches} \]

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

\[ I_{5,5} = D_{5,5} / \text{Duration} = 0.18 / (5/60) = 2.16 \text{ inches/hour} \]
\[ I_{5,10} = D_{5,10} / \text{Duration} = 0.26 / (10/60) = 1.56 \text{ inches/hour} \]
\[ I_{5,15} = D_{5,15} / \text{Duration} = 0.32 / (15/60) = 1.28 \text{ inches/hour} \]
\[ I_{5,30} = D_{5,30} / \text{Duration} = 0.44 / (30/60) = 0.88 \text{ inches/hour} \]
\[ I_{5,60} = D_{5,60} / \text{Duration} = 0.54 / (60/60) = 0.54 \text{ inches/hour} \]
Step 6: Calculate 100-year rainfall intensity values for storm durations of 5 minutes, 10 minutes, 15 minutes, 30 minutes, and 60 minutes.

\[
\begin{align*}
I_{100,5} &= \frac{D_{100.5}}{\text{Duration}} = \frac{0.48}{(5/60)} = 5.76 \text{ inches/hour} \\
I_{100,10} &= \frac{D_{100.10}}{\text{Duration}} = \frac{0.71}{(10/60)} = 4.26 \text{ inches/hour} \\
I_{100,15} &= \frac{D_{100.15}}{\text{Duration}} = \frac{0.87}{(15/60)} = 3.48 \text{ inches/hour} \\
I_{100,30} &= \frac{D_{100.30}}{\text{Duration}} = \frac{1.19}{(30/60)} = 2.38 \text{ inches/hour} \\
I_{100,60} &= \frac{D_{100.60}}{\text{Duration}} = \frac{1.45}{(60/60)} = 1.45 \text{ inches/hour}
\end{align*}
\]

Step 7: Plot the time-intensity curves for the 5-year and 100-year frequencies for Rose Subdivision. (See Figure 608).

Application: The time-intensity-frequency curve is used to determine the rainfall intensities used in the Rational Method of determining runoff described in Section 700.
## Regional Growth Factors

**Non-dimensional**

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<th>10-Yr.</th>
<th>25-Yr.</th>
<th>50-Yr.</th>
<th>100-Yr.</th>
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## Conversion Ratios for Durations of Less Than One-Hour

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<td>.60</td>
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<tr>
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<td>.82</td>
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### Time-Intensity-Frequency Values for Zone I and Zone II

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RAINFALL DEPTH 2-YEAR, 1-HOUR: SOUTHERN WASHOE COUNTY
RAINFALL DEPTH 2-YEAR, 1-HOUR: NORTHERN WASHOE COUNTY
NOTE:

1. CONSULT WITH THE LOCAL ENTITY AND/OR WASHOE COUNTY FOR GUIDANCE IN USING THE DEPTH–AREA REDUCTION FACTORS AND STORM CENTERING FOR MODELLING OF DRAINAGE AREAS GREATER THAN 200 MILES.
## TABLE OF CONTENTS

<table>
<thead>
<tr>
<th>Section</th>
<th>Description</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>701 INTRODUCTION</td>
<td>Basin Characteristics</td>
<td>702</td>
</tr>
<tr>
<td>702 TIME OF CONCENTRATION</td>
<td>Urbanized Basins</td>
<td>704</td>
</tr>
<tr>
<td>703 PRECIPITATION LOSSES</td>
<td>Introduction</td>
<td>704</td>
</tr>
<tr>
<td>703.2.1 SCS Curve Number Method</td>
<td>CN Determination</td>
<td>706</td>
</tr>
<tr>
<td>704 RATIONAL FORMULA METHOD</td>
<td>Methodology</td>
<td>707</td>
</tr>
<tr>
<td>704.2 Assumptions</td>
<td></td>
<td>707</td>
</tr>
<tr>
<td>704.3 Limitations on Methodology</td>
<td></td>
<td>707</td>
</tr>
<tr>
<td>704.4 Rainfall Intensity</td>
<td></td>
<td>708</td>
</tr>
<tr>
<td>704.5 Runoff Coefficient</td>
<td></td>
<td>708</td>
</tr>
<tr>
<td>704.6 Application of the Rational Formula Method</td>
<td></td>
<td>708</td>
</tr>
<tr>
<td>704.7 Major Storm Analysis</td>
<td></td>
<td>708</td>
</tr>
<tr>
<td>705 SCS UNIT HYDROGRAPH METHOD</td>
<td>Methodology</td>
<td>709</td>
</tr>
<tr>
<td>705.1 Assumptions</td>
<td></td>
<td>710</td>
</tr>
<tr>
<td>705.3 Lag Time</td>
<td>Roughness Factor</td>
<td>711</td>
</tr>
<tr>
<td>705.4 Unit Storm Duration</td>
<td></td>
<td>711</td>
</tr>
<tr>
<td>705.5 Sub-basin Sizing</td>
<td></td>
<td>711</td>
</tr>
<tr>
<td>706 CHANNEL ROUTING OF HYDROGRAPHS</td>
<td></td>
<td>711</td>
</tr>
<tr>
<td>707 RESERVOIR ROUTING OF HYDROGRAPHS</td>
<td>Modified Puls Method</td>
<td>712</td>
</tr>
</tbody>
</table>
WASHOE COUNTY
HYDROLOGIC CRITERIA AND DRAINAGE DESIGN MANUAL

708 STATISTICAL ANALYSIS

709 EXAMPLE APPLICATIONS
709.1 - Example: Rational Formula Method
709.2 - Example: SCS Unit Hydrograph Method

LIST OF TABLES

701 RATIONAL FORMULA METHOD RUNOFF COEFFICIENTS AND AVERAGE PERCENT IMPERVIOUS AREA
702 RUNOFF CURVE NUMBERS
703 LAG EQUATION ROUGHNESS FACTORS
704 EXAMPLE: RATIONAL FORMULA METHOD RESULTS

LIST OF FIGURES

701 TRAVEL TIME VELOCITY
702 PERCENTAGE OF IMPERVIOUS AREA VS. COMPOSITE CN'S FOR GIVEN PERVIOUS AREA CN'S
703 EXAMPLE: ROSE SUBDIVISION
704 EXAMPLE: HEC-1 INPUT DATA AND OUTPUT DATA FOR BASIN A

December 2, 1996
Storm Runoff
SECTION 700
STORM RUNOFF

701  INTRODUCTION

For the area within the jurisdiction of this Manual, two deterministic hydrological models can be used to predict storm runoff (POLICY Section 304.3). These models are the Rational Formula method and the SCS Unit Hydrograph method. The procedures for using these methods are presented in this section. The Rational Formula method may be employed without the use of computers. Computer modelling using the U.S. Army Corps of Engineers HEC-1 Flood Hydrograph Package or other hydrologic computer modelling programs (i.e. TR-20) required for the SCS method. For certain circumstances, where adequate recorded stream flow data are available and the drainage area is large (> 10 square miles), a statistical analysis may be required to predict the storm runoff peaks or for calibration of deterministic models (see Section 708).

701.1  BASIN CHARACTERISTICS

The basin characteristics needed for the subject runoff computation methods include the drainage area, soil type, the various flow path lengths, slopes, and characteristics (i.e., overland, grassed channel, gutter) and land use types. The drainage basin boundary and area may be determined from available topographic maps or site-specific mapping depending upon the level of detail required. A field investigation is recommended to verify drainage boundaries. The land use and flow path characteristics can be obtained from zoning maps, aerial photographs, field investigations, or detailed topographic maps.

702  TIME OF CONCENTRATION

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

In the past, several different time of concentration equations have been used with the runoff methods discussed in the following sections. However, as both methods have the same definition of the time of concentration, and to promote consistency between the two runoff methods, the time of concentration equations presented in this section shall be used for all watersheds of total area less than one square mile and who's basin slope is less than ten percent. For larger watersheds and for watersheds with basin slopes equal to or greater than ten percent, the basin lag equation shall be used (see Section 705.3).

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

\[ t_c = t_i + t_t \]  

(701)

In which
- \( t_c \) = time of concentration (minutes)
- \( t_i \) = initial, inlet, or overland flow time (minutes)
- \( t_t \) = travel time in the ditch, channel, gutter, storm sewer, etc. (minutes)

To aid in the computation of \( t_c \), Standard Form 2 (see Section - 1500) has been developed to organize the computation. In all drainage studies, \( t_c \) calculations should be submitted using Standard Form 2.

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

\[ t_i = \frac{1.8 \times (1.1 - R) \times L_{o}}{S^{0.3}} \]  

(702)

Where
- \( t_i \) = initial or overland flow time (minutes)
- \( R \) = flow runoff coefficient
- \( L_o \) = length of overland flow (feet, 500-feet maximum)
- \( S \) = average overland basin 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 using the appropriate flow runoff coefficient.

For the Rational Formula method, the 5-year runoff coefficient, \( C_s \), presented in Table 701 shall be used as the flow runoff coefficient, \( R \). For the SCS Unit Hydrograph method, \( R \) shall be calculated using the following equation:

\[ R = 0.0132 \times CN - 0.39 \]  

(703)

This equation was developed by converting CN factors to typical \( C_s \) runoff coefficients.

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

For longer basin lengths, initial or overland flow needs to be considered in combination with the travel time, \( t_t \), which is calculated using the hydraulic properties of the swale, ditch, or channel. For preliminary work, travel time can be estimated with the help of Figure 701 (SCS, 1985). The time of concentration is then the sum of the initial flow time, \( t_i \) and the travel time \( t_t \) (Equation 701). The minimum \( t_c \) in Washoe County for non-urban watersheds shall be 10 minutes.
702.1 URBANIZED BASINS

Overland flow in urbanized basins can occur from the back of the lot to the street, in parking lots, in greenbelt areas, or within park areas. It can be calculated using the procedure described in Section 701 except that the travel time, $t_c$, to the first design point or inlet is estimated using the "Paved Area (Sheet Flow) & Shallow Gutter Flow" line in Figure 701 or an estimated velocity based on the allowable street flow figures in Section 1000. The time of concentration for the first design point in an urbanized basin using this procedure should not exceed the time of concentration calculated using Equation 704, which was developed using rainfall/runoff data collected in urbanized regions (USDCM, 1969).

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

Where $t_c =$ time of concentration at the first design point in an urban watershed (minutes)

$L =$ watershed length (feet)

Equation 704 may result in a lesser time of concentration at the first design point and thus would govern in an urbanized watershed. For subsequent design points, the time of concentration is calculated by accumulating the travel times in downstream reaches. The minimum $t_c$ in Washoe County for urbanized paved areas shall be 5 minutes and 10 minutes for vegetated landscaped areas.

A common mistake in calculating $t_c$ is to assume travel velocities (for $t_c$) that are too small or not using post development slopes. Another common error is to not analyze the portion of basin which would result in the longest computed time of concentration. This error is most often encountered in long basins, or a basin where the upper portion contains grassy park land and the lower developed urban land. However, a check should be performed to assure that runoff from the total area is not exceeded by runoff from only the urbanized area.

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

703 PRECIPITATION LOSSES

Precipitation loss calculations are required for the SCS Unit Hydrograph method. The calculation methodology for precipitation losses within the Washoe County is presented in the following section. For the Rational Formula method, the precipitation losses are not computed separately. Therefore, the following methodology does not apply to the Rational Formula method.

703.1 INTRODUCTION

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

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

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

There are several methods that can be used to calculate the precipitation loss. These methods include the Initial and Uniform Loss Rate, Exponential Loss Rate, Holtan Loss Rate, Horton Loss Rate, Green-Ampt and SCS Curve Number method to name a few. The SCS Curve Number method is recommended for the Washoe County area because there is a lack of data for use in other methods and the local consultants are familiar with using this method. In addition, modelling of sample areas within Washoe County has shown that this method will result in reasonable and justifiable runoff rates. The dynamic nature of flood hydrology may show that, in the future, a different method of computing rainfall loss (i.e. Green-Ampt for example) may be more accurate than the CN method shown herein. However, a change in the loss methodology should only be made upon substantial showing that said method can be easily applied by the local consultants and that data is available to make applicable changes in design rainfall distribution/timing. In the SCS Curve Number method, an average precipitation loss is determined for a computation interval and subtracted from the rainfall hyetograph. The resulting precipitation excess is used to compute an outflow hydrograph for a sub-basin.

703.2 SCS CURVE NUMBER METHOD

The Soil Conservation Service (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 drainage characteristics of soil groups to a curve number, CN (SCS, 1985). The SCS provides information on relating soil group type to the curve number as a function of soil cover, land use type and antecedent moisture conditions.

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

\[ Q = \frac{(P-IA)^2}{(P-IA) + S} \]  \hspace{1cm} (705)

\[ S = \frac{1000}{CN} - 10 \]  \hspace{1cm} (706)

where 
- \( Q \) = Accumulated excess (inches)
- \( P \) = Accumulated rainfall depth (inches)
- \( IA \) = Initial surface moisture storage capacity (inches)
- \( S \) = Currently available soil moisture storage deficit (inches)

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

\[ IA = 0.2S \]  \hspace{1cm} (707)

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

703.2.1 CN Determination

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

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

For the Washoe County area, an AMC-II condition shall be used for determining storm runoff.

Having determined the soil group, land use and treatment class and the antecedent moisture condition, CN values can be determined from Table 702.

There will be areas to which the values in Table 702 do not apply. The percentage of impervious area for the various types of residential areas or the land use condition for the pervious portions may vary from the conditions assumed in Table 702. A curve for each pervious CN can be developed to determine the composite CN for any density of impervious area. Figure 702 has been developed assuming a CN of 98 for the impervious area. The curves in Figure 702 can help in estimating the increase in runoff as more and more land within a given area is covered with impervious material.

There are a number of methods available for computing the percentage of impervious area in a watershed. Some methods include using U.S. Geological Survey topographic maps, land use maps, aerial photographs, and field reconnaissance. Care must be exercised when using methods based on such parameters as population density, street density, and age of the development as a means of determining the percentage of impervious area. The available data on runoff from urban areas are not yet sufficient to validate widespread use of these methods. Therefore, the CN to be used in the Washoe County area shall be based on Table 702 or Figure 702 in this Manual. A CN computation example is included in Section 711.
RATIONAL FORMULA METHOD

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

704.1 METHODOLOGY

The Rational Formula method is based on the formula:

\[ Q = CIA \]  \hspace{1cm} (708)

Q is defined as the maximum rate of runoff in cubic feet per second (actually, Q has units of acre inches per hour, which is approximately equal to the units of cubic feet per second). C is a runoff coefficient and represents the runoff-producing conditions of the subject land area (see Section 704.5). I is the average intensity of rainfall in inches per hour for a duration equal to the time of concentration. A is the contributing basin area in acres.

704.2 ASSUMPTIONS

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

1. The computed maximum rate of runoff to the design point is a function of the average rainfall rate during the time of concentration to that point.
2. The maximum rate of rainfall occurs during the time of concentration, and the design rainfall depth during the time of concentration is converted to the average rainfall intensity for the time of concentration.
3. The maximum runoff rate occurs when the entire area is contributing flow. However, this assumption has been modified from time to time when local rainfall/runoff data was used to improve calculated results.

704.3 LIMITATIONS ON METHODOLOGY

The Rational Formula method adequately approximates the peak rate of runoff from a rainstorm in a given basin. The critics of the method usually are unsatisfied with the fact that the answers are only approximations. A shortcoming of the Rational Formula method is that only one point on the runoff hydrograph is computed (the peak runoff rate).

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

704.4 RAINFALL INTENSITY

The rainfall intensity, I, is the average rainfall rate in inches per hour for the period of maximum rainfall of a given frequency having a duration equal to the time of concentration. After the design storm frequency has been selected, a graph should be prepared showing rainfall intensity versus time. The procedure for obtaining the local data and preparing the graph is explained and illustrated in Section 603 of this Manual.

704.5 RUNOFF COEFFICIENT

The runoff coefficient, C, represents the integrated effects of infiltration, evaporation, retention, flow routing, and interception, all which affect the time distribution and peak rate of runoff. Determination of the coefficient requires judgment and understanding on the part of the engineer. Table 701 presents the recommended values of C for the various recurrence frequency storms. The values are presented for different surface characteristics as well as for different aggregate land uses.

A composite runoff coefficient is computed on the basis of the percentage of different types of surfaces in the drainage area. For existing developed areas, this procedure is often applied to a typical "sample" area as a guide to selection of reasonable values of the coefficient for an entire area. Suggested coefficients with respect to surface type are also given in Table 701 under the column labeled "Percent Impervious". Where land use features are existing, a composite C analysis will result in more accurate results. The runoff coefficients in Table 701 vary with recurrence frequency and therefore, further adjustments of the C factor are not needed.

704.6 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 drainage basins. Basins to be defined include all basins tributary to the area of study and sub-basins within the study area. A field check and possibly field surveys should be made for each basin. At this stage of planning, the possibility for the diversion of transbasin waters should be identified.

The major storm drainage basin does not always coincide with the minor storm drainage basin. This is often the case in urban areas where a low flow will stay next to a curb and follow the lowest grade, but when a large flow occurs the water will be deep enough so that part of the water will overflow street crowns and flow into a new sub-basin. An example of how to apply the Rational Formula method is presented in Section 709.

704.7 MAJOR STORM ANALYSIS

When analyzing the major runoff occurring on an area that has a storm sewer system sized for the minor storm, care must be used when applying the Rational Formula method. Normal application of the Rational Method assumes that all of the runoff is collected by the storm sewer. For the minor storm design, the time of concentration is dependent upon the flow time in the sewer. However, during the major storm runoff, the sewers will probably be at capacity and would not carry the additional water flowing to the inlets. This additional water then flows overland past the inlets, generally at a lower velocity than the flow in the storm sewers.
If a separate time of concentration analysis is made for the pipe flow and surface flow, a time lag between the surface flow peak and the pipe flow peak will occur. This lag, in effect, will allow the pipe to carry a larger portion of the major storm runoff than would be predicted using the minor storm time of concentration. The basis for this increased benefit is that the excess water from one inlet will flow to the next inlet downhill, using the overland route. If that inlet is also at capacity, the water will often continue until capacity is available in the storm sewer. The analysis of this aspect of the interaction between the storm sewer system and the major storm runoff is complex. The simplified approach of using the minor storm time of concentration for all frequency analysis is acceptable for use in Washoe County.

705 SCS UNIT HYDROGRAPH METHOD

The SCS Unit Hydrograph method was developed for the SCS by Mr. Victor Mockus. The SCS Unit Hydrograph was derived from a large number of natural unit hydrographs from watersheds varying widely in size and geographic location. The SCS Unit Hydrograph has been in use for many years and has produced satisfactory results for many applications. This method may be used for drainage areas within the Washoe County area in accordance with Section 304.3.

705.1 METHODOLOGY

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

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

1. The time-to-peak, $T_p$, of the unit hydrograph approximately equals 0.2 times the time-of-base, $T_b$.

2. The point of inflection of the falling leg of the unit hydrograph approximately equals 1.7 times $T_p$.

For ease of calculation, an equivalent triangular unit hydrograph was derived from the natural curvilinear unit hydrograph. From the triangular unit hydrograph, equations for the peak discharge, $Q_p$, time-to-peak, $T_p$, and the time of concentration, $t_c$, were developed based on a single lag factor (TLAG). The discharge hydrograph is then determined for the SCS Unit Hydrograph method based on the storm excess precipitation applied to the unit hydrograph whose parameters are determined by TLAG. TLAG is defined and discussed in Section 705.3.

705.2 ASSUMPTIONS

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

1. The effects of all physical characteristics of a given drainage basin are reflected in the shape of the storm runoff hydrograph for that basin.
2. At a given point on a stream, discharge ordinates of different unit graphs of the same unit time of rainfall excess are mutually proportional to respective volumes.

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

705.3 LAG TIME

Input data for the Soil Conservation Service dimensionless unit hydrograph method (SCS, 1985) consists of a single parameter, TLAG, which is equal to the lag (in hours) between the center of mass of rainfall excess and the peak of the unit hydrograph. For small drainage basins (less than one square mile) and basin slopes less than ten percent the lag time may be related to the time of concentration, \( t_c \), by the following empirical relationship:

\[
TLAG = 0.6 t_c
\]  
(709)

The \( t_c \) is computed as presented in Section 702.

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

\[
TLAG = 22.1 K_n \left( L L_s / S^{0.5} \right)^{0.33}
\]  
(710)

where

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

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

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

705.3.1 Roughness Factor

The selection of a proper roughness factor for use in the lag time calculation is highly subjective. Therefore, in order to obtain more consistent lag time and runoff analysis results, the roughness factor, \( K_n \), shall be determined using the factors presented in Table 703. These factors are based on roughness factor analysis performed in the Washoe County and Carson City areas, analysis performed for the Sacramento, California area, and by USBR (1989) as
compared to the typical watershed channels found in the Washoe County area. The reader is referred to these documents for further discussion on selection of a proper roughness factor.

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

705.4 **UNIT STORM DURATION**

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

705.5 **SUB-BASIN SIZING**

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

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

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

706 **CHANNEL ROUTING OF HYDROGRAPHS**

Whenever a large or a non-homogeneous basin is being investigated, the basin should be divided into smaller and more homogeneous sub-basins. The storm hydrograph for each sub-basin can then be calculated using the procedures described in Sections 704 or 705. The user then must route and combine the individual sub-basin hydrographs to develop a storm hydrograph for the entire watershed. There are several methods available for use in channel routing which include:

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

The two most commonly used routing techniques are the Muskingum-Cunge (an approximate diffusion router) and the Kinematic Wave (a finite-difference technique). Of these, the Muskingum-Cunge is the
preferable method for use in Washoe County. The Muskingum-Cunge method provides accurate results over a wide range of flow conditions, where as the Kinematic Wave method should only be used in relatively short reaches such as those encountered in an urban environment. Numerical errors introduced when solving the Kinematic Wave technique may cause a greater attenuation of the peak flow than actually occurs. The Kinematic Wave technique can only be used for specific types of channel shapes (i.e., trapezoidal, rectangular, etc.), but the Muskingum-Cunge technique can be used for channels with standard prismatic shapes or channels with irregular cross section shapes. Since the HEC-1 program computes hydrograph lagging based on internally selected computation interval, the user should always check that the peak generated from the internally selected computation interval are comparable to the result peaks shown in the output at the user determined intervals. In some instances, an error message will occur with Muskingum-Cunge method which terminates the program computations. In this instance the Muskingum method should be used.

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

707 RESERVOIR ROUTING OF HYDROGRAPHS

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

Storm runoff detention is required for some new development (Section 303.7) and therefore detention reservoirs will be required (see Section 1300). In some instances, the sizing of the detention storage will be based upon hydrograph storage routing techniques rather than direct calculation of volume and discharge requirements. The methodology for Manual computation of reservoir routing is presented in this section. This method is computerized and is part of the HEC-1 program. The input requirements are explained in the HEC-1 Users Manual.

707.1 MODIFIED PULS METHOD

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

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

\[ (I-D)t = \Delta S \]  

(720)

where I is the inflow rate, D is the discharge rate, t is the time interval, and \( \Delta S \) 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:

\[ \frac{(I_1 + I_2)}{2} - \frac{(D_1 + D_2)}{2} \cdot \frac{t}{2} = S_2 - S_1 \]  

(721)
where the subscripts 1 and 2 refer to the beginning and end of time period \( t \). Rearranging the equation gives the following form used for the Modified Puls method:

\[
I_1 + I_2 + (2S_1/t - D_1) = (2S_2/t + D_2)
\]  

(722)

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

708  

STATISTICAL ANALYSIS

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

Before proceeding with a statistical analysis, the analyst shall contact the Washoe County Engineering Department and the USGS to obtain (when available) applicable data, information, other relevant studies, and criteria for evaluation.

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

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

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

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

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

709 EXAMPLE APPLICATIONS

709.1 EXAMPLE: RATIONAL FORMULA METHOD

Problem: Determine the 5-year flow at the design points within Rose Subdivision shown in Figure 703. The flow sequences is as follows: Design Point 1 flows to Design Point 2. Design Point 2 flows to Design Point 3. Design Point 5 flows to Design Point 6. Design Points 3 and 6 flow to Design Point 4. Design Point 4 flows into the proposed detention basin represented by Design Point 7 and Design Point 7 finally flows to Design Point 8 located in Doe Creek.

Solution:

Step 1: Estimate the flow runoff coefficients for each sub-basin in Rose Subdivision. Rose Subdivision is a single family residential area with an average lot size of ⅓ acre. The flow runoff coefficient, R, is assumed to be equal to the 5-year runoff coefficient, C, which are provided in Table 701.

\[ R_A = R_B = R_C = R_D = R_E = R_F = R_G = C = 0.45 \]

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

\[ t_i = \frac{1.8(1.1-R)L}{S^{\frac{1}{6}}} = \frac{1.8(1.1-0.45)(150)^{\frac{1}{6}}}{(1.5)^{\frac{1}{6}}} = 12.5 \text{ Minutes} \]

Step 3: Compute the travel time of the runoff in the street gutter to the designated design point using Figure 701. Only the calculation for the travel time to Design Point 1 is shown in the example. The results of the remaining travel time calculations are shown in Table 704.

Assuming the runoff combines and flows down the street at a 2.5% grade, Figure 701 estimates the runoff velocity in the street to be:

\[ V_A = 3.4 \text{ feet per second (fps)} \]

The gutter flow length in sub-basin A is:

\[ L_A = 900 \text{ feet} \]

and the travel time will be:

\[ t_i = \frac{L}{60V} = \frac{900}{60 \times 3.4} = 4.4 \text{ Minutes} \]
Step 4:
Calculate the time of concentration using Equations 701 and 704 at Design Point 1. Select the smaller time delivered by the two equations as the final time of concentration at each design point.

\[ t_c = t_1 + t_4 = 12.5 + 4.4 = 16.9 \text{ Minutes} \]

\[ t_1 = \frac{L}{180} + 10 = \frac{1050}{180} + 10 = 15.8 \text{ Minutes} \]

Since Equation 704 gives the smaller value, the time of concentration at Design Point 1 is:

\[ t_{c1} = 15.8 \text{ Minutes} \]

Step 5:
Estimate the time of concentration at downstream design points. When multiple subbasins drain to a common design point, continue the time of concentration calculations in the downstream direction. The flow calculated at each design point is used to estimate the flow velocity in the downstream pipe, gutter, swale, or channel.

This flow velocity is then used to calculate the time of travel to the next downstream design point. Table 704 shows the use of Standard Form 2 and presents the results of the remaining calculations to determine the time of concentration at each design point.

Step 6:
Determine the 5-year runoff coefficient \((C_s)\) at each design point from Table 701.

\[ C_{s1} = C_{s2} = C_{s3} = C_{s4} = C_{s5} = C_{s6} = C_{s7} = 0.45 \]

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

Step 7:
Determine the 5-year rainfall intensity \((I_s)\) at each design point using the time of concentration calculations in Step 4 (and shown in Table 704) and Figure 608.

\[ I_{s1} = 1.23 \text{ Inches/hour} \]
\[ I_{s2} = 1.22 \text{ Inches/hour} \]
\[ I_{s3} = 1.18 \text{ Inches/hour} \]
\[ I_{s4} = 1.16 \text{ Inches/hour} \]
\[ I_{s5} = 1.23 \text{ Inches/hour} \]
\[ I_{s6} = 1.18 \text{ Inches/hour} \]
\[ I_{s7} = 1.14 \text{ Inches/hour} \]

Step 8:
Calculate the 5-year peak flow \((Q_s)\) at each design point using Equation 708.

\[ Q_{s1} = C_s * I_{s1} * A_i = 0.45 * 1.23 * 4.13 = 2.3 \text{ cfs} \]
\[ Q_{s2} = 0.45 * 1.22 * 5.94 = 3.3 \text{ cfs} \]
\[ Q_{s3} = 0.45 * 1.18 * 8.26 = 4.4 \text{ cfs} \]
\[ Q_{s4} = 0.45 * 1.16 * 14.72 = 7.7 \text{ cfs} \]
\[ Q_{s5} = 0.45 * 1.23 * 3.36 = 1.9 \text{ cfs} \]
\[ Q_s = 0.45 \times 1.18 \times 4.65 = 2.5 \text{ cfs} \]
\[ Q_{se} = 0.45 \times 1.14 \times 15.5 = 8.0 \text{ cfs} \]

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

APPLICATION: The results from the Rational Formula Method are used to design the drainage system in an urban environment. The results from this example problem will be used in subsequent example problems.

709.2 EXAMPLE: SCS UNIT HYDROGRAPH METHOD

Problem: Determine the 100-year, 24-hour runoff hydrograph on Doe Creek immediately upstream of John Boulevard and Rose Subdivision.

Solution:

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

\[ DA = 3.34 \text{ square miles} = 2140 \text{ acres} \]

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

<table>
<thead>
<tr>
<th>Land Use</th>
<th>Soil Type</th>
<th>CN</th>
<th>Area (Acres)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Forest</td>
<td>B</td>
<td>54</td>
<td>200</td>
</tr>
<tr>
<td>Forest</td>
<td>C</td>
<td>66</td>
<td>1100</td>
</tr>
<tr>
<td>Shrub/Brush</td>
<td>B</td>
<td>56</td>
<td>840</td>
</tr>
</tbody>
</table>

\[ CN_{ave} = \frac{(54 \times 200 + 66 \times 1100 + 56 \times 840)}{2140} = 61.0 \]

Step 3: Measure the length of the longest water course \((L)\).

\[ L = 22100 \text{ feet} = 4.19 \text{ miles} \]

Step 4: Measure the length along Doe Creek from the John Boulevard Bridge to the point opposite the centroid of the basin \((L_w)\).
\[ L_e = 2.05 \text{ miles} \]

Step 5: Calculate the average slope of Doe Creek.

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

\[ \text{Slope} = \frac{7276 - 4920}{4.19} = 563 \text{ feet/mile} \]

Step 6: Estimate the average roughness factor, \( K_a \) for Doe Creek using Table 703.

<table>
<thead>
<tr>
<th>Land Use</th>
<th>( K_a )</th>
<th>Area</th>
</tr>
</thead>
<tbody>
<tr>
<td>Forest</td>
<td>0.15</td>
<td>1300</td>
</tr>
<tr>
<td>Shrub/Brush</td>
<td>0.1</td>
<td>840</td>
</tr>
</tbody>
</table>

\[ K_a = (0.15 \times 1300 + 0.1 \times 840) / 2140 = 0.130 \]

Step 7: Calculate the lag time (TLAG) for the SCS dimensionless unit hydrograph using Equation 710.

\[ \text{TLAG} = 22.1 \times K_a \times (L \times L_e / S^{0.5})^{0.33} \]

\[ \text{TLAG} = 22.1 \times 0.13 \times (4.19 \times 2.05 / 563^{0.5})^{0.33} = 2.05 \text{ hours} \]

Step 8: Input the necessary information into the HEC-1 program and run HEC-1 to obtain the 100-year, 24-hour storm hydrograph at John Boulevard Bridge. The HEC-1 program will require KK, BA, LS, PH, and UD cards. The rainfall distribution information was obtained from Section 604.2. The results from HEC-1 model are provided in Figure 704.
## Rational Formula Method

### Runoff Coefficients

<table>
<thead>
<tr>
<th>Land Use or Surface Characteristics</th>
<th>Aver. % Impervious Area</th>
<th>5-Year ( C_s )</th>
<th>100-Year ( C_{100} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Business/Commercial:</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Downtown Areas</td>
<td>85</td>
<td>.82</td>
<td>.85</td>
</tr>
<tr>
<td>Neighborhood Areas</td>
<td>70</td>
<td>.65</td>
<td>.80</td>
</tr>
<tr>
<td>Residential:</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(Average Lot Size)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>¼ Acre or Less (Multi-Unit)</td>
<td>65</td>
<td>.60</td>
<td>.78</td>
</tr>
<tr>
<td>¼ Acre</td>
<td>38</td>
<td>.50</td>
<td>.65</td>
</tr>
<tr>
<td>½ Acre</td>
<td>30</td>
<td>.45</td>
<td>.60</td>
</tr>
<tr>
<td>¾ Acre</td>
<td>25</td>
<td>.40</td>
<td>.55</td>
</tr>
<tr>
<td>1 Acre</td>
<td>20</td>
<td>.35</td>
<td>.50</td>
</tr>
<tr>
<td>Industrial:</td>
<td>72</td>
<td>.68</td>
<td>.82</td>
</tr>
<tr>
<td>Open Space:</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(Lawns, Parks, Golf Courses)</td>
<td>5</td>
<td>.05</td>
<td>.30</td>
</tr>
<tr>
<td>Undeveloped Areas:</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Range</td>
<td>0</td>
<td>.20</td>
<td>.50</td>
</tr>
<tr>
<td>Forest</td>
<td>0</td>
<td>.05</td>
<td>.30</td>
</tr>
<tr>
<td>Streets/Roads:</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Paved</td>
<td>100</td>
<td>.88</td>
<td>.93</td>
</tr>
<tr>
<td>Gravel</td>
<td>20</td>
<td>.25</td>
<td>.50</td>
</tr>
<tr>
<td>Drives/Walks:</td>
<td>95</td>
<td>.87</td>
<td>.90</td>
</tr>
<tr>
<td>Roofs</td>
<td>90</td>
<td>.85</td>
<td>.87</td>
</tr>
</tbody>
</table>

**Notes:**

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

---

**VERSION:** December 2, 1996

**REFERENCE:** USDCM, DROCOG, 1969 (with modifications)

**WRC ENGINEERING, INC.**

**TABLE 701**
## Runoff Curve Numbers

<table>
<thead>
<tr>
<th>Land Use or Surface Characteristics</th>
<th>Aver. % Impervious Area</th>
<th>Soil Comp A</th>
<th>Soil Comp B</th>
<th>Soil Comp C</th>
<th>Soil Comp D</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Business/Commercial:</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Downtown Areas</td>
<td>85</td>
<td>89</td>
<td>92</td>
<td>94</td>
<td>95</td>
</tr>
<tr>
<td>Neighborhood Areas</td>
<td>70</td>
<td>80</td>
<td>87</td>
<td>91</td>
<td>93</td>
</tr>
<tr>
<td><strong>Residential:</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(Average Lot Size)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1/8 Acre or Less (Multi-Unit)</td>
<td>65</td>
<td>77</td>
<td>85</td>
<td>90</td>
<td>92</td>
</tr>
<tr>
<td>1/4 Acre</td>
<td>38</td>
<td>61</td>
<td>75</td>
<td>83</td>
<td>87</td>
</tr>
<tr>
<td>1/3 Acre</td>
<td>30</td>
<td>57</td>
<td>72</td>
<td>81</td>
<td>86</td>
</tr>
<tr>
<td>1/2 Acre</td>
<td>25</td>
<td>54</td>
<td>70</td>
<td>80</td>
<td>85</td>
</tr>
<tr>
<td>1 Acre</td>
<td>20</td>
<td>51</td>
<td>68</td>
<td>79</td>
<td>84</td>
</tr>
<tr>
<td><strong>Industrial:</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Irrigated Areas:</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Lawns, Parks, Golf Courses/Agriculture</td>
<td>5</td>
<td>41</td>
<td>62</td>
<td>75</td>
<td>81</td>
</tr>
<tr>
<td>0</td>
<td>0</td>
<td>39</td>
<td>61</td>
<td>74</td>
<td>80</td>
</tr>
<tr>
<td><strong>Undeveloped Areas (Open Space):</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Herbaceous (grasses)</td>
<td>0</td>
<td>40</td>
<td>62</td>
<td>74</td>
<td>85</td>
</tr>
<tr>
<td>Mixed Grass and Shrub</td>
<td>0</td>
<td>39</td>
<td>61</td>
<td>73</td>
<td>82</td>
</tr>
<tr>
<td>Shrub/Brush</td>
<td>0</td>
<td>35</td>
<td>56</td>
<td>70</td>
<td>77</td>
</tr>
<tr>
<td>Forest (Evergreen)</td>
<td>0</td>
<td>30</td>
<td>54</td>
<td>66</td>
<td>75</td>
</tr>
<tr>
<td>Outcrops</td>
<td>70</td>
<td>77</td>
<td>86</td>
<td>91</td>
<td>94</td>
</tr>
<tr>
<td><strong>Street/Roads:</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
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</tr>
<tr>
<td>Paved</td>
<td>100</td>
<td>98</td>
<td>98</td>
<td>98</td>
<td>98</td>
</tr>
<tr>
<td>Gravel</td>
<td>20</td>
<td>76</td>
<td>85</td>
<td>89</td>
<td>91</td>
</tr>
<tr>
<td><strong>Drives/Walks:</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Roofs</td>
<td>95</td>
<td>97</td>
<td>97</td>
<td>97</td>
<td>97</td>
</tr>
</tbody>
</table>

Notes:

1. Grass - Grossed Landscaping or Irrigated Vegetation

---

**Version:** December 2, 1996

**Reference:** SCS TR-55, USDA, June 1986 (with modifications)

**Table 702**

**WRC Engineering, Inc.**
# LAG EQUATION ROUGHNESS FACTORS

<table>
<thead>
<tr>
<th>LAND USE</th>
<th>RANGE OF AVERAGE IMPERVIOUS AREA</th>
<th>$K_n$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Developed Areas</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Commercial/Industrial/Office/Business</td>
<td>70 - 85</td>
<td>.05</td>
</tr>
<tr>
<td>High and Medium Density Residential</td>
<td>30 - 65</td>
<td>.05</td>
</tr>
<tr>
<td>Low Density Residential</td>
<td>20 - 25</td>
<td>.07</td>
</tr>
<tr>
<td>Rural Residential</td>
<td>10 - 15</td>
<td>.08</td>
</tr>
<tr>
<td>Irrigated Grass (Golf Course/Parks/Cemeteries)</td>
<td>0 - 5</td>
<td>.10</td>
</tr>
<tr>
<td>Undeveloped Areas</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Rock Outcroppings</td>
<td>-</td>
<td>.04</td>
</tr>
<tr>
<td>Irrigated Agriculture</td>
<td>-</td>
<td>.10</td>
</tr>
<tr>
<td>Rangelands:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Herbaceous (grasses)</td>
<td>-</td>
<td>.08</td>
</tr>
<tr>
<td>Mixed grass and shrub</td>
<td>-</td>
<td>.09</td>
</tr>
<tr>
<td>Heavy shrub/brush</td>
<td>-</td>
<td>.10</td>
</tr>
<tr>
<td>Forest (Evergreen)</td>
<td>-</td>
<td>.15</td>
</tr>
<tr>
<td>DESIGN</td>
<td>R</td>
<td>AREA</td>
</tr>
<tr>
<td>--------</td>
<td>-----</td>
<td>------</td>
</tr>
<tr>
<td>A</td>
<td>.45</td>
<td>2.32</td>
</tr>
<tr>
<td>B</td>
<td>.45</td>
<td>1.81</td>
</tr>
<tr>
<td>DP1</td>
<td></td>
<td></td>
</tr>
<tr>
<td>C</td>
<td>.45</td>
<td>1.81</td>
</tr>
<tr>
<td>DP2</td>
<td></td>
<td></td>
</tr>
<tr>
<td>D</td>
<td>.45</td>
<td>2.32</td>
</tr>
<tr>
<td>DP3</td>
<td></td>
<td></td>
</tr>
<tr>
<td>DP5</td>
<td>.45</td>
<td>3.36</td>
</tr>
<tr>
<td>G</td>
<td>.45</td>
<td>1.29</td>
</tr>
<tr>
<td>DP6</td>
<td></td>
<td></td>
</tr>
<tr>
<td>E</td>
<td>.45</td>
<td>1.81</td>
</tr>
<tr>
<td>DP4</td>
<td></td>
<td></td>
</tr>
<tr>
<td>H</td>
<td>.45</td>
<td>0.78</td>
</tr>
<tr>
<td>DP7</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

\[ tᵣ = 1.8 \left(1.1 - R\right) \frac{L^{1/2}}{S^{1/3}} \]
TRAVEL TIME VELOCITY

SLOPE IN PERCENT

VELOCITY IN FEET PER SECOND

VERSION: December 2, 1998
REFERENCE:
Soil Conservation Service, 1985 (Modified)

FIGURE 701
PERCENTAGE OF IMPERVIOUS AREA VS. COMPOSITE CN'S FOR GIVEN PERVIOUS AREA CN'S

NOTE:
1. REFER TO TABLE 701 FOR CN VALUES FOR VARIOUS LAND USE CONDITIONS.
EXAMPLE: HEC-1 INPUT AND OUTPUT FOR BASIN A

**HEC-1 INPUT**

```
LINE | ID ........ 1 ........ 2 ........ 3 ........ 4 ........ 5 ........ 6 ........ 7 ........ 8 ........ 9 ........ 10  
1    | ID ........ DOE CREEK  
2    | ID ........ WASHOE COUNTY DRAINAGE CRITERIA MANUAL EXAMPLE  
3    | ID ........ 24-HR. 100-YEAR EVENT  
4    | ID ........ WRC ENGINEERING, INC.  
5    | IT ........ 5 ........ 0 ........ 300  
6    | IO ........ 2  
7    | JR ........ PREC ........ .995  
8    | KK ........ A ........ BASIN A HYDROGRAPH  
9    | BA ........ 3.34  
10   | LS ........ 0 ........ 61  
11   | PH ........ 0.01 ........ .48 ........ .87 ........ 1.45 ........ 1.69 ........ 1.88 ........ 2.26 ........ 2.91 ........ 3.55  
12   | UD ........ 2.05  
13   | ZZ ........  
```

**HEC-1 OUTPUT**

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

<table>
<thead>
<tr>
<th>OPERATION</th>
<th>STATION</th>
<th>AREA</th>
<th>PLAN</th>
<th>RATIO 1</th>
</tr>
</thead>
<tbody>
<tr>
<td>HYDROGRAPH AT +</td>
<td>A</td>
<td>3.34</td>
<td>1</td>
<td>FLOW 191</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>TIME 14.58</td>
</tr>
</tbody>
</table>

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

**VERSION:** December 2, 1996  
**REFERENCE:**  
**WRC ENGINEERING, INC.**
SECTION 800
OPEN CHANNELS

TABLE OF CONTENTS

801 INTRODUCTION .......................... 805

802 OPEN-CHANNEL HYdraulics ............. 805
   802.1 Uniform Flow ...................... 805
   802.2 Uniform Critical Flow Analysis ... 806
   802.3 Gradually Varying Flow .......... 807
   802.4 Rapidly Varying Flow ............ 808
   802.5 Transitions ...................... 808

803 CHANNEL SELECTION .................... 809
   803.1 Channel Types .................... 809
      803.1.1 Natural Channels ............. 809
      803.1.2 Grass-lined Channels .......... 809
      803.1.3 Wetland Bottom Channels ...... 809
      803.1.4 Concrete-lined Channels ..... 809
      803.1.5 Riprap-lined Channels ....... 809
      803.1.6 Other Channel Liners ........ 809
   803.2 Channel Selection ................. 810
      803.2.1 Hydraulic Factors .......... 810
      803.2.2 Structural Factors ............ 810
      803.2.3 Environmental Factors ........ 810
      803.2.4 Sociological Factors .......... 810
      803.2.5 Maintenance Factors ......... 811
      803.2.6 Regulatory Factors .......... 811
   803.3 Maximum Permissible Velocities ... 811

804 NATURAL CHANNEL DESIGN .............. 811
   804.1 Introduction ..................... 811
      804.1.1 Natural Unencroached Channels 812
      804.1.2 Natural Encroached Channels ... 812
      804.1.3 Bank-lined Channels ........... 812
      804.1.4 Partially Lined Channels ...... 813
   804.2 Natural Channel Systems ........... 813
   804.3 Floodplain Management of Natural Channels 814
IMPROVED CHANNEL DESIGN

805.1 - Introduction
805.2 - Permanent Unlined Channels
805.3 - Non-reinforced Grass-lined Channels

805.3.1 - Design Parameters
  805.3.1.1 - Longitudinal Channel Slopes
  805.3.1.2 - Roughness Coefficient
  805.3.1.3 - Low Flow and Trickle Channels
  805.3.1.4 - Bottom Width
  805.3.1.5 - Flow Depth
  805.3.1.6 - Side Slopes
  805.3.1.7 - Grass Lining
  805.3.1.8 - Establishing Vegetation

805.3.2 - Channel Bend Protection

805.4 - Wetland Bottom Channel

805.4.1 - Design Parameters
  805.4.1.1 - Longitudinal Channel Slope
  805.4.1.2 - Roughness Coefficients
  805.4.1.3 - Low-Flow Channel
  805.4.1.4 - Bottom Width
  805.4.1.5 - Flow Depth
  805.4.1.6 - Side Slopes
  805.4.1.7 - Grass Lining

805.4.2 - Channel Bend Protection
805.4.3 - Channel Crossing
805.4.4 - Life Expectancy

805.5 - Riprap-lined Channels

805.5.1 - Design Parameters
  805.5.1.1 - Longitudinal Channel Slope
  805.5.1.2 - Roughness Coefficients
  805.5.1.3 - Low Flow Channel
  805.5.1.4 - Bottom Width
  805.5.1.5 - Flow Depth
  805.5.1.6 - Side Slopes
  805.5.1.7 - Toe Protection
  805.5.1.8 - Beginning and End of Riprap-lined Channel
  805.5.1.9 - Loose Riprap Lining
  805.5.1.10 - Grouted Riprap Lining

805.5.2 - Channel Bend Protection
805.5.3 - Transition Protection
805.5.4 - Concrete Bend Protection
805.5.5 - Riprap-lined Channels on Steep Slopes

805.5.5.1 - Introduction
805.5.5.2 - Rock Size
805.5.5.3 - Riprap Gradation for Steep Slopes
805.5.5.4 - Riprap Thickness for Steep Slopes
805.5.5.5 - Riprap Placement on Steep Slopes
805.5.5.6 - Freeboard
805.5.5.7 - Bedding Requirements on Steep Slopes
805.6 - Concrete-lined Channels
   805.6.1 - Design Parameters
   805.6.1.1 - Longitudinal Channel Slope
   805.6.1.2 - Roughness Coefficients
   805.6.1.3 - Low Flow Channel
   805.6.1.4 - Bottom Width
   805.6.1.5 - Flow Depth
   805.6.1.6 - Side Slopes
   805.6.2 - Concrete Lining Section
   805.6.2.1 - Thickness
   805.6.2.2 - Concrete Joints
   805.6.2.3 - Concrete Finish
   805.6.2.4 - Concrete Curing
   805.6.2.5 - Reinforcement Steel
   805.6.2.6 - Earthwork
   805.6.2.7 - Bedding
   805.6.2.8 - Underdrain and Weepholes
   805.6.2.9 - Concrete Cutoffs
   805.6.3 - Special Consideration for Supercritical Flow
   805.7 - Other Channel Linings

806 ADDITIONAL HYDRAULIC DESIGN STANDARDS
   806.1 - Subcritical Flow Design Standards
   806.1.1 - Transitions
   806.1.1.1 - Transition Energy Loss
   806.1.1.2 - Transition Length
   806.1.2 - Superelevation in Bends
   806.1.3 - Freeboard

806.2 - Supercritical Flow Design Standards
   806.2.1 - Super Critical Transitions
   .806.2.1.1 - Contracting Transitions
   .806.2.1.2 - Expanding Transitions
   806.2.2 - Superelevation in Bends
   806.2.3 - Circular Transition Curves
   806.2.4 - Freeboard
   806.2.5 - Slug Flow

807 CHANNEL APPURTENANCES
   807.1 - Maintenance Access Road
   807.2 - Safety Requirements
   807.3 - Culvert Outlet Protection
   807.3.1 - Basin Configuration
   807.3.2 - Rock Size
   807.4 - Grade Control Structures
   807.4.1 - Introduction
   807.4.2 - Drop Structure Grade Control Structures
   807.4.3 - Control Sill Grade Control Structures

December 2, 1996
LIST OF TABLES

- GEOMETRIC ELEMENTS OF CHANNEL SECTIONS
- TYPICAL ROUGHNESS COEFFICIENTS FOR OPEN CHANNELS
- MAXIMUM PERMISSIBLE MEAN CHANNEL VELOCITIES
- CLASSIFICATION AND GRADATION OF LOOSE RIPRAP
- GRADATION FOR GRANULAR BEDDING
- CLASSIFICATION AND GRADATION OF ROCK FOR GROUTED RIPRAP
- DESIGN D_{50} VALUES FOR STEEP CHANNELS
- RIPRAP GRADATION FOR STEEP SLOPES

LIST OF FIGURES

- OPEN-CHANNEL FLOW CONDITIONS
- CRITICAL DEPTH FOR TRAPEZOIDAL AND CIRCULAR SECTIONS
- TYPICAL OPEN-CHANNEL DESIGN SECTIONS (NATURAL CHANNELS)
- TYPICAL OPEN-CHANNEL DESIGN SECTIONS (IMPROVED CHANNELS)
- ROUGHNESS COEFFICIENT FOR GRASS-LINED CHANNELS
- TYPICAL CROSS-SECTION OF CONCRETE-LINED TRICKLE CHANNEL
- TYPICAL CROSS-SECTION OF RIPRAP-LINED TRICKLE CHANNEL
- TYPICAL CROSS-SECTION OF LOW-FLOW CHANNEL
- MANNING'S ROUGHNESS COEFFICIENT FOR WETLAND BOTTOM
- TYPICAL CROSS-SECTIONS FOR RIPRAP-LINED CHANNELS
- TYPICAL CROSS-SECTION FOR GROUTED RIPRAP LINING
- STEEP SLOPE RIPRAP DESIGN, TRIANGULAR CHANNEL, 2:1 SIDE SLOPES

December 2, 1996
813 STEEP SLOPE RIPRAP DESIGN, TRAPEZOIDAL CHANNEL, 2:1 SIDE SLOPES, 6-FOOT BOTTOM WIDTH
814 STEEP SLOPE RIPRAP DESIGN, TRAPEZOIDAL CHANNEL, 2:1 SIDE SLOPES, 10-FOOT BOTTOM WIDTH
815 STEEP SLOPE RIPRAP DESIGN, TRAPEZOIDAL CHANNEL, 2:1 SIDE SLOPES, 14-FOOT BOTTOM WIDTH
816 STEEP SLOPE RIPRAP DESIGN, TRAPEZOIDAL CHANNEL, 2:1 SIDE SLOPES, 20-FOOT BOTTOM WIDTH
817 CHANNEL TRANSITION TYPES
818 TYPICAL CHANNEL TRANSITION SECTIONS AND ENERGY LOSS COEFFICIENTS
819 TYPICAL CONTRACTING TRANSITION FOR SUPERCRITICAL FLOW
820 DESIGN CHART FOR CONTRACTING TRANSITION FOR SUPERCRITICAL FLOW
821 CONFIGURATION OF CULVERT OUTLET PROTECTION
822 PREFORMED SCOUR HOLE
823 CONTROL SILL GRADE CONTROL STRUCTURE
824 EXAMPLE: CROSS-SECTION OF DOE CREEK
SECTION 800
OPEN CHANNELS

801 INTRODUCTION

Presented in this section is the technical criteria and design standards for the hydraulic evaluation and design of open channels. Discussion and standards are provided for the various channel linings and design sections anticipated to be encountered or used in Washoe County. Since the design of channel sections depends on site conditions, the "best" channel design can vary significantly within Washoe County. The ultimate responsibility for a safe and stable channel design rests with the designer. The information presented in this section should be considered to be the minimum standards upon which channel evaluation and design should be based. Additional analysis beyond the scope of this MANUAL may be necessary for unique or unusual channel conditions. In addition, Washoe County may require submittal of additional design and analysis information for any of the proposed channel sections and linings in order to assess the adequacy of the design for the proposed application. Therefore, the designer is recommended to contact Washoe County prior to design of an open channel to discuss additional requirements (if any) for the selected channel. If the designer is proposing a different channel design than presented in this section, Washoe County must be contacted prior to designing the channel.

802 OPEN-CHANNEL HYDRAULICS

An open channel is a conduit in which water flows with a free surface (non-pressurized flow). The hydraulics of an open channel can be very complex, encompassing many different flow conditions from steady-state uniform flow to unsteady, rapidly varying flow. Most of the problems in storm water drainage involve uniform, gradually varying or rapidly varying flow states. An example of these flow conditions is illustrated in Figure 801. The calculations for uniform and gradually varying flow are relatively straightforward and are based upon similar assumptions (i.e., parallel streamlines). Rapidly varying flow computations (i.e., hydraulic jumps and flow over spillways), however, can be very complex, and the solutions are generally empirical in nature.

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

802.1 UNIFORM FLOW

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

Uniform flow rarely occurs in nature and is difficult to achieve in a laboratory, because not all of the parameters remain exactly the same. However, channels are designed assuming uniform flow as an approximation, which is adequate for planning and design purposes.
The computation of uniform flow and normal depth shall be based upon the Manning formula as follows:

\[ Q = \left( \frac{1.49}{n} \right) R^{2/3} S^{1/6} A \]  

(801)

Where

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

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

Presented in Table 801 are equations for calculating many of the parameters required for hydraulic analysis of different channel sections. Table 802 provides a list of Manning roughness coefficient values for many types of conditions that may occur in Washoe County. These parameters and the Manning equation may also be readily computed using hand-held calculators and personal computers.

### 802.2 UNIFORM CRITICAL FLOW ANALYSIS

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

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

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

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

\[ F_r = \frac{V}{(gD)^{0.5}} \]  

(802)
The Froude Number for a given channel section and flow can be easily computed using the above equation. The critical depth in a given trapezoidal channel section with a known flow rate can be determined using the following methodology. First, the section factor, Z, is computed.

\[ Z = \frac{Q}{g^{0.5}} \]  

\[ (803) \]

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

Utilizing values for \( Z \), the channel bottom width, \( b \), and the side slope, \( z \), the critical depth in the channel, \( y \), can be determined from Figure 802. For other prismatic channel shapes, Equation 803 above can be used with the section factors provided in Table 801 to determine the critical depth.

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

<table>
<thead>
<tr>
<th>Flow Condition</th>
<th>Froude Number ( (F_r) )</th>
<th>Flow Depth</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sub-Critical</td>
<td>&lt;0.8</td>
<td>&gt;1.1( d_c )</td>
</tr>
<tr>
<td>Super-Critical</td>
<td>&gt;1.13</td>
<td>&lt;0.9( d_c )</td>
</tr>
</tbody>
</table>

where \( d_c \) = critical depth

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

**802.3 GRADUALLY VARYING FLOW**

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

Backwater computations can be made using the methods presented in Chow, 1959. Many computer programs are available for computation of backwater curves. The most general and widely used program is U.S. Army Corps of Engineers' HEC-2, Water-Surface Profiles, 1990. This program is
recommended for floodwater profile computations for channel and floodplain analyses within Washoe County.

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

802.4 RAPIDLY VARYING FLOW

Rapidly varying flow is characterized by very pronounced curvature of the flow streamlines. The change in curvature may become so abrupt that the flow profile is virtually broken, resulting in a state of high turbulence. There are mathematical solutions to some specific cases of rapidly varying flow, but empirical solutions are generally relied on for most rapidly varying flow problems. The most common occurrence of rapidly varying flow in storm drainage applications involves weirs, orifices, hydraulic jumps, non-prismatic channel sections (transitions, culverts and bridges), and non-linear channel alignments (bends). Discussions of rapidly varying flow for these applications are presented in this MANUAL as follows:

<table>
<thead>
<tr>
<th>Application</th>
<th>Section</th>
</tr>
</thead>
<tbody>
<tr>
<td>Weirs and Orifices</td>
<td>1300 - Detention</td>
</tr>
<tr>
<td>Hydraulic Jumps (Channels)</td>
<td>1200 - Additional Hydraulic Structures</td>
</tr>
<tr>
<td>Hydraulic Jumps (Conduits)</td>
<td>900 - Storm Sewer Systems</td>
</tr>
<tr>
<td>Culverts and Bridges</td>
<td>1100 - Culverts and Bridges</td>
</tr>
<tr>
<td>Channel Transitions &amp; Bends</td>
<td>800 - Open Channels</td>
</tr>
</tbody>
</table>

Each of these flow conditions require extensive and detailed calculations to properly identify the flow capacities and depths of flow in the given section. The designer should be cognizant of the design requirements for each of the above conditions and must include all necessary calculations as part of the design submittal documents. The designer is referred to the many hydraulic references for the proper calculation methods to use in the design of rapidly varying flow facilities.

802.5 TRANSITIONS

Channel transitions occur in open channel design whenever there is a change in channel slope or shape and at junctions with other open channels or storm sewers. The goal of a good transition design is to minimize the loss of energy as well as minimize surface disturbances from cross-waves and turbulence. A special case of transitions where excess energy is dissipated by design are drop structures and hydraulic jumps. Channel drop structures are discussed in Section 1200 (Additional Hydraulic Structures).

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

1. Sub-critical flow to sub-critical flow.
2. Sub-critical flow to super-critical flow.

For definition purposes, Conditions 1 and 2 will be considered as sub-critical transitions and are discussed in Section 806.1. Conditions 3 and 4 will be considered as super-critical transitions and are discussed in Section 806.2.

803 CHANNEL SELECTION

803.1 CHANNEL TYPES

Essentially, open channels can be separated into 6 different categories:

803.1.1 Natural Channels - Drainageways which are carved or shaped by nature before urbanization occurs. They often, but not always, have mild slopes and are reasonably stable. As the channel's tributary watershed urbanizes, natural channels often experience erosion and may need grade control checks and localized bank protection to stabilize.

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

803.1.3 Wetland 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. In low-flow areas the banks need rock lining to protect against undermining.

803.1.4 Concrete-lined Channels - Concrete-lined channels are high velocity artificial drainageways that are not encouraged in urban areas. 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.

803.1.5 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 and maintenance attention as compared to grass-lined channels and avoid the higher costs of concrete lined channels.

803.1.6 Other Channel Liners - 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.
803.2 CHANNEL SELECTION

The actual selection must be based upon a variety of multi-disciplinary factors which include:

803.2.1 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

803.2.2 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

803.2.3 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

803.2.4 Sociological Factors

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

803.2.5 Maintenance Factors

1) Life expectancy
2) Repair and reconstruction needs
3) Maintainability
4) Proven performance
5) Accessibility
803.2.6 Regulatory Factors

1) Federal Regulations
2) State Regulations
3) Local Regulations

803.3 MAXIMUM PERMISSIBLE VELOCITIES

The design of all channels in Washoe County shall be based on maximum permissible velocities. This method of design assumes that the given channel section will remain stable up to the stated maximum permissible velocity provided that the channel is designed in accordance with the provisions of this MANUAL. Presented in Table 803 are the maximum permissible velocities for natural or improved, unlined and lined channels in Washoe County. These values shall be used for all channel designs in Washoe County. If a higher velocity is desired, the design engineer must demonstrate to the satisfaction of the local entity and/or Washoe County that the higher velocity would not endanger the health or safety of the public and would not increase maintenance of the channel section. For natural and improved unlined channels, a geotechnical report shall be submitted to the local entity and/or Washoe County upon which the existing or proposed soil material classification used for the maximum permissible velocity determination was selected. Additional analysis may be required for natural channels or improved unlined channels to verify that the channel will remain stable based on the stated maximum permissible velocities.

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

804 NATURAL CHANNEL DESIGN

804.1 INTRODUCTION

Presented in this section are the typical natural open channel sections which are encountered in the Washoe County area. A graphical illustration of the typical design sections is presented in Figure 803. The selection of a design section for a natural channel is generally dependent on the value of developable land versus the cost to remove said land from a floodplain. The costs for the removal depend on the rate of flow, slope, alignment, and depth of the channel as well as material and fill costs for construction of the encroachment. The design sections discussed herein vary from no encroachment to the level of encroachment at which point an improved channel (unlined or lined) becomes more economical or is required to adequately protect the proposed development.

The design standards presented in this section are the minimum standards by which natural channel analysis and design shall be completed within Washoe County. The channel designer is reminded that the ultimate responsibility for a safe and stable channel design lies solely with the engineer responsible for the design. Thus, the execution of this responsibility may require additional analysis and stricter standards than are presented in this section. In addition, Washoe County may require additional design
analysis be performed to verify the suitability of the proposed design for the location under consideration.

For natural channel sections, the engineer shall 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 Washoe County to determine: a) what additional analysis shall be prepared to estimate the potential extent of lateral and vertical channel movement, b) what is the potential risk to the proposed development from channel degradation and/or bank failure, c) what solutions and/or remedies are available which can mitigate the potential risk to the proposed development, and d) what improvements and/or reduction in encroachment in or adjacent to the subject channel will be required to allow approval of the subject development.

804.1.1 Natural Unencroached Channels

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

804.1.2 Natural Encroached Channels

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

804.1.3 Bank-lined Channels

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

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

Partially lined channels will only be allowed if:

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

b) Erosion in the unlined section is addressed to the satisfaction of Washoe County.

c) Scour below the lining is addressed to the satisfaction of Washoe County.

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

804.2 NATURAL CHANNEL SYSTEMS

A natural channel system generally is continually changing its position and shape as a consequence to hydraulic forces acting on its bed and banks and related biological forces interacting with these physical forces. These changes may be slow or rapid and may result from natural environmental changes or from changes caused by 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 there of; (7) geology, including type of sediment; and (8) constructed improvements.

804.3 FLOODPLAIN MANAGEMENT OF NATURAL CHANNELS

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

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

2. Natural channel segments which have a calculated flow velocity greater than the allowable flow velocity determined herein shall be analyzed for erosion potential 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 floodplain can be delineated.

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

5. Roughness factors which are representative of unmaintained conditions shall be used for the analysis of water surface profiles.

6. Erosion control structures, such as drop structures (Section 1200) or check structures (Section 807), may be required to control flow velocities for both the minor storm and major storm events.

7. A general plan and profile (i.e., HEC-2 output) of the floodplain shall be prepared which includes appropriate allowances for known future bridges or culverts which will increase the water surface profile and cause the floodplain to be larger.

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

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

The usual rules of freeboard depth, curvature, and other rules which are applicable to artificial channels do not apply for natural channels. There are significant advantages which 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 maintained or encroached upon for a development, then the applicant shall meet with Washoe County (if applicable) to discuss the concept and to obtain the requirements for planning and design analysis and documentation.
805 IMPROVED CHANNEL DESIGN

805.1 INTRODUCTION

Presented in this section are the typical improved channel design sections which may be used in Washoe County. A graphical illustration of the typical design sections is presented in Figure 804. The selection of a channel section and lining is generally dependent on physical and economic channel restrictions (i.e. value of developable land), the slope of the proposed channel alignment, the rate of flow to be conveyed by the channel, and the comparative costs of the lining materials. The channel sections and linings discussed herein provide a range of options from which an appropriate channel may be selected. Specific hydraulic design standards which are applicable to all improved channels (i.e. transition, freeboard, etc.) are presented in Section 806.

The design standards presented in this section are the minimum standards by which channel design shall be completed within Washoe County. The channel designer is reminded that the ultimate responsibility for a safe and stable channel design lies solely with the engineer responsible for the design. Thus, the execution of this responsibility may require additional analysis and stricter standards than are presented in this section. In addition, Washoe County may require additional design analysis be performed to verify the suitability of the proposed design for the location under consideration.

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

805.2 PERMANENT UNLINED CHANNELS

Permanent unlined channels are improved channels which are constructed to the shape of vegetation-lined channels but are not revegetated. The cost of construction of these channels is relatively low for areas with flat slopes and where the design flow rates and velocities are small. The designer must adequately address potential erosion problem areas (i.e. bends, transitions, structures) as well as the overall stability of the unlined channel and the effect that possible future natural revegetation may have on the channel hydraulics.

The stability of the channel shall be analyzed as if the channel was a natural channel using the design standards in Section 804. In addition, the layout, alignment, and cross-section of the channel shall be designed as if the channel was to be revegetated using the design standards in Section 805.3.

805.3 NON-REINFORCED GRASS-LINED CHANNELS

Grass-lined channels may be considered to be the most desirable artificial channels from an esthetics viewpoint. The channel storage, lower velocities, and the sociological benefits create significant advantages over other types of channels. The designer must give full consideration to flow hydraulics for which calculations shall be submitted for review to Washoe County.

The satisfactory performance of a grass-lined channel depends on constructing the channel with the proper shape and preparing the area in a manner to provide conditions favorable to vegetative growth. Between the time of seeding and the actual establishment of the grass, the channel is unprotected and subject to considerable damage unless special protection is provided. Channels subject to constant or prolonged flows require special supplemental treatment, such as grade control structures, stone centers,
ii) Riprap Trickle Channel: The riprap trickle channel shall have a minimum bottom width of 5 feet and a minimum depth of 2 feet. Manning’s roughness coefficient will be determined by Equation 805. Figure 807 is a typical cross-section of a riprap trickle channel.

b) Low Flow Channels

Low-flow channels will be used in channels with a 100-year flow greater than 100 cfs. The low-flow channel will have the capacity to carry the 5-year flow event with no freeboard. Low-flow channels are used to contain relatively frequently occurring flows within a recognizable channel section. The flow capacity of the main channel should include the flow in the low flow channel. Figure 808 illustrates an example of a low-flow channel.

Low-flow channels shall have a minimum bottom width of 8 feet and a depth between 1 foot and 4 feet. The riprap-lined side slopes of the low-flow channel will be 2.5:1 to 3:1. The main channel depth limitation does not apply to the low-flow channel area of the total channel cross-section.

805.3.1.4 Bottom Width
The minimum channel bottom width shall be 5 feet in 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.

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

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

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

805.3.1.8 Establishing Vegetation
Channel vegetation is usually established by seeding. In the more critical sections of some channels, it may be desirable to provide immediate protection by transplanting a complete sod cover.
Jute, plastic, or paper mesh and straw or hay mulch may be used to protect the entire width and side slopes of a waterway until the vegetation becomes established. All seeding, planting, and sodding should conform to local agronomic recommendations.

**805.3.2 Channel Bend Protection**

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

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

**805.4 WETLAND BOTTOM CHANNEL**

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

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

The down side of this practice is that the channel bottom is "boggy" and can become overgrown. As a result, it is impossible to mow the bottom grasses and very difficult to control the density of vegetation. This more abundant bottom vegetation traps sediments, thereby reducing channel flood carrying capacity as the bottom fills with sediments. Eventually, depending on the sediment loads being carried by the flows, the channel bottom will have to be dredged to restore its flood carrying capacity. Wetland bottom channels can provide habitat for mosquito breeding, and because the abundant vegetation can dislodge during a flood, an increased potential exists for blockage of drainageway crossing structures.

Since wetland bottoms will decrease flow conveyance and accelerate channel bottom aggradation, the channel cross-section needs to be enlarged for flood conveyance. As a result, more right-of-way will be needed than required for a well groomed grass-lined channel. In areas where urbanization has already taken place, wetland bottom channels may not be feasible. Where right-of-way is limited, mitigating flood damages should take precedence over other considerations during project design. In cases when existing wetlands are eliminated or reduced, off-site wetland mitigation may be required by the Corps of Engineers’ 404 Permit. The design of channels with wetland bottoms can be a
complicated, iterative process. In order to simplify the design procedure for this manual, assumptions have been made concerning how the flow depth in a channel interacts with the wetland vegetation and affects the channel roughness and the rate of sediment deposition on the bottom.

805.4.1 Design Parameters

805.4.1.1 Longitudinal Channel Slope

The longitudinal channel slope will be set so the maximum permissible velocity criteria provided in Table 803 is not violated. To prevent channel degradation, the channel slope should be determined assuming there is no wetland vegetation on the bottom (i.e., "New Channel"). In addition to the velocity requirements, the Froude Number for the New Channel condition shall be less than 0.7.

805.4.1.2 Roughness Coefficients

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

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

\[
n_c = \left(\frac{n_c^2 P_o + n_w^2 P_w}{P_o + P_w}\right)^{0.5}
\]

where

\( n_c \) = Manning's roughness coefficient for the composite channel (Dimensionless)

\( n_b \) = Manning's roughness coefficient for areas above the wetland area (Dimensionless)

\( n_w \) = Manning's roughness coefficient for the wetland area (Dimensionless)

\( P_o \) = Wetland perimeter of channel cross-section above the wetland area (feet)

\( P_w \) = Wetland perimeter of the wetland channel bottom (feet)

For grass-lined areas above the wetland area, use a Manning's roughness coefficient, \( n_w \), of 0.035. Manning's roughness coefficients for the wetland area (\( N_w \)) are supplied by Figure 809.

805.4.1.3 Low-Flow Channel

Trickle channels are not permitted in wetland bottom channels. Low-flow channels are used when the 100-year flow exceeds 1,000 cfs. The design of the low flow channel is according to Section 805.3.1.3.
Bottom Width

The minimum bottom width will be designed according to Section 805.3.1.4.

Flow Depth

The maximum flow depth shall be determined according to Section 805.3.1.5.

Side Slopes

The side slopes shall be designed according to Section 805.3.1.6.

Grass Lining

The side slopes shall be grass-lined according to Sections 805.3.1.7 and 805.3.1.8.

Channel Bend Protection

Channel bends shall be designed according to the criteria discussed in Section 805.3.2.

Channel Crossings

Whenever a wetland bottom channel is crossed by a road, railroad or a trail requiring a culvert or a bridge, a drop structure should be provided immediately downstream of such a crossing. This will help reduce the silting-in of the crossing with sediments. A 1-foot to 2-foot drop is recommended (the larger drop preferred on the downstream side of each culvert and crossing of a wetland bottom channel.

Life Expectancy

Wetland vegetation bottom channels are expected to fill with sediment over time. This occurs because the bottom vegetation traps some of the sediments carried by the flow. The life expectancy of such a channel will depend primarily on the land use of the tributary watershed and could range anywhere from 20 to 40 years before major channel dredging is needed. However, life expectancy can be dramatically reduced, to a 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.

RIPRAP-LINED CHANNELS

Riprap-lined channels are defined as channels in which riprap is used for lining of the channel banks and the channel bottom, if required. Riprap used for erosion protection at transitions and bends is also considered as a riprap-lined channel and those portions shall be designed in accordance with the riprap-lined channel and transition design standards. The design standards presented in this section are the minimum hydraulic design parameters.
Riprap has proven to be an effective means to deter erosion along channel banks, in channel beds, upstream and downstream from hydraulic structures, at bends, at bridges, and in other areas where erosive tendencies exist. Riprap is a popular choice for erosion protection because the initial installation costs are often less than alternative methods for preventing erosion. However, the designer needs to bear in mind that there are additional costs associated with riprap erosion protection since riprap installations require periodic inspection and maintenance.

Channel linings constructed from loose riprap or grooved riprap to control channel erosion have been found to be cost effective where channel reaches are relatively short (less than ¼ mile). Situations for which riprap lining might be appropriate are: 1) where major flows, such as the 100-year flood are found to produce channel velocities in excess of allowable non-eroding values; 2) where channels side slopes must be steeper than 3:1; 3) for low flow channels, and 4) where rapid changes in channel geometry occur such as channel bends and transitions. Design criteria applicable to these situations are presented in the following sections.

805.5.1 Design Parameters

805.5.1.1 Longitudinal Channel Slope

Riprap-lined channel slopes are dictated by the maximum permissible velocity requirements (Table 803). Where topography is steeper than desirable, drop structures (Section 1200) shall be utilized to maintain design velocities.

805.5.1.2 Roughness Coefficients

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

\[ n = 0.0395 \left( \frac{d_{50}}{10} \right)^{1.06} \]  

(805)

where

\[ d_{50} = \text{mean stone size (feet)} \]

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.

805.5.1.3 Low Flow Channel

The design of the low-flow channel is discussed in Section 805.3.1.3.

805.5.1.4 Bottom Width

The minimum channel bottom width for a riprap-lined channel should be designed according to Section 805.3.1.4.
805.5.1.5 Flow Depth

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

805.5.1.6 Side Slopes

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

805.5.1.7 Toe Protection

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

805.5.1.8 Beginning and End of Riprap-Lined Channel

At the upstream and downstream termination of a riprap lining, the thickness should be increased 50 percent for at least 3 feet to prevent undercutting.

805.5.1.9 Loose Riprap Lining

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

Many factors govern the size of the rock necessary to resist the 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.

a) Riprap Material

Rock used for loose riprap, grouted riprap, or wire enclosed riprap should be hard, durable, angular in shape, and free from cracks, overburden, shale and organic matter. Neither breadth nor thickness of a single stone should be less than ½ its length and rounded stone should be avoided. Rock having minimum specific gravity
or 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 804 and are based on a minimum specific gravity of 2.50 for the rock. Because of its relative small size and weight, riprap Class 150 must be buried with native topsoil and revegetated to protect the rock from vandalism.

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

\[
d_{50} = \frac{0.05 \times V^2 \times S^{0.34}}{(S_s - 1)^{1.332}}
\]  

(806)

where

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

The riprap blanket thickness should be at least 2.0 times \(d_{50}\) and should extend up the side slopes to an elevation of the design water surface plus the calculated freeboard (Section 806.1.3) and superelevation (Section 806.1.2).

b) Bedding Requirements

Long term stability of riprap erosion protection is strongly influenced by proper bedding conditions. A large percentage of all riprap failures is directly attributable to bedding failures.

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

1) Granular Bedding - Generic Design

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

A single 12-inch layer of said granular bedding can be used except at drop structures. At drop structures, filter fabric must be added below the 12-inch layer of granular bedding.
2) Granular Bedding - T-V Design

The T-V (Terzughi-Vicksburg) design establishes an optimum granular bedding gradation for a specific channel soil. Since this method designs the granular bedding for a particular soil, the allowable granular bedding thickness may be much less than the generic design.

The specifications for the T-V reverse filter method relate the gradation of the protective layer (filter) to that of the bed material (base) by the following inequalities:

\[ D_{15(\text{filter})} \leq 5d_{15(\text{base})} \]  \hspace{1cm} (807)

\[ 4d_{15(\text{base})} \leq D_{15(\text{filter})} \leq 20d_{15(\text{base})} \]  \hspace{1cm} (808)

\[ D_{50(\text{filter})} < 25d_{50(\text{base})} \]  \hspace{1cm} (809)

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 \( D_{15(\text{filter})} \) and 85 percent of the base material is finer than \( d_{50(\text{base})} \).

When the T-V method is used, the thickness of the resulting layer of granular bedding may be reduced to six inches. However, if a gradation analysis of the existing soils shows that a single layer of T-V Method designed granular bedding can not bridge the gap between the riprap specification and the existing soils, then two or more layers of granular bedding shall be used. The design of the bedding layer closest to the existing soils shall be based on the existing soil gradation. The design of the upper bedding layer shall be based on the gradation of the lower bedding layer. The thickness of each of the two or more layers shall be four inches.

3) Filter Fabrics

Filter fabric is not a complete substitute for granular bedding. Filter fabric provides filtering action only perpendicular to the fabric and has only a single equivalent pore opening between the channel bed and the riprap. Filter fabric has a relatively smooth surface which provides less resistance to stone movement. As a result, it is recommended that the use of filter fabric in place of granular bedding be restricted to slopes no steeper than 2.5 horizontal to 1 vertical, and that such filter fabric only replace the bottom layer in a multi-layer T-V Method granular bedding design. The granular bedding shall be placed on top of the filter fabric to act as a cushion when placing the riprap. Tears in the fabric greatly reduce its effectiveness so that direct dumping of riprap on the filter fabric is not allowed and due care must be exercised during construction. Nonetheless, filter fabric has proven to be an adequate replacement for granular bedding in many instances. Filter fabric provides an adequate bedding for channel linings along uniform mild
sloping channels where leaching forces are primarily perpendicular to the fabric.

At drop structures and sloped channel drops, where seepage forces may run parallel with the fabric and cause piping along the bottom surface of the fabric, special care is required in the use of filter fabric. Seepage parallel with the fabric may be reduced by folding the edge of the fabric vertically downward about 2 feet (similar to a cutoff wall) at 12-foot intervals along the installation, particularly at the entrance and exit of the channel reach. Filter fabric has to be lapped a minimum of 12 inches at roll edges with upstream fabric being placed on top of downstream fabric at the lap.

Fine silt and clay has been found to clog the openings in filter fabric. This prevents free drainage which increases failure potential due to uplift. For this reason, a granular filter is often a more appropriate bedding for fine silt and clay channel beds.

805.5.1.10 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.

a) Riprap Material

The rock used for grouted riprap is different from the standard gradation of riprap in that the smaller rock has been reduced to allow greater penetration by the grout. The riprap specifications are shown on Table 806. Riprap smaller than Class 400 should not be grouted.

b) Bedding Material

The bedding material will be the same as for loose riprap.

c) Cutoff Trench

As the riprap layer is placed, a cutoff trench should be excavated around the rock section at the top of the slope and at the upstream and downstream edges. The trench should be the full depth of the riprap layer and at least 1-foot thick. This trench is filled with grout to prevent water from undermining the grouted rock mass.

d) Grout

After the riprap has been placed to the required thickness and the trench excavated, the rock is sprayed with clean water which cleans the rock and allows better adherence by the grout. The rock is then grouted using a low pressure (less than 10 psi) grout
pump with a 2" maximum diameter hose. Using a low pressure grout pump allows the work crew time to move the hose and vibrate the grout. Vibrating the grout with a pencil vibrator assures complete penetration and filling of the voids. After the grout has been placed and vibrated, a small hand broom or gloved hand is used to smooth the grout and remove any excess grout from the rock. The finished surface is sealed with a curing compound.

The grout should consist of 6 sacks (564 pounds) of cement per cubic yard, and the aggregate should consist of 30% of %-inch coarse gravel and 70% natural sand. The grout should contain 7.5% ± 1.5% air entrainment, have a 28-day compressive strength of at least 2,000 p.s.i., and have a slump of 7 inches ± 2 inches. Fiber reinforcement should be used such as 1.5 pounds per cubic yard of Fibermesh or an approved equivalent amount. A maximum of 25% flyash maybe substituted for the cementations material.

805.5.2 Channel Bend Protection

When riprap protection is required for a straight channel, increase the rock size by one category (e.g., Class 300 to Class 400) through bends. The minimum radius for a riprap-lined bend is 1.2 times the top width and in no case less than 50 feet. Riprap protection should extend downstream from the end of the bend a distance that is equal to the length of the bend measured along the channel centerline.

805.5.3 Transition Protection

Scour potential is amplified by turbulent eddies in the vicinity of rapid changes in channel geometry such at transitions and bridges. For these locations, the riprap lining thickness shall be increased by one size category.

Protection should extend upstream from the transition entrance at least 5 feet and extend downstream from the transition exit at least 10 feet. See Section 806 for further a discussion on transitions.

805.5.4 Concrete Cutoff Walls

Transverse concrete cutoff walls may be required by Washoe County for riprap lined channels where a resulting failure of the riprap lining could seriously affect the health and safety of the public. The designer shall consult with Washoe County prior to design of riprap lined-channels to determine if concrete cutoff walls are required as well as their sizing and spacing, if required.

805.5.5 Riprap-Lined Channels on Steep Slopes

805.5.5.1 Introduction

Achieving channel stability on steep slopes usually requires some type of channel lining. The only exception is a channel constructed in durable bedrock.
On mild slopes, the water velocity is slow enough and the depth of flow is large enough (relative to the riprap size) that a reasonable estimate of the resistance to flow can be made. On steep channels, the riprap size required to stabilize the channel is on the same order of magnitude or greater than the flow depth, which invalidates the Manning's relation. Since the resistance to flow is now unknown, an estimate of the velocity needed for the design of the riprap cannot be accurately estimated.

A graphically based methodology was developed for the U.S. Department of Interior, Office of Surface Mining Reclamation and Enforcement (SIMONS, 1989) to design riprap-lined channels on steep slopes (supercritical flow). This methodology was based on a study by BRUTHURST, 1979 that analyzed the hydraulics of mountain rivers where roughness elements are on the same order of magnitude as the depth of flow. Using the resistance equation developed by Bathurst, the velocity can be estimated for a given riprap size. The velocity is then used to predict the stability of the riprap.

This procedure shall be used for all riprap lined channels whose depth of flow is equal to or less than $d_{50}$ as computed initially using Equation 806.

805.5.2 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 should be given in 0.25-foot increments. The final minimum design size is determined using Table 807.

805.5.3 Riprap Gradation For Steep Slopes

Lack of proper riprap gradation is one of the most common causes of riprap failure. With the proper rock gradation, the voids formed by large stones are filled with smaller sizes in an interlocking fashion that prevents jets of water from contacting the underlying soil and ultimately eroding the soil supporting the riprap layer.

Table 808 provides ratios used to determine the $D_{10}$, $D_{20}$, and $D_{50}$ 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.
805.5.4 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.

805.5.5 Riprap Placement On Steep Slopes

Improper placement is another major cause of failure in riprap-lined channels. To prevent segregation of rock sizes, riprap should never be placed by dropping it down the slope or pushing it down with a bulldozer. Rock can be dumped directly from trucks from the top of the embankment, and draglines with orange peel buckets, backhoes, and other power equipment can also be used to place riprap with a minimum of handwork.

805.5.6 Freeboard

Figures 812 through 816 also provide the depth of flow for a given flow, channel slope, and channel dimensions. The required freeboard is given by Equation 821. The velocity can be estimated by dividing the flow rate by the area of flow.

805.5.7 Bedding Requirements on Steep Slopes

Either a granular bedding material or filter fabric may be used on steep slopes according to the requirements specified in Section 805.5.1.9.

805.6 CONCRETE-LINED CHANNELS

Concrete-lined channels are defined as rectangular or trapezoidal channels in which reinforced concrete is used for lining of the channel banks and channel bottom. The cost of concrete channels generally can be more economical than other lining types in an urban environment due to their greater flow carrying capacity resulting in less land area requirements.

805.6.1 Design Parameters

The following sections present design parameters for concrete-lined channels. The design parameters presented do not relieve the designer of performing appropriate engineering analyses.

805.6.1.1 Longitudinal Channel Slope

The maximum slope of concrete-lined channels is determined by the maximum permissible velocity requirements (Table 803). Concrete-lined channels have the ability to accommodate super-critical flow conditions and thus can be constructed to almost any naturally occurring slope.
WASHOE COUNTY
HYDROLOGIC CRITERIA AND DRAINAGE DESIGN MANUAL

805.6.1.2 Roughness Coefficients

The Manning's roughness coefficient for concrete-lined channels is as shown in Table 802. For concrete-lined channels with subcritical flow, check the Froude Number using a roughness coefficient of 0.011.

805.6.1.3 Low Flow Channel

The bottom of the concrete channel shall be constructed with a defined low flow channel but shall be adequately sloped to confine the low flows to the middle or one side of the channel. Low flows are defined in Section 805.3.1.3.

805.6.1.4 Bottom Width

There are no bottom width requirements for concrete-lined channels.

805.6.1.5 Flow Depth

There are no flow depth requirements for concrete-lined channels.

805.6.1.6 Side Slopes

Concrete-lined channels may have side slopes that are vertical or flatter.

805.6.2 Concrete Lining Section

805.6.2.1 Thickness

All concrete lining shall have a minimum thickness of 6 inches for flow velocities less than 30 fps and a minimum thickness of 7 inches for flow velocities of 30 fps and greater.

805.6.2.2 Concrete Joints

The following design standards, found to work in similar conditions, are suggested for use in the Washoe County. Alternatives will be considered on a case by case basis.

a. 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 Washoe County.

b. Longitudinal joints, where required, shall be constructed on the sidewalls at least one foot vertically above the channel invert.

c. All joints shall be designed to prevent differential movement.
d. 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.

805.6.2.3 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.

805.6.2.4 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 Section 501.03.09 of the Standard Specifications.

805.6.2.5 Reinforcement Steel

a. Steel reinforcement shall be a minimum grade - 40 deformed bars. Wire mesh shall not be used.

b. Ratio of longitudinal steel area to concrete cross sectional area shall be greater than .0905 but not less than a #4 rebar placed at a 12-inch spacing. the longitudinal steel shall be placed on top of the transverse steel.

c. Ratio of transverse steel area to concrete cross sectional area shall be greater than .0025 but not less than a #4 rebar placed at a 12-inch spacing.

d. Reinforcing steel shall be place near the center of the section with a minimum clear cover of three inches adjacent to the earth.

e. Additional steel shall be added as needed. If a retaining wall structure is used, the structure must be designed by a registered structural engineer with structural design calculations submitted to Washoe County for review.

805.6.2.6 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 must be required by the geotechnical report.

a. The 12 inches of subgrade immediately beneath concrete lining (both channel bottom and side slopes).

b. Top 12 inches of maintenance road.

c. Top 12 inches of earth surface within 10 feet of concrete channel lip.

d. All fill material.
805.6.2.7 **Bedding**

A geotechnical report shall be submitted to Washoe County which addresses the required bedding necessary for the specific concrete section under consideration.

805.6.2.8 **Underdrain and Weepholes**

The necessity for longitudinal underdrains and weepholes shall be addressed in a geotechnical report submitted to Washoe County for the specific concrete channel section under consideration.

805.6.2.9 **Concrete Cutoffs**

A transverse concrete cutoff shall be installed at the beginning and end of the concrete-lined section of channel and at a maximum spacing of 90 feet. The concrete cutoffs shall extend a minimum of three feet below the bottom of the concrete slab and across the entire width of the channel lining. Longitudinal cutoffs, a minimum of 3 feet in depth, at top lining are required to ensure integrity of the concrete lining.

If the channel is continuously reinforced without transverse joints than a concrete cutoff is required to be incorporated into the expansion/concrete joint.

805.6.3 **Special Consideration for Supercritical Flow**

Supercritical flow in an open channel in an urbanized area creates hazards which the designer must take into consideration. Careful attention must be taken to insure against excessive waves which may extend down the entire length of the channel from only minor obstructions. Imperfections at joints may rapidly cause a deterioration of the joints, in which case a complete failure of the channel can readily occur. In addition, high velocity flow entering cracks or joints creates an uplift force by the conversion of velocity head to pressure head which can damage the channel lining.

Generally, there should not be a drastic reduction in cross section shape and diligent care should be taken to minimize the change in wetted area of the cross-section at bridges and culverts. Bridges and other structures crossing the channel must be anchored satisfactorily to withstand the full dynamic load which might be imposed upon the structure in the event of major debris plugging.

The concrete lining must be protected from hydrostatic uplift forces which are often created by a high-water table or momentary inflow behind the lining form localized flooding. Generally an underdrain will be required under and/or adjacent to the lining.

The underdrain must be designed to be free draining. With supercritical flows, minor downstream obstructions do not create any backwater effect. Backwater computation methods are applicable for computing the water-surface profile or the energy gradient in channels having a supercritical flow; however, the computations must proceed in a downstream direction. The designer must take care to insure against the possibility of unanticipated hydraulic jumps forming in the channel.
805.7 OTHER CHANNEL LININGS

Other channel linings include all channel linings which are not discussed in the previous sections. These include composite-lined channels which are channels in which two or more different lining materials are used (i.e. riprap bottom with concrete side slope lining). They also include gabions, soil cement linings, synthetic fabric and geotextile linings, preformed block linings, reinforced soil linings, and floodwalls (vertical walls constructed on both sides of an existing floodplain). The wide range of composite combinations and other lining types does not allow a discussion of all potential linings in this MANUAL. For those linings not discussed in this MANUAL, supporting documentation will be required to support the use of the desired lining. A guideline of some of the items which must be addressed in the supporting documentation is as follows:

a. Structural integrity of the proposed lining.
b. Interfacing between different linings.
c. The maximum velocity under which the lining will remain stable.
d. Potential erosion and scour problems.
e. Access for operations and maintenance.
f. Long term durability of the product under the extreme meteorological and soil conditions in the Washoe County.
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. Because of the potential significant unknown problems with these lining types, concurrence with Washoe County on the design items to be addressed as well as the final design will be required. Washoe County reserve the right to reject the proposed lining system in the interests of operation, maintenance, and protecting the public safety.

806 ADDITIONAL HYDRAULIC DESIGN STANDARDS

Presented in this section are the hydraulic design standards for design of improved channels. The standards included herein are those standards which are the same for all improved channels. Standards which are specific to a lining type are included in the discussion for the specific lining under consideration.

806.1 SUBCRITICAL FLOW DESIGN STANDARDS

The following design standards are to be used when the design runoff in the channel is flowing in a Subcritical condition ($F_{c} < 0.8$). Furthermore, all super critical channels ($F_{c} > 0.8$) must be designed with the limits as stated in Section 802.2.
806.1.1 Transitions

For the purposes of this manual, 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 super critical flow condition downstream (i.e. sloping spillway entrance). Several typical Subcritical transition sections are presented in Figures 817 and 818. The warped transition section, although most efficient, should only be used in extreme cases where minimum loss of energy is required since the section is very difficult and costly to construct. Conversely, the square-ended transition should only be used when either a straight-line transition or a cylinder-quadrant transition cannot be used due to topographic constraints or utility conflicts.

806.1.1.1 Transition Energy Loss

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

\[ H_t = K_{tc} \left( \frac{V_2^2 - V_1^2}{2g} \right) \] (810)

Where
- \( H_t \) = Energy loss (feet)
- \( K_{tc} \) = Transition coefficient - contraction
- \( V_1 \) = Upstream velocity (feet per second)
- \( V_2 \) = Downstream velocity (feet per second)
- \( g \) = Acceleration of gravity (feet per second squared)

\( K_{tc} \) values for the typical transition sections are presented in Figure 818.

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

\[ H_t = K_{te} \left( \frac{V_1^2 - V_2^2}{2g} \right) \] (811)

Where
- \( H_t \) = Energy loss (feet)
- \( K_{te} \) = Transition coefficient - expansion
- \( V_1 \) = Upstream velocity (feet per second)
- \( V_2 \) = Downstream velocity (feet per second)
- \( g \) = Acceleration of gravity (feet per second squared)

\( K_{te} \) values for the typical transition sections are also presented in Figure 818.

The energy loss in a contracting transition for straight-line or warped transitions is allowed to be partially or totally accommodated by sloping the transition channel bottom from the transition entrance to the exit.
806.1.2 **Transition Length**

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

\[ L_t \geq 0.5L_c(\Delta T_w) \]  \hspace{1cm} (812)

Where

- \( L_t \) = Minimum transition length (feet)
- \( L_c \) = Length coefficient (dimensionless)
- \( \Delta T_w \) = Difference in the top width of the normal water surface upstream and downstream of the transition (feet)

For an approach flow velocity less than 12 feet per second, \( L_c = 4.5 \). This represents a 4.5 (length) to 1.0 (width) wall expansion or contraction with the angle of expansion or contraction of 12.5 degrees from the channel centerline. For an approach flow velocity equal to or greater than 12 feet per second, \( L_c = 10.0 \). This represents a 10.0 (length) to 1.0 (width) expansion or contraction with the angle of expansion or contraction of about 5.75 degrees from the channel centerline.

The transition length equation is not applicable to cylinder-quadrant or square-ended transitions.

806.2 **Superelevation in Bends**

Superelevation in bends is estimated from the following equations:

\[ S_e = \frac{CV^2T_w}{rg} \]  \hspace{1cm} (813)

Where

- \( r \) = Radius of curvature (feet)
- \( C \) = Superelevation coefficient (=0.5 for subcritical flow)
- \( S_e \) = Superelevation water surface increase (feet)
- \( T_w \) = Top width of the design water surface (feet)
- \( V \) = Mean design velocity (feet per second)
- \( g \) = Acceleration of gravity (feet per second squared)

Within Washoe County Superelevation shall be limited to a maximum of 1.0 feet, and the radius of curvature shall conform to the requirements provided in Section 805.4.8.
806.1.3 Freeboard

All subcritical channels shall be constructed with a minimum freeboard determined as follows:

\[ F_b = 0.5 + \frac{V^2}{2g} \]  

(814)

Where

- \( F_b \) = Freeboard height (feet)
- \( V \) = Mean design velocity (feet per second)
- \( g \) = Acceleration of gravity (feet per second squared)

In no case shall the freeboard be less than 1.0 feet. All channel linings must extend to the freeboard height plus the increase in water surface elevation due to superelevation.

806.2 SUPERCritical Flow Design Standards

The following design standards are to be used when the design runoff in the channel is flowing in a supercritical condition (\( F_r > 0.8 \)). Since flow with a Froude Number between 0.8 and 1.13 is relatively unstable, channels with a Froude Number within this range should be designed as a supercritical channel. Furthermore, all supercritical channels must be designed within the limits as situated in Section 802.2.

806.2.1 Super Critical Transitions

The design of supercritical flow in a transition is much more complicated and requires more special attention than a subcritical transition design due to the potential damaging effects of the oblique jump which is created by the transition. The oblique jump results in cross waves and higher flow depths which can cause severe damage if not properly accounted for in the design. A simpler design analysis is to force a hydraulic jump (supercritical flow to subcritical flow). However, hydraulic jumps must also be carefully designed to assure the jump will remain where the jump is designed to occur. For Washoe County, hydraulic jumps shall not be designed to occur in an erodible channel section but only in an energy dissipation or drop structure. The design of these structures are presented in Section 1200 (Additional Hydraulic Structures).

806.2.1.1 Contracting Transitions

Presented in Figure 819 is an example of a supercritical contracting transition. As shown in this figure, the upstream flow is contracted from width \( b_1 \) to \( b_3 \) with a wall diffraction angle of \( \Theta \). 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 \( \beta_1 \). Since the flow pattern is symmetric, the centerline acts as if there was a solid wall that causes a subsequent oblique jump and generates a backward wave front toward the wall with another angle \( \beta_2 \). These continuous oblique jumps result in turbulent fluctuations in the water surface.
To minimize the turbulence, the first two wave fronts are designed to meet at the center and then end at the exit of the contraction. Using the contraction geometry, the length of the transition shall be as follows:

\[ L_t = \frac{b_1 - b_2}{2\tan \theta} \]  

(815)

Where

- \( L_t \) = Transition length (feet)
- \( b_1 \) = Upstream top width of flow (feet)
- \( b_2 \) = Downstream top width of flow (feet)
- \( \theta \) = Wall angle as related to the channel centerline (degrees)

Using the continuity principle,

\[ \frac{b_1}{b_3} = \left( \frac{Y_3^{1.5}}{Y_1} \right) \left( \frac{F_3}{F_1} \right) \]  

(816)

Where

- \( Y_1 \) = Upstream depth of flow (feet)
- \( Y_3 \) = Downstream depth of flow (feet)
- \( F_1 \) = Upstream Froude Number
- \( F_3 \) = Downstream Froude Number

Also, by the continuity and momentum principals, the following relationship between the Froude Number, wave angle, and wall angle is found to be:

\[ \tan \theta = \frac{\tan \beta_1 \left[ (1 + 8F_1^2 \sin^2 \beta_1)^{\frac{1}{4}} - 3 \right]}{2\tan^2 \beta_1 + (1 + 8F_1^2 \sin^2 \beta_1)^{\frac{3}{4}} - 1} \]  

(817)

Where \( \beta_1 \) = Initial wave angle (degrees)

Equations 815, 816, and 817 can be used by trial and error to determine the transition length and wall angle. However, Figure 820 is provided to allow a quicker trial and error solution than by using the equations. The procedure to determine the transition length and wall angle between two predetermined channel sections using Figure 820 is as follows:

**Step 1:** Determine the upstream and downstream channel flow conditions including flow depths, velocities, and Froude numbers.

**Step 2:** If either or both sections are trapezoidal, convert the trapezoidal flow parameters to equivalent rectangular flow parameters by calculating an equivalent flow width equal to the flow area divided by the flow depth. This computed flow width is used for all calculations.
Step 3: Compute \( Y_2/Y_1 \)

Step 4: Assume a trial wall angle, \( \Theta \)

Step 5: Using \( \Theta \) and \( F_1 \), read the values of \( F_2 \) and \( Y_2/Y_1 \) for Section 1 from Figure 820. Then, replacing \( F_1 \) with \( F_2 \) read a second \( F_2 \) (really \( F_3 \)) and second \( Y_2/Y_1 \) (really \( Y_3/Y_2 \)) from Figure 820 for Section 2.

Step 6: Compute the first trial value of \( Y_3/Y_1 \) by multiplying the \( Y_2/Y_1 \) for Section 1 by the \( Y_2/Y_1 \) (really \( Y_3/Y_2 \)) for Section 2.

Step 7: Compare the first trial \( Y_3/Y_1 \) to the actual \( Y_3/Y_1 \) (Step 3). If the trial value \( Y_3/Y_1 \) is larger than the actual \( Y_3/Y_1 \), assume a smaller \( \Theta \) and redo Steps 5 through 7. If the trial value \( Y_3/Y_1 \) is smaller than the actual \( Y_3/Y_1 \), assume a larger \( \Theta \) and redo Steps 5 through 7.

Step 8: Repeat the trial and error procedure until the computed \( Y_3/Y_1 \) is within the five percent of the actual \( Y_3/Y_1 \).

Step 9: Compute the transition length using Equation 820 and the last assumed value of \( \Theta \).

Figure 820 can also be used to determine the wave angle, \( \beta \), or may be used with the equations to determine the required downstream depth or width parameter if a certain transition length is desired or required.

To minimize the length of the transition section, \( Y_3/Y_1 \) should generally be between 2 and 3. However, \( F_3 \) shall not be less than 1.7 for all transition designs. For further discussion on oblique jumps and supercritical contractions, refer to Chow, 1959.

806.2.1.2 Expanding Transitions

The goal of a properly designed expansion transition is to expand the flow boundaries at the same rate as the natural flow expansion. Based on experimental and analytical data results, the minimum length of a supercritical expansion shall be as follows:

\[
L_t \geq 1.5(\Delta T_w)F_{r1}
\]  

Where

- \( L_t \) = Minimum transition length (feet)
- \( \Delta T_w \) = Difference in the top width of the normal water surface upstream and downstream of the transition
- \( F_{r1} \) = Upstream Froude number

806.2.2 Superelevation in Bends

Bends in supercritical channels create cross waves and superelevated flow in the bend section as well as further downstream from the bend. In order to minimize these disturbances, the radius of curvature in the bend shall not cause superelevation of the water surface exceeding...
two feet. Equation 813 can be modified to determine the allowable radius of curvature of a channel for a given superelevation value. In no case shall the radius of curvature be less than 50 feet.

\[ r = \frac{C(V^2T_w)}{(S_e \cdot g)} \]  \hspace{1cm} (819)

C shall equal 1.0 for all trapezoidal channels and for rectangular channels without transition curves. For rectangular channels with transition curves, C shall equal 0.5.

806.2.3 Circular Transition Curves

When a designer desires to reduce the required amount of freeboard and radius of curvature in a rectangular channel, a circular transition curve may be used. The length of the transition curve measured along the channel centerline shall be determined as follows:

\[ L_e = \frac{0.32T_wV}{y^{0.5}} \]  \hspace{1cm} (820)

Where

- \( L_e \) = Length of transition curve (feet)
- \( T_w \) = Top width of design water surface (feet)
- \( V \) = Mean design velocity (feet per second)
- \( y \) = Depth of design flow (feet)

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

806.2.4 Freeboard

In supercritical channels, adequate channel freeboard above the designed water surface shall be provided and shall not be less than that determined by the following:

\[ F_b = 1.0 + 0.025V(d)^{0.5} \]  \hspace{1cm} (821)

Where

- \( F_b \) = Freeboard height (feet)
- \( V \) = Velocity (feet per second)
- \( d \) = depth of flow (feet)

Freeboard shall be in addition to superelevation, standing waves, and/or other water surface disturbances.

The channel lining side slopes shall be extended, as a minimum, to the freeboard elevation.
806.2.5 Slug Flow

Slug flow is a series of shallow-water shock waves which occur in steep super critical channels. The resulting wave heights may easily overtop channel linings using the typical freeboard requirements presented in this MANUAL or damage the channel lining. Therefore, all channels in Washoe County shall be designed to avoid the occurrence of slug flow. To avoid slug flow when the Froude Number is greater than 2.0, the channel slope shall be as follows:

\[ S \leq \frac{12}{R_e} \]  \hspace{1cm} (822)

Where

- \( S \) = Channel slope (feet per feet)  
- \( R_e \) = Reynolds Number

and  

\[ R_e = \frac{VR}{\nu} \]  \hspace{1cm} (823)

Where

- \( V \) = Mean design velocity (feet per second)  
- \( R \) = Hydraulic radius (feet)  
- \( \nu \) = Kinematic viscosity of water (feet squared per second)

Theoretically, slug flow will not occur with \( F_r < 2.0 \).

807 CHANNEL APPURTENANCES

Presented in this section are the design standards for appurtenances to improved channels. All channels in Washoe County shall be designed to include these appurtenances.

807.1 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 less than 50 feet in top width, one maintenance access shall be provided as part of the channel improvements. For channels greater than 50 feet in top width, the maintenance road shall be located in or within 10 feet horizontal distance from the bottom of the channel or on both sides at the channel top.

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.

807.2 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.

807.3 CULVERT OUTLET PROTECTION

If the flow velocity at a culvert or storm sewer outlet exceeds the maximum permissible velocity for the local soil or channel lining, channel protection is required. This protection usually consists of an erosion resistant reach, such as riprap, to provide a stable reach at the outlet in which the exit velocity is reduced to a velocity allowable in the downstream channel.

The following basin sizing procedure shall be used for culvert sizes less than or equal to 36-inches in diameter or equivalent open area and outlet velocities less than 15 fps. For larger culverts or outlet velocities greater than 15 fps, the outlet protection design provided for in USDOT, 1983 shall be used.

807.3.1 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):

\[ L_a = \frac{1.8Q}{D_o^{3/2}} + 7D_o, \text{ for } TW < \frac{D_o}{2} \]  
(824)

and

\[ L_a = \frac{3Q}{D_o^{3/2}} + 7D_o, \text{ for } TW \geq \frac{D_o}{2} \]  
(825)

where

- \( D_o \) = Maximum inside culvert width (ft)
- \( Q \) = Pipe discharge (cfs)
- \( TW \) = Tailwater depth (ft)

Where there is no well defined channel downstream of the apron, the width, \( W \), of the outlet and of the apron (as shown in Figure 821) should be as follows:

\[ W = 3D_o + 0.4L_a, \text{ for } TW \geq \frac{D_o}{2} \]  
(826)

and

\[ W = 3D_o + L_a, \text{ for } TW < \frac{D_o}{2} \]  
(827)

The width of the apron at the culvert outlet should be at least 3 times the culvert width.
WASHOE COUNTY
HYDROLOGIC CRITERIA AND DRAINAGE DESIGN MANUAL

Where there is a well-defined channel downstream of the apron, the bottom width of the apron should be at least equal to the bottom width of the channel and the lining should extend at least one foot above the tailwater elevation and at least two-thirds of the vertical conduit dimension above the invert.

The apron side slopes should be 2:1 or flatter, and the bottom grade should be level.

807.3.2 Rock Size

The median stone diameter, \( d_{50} \), is determined from the following equation:

\[
 d_{50} = 0.02 \frac{(Q)^{4/3}}{TW(D_p)} \tag{828}
\]

Existing scour holes may be used where flat aprons are impractical. Figure 822 shows a general design of a scour hole. The stone diameter is determined using the following equations:

\[
 d_{50} = \frac{0.0125(Q)^{4/3}}{TW(D_p)} \quad \text{for} \quad Y = \frac{D_e}{2} \tag{829}
\]

Also,

\[
 d_{50} = \frac{0.0082(Q)^{4/3}}{TW(D_e)} \quad \text{for} \quad Y = D_o \tag{830}
\]

where \( Y \) = depth of scour hole below culvert invert

The other riprap requirements are as indicated in the previous sections for channel lining.

807.4 GRADE CONTROL STRUCTURES

807.4.1 Introduction

With the advent of floodplain management programs, developers and local governments frequently decided to preserve the floodplain. Since urbanization causes more frequent and sustained flows, the trickle/low flow channel becomes more susceptible to erosion even though the overall floodplain may remain stable and able to resist major flood events.

Erosion of the low flow channel, if left uncontrolled, can cause degradation and 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 elsewhere in this chapter 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.

807.4.2 Drop Structure Grade Control Structures

The grouted sloping boulder drop structure and the vertical riprap drop structure designs can be adapted for use as check structures. The analysis steps are the same with the additional
consideration of 1) stable bed slope for the unlined trickle or low flow channel and 2) potential overflow erosion during submergence of the check structure and where flow converges back from the main channel sides or below the check structure.

The basic design steps for this type of structure include the following:

a. Determine a stable slope and configuration for the low flow zone. For unlined channels, discharges from full floodplain flow to the dominant discharge should first be considered. The dominant discharge is more fully explained in sediment transport texts such as Simons, Li and Associates (1982).

b. The configuration of the low flow zone, and number and placement of the check structures has to be reviewed. Typically, the floodplain slope is steeper, often on the order of critical conditions. If the checks are widely spaced, the trickle channel depth can be quite deep downstream of the check, leading to concentration of higher flows into the trickle channel and the check. A good rule of thumb is to not have the trickle channel more than 2 feet deep at the crest of the check, or more than 4 feet deep below the check structure (relative to the overbank).

c. A hydraulic analysis should be performed using the discharge that completely fills the check structure at its crest (the primary design flow).

d. The secondary design flow is that flow which causes the worst condition for lateral overflow around the abutments and back into the basin or trickle channel below. The goal is to have the check structure survive such an event with minimal or reasonable damage to the floodplain below. The best approach is to estimate unit discharges, velocities and depths along overflow paths. The unit discharges can be estimated at the crest or critical section for the given total flow. Estimating the overflow path around the check abutment is difficult and requires practical judgement. Slopes can be derived for the anticipated overflow routes and protective measures devised such as grouted rock.

e. Seepage control is also important, as piping and erosion through or around these structures is a frequent problem. It is advisable to provide a cutoff which extends laterally at least 5 to 10 feet into undisturbed bank at minimum and has cutoff depth appropriate to the profile dimensions of the check.

**807.4.3 Control Sill Grade Control Structures**

Another type of check structure which can be used to stabilize low flow channels within wide, relatively stable floodplains is the control sill shown in Figure 823. The sill can be constructed by filling an excavated trench with concrete, if soil conditions are acceptable for trenching, or forming a simple wall if a trench will not work.

The sill crosses the low flow channel and should extend a significant distance into the adjacent floodplain on both sides. The top of the sill conforms with 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:
WASHOE COUNTY
HYDROLOGIC CRITERIA AND DRAINAGE DESIGN MANUAL

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).

808 EXAMPLE APPLICATION

808.1 EXAMPLE: OPEN CHANNEL DESIGN FOR DOE CREEK

Problem: An open channel is to be constructed for Doe Creek downstream of John Boulevard and north of Rose Subdivision. Assume the following conditions for this problem.

\[ Q_{100} = 191 \text{ cfs} \]
Invert elevation downstream of John Boulevard = 4,918
Invert elevation downstream of Rose Subdivision = 4,917
Channel improvement length = 900 feet
Due to aesthetics and sufficient right-of-way, a grass-lined channel shall be constructed.

Side Slope = \( z = 3 \)
Bottom Width = \( b = 10 \text{ feet} \)
\( n = 0.035 \) for grass-lined channel

Since the 100-year, 24-hour flow is less than 200 cfs, a trickle channel shall be constructed in the proposed channel bottom.

Solution: Step 1: Determine the depth of water during a 100-year flow event.

\[
\text{Slope} = \frac{4918 - 4917}{900} = 0.0011 \text{ feet/feet}
\]

The Manning Equation can be re-written so that the depth of flow, \( y \), in a trapezoidal channel is on one side of the equation.

\[
\left(\frac{by + zy^2}{b + 2y(1 + z^2)^{1/4}}\right)^{5/3} = \left(\frac{Q}{S^{1/3}}\right) \left(\frac{n}{1.49}\right)
\]

Solving by trial and error,

\( Y = 3.7 \text{ feet} \)

Step 2: Calculate the water velocity in the proposed channel during a 100-year flow event using the Manning Equation.

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

\[
= \left(\frac{1.49}{0.035}\right) \times (0.011)^{2/3} \times \left(\frac{10 + 3 + 3.7 \times 3.7}{10 + 2 \times 3.7 + (1 + 3^3)^{1/3}}\right)^{2/3}
\]

December 2, 1996
Open Channels 843
- 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 the dimensions for a concrete trickle channel are:

Bottom width = 5 feet
Depth = 1 foot
Side Slopes = vertical

The capacity of the trickle channel is:

\[
Q = \left( \frac{1.49}{n} \right) \left( S^{1/4} \right) R^{3/8} (A)
\]

\[
Q = \left( \frac{1.49}{n} \right) \left( S^{1/4} \right) \left( \frac{by}{b + 2y} \right)^{36} (by)
\]

\[
Q = \left( \frac{1.49}{0.015} \right) \times (0.011)^{36} \times ((5+1)/(5+(2+1)))^{36} \times (5 \times 1)
\]

\[Q = 13.16 \text{ cfs}\]

Step 4: Verify that trickle channel has sufficient capacity.

The minimum capacity of the trickle channel is:

\[\text{Min. } Q_{T_e} = 0.03 \times Q_{100}\]

\[\text{Min. } Q_{e} = 5.6 \text{ cfs}\]

Since the capacity of the proposed trickle channel (13.2 cfs) is greater than the required capacity (5.6 cfs), the proposed trickle channel is adequate.

Step 5: Determine the freeboard required for the proposed channel.

\[F_b = 0.5 + \frac{V^2}{2g}\]

\[F_b = 0.5 + \frac{(2.5)^2}{2 \times 32.2} = 0.6 \text{ feet, but minimum = 1.0 feet}\]

Therefore use \(F_b = 1.0 \text{ feet.}\)

Step 6: The cross-section of the proposed channel is shown in Figure 824.
# Geometric Elements of Channel Sections

<table>
<thead>
<tr>
<th>Section</th>
<th>Area, $A$</th>
<th>Wetted Perimeter, $P$</th>
<th>Hydraulic Radius, $R$</th>
<th>Top Width, $T$</th>
<th>Hydraulic Depth, $D$</th>
<th>Section Factor, $Z$</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Rectangle</strong></td>
<td>$by$</td>
<td>$b+2y$</td>
<td>$\frac{by}{b+2y}$</td>
<td>$b$</td>
<td>$y$</td>
<td>$by^{1.8}$</td>
</tr>
<tr>
<td><strong>Trapezoid</strong></td>
<td>$(b+zy)y$</td>
<td>$b+2y\sqrt{1+z^2}$</td>
<td>$\frac{(b+zy)y}{b+2y\sqrt{1+z^2}}$</td>
<td>$b+2zy$</td>
<td>$(b+zy)y$</td>
<td>$\frac{[(b+zy)y]^{1.8}}{\sqrt{b+2zy}}$</td>
</tr>
<tr>
<td><strong>Triangle</strong></td>
<td>$zy^2$</td>
<td>$2y\sqrt{1+z^2}$</td>
<td>$\frac{zy}{2\sqrt{1+z^2}}$</td>
<td>$2zy$</td>
<td>$\frac{y}{2}$</td>
<td>$\frac{\sqrt{2}}{2}2y^{1.8}$</td>
</tr>
<tr>
<td><strong>Circle</strong></td>
<td>$\frac{1}{4}(\theta-\sin\theta)d_s^2$</td>
<td>$\frac{1}{4}(1-\frac{\sin\theta}{\theta})d_s^2$</td>
<td>or $\frac{(\sin\frac{\theta}{2})d_s}{2\sqrt{\gamma(d_s-y)}}$</td>
<td>$\frac{1}{3}(\theta-\sin\theta)d_s^{1.8}$</td>
<td>$\frac{\sqrt{2}}{32}\frac{(\theta-\sin\theta)^{1.8}}{(\sin\frac{\theta}{2})^{1.8}}$</td>
<td></td>
</tr>
<tr>
<td><strong>Parabola</strong></td>
<td>$\frac{\pi}{4}\gamma y$</td>
<td>$\gamma+\frac{2\gamma^2}{3}\gamma^2$</td>
<td>$\frac{2\gamma^2}{3}(\gamma^2+8\gamma^2)$</td>
<td>$\frac{3A}{2\gamma}$</td>
<td>$\frac{2\gamma}{3}$</td>
<td>$\frac{3}{4}\sqrt{6}\gamma^{1.8}$</td>
</tr>
<tr>
<td><strong>Round-cornered rectangle (y&gt;r)</strong></td>
<td>$\frac{(\pi/2-2)r^2+(b+2r)y}{(\pi/2)r+b+2y}$</td>
<td>$b+2r$</td>
<td>$(\frac{\pi/2-2)r^2+(b+2r)y}{b+2r}+y$</td>
<td>$\frac{[(\pi/2-2)r^2+(b+2r)y]^{1.8}}{\sqrt{b+2y}}$</td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Round-bottom triangle</strong></td>
<td>$\frac{7}{4z}\gamma z^2-\frac{\gamma^2}{2}(1-z\cot^2\gamma)$</td>
<td>$\frac{T}{2}\sqrt{1+\gamma^2-\frac{2r}{z}(1-z\cot^2\gamma)}$</td>
<td>$\frac{T}{2}\gamma z^2-\frac{\gamma^2}{2}(1-z\cot^2\gamma)$</td>
<td>$\frac{A}{P}$</td>
<td>$2[y(y-r)+\gamma\sqrt{1+z^2}]$</td>
<td>$\frac{A}{T}$</td>
</tr>
</tbody>
</table>

*Satisfactory approximation for the interval $0<x\leq 1$, where $x=4y/T$. When $x>1$, use the exact expression $P=(r/2)[\sqrt{1+x^2}+1/x \ln(x+\sqrt{1+x^2})]$
# Typical Roughness Coefficients for Open Channels

## Type of Channel and Description

<table>
<thead>
<tr>
<th>EXCAVATED OR DREDGED</th>
<th>MINIMUM</th>
<th>NORMAL</th>
<th>MAXIMUM</th>
</tr>
</thead>
<tbody>
<tr>
<td>a. Earth, straight and uniform</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1. Clean, recently completed</td>
<td>0.016</td>
<td>0.018</td>
<td>0.020</td>
</tr>
<tr>
<td>2. Clean, after weathering</td>
<td>0.018</td>
<td>0.022</td>
<td>0.025</td>
</tr>
<tr>
<td>3. Gravel, uniform section, clean</td>
<td>0.022</td>
<td>0.025</td>
<td>0.030</td>
</tr>
<tr>
<td>4. With short grass, few weeds</td>
<td>0.022</td>
<td>0.027</td>
<td>0.033</td>
</tr>
<tr>
<td>b. Earth, winding and sluggish</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1. No vegetation</td>
<td>0.023</td>
<td>0.025</td>
<td>0.030</td>
</tr>
<tr>
<td>2. Grass, some weeds</td>
<td>0.025</td>
<td>0.030</td>
<td>0.033</td>
</tr>
<tr>
<td>3. Dense weeds or aquatic plants in deep channels</td>
<td>0.030</td>
<td>0.035</td>
<td>0.040</td>
</tr>
<tr>
<td>4. Earth bottom and rubble sides</td>
<td>0.028</td>
<td>0.030</td>
<td>0.035</td>
</tr>
<tr>
<td>5. Stony bottom and weedy banks</td>
<td>0.025</td>
<td>0.035</td>
<td>0.040</td>
</tr>
<tr>
<td>6. Cobble bottom and clean sides</td>
<td>0.030</td>
<td>0.040</td>
<td>0.050</td>
</tr>
<tr>
<td>c. Dragline-excavated or dredged</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1. No vegetation</td>
<td>0.025</td>
<td>0.028</td>
<td>0.033</td>
</tr>
<tr>
<td>2. Light brush on banks</td>
<td>0.035</td>
<td>0.050</td>
<td>0.060</td>
</tr>
<tr>
<td>d. Rock cuts</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1. Smooth and uniform</td>
<td>0.025</td>
<td>0.035</td>
<td>0.040</td>
</tr>
<tr>
<td>2. Jagged and irregular</td>
<td>0.035</td>
<td>0.040</td>
<td>0.050</td>
</tr>
<tr>
<td>e. Channels not maintained, weeds and brush</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1. Dense weeds, high as flow depth</td>
<td>0.050</td>
<td>0.080</td>
<td>0.120</td>
</tr>
<tr>
<td>2. Clean bottom, brush on sides</td>
<td>0.040</td>
<td>0.050</td>
<td>0.080</td>
</tr>
<tr>
<td>3. Same as above, but highest state of flow</td>
<td>0.045</td>
<td>0.070</td>
<td>0.110</td>
</tr>
<tr>
<td>4. Dense brush, high state</td>
<td>0.080</td>
<td>0.100</td>
<td>0.140</td>
</tr>
</tbody>
</table>

## Natural Streams

Minor Streams (top width at flood stage < 100 ft)

<table>
<thead>
<tr>
<th>a. Streams on plain</th>
<th>MINIMUM</th>
<th>NORMAL</th>
<th>MAXIMUM</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Clean, straight, full stage, no rifts or deep pools</td>
<td>0.025</td>
<td>0.030</td>
<td>0.033</td>
</tr>
<tr>
<td>2. Same as above, but more stones and weeds</td>
<td>0.030</td>
<td>0.035</td>
<td>0.040</td>
</tr>
<tr>
<td>3. Clean, winding, some pools and shoals</td>
<td>0.033</td>
<td>0.040</td>
<td>0.045</td>
</tr>
<tr>
<td>4. Same as above, but some weeds and stones</td>
<td>0.035</td>
<td>0.045</td>
<td>0.050</td>
</tr>
<tr>
<td>5. Same as above, but lower stages, and more ineffective slopes and sections</td>
<td>0.040</td>
<td>0.048</td>
<td>0.055</td>
</tr>
</tbody>
</table>
TYPICAL ROUGHNESS COEFFICIENTS FOR OPEN CHANNELS

<table>
<thead>
<tr>
<th>TYPE OF CHANNEL AND DESCRIPTION</th>
<th>MINIMUM</th>
<th>NORMAL</th>
<th>MAXIMUM</th>
</tr>
</thead>
<tbody>
<tr>
<td>6. Same as 4, but more stones</td>
<td>0.045</td>
<td>0.050</td>
<td>0.060</td>
</tr>
<tr>
<td>7. Sluggish reaches, weedy, deep pools</td>
<td>0.050</td>
<td>0.070</td>
<td>0.080</td>
</tr>
<tr>
<td>8. Very weedy reaches, deep pools, or floodways with heavy stand of timber and underbrush</td>
<td>0.075</td>
<td>0.100</td>
<td>0.150</td>
</tr>
<tr>
<td>b. Mountain streams, no vegetation in channel, banks usually steep, trees and brush along banks submerged at high stages</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1. Bottom: gravel, cobbles, and few boulders</td>
<td>0.030</td>
<td>0.040</td>
<td>0.050</td>
</tr>
<tr>
<td>2. Bottom: cobbles with large boulders</td>
<td>0.040</td>
<td>0.050</td>
<td>0.070</td>
</tr>
</tbody>
</table>

Floodplains

a. Pasture, no brush
   1. Short grass | 0.025 | 0.030 | 0.035 |
   2. High grass  | 0.030 | 0.035 | 0.050 |

b. Cultivated areas
   1. No crop     | 0.020 | 0.030 | 0.040 |
   2. Mature row crops | 0.025 | 0.035 | 0.045 |
   3. Mature field crops | 0.030 | 0.040 | 0.050 |

c. Brush
   1. Scattered brush, heavy weeds | 0.035 | 0.050 | 0.070 |
   2. Light brush and trees, in winter | 0.035 | 0.050 | 0.060 |
   3. Light brush and trees, in summer | 0.040 | 0.060 | 0.080 |
   4. Medium to dense brush, in winter | 0.045 | 0.070 | 0.110 |
   5. Medium to dense brush, in summer | 0.070 | 0.100 | 0.160 |

d. Trees
   1. Dense willows, summer, straight | 0.110 | 0.105 | 0.200 |
   2. Cleared land with tree stumps, no sprouts | 0.030 | 0.040 | 0.050 |
   3. Same as above, but with heavy growth of sprouts | 0.050 | 0.060 | 0.080 |
   4. Heavy stand of timber, a few down trees, little undergrowth, flood stage below branches | 0.080 | 0.100 | 0.120 |
   5. Same as above, but with flood stage reaching branches | 0.100 | 0.120 | 0.160 |

Major streams (top width at flood state 100 ft). The n value is less than that for minor streams of similar description, because banks offer less effective resistance.

a. Regular section with no boulders or brush | 0.025 |   | 0.060 |

b. Irregular and rough section | 0.05 |   | 0.100 |
## Typical Roughness Coefficients for Open Channels

<table>
<thead>
<tr>
<th>Type of Channel and Description</th>
<th>Minimum</th>
<th>Normal</th>
<th>Maximum</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Lined or Built-up Channels</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>a. Concrete</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1. Trowel finish</td>
<td>0.011</td>
<td>0.013</td>
<td>0.015</td>
</tr>
<tr>
<td>2. Float finish</td>
<td>0.013</td>
<td>0.015</td>
<td>0.016</td>
</tr>
<tr>
<td>3. Gunite, good section</td>
<td>0.016</td>
<td>0.019</td>
<td>0.023</td>
</tr>
<tr>
<td>4. Gunite, wavy section</td>
<td>0.018</td>
<td>0.022</td>
<td>0.023</td>
</tr>
<tr>
<td>b. Concrete bottom float finished with side of</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1. Dressed stone in mortar</td>
<td>0.015</td>
<td>0.017</td>
<td>0.020</td>
</tr>
<tr>
<td>2. Random stone in mortar</td>
<td>0.017</td>
<td>0.020</td>
<td>0.024</td>
</tr>
<tr>
<td>3. Dry rubble or riprap</td>
<td>0.020</td>
<td>0.030</td>
<td>0.035</td>
</tr>
<tr>
<td>c. Gravel bottom with sides of</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1. Formed concrete</td>
<td>0.017</td>
<td>0.020</td>
<td>0.025</td>
</tr>
<tr>
<td>2. Random stone in mortar</td>
<td>0.020</td>
<td>0.023</td>
<td>0.026</td>
</tr>
<tr>
<td>3. Dry rubble or riprap</td>
<td>0.023</td>
<td>0.033</td>
<td>0.036</td>
</tr>
<tr>
<td>d. Asphalt</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1. Smooth</td>
<td>0.013</td>
<td>0.013</td>
<td>–</td>
</tr>
<tr>
<td>2. Rough</td>
<td>0.016</td>
<td>0.016</td>
<td>–</td>
</tr>
<tr>
<td>e. Grassed</td>
<td>0.030</td>
<td>0.040</td>
<td>0.050</td>
</tr>
</tbody>
</table>
## Maximum Permissible Mean Channel Velocities

<table>
<thead>
<tr>
<th>Material/Lining</th>
<th>Maximum Permissible Mean Velocity (fps)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Natural and Improved Unlined Channels</strong></td>
<td></td>
</tr>
<tr>
<td>Erosive Soils:</td>
<td></td>
</tr>
<tr>
<td>Loams, Sands, Noncolloidal Silts</td>
<td>3.0</td>
</tr>
<tr>
<td>Less Erosive Soils:</td>
<td></td>
</tr>
<tr>
<td>Clays, Shales, Cobbles, Gravel</td>
<td>5.0</td>
</tr>
<tr>
<td><strong>Fully Lined Channels</strong></td>
<td></td>
</tr>
<tr>
<td>Unreinforced vegetation</td>
<td>5.5</td>
</tr>
<tr>
<td>Loose riprap</td>
<td>15.0</td>
</tr>
<tr>
<td>Grouted riprap</td>
<td>15.0</td>
</tr>
<tr>
<td>Gibbons</td>
<td>15.0</td>
</tr>
<tr>
<td>Soil-Cement</td>
<td>15.0</td>
</tr>
<tr>
<td>Concrete</td>
<td>35.0</td>
</tr>
</tbody>
</table>

**Notes:**

1. For composite lined channels, use the lowest of the maximum mean velocities for the materials used in the composite lining.

2. Deviations from the above values are only allowed with appropriate engineering analysis and/or suitable agreements for maintenance responsibilities.

3. Maximum permissible velocities based upon non-clear water conditions.
## Classification and Gradation of Loose Riprap

<table>
<thead>
<tr>
<th>Riprap Class Designation</th>
<th>% Smaller Than Given Size by Weight</th>
<th>Riprap Gradation (Inches)</th>
<th>(d_{50})* (Inches)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Class 150</td>
<td></td>
<td></td>
<td>10</td>
</tr>
<tr>
<td></td>
<td>100</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>35 - 50</td>
<td>6</td>
<td></td>
</tr>
<tr>
<td></td>
<td>0 - 15</td>
<td>2</td>
<td>6**</td>
</tr>
<tr>
<td>Class 300</td>
<td></td>
<td></td>
<td>20</td>
</tr>
<tr>
<td></td>
<td>100</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>35 - 50</td>
<td>12</td>
<td></td>
</tr>
<tr>
<td></td>
<td>0 - 15</td>
<td>4</td>
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</tr>
<tr>
<td>Class 400</td>
<td></td>
<td></td>
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</tr>
<tr>
<td></td>
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<td></td>
</tr>
<tr>
<td></td>
<td>35 - 50</td>
<td>16</td>
<td></td>
</tr>
<tr>
<td></td>
<td>0 - 15</td>
<td>6</td>
<td></td>
</tr>
<tr>
<td>Class 550</td>
<td></td>
<td></td>
<td>37</td>
</tr>
<tr>
<td></td>
<td>100</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>35 - 50</td>
<td>22</td>
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<td>0 - 15</td>
<td>8</td>
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<tr>
<td>Class 700</td>
<td></td>
<td></td>
<td>45</td>
</tr>
<tr>
<td></td>
<td>100</td>
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<tr>
<td></td>
<td>35 - 50</td>
<td>28</td>
<td></td>
</tr>
<tr>
<td></td>
<td>0 - 15</td>
<td>10</td>
<td></td>
</tr>
<tr>
<td>Class 900</td>
<td></td>
<td></td>
<td>57</td>
</tr>
<tr>
<td></td>
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<tr>
<td></td>
<td>35 - 50</td>
<td>35</td>
<td></td>
</tr>
<tr>
<td></td>
<td>0 - 15</td>
<td>14</td>
<td></td>
</tr>
</tbody>
</table>

*\(d_{50}\) = mean stone size

** Bury Class 150 riprap with native top soil and re-vegetate to protect from vandalism
### Gradation for Granular Riprap Bedding

<table>
<thead>
<tr>
<th>Riprap Designation</th>
<th>Granular Bedding Sieve Size (MM)</th>
<th>Granular Bedding Percent Passing by Weight</th>
</tr>
</thead>
<tbody>
<tr>
<td>Class 150</td>
<td>37.5</td>
<td>100</td>
</tr>
<tr>
<td></td>
<td>19</td>
<td>35 - 100</td>
</tr>
<tr>
<td></td>
<td>12.5</td>
<td>15 - 80</td>
</tr>
<tr>
<td></td>
<td>9.5</td>
<td>5 - 60</td>
</tr>
<tr>
<td></td>
<td>4.75</td>
<td>0 - 35</td>
</tr>
<tr>
<td></td>
<td>1.18</td>
<td>0 - 5</td>
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<tr>
<td>Class 300</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td></td>
<td>37.5</td>
<td>30 - 100</td>
</tr>
<tr>
<td></td>
<td>25</td>
<td>15 - 80</td>
</tr>
<tr>
<td></td>
<td>12.5</td>
<td>0 - 50</td>
</tr>
<tr>
<td></td>
<td>4.75</td>
<td>0 - 20</td>
</tr>
<tr>
<td></td>
<td>2.36</td>
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<tr>
<td>Class 400</td>
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<tr>
<td></td>
<td>50</td>
<td>30 - 100</td>
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<td>37.5</td>
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<tr>
<td></td>
<td>19</td>
<td>0 - 45</td>
</tr>
<tr>
<td></td>
<td>6.3</td>
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<tr>
<td></td>
<td>4.75</td>
<td>0 - 10</td>
</tr>
<tr>
<td>Class 550</td>
<td>150</td>
<td>100</td>
</tr>
<tr>
<td></td>
<td>75</td>
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<td>25</td>
<td>0 - 50</td>
</tr>
<tr>
<td></td>
<td>12.5</td>
<td>0 - 30</td>
</tr>
<tr>
<td></td>
<td>6.3</td>
<td>0 - 10</td>
</tr>
<tr>
<td>Class 700</td>
<td>200</td>
<td>100</td>
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<tr>
<td></td>
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<td>25 - 85</td>
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<tr>
<td></td>
<td>50</td>
<td>5 - 70</td>
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<tr>
<td></td>
<td>19</td>
<td>0 - 40</td>
</tr>
<tr>
<td></td>
<td>9.5</td>
<td>0 - 15</td>
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<td>6.3</td>
<td>0 - 5</td>
</tr>
<tr>
<td>Class 900</td>
<td>250</td>
<td>100</td>
</tr>
<tr>
<td></td>
<td>100</td>
<td>25 - 90</td>
</tr>
<tr>
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<td>75</td>
<td>15 - 75</td>
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<tr>
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<td>25</td>
<td>0 - 35</td>
</tr>
<tr>
<td></td>
<td>12.5</td>
<td>0 - 15</td>
</tr>
<tr>
<td></td>
<td>6.3</td>
<td>0 - 5</td>
</tr>
</tbody>
</table>
## Classification and Gradation of Rock for Grouted Riprap

<table>
<thead>
<tr>
<th>Riprap Designation</th>
<th>% Smaller Than Given Size By Weight</th>
<th>Intermediate Rock Dimension (Inches)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Class 400</td>
<td>100</td>
<td>26</td>
</tr>
<tr>
<td></td>
<td>35 - 50</td>
<td>16</td>
</tr>
<tr>
<td></td>
<td>0 - 5</td>
<td>12</td>
</tr>
<tr>
<td>Class 550</td>
<td>100</td>
<td>37</td>
</tr>
<tr>
<td></td>
<td>35 - 50</td>
<td>22</td>
</tr>
<tr>
<td></td>
<td>0 - 5</td>
<td>16</td>
</tr>
<tr>
<td>Class 700</td>
<td>100</td>
<td>45</td>
</tr>
<tr>
<td></td>
<td>35 - 50</td>
<td>28</td>
</tr>
<tr>
<td></td>
<td>0 - 5</td>
<td>20</td>
</tr>
<tr>
<td>Class 900</td>
<td>100</td>
<td>57</td>
</tr>
<tr>
<td></td>
<td>35 - 50</td>
<td>35</td>
</tr>
<tr>
<td></td>
<td>0 - 5</td>
<td>28</td>
</tr>
</tbody>
</table>
## DESIGN $D_{50}$ VALUES

<table>
<thead>
<tr>
<th>$D_{50}$ DETERMINED FROM DESIGN CURVE (ft)</th>
<th>MINIMUM DESIGN $D_{50}$ (ft)</th>
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<tbody>
<tr>
<td>&lt; 0.25</td>
<td>0.25</td>
</tr>
<tr>
<td>0.26 - 0.50</td>
<td>0.50</td>
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<tr>
<td>0.51 - 0.75</td>
<td>0.75</td>
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<tr>
<td>0.76 - 1.00</td>
<td>1.00</td>
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<tr>
<td>1.01 - 1.25</td>
<td>1.25</td>
</tr>
<tr>
<td>1.26 - 1.50</td>
<td>1.50</td>
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<tr>
<td>1.51 - 1.75</td>
<td>1.75</td>
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<tr>
<td>1.76 - 2.00</td>
<td>2.00</td>
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<tr>
<td>2.01 - 2.25</td>
<td>2.25</td>
</tr>
<tr>
<td>2.26 - 2.50</td>
<td>2.50</td>
</tr>
<tr>
<td>2.51 - 2.75</td>
<td>2.75</td>
</tr>
<tr>
<td>2.76 - 3.00</td>
<td>3.00</td>
</tr>
</tbody>
</table>
RIPRAPPGRADATIONFORSTEEPSLOPES

\[
\frac{D_{\text{max}}}{D_{10}} = 1.25
\]

\[
\frac{D_{50}}{D_{20}} = 2
\]

\[
\frac{D_{50}}{D_{10}} = 3
\]
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
TYPICAL OPEN-CHANNEL DESIGN SECTIONS
(IMPROVED CHANNELS)

100-YR. WATER SURFACE

UNLINED AND GRASS-LINED CHANNEL

100-YR. WATER SURFACE

RIPRAP-LINED CHANNEL

GRANULAR BEDDING MATERIAL

MAINTENANCE ROAD

MAINTENANCE ROAD
WASHOE COUNTY
HYDROLOGIC CRITERIA AND DRAINAGE DESIGN MANUAL

TYPICAL OPEN-CHANNEL DESIGN SECTIONS
(IMPROVED CHANNELS)

MAINTENANCE ROAD

100-YR.
WATER SURFACE

slope ←→ slope

CONCRETE-LINED CHANNEL

MAINTENANCE ROAD

100-YR.
WATER SURFACE

CONCRETE

3' MIN.

COMPOSITE-LINED CHANNEL

RIPRAP

Granular Bedding Material

NOTE:
REFER TO SECTION 805.7 FOR DISCUSSION OF DESIGN CONSIDERATIONS.

VERSION: December 2, 1998
REFERENCE:
WRC ENGINEERING, INC.

FIGURE
804
2 OF 2
ROUGHNESS COEFFICIENT FOR GRASS-LINED

Channel capacity design curve

V*R
PRODUCT OF VELOCITY AND HYDRAULIC RADIUS

TYPICAL CROSS-SECTION OF
CONCRETE-LINED TRICKLE CHANNEL

Freeboard

Grassed Slope

Trickle Channel

ALTERNATIVE SECTION

WASHOE COUNTY
HYDROLOGIC CRITERIA AND DRAINAGE DESIGN MANUAL

VERSION: December 2, 1996
REFERENCE: WRC ENGINEERING INC.
FIGURE 806
TYPICAL CROSS-SECTION OF RIPRAP-LINED TRICKLE CHANNEL

- Grassed Slope
- Freeboard
- 1% to 2% incline
- 2.0' Min.
- TRICKLE CHANNEL
- Granular Bedding Material
- 2.5
- 1
- 1
- 3 Minimum
- 20' Min.
- 5' Min.
TYPICAL CROSS-SECTION OF LOW-FLOW CHANNEL

- Freeboard
- 1% to 2%
- 3.0' to 5.0'
- RIPRAPH
- 2.5
- Granular Bedding Material
- Low Flow Channel

30' Min. 8' Min.
MANNING'S ROUGHNESS COEFFICIENT FOR UNLINED LOW FLOW CHANNELS

Depth Of Flow (Feet)

* Use normal depth, ignoring all backwater effects
TYPICAL CROSS-SECTIONS FOR RIPRAP-LINED CHANNELS

- Freesboard + Superelevation (1.0' minimum)
- Not steeper than 2H to 1V
- Scour Depth or 3 ft. minimum whichever is greater.

- Freesboard + Superelevation (1.0' minimum)
- Not steeper than 2H to 1V
- Riprap
- Granular Bedding Material
- Scour Depth or 3 ft. minimum whichever is greater.
TYPICAL CROSS-SECTION FOR GROUTED RIPRAP LINING

GROUT CUT-OFF. EXTEND TO FULL DEPTH OF RIPRAP AND BEDDING LAYER. PLACE CUT-OFF ALONG TOP OF SLOPE AND ALONG UPSTREAM AND DOWNSTREAM ENDS OF RIPRAP INSTALLATION.

DESIGN RIPRAP GRADE

LARGER ROCK AT SURFACE

MIN. 8" COMPACTED TOPSOIL

FINISHED GRADE

SLOPE VARIES

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.

REFERENCE: UDFCD, 1990

FIGURE 811
STEEP SLOPE RIPRAP DESIGN,
TRIANGULAR CHANNELS, 2:1 SIDESLOPES

Graphs showing the relationship between discharge (Q) and critical size (D_{50} and d) for triangular channels with 2:1 sideslopes.
STEEP SLOPE RIPRAP DESIGN, TRAPEZOIDAL CHANNELS, 2:1 SIDESLOPES, 6 FT BASE WIDTH

REFERENCE: Simons, Li and Assoc., 1989

FIGURE 813
WASHOE COUNTY
HYDROLOGIC CRITERIA AND DRAINAGE DESIGN MANUAL

STEEP SLOPE RIPRAP DESIGN, TRAPEZOIDAL CHANNELS, 2:1 SIDESLOPES, 10 FT BASE WIDTH

Q (cfs)

REFERENCE: Simons, Li and Assoc., 1989

FIGURE 814
STEEP SLOPE RIPRAP DESIGN, TRAPEZOIDAL CHANNELS, 2:1 SIDESLOPES, 20 FT BASE WIDTH

REFERENCES:
Simons, Li and Assoc., 1989

FIGURE 818
CHANNEL TRANSITION TYPES

\[ Froude\ Number = \frac{v_A}{\sqrt{gA}} \]

- CYLINDRICAL QUADRANT
- STRAIGHT LINE
- WARPED
- WEDGE
- SQUARE-ENDED
- SQUARE-ENDED
WASHOE COUNTY
HYDROLOGIC CRITERIA AND DRAINAGE DESIGN MANUAL

TYPICAL CHANNEL TRANSITION SECTIONS
AND ENERGY LOSS COEFFICIENTS

WARPED TRANSITION
\( K_{tc} \) (CONTRACTION) = 0.1
\( K_{te} \) (EXPANSION) = 0.2

STRAIGHT-LINE TRANSITION
\( K_{tc} \) (CONTRACTION) = 0.3
\( K_{te} \) (EXPANSION) = 0.5

CYLINDER-QUADRANT
\( K_{tc} = 0.15 \)
\( K_{te} = 0.25 \)

SQUARE-ENDED TRANSITION
\( K_{tc} = 0.30 \)
\( K_{te} = 0.75 \)
NOTES:
(a) GENERAL DISTURBANCE PATTERNS
(b) MINIMUM DOWNSTREAM DISTURBANCE
(c) SCHEMATIC PROFILE

REFERENCE:

FIGURE 819
EXAMPLE: For $\theta = 5'$ and $F_1 = 5.0$

1. Read $F_2 = 4.1$
2. Read $\frac{y_2}{y_1} = 1.5$
3. Read $\beta_1 = 16'$
4. Read $\theta = 5'$ (check)
CONFIGURATION OF CULVERT OUTLET PROTECTION

\[ W = 3D_0 + 0.4L_0 \]  
(Tailwater \( \geq 0.5 \ D_0 \))

\[ W = 3D_0 + L_0 \]  
(Tailwater \( < 0.5 \ D_0 \))
PREFORMED SCOUR HOLE

PLAN VIEW

SECTION VIEW

REFERENCE:

ASCE, 1975

FIGURE

822
CONTROL SILL GRADE CONTROL STRUCTURE

(a) PLAN

LOW FLOW CHANNEL BANKS

10' MIN

NATURAL CHANNEL SLOPE

PROJECTED FUTURE DOWNSTREAM GRADE BELOW SILL

2'

CONTROL SILLS ARE SPACED TO PRODUCE MAXIMUM 3 FT. DROP

(b) PROFILE

PROJECTED SLOPE

NATURAL BANK SLOPE

MINIMUM DEPTH TO CARRY 2-YEAR FLOOD FOR FUTURE DEVELOPED BASIN

MATCH TO EXISTING GRADE SURFACE

MINIMUM WEIR CAPACITY = 3% OF Q_{100} WITHOUT USE OF DETENTION

(c) SECTION A-A'

PROJECTED GRADE FUTURE DOWNSTREAM

MINIMUM DEPTH TO CARRY 2-YEAR FLOOD FOR FUTURE DEVELOPED BASIN

MATCH TO EXISTING GRADE SURFACE

MINIMUM WEIR CAPACITY = 3% OF Q_{100} WITHOUT USE OF DETENTION

(c) SECTION A-A'

WASHOE COUNTY
HYDROLOGIC CRITERIA AND DRAINAGE DESIGN MANUAL

VERSION: December 2, 1988
REFERENCE: UDFCD, 1990
FIGURE 823
EXAMPLE: CROSS-SECTION OF DOE CREEK

CONCRETE-LINED TRICKLE CHANNEL

1.0'  4.7'  3

2.5'  5.0'  2.5'  3.7'  1  1

WASHOE COUNTY
HYDROLOGIC CRITERIA AND DRAINAGE DESIGN MANUAL

VERSION: December 2, 1996
REFERENCE:
WRC ENGINEERING, INC.
SECTION 900
STORM SEWER SYSTEM

TABLE OF CONTENTS

901 INTRODUCTION

902 DESIGN PARAMETERS
  902.1 Allowable Storm Sewer Capacity
  902.2 Allowable Storm Sewer Velocity
  902.3 Manning's Roughness Coefficient
  902.4 Storm Sewer Layout
    902.4.1 Vertical Alignment
      902.4.1.1 Minimum and Maximum Cover
      902.4.1.2 Manhole
    902.4.2 Horizontal Alignment
    902.4.3 Utility Clearances
    902.4.3.1 Water Mains
    902.4.3.2 Sewer Mains
  902.5 Allowable Storm Inlet Types and Capacity Factors

903 CONSTRUCTION STANDARDS
  903.1 Storm Sewer Pipe
    903.1.1 Size
    903.1.2 Material and Shape
    903.1.3 Joint Sealants and Gaskets
  903.2 Manholes
  903.3 Storm Sewer Inlets
  903.4 Storm Sewer Outlets

904 STORM SEWER HYDRAULICS
  904.1 Hydraulic Analysis
    904.1.1 Pressure Flow Analysis
    904.1.2 Partial Full Flow Analysis
  904.2 Energy Loss Calculations
    904.2.1 Pipe Friction Losses
    904.2.2 Transition Losses
    904.2.2.1 Expansion Losses
    904.2.2.2 Contraction Losses
    904.2.2.3 Bend Losses
    904.2.2.4 Junction and Manhole Losses
    904.2.2.5 Inlet Losses

Page
903
903
904
904
905
905
905
906
906
906
906
907
907
907
907
908
908
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909
909
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910
911
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911
900

December 2, 1996
Storm Sewer Systems
WASHOE COUNTY
HYDROLOGIC CRITERIA AND DRAINAGE DESIGN MANUAL

904.2.2.6 - Outlet Losses 912
904.2.2.7 - Drop Manholes 912

905 STORM INLET HYDRAULICS 912
905.1 - Inlets on Continuous Grade 913
905.2 - Inlets in a Sump Condition 913
905.3 - Inlet Spacing 914
905.4 - Inlet Capacity for Major Storm Analysis 914

906 STORM SEWER SYSTEM DESIGN 914
906.1 - Initial Storm Sewer Sizing 914
906.2 - Final Storm Sewer Sizing 915

907 EXAMPLE APPLICATION 915
907.1 - Introduction 915
907.2 - Example: Storm Sewer Hydraulic Analysis 915

LIST OF TABLES
901 STORM SEWER DESIGN AND ANALYSIS PARAMETERS
902 ALLOWABLE STORM INLET TYPES AND CAPACITY FACTORS
903 STORM SEWER ENERGY LOSS COEFFICIENTS

LIST OF FIGURES
901 HYDRAULIC PROPERTIES OF CIRCULAR PIPE
902 HYDRAULIC PROPERTIES OF HORIZONTAL ELLIPTICAL PIPE
903 HYDRAULIC PROPERTIES OF ARCH PIPE
904 ENERGY LOSS COEFFICIENT IN STRAIGHT THROUGH MANHOLE
905 ALLOWABLE INLET CAPACITY OF TYPE 2 CATCH BASIN - CONTINUOUS GRADE CONDITION
906 ALLOWABLE INLET CAPACITY OF TYPE 4R CATCH BASIN - CONTINUOUS GRADE CONDITION
907 ALLOWABLE INLET CAPACITIES - SUMP CONDITION
908 EXAMPLE PROBLEM: STORM SEWER PROFILE

December 2, 1996  Storm Sewer Systems 901
EXAMPLE PROBLEM: SCHEMATIC DRAWING OF STORM SEWER SYSTEM

EXAMPLE PROBLEM: STORM SEWER HYDRAULIC CALCULATIONS (STANDARD FORM - 3)
SECTION 900
STORM SEWER SYSTEM

901 INTRODUCTION

A storm sewer system consists of a series of pipes, manholes, and inlets which generally convey storm runoff from streets (gutter flow) to open channels or detention facilities. Storm sewers are generally utilized when the flow carrying capacity of a street (gutter) is exceeded by the calculated storm runoff contributing to the said street (gutter). Inlets to the storm sewer are sized to reduce the amount of street (gutter) flow to a level where the downstream street (gutter) flow is not exceeded before the location of the next inlet. Manholes in the sewer system are provided to allow access to the storm sewer for inspection and maintenance of the storm sewer.

The size of the storm sewer system is generally governed by the minor storm flows. This is a result of the incremental flow capacity between the allowable street flow during major and minor storms being generally greater than the incremental difference in the peak runoff from major and minor storms. In addition, the storm sewer system will naturally carry some runoff in excess of the required minor storm capacity during major storms due to natural surcharging of the storm sewer system.

There are conditions, however, when the storm sewer system design will be governed by the major storm flows. A partial listing of some of the possible situations are as follows:

1. Locations where street flow is collected in a sump with no allowable overflow capacity.
2. Locations where the street cross-section is such that the allowable depth of flow in the street is limited to the curb height (i.e. elevated streets with negative slopes at the ROW line).
3. Locations where the desired major storm flow direction is not reflected by the street flow direction during a major storm (i.e. flow splits at intersections).
4. Locations where the subject storm sewer system is accepting flow from an upstream storm sewer system or branch which is designed for major storm capacity.
5. Regional storm sewers.

The storm sewer system designer must be aware that if a storm sewer is to be designed to carry major storm flows, then the inlets to the storm sewer must be designed accordingly.

902 DESIGN PARAMETERS

902.1 ALLOWABLE STORM SEWER CAPACITY

The storm sewer system shall be designed to convey a part or all of the minor or major storm (design storm) under open channel or surcharged (pressure flow) conditions. The storm sewer shall be considered surcharged when the depth of flow (hydraulic grade line - HGL) in the storm sewer is greater than eighty percent of full flow depth. The maximum level of surcharging for the capacity analysis shall be limited to maintaining the HGL to one foot below the final grade above the storm...
sewer at all locations. Special site conditions that warrant additional surcharging will require locking type manhole covers or grated covers and will be reviewed on a case by case basis.

The energy grade line (EGL) and HGL shall be calculated to include all hydraulic losses including friction, expansion, contraction, bend, and junction losses. The methods for estimating these losses and for calculating the EGL and HGL are presented in the following sections.

902.2 ALLOWABLE STORM SEWER VELOCITY

The maximum allowable storm sewer velocity is dependent on many factors including the type of pipe, the acceptable water level during the pipe design life, proposed flow conditions (open channel versus pressure flows), and the type and quality of construction of joints, manholes, and junctions. In consideration of the above factors, the maximum velocity in all storm sewers shall be limited to 20 fps.

The need to maintain a self-cleaning storm sewer system is recognized as a goal to minimize the costs for maintenance of storm sewer facilities. Sediment deposits, once established, are generally difficult to remove without pressure cleaning equipment. However, the infrequency of storm runoff also possesses a problem in obtaining flows large enough to maintain the self-cleaning quality of the design. Thus, a balance must be drawn between obtaining a self-cleaning system and constructing a reasonably sized and sloped storm sewer.

A generally accepted criteria is to maintain a minimum velocity of 3 fps at half or full conduit flow conditions. At half full, the storm sewer will flow under open channel flow conditions and thus, the velocity in a given storm sewer is governed by the pipe slope. However, storm sewers generally cannot be constructed at slopes less than 0.25 percent and maintain a smooth even invert. Therefore, the minimum allowable storm sewer slope shall be 0.25 percent.

902.3 MANNING'S ROUGHNESS COEFFICIENT

All storm sewer system hydraulic calculations shall be performed using Manning's Formula (see Equation 801). Manning's roughness factor or "n" value is determined based on the surface roughness of the storm sewer pipe material. In addition, for a given pipe material, Manning's roughness coefficient theoretically varies based on depth of flow in the pipe. For the purposes of this MANUAL, Manning's roughness coefficient is assumed to be constant for all depths of pipe flow.

Various pipe manufacturers have determined Manning's roughness coefficients for use with their specific product. However, for storm sewer hydraulic design, Manning's roughness coefficient should also account for additional friction losses from pipe joints, potential debris and sediment in the storm runoff, and the pipe interior surface condition over the entire design life of the pipe. Therefore, presented on Table 901 are the Manning's roughness coefficients to be used for all storm sewer design and analysis prepared in accordance with this MANUAL.

902.4 STORM SEWER LAYOUT

The layout of a storm sewer system is governed by many factors including existing utility locations; street alignment, inlet placement, outfall location, and surface topography. These factors place constraints around which the storm sewer must be designed and still operate as an effective system. In addition, these constraints have inherent priorities as to which constraint takes present over the other constraints (i.e. relocating water lines versus designing around sanitary sewers).
The storm sewer system, however, must also take priority when other constraints would cause undesirable hydraulic conditions to occur in the storm sewer system, if the system were to be designed around the constraint. Therefore, limits are necessary in the storm sewer layout to prevent undesirable hydraulic conditions. The limits on vertical and horizontal alignments are presented in the following sections.

902.4.1 Vertical Alignment

902.4.1.1 Minimum and Maximum Cover

The required cover over a storm sewer pipe is dependent on many factors including the design pipe strength, pipe size, and cover material. For practical purposes, the storm sewer should be protected from potential surface disturbances and displacements. Therefore, the minimum allowable cover over the storm sewer pipe shall be one foot or greater at any point along the pipe. The maximum cover is contingent upon the design pipe strength.

902.4.1.2 Manhole

Manholes are used to provide a hydraulically efficient transition section at changes in the storm sewer system. Manholes are also used to provide access to the storm sewer for maintenance purposes. Therefore, to maintain hydraulic efficiency and adequate maintenance access, a manhole shall be located at all changes in pipe size, direction (including bends where allowed), elevation and grade for all pipes with a diameter (or rise dimension) of less than 48 inches. Manholes will be required at inlet laterals and at the end of public lines. For pipes with a diameter (or rise dimension) of 48 inches and greater, the designer shall consult with Washoe County for location of manholes based on hydraulic and maintenance considerations. In addition, the maximum spacing between manholes for various pipe sizes shall be in accordance with Table 901.

902.4.2 Horizontal Alignment

The horizontal alignment of storm sewers shall generally be straight between manholes. However, if approved by the local entity, the storm sewer may be constructed with curvilinear alignment by the pulled-joint method, pipe bends, or using radius pipe. The radius of curvature for pulled-joint construction is dependent on the pipe length, diameter, and the permitted opening in the joint. The maximum allowable joint pull for pulled-joint construction shall be as presented in Table 901. For radius pipe, the maximum bevel angle shall not exceed 5-degrees.

902.4.3 Utility Clearances

Storm sewers should generally be located to minimize potential contamination of water supply and sanitary sewer mains from or to storm sewers and minimize the disturbance of existing and/or future water supply and sanitary sewer mains due to storm sewer construction. This should be accomplished through distancing the storm sewer from water and sanitary sewer mains where at all possible or adding additional leakage protection at joints. The requirements for utility separations are presented in the following sections. Additional requirements may be imposed by the local utility companies. The storm sewer designer is responsible for adhering to the more stringent criteria.
WASHOE COUNTY
HYDROLOGIC CRITERIA AND DRAINAGE DESIGN MANUAL

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904.3.1 **Water Mains**

Where a storm sewer or storm inlet run crosses a water main or comes within ten horizontal feet (clear distance) of a water main, the storm sewer pipe shall be located a minimum of eighteen inches clear distance vertically below the water main. If this clear distance cannot be obtained, then the storm sewer pipe section must be designed and constructed so as to protect the water main. Minimum protection shall consist of a twenty foot section of storm sewer centered over the water main being encased in concrete at least four inches thick. In addition, water tight joints shall be used within the twenty foot section. In no case shall the clearance between the water main and the storm sewer be less than twelve inches.

902.4.3.2 **Sewer Mains**

Where a storm sewer or storm inlet run crosses a sanitary sewer main or comes within ten horizontal feet (clear distance) of each other, the storm sewer pipe shall be located a minimum of twelve inches clear distance vertically above or below the sanitary sewer main. If this clear distance cannot be obtained, then the sanitary sewer pipe section must be designed or improved to provide a structurally sound sewer main. This may be accomplished by either of the following methods:

a) Install one length of structural sanitary sewer pipe at least eighteen feet long centered at the storm sewer. Joints between the sanitary sewer pipe and the structural pipe shall be encased in a concrete collar at least four inches thick and extending at least six inches either side of the joint.

b) Concrete encase the sanitary sewer with concrete at least four inches thick and extending a distance of ten feet either side of the storm sewer.

Special additional backfill or structural provisions may also be required to preclude settling and/or failure of the higher pipe (storm sewer or sanitary sewer). Also, all distances shall be measured from outside pipe edge to outside pipe edge.

902.5 **ALLOWABLE STORM INLET TYPES AND CAPACITY FACTORS**

Standard storm inlet types have been adopted as part of the STANDARD DETAILS FOR PUBLIC WORKS CONSTRUCTION for Washoe County. The allowable use of these storm inlet types is presented on Table 902. Also presented on Table 902 are the allowable inlet capacity factors for each of the standard inlets. These capacity factors are applied to the theoretical capacity of the inlets to account for conditions which decrease the capacity of the standard inlets. These conditions include plugging from debris and sediment, pavement overlaying, variations in design assumptions, and the general deterioration of the inlet conditions over time. Storm inlet hydraulics are discussed in Section 905.

903 **CONSTRUCTION STANDARDS**

The following sections present the standards for construction of storm sewer systems. Detailed specifications for specific parts of the following standards are presented in the STANDARD SPECIFICATIONS FOR PUBLIC WORKS CONSTRUCTION (latest edition) for Washoe County including all future amendments. Where these detailed STANDARD SPECIFICATIONS are available, they shall be considered as an addition to the generalized standards presented in the following sections.
The designer shall be responsible for referencing the most current version of the STANDARD SPECIFICATIONS.

903.1 STORM SEWER PIPE

903.1.1 Size

The minimum allowable pipe size for storm sewers is dependent upon a practical size and length for maintenance and inspection of the storm sewer. Therefore, the minimum pipe size for storm inlet laterals to the storm sewer mains shall be 15 inches and for storm sewer mains shall be 18 inches in diameter or equivalent for non-round pipe.

903.1.2 Material and Shape

The material and shape of the storm sewer will be in accordance with the STANDARD SPECIFICATIONS. Public storm sewers shall be reinforced concrete pipe.

Square or rectangular Reinforced Concrete Box (RCB) pipe in accordance with ASTM C-789 or C-850 is acceptable for use in storm sewer construction.

Other pipe materials may be used for private storm sewer construction upon approval by the County. Documentation must be submitted for review which shows that the subject pipe material has a design life similar to the above materials and that the interior lining, if any, will maintain the design Manning's roughness coefficient value for the life of the pipe material.

903.1.3 Joint Sealants and Gaskets

Pipe joints for concrete pipe are generally sealed with either joint sealants or gaskets. Joint sealants are generally mastics which consist of bitumen and inert mineral fillers or joint mortar. The mastic is easily applied in the field but may not always provide a water tight joint. Joint gaskets are generally made of rubber and are either cemented to, recessed in, or rolled on the pipe joint. These gaskets generally provide a water tight seal and can withstand some internal pressure. Since all storm sewers within Washoe County will be generally designed for pressure flow conditions, rubber gasket joints shall be used for all installations where the pressure head exceeds 5 feet for the design flow. The pressure head is computed as the difference between the hydraulic grade line and the inside top of pipe.

903.2 MANHOLES

Manholes shall be constructed in accordance with the STANDARD DETAILS for the Washoe County area. Pipes may be directly cast into the manhole base. The County may require gasketed joints, locking type manhole covers, and/or grated manhole covers for pressure flow conditions.

903 STORM SEWER INLETS

Storm sewer inlets shall be constructed in accordance with the STANDARD DETAILS for the Washoe County area.
903.4 STORM SEWER OUTLETS

Storm sewer outlets shall be constructed with outlet erosion protection for discharges to channels with unlined bottoms in accordance with the following:

<table>
<thead>
<tr>
<th>Outlet Velocity (fps)</th>
<th>Required Outlet Protection</th>
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<tr>
<td>less than 5</td>
<td>Riprap Protection (Section 807.3)</td>
</tr>
<tr>
<td>between 5 and 15</td>
<td>Riprap Protection (Section 807.3) or Energy Dissipater (Section 1202.2)</td>
</tr>
<tr>
<td>greater than 15</td>
<td>Energy Dissipater (Section 1202.2)</td>
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</tbody>
</table>

For channels with lined bottoms, the outlet discharge velocity must not exceed the maximum allowable channel velocity without an energy dissipation structure.

904 STORM SEWER HYDRAULICS

Presented in this section are the general procedures for hydraulic design and evaluation of storm sewers. The user is assumed to possess a basic working knowledge of storm sewer hydraulics and is encouraged to review the text books and other technical literature available on the subject.

904.1 HYDRAULIC ANALYSIS

Storm sewers in Washoe County will typically be designed for pressure flow conditions. However, portions of the storm sewer may also act like open channels (i.e. very steep slopes, segments of storm sewers discharging to open channels). Therefore, the storm sewer capacity analysis must account for changes in flow conditions (open channel versus pressure flow) in the hydraulic grade line (HGL) and energy grade line (EGL) calculations. Both the HGL and the EGL for the design flow shall be included on all final storm sewer improvement construction plans.

Many computer programs are now available which perform hydraulic computations for storm sewer hydraulics. In order for Washoe County to be assured that said programs correctly compute storm sewer hydraulics in accordance with the provisions of this MANUAL, Washoe County will maintain a list of computer programs approved for use in the County for storm sewer hydraulic computations. Any program not on this list may be eligible for inclusion upon submittal of backup documentation and other information as may be deemed necessary by the County. Any such program, however, must be able to accommodate the use of the loss coefficients provided herein.

904.1.1 Pressure Flow Analysis

When a storm sewer is flowing under a pressure flow condition, the energy and hydraulic grade lines may be calculated using the pressure-momentum theory. The capacity calculations generally proceed from the storm sewer outlet upstream accounting for all energy losses. These losses are added to the EGL and accumulate to the upstream end of the storm sewer. The HGL is then determined by subtracting the velocity head, H_v, from the EGL at each change in the EGL slope. To assist in accounting for and computing the energy losses and
EGL, a pressure storm sewer computation form (Standard Form 6) is provided in this MANUAL.

904.1.2 Partial Full Flow Analysis

When a storm sewer is not flowing full, the sewer acts like an open channel and the hydraulic properties can be calculated using open channel techniques. For convenience, charts for various culvert shapes have been developed by the pipe manufacturers for calculating the hydraulic properties associated with partial full flow (Figures 901, 902, and 903). The data presented assumes that the friction coefficient, Manning’s roughness coefficient, does not vary throughout the depth.

For partial full flow analysis, the HGL and EGL are parallel when the flow reaches normal depth. The designer should check the available energy at all junctions and transitions to determine whether or not the flow in the storm sewer will be pressurized due to backwater effects even if the design flow is less than the full flow capacity of the storm sewer. In this case, a hydraulic jump will occur and the pipe should be structurally designed to accommodate the jump.

904.2 ENERGY LOSS CALCULATIONS

Presented in this section are the energy loss equations and coefficients for use in the hydraulic analysis of storm sewer systems. All storm sewer analysis in Washoe County shall account for energy losses using the equations and coefficients in this section.

904.2.1 Pipe Friction Losses

Pipe friction losses shall be calculated using an equation for full flow conditions derived from Manning’s equation as follows:

\[ S_f = \frac{\phi H}{R^{1.33}} \]  

(901)

Where  
- \( S_f \) = Friction slope (feet/feet)  
- \( H \) = Velocity head (feet)  
- \( R \) = Hydraulic radius (feet)

The flow coefficient, \( \phi \), is related to the Manning’s "n" value for the pipe as follows:

\[ \phi = \frac{2g n^2}{2.21} \]  

(902)

Where  
- \( n \) = Manning’s roughness coefficient (Dimensionless)  
- \( g \) = Gravitational acceleration 32.2 ft/sec²

The total head loss due to friction in a length of pipe is then equal to the friction slope times the pipe length.
Transition Losses

Generally, between the inlet and outlet, storm sewer flow encounters a variety of configurations in the flow passageway such as changes in pipe size, branches, bends, junctions, expansions, and contractions. These shape variations impose losses in addition to those resulting from pipe friction. Transition losses are the result of fully developed turbulence and are expressed as:

\[ H_L = K \left( \frac{V^2}{2g} \right) \]  

(903)

Where

- \( H_L \) = Head loss (feet)
- \( K \) = Loss coefficient
- \( \frac{V^2}{2g} \) = Velocity head (feet)
- \( g \) = Gravitational acceleration = 32.2 ft/sec\(^2\)

The following is a discussion of a few of the common types of Transition losses encountered in sewer system design. The reader is referred to standard hydraulic references and text books for additional transition loss discussion. In the following equations, subscripts 1 and 2 denote the upstream and downstream sections, respectively.

Expansion Losses

Expansion in a storm sewer conduit will result in a shearing action between the incoming high velocity jet and the surrounding sewer boundary. As a result, much of the kinetic energy is dissipated by eddy currents and turbulence. The head loss is expressed as:

\[ H_L = K_e \left( \frac{V^2}{2g} \right) \left( \frac{A_2}{A_1} \right)^2 - 1 \]  

(904)

in which \( A \) is the cross-sectional flow area, \( V \) is the average flow velocity, and \( K_e \) is the expansion loss coefficient. The value of \( K_e \) varies from about 1.0 for a sudden expansion to about 0.2 for a well designed expansion transition. Table 903(1 of 3) presents the expansion loss coefficients for various flow conditions.

Contraction Losses

The form loss due to contraction is expressed as:

\[ H_L = K_c \left( \frac{V^2}{2g} \right) \]  

(905)

where \( K_c \) is the contraction loss coefficient. \( K_c \) varies from about 0.4 for large pipe size differences (> 10:1) to about 0.1 for minor pipe size differences. Table 903(1 of 3) presents the contraction loss coefficients for various flow conditions.
904.2.2.3 Bend Losses

The head losses for bends, in excess of that caused by an equivalent length of straight pipe, is expressed as:

\[ H_L = K_b \left( \frac{V^2}{2g} \right) \]  \hspace{1cm} (906)

in which \( K_b \) is the bend loss coefficient. The bend coefficient has been found to be a function of, (a) the ratio of the radius of curvature of the bend to the width of the conduit, (b) deflection angle of the conduit, (c) geometry of the cross section of flow, and (d) the Reynolds Number and relative roughness. Tables showing the recommended bend loss coefficients are presented in Table 903(2 of 3).

904.2.2.4 Junction and Manhole Losses

A junction occurs where one or more branch sewers enter a main sewer, usually at manholes. The hydraulic design of a junction is in effect the design of two or more transitions, one for each flow path. Allowances are made for head losses due to the impacts at the junctions. The head loss at a junction is expressed as:

\[ H_L = \frac{V^2}{2g} - K_j \left( \frac{V^1}{2g} \right) \]  \hspace{1cm} (907)

where \( V_o \) is the outfall flow velocity, \( V_i \) is the inlet velocity, and \( K_j \) is the junction coefficient. Because of the difficulty in evaluating hydraulic losses at junctions due to the many complex conditions of pipe size, geometry of the junction and flow combinations, a simplified table of loss coefficients has been prepared. Table 903(3 of 3) presents the recommended energy loss coefficients for typical manhole or junction conditions that will be encountered in the urban storm sewer system.

For straight flow through manholes (single pipe with no inlet laterals), the head loss through the manhole is similar to a pipe bend. For this condition, the head loss at the manhole is expressed as:

\[ H_L = K_m \left( \frac{V^2}{2g} \right) \]  \hspace{1cm} (909)

in which \( K_m \) is the manhole loss coefficient. Figure 904 presents values of \( K_m \) for various deflection angles.

904.2.2.5 Inlet Losses

When runoff enters a storm sewer system from locations other than street inlets (i.e. open channels) an energy loss occurs at the entrance in the form of a contraction loss. The head loss at storm sewer entrances is expressed as:
\[ H_L = K_i \left( \frac{V_i^2}{2g} \right) \]  

(909)

in which \( K_i \) is the inlet (entrance) loss coefficient. The coefficient \( K_i \) is the same as the \( K_e \) coefficient used for the entrance loss calculation for culverts. A list of various \( K_i \) \( (K_e) \) coefficients is presented in Table 1101 in Section 1100.

904.2.2.6 Outlet Losses

When the storm sewer system discharges into open channels, additional losses occur at the outlet in the form of expansion losses.

For most storm sewer outlets, the flow velocity in the storm sewer is greater than the allowable or actual flow velocity in the downstream channel. Therefore, energy dissipating facilities are used to remove excess energy from the storm sewer flow. In addition, the alignment of the storm sewer at the outlet may not be the same as the downstream channel. Therefore, energy is lost in changing the flow direction between the storm sewer to the downstream channel. The head loss at storm sewer outlets is expressed as:

\[ H_L = K_o \left( \frac{V_o^2}{2g} \right) \]  

(910)

where \( K_o \) is the outlet loss coefficient. For all storm sewer outlets, an outlet loss coefficient \( K_o \) of 1.0 shall be used.

The outlet loss shall be added to the downstream EGL and compared to the critical depth EGL in the storm sewer. The larger (higher) EGL shall be used for starting the storm sewer hydraulic calculations.

904.2.2.7 Drop Manholes

Energy losses in drop manholes depends on the amount of drop and the size of pipes entering and leaving the manhole. For purposes of this MANUAL, energy losses in drop manholes shall be computed as two 90° bend losses when the invert of the upstream pipe is higher than the crown of the downstream pipe. When the upstream pipe invert is lower than the downstream pipe crown the above computed energy loss shall be prorated by the ratio of difference of invert elevations to the downstream pipe size.

905 STORM INLET HYDRAULICS

Presented in this section is discussion and criteria for sizing and locating storm sewer inlets. In Washoe County the allowed standard inlet types are presented in Table 902. For capacity calculations, the inlets are further classified as being on a "continuous grade" or in a "sump". The term "continuous grade" refers to an inlet so located that the grade of the street has a continuous slope past the inlet and therefore ponding does not occur at the inlet. The sump condition exists whenever water is restricted to the inlet area because the inlet is located at a low point. A sump condition can occur at locations.
such as a change in grade of the street from positive to negative or at an intersection due to the crown slope of a cross street.

The procedures and basic data used to define the capacities of the standard inlets under various flow conditions were obtained from IZZARD, 1977 and LINSLEY 1964. The procedure consists of defining the amount and depth of flow in the gutter and determining the theoretical flow interception by the inlet. To account for effects which decrease the capacity of the various types of inlets, such as debris plugging, pavement overlaying and variations in design assumptions, the theoretical capacity calculated for the inlets is reduced by the factors presented in Table 902.

Allowable inlet capacities for the standard inlets have been developed and are presented in Figures 905 and 906 for "continuous grade" and Figure 907 for "sump" conditions. The allowable inlet capacity is dependent on the depth of flow as determined from the street capacity calculations (for continuous grade inlets) or on the depth of ponding necessary to accept the desired flow rate (sump conditions). These depths must be kept at or below the allowable flow or ponding depths as presented in Section 304.4.

905.1 INLETS ON CONTINUOUS GRADE

For the "continuous grade" conditions (Figures 905 and 906), the capacity of an inlet is dependent upon many factors including gutter slope, depth of flow in the gutter, height and length of curb opening, street cross slope, and the amount of depression at the inlet. In addition, all of the gutter flow will not be intercepted and some flow will continue past the inlet area ("inlet carryover"). The amount of carryover must be included in the drainage facility evaluation as well as in the design of the inlet.

The hydraulic calculations for the curb openings were based on the Izzard, 1977 method of capacity calculations up to about 1.3 feet in depth. At greater depths, the curb opening will act as an orifice with C = 0.7 (Linsley, 1964). For the grated inlets, the John Hopkins University method (Hopkins, 1956) was used to determine the theoretical inlet capacity.

905.2 INLETS IN A SUMP CONDITION

The capacity of an inlet in a sump condition (Figure 907) is dependent on the depth of ponding above the inlet. Typically, the problem consists of determining the quantity or length of inlets required to reduce the depth of ponding to an acceptable level. The designer should be aware that several inlets or additional inlet length will generally be required when an inlet must be designed to accommodate major storm flow. Also, additional continuous grade inlets may be necessary upstream of the sump location to reduce the depth of ponding at the sump inlets to an acceptable level during major storm events.

At all sump locations, the design shall include provisions for emergency overflow if the inlets become completely plugged. The emergency overflow shall be paved or riprapped and include an easement for access and maintenance. If an emergency overflow area is not available, then flanker inlets on each side of the sump inlet shall be provided. The flanker inlets shall be placed so as to provide in the two flanker inlets the same total capacity as the sump inlet. The water surface used in design of the flanker inlets shall be the allowable depth of ponding on the sump inlet. In no case shall the flanker inlets be located nearer than 10 feet to the sump inlet or further than 50 feet from the sump inlet.
The hydraulic calculations for the curb opening were based on the weir equation with \( C = 3.3 \) for shallow depths and were transitioned to the orifice equation with \( C = 0.7 \) (Linsley, 1964) for deeper flow depths. For the grated inlets, a \( C = 0.6 \) was used in the orifice equation.

905.3 INLET SPACING

The optimum spacing of storm inlets is dependent upon several factors, including traffic requirements, contributing land use, street slope, and distance to the nearest outfall system. The suggested sizing and spacing of the inlets is based upon an interception rate of 70 percent to 80 percent. This spacing has been found to be more efficient than a spacing using 100 percent interception rate. Using the suggested spacing, only the most downstream inlet in a development would be designed to intercept 100 percent of the flow. Also, considerable improvement in overall inlet system efficiency can be achieved if the inlets are located in the sumps created by street intersections, if possible, without overloading of the sump inlets.

Inlets shall be installed at low points of vertical curves, at street intersection sumps, and at sufficient intervals to intake the design peak flow so that said flows will not interfere with traffic or flood adjoining property.

905.4 INLET CAPACITY FOR MAJOR STORM ANALYSIS

Inlet capacities may be read directly from Figure 907 for sump conditions for major storm events. However, for the continuous grade condition, the use of the allowable street capacity charts in Section 1000 may result in an actual flow depth of less than that indicated due to the restriction of flow in the right-of-way limits. Therefore, for the major storm inlet capacity analysis, the continuous grade inlet capacities shall be reduced by an additional fifteen percent.

906 STORM SEWER SYSTEM DESIGN

Presented in this section is the design procedures for a storm sewer system from preliminary design consideration to final design. A typical drainage system within a development consists of flow in the storm sewer and allowable flow in the gutter, which combined would carry both the minor and major storm flows. The design flow for the storm sewer is generally governed by the amount of runoff in excess of the minor storm street capacity. In some cases, however, the amount of runoff from the major storm in excess of the major storm street capacity may be larger than the excess from the minor storm. In this case, the storm sewer and inlets would need to be designed to accommodate the excess major storm flow. To assist in this analysis, the allowable minor and major storm street capacity should be determined prior to sizing of the storm sewer system. (See Section 1000/Streets).

906.1 INITIAL STORM SEWER SIZING

Preliminary street grades and cross sections must be available to the storm sewer designer so he can calculate the allowable carrying capacity for these streets. Beginning at the upper end of the basin in question, the designer should calculate the quantity of flow in the street until the point is reached at which the allowable carrying capacity of the street matches the design runoff. Initiation of the storm sewer system would start at this point if there is no alternate method of removing runoff from the street surface. Removal of all the street flow by the storm sewer system is not required except at sump areas. However, the sum of the flow in the sewer plus the flow in the street must be less than or equal to the allowable capacity of the street and storm sewer.
For preliminary sizing purposes, the diameter, type of pipe, and pipe slope may be determined assuming a full flow pipe capacity based on slope-area calculations. If large energy losses are anticipated (i.e., large junctions, bends), then the preliminary pipe size may need to be upsized to assure that the final pressure calculations result in an acceptable HGL and EGL. In some instances, a profile may be required to check utility conflicts or to assure compatibility with the downstream drainage system.

At this point, the preliminary system should be reviewed to check that the system is hydraulically efficient as well as to locate segments which have potentially large energy losses. These segments should be examined carefully and options explored to minimize the energy loss. The designer should also check potential inlet locations to assure that the required inlet capacity is not larger than the allowable inlet capacities.

906.2 FINAL STORM SEWER SIZING

Final design consists of the preparation of plan, profiles, and specifications for the storm sewer system in sufficient detail for construction. The first step consists of the review and verification of the basic data, hydrologic analysis, and storm sewer inlet sizing performed for the preliminary design. Plan and profile drawings are prepared containing the basic data. Drainage sub-basins are revised as necessary, and the design flood peaks recalculated. The storm sewer and inlets are then sized taking into account actual street and storm sewer grades, locations of existing and proposed utilities, and the design of the downstream drainage system. The calculations also include the determination of the hydraulic and energy grade lines. The manholes, junction structures, or other appurtenant structures must be evaluated for energy losses. If special transitions are required to reduce losses, the structural design of the facilities must include these requirements when detailing the structures.

907 EXAMPLE APPLICATION

907.1 INTRODUCTION

The following example presents the hydraulic analysis of a storm sewer system and demonstrates the use of the energy loss coefficients and the Hydraulic Calculations Standard Form 6.

The following procedure is based on full-flow pipe conditions. If the pipe is flowing substantially full (i.e., greater than 80 percent), the following procedures can be used with minimal loss of accuracy. However, the designer is responsible for checking the assumptions (i.e., check for full flow) to assure that the calculations are correct.

907.2 EXAMPLE: STORM SEWER HYDRAULIC ANALYSIS

Problem: Compute the Energy Grade Line (EGL) and Hydraulic Grade Line (HGL) for Rose Subdivision shown in Figure 908. This example problem utilizes allowable street flow calculations performed in Section 1007.2 and runoff calculations performed in Section 709.1 Assume the water surface elevation at the outlet in the detention basin (Point 7) is 4922.0 feet.

Solution:

Step 1: Based on the allowable street flow calculations performed in the example problem in Section 1007.2, draw a plan view of the necessary storm sewer system (See Figure 909).
Step 2: Determine the location that the calculations will begin and the direction in which they will proceed. In this example, assume the normal depth at the storm sewer outlet is greater than the critical depth \( d_c > d_o \), so the calculations will begin at Point 7 and proceed upstream.

Step 3: Enter the known data into Standard Form 6 (See Figure 910). In this example, the assumed known data is input in columns 1, 2, 6, 10, and 27 and the first row of column 4.

Step 4: Assume a storm sewer type and diameter for the first reach of the storm sewer system and fill in the first row of columns 3, 8, 11 and 12.

Assume \( D_{v4} = 1.5 \) feet

The storm sewer velocity is:

\[
V_{7-4} = \frac{Q}{A} = \frac{7.7}{\pi \left( \frac{1.5}{2} \right)^2} = 4.4 \text{ fps}
\]

and the velocity head is:

\[
H_{V7-4} = \frac{V^2}{2g} = \frac{4.4^2}{2 \times 32.2} = 0.3 \text{ feet}
\]

Step 5: Determine the starting HGL and EGL elevations.

As previously mentioned, the starting HGL is:

\[
HGL_7 = 4922.0
\]

The energy, or head, loss at the storm sewer outlet is:

\[
H_{LO} = K_o \times H_{V7-4} = 1 \times 0.3 = 0.3 \text{ feet}
\]

The initial EGL will be:

\[
EGL_7 = HGL_7 + H_{LO} = 4922.0 + 0.3 = 4922.3 \text{ feet}
\]

Input the starting HGL (HGL_7) and EGL (EGL_7) above the first row of columns 24 and 25.

Step 6: Assume a value for the upstream invert elevation of the first storm sewer reach, and fill the first row of columns 5 and 7.

Assume the storm sewer invert elevation at Design Point 4 is 4619.0 feet. The slope in the first reach will be:

\[
S_{7-4} = \frac{4919 - 4918}{100} = 0.01 \text{ ft/ft}
\]
Step 7: Calculate the friction slope for this reach.

The flow coefficient is:

\[
\phi = \frac{2gn^2}{2.21} = \frac{2 \times 32.2 \times 0.013^2}{2.21} = 0.0049
\]

and the hydraulic radius is:

\[
R_{7/4} = \frac{D_{7/4}}{4} = \frac{1.5}{4} = 0.375 \text{ feet}
\]

The friction slope is:

\[
S_{f,7/4} = \frac{\phi H_v}{R^{1.53}} = \frac{0.0049 \times 0.3}{(0.375)^{1.53}} = 0.005 \text{ ft/ft}
\]

Enter the flow coefficient and the friction slope into the first row of columns 9 and 13, respectively.

Step 8: Compute the average friction slope and input this value into column 14 of the first row.

The average friction slope is the average value of \( S_f \) for the current reach. When analyzing losses across long transitions, the average friction slope is the \( S_f \) for the upstream reach and the preceding reach averaged together.

\[
\text{Ave } S_f = S_{n,4} = 0.005 \text{ ft/ft}
\]

Step 9: Calculate the energy loss due to pipe friction in the first reach.

\[
H_{7/4} = (\text{Ave. } S_{7/4}) (L) = 0.005 \times 100 = 0.5 \text{ feet}
\]

Enter \( H_{7/4} \) in the first row of column 15.

Step 10: Determine the EGL and HGL at the upstream station.

\[
\text{EGL}_4 = \text{EGL}_7 + H_{7/4} = 4922.3 + 0.5 = 4922.8 \text{ feet}
\]

\[
\text{HGL}_4 = \text{EGL}_4 - H_v = 4922.8 - 0.3 = 4922.5 \text{ feet}
\]

Enter EGL\(_4\) in the first row of column 22 and HGL\(_4\) in the first row of column 23.

Step 11: Check that full flow still exists (i.e., WSEL. > 0.8D)

\[
\text{Flow Depth} = \text{HGL}_4 - \text{Invert elevation at Design Point 4}
\]
Flow Depth = 4922.5 - 4919.0 = 3.5 feet

0.8D = 0.8 * 1.5 = 1.2 feet

Since 3.5 > 1.2, pressure flow exists. Enter "yes" in the first row of column 26.

Step 12: Assume a storm sewer type and diameter from Design Point 4 to Design Point 6 (Reach 2) and a storm sewer invert elevation at Design Point 6, and fill in the second row of columns 3, 4, 5, 7, 8, 9, 11, and 12.

Assume D_{4-6} = 1.5 feet

Upstream Invert Elevation = 4919.4

A_{4-6} = \pi r^2 = \pi (0.75)^2 = 1.77 \text{ ft}^2

\phi = \frac{2g}{2.21} = 0.0049

V_{4-6} = \frac{Q}{A} = \frac{2.5}{1.77} = 1.4 \text{ fps}

H_v = \frac{V^2}{2g} = \frac{1.4^2}{2 * 32.2} = 0.1 \text{ feet}

Step 13: Check the controlling downstream flow condition for Reach 2. Compare the downstream flow condition for Reach 2 to the upstream flow condition for Reach 1. The highest value controls.

EGL_4 (Downstream of Reach 2) = D/S Invert E_4 + D + H_v

EGL_4 (Downstream of Reach 2) = 4919.0 + 1.5 + 0.1 = 4920.6 feet

EGL_4 (Upstream of Reach 1) = 4922.8 feet

Since EGL_4 (U/S of Reach 1) is greater than EGL_4 (D/S of Reach 2), the controlling downstream energy gradeline elevation is 4922.8 feet. If the downstream EGL of Reach 2 had been greater, this value would be the controlling EGL (and HGL) and entered in columns 24 and 25, respectively. Step 11 would be repeated in the next row down and the calculations would continue in this row with Step 13.

Step 14: Calculate the friction slope for this reach. The flow coefficient does not change.

The hydraulic radius is:

R_{4-6} = \frac{D_{4-6}}{4} = \frac{1.5}{4} = 0.375 \text{ feet}
The friction slope is:

\[ S_{t,x} = \frac{\phi H}{R^{1.33}} = \frac{0.0049 \times 0.1}{(0.375)^{1.33}} = 0.002 \text{ ft/ft} \]

Enter this value into the second row of column 13.

**Step 15:**
Since the transition is relatively abrupt, the average friction slope is equal to the friction slope of the upstream reach. Input this value into the second row of column 14.

\[ \text{Ave } S_t = S_t = 0.002 \text{ ft/ft} \]

**Step 16:**
Determine the head loss due to friction in the storm sewer in Reach 2.

\[ H_{t,x} = (\text{Ave } S_t) (L) = (0.002) (40) = 0.1 \text{ feet} \]

Enter this value in the second row of column 15.

**Step 17:**
Calculate transition energy losses. In this case, there is a transition loss due to the junction.

Assume Reach 2 enters the junction at Design Point 4 at a 45° skew to the main storm sewer alignment. From Table 903(D), the loss coefficient will be:

\[ k_j = 0.5 \]

and the transition loss at the junction will be:

\[ H_j = \frac{V^2}{2g} - K_j \left( \frac{V^2}{2g} \right) = 0.3 - (0.5)(0.1) = 0.3 \text{ feet} \]

Enter this value in the second row of column 17.

For columns 17 through 20, enter the K values acquired from the appropriate tables and figures and the head values calculated from Equations 903 through 910. Separate the loss coefficient, K, and the head value, H, by a slash (/).

**Step 18:**
Calculate the total energy loss and input this value into the second row of column 21.

\[ H_{\text{total}} = H_{t,x} + H_j = 0.1 + 0.3 = 0.4 \text{ feet} \]

**Step 19:**
Compute the EGL and the HGL at the upstream station (Design Point 6).

\[ \text{EGL}_u = \text{EGL}_4 + H_{\text{total}} = 4922.8 + 0.4 = 4923.2 \text{ feet} \]
HGL₆ = EGL₆ - Hᵢ = 4923.2 - 0.1 = 4923.1 feet

Enter EGL₆ and HGL₆ in the second row of columns 24 and 25, respectively.

Step 20: Check the full flow still exists.

Flow Depth = HGL₆ - Invert Elevation at DP6

Flow Depth = 4923.1 - 4919.4 = 3.7 feet

Since the flow depth is greater than 0.8*D (3.7 > 1.2), pressure flow exists, and "Yes" should be entered in the second row of column 27.

Step 21: Repeat Steps 11 through 19, as needed, to obtain the EGL and HGL for the entire storm sewer system. The results of this analysis are supplied by Figure 910, and the final EGL and HGL are plotted on Figure 908.

Note: The flow velocity in Reaches 2 and 3 under full flow conditions is less than 3 fps. Due to the small amount of flow needed to be carried in these reaches of storm sewer, a low velocity is unavoidable. If the storm sewer flow was not being controlled by the backwater conditions created by the detention basin, the flow velocities in Reaches 2 and 3 would be 4.9 fps, and 5.7 fps, respectively. These velocities should be sufficient to clean the storm sewer of sediment and debris.
A. MANNING'S ROUGHNESS COEFFICIENT (n):

<table>
<thead>
<tr>
<th>STORM SEWER TYPE</th>
<th>n</th>
</tr>
</thead>
<tbody>
<tr>
<td>CONCRETE</td>
<td>0.013</td>
</tr>
<tr>
<td>CORRUGATED METAL (CORRUGATED INTERIOR)</td>
<td>0.024</td>
</tr>
<tr>
<td>CORRUGATED METAL (SMOOTH LINED INTERIOR)</td>
<td>0.013</td>
</tr>
</tbody>
</table>

B. MANHOLE SPACING:

<table>
<thead>
<tr>
<th>EQUIVALENT PIPE SIZE (INCHES)</th>
<th>MAXIMUM ALLOWABLE DISTANCE BETWEEN MANHOLES</th>
</tr>
</thead>
<tbody>
<tr>
<td>12 to 48</td>
<td>300 feet</td>
</tr>
<tr>
<td>48 and larger</td>
<td>600 feet</td>
</tr>
</tbody>
</table>

C. MAXIMUM ALLOWED DEFLECTION FOR PULLED JOINT CONSTRUCTION

<table>
<thead>
<tr>
<th>PIPE DIAMETER (SPAN) (INCHES)</th>
<th>ALLOWED DEFLECTION (PULL) PER JOINT (INCHES)</th>
</tr>
</thead>
<tbody>
<tr>
<td>8 - 33</td>
<td>$\frac{1}{2}$</td>
</tr>
<tr>
<td>36 - 54</td>
<td>$%$</td>
</tr>
<tr>
<td>60 - 78</td>
<td>$%$</td>
</tr>
<tr>
<td>84 - 102</td>
<td>$%$</td>
</tr>
<tr>
<td>108 - 144</td>
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# ALLOWABLE STORM INLET TYPES AND CAPACITY FACTORS

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<td>0.7 for Grate/0.8 for Curb Opening</td>
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<td>SUMP</td>
<td>0.5 for Grate/0.7 for Curb Opening</td>
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**NOTES:**

1. C.G. = Continuous Grade
2. Standard Drawing Numbers refer to the "Standard Details for Public Works Construction," as adopted by Washoe County, City of Reno and City of Sparks, including future amendments.
3. Capacity factor is applied to the theoretical inlet capacity to obtain the allowable inlet capacity to account for factors which reduce actual inlet capacity.
STORM SEWER ENERGY LOSS COEFFICIENTS

(A) EXPANSIONS

* The angle $\theta$ is the angle in degrees between the sides of the tapering section.

(B) CONTRACTIONS

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NOTE: Losses due to angular expansions and abrupt contractions will normally only occur in retro-fit storm sewer design.

REFERENCE:
(A) Fluid Mechanics, Daugherty and Franzini, 1977
(B) Fluid Mechanics, Streeter and Wylie, 1979
STORM SEWER ENERGY LOSS COEFFICIENTS

(C) BENDS

I. Large Radius Bends
(Pipe Diameter > Bend Radius)

\[ K_b = 0.25 \left( \frac{\theta}{90} \right)^{0.5} \]

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Note: Head loss applied at P.C.

II. Sharp Radius Bends
(Pipe Diameter = Bend Radius)

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Note: Head loss applied at entrance to bend.
STORM SEWER ENERGY LOSS COEFFICIENTS

(D) JUNCTIONS

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NOTE: Head loss applied at exit of junction
HYDRAULIC PROPERTIES
HORIZONTAL ELLIPTICAL PIPE

REFERENCE:
"Concrete Pipe Design Manual," ACPA, 1970
HYDRAULIC PROPERTIES
ARCH PIPE

DEPTH OF FLOW

PROPORTION OF VALUE FOR FULL FLOW

VERSIO\N: December 2, 1996
REFERENCE:
"Concrete Pipe Design Manual," ACPA, 1970

WASHOE COUNTY
HYDROLOGIC CRITERIA AND DRAINAGE DESIGN MANUAL

FIGURE 903
ENERGY LOSS COEFFICIENT IN STRAIGHT THROUGH MANHOLE

NOTE: Head loss applied at outlet of manhole.

DEFLECTION ANGLE $\gamma$, DEGREES

Loss Coefficient, $K_m$

Bend at Manhole, no Special Shaping
Deflector
Curved
Bend at Manhole, Curved or Deflector
Manhole

VERSIO: December 2, 1998

FIGURE 904
ALLOWABLE INLET CAPACITY
CONTINUOUS GRADE CONDITION
CATCH BASIN TYPE 2

NOTE: Includes a capacity reduction factor of 0.8
ALLOWABLE INLET CAPACITY
CONTINUOUS GRADE CONDITION
CATCH BASIN TYPE 2

L = 10 ft

L = 12 ft

L = 14 ft

NOTE: Includes a capacity reduction factor of 0.8
ALLOWABLE INLET CAPACITY
CONTINUOUS GRADE CONDITION
CATCH BASIN TYPE 2

L = 16 ft

L = 18 ft

L = 20 ft

NOTE: Includes a capacity reduction factor of 0.8
ALLOWABLE INLET CAPACITY
CONTINUOUS GRADE CONDITION
CATCH BASIN TYPE 4-R

TOTAL FOR CURB AND GRATE OPENING (L = 33")

TOTAL FOR CURB AND GRATE OPENING (L = 33" x 2)

NOTE: 1) Includes a capacity reduction factor of 0.70 for grate and 0.80 for curb opening
ALLOWABLE INLET CAPACITY
CONTINUOUS GRADE CONDITION
CATCH BASIN TYPE 4-R

TOTAL FOR CURB AND GRATE OPENING (L = 33")

![Graph 1]

TOTAL FOR CURB AND GRATE OPENING (L = 33" x 2)

![Graph 2]

NOTE: 1) Includes a capacity reduction factor of 0.70 for grate and 0.80 for curb opening

VERSION: December 2, 1996
REFERENCE:
WRC ENGINEERING, INC.
NOTE: 1) INCLUDES CAPACITY REDUCTION FACTOR OF 0.5 FOR GRATE AND 0.7 FOR CURB OPENING
EXAMPLE PROBLEM:
Schematic Drawing Storm Sewer System

REACH 1
Q = 7.7 cfs
L = 40 ft
S = 1.0%

REACH 2
Q = 2.5 cfs
L = 40 ft
S = 1.0%

REACH 3
Q = 4.4 cfs
L = 40 ft
S = 1.0%
### Storm Sewer Hydraulic Calculations

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<th>A (ft²)</th>
<th>I</th>
<th>Q (cfs)</th>
<th>V (fps)</th>
<th>V²/g</th>
<th>H₀ (ft)</th>
<th>H₁ (ft)</th>
<th>H₂ (ft)</th>
<th>H₃ (ft)</th>
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Storm Sewer 'n' Value = 0.013
SECTION 1000
STREETS

TABLE OF CONTENTS

1000 INTRODUCTION 1001
1002 FUNCTION OF STREETS IN THE DRAINAGE SYSTEM 1001

1003 DRAINAGE IMPACT ON STREET MAINTENANCE 1001
1003.1 - Sheet Flow 1002
1003.2 - Gutter Flow 1002
1003.3 - Storm Duration 1002
1003.4 - Cross Flow 1003

1004 DRAINAGE IMPACT ON STREET MAINTENANCE 1003
1004.1 - Pavement Deterioration 1003
1004.2 - Sedimentation and Debris 1003

1005 STREET CLASSIFICATION AND ALLOWABLE FLOW DEPTH 1004

1006 HYDRAULIC EVALUATION 1004

1007 EXAMPLE APPLICATION 1005
1007.1 - Example: Allowable Flow in Street With 42-Foot Right-of-Way 1005

LIST OF FIGURES

1001 STREET FLOW CAPACITY - 42 FOOT ROW
1002 STREET FLOW CAPACITY - 52 FOOT ROW
1003 STREET FLOW CAPACITY - 80 FOOT ROW
1004 STREET FLOW CAPACITY - 100 FOOT ROW WITH MEDIAN

December 2, 1996
1000 Streets
1003.1 SHEET FLOW

Rainfall on the paved surface of a street or road must flow overland in what is referred to as sheet flow until it reaches a channel. Streets which have curbs and gutters become the channel, while on roads which have a drainage ditch, the ditch becomes the channel. In situations where the street is not inundated due to runoff originating from upgradient, the depth of sheet flow will be essentially zero at the crown of the street and will increase in the direction of the curb and gutter or drainage ditch.

Traffic interference due to sheet flow is by hydroplaning or by splash. Hydroplaning is the phenomenon of vehicle tires becoming supported by a film of water which acts as a lubricant between the pavement and the vehicle. This generally occurs at higher speeds associated with arterials and freeways and can result in loss of vehicle control. Drainage design can reduce the hydroplaning potential by increasing the street cross slope which drains the runoff more quickly.

Splashing of the sheet flows interferes with traffic movement by reducing visibility. The increase in cross slope of the street crown also reduces the splash potential. In general, a 2 percent cross slope is a desirable practical slope to promote swift removal of runoff while minimizing potential vehicle side-slippage from ice buildup during winter months.

1003.2 GUTTER FLOW

Water which enters a street as sheet flow from the pavement surface or as overland flow from adjacent land area will flow in the gutter and possibly a portion of the street section until reaching some outlet, such as a storm sewer inlet or a channel. As the flow progresses downstream and additional areas contribute to the runoff, the width of flow will increase and progressively infringe upon the traffic lane. If the roadway width allows vehicles to be parked adjacent to the curb, the flow width will have little influence on traffic capacity until it exceeds the width of the parking lane by several feet. However, on streets where parking is not permitted, the flow width significantly effects traffic movement after exceeding a few feet, since the flow encroaches on a moving lane rather than a normal parking lane. Field observations show that vehicles will crowd adjacent lanes to avoid curb flow. This creates a traffic hazard which contributes to the rash of minor accidents that occur during rain storms.

As the flow width increases, the traffic must eventually move through the inundated lanes, progressively reducing traffic movement as the depth of flow increases. Although some reduction of traffic movement caused by runoff is acceptable, emergency vehicles (i.e., fire equipment, ambulances, police vehicles) must be able to travel the streets. Therefore, certain limitations on the depth of flow in the street are required.

1003.3 TEMPORARY PONDING

Storm runoff temporary ponded on the street due to grade changes or intersection street crowns affects traffic movement by increasing flow depths and the duration of flow at the greater depths. This temporary ponding is localized and vehicles may enter the ponded area at high speeds unaware of the ponded water until the vehicle is out of control. Ponding will often cause traffic to halt to avoid vehicle stalling, resulting in reduced traffic movement. Therefore, depths of temporary ponding must be controlled in a manner similar to gutter flow and in some cases eliminated on high traffic volume streets.
1003.4 CROSS FLOW

Whenever storm runoff, other than sheet flow, moves across a traffic lane, traffic movement is affected. The cross flow may be caused by super-elevation of a curve, by the intersection of two streets, by exceeding the capacity of the higher gutter on a street with cross fall, or simply poor street design. The problem associated with this type of flow is the same as for ponding in that it is localized in nature and vehicles may be traveling at high speed when they reach the location. If the speed limits are slow and the traffic volume is light; then the influence of cross street flow may be within acceptable limits.

1004 DRAINAGE IMPACT ON STREET MAINTENANCE

The use of the roadway system for drainage of runoff during and immediately after storm events also has an impact on the structural integrity of the pavement system and the roadway maintenance required. If water penetrates the road surface and saturates the sub-grade material, the sub-grade may fail and cause failure of the pavement.

Additionally, runoff from rural and urban areas can carry large amounts of debris and sediment, which may reduce the performance of hydraulic structures or become a safety hazard and must be removed.

1004.1 PAVEMENT DETERIORATION

The efficient removal of storm runoff from pavement surfaces has a positive effect on street maintenance and repair. Street maintenance and repair procedures can in turn affect the efficiency of a street as part of the runoff collection system. Research has indicated that pavement deterioration is accelerated by the presence of storm runoff.

Pavement surfaces are subject to numerous types of distress such as weathering, raveling, long cracks, alligator cracks, chuck holes, bleeding, depression, and edge breakup. Water is probably the greatest cause of distress in a pavement structure. Flow of water across a bituminous pavement surface has little effect on the pavement so long as the pavement retains its watertight condition. A number of types of pavement distress may cause the pavement to become permeable, allowing water to reach the sub-grade. Once the water reaches the sub-grade, the problems multiply as the sub-base and sub-grade weaken, causing an increase in cracks through the surface.

A common practice to reduce the problem of bituminous surface deterioration is to seal-coat or overlay the surface. This reduces the problem of pavement deterioration, but indirectly creates a problem with the carrying capacity of the adjacent gutter. As the street section is resurfaced, the flow area of the section is decreased. Over a period of 20 to 30 years, a considerable portion of the runoff carrying capacity of the street may be lost. Scarifying the surface to remove the upper layer of asphalt prior to resurfacing reduces the problem, but is not always the selected method. In any case, the street section flow capacity should be maintained.

1004.2 SEDIMENTATION AND DEBRIS

A common problem in Washoe County is the deposition of sediment and debris on the street surface during and after a storm event. During the flow event, this sedimentation can cause problems by reducing the flow carrying capacity of the street section and causing increased encroachment into the
traffic lanes. This problem is most prevalent at major grade changes where the flow velocity in the street section is reduced. Reducing the flow velocity decreases its sediment and debris transport ability and sediment and debris is deposited.

Additionally, sediment and other debris carried by runoff can impair the operation of hydraulic structures such as curb inlets and grated drop inlet structures. The sediment and debris can block a portion of the flow area into these facilities and cause artificially increased water surface elevations.

Immediately after a storm event, identified problem areas should be reviewed and street sweeping initiated to remove accumulated sediment and debris. By regularly scheduled sweeping of upstream areas, the source of some of the sediment can be eliminated. Also, runoff from construction sites may cause site-specific sedimentation problems and should be controlled as recommended in Section 1400.

1005 STREET CLASSIFICATION AND ALLOWABLE FLOW DEPTH

The streets in Washoe County are classified according to traffic volume and right of way width. The standard street sections are provided in the Washoe County Development Code. The street classifications, ROW requirements, and allowable storm flow depth criteria are provided in Policy Section 304.4.

The calculation of the water surface elevation and velocity must be based on limiting the flow to the width of the right of way. This implies that, for calculation purposes only, an infinitely high vertical wall exists at the right-of-way boundary and any flow area outside of the right of way is not considered in the analysis. This provides a conservative analysis for street capacity requirements. In addition, whenever flow depths are such that crown overtopping would occur, the one-half street calculations assume a vertical wall at the street crown with no associated wetted perimeter.

For street sag locations, provisions must be included to carry the 100-year runoff in a pipe or an overflow section and include an access and maintenance easement.

1006 HYDRAULIC EVALUATION

The hydraulic analysis of flow in street sections is similar to open channel flow analysis for larger flood control channels (Section 800). The basic governing equation, Manning's equation, is as follows:

\[ Q = \left( \frac{1.49}{n} \right) A R^{\frac{2}{3}} S^{\frac{1}{2}} \]  

where
- \( Q \) = discharge (cfs)
- \( n \) = roughness coefficient (0.016 for streets)
- \( A \) = flow area (square feet)
- \( R \) = hydraulic radius = \( A/P \) (feet)
- \( P \) = wetted perimeter (feet)
- \( S \) = slope of the energy grade line (EGL), generally assumed equal to the street slope (ft/ft)
Based upon the policy of Section 304.4, the allowable storm capacity of the minor storm of each street section is calculated using Equation 1001.

The calculation of depth of flow for the major storm event is also based on Equation 1001. The major difference is in the assumed flow area. For the calculation of flow depth and velocity, the area outside the limits of the right of way is not considered in the calculation of conveyance. Even though water will flow in the area outside of the right of way, the depth of flow allowed is based on containment of the flow within the right of way.

The maximum allowable capacity for standard Washoe County street cross-sections has been calculated and is presented in Figures 1001 through 1004. The calculations were performed for various allowable flow depths and street slopes. A Manning's n-value of 0.016 was assumed for the gutter and street flow areas and a cross slope of 2% was used. If standard street sections are used, the maximum allowable street capacity shall be obtained from Figures 1001 through 1006. If non-standard sections are used, the standard Manning's equation with a Manning's n-value of .016 shall be used to calculate allowable flows.

Streets with grades flatter than 0.5% must be given special consideration when calculating allowable flow depth. These streets are subject to ponding and are candidates for storm sewers. Storm sewers and their inlets are described in Section 900.

1007 EXAMPLE APPLICATION

1007.1 EXAMPLE: ALLOWABLE FLOW IN STREET WITH 42-FOOT RIGHT-OF-WAY

Problem: Determine the allowable street capacity for the streets in Rose Subdivision shown in Figure 703. Assume the following design criteria:

a) 42-foot right-of-way  
b) South Street slope = 2.5%  
c) Middle Street slope = 2.5%  
d) North Street slope = 2.5%  
e) East Street slope = 2.5%  
f) 6-inch curb height  
g) Velocity < 6 fps  
h) All streets are classified as local streets

Solution:

Step 1: Determine the street flow capacity for a minor storm event using Figure 1001.

\[ Q_{east} = Q_{south} = Q_{middle} = Q_{north} = 3.8 \text{ cfs/gutter} \]

Step 2: Determine the street flow capacity for a major storm event using Figure 1001.

\[ Q_{east} = Q_{south} = Q_{middle} = Q_{north} = 33 \text{ cfs/gutter} \]
Step 3: Determine if storm sewer inlets are needed at each design point by comparing the 5-year peak flow obtained from the example problem in Section 709.1 to the street flow capacity for a minor storm event.

Design Point 1: Since the allowable street capacity is greater than the 5-year design flow ($3.8 \text{ cfs} \geq 2.3 \text{ cfs}$) an inlet is not needed at Design Point 1.

Design Point 2: Since the allowable street capacity is greater than the 5-year design flow ($3.8 \text{ cfs} \geq 3.3 \text{ cfs}$) an inlet and storm sewer are not needed at Design Point 2.

Design Point 3: Since the entire flow needs to be conveyed beneath North Street, an inlet and storm sewer will need to be placed at this location with a capacity of 4.4 cfs.

Design Point 4: Inlet and storm sewer required. Assuming the entire flow is to be conveyed to Design Point 7 (the detention basin) through a storm sewer, the storm required sewer capacity is 7.7 cfs.

Design Point 5: An inlet is not required ($3.8 \text{ cfs} \geq 1.9 \text{ cfs}$).

Design Point 6: The street flow does not exceed its allowable capacity at this point ($34.8 \text{ cfs} \geq 2.5 \text{ cfs}$), but an inlet and storm sewer will be placed here to collect and convey flow beneath North Street to Design Point 4.

Step 4: Determine the need for a storm sewer at each design point based on the criteria for a major storm event (100-year). The major storm event calculations have not been performed for this example problem, but the procedure is the same as that used in Step 3.

APPLICATION: The allowable street capacity is used in conjunction with the total 5-year and 100-year flows to estimate the location and sizes of inlets and storm sewers.
STREET FLOW CAPACITY
100 FOOT R.O.W. WITH MEDIAN
(ARTERIAL)
## Section 1100

### Culverts and Bridges

**Table of Contents**

<table>
<thead>
<tr>
<th>Section</th>
<th>Description</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>1101</td>
<td>Introduction</td>
<td>1102</td>
</tr>
<tr>
<td>1102</td>
<td>Design Standards for Culverts</td>
<td>1102</td>
</tr>
<tr>
<td>1102.1</td>
<td>Culvert Sizing Criteria</td>
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<td>1102.2</td>
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<td>1102</td>
</tr>
<tr>
<td>1102.3</td>
<td>Velocity Limitations and Inlet/Outlet Protection</td>
<td>1103</td>
</tr>
<tr>
<td>1102.4</td>
<td>Headwater Criteria</td>
<td>1103</td>
</tr>
<tr>
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<td>1104.1</td>
<td>Bridge Sizing Criteria</td>
<td>1108</td>
</tr>
<tr>
<td>1104.2</td>
<td>Velocity Limitations</td>
<td>1108</td>
</tr>
<tr>
<td>1105</td>
<td>Bridge Hydraulics</td>
<td>1108</td>
</tr>
<tr>
<td>1105.1</td>
<td>Hydraulic Analysis</td>
<td>1108</td>
</tr>
<tr>
<td>1105.2</td>
<td>Inlet and Outlet Configuration</td>
<td>1109</td>
</tr>
<tr>
<td>1106</td>
<td>Example Application</td>
<td>1109</td>
</tr>
<tr>
<td>1106.1</td>
<td>Example: Culvert Sizing</td>
<td>1109</td>
</tr>
<tr>
<td>Table/Example</td>
<td>Description</td>
<td></td>
</tr>
<tr>
<td>---------------</td>
<td>-------------------------------------------------</td>
<td></td>
</tr>
<tr>
<td>1101</td>
<td>Hydraulic Data for Culverts</td>
<td></td>
</tr>
<tr>
<td>1102</td>
<td>Example: Culvert Design</td>
<td></td>
</tr>
<tr>
<td>1101</td>
<td>Culvert Alignment</td>
<td></td>
</tr>
<tr>
<td>1102</td>
<td>Culvert Flow Types</td>
<td></td>
</tr>
<tr>
<td>1103</td>
<td>Nomograph - Inlet Control Box Culvert</td>
<td></td>
</tr>
<tr>
<td>1104</td>
<td>Nomograph - Inlet Control RCP</td>
<td></td>
</tr>
<tr>
<td>1105</td>
<td>Nomograph - Inlet Control Elliptical Pipe</td>
<td></td>
</tr>
<tr>
<td>1106</td>
<td>Outlet Control Culvert Hydraulics</td>
<td></td>
</tr>
<tr>
<td>1107</td>
<td>Nomograph - Critical Depth Box Culvert</td>
<td></td>
</tr>
<tr>
<td>1108</td>
<td>Nomograph - Critical Depth RCP</td>
<td></td>
</tr>
<tr>
<td>1109</td>
<td>Nomograph - Critical Depth Elliptical Pipe</td>
<td></td>
</tr>
<tr>
<td>1110</td>
<td>Nomograph - Outlet Control Box Culvert</td>
<td></td>
</tr>
<tr>
<td>1111</td>
<td>Nomograph - Outlet Control RCP</td>
<td></td>
</tr>
<tr>
<td>1112</td>
<td>Nomograph - Outlet Control Elliptical Pipe</td>
<td></td>
</tr>
<tr>
<td>1113</td>
<td>Example: Nomograph - Inlet Control 5-Foot Diameter RCP</td>
<td></td>
</tr>
<tr>
<td>1114</td>
<td>Example: Nomograph - Critical Depth in 5-Foot Diameter RCP</td>
<td></td>
</tr>
<tr>
<td>1115</td>
<td>Example: Nomograph - Outlet Control 5-Foot Diameter RCP</td>
<td></td>
</tr>
</tbody>
</table>
SECTION 1100
CULVERTS AND BRIDGES

1101 INTRODUCTION

Culverts and bridges are used to convey water through or beneath engineered structures. The size, alignment, and support structures of a bridge or culvert will directly affect the carrying capacity of the drainage system. Inadequate culvert or bridge capacity can force water out of the conveyance system, and the flood water may take an alternate path and cause damage away from the channel.

The primary distinction between a culvert and a bridge is the change in flow area from the upstream channel cross-section. A culvert is usually designed to allow the design upstream water surface elevation to be greater than the top of the culvert, while bridge design generally provides freeboard between the design flood water surface and the low chord of the bridge.

1102 DESIGN STANDARDS FOR CULVERTS

All culverts within Washoe County 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.

1102.1 CULVERT SIZING CRITERIA

For hydraulic analysis, sizing of culverts is important because of potential effects on water surface elevations in a channel. Larger culverts do not encroach into the channel cross-section as much as smaller culverts and will cause a smaller rise in water surface elevations. The trade-off is that larger culverts are more expensive to construct than small culverts.

1102.1.1 Design Frequency

As indicated in Policy Section 304.5, all culverts in Washoe County will be designed to pass the flow from the major storm including an overflow section where permitted.

1102.1.2 Minimum Size

The minimum culvert size shall be 18-inch diameter for round pipe or shall have a minimum flow area of 2.2 square feet for other pipe shapes.

Culverts for driveways for single family residences shall be sized for the equivalent roadside ditch flow area and be a minimum 15-inch diameter round pipe or equivalent pipe size for other than round pipe.

1102.2 CONSTRUCTION MATERIALS

Culverts shall be constructed with Reinforced Concrete Pipe. The pipe shape may be round, square, rectangular, or elliptical.
Other pipe materials may be used for culvert construction upon approval by Washoe County. Documentation must be submitted for review which shows that the subject pipe material has a design life similar to the above materials and that the interior lining, if any, will maintain the design Manning's roughness coefficient ("n") value for the life of the pipe material.

Culvert headwalls and wingwalls shall be provided with guardrails or handrails in conformance with local building codes and roadway design safety requirements.

1102.3 VELOCITY LIMITATIONS AND INLET/OUTLET PROTECTION

In the proper design of culverts, the velocity of the flow through the culvert is very important. If the velocity is too low, suspended sediment in the flow may settle. This decreases the effective area of the culvert and increases the frequency of required maintenance. If the velocity of the flow exiting the culvert is too high, erosion may take place, possibly jeopardizing the integrity of the culvert and roadway.

All culverts shall include headwalls/wingwalls or flared-end sections at inlets and outlets. Culvert designs which include road overtopping sections shall include a road profile which will adequately confine flows within the design overtopping section and convey such flows into the downstream channel. Adequate erosion protection shall be provided to prevent degradation of the roadway and embankments.

All culverts shall be designed to provide a minimum flow velocity of 3 fps at the culvert outlet under minor storm conditions. In addition, the culvert slope shall be a minimum 0.25 percent.

The criteria for outlet erosion protection for discharges to channels with unlined bottoms are as follows:

<table>
<thead>
<tr>
<th>Outlet Velocity (fps)</th>
<th>Required Outlet Protection</th>
</tr>
</thead>
<tbody>
<tr>
<td>less than 5</td>
<td>Minimum riprap protection (Section 807.3)</td>
</tr>
<tr>
<td>between 5 and 15</td>
<td>Riprap protection (Section 807.3) or Energy dissipator (Section 1202.2)</td>
</tr>
<tr>
<td>greater than 15</td>
<td>Energy dissipator (Section 1202.2)</td>
</tr>
</tbody>
</table>

1102.4 HEADWATER CRITERIA

For culvert designs based on standard inlet (headwall, wingwalls, etc.) and outlet configurations, the maximum headwater for the design storm flow for culverts greater than 36" diameter or a culvert rise of greater than 36" shall be 1.5 times the culvert height. The maximum headwater for culverts with a height of 36" or less shall be 5 feet if adjacent properties are not adversely affected.

If site conditions are such that the maximum headwater conditions can not be met, additional engineering analysis shall be performed. This analysis is necessary to determine scour potential, embankment stability and any other factors that may influence the long-term stability of the structure. Additional erosion protection around the culvert inlet or other design considerations shall be included as appropriate to ensure the long-term stability of the culvert and approaches.
Culverts which do not include an overtopping section shall have a minimum of 1 foot freeboard from the hydraulic grade line elevation at the culvert entrance to the edge of the overlying roadway. Levees shall not be used to provide increased headwater at culvert inlets.

The extent of impact on adjacent properties from backwater created by culvert installations shall be analyzed for all culverts.

1102.5 ALIGNMENT

The alignment of the culvert with respect to the natural channel is very important for proper hydraulic performance. Culverts may pass beneath the roadway normal to the centerline or they may pass at an angle (skewed). Whenever possible, culverts should be aligned with the natural channel. This reduces inlet and outlet transition problems.

Where the natural channel alignment would result in an exceptionally long culvert, modification to the natural alignment may be necessary. Since such modifications will change the natural stability of the channel, such modification should be thoroughly investigated. Although the economic factors are important, the hydraulic effectiveness of the culvert must be given major consideration. Improper culvert alignment may cause erosion to adjacent properties or siltation of the culvert. Culvert Alignment considerations are shown in Figure 1101.

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 unavailable will require additional structural analysis and other provisions (i.e. full depth concrete paving to compensate for the loss of proper cover).

1102.6 TEMPORARY CROSSING

Temporary crossings are defined as dip road sections with a culvert sized to pass nuisance flow, or a culvert system that does not meet criteria presented in Section 1100 of this Manual.

Temporary crossings will be reviewed on a case-by-case basis. Major consideration will be given to the following items:

1. Drainage area contributing to crossing.
2. Volume of roadway traffic.
3. Vertical and horizontal roadway alignment (sight distance).
4. Alternate access routes.
5. Time frame for temporary crossing.
7. 5-year and 100-year storm flows.
1102.7 **MULTIPLE-BARREL CULVERTS**

If the available fill height limits the size of culvert necessary to convey the flood flow, multiple culverts can be used. If each barrel of a multiple-barrel culvert are of the same type and size and constructed such that all hydraulic parameters are equal, the total flow should be assumed to be equally divided among each of the barrels.

1102.8 **TRASH RACKS/SAFETY GRATES**

During culvert design, engineering judgement shall be used to determine if trash racks or safety grates should be included. Factors which may influence whether or not trash racks or safety grates should be used include the following:

- Tributary Land Use (urban, rural, forest)
- Location (urban/rural)
- Design flow
- Size of culvert
- Anticipated debris loading
- Performance of nearby existing structures

Additionally, trash racks or safety grates shall be used 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. The open area through the grate at the design water surface shall be four times the design flow area of the culvert or in lieu of this, a trashrack/entrance section design provided in the "Standard Details" for Washoe County shall be used.

1102.9 **AIR VENTS**

All culverts greater than 48 inches in diameter for which both the inlet and outlet are sealed by water under less than full flow conditions shall include an air vent pipe to prevent air accumulation/partial vacuums. Said vent shall have a diameter equal to or greater than one-sixth of the culvert pipe diameter.

1102.10 **MAINTENANCE ACCESS**

Access (including necessary easements) for culvert maintenance/cleaning shall be provided at all culvert locations.

1103 **CULVERT HYDRAULICS**

This section presents the general procedures for hydraulic design and evaluation of culverts. The user is assumed to possess a basic working knowledge of culvert hydraulics and is encouraged to review the textbooks and other technical literature on the subject.

The two categories of flow in culverts are inlet control and outlet control. Under inlet control, the flow through the culvert is controlled by the headwater on the culvert and the inlet geometry. Under outlet control, the flow through the culvert is controlled 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 constraint (e.g. headwater depth, flow capacity, etc.). The control condition which produces the greater energy loss for the design condition determines the appropriate control to use for culvert design. Culvert hydraulic calculations shall be performed using rating nomographs and/or culvert hydraulic analysis programs (i.e. H-8).

1103.1 INLET CONTROL CONDITION

Inlet control for culverts may occur in two ways (see Figure 1102):

1. **Unsubmerged** - The headwater is not sufficient to submerge the top of the culvert and the culvert invert slope is supercritical. The culvert entrance acts like a weir (Condition A, Figure 1102).

2. **Submerged** - The headwater submerges the top of the culvert but the pipe does not flow full. The culvert inlet acts like an orifice (Condition B and C, Figure 1102).

The inlet control rating for typical shapes and inlet configurations are presented in Figures 1103 to 1105. Additional nomographs are available in the U.S. Department of Transportation's Hydraulic Design Series Number 5 (USDOT, 1985). These nomographs were developed empirically by pipe manufacturers, Bureau of Public Roads, and the Federal Highway Administration. The nomographs shall be used in Washoe County rather than the orifice and weir equations, due to the uncertainty in estimating the orifice and weir coefficients.

1103.2 OUTLET CONTROL CONDITION

Outlet control will govern if the headwater and/or tailwater is deep enough, the culvert slope is relatively flat, and the culvert is relatively long. There are three types of outlet control culvert flow conditions:

1. The headwater submerges the culvert top, and the culvert outlet is submerged by the tailwater. The culvert will flow full (Condition A, Figure 1102).

2. The headwater submerges the top of the culvert and the culvert is unsubmerged by the tailwater (condition B or C, Figure 1102).

3. The headwater is insufficient to submerge the top of the culvert. The culvert slope is subcritical and the tailwater depth is lower than the pipe critical depth (Condition D, Figure 1102).

The factors affecting the capacity of a culvert in outlet control include the headwater elevation, the inlet geometry and associated losses, the culvert material friction losses, and the tailwater condition.

The capacity of the culvert is calculated using the conservation of energy principle (Bernoulli's Equation). An energy balance exists between the total energy of the flow at the culvert inlet and at the culvert outlet, which includes the inlet losses, the friction losses, and the velocity head (see Figure 1106). The equation is then expressed as:

\[ H = h_e + h_t + h_f \]  \hspace{1cm} (1101)
where \( H \) = total energy difference, inlet through outlet (ft)
\( h_e \) = entrance head losses (ft)
\( h_f \) = friction losses (ft)
\( h_v \) = velocity head = \( V^2/2g \) (feet)  

\[
(1102)
\]

For inlet losses, the governing equation is:

\[
h_e = K_e \left( \frac{V^2}{2g} \right)
\]

\[
(1103)
\]

where \( K_e \) is the entrance loss coefficient. Typical entrance loss coefficients recommended for use are given in Table 1101.

Friction loss is the energy required to overcome the roughness of the culvert and is expressed as follows:

\[
h_f = (29n^2L/R^{1.33})(V^2/2g)
\]

\[
(1104)
\]

where
- \( n \) = Manning's coefficient (see Table 1102)
- \( L \) = Length of culvert (ft)
- \( R \) = Hydraulic radius (ft)
- \( V \) = Velocity of flow (fps)
- \( G \) = Gravitational acceleration constant (32.2 ft/s²)

Substituting equivalent terms from equations 1102, 1103, and 1104 into equation 1101 and simplifying the terms results in the following equation:

\[
H = \left[ K_e + (29n^2L/R^{1.33}) + 1 \right] \frac{V^2}{2g}
\]

\[
(1105)
\]

Equation 1105 can be used to calculate the culvert capacity directly when the culvert is flowing under outlet control conditions A or B as shown on Figure 1102. The actual headwater (\( H_w \)) is calculated by adding \( H \) to the tailwater elevation (see Figure 1106). For conditions C or D in Figure 1102, the hydraulic grade line at the outlet is approximated by averaging the critical depth and the culvert diameter. This value is used to compute headwater depth (\( H_w \)) if it is greater than the tailwater depth (\( T_w \)). This is an approximate method and is more fully described in HDS No. 5 (USDOT, 1985). Estimates of critical depth for box culverts, circular pipe, and elliptical pipe can be obtained from Figures 1107, 1108, and 1109 respectively.

A series of outlet control nomographs for various culvert shapes have been developed by pipe manufacturers, Bureau of Public Roads, and the Federal Highway Administration. The nomographs are presented in Figures 1110 to 1112. Additional nomographs are available in HDS No. 5 (USDOT, 1985). When rating a culvert, either the outlet control nomographs or Equation 1105 can be used to calculate the headwater requirements.

1103.3 HYDRAULIC DATA

The hydraulic data provided in Table 1101 shall be used in the hydraulic design of all culverts within Washoe County. The design capacity of culverts shall be calculated using the computation sheet provided as Standard Form 4. Manning's roughness coefficients ("n") used for velocity and capacity calculations shall be those presented in Table 1102 for storm sewers.
Alternatively, computer programs may be used for hydraulic analysis. However the designer should thoroughly review the modelling results to determine if the analysis has properly modelled the hydraulic conditions.

1103.4 STRUCTURAL DESIGN

All culverts shall be designed as a minimum to withstand an H-20 loading in accordance with the design procedures of AASHTO "Standard Specifications for Highway Bridges" and with the pipe manufacturer's recommendations.

1104 DESIGN STANDARDS FOR BRIDGES

All bridges shall be in accordance with the "Standard Specifications for Highway Bridges" by AASHTO and the "Standard Plans for Road and Bridge Construction" by the State of Nevada Department of Transportation. Hydraulic design and analysis shall be in accordance with the following criteria.

1104.1 BRIDGE SIZING CRITERIA

All bridges within Washoe County shall be designed to pass the 100-year design flow. Additionally, the design water surface elevation within the bridge shall be a minimum of 2 foot below the bridge low chord or measures taken to avoid floatation of the bridge due to blockage. Additional freeboard may be required for special hydraulic conditions. In special flood hazard areas, the bridge shall not back up the 100-year storm flow greater than 1 foot above the natural water surface elevation without mitigation measures. The designer must also ensure that no adjacent properties are adversely affected.

1104.2 VELOCITY LIMITATIONS

The velocity limitations through the bridge opening are controlled by the potential abutment scour and subsequent erosion protection provided. Flow velocities through the bridge and approaches shall be in accordance with the appropriate allowable channel velocities as discussed in Section 803.5, dependent on channel lining type.

1105 BRIDGE HYDRAULICS

1105.1 HYDRAULIC ANALYSIS

The procedures for analysis and design as outlined in the publications Stream Stability at Highway Structures (USDOT, 1991A) and Evaluating Scour at Bridges (USDOT, 1991B) shall be used for the hydraulic design and scour analysis of all bridges in Washoe County. This analysis shall be supplemented by an appropriate backwater analysis (see Section 802) to verify the resulting hydraulic performance. The extent of the bridge backwater shall be shown on a topographic map.

Analysis shall be prepared for the design flow condition as well as an approximate 500-year condition (1.7 times the 100-year flow) and the flow condition at which the water surface just inundates the bridge soffit.
1105.2 INLET AND OUTLET CONFIGURATION

The design of all bridges shall include adequate wingwalls of sufficient length to prevent abutment erosion and to provide slope stabilization from the embankment to the channel. Erosion protection on the inlet and outlet transition slopes shall be provided to protect the channel from the erosive forces of eddy currents.

1106 EXAMPLE APPLICATION

1106.1 EXAMPLE: CULVERT SIZING

Problem: Determine the culvert size necessary to convey the 100-year, 24-hour peak flow in Doe Creek beneath John Boulevard. The results of this analysis are provided in Table 1103.

- Top of road elevation = 4928 feet
- Culvert inlet elevation = 4920 feet
- Culvert outlet elevation = 4918 feet
- Culvert length = 200 feet
- Inlet - Groove end with headwall and wingwalls at 45°
- Outlet - Groove end with headwall and wingwalls at 45°
- Flow = 191 cfs from Section 700
- Tailwater Depth = 4 feet

Solution:

Step 1: Assume a pipe diameter or box culvert dimensions and determine the headwater to depth ratio for inlet control conditions from Figure 1104. Assuming a 5-foot diameter reinforced concrete pipe (RCP), the headwater to depth ratio, is 1.38 (see Figure 1104).

Step 2: Calculate the headwater assuming inlet control conditions. Multiply the pipe diameter times the headwater to depth ratio.

\[ \text{Headwater} = HW_1 = D \times HW/D = 5 \times 1.38 = 6.9 \text{ feet} \]

Step 3: Estimate the critical depth, \(d_c\), in the culvert from Figure 1108 (see Figure 1114).

\( d_c = 3.9 \text{ feet} \)

Step 4: Since the tailwater depth is less than the culvert diameter, compute the estimated water depth at the culvert outlet assuming the tailwater does not control the outlet conditions.

\[ \text{Outlet Depth} = \frac{d_c + D}{2} = \frac{3.9 + 5.0}{2} = 4.5 \text{ feet} \]

Step 5: Determine the flow depth at the culvert outlet, \(h_o\). The estimated depth is the maximum value of the tailwater depth and the water depth assuming no tailwater.

\( h_o = 4.5 \text{ feet} \)
Step 6: Estimate the head, H, for outlet control conditions from Figure 1111.

\[ H = 2.6 \text{ feet (see Figure 1115).} \]

Step 7: Calculate the headwater depth for outlet control conditions.

\[ HW_o = H + h_o + LS_o = 2.6 + 4.5 - 2.0 = 5.1 \]

Step 8: Determine if the culvert is under inlet control or outlet control and provide the resulting headwater depth and elevation.

Since \( HW_l \geq HW_o \) (6.4 \( \geq \) 5.5), the culvert is under inlet control.

\[ HW = 6.4 \]

Step 9: Calculate the outlet velocity by an appropriate method, and determine the type of outlet protection needed in Section 800.

\[ V = 10.0 \text{ fps} \]

Riprap protection or an energy dissipater is necessary.
# Hydraulic Data for Culverts

## Culvert Entrance Losses

<table>
<thead>
<tr>
<th>Type of Entrance</th>
<th>Entrance Coefficient, ( K_e )</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Pipes</strong></td>
<td></td>
</tr>
<tr>
<td>Headwall</td>
<td></td>
</tr>
<tr>
<td>Grooved edge</td>
<td>0.20</td>
</tr>
<tr>
<td>Rounded edge (0.15D radius)</td>
<td>0.15</td>
</tr>
<tr>
<td>Rounded edge (0.25D radius)</td>
<td>0.10</td>
</tr>
<tr>
<td>Square edge (cut concrete and CMP)</td>
<td>0.40</td>
</tr>
<tr>
<td>Headwall &amp; 45° Wingwall</td>
<td></td>
</tr>
<tr>
<td>Grooved edge</td>
<td>0.20</td>
</tr>
<tr>
<td>Square edge</td>
<td>0.35</td>
</tr>
<tr>
<td>Headwall with Parallel Wingwalls Spaced 1.25D apart</td>
<td></td>
</tr>
<tr>
<td>Grooved edge</td>
<td>0.30</td>
</tr>
<tr>
<td>Square edge</td>
<td>0.40</td>
</tr>
<tr>
<td>Beveled edge</td>
<td>0.25</td>
</tr>
<tr>
<td>Projecting Entrance</td>
<td></td>
</tr>
<tr>
<td>Grooved edge (RCP)</td>
<td>0.25</td>
</tr>
<tr>
<td>Squared edge (RCP)</td>
<td>0.50</td>
</tr>
<tr>
<td>Sharp edge, thin wall (CMP)</td>
<td>0.90</td>
</tr>
<tr>
<td>Sloping Entrance</td>
<td></td>
</tr>
<tr>
<td>Mitered to conform to slope</td>
<td>0.70</td>
</tr>
<tr>
<td>Flared-end Section</td>
<td>0.50</td>
</tr>
<tr>
<td><strong>Box, Reinforced Concrete</strong></td>
<td></td>
</tr>
<tr>
<td>Headwall Parallel to Embankment (no wingwalls)</td>
<td></td>
</tr>
<tr>
<td>Square edge on 3 edges</td>
<td>0.50</td>
</tr>
<tr>
<td>Rounded on 3 edges to radius of 1/12 barrel dimension</td>
<td>0.20</td>
</tr>
<tr>
<td>Wingwalls at 30° to 75° to barrel</td>
<td></td>
</tr>
<tr>
<td>Square edge at crown</td>
<td>0.40</td>
</tr>
<tr>
<td>Crown edge rounded to radius of 1/12 barrel dimension</td>
<td>0.20</td>
</tr>
<tr>
<td>Wingwalls at 10° to 30° to barrel</td>
<td></td>
</tr>
<tr>
<td>Square edge at crown</td>
<td>0.50</td>
</tr>
<tr>
<td>Wingwalls parallel (extension of sides)</td>
<td></td>
</tr>
<tr>
<td>Square edge at crown</td>
<td>0.70</td>
</tr>
</tbody>
</table>

**NOTE:** The entrance loss coefficients are used to evaluate the culvert or sewer capacity operating under outlet control
NOMOGRAPH - INLET CONTROL RCP

**WASHOE COUNTY**
**HYDROLOGIC CRITERIA AND DRAINAGE DESIGN MANUAL**

**DIAMETER OF CULVERT (D) IN INCHES**

**DISCHARGE (Q) IN CFS**

**HW D SCALE**

**ENTRANCE TYPE**

- (1) Square edge with headwall
- (2) Groove and with headwall
- (3) Groove and projecting

**HEADWATER DEPTH IN DIAMETERS (HW/D)**

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.
To use scale (2) or (3) draw a straight line through known values of size and discharge to intersect scale (1). From point on scale (1) project horizontally to solution on other scale (2) or (3).

ENTRANCE TYPE
(1) Square edge with headwall
(2) Groove and with headwall
(3) Groove and projecting
NOMOGRAM - OUTLET CONTROL BOX CULVERT
(n=0.012)

For outlet crown not submerged, compute HW by methods described in the design procedure.

DISCHARGE (Q) IN CFS

DIMENSION OF SQUARE BOX IN FEET

AREA OF RECTANGULAR BOX IN SQUARE FEET

LENGTH (L) IN FEET

HEAD (H) IN FEET

VERSION: December 2, 1993
REFERENCE:
WRC ENGINEERING, Inc.

FIGURE 1110
WASHOE COUNTY
HYDROLOGIC CRITERIA AND DRAINAGE DESIGN MANUAL

NOMOGRAPH - OUTLET CONTROL RCP
(n=0.012)

For outlet crown not submerged, compute HW by methods described in the design procedure.

FIGURE
1111
NOMOGRAPH - OUTLET CONTROL ELLIPTICAL PIPE
(n=0.012)

For outlet crown not submerged, compute HW by
methods described in the design procedure.

Dimensions on size scale are
ordered for long axis horizontal
installation. They should be
reversed for long axis vertical.
WASHOE COUNTY
HYDROLOGIC CRITERIA AND DRAINAGE DESIGN MANUAL

NOMOGRAPH - OUTLET CONTROL RCP
(n=0.012)

For outlet crown not submerged, compute HW by methods described in the design procedures.

DISCHARGE (Q) IN CFS

DIAETR (D) IN INCHES

HEAD (H) IN FEET

LENGTH (L) IN FEET

SLOPE S

TWIRING LINE

SUBMERGED OUTLET COULVERT FLOWING FULL

VERSION: December 2, 1998
REFERENCE:
WFC ENGINEERING, Inc.

FIGURE
1115
WASHOE COUNTY
HYDROLOGIC CRITERIA AND DRAINAGE DESIGN MANUAL

SECTION 1200
ADDITIONAL HYDRAULIC STRUCTURES

TABLE OF CONTENTS

1201 INTRODUCTION 1202

1202 CHANNEL DROPS AND ENERGY DISSIPATION STRUCTURES 1202
1202.1 Channel Drop Structures 1202
1202.1.1 Sloping Grouted Boulder Drop Structures 1203
1202.1.1.1 Design 1203
1202.1.2 Vertical Riprap Drop Structures 1205
1202.1.2.1 Introduction 1205
1202.1.2.2 Design 1208
1202.1.3 Straight Drop Spillways 1208
1202.1.4 Baffled Aprons (USBR Type IX) 1209
1202.2 Energy Dissipation Structures 1209
1202.2.1 Types of Energy Dissipation Structures 1209
1202.2.2 Stilling Basins with Horizontal Sloping Aprons 1210
1202.2.2.1 Short Stilling Basin (USBR Type III) 1210
1202.2.2.2 Low Froude Number Basins (USBR Type IV) 1210
1202.2.2.3 Impact Stilling Basin (USBR Type VI) 1210
1202.2.3 Hydraulic Design 1210
1202.2.4 Riprap Protection 1211
1202.2.5 Design Flow Rates 1211
1202.2.6 Trajectory Transition Section 1211

LIST OF TABLES

1201 CHANNEL DROP AND ENERGY DISSIPATION STRUCTURES
1202 SLOPING GROUTED BOULDER DROP DESIGN PARAMETERS
1203 SLOPING GROUTED BOULDER DROP ROCK AND GROUT THICKNESS

December 2, 1996  Additional Hydraulic Structures 1200
LIST OF FIGURES

1201  GENERALIZED PROFILES OF RIPRAP DROP STRUCTURES
1202  SLOPING GROUTED BOULDER DROP
1203  SLOPING GROUTED BOULDER DROP SCHEMATIC
1204  TYPICAL GROUTED BOULDER PLACEMENT
1205  WEEP DRAIN SYSTEM DETAILS
1206  VERTICAL RIPRAP DROP
1207  CURVES FOR SCOUR DEPTH AT VERTICAL DROP
1208  STRAIGHT DROP SPILLWAY
1209  BAFFLED APRON STILLING BASIN (USBR TYPE IX)
1210  DESIGN DATA - USBR TYPE STILLING BASINS
1211  SHORT STILLING BASIN (USBR TYPE III)
1212  LOW FROUDE NUMBER STILLING BASIN (USBR TYPE IV)
1213  IMPACT STILLING BASIN (USBR TYPE VI)
1214  TYPICAL TRAJECTORY TRANSITION
SECTION 1200
ADDITIONAL HYDRAULIC STRUCTURES

1201 INTRODUCTION

Presented in this section are design guidelines and standards for hydraulic structures which are appurtenant to both storm sewer outlet and open channel design. These guidelines and standards are generalized since each structure is unique, with the possible exception of channel drops. The user is encouraged to coordinate with the local entity and/or Washoe County when planning and designing these types of hydraulic structures.

1202 CHANNEL DROPS AND ENERGY DISSIPATION STRUCTURES

The design of open channels often requires the use of channel drop and/or energy dissipation structures to dissipate excess energy created by gravity acting on the storm water flow. The most common use of these structures is to control the longitudinal slope of channels to keep design velocities within acceptable limits (Section 800). These structures are also used to dissipate excess energy at storm sewer outlets.

For the purposes of this MANUAL, channel drop and energy dissipation structures are classified into two groups. Channel Drops are classified as structures which shall only be used when the inflow channel flow is sub-critical (Froude Number, \( F_r < 0.8 \)). Energy Dissipators (and Still ing Basins) are classified as structures which may be used for either sub-critical (\( F_r < 0.8 \)) or super-critical (\( F_r > 1.13 \)) inflow conditions.

Presented in Table 1201 is a listing of the structures discussed in this section along with the hydraulic limitations under which these structures are allowed to be used within Washoe County. The designer must obtain prior approval from Washoe County to use any of the listed structures outside of the stated limits. Also, if the designer desires to use a structure not discussed in the section, pertinent detailed information on the structure must be submitted to Washoe County for review and approval prior to designing the facility.

Criteria and charts to aid in the design of these types of structures have been developed based on numerous hydraulic studies and are provided in the following sections.

The reader should refer to the standard channel drop and energy dissipation design references to become familiar with the detailed information available on each type of structure prior to design. Suggested references include PETERKA, 1978; USBR, 1987; and USACE, 1970.

1202.1 CHANNEL DROP STRUCTURES

Presented in Table 1201 are the types of channel drop structures allowed in Washoe County. By definition, channel drop structures are used only when the upstream channel flow is sub-critical. Figure 1201 presents the generalized profiles and nomenclature for these types of drop structures. This nomenclature is used throughout this section for discussion of specific standards for each part of the structure.
1201.1.1 Sloping Grouted Boulder Drop Structures

This type of structure has gained popularity due to design aesthetics and successful applications. The quality of rock used and proper grouting procedure are very important to the structural integrity. There is no maximum height requirement for this type of structure.

The GSB drop is designed to operate as a hydraulic jump dissipator, although some energy loss is incurred due to the roughness of the grouted rock slope. Structure integrity and containment of the erosive turbulence within the basin area, are the main design objectives.

Grouted boulder drops must be constructed of uniform size boulders grouted in place through the approach, sloping face, basin, and exit areas of the drop. Figures 1202, 1203, and 1204 illustrate the general configuration of the sloping grouted boulder drop structure. Table 1202 supplies the design criteria for the drop structure based on the flow, drop height, and soil's erosiveness.

1202.1.1.1 Design

a) Rock and Grout: The grout thickness, $D_g$, and rock thickness, $D_r$, are determined based on a minimum surplus net downward force of 30 pounds. The grouted boulder section is only one layer thick. The rock size with its corresponding depth of grout is provided in Table 1203. A thicker layer of grout will reduce the appearance and the energy dissipation characteristics of the drop structure.

The rock used for the grouted boulder drop structure is different from the standard riprap gradation in that the smaller rock has been removed to allow greater penetration by the grout. The boulders are placed directly on the subgrade with no bedding. The boulders should be placed as closely together as possible without disturbing the subgrade. Boulders should also be placed with the flattest surface horizontal and on top. Before the grout is placed, the rock should be sprayed with clean water to clean the rock and allow better adherence by the grout to the rock.

The voids between the boulders is the filled with grout meeting the specifications outlined in Section 800. The grout should be vibrated with a pencil vibrator to ensure complete penetration and filling of the voids. A small hand broom or gloved hand is used to smooth the grout and remove any excess grout from the rock. The finished surface should be sealed with a curing compound.

b) Approach Apron: The upstream channel will have a trapezoidal section designed according to Section 800. The length of the approach apron will be as shown in Table 1202. The width of the approach apron and the side slopes will be identical to the upstream channel. The height of grouted boulder channel sides will be equal to the depth of water in the upstream channel plus the required freeboard as described in Section 800. The approach apron is provided to protect against the increasing velocities and turbulence which result as the water approaches the sloping portion of the drop structure.

A concrete or grout cutoff wall shall be placed at the top of the slope and on the upstream side if the approach apron to reduce or eliminate seepage and piping along with the failures which can result from these problems. The depth of the cutoff wall...
should be at least the full depth of the riprap layer and at least 1-foot thick. Depending on the soil type and hydraulic forces acting on the drop structure, the cutoff wall may need to be deeper to lengthen the seepage flow path.

c) **Drop**: The slope of the drop structure should not be steeper than 4:1. Slopes flatter than 4:1 usually increase expense, but some improvement in appearance may be gained. The side slopes and bottom width of the drop should be the same as the upstream channel. The grouted boulders should extend up the side slopes a height of the tailwater depth plus the required freeboard as projected from the downstream channel or the critical depth plus 1 foot, whichever is greater.

d) **Basin**: The main stilling basin is depressed 1 to 2 feet in order to stabilize the hydraulic jump. The basin shall be constructed to the dimensions provided in Table 1202 and shown in Figures 1202 and 1203. The grouted boulder sides should extend up the side slopes a height of the tailwater depth plus the required freeboard as projected from the downstream channel.

e) **Exit Apron**: The exit apron is necessary to minimize any erosion that may occur due to secondary currents. The bottom width and side slopes of the exit apron should be the same as the downstream channel. The grouted boulder channel sides should extend to a height equal to the tailwater depth plus the required freeboard. The length of the exit apron should be according to Table 1202.

f) **Low Flow Channel**: A low flow or trickle channel will extend through the drop structure as shown in Figure 1202 connecting the upstream low flow channel to the downstream low flow channel. Due to the greater depth of flow in the trickle channel as compared to the main channel, the trickle channel will have higher velocities and greater energy, and the jump will tend to wash downstream of the basin. Large boulders or should be placed in the trickle channel in the bottom of the basin to help dissipate the higher flow energy.

g) **Drainage**: Subgrade erosion caused by seepage and structure failures caused by high seepage pressures or inadequate mass are of critical concern. These factors are very important in the design and must be analyzed.

The most sophisticated means of seepage analysis involves computerized groundwater flow modeling. Advanced geotechnical field and laboratory testing techniques may be used to confirm the accepted permeability values where complicated seepage problems are anticipated. Several flow net analysis programs are currently available that are suitable for this purpose. These methods are discussed in CEDEGREN, 1967, TAYLOR, 1967, and USBR, 1987.

A minimal approach is Lane's Weighted Creep method (LANE, 1935). It can be used to determine dimensions or cutoff improvements which would provide an adequate seepage length. It should only be used as a guideline, and when marginal conditions or complicated geological conditions exist, a more precise analysis should be used.

Weep drains are needed for seepage and uplift control. Weep drains for grouted sloping boulder drop structures are shown in Figure 1205. This type of system is appropriate for smaller drops and other locations where space is limited.
continuous manifold is preferred over a "point" system for weep drainage of a drop structure as it provides more complete interception of subsurface drainage.

Weep systems require special attention during construction. The pipes can be crushed by the boulders and alignment of the pipes between the boulders are difficult. Flexible outlet pipes should be used to allow alignment of the pipes around the boulders.

1202.1.2 Vertical Riprap Drop Structures

1202.1.2.1 Introduction

Energy dissipation is achieved in this type of drop by flow plunging into a pool where the energy is expended by turbulence. The pool is created by specific placement and construction of a basin or by a "planned" rearrangement of rock by the flow.

The structural design for the vertical crest wall is complicated by the lack of downstream support, seepage, soil saturation and hydraulic loading on the upstream side. In sandy or erosive soils, it is quite common to use sheet pile for crest wall construction, while caissons may prove acceptable for certain other applications. Commonly a retaining wall is used after evaluating seepage control.

Figures 1206 and 1207 provide the design standards and details for vertical riprap drop structures. The design curves for the vertical channel drop structures are based upon the height of the drop and the normal depth and velocity of the approach and exit channels. The channel must be prismatic throughout, from the upstream channel through the drop to the downstream channel.

The maximum (steepest) allowable side slope for the riprap stilling basin is 4:1. Flatter side slopes are allowable and encouraged when available right-of-way permits. The riprap should extend up the side slopes to a depth equal to 1-foot above the normal depth projected upstream from the downstream channel.

The crest wall is a structural retaining wall which is buried at least 3 feet below the level of the rock bedding layer in the drop basin. A trickle channel is carried through the wall. The top of the crest wall should not extend above the upstream invert elevation. The trickle slab should be tied to the structure and consider wall movement.

Crest wall and footer dimensions are determined by conventional structural methods. Underdrain requirements are determined from seepage analysis.

The following design methodology is adapted from STEVENS, 1981. The design is essentially that which was developed and model tested by SMITH, 1965. The structure is an adaptation of the reinforced concrete vertical drop structure adapted to smaller heads and drop heights.

1202.1.2.2 Design

The crest wall height extends to the energy gradeline above the drop crest. The crest wall height, \( H_m \), is given by the following equation.
\[ H_m = EGL_m - Elev_c \]

where

\[ EGL_m = \text{Energy gradeline elevation at main crest of drop} \]
\[ Elev_c = \text{Invert elevation of main channel at crest of drop} \]

Since the flow is at critical depth at the crest of the drop, the energy gradeline elevation can be calculated with the following equations:

\[ V_{cm} = (g \cdot y_{cm})^{0.5} \]

where

\[ V_{cm} = \text{Critical velocity of main channel} \]
\[ y_{cm} = \text{Critical depth of main channel} \]

\[ EGL_m = y_{cm} + \frac{V_{cm}^2}{2g} + Elev_c \]

The wingwalls (Figure 1206) are required to direct the flow coming along the sides of the approach channel into the plunge pool. The width of the crest is the same as the bed of the approach section. The height of the wingwalls above the main crest is the same as the crest wall height calculated above.

The wingwalls must extend below the depth of excavation for the plunge pool and must provide an adequately long seepage path to prevent piping. A separate analysis at the trickle channel is required as follows:

\[ H_t = EGL_t - Elev_t \]

where

\[ EGL_t = \text{Energy gradeline elevation of trickle channel at main crest of drop} \]
\[ Elev_t = \text{Invert elevation of trickle channel at crest of drop} \]

and

\[ EGL_t = y_{ct} + \frac{V_{ct}^2}{2g} + Elev_t \]

where

\[ y_{ct} = \text{Critical depth of trickle channel} \]
\[ V_{ct} = \text{Velocity of trickle channel} \]

The plunge pool is a deep bed of rock riprap initially placed level across the floor of plunge pool and extending downstream.

\[ L_b = 4H_m + 0.25D \]
where

\[ L_b = \text{Length of basin} \]
\[ D = \text{Depth of drop} \]

The first flow over the weir initially falls on the rock bed and begins to form a scour hole. The rocks removed from the scour hole are deposited in the area between the scour hole and the beginning of the downstream channel. With substantial flow or a repetition of flow, a mound of stones forms downstream from the scour hole. The mound is an integral part of the energy dissipating structure and must be maintained. This is achieved by initially placing the top of the stone bed below the downstream channel bed by an amount equal to two-thirds of the scour depth, \( d_s \), at the design discharge. The scour hole must be allowed to develop by natural means and generally should not be preformed.

The desired drop across the structure is the difference in the bed elevations of the approach channel at the weir and the downstream channel at the end of the structure. Let this difference be \( H_d \). It follows from Figure 1206 that

\[ H_d = D - 0.67 \ d_s \] (1207)

The designer must find the combination of rock size and jet plunge height, \( D \), that gives a depth of scour which balances Equation 1207. The relation between rock size, \( d_{50} \), jet plunge height, \( D \), head on the weir, \( H \), and depth of scour, \( d_s \), is given in Figure 1207. As these values will be different in the main drop and the trickle, the design \( d_{50} \) and/or \( d_s \) will vary. This assumes that this is an appropriate extrapolation of the modeling work, which would appear reasonable if the trickle and adjacent areas are treated conservatively.

To obtain an adequate cutoff, the depth of the vertical wall that forms the weir crest must extend below the bottom of the excavation for the riprap. Therefore, it is usually uneconomical to design a scour depth \( d_s \) any greater than 0.3D. To meet this limitation in the field, it is necessary to increase the rock size \( d_{50} \), decrease the jet plunge height \( D \) (by using more drops), decrease \( H \) (by using a wider structure), or use another type of drop structure.

A contingency factor of 25% to 50% should be applied to the rock depth in areas of erosive soils since experience has shown that basin rock rearrangement can cause collapse into the basin center.

The side slopes in the basin must be riprapped also as there are strong back currents in the basin. A sand and gravel or cloth filter is required under this riprap. The side slopes in the basin should be the same slope as for the downstream channel (but no steeper than 4 horizontal to 1 vertical).

The following provides a summary of the design parameters used in Figure 1206.

- \( b_t \) = Trickle channel width
- \( Y_s \) = Depths of flow upstream of drop
- \( H_{cw} \) = Depth of upstream cutoff wall
- \( Y_2 \) = Tailwater depth
- \( B \) = \( 0.67 \ d_s = \) basin depth
- \( D_R \) = \( 1.5 \ d_s = \) riprap depth
WASHOE COUNTY
HYDROLOGIC CRITERIA AND DRAINAGE DESIGN MANUAL

\[ L_T = \text{Length of endsill} \]
\[ H_d = D - 0.67 d_s \]
\[ H_{sw} = \text{Depth of downstream cutoff wall} \]
\[ L_A = \text{Length of riprap upstream scour drop} \]
\[ H_m = \text{Crest of wall height above main channel invert} \]
\[ D = \text{Depth of drop} \]
\[ d_{50} = \text{Median diameter of riprap} \]
\[ L_b = \text{Length of basin} = 4H_m + 0.25D \]
\[ d_s = y \text{Depth of scour} \leq 0.3D \text{ (Determine from Figure 1207)} \]

1202.1.3 Straight Drop Spillways

Presented in Figure 1208 are the design details for a straight drop spillway. The spillway produces a controlled overflow jet which is dissipated through impact on the structure floor and baffle blocks. The jet energy is also dissipated in the plunge pool created when impact blocks are used or through a hydraulic jump using the typical baffle block arrangements from the USBR stilling basin designs.

The basin design is based on the drop distance, \(Y\), and the unit discharge, \(q\), as related through the drop number, \(D\), computed as follows:

\[ D = \frac{q^2}{gY^2} \]  \hspace{1cm} (1208)

where
\[ D = \text{Drop Number (dimensionless)} \]
\[ q = \text{Unit discharge (cubic feet per second per foot of width)} \]
\[ Y = \text{Drop distance (feet)} \]

The remaining design parameters can be obtained from Figure 1208.

The impact block basin is applicable for low heads with a wide range of tailwater depths. The hydraulic jump basin may be used as long as the design parameters for the selected basin type are meet. The designer is referred to USBR, 1987, for detailed design information, guidelines, and examples.

1202.1.4 Baffled Aprons (USBR Type IX)

Presented in Figure 1209 is the Baffled Apron Stillling Basin. This structure requires no initial tailwater to be effective, although when the tailwater forms a pool into which the flow discharges, the channel bed scour is not as deep and is less extensive. The chutes are constructed on an excavated slope, 2:1 or flatter, extending to below the channel bottom. Backfill is placed over one or more rows of baffles to restore the original streambed elevation. When scour or downstream channel degradation occurs, successive rows of baffle piers are exposed to prevent excessive acceleration of the flow entering the channel. If degradation does not occur the scour creates a stilling pool at the downstream end of the chute, stabilizing the scour pattern.

Generalized design information is presented in Figure 1210. The designer is referred to PETERKA, 1978 for detailed design information, guidelines, and examples.
1202.2 ENERGY DISSIPATION STRUCTURES

Presented in Table 1201 are the types of energy dissipation structures allowed in Washoe County. By definition, energy dissipation structures may be used for both sub-critical and super-critical upstream channel (or pipe) flow conditions. For sub-critical flow conditions, these structures are designed similar to the channel drop structures discussed in the previous section. For super-critical flow conditions, the upstream channel is tied directly into the stilling basin floor (hydraulic rise) or the upstream channel is transitioned into the structure through the use of a trajectory transition section. The hydraulic design of trajectory transition sections is discussed in Section 1202.2.6.

1202.2.1 Types of Energy Dissipation Structures

Many types of stilling basins and energy-dissipating devices are available in conjunction with spillways, outlet works, and canal structures. These structures utilize blocks, sills, or other roughness elements to impose exaggerated resistance to the flow and dissipate excessive energy. The type of stilling basin selected is based upon hydraulic requirements, available space, and cost. The hydraulic jump which occurs in a stilling basin has distinctive characteristics depending on the energy of the flow which must be dissipated in relation to the depth of the flow. A comprehensive series of tests have been performed by the Bureau of Reclamation for determining the most efficient energy dissipators (PETERKA, 1978).

The energy dissipation structures discussed herein provide a wide range of structures from which to choose the most hydraulically and cost efficient structure. The reader is encouraged to review the analysis, results, and recommendation in PETERKA, 1978, prior to the final selection of the energy dissipation structure.

1202.2.2 Stilling Basins With Horizontal Sloping Aprons

The basis for design of all of the USBR stilling basins is the analysis of the hydraulic jump characteristics on horizontal and sloping aprons. The governing equation for hydraulic jumps is based on pressure-momentum theory and may be written as follows:

\[ \frac{D_2}{D_1} = 0.5 \left( (1 + 8F_{rd})^{0.2} - 1 \right) \]  (1209)

where
- \( D_1 \) = Depth of flow at jump entrance (feet)
- \( D_2 \) = Depth of flow at jump exit (feet)
- \( F_{rd} \) = Froude Number at jump entrance

The results of the USBR analysis is presented in Figure 1210. In this figure \( T_w \) is the tailwater depth necessary to create or assist in forming the hydraulic jump. Generally, \( T_w \) is greater than \( D_2 \).

The above equation is generally used to determine the approximate location of a hydraulic jump in a channel. In practical application, the actual flow depths and location of the jump will vary due to inaccuracies in estimating actual flow parameters (i.e. channel roughness, flow characteristics). The location of the jump will also vary depending on the flow rate in the channel. Therefore, from a structural and safety standpoint, horizontal and sloping apron stilling basins should not be used as energy dissipation structures without the addition of appurtenances (i.e. baffle blocks, end sills, etc.) to control the location of the hydraulic jump. Standard designs for these types of structures are discussed in the following sections.
1202.2.2.1 Short Stilling Basin (USBR Type III)

Presented in Figure 1210 and Figure 1211 is the standard design for a Type III stilling basin. The chute blocks at the upstream end of a basin tend to corrugate the jet, lifting a portion of it from the floor to create a greater number of energy dissipating eddies. These eddies result in a shorter length of jump than would be possible without them, and tend to stabilize the jump. The baffle piers act as an impact dissipation device and the end sill is for scour control. The end sill has little or no effect on the jump. The only purpose of the end sill in a stilling basin is to direct the remaining bottom currents upward and away from the channel bed.

This type of a basin is recommended at the outlet of a sloping channel drop when there is limited available space for a drop structure and adequate tailwater. This basin is relatively less expensive than other basins under similar hydraulic conditions. For insufficient tailwater, a USBR Type VI basin is recommended.

1202.2.2 Low Froude Number Basins (USBR Type IV)

Presented in Figure 1212 is the standard design of a low Froude number basin. The basin is used instead of the USBR Type II and Type III basins in order to achieve better jump characteristic at low Froude numbers (2.5 < Fr < 4.5). At these low Froude numbers, excess waves are created because the jump is not fully developed.

This basin minimizes the waves by directing jets from the tops of the baffle blocks into the roller to strengthen and intensify it. In addition, the tailwater depth (T) should be at least 1.1 x D2 (Conjugate Depth) to minimize the chance of the jump sweeping out of the basin. (See Figure 1210.) The end sill has little or no effect on the jump but rather directs the bottom currents upward and away from the channel bed.

1202.2.3 Impact Stilling Basin (USBR Type VI)

This stilling basin is an impact-type energy dissipator, contained in a relatively small box-like structure, and requiring little or no tailwater for successful performance. The general arrangement of the basin is shown on Figure 1213. This type of basin is subjected to large dynamic forces and turbulences which must be considered in the structural design. The structure should be made sufficiently stable to resist sliding against the impact load on the baffle wall and must resist the severe vibrations. Riprap should also be provided along the bottom and sides adjacent to the structure to avoid the tendency for scour of the outlet channel downstream from the end sill when shallow tailwater exists. This type of stilling basin is very effective at the outlet of storm drains or culverts where there is little or no tailwater.

1202.2.3 Hydraulic Design

The three different stilling basin configurations can be divided into two categories, basins for spillways or channels (Type III or IV) and basins for pipe outlets (Type VI). A summary of the design data for all three basin types is presented in Figure 1210. The reader is referred to Peterka, 1978, for a detailed discussion of the structural design requirements.
1202.2.4 Riprap Protection

Riprap protection shall be provided downstream of the Type III, IV, and VI stilling basins (except in fully concrete lined channels). This protection is necessary to protect the downstream channel from erosion due to eddy currents and excess velocities in the transition zone between the structure and the design channel section.

For the Type III and Type IV basin, riprap shall be installed from the end sill a distance of 4 to 10 times the design depth of flow in the downstream channel. The riprap size and thickness shall be designed in accordance with Section 800.

For the Type VI stilling basin, riprap protection shall extend downstream a distance equal to the outlet width, W, of the basin. The minimum downstream distance shall be 5 feet. A 2-foot layer of regular riprap shall be used for all basin widths of 12 feet or less. For basin widths between 12 feet and 20 feet, a 3-foot layer of heavy riprap shall be used. For basin widths greater than 20 feet, a 2-foot layer of grouted riprap shall be used.

1202.2.5 Design Flow Rates

The effectiveness of energy dissipations structures is dependent on many factors including flow rates, tail water depths, and type of dissipation structure. The structures also must function over a wide range of flow rates typical of stormwater runoff. Therefore, a minimum of the minor and major storm flow rates should be analyzed to assist in protecting the structure against drowning of the hydraulic jump or sweepout of the jump into the downstream channel. The design of the impact stilling basin shall be based on the design flow rate for the upstream pipe or channel.

1202.2.6 Trajectory Transition Section

Energy dissipation structures may be designed for either subcritical or supercritical upstream flow conditions. For subcritical flow, an abrupt change in grade at the structure entrance performs satisfactorily. However, for supercritical flow, the flow tends to separate and spring away at any abrupt change in grade. Therefore, to avoid the possibility of flow separation from the channel floor, the floor shape should be flatter than the trajectory of a free discharging flow jet.

Presented in Figure 1214 is a typical design of a trajectory transition section. The curvane of the trajectory section can be determined by the following equation (USBR, 1987).

\[
y = x \tan \theta + \frac{x^2}{K(4(d + h_\nu) \cos^2 \theta)}
\] (1210)

where
- \(Y\) = Change in vertical elevation (feet)
- \(X\) = Change in horizontal location (feet)
- \(K\) = Safety factor
- \(d\) = Depth of flow at trajectory entrance (feet)
- \(h_\nu\) = Velocity head at trajectory entrance (feet)
- \(\theta\) = Slope angle from horizontal of the upstream channel (degrees)
The safety factor, $K$, should be equal to or greater than 1.5 to assure positive contact pressure.

The trajectory section should be connected to the stilling basin apron by a short, steep chute section. This section should be at a slope between 1.5 horizontal to 1 vertical and 3 horizontal to 1 vertical with 2 horizontal to 1 vertical preferred. In no case should the slope be flatter than 6 horizontal to 1 vertical.
# CHANNEL DROP AND ENERGY DISSIPATION STRUCTURES

<table>
<thead>
<tr>
<th>Structure</th>
<th>Upstream Flow Class</th>
<th>Maximum Allowed Drop Height (feet)</th>
<th>Maximum Allowed Flow Rate (cfs/ft)</th>
<th>Maximum Allowed Inflow Velocity (fps)</th>
<th>Basin Entrance Froude No.</th>
<th>Maximum Basin Entrance Velocity (fps)</th>
<th>Required Cross-Section Geometry</th>
<th>Reference Figure Number</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sloping Grouted Boulder</td>
<td>SUB</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>TRAP</td>
<td>1202 and 1203</td>
</tr>
<tr>
<td>Vertical Riprap Drop</td>
<td>SUB</td>
<td>4</td>
<td>35</td>
<td>7</td>
<td>-</td>
<td>-</td>
<td>TRAP</td>
<td>1206</td>
</tr>
<tr>
<td>Straight Drop</td>
<td>SUB</td>
<td>4</td>
<td>4</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>RECT</td>
<td>1208</td>
</tr>
<tr>
<td>Baffled Apron (USBR Type IX)</td>
<td>SUB</td>
<td>-</td>
<td>60</td>
<td>12</td>
<td>-</td>
<td>-</td>
<td>RECT</td>
<td>1209</td>
</tr>
<tr>
<td>Short Stilling Basin (USBR Type II)</td>
<td>SUB or SUPER</td>
<td>-</td>
<td>200</td>
<td>-</td>
<td>4.5 to 17</td>
<td>60</td>
<td>RECT</td>
<td>1211</td>
</tr>
<tr>
<td>Low Froude Number Basin (USBR Type IV)</td>
<td>SUB or SUPER</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>2.5 to 4.5</td>
<td>-</td>
<td>RECT</td>
<td>1212</td>
</tr>
<tr>
<td>Baffled Outlet (USBR Type VI)</td>
<td>SUB or SUPER</td>
<td>NA</td>
<td>30</td>
<td>NA</td>
<td>50</td>
<td>NA</td>
<td>-</td>
<td>1213</td>
</tr>
</tbody>
</table>

Column Descriptions

1. **SUB** = Subcritical (Fr < 0.8)
   **SUPER** = Supercritical (F > 1.15)

2. Drop height measured from bottom of upstream channel to bottom of downstream channel.

3. Flow Rate = Normal depth (Yn) multiplied by normal velocity (Vn).

4. Inflow Velocity = Upstream normal channel velocity.

5. Froude number for flow conditions at entrance to structure apron.

6. Velocity for flow conditions at entrance to structure apron.

7. Cross-section of chute and stilling basin.

8. Reference to figures in this manual.

* Total flow should be less than 400 cfs.

**Note:** Tail water (TW) is measured from invert of basin to water surface in channel immediately downstream of end sill.
<table>
<thead>
<tr>
<th>DESIGN PARAMETER</th>
<th>NON-EROSSIVE SOILS</th>
<th>EROSSIVE SOIL</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>DROP HEIGHT ≤ 3 feet</td>
<td>DROP HEIGHT ≤ 3 feet</td>
</tr>
<tr>
<td>Maximum Slope of Drop, $S_d$</td>
<td>4 Horizontal to 1 Vertical</td>
<td>4 Horizontal to 1 Vertical</td>
</tr>
<tr>
<td>Uniform Rock Size, $D_u$</td>
<td>See Table 1203</td>
<td>See Table 1203</td>
</tr>
<tr>
<td>Grout Thickness, $D_g$</td>
<td>See Table 1203</td>
<td>See Table 1203</td>
</tr>
<tr>
<td>Basin Depression, $B$</td>
<td>2.0 feet for $Q ≤ 1000$ cfs</td>
<td>2.25 feet for $Q ≤ 1000$ cfs</td>
</tr>
<tr>
<td></td>
<td>1.75 feet for $1000 &lt; Q ≤ 3000$ cfs</td>
<td>2.0 feet for $1000 &lt; Q ≤ 3000$ cfs</td>
</tr>
<tr>
<td></td>
<td>1.5 feet for $Q &gt; 3000$ cfs</td>
<td>1.75 feet for $Q &gt; 3000$ cfs</td>
</tr>
<tr>
<td>Grouted Rock Approach, $A_D$</td>
<td>5 feet for $Q ≤ 1000$ cfs</td>
<td>5 feet for $Q ≤ 1000$ cfs</td>
</tr>
<tr>
<td></td>
<td>10 feet for $1000 &lt; Q ≤ 3000$ cfs</td>
<td>10 feet for $1000 &lt; Q ≤ 3000$ cfs</td>
</tr>
<tr>
<td></td>
<td>15 feet for $Q &gt; 3000$ cfs</td>
<td>15 feet for $Q &gt; 3000$ cfs</td>
</tr>
<tr>
<td>Basin Length, $L_b$</td>
<td>5 * $W$ depth but not less than 20 feet</td>
<td>5 * $W$ depth but not less than 25 feet</td>
</tr>
<tr>
<td></td>
<td>See Section 800</td>
<td>See Section 800</td>
</tr>
<tr>
<td>Channel width, $B_c$</td>
<td>See Section 800</td>
<td>See Section 800</td>
</tr>
<tr>
<td>Slope face length, $L_s$</td>
<td>$S_d$ * Drop Height</td>
<td>$S_d$ * Drop Height</td>
</tr>
<tr>
<td></td>
<td>13 feet</td>
<td>15 feet</td>
</tr>
<tr>
<td>Grounded rock exist $A_D$, $L$</td>
<td>10 feet min.</td>
<td>10 feet min.</td>
</tr>
</tbody>
</table>

* For submerged drops add 10 feet to the basin length or use a hydraulic jump analysis to refine the main basin length.
## Sloping GROUTED Boulder DROP Rock and GROUT Thickness

<table>
<thead>
<tr>
<th>Depth of Rock Layer Which is Equivalent to the Minimum Boulder Size, ( D_r ) (Inches)</th>
<th>Depth of GROUT Layer, ( D_s ) (Inches)</th>
</tr>
</thead>
<tbody>
<tr>
<td>18</td>
<td>12</td>
</tr>
<tr>
<td>24</td>
<td>18</td>
</tr>
<tr>
<td>30</td>
<td>24</td>
</tr>
<tr>
<td>36</td>
<td>28</td>
</tr>
<tr>
<td>42</td>
<td>32</td>
</tr>
</tbody>
</table>

**Reference:** Urban Drainage and Flood Control District, 1990
WASHOE COUNTY
HYDROLOGIC CRITERIA AND DRAINAGE DESIGN MANUAL

DROP STRUCTURES

A. SLOPING CHANNEL DROP

B. VERTICAL CHANNEL DROP
SLOPING GROUTED BOULDER DROP

CONTROL GRADE AT CREST
Dg THICKNESS OF GROUT
TOP OF GROUTED ROCK AT SIDES

Yt
Ym
MAIN CHANNEL

TRICKLE INVERT

Lx

LF

LB

LT

Hd

Y+FREEBOARD

EXCAVATE 12" MIN. TRENCH AND BACKFILL WITH CONC., OTHER OPTIONS POSSIBLE, BUT CUTOFF ESSENTIAL.

PERPENDICULAR WEEP DRAINS (ON 10' CENTERS ± ACROSS THE DROP)
DRAIN MATERIAL BETWEEN WEEP PIPES AND ACROSS THE DROP FACE

PROFILE

OUTLINE OF PROJECTING BOULDER DOWNSTREAM
0.6 TO 0.8 OF Yc IN TRICKLE

GROUT VOID SPACE TO SUBGRADE

SECTION A
NOTE:
SEE TABLE 1202
FOR DEFINITIONS AND
VALUES OF DESIGN
PARAMETERS
TYPICAL GROUTED BOULDER PLACEMENT

PLACE BOULDERS IN STAIRSTEP FASHION ON
SLOPE WITH FLATTEST SURFACE SET HORIZONTAL
AND ON THE TOP

SURFACES OF
BOULDERS ABOVE
DESIGN TOP OF
GROUT TO REMAIN
CLEAN AND FREE
OF GROUT

SLOPE VARIES

PREPARED SUBGRADE

\[ D_r = \text{DEPTH OF ROCK LAYER WHICH IS EQUIVALENT TO THE MINIMUM BOULDER SIZE} \]

\[ D_g = \text{DEPTH OF GROUT LAYER} \]
4" ADS non-perforated pipes or approved equal, spaced 10' O.C. maximum. May be adjusted to fit between boulders. Crushed or punctured pipe shall be replaced.

Use 90° elbow to outlet pipe at level of grout clean excess grout and provide slope for free draining.

Prepared subgrade

Weep drain manifold (end view) 4" ADS perforated pipe (or approved equal) provide 4" tees to outlet pipes and end caps as required.

Place approved filter fabric over granular material to prevent contamination by grout.

Granular weep drain filter material minimum 6" thickness surrounding pipe system at all points.
VERTICAL RIPRAP DROP

SECTION B

PARTIAL PLAN 3bt = ZONE OF HEAVIER ROCK SHAPE AS SHALLOWER TRICKLE DEPENDING ON GEOMETRY

PROFILE

VERSION: December 2, 1998 REFERENCE: McLAUGHLIN WATER ENGINEERS, 1986 FIGURE 1206
CURVES FOR SCOUR DEPTH AT VERTICAL DROP
NUMBERS ON CURVES ARE VALUES OF \( Y_2 /D \)

VALUES OF \( d_8 /D \)

VALUES OF \( H_m /D \)

D/d_{50} = 0.2
0.6
0.8
0.4
0.2
0.0

D/d_{50} = 0.4
1.0
1.2
0.6
0.4
0.2
0.0

D/d_{50} = 0.8
1.0
1.2
0.6
0.4
0.2
0.0

D/d_{50} = 3 (EXTAPOLATED)

VALUES OF \( d_8 /D \)

VALUES OF \( H_m /D \)

0.2
0.4
0.6
0.8

0.2
0.4
0.6
0.8
1.0
1.2
1.3
1.4

REFERENCE:
McLaughlin Water Engineers, Ltd., Evaluation of and Design Recommendations For Drop Structures In The Denver Metropolitan Area, December, 1986,

FIGURE 1207
NOTE: See Figure 1210 for design data
DESIGN DATA-USBR TYPE STILLING BASINS

GENERAL INVESTIGATION OF THE HYDRAULIC JUMP ON HORIZONTAL APRONS
(BASIN I)

Jumps occur on flat floor with no chute blocks, bottle sides, or end sills. Usually not a practical basin because of excessive length. Elements and characteristics of jumps for complete range of Froude numbers is determined to aid designers in selecting more practical basins II, III, IV, V, and VI.

STILLING BASIN WITH SLOPING APRON
(BASIN II)

For use where structural economies dictate desirability of sloping apron, usually on high dam spillways. Needs greater tailwater depth than horizontal aprons.

REFERENCE:
"HYDRAULIC DESIGN OF STILLING BASINS AND ENERGY DISSIPATORS," EM25 BR, JANUARY 1978
WASHOE COUNTY
HYDROLOGIC CRITERIA AND DRAINAGE DESIGN MANUAL

DESIGN DATA-USBR TYPE STILLING BASINS

BAFFLED APRON FOR CANAL OR SPILLWAY DROPS
(BASIN IX)

For use in flow ways where water is to be
leveled from one level to another. The
baffles are present under acceleration of
the flow as it passes down the chute.

Since the flow velocities entering the
downstream channel are relatively low, no
stilling basin is required. The chute may be
designed to discharge up to 60 cubic feet per
second per foot of width and the area may be
designed as structurally feasible.

DESIGN PROCEDURE

The baffles should be spaced for the
maximum respective discharges @, to
60 c.f.s. per foot of width.

The discharge per inch of width should be no
less than 1 ft. per second.

See Figures 10-12, 16-17, and 19 for sample
application.

The total area of the baffles should be
10% of the total cross-sectional
area of the flow. The area of the
baffles may be increased to maintain
flow velocities below 3 ft. per second.

The slope of the baffles should be
at least 1:2, but not greater than
1:5.

The area of the baffles should be
arranged to maintain flow velocities
below 3 ft. per second.

The overall design should be such that
flow velocities are maintained at
1 ft. per second or less.

The baffles should be spaced
at least 10 ft. apart to
prevent flow disturbance.

The design should be such that
flow velocities are maintained at
1 ft. per second or less.

The overall design should be such that
flow velocities are maintained at
1 ft. per second or less.

The baffles should be spaced
at least 10 ft. apart to
prevent flow disturbance.

The design should be such that
flow velocities are maintained at
1 ft. per second or less.

The baffles should be spaced
at least 10 ft. apart to
prevent flow disturbance.

The design should be such that
flow velocities are maintained at
1 ft. per second or less.
DESIGN DATA-USBR TYPE STILLING BASINS

SHORT STILLING BASINS FOR CANAL STRUCTURES, SMALL OUTLET WORKS AND SMALL SPILLWAYS (BASIN III)

Jump and basin lengths reduced about 50 percent with chess blocks, bolted plates, and solid end sill.
For use on small spillways, outlet works, small canal structures where J does not exceed 50-60 feet per second and Froude number is about 0.5.

MINIMUM TW DEPTHS (SWEETWATER)

LENGTH OF JUMP

HEIGHT OF SWEETWATER AND END SILLS

WATER SURFACE AND PRESSURE PROFILES

REFERENCE:
"HYDRAULIC DESIGN OF STILLING BASINS AND ENERGY DISSIPATORS," EM25 BR, JANUARY 1978

FIGURE 1210
3 OF 3
SHORT STILLING BASIN  
(USBR TYPE III)

NOTE: See Figure 1210 for design data

LOW FROUDE NUMBER STILLING BASIN
( USBR TYPE IV )

Fractional space

w = Max. tooth width Dₜ

Space = 2.5 w

Top surface on 5° slope

ZDₜ

ZDᵢ, min.

NOTE: See Figure 1210 for design data

REFERENCES:
IMPACT STILLING BASIN
(USBR TYPE VI)

NOTE:
1. See Figure 1210 for design data
2. Refer to reference for structural details

REFERENCE:
"Design of Small Canal Structures",
USDI, BR, Denver 1974
# TABLE OF CONTENTS

<table>
<thead>
<tr>
<th>Section</th>
<th>Title</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>1301</td>
<td>INTRODUCTION</td>
<td>1301-1302</td>
</tr>
<tr>
<td>1301.1</td>
<td>Definition of Regional Facilities</td>
<td>1302</td>
</tr>
<tr>
<td>1301.2</td>
<td>Definition of Local Facilities</td>
<td>1302</td>
</tr>
<tr>
<td>1301.2.1</td>
<td>Local Minor Facilities</td>
<td>1302</td>
</tr>
<tr>
<td>1301.2.2</td>
<td>Local Major Facilities</td>
<td>1302</td>
</tr>
<tr>
<td>1302</td>
<td>DETENTION DESIGN GUIDELINES AND STANDARDS</td>
<td>1303-1304</td>
</tr>
<tr>
<td>1302.1</td>
<td>Regional Detention</td>
<td>1303</td>
</tr>
<tr>
<td>1302.2</td>
<td>Local Detention</td>
<td>1304</td>
</tr>
<tr>
<td>1302.2.1</td>
<td>Local Minor Detention</td>
<td>1304</td>
</tr>
<tr>
<td>1302.2.2</td>
<td>Local Major Detention</td>
<td>1304</td>
</tr>
<tr>
<td>1303</td>
<td>HYDROLOGIC DESIGN METHODS AND CRITERIA</td>
<td>1305-1307</td>
</tr>
<tr>
<td>1303.1</td>
<td>Inflow Hydrograph</td>
<td>1305</td>
</tr>
<tr>
<td>1303.1.1</td>
<td>HEC-1/TR-20 Method</td>
<td>1305</td>
</tr>
<tr>
<td>1303.1.2</td>
<td>Rational Method</td>
<td>1306</td>
</tr>
<tr>
<td>1303.2</td>
<td>Detention Basin Design Outflow Limitations</td>
<td>1306</td>
</tr>
<tr>
<td>1303.2.1</td>
<td>Regional Facilities</td>
<td>1306</td>
</tr>
<tr>
<td>1303.2.2</td>
<td>Local Facilities</td>
<td>1306</td>
</tr>
<tr>
<td>1303.3</td>
<td>Hydrologic Calculation Methods</td>
<td>1306</td>
</tr>
<tr>
<td>1303.3.1</td>
<td>HEC-1/TR-20 Method</td>
<td>1307</td>
</tr>
<tr>
<td>1303.3.2</td>
<td>Rational Method</td>
<td>1307</td>
</tr>
<tr>
<td>1304</td>
<td>OUTLET STRUCTURES</td>
<td>1307-1311</td>
</tr>
<tr>
<td>1304.1</td>
<td>Low Flow Outlets</td>
<td>1307</td>
</tr>
<tr>
<td>1304.1.1</td>
<td>Minimum Conduit Size</td>
<td>1307</td>
</tr>
<tr>
<td>1304.1.2</td>
<td>Flow Calculations</td>
<td>1307</td>
</tr>
<tr>
<td>1304.1.2.1</td>
<td>Pipe Outlets</td>
<td>1307</td>
</tr>
<tr>
<td>1304.1.2.2</td>
<td>Orifices</td>
<td>1308</td>
</tr>
<tr>
<td>1304.1.2.3</td>
<td>Weirs</td>
<td>1310</td>
</tr>
<tr>
<td>1304.2</td>
<td>Spillways</td>
<td>1310</td>
</tr>
<tr>
<td>1304.2.1</td>
<td>Sizing Requirements</td>
<td>1310</td>
</tr>
<tr>
<td>1304.2.2</td>
<td>Flow Calculations</td>
<td>1311</td>
</tr>
<tr>
<td>1304.2.3</td>
<td>Spillway Design</td>
<td>1311</td>
</tr>
<tr>
<td>1305</td>
<td>DEBRIS AND SEDIMENTATION</td>
<td>1311</td>
</tr>
<tr>
<td>1305.1</td>
<td>Trash Racks</td>
<td>1311</td>
</tr>
<tr>
<td>1305.2</td>
<td>Sedimentation</td>
<td>1311</td>
</tr>
</tbody>
</table>

December 2, 1996
WASHOE COUNTY
HYDROLOGIC CRITERIA AND DRAINAGE DESIGN MANUAL

1306 DESIGN STANDARDS AND CONSIDERATIONS
1306.1 - Dam Safety
1306.2 - Grading Requirements
1306.3 - Depth Limits
1306.4 - Trickle Flow and Basin Dewatering
1306.5 - Embankment Protection
1306.6 - Maintenance Requirements
1306.7 - Local Detention Basin Sitting Guidelines

1307 EXAMPLE APPLICATIONS
1307.1 - Example: Detention Pond Outlet Sizing
1307.2 - Example: Rational Formula detention Method

LIST OF TABLES

1301 INFLOW HYDROGRAPH FOR EXAMPLE IN SECTION 1308.1
1302 HEC-1 RUN FOR EXAMPLE IN SECTION 1308.2

LIST OF FIGURES

1301 BASIN GEOMETRY
1302 V-NOTCH WEIR COEFFICIENTS
1303 OGEE-CRESTED WEIR COEFFICIENTS
1304 HYDROGRAPH FOR EXAMPLE IN SECTION 1208.2
SECTION 1300
DETENTION

1301 INTRODUCTION

The main purpose of a detention basin is to temporarily store runoff and reduce peak discharge by allowing flow to be discharged at a controlled rate. This controlled discharge rate is based on either limited downstream capacity (regional and local facilities) or on a limit on the increase in flows over pre-development conditions (local facilities only). Regional and local detention facilities are more fully discussed below. Washoe County Policy regarding detention basin design is presented in the "Policy" Section 303.7.

1301.1 DEFINITION OF REGIONAL FACILITIES

Regional detention facilities are those identified in the current Washoe County flood control master plan or as designated by Washoe County. Generally, these facilities control flow on major drainageways, are of major proportion, 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.

1301.2 DEFINITION OF LOCAL 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.

1301.2.1 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 pre-development hydrology, and the detention structures are generally small (0.01 to 1 Ac-Ft). Detention storage volume may be provided as small landscaped or turfed basins, parking lot storage, roof top storage, or a suitable combination of all three.

1301.2.2 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 a double function. 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.
1302 DETENTION DESIGN GUIDELINES AND STANDARDS

Certain guidelines for detention basin design need to be identified in order to properly design facilities. These guidelines cover items such as outlet flows, spillway sizing, and sedimentation. The following sections describe major guidelines governing detention basin designs.

1302.1 REGIONAL DETENTION

The design of regional detention facilities shall be coordinated with Washoe County. Regional detention guidelines include:

1. Regional detention basins are preferred to smaller local detention basins.
2. Off-channel detention basins are preferred.
3. Multi-use (e.g. recreation) must be considered in the design of detention basins.
4. Below-grade detention basins are preferred to above-grade facilities.
5. Basins should be sited on publicly-owned lands whenever possible.

Regional Detention Standards include:

1. Detention basin outlet capacity shall be based on the downstream channel capacities (existing or Master Planned) with consideration given to inflows occurring below the detention basin.
2. All detention basins are required to properly function under all debris and sedimentation conditions.
3. In-channel detention basins typically will be required to safely pass the PMF discharge as a minimum. Hydrometeorological Report No. 49 (HMR 49, 1977) shall be used to calculate PMF flows unless a different analysis is required by the State Engineer for jurisdictional dams.
4. Detention ponds shall be designed to include provisions for security/public safety. This may include fencing of non-multi-use facilities or additional safety related improvements for multi-use facilities.
5. Basins should be drained in not more than 7 days with the preferred standard drain time set at 24 hours. (Drain time is defined as the time from the end of precipitation (end of the 24 hour storm) until the basin is drained of 90% of design capacity).
6. A minimum of one foot of freeboard is required above the emergency spillway design water surface elevation (See Figure 1301) (Additional freeboard may be required by the State Engineer for jurisdictional dams).
7. Basins shall be self-regulating (passive).
8. Dams greater than 20 feet in height or impounding more than 20 acre-feet of water above grade must be approved by the State Engineer (The designer shall verify the current State Engineer jurisdictional limits as these may change from time to time).
9. Inflows shall be based on ultimate development conditions and Master Planned tributary area.

10. Design of all detention basins shall include emergency spillways.

11. Embankment protection will be considered for each basin.

1302.2 LOCAL DETENTION

Since the functions of local minor and local major detention facilities are different, the development guidelines for each are described separately below:

1302.2.1 Local Minor Detention

Local minor detention may be required for developments in hydrologic basins of less than 20 acres in size. The need for local minor detention is based on analysis of pre- and post-development hydrology.

Local Minor Detention Guidelines include:

1. Public safety should be paramount in all designs.

2. Accommodation of debris and sedimentation should be considered in all designs.

Local Minor Detention Standards include:

1. Post-development peak discharges must not exceed pre-development discharges if downstream facilities lack adequate capacity to handle the increased flow rates.

2. Basins must drain completely in less than 24 hours after the end of the 24-hour design storm.

3. A minimum of one foot of freeboard is required above the major design storm water surface elevation.

1302.2.2 Local Major Detention

Local major detention may be required in accordance with Section 303.7 or where upstream off-site flows must be intercepted and controlled to protect the proposed downstream development. Design of such basins should be coordinated with Washoe County.

Local Major Detention Guidelines include:

1. Off-channel detention basins are preferred.

2. All basins are required to properly function under debris and sedimentation conditions. Adequate access must be provided for the necessary equipment to periodically remove accumulated sediment and debris.

3. Multi-use (e.g., recreation) should be considered for all detention basins.
4. Below-grade detention basins are preferred to above-grade detention basins.

Local Major Detention Standards include:

1. Detention basin outlet capacity will be based on either (a) downstream conveyance system capacities with consideration given to inflows below the detention basin or (b) pre- and post-development hydrology.

2. Detention basins shall be drained in not more than 3 days with the preferred drain time set at 24 hours after the end of the 24-hour design storm.

3. A minimum of one foot of freeboard will be required above the emergency spillway design water surface elevation.

4. Detention basins shall be self-regulating (passive).

5. Emergency outlets shall be incorporated on all detention basins.

1303 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/TR-20 vs Rational method). If HEC-1/TR-20 is used, a full hydrograph is available for traditional storage routing. If the Rational method is used, a simplified triangular procedure has been developed for use in Washoe County.

1303.1 INFLOW HYDROGRAPH

The determination of required detention storage is based on volume calculations derived from the inflow hydrograph and the maximum outlet flow. The inflow hydrograph shall be based on ultimate development conditions.

1303.1.1 HEC-1/SCS TR-20 Method

The hydrograph for local and regional facilities may be calculated using HEC-1/TR-20 (Section 700). HEC-1/TR-20 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.

1303.1.2 Rational Method

For the design of detention facilities in hydrologic basins of less than 5 acres, a simple, "triangular" hydrograph has been developed using the Rational method. The application of the Rational method is described in Section 704.

The Rational method is traditionally used solely for peak runoff estimation, but a hydrograph can be constructed using the following assumptions:

a) Peak flow occurs at the \( t_r \);

b) Flow increases linearly from \( Q = 0 \) to \( Q = Q_{\text{peak}} \) for \( t = 0 \) to \( t = t_r \);
WASHOE COUNTY
HYDROLOGIC CRITERIA AND DRAINAGE DESIGN MANUAL

c) Flow decreases linearly from \( q = Q_{\text{peak}} \) to \( Q = 0 \) for \( t = t_c \) to \( t = 2t_c \).

The resulting hydrograph is triangular in shape and has a volume given by

\[
V = 60 \left( t_c \cdot Q_p \right)
\]

(1301)

Where

- \( V \) = Volume (cubic feet)
- \( t_c \) = Time of concentration (minutes)
- \( Q_p \) = Peak flow rate (cubic feet per second)

1303.2 DETENTION BASIN DESIGN 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).

1303.2.1 Regional Facilities

The allowable release rate for regional facilities in the Master Plan is typically based on the non-damaging capacity of the downstream conveyance system or on the conveyance capacity of the system as improved by the detention project. The design maximum outlet capacity of a regional facility must be coordinated with Washoe County.

1303.2.2 Local Facilities

The outflow limitation for local facilities is stated in Section 303.7. Existing flow conditions will be calculated based on development conditions that exist prior to construction of the project. The allowable outlet rate is equal to the existing peak runoff rate (and/or downstream conveyance capacity for local major detention facilities).

1303.3 HYDROLOGIC CALCULATION METHODS

After the inflow hydrograph has been calculated (Section 1303.1) and the outflow limits (Section 1303.2) have been established, the storage volume requirement can be estimated. Separate methods for calculating required storage are used depending on the method used to estimate the inflow hydrograph.

1303.3.1 HEC-1/SCS TR-20 Method

In order to calculate the required storage volume at a particular detention basin site, the following information must be available or prepared:

a) Inflow hydrograph;

b) Outlet capacity limitation;

c) Proposed outlet discharge vs. elevation data for the proposed basin site;
d) Proposal storage vs. elevation data for the proposed basin site;
e) Proposed drain time for the proposed basin site.

The HEC-1 or TR-20 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. This method is shown in the example in Section 1308.1.

1303.3.2 Rational Method

After the inflow hydrograph (Section 1303.1) and the outflow limitation (Section 1303.2) have been determined, the required storage volume can be calculated. The estimated hydrograph is plotted at a suitable scale. The maximum outflow rate is plotted on the receding limb of the hydrograph. A straight line is constructed from the origin to the outlet limit on the receding limb. The area above this line is the required storage volume. The estimation of required storage volume is shown in the example in Section 1308.2.

1304 OUTLET STRUCTURES

This section describes the methods to be used to size outlet structures for detention facilities. Although the methods presented are recommended for the hydraulic structures described, alternative hydraulic techniques may be appropriate depending upon the configuration of the outlet structure.

1304.1 LOW FLOW OUTLETS

The low flow outlet (principal spillway) is sized to control discharge from a basin as set forth in Section 1303.2.

In traditional detention basins, outlet control is usually provided by a culvert or large (> 18" diameter) pipe conduit. The types of low flow control typically used for parking lot detention are small under-sidewalk weirs or pipes.

1304.1.1 Minimum Conduit Size

To reduce the potential for outlet clogging by debris, minimum conduit sizes have been set for Washoe County. The minimum conduit size for use in detention facilities is 18-inch diameter or equivalent. Orifice plates shall be utilized to reduce flows from these minimum pipe sizes.

1304.1.2 Flow Calculations

1304.1.2.1 Pipe Outlets

The capacity of outlets shall be calculated using nomographs in Section 1100.

1304.1.2.2 Orifices

The capacity of a small closed conduit (Section 1100 nomographs are not applicable) is estimated assuming inlet control using the orifice equation shown below:
Q = CA (2gh)^n \hspace{1cm} (1302)

Where

Q = discharge (cubic feet per second)
A = cross-sectional area of conduit (square feet)
g = gravitational constant = 32.2 ft/sec^2
h = head, measured from line of orifice opening (feet)
C = orifice coefficient = 0.65 (dimensionless)

The orifice coefficient to be used in all calculations is 0.65 unless a deviation from this value is approved by Washoe County. An example of this calculation is provided in Example 1308.1.

1304.1.2.3 Weirs

There are two main types of weirs used in detention basin outlet structures: sharp-crested and broad-crested. Sharp-crested weirs have a sharp upstream edge so formed that water springs clear of the crest. A broad-crested weir has a horizontal or nearly horizontal crest sufficiently long in the direction of flow so that the overflowing sheet of water, or nappe, will be supported and hydrostatic pressures will be fully developed for at least a short distance. The V-notch weir is a type of sharp-crested weir that is sometimes used in outlet structures. The V-notch weir has a triangular opening.

The capacity of a weir can be estimated using the following equations (Brater and King, 1976):

1. Horizontal crested weirs

   For horizontal crested weirs (broad-crested and sharp-crested):

   \[ Q = CLH^{3/2} \hspace{1cm} (1303) \]

   Where

   \begin{align*}
   Q &= \text{Flow (cubic feet per second)} \\
   C &= \text{Weir coefficient} \\
   &= 3.3 \text{ for a sharp-crested weir} \\
   &= 2.65 \text{ for a broad-crested weir} \\
   L &= \text{Effective horizontal length of weir (feet)} \\
   H &= \text{Head (feet)}
   \end{align*}

True sharp-crested weirs are seldom used in hydraulic structures and are normally used to measure the flow of water, but weirs can sometimes be treated as sharp-crested weirs under the correct conditions. When the head is greater than or equal to two times the breadth of the weir crest, the weir may be considered a sharp-crested weir. When the head is less than or equal to one-half the breadth of the weir crest, the weir is considered a broad-crested weir. This relationship is summarized below:

\[ C_1 = 2.63 \text{ when } H \geq 2 \text{ (W)} \]

\[ C_1 = 3.3 \text{ when } H \leq 0.5 \text{ (W)} \]
Where

\( W = \) breadth of weir crest (feet)

When 0.5 \((W) \leq H \leq 2 \)(W), then a straight line approximation may be used to obtain a value of \( C \).

End contraction occur when the horizontal weir opening does not extend the full width of the approach channel. Water flowing near the walls must move toward the center of the channel to pass over the weir, thus causing a contraction of the flow. The flow width continues to contact as it passes over the crest. Below the crest, the flow has a width less than the crest width. Flow will also be contracted at bends in the weir (i.e. 4-sided drop inlet).

The effective length of a weir with contracted flow is:

\[
L_e = L - 0.1 NH
\]  \hspace{1cm} (1305)

Where

- \( L_e \) = Effective horizontal length of weir (feet)
- \( L \) = Measured length of weir crest (feet)
- \( N \) = Number of end contractions + \( \Sigma \) of bends
- \( H \) = Head (feet)

For instance, if the outlet from the detention basin was a 3-sided weir with 2-90° bends and flow contractions at both ends of the weir, \( N \) would be:

\( N = 2 + 2 = 4 \)

The head is measured from the weir crest to the water surface elevation at a distance 2.5 \((H)\) upstream from the weir, to be beyond the drop in the water surface (surface contraction) near the weir.

2. V-notch weirs

For V-notch weirs (BRATER, 1976):

\[
Q = C_i \tan (\theta/2)^{1/2}
\]  \hspace{1cm} (1304)

Where

- \( C_i \) = Weir coefficient
- \( \theta \) = Angle of V-notch (degrees)
- \( H \) = Head (feet)

Figure 1302 provides values of \( C_i \) for a V-notch weir for values of Head from 0.2 feet to 0.8 feet. For values of head greater than 0.8 feet, assume that \( C_i \) is 2.5 (BRATER, 1976).

The head is measured from the notch elevation to the water surface elevation at a distance 2.5 \((H)\) upstream from the weir.

The V-notch weir is better than a rectangular sharp-crested weir for measuring low discharges.
since flow over a V-notch weir starts at a point and the discharge and width of flow increases as a function of depth.

1304.2 SPILLWAYS

Since storm flows may enter a detention facility in excess of the maximum design flow of the outlet works, a safe method of passing these flows must be provided. All detention facilities must have the ability to pass flows in excess of the major design storm without endangering the structural integrity of the facility or diverting flows from their historic drainage pattern.

A detention basin may have more than one spillway, or in the case of local facilities, the complete structure may be designed to act as an overflow section. If a basin has only one spillway, it must be able to pass both the design flow and a larger flow to provide a margin of safety. These larger flows are discussed in Section 1302. If the geometry of the basin site does not allow for a single spillway to serve these two flows, two spillways may be provided. The principal spillway will be designed to handle the major design storm flow. If the flow is greater than the major design storm flow, the emergency spillway would allow these greater flows to be passed safely. For minor local detention structures, the structure may be designed to be safely overtopped and the structure itself is the emergency spillway.

1304.2.1 Sizing Requirements

All detention basins in Washoe County shall have emergency spillways which safely pass the following peak flow rates:

1. Regional Facilities: The spillway will be required to pass, as a minimum, half the runoff from a PMF event if approval of the State Engineer’s Office is not required (1306.1).

2. Local Major Facilities: The spillway will be required to pass, as a minimum, an inflow hydrograph developed by using twice the point precipitation of the major storm if approval of the State Engineer’s Office is not required (1306.1).

3. Local Minor Facilities: Emergency spillways for local minor facilities shall be designed to pass the major storm if approval of the State Engineer’s Office is not required (1306.1).

Off line detention basins (basins for which a controlled amount of runoff is diverted to the basin for detention purposes) shall only be required to have an emergency spillway sized to safely pass the design 100-year inflow rate plus all uncontrolled inflow tributary to said basin.

1304.2.2 Flow Calculations

The equation for flow over a spillway is the same as that for flow over a horizontal crested weir given in Section 1304.1.2 (Equation 1303). The discharge coefficient, C, for a broad or ogee-crested weirs is normally used to estimate the flow over the spillway, depending on the type of spillway used. A graph for coefficient estimation for ogee-crested weirs is provided in Figure 1303.
1304.2.3 **Spillway Design**

All spillways (except vertical drops) should be designed so the water is always in positive contact with the spillway invert. The profile of the spillway can be designed according to the trajectory transition section discussed in Section 1202.2.6 and shown in Figure 1214.

1305 **DEBRIS AND SEDIMENTATION**

The performance and reliability of detention facilities can be reduced by natural and man-made debris. Naturally occurring sedimentation can, over a period of time, reduce the storage capacity of a detention basin and thereby reduce the degree of flood protection provided. The obstruction of low flow conduits by debris can reduce outlet capacity and cause the premature filling of the detention basin with storm water, again reducing the flood protection provided by the structure. Consequently, adequate care must be exercised in design to provide for protection of the outlet works from debris and for the control and removal of sedimentation in the basin. The designer should become familiar with the provisions of Section 1400 as related to the potential for debris flow into the proposed detention basin.

1305.1 **TRASH RACKS**

All outlet works and low flow conduits shall be provided with a trash rack for debris control. The trash rack shall provide a maximum bar spacing not to exceed two-thirds of the outlet opening or diameter or 6-inches, whichever is less. In addition, the open area of the trash rack shall be at least four times larger than the open area of the detention pond outlet. Trash racks should be hinged at the top to permit lifting and cleaning and should slope at 3:1 to 5:1 (horizontal to vertical) to permit debris to float up and down as the water level rises and falls. Water velocity through the rack should be minimized for safety concerns and to minimize head loss.

1305.2 **SEDIMENTATION**

The storage volume of a detention basin can be reduced and/or eliminated by sediment deposition. Depending on the cover and soil conditions in a watershed, detention basin filling may happen slowly over a period of many years or, in extreme cases, during one storm event.

Sedimentation effects may be reduced by the construction of debris basins (Section 1400) upstream of the detention facility or by providing additional storage capacity in the detention facility for storage of sediment. Section 1400 presents some basic information regarding debris and sedimentation control facilities.

Washoe County will review detention basins for sedimentation risks and may require additional sediment storage volume.

1306 **DESIGN STANDARDS AND CONSIDERATIONS**

The following section describes current standards and special considerations for detention design.

1306.1 **DAM SAFETY**

All dams which store more than 20 acre-feet of water or have an embankment 20 feet or greater in height must be approved by the State Engineer (The designer shall verify the current State Engineer jurisdictional limits as these may change from time to time).
1306.2 GRADING REQUIREMENTS

All detention facilities will be graded to allow for complete drainage by the low flow outlet of the principal spillway. No permanent standing water will be allowed. Minimum grade is 0.5%.

1306.3 DEPTH LIMITS

The maximum ponding depth for parking lot detention facilities is 12 inches and shall include signage warning the general public as to the use of the parking lot for detention ponding.

1306.4 TRICKLE FLOW AND BASIN DEWATERING

All detention basins shall include provisions for a concrete low flow channel and/or a storm drain to ensure positive dewatering of the basin. Low flow criteria are presented in Section 800.

1306.5 EMBANKMENT PROTECTION

Embankments shall be protected from structural failure from overtopping. Overtopping can be caused by a larger than design inflow or from obstruction of the low flow outlet. Embankment protection may be provided by embankment armoring (i.e. riprap) or by a design overflow section (i.e. emergency spillway). The invert of the emergency spillway shall be set equal to or above the major design storm water surface elevation.

1306.6 MAINTENANCE REQUIREMENTS

All detention facilities will be designed to minimize required maintenance and to allow access by equipment and workers to perform maintenance. Maintenance for facilities on public lands or within dedicated easements will generally be maintained by the local entity. Regional facilities will be maintained by the local entities and/or Washoe County. Facilities on private land will be the responsibility of the owner (See Section 303.10).

1306.7 LOCAL DETENTION BASIN SITING GUIDELINES

Local detention basins should be located as to minimize their impact on the site and to ensure public safety. Basins should not be located adjacent to building because of the potential of saturating foundation materials. Basins should also be placed to minimize detrimental impact on public facilities (e.g. roadway and sidewalk deterioration).

1307 EXAMPLE APPLICATIONS

1307.1 EXAMPLE: DETENTION POND OUTLET SIZING

Problem: Size the principal and emergency spillway for a detention pond given the following information:

- Inflow hydrograph in Table 1301 (A)
- Basin Site characteristics in Table 1301 (B)
- Outflow limitation of 300 cfs (Major Storm)
- Emergency spillway design flow = 1000 cfs
Solution:

Step 1  Size low flow conduit;

\[ Q = C_d A (2gh)^{0.5} \]

300 cfs  = 0.65 A (2gh)^{0.5}

\[
A = 21.8 \text{ ft}^2
\]

Diameter  = 5.3 ft, Use 72" RCP

Step 2  Develop depth-outflow date for low flow conduit as presented in Table 1201 (C).

Step 3  Perform storage routing using HEC-1. The input data listing and resulting outflow summary is presented in Table 1302.

The results show that a volume of 31.4 acre-feet is sufficient to limit the pond outflow to less than 300 cfs (actual outflow = 302 cfs).

Step 4  Size emergency spillway

Assume \( H = 2.0 \text{ ft} \) and \( w = 0.5 \text{ ft} \)

Therefore, a broad crested weir, \( C_d = 2.65 \)

\[ 1,000 \text{ cfs} = 2.65 \times 2.0^{1.5} \]

\[ L = 133 \text{ feet} \]

Use 135 feet

Step 5  The actual water surface elevation for the emergency spillway design flow is then found by repeating the storage routing procedure for the required emergency spillway design hydrograph.

1307.2 EXAMPLE: RATIONAL FORMULA DETENTION METHOD

Problem:  Determine the required detention volume given the following parameters:

Peak flow from Rational method is 29 cfs

Time of concentration is 15.2 minutes

Outflow is limited to an existing flow rate of 13 cfs

Solution:  Step 1  Plot triangular hydrograph as described in Section 1303.1.2 (See Figure 1304).

Step 2  Plot outflow limitation of 10 cfs on falling limb of hydrograph (Point D on Figure 1304).

Step 3  Calculate area under triangle above line A-D (Figure 1304)

\[ V = 15,550 \text{ ft}^3 \]
## Inflow Hydrograph and Basin Characteristics for Example in Section 1308.1

### (A) Inflow Hydrograph

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<th>RUNOFF (CFS)</th>
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### (B) Basin Characteristics

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### (C) 72-inch RCP Discharge Rating

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**WASHOE COUNTY**  
HYDROLOGIC CRITERIA AND DRAINAGE DESIGN MANUAL

**HEC - 1 RUN FOR EXAMPLE IN SECTION 1308.2**

**HEC - 1 INPUT**

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**RUNOFF SUMMARY**
FLOW IN CUBIC FEET PER SECOND  
TIME IN HOURS, AREA IN SQUARE MILES

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Peak Storage = 31 AF., Peak Storage = 110.79
TYPICAL BASIN GEOMETRY

NOTES:
1. FOR LOCAL MINOR DETENTION FACILITIES, THE REQUIRED ONE-FOOT FREEBOARD SHALL BE ABOVE THE 100-YEAR WATER SURFACE ELEVATION.
V-NOTCH WEIR COEFFICIENTS

\[ C_1 = \frac{2.9 \sin \theta}{\sqrt{1 + 2.57 \sin \theta}} \]

where:
- \( C_1 \) is the weir coefficient
- \( \theta \) is the angle of the weir face
- \( H \) is the depth of flow

Graph showing variations of \( C_1 \) with \( H \) for different angles of \( \theta \):
- 20°
- 45°
- 60°
- 90°

Reference:
Brater and King, Handbook of Hydraulics, 1976
OGEE-CRESTED WEIR COEFFICIENTS

VALUES OF COEFFICIENT $Q_l$ vs. VALUES OF $P_{H_d}$

$Q = C_d L H_d^{3/2}$
# SECTION 1400
## EROSION AND SEDIMENTATION

### TABLE OF CONTENTS

<table>
<thead>
<tr>
<th>1401</th>
<th>DEBRIS CONTROL STRUCTURES AND BASINS</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1401.1 - Introduction</td>
<td>1401</td>
</tr>
<tr>
<td></td>
<td>1401.2 - Debris Deflectors</td>
<td>1401</td>
</tr>
<tr>
<td></td>
<td>1401.3 - Debris Racks</td>
<td>1402</td>
</tr>
<tr>
<td></td>
<td>1401.4 - Debris Risers</td>
<td>1402</td>
</tr>
<tr>
<td></td>
<td>1401.5 - Debris Cribs</td>
<td>1402</td>
</tr>
<tr>
<td></td>
<td>1401.6 - Debris Dams and Basins</td>
<td>1403</td>
</tr>
<tr>
<td></td>
<td>1401.7 - Sizing of Control Structures and Basins</td>
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<td>1401.8 - Siting of Control Structures and Basins</td>
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</table>

| 1402 | CONTROL OF EROSION FROM CONSTRUCTION ACTIVITIES               | 1403 |

### LIST OF TABLES

### LIST OF FIGURES

| 1401 | TYPICAL DEBRIS CRIB                                          | 1400 |
SECTION 1400
EROSION AND SEDIMENTATION

1401  DEBRIS CONTROL STRUCTURES AND BASINS

1401.1  INTRODUCTION

Debris transported by storm water can cause severe problems with flood control structures and other public facilities. Debris-related problems include: clogging of channels and culverts, filling of detention ponds, and burial of or physical damage to roadways and other property. Consequently, the need for debris control is an essential consideration in the design of hydraulic structures, particularly culverts and detention basin outlets.

In order to select an appropriate debris-control measure, the debris within a particular basin should be classified. A classification used by the U.S. Department of Transportation (USDOT, 1971) follows:

1. Light floating debris — small limbs or sticks, orchard prunings, twigs and refuse.
2. Medium floating debris — limbs or large sticks.
3. Heavy floating debris — logs or trees.
4. Flowing debris — heterogeneous fluid mass or clay, silt, sand, gravel, rock, refuse, or sticks.
5. Fine detritus — fairly uniform bedload of silt, sand, gravel more or less devoid of floating debris, tending to deposit upon diminution of velocity.
6. Coarse detritus — coarse gravel or rock fragments carried as channel bedload at flood stage.
7. Boulders

Debris can be controlled by three methods: (a) interception near the debris source or above a critical hydraulic structure downstream of the source; (b) deflecting the debris for detention near (usually above) a culvert or inlet; or (c) passing the debris through the channel or inlet structure. Commonly used structures for controlling various types of debris are listed in Table 1401 and described in the following sections.

1401.2  DEBRIS DEFLECTORS

Debris deflectors are used to divert medium and heavy floating debris and large rocks from the culverts (or other inlets) for accumulation in a storage area and subsequent removal after the flood subsides. The storage area must be adequate to retain the anticipated type and quantity of debris during any one storm or between clean-outs.
1401.3 DEBRIS RACKS

Debris racks provide barriers across stream channels to stop debris that is too large to pass through downstream channels or culverts. Debris racks vary greatly in size and in construction material. Height of racks should allow some freeboard above the expected depth of flow in the upstream channel for the design flood. Racks should not be placed in the plane of the culvert entrance, since they induce plugging when thus positioned. Access to the rack is necessary for maintenance. The rack should be placed well upstream from the culvert or improved channel inlet in those situations where a well-defined upstream channel exists. However, they should not be placed so far upstream that debris enters the channel between the rack and the inlet.

1401.4 DEBRIS RISERS

Debris risers generally consist of a vertical culvert pipe and are usually suitable for installations of less than 54-inch diameter. Risers are normally used with detention ponds or debris basins or where a considerable height of embankment is available above a culvert crossing. The riser is particularly effective where debris consists of flowing masses of clay, silt, sand, sticks, or medium floating debris without boulders. Risers are seldom structurally stable under high-velocity flow conditions because of their vulnerability to damage by impact.

1401.5 DEBRIS CRIBS

Debris cribs are particularly adapted to small-size culverts where a sharp change in stream grade or constriction of the channel causes deposition of detritus at the culvert inlet. The crib is usually placed directly over the culvert inlet and in "log cabin" fashion.

Figure 1401 shows the general dimensional details of a typical debris crib. Spacing between bars should be about 6 inches. A crib may be open or covered with horizontal top members spaced equal to the crib members. Debris can almost envelop a crib without completely blocking the flow and plugging the culvert. When an open crib is used as a riser and an accumulation of detritus is expected, provision can be made for increasing the height.

1401.6 DEBRIS DAMS AND BASINS

On channels carrying heavy sediment and debris loads, it is often economically impracticable to provide culverts large enough to carry surges of debris. If the height of an embankment and storage area are not sufficient for a riser or crib, a debris dam and/or basin placed some distance upstream from the culvert may be feasible. These are sometimes used to trap heavy boulders or coarse gravel that would clog culverts.

Detention basins located in the mountain canyon areas can accumulate large deposits of rocky debris, either over the course of several years or after each extremely large load event. Design of detention ponds (Section 1300) in these areas must include provisions for debris (and suspended sediment) deposits and control of floating debris using debris racks and/or risers.

Much of the rock debris will deposit in the upper reaches of detention ponds where high-velocity flood waters first encounter slack, ponded water. If regularly maintained and cleaned of these deposits, detention ponds can effectively serve multiple purposes of attenuation of flood peaks and entrapment of sediment and debris (see Section 1200 for further discussion of detention pond design).
1401.7 SIZING OF CONTROL STRUCTURES AND BASINS

The spacing of bars on trash racks, debris racks, debris deflectors, debris risers and debris cribs is based on the size of the structure to be protected and the anticipated size and gradation of the debris. To minimize the potential for clogging, in no case shall the barrier members be spaced more than two-thirds of the conduit diameter.

The size of debris basins is most dependent on the physical properties of the watershed and the intensity of flood events. Specific sedimentation data have not been developed for the Clark County area, and designs must be based on site specific data from other areas. The U.S. Department of Agriculture reports sedimentation rates for reservoirs nation-wide in a report "Sedimentation Deposition in U.S. Reservoirs: Summary of Data Reported Through 1975" (USDA, 1976). The average annual sedimentation rates reported vary over five orders of magnitude. For this reason, the use of data from other areas is limited.

The major threat to debris basins is from a single rare flood event. The Los Angeles Department of Public Works has published curves for debris production per storm event for the Los Angeles area (LADPW, 1989). These rates vary from approximately 250,000 yd³/square mile to 4200 yd³/square mile. Again the soil types and storm patterns vary considerably between Los Angeles and Washoe County, but the data developed for Los Angeles does illustrate the problem.

1401.8 SITTING OF CONTROL STRUCTURES AND BASINS

Debris control structures which protect other hydraulic structures (e.g. culverts, bridges, channel) are placed based on structure cost, debris production potential and the importance of the structure. Minor culverts whose failure would have a limited impact on downstream structures would require less debris protection than a major lined channel. Generally speaking, debris control structures should be placed at the source of debris.

1402 CONTROL OF EROSION FROM CONSTRUCTION ACTIVITIES

WASHOE COUNTY has adopted Best Management Practices Handbook (BMPH) for control of erosion and sedimentation for construction activities. This BMPH is hereby adopted and made a part of this MANUAL by reference.
<table>
<thead>
<tr>
<th>Debris Classification</th>
<th>Debris Deflector</th>
<th>Debris Rack</th>
<th>Debris Riser</th>
<th>Debris Crib</th>
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</table>
DEBRIS CRIB
(TYPICAL)

(D = Diameter of pipe)

PLAN

CRIB MEMBER
When 'L' is 4' or more use double amount of R.S. shown.

ELEVATION

Intermediate to be placed if, and as directed by the Eng'r.

Filler 1/4" (if intermediate crib member is used)

1" I.P. Driven securely into the ground

Note: I" I.P. & 1/2" I.P. Filler to be paid for at the unit price paid for reinforcing steel

HEIGHT to be determined by the Eng'r.
SECTION 1500
STANDARD FORMS

STANDARD FORM 1 - DRAINAGE REPORT SUBMITTAL CHECKLIST
STANDARD FORM 2 - TIME OF CONCENTRATION
STANDARD FORM 3 - STORM SEWER HYDRAULIC CALCULATIONS
STANDARD FORM 4 - CULVERT DESIGN
# DRAINAGE REPORT SUBMITTAL CHECKLIST

**PREPARED BY:** ____________________________  **DATE:** ____________________________

The drainage report for the development as noted below has been received and found to lack the information needed. This information must be submitted before the report will be accepted for review. Please provide the required information.

**DEVELOPMENT:**

**LOCATION:**

**DATE SUBMITTED:**

**SUBMITTED BY:**

**FIRM**

**CONTACT**

**PHONE**

☐ Conceptual Report  ☐ Technical Report  ☐ Floodplain Study

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**VERSION:** December 2, 1996

WASHOE COUNTY
HYDROLOGIC CRITERIA AND DRAINAGE DESIGN MANUAL

STANDARD FORM 1

WRC ENGINEERING, INC.
<table>
<thead>
<tr>
<th>DESIGN</th>
<th>R</th>
<th>AREA Ac</th>
<th>LENGTH Ft</th>
<th>SLOPE %</th>
<th>$t_i$ Min</th>
<th>LENGTH Ft</th>
<th>SLOPE %</th>
<th>VEL. FPS</th>
<th>$t_f$ Min</th>
<th>$t_c$ Min</th>
<th>TOTAL LENGTH Ft</th>
<th>$t_c = (L/180) + 10$ Min</th>
<th>REMARKS</th>
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</thead>
</table>

$t_i = 1.8 (1.1 - R) L^{1/2} / S^{1/3}$
## Washoe County
### Hydrologic Criteria and Drainage Design Manual

### Storm Sewer Hydraulic Calculations

<table>
<thead>
<tr>
<th>Station</th>
<th>Conduit Data</th>
<th>Flow Data</th>
<th>Energy Loss Data</th>
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<td>From</td>
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<td>D/S (elev)</td>
<td>L/S (elev)</td>
<td>Length (ft)</td>
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Enter Starting EGL and HGL Here:

### Storm Sewer "n" Value

\[ s_n = \frac{I \cdot V}{R^{1.35}} \]

\[ I = \frac{2a}{c^2} \]

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 Revision: December 2, 1996

Reference:

Standard Form 3
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**WRC ENGINEERING, INC.**
### OUTLET CONTROL EQUATIONS

1. \( H_w = H + (h_o - L)S_o \)
2. For \( T_w = D \), \( h_o = \frac{d_e + D}{2} \) or \( T_w \) (whichever is greater)
   \( T_w \geq D \), \( h_o = T_w \)
3. For Box Culvert: \( d_e = 0.315(Q/D)^{2/3} \leq D \)

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<table>
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<th>OUTLET CONTROL</th>
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<td>( H_w )</td>
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REFERENCES


City of Reno, Section 18.06.315 Hillside Developments, Ordinance 3841, August 17, 1989.


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Humphrey, John H., Ph.D., Washoe County Flood Control Master Plan/Meteorological Analysis, April 1994.


NRS 278, Nevada Statutes Related to Regional Planning.

NRS 487, Nevada Statutes Creating the Regional Water Planning and Advisory Board of Washoe County (Regional Water Resources Management Agency/RWRMA).

NRS 543, Nevada Statutes Related to Flood Control Districts that cover the entire county.


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HYDROLOGIC CRITERIA AND DRAINAGE DESIGN MANUAL


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Washoe County & Cities of Reno and Sparks, Washoe County Flood Control Master Plan, Volumes I & II, April 1991.


