CITY OF MARBLE FALLS
DRAINAGE CRITERIA MANUAL

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THE CITY OF
MARBLE
FALLS
T E X A S

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MR. MIKE PILLEY, MAYOR PRO-TEM
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MR. PERRY MALKEMUS, PUBLIC WORKS DIRECTOR

KC

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STATE OF TEXAS
JOHN G. FOWLER
70382
REGISTERED
PROFESSIONAL ENGINEER

K.C. ENGINEERING, INC.
705 North Hwy 281, Suite 103, Marble Falls, TX 78654 (830) 693-5635

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II. INTRODUCTION

Authorization
II.1

Drainage and management of storm water and runoff from existing and newly developed areas requires the application of a sound and consistent set of policies and methodologies for the design and acceptance of proposed storm drain systems for proposed roadways, subdivisions, and commercially developed areas. To that end, the City Council of the City of Marble Falls, Texas has adopted these policies and design criteria, in its capacity as the governing body of the City of Marble Falls, Texas and the area within its Extraterritorial Jurisdiction.

Area Covered by Policies, Design Criteria, and Procedures
II.2

These policies apply:

- To areas within the City of Marble Falls.
- To areas where the City of Marble Falls owns and/or maintains property, right of way, or easements.
- To areas within the Extraterritorial Jurisdiction of the City of Marble Falls.

To any other areas where specifically required by regulations and / or ordinance of the City of Marble Falls.

Scope
II.3

This Manual has been compiled to provide written policies, design criteria, and acceptable methodologies for use in the design of efficient and cost effective drainage systems and storm water facilities within the City of Marble Falls and its ETJ.

Discussions of technical data and methodologies are included in this Manual in order to facilitate completion of new designs and analyses to meet the criteria stated within. However, this Manual is not intended to serve as a textbook or to provide definitive technical guidance on hydraulic and / or hydrologic methods. For such definitive technical guidance, appropriate texts and reports should be consulted.

The solutions described in this Manual are based upon mathematical methods, and are well suited for computer solutions, such as spreadsheets. However, numerous nomograph solutions are included in the references quoted in this Manual, and such nomograph solutions are acceptable, provided that an adequate level of accuracy is maintained.

Limitations
II.4

This Manual is intended to provide a guideline for the design of the most commonly encountered drainage and flood control systems in the Marble Falls area. The Manual was written for users with knowledge and experience in the applications of standard engineering principles and practices of storm water analysis, design, and management.

There will be situations not covered by this Manual which merit variations from the criteria and methodologies set forth within. Other methods of design or exceptions to the criteria may be permitted on a case by case basis. The designer should obtain prior agreement from the City of Marble Falls before beginning any required design or analysis that relies upon means and methods other than those described in this Manual.

Design Responsibility II.5

The requirements and methods included in this Manual are intended to outline the minimum design and analysis effort required for drainage infrastructure within the City of Marble Falls. The designer may choose to apply more detailed methods to obtain a more efficient design, or it may be necessary to apply other more involved methods to produce an appropriate design for specific projects. Additionally, the designer is responsible for expanded studies, such as geotechnical and environmental investigations and any other studies that may be needed as a basis for sound design. In any case, the recommendations and requirements in this Manual do not relieve the designer of any portion of his professional responsibility. **The full responsibility for all designs, plans, and specifications will rest with the design professional who produced them.**

Acknowledgements II.6

The engineering data and methodology presented in this Criteria Manual are not represented as original academic work or theory. The information provided in this Criteria Manual relies heavily on work previously completed by other public entities and jurisdictions, such as: The Federal Highway Administration, The Texas Department of Transportation, The City of Austin, the Harris County Flood Control District, and the Natural Resources Conservation Service (formerly Soil Conservation Service) of the United States Department of Agriculture. To a large extent, the criteria and methodologies in use by those entities have been included in this Manual, with revisions and modifications as appropriate to the local area.

Objective II.7

In order to accommodate the increasingly rapid changes in the area, the City of Marble Falls has elected to develop a formal policy regulating new construction in the area, specifically as related to drainage of storm water and potential changes in drainage patterns and associated effects.

The goal of the City of Marble Falls, as addressed in this Manual, is to provide for and encourage construction of new housing, commercial developments, streets, and parking areas, while controlling the aspects of increased storm water runoff from those proposed projects. The design criteria detailed in this Manual are intended to control potential increases in runoff and possible flooding to the extent that existing development, streets, and facilities are not overwhelmed from the possible increases in storm water runoff and associated flooding potential.

The design of all drainage systems is based upon a compromise between the desired level of flood protection to be provided and the cost of providing such protection. With this compromise in mind, the objective of this Manual is to outline policies and associated methodologies that can be expected to assure adequate drainage of newly developed land without adversely affecting existing drainage patterns or systems, and without increasing the potential for flooding in downstream areas. To that end, the policies and procedures developed and described in this Criteria Manual have been adopted to guide the design of storm drainage systems for proposed streets, roadways, bridges, open channel drainage systems and flood control facilities within the geographic area covered by this Criteria Manual.

Organization of Manual
II.8

This Manual is intended for use by at least three distinct groups:

- Public officials, municipal staff members, and other decision makers who are primarily interested in policy definitions and requirements.
- Developers, planners, and others who need a means of determining how established policy requirements may impact planned development or improvement costs and design requirements.
- Engineers, designers, and technicians who are charged with the responsibility of designing, analyzing, and detailing the proposed infrastructure that is affected by the established drainage policy requirements.

In an effort to present the information in this Manual in a way that is useful to all the groups listed above, the information in this Manual is subdivided as follows:

Division 1 contains a discussion of policy requirements and guidelines, and is intended to provide a broad insight into drainage requirements.

Divisions 2 through 8 include technical design requirements, methods, and analysis along with design requirements for specific infrastructure items. The methods and data are intended to describe the minimum technical requirements that must be met, and are not intended to be exhaustive discussions on any of the items addressed. The specified design requirements rely upon numerous technical references, which should be consulted if additional data or discussion is needed.

A List of References is included following Division 2.

Appendix A contains tables and charts referenced in the discussions.

The information in this Manual has been separated into Divisions for purposes of convenience; however, the data is intended to be complementary and cumulative. That is, information from the various Divisions should not be interpreted to stand alone, but should be applied in conjunction with related information and / or requirements from all Divisions.

DIVISION 1 – POLICY REQUIREMENTS

Summary of Requirements 1.1

For ease of reference, an abbreviated summary of drainage policy requirements follows. A full discussion of policy requirements is contained in the remainder of this section, and should be consulted for additional details.

Flood Plains: All development in 100-year flood plains is regulated by the Flood Plain Administrator for the City of Marble Falls, Texas. For newly developed properties meeting minimum size standards, 100-year flood plains and base flood elevations must be developed using detailed engineering analysis and current survey data.

Peak Flows: Peak flows for the 2, 5, 10, 25, and 100-year events shall not be allowed to increase unless full conveyance is provided downstream to the Colorado River.

Public Easements: Public easements must be dedicated in new development to convey the 100-year flow downstream to the limits of the proposed development.

Erosion Protection: All flows from a site must be returned to a channel or storm drain, or must be returned as sheet flow with adequate erosion protection.

Rainfall Runoff / Hydrology: Peak flows may be calculated by the rational method or by the NRCS curve number method as described in this Manual. Rainfall data shall be NRCS Type II as described in this Manual.

Streets: Streets shall be designed to convey the 2-year flow with a maximum ponded width as follows:

- For Local Streets, maintain one 11-foot lane with water ponded no deeper than 6 inches.
- For Collector Streets, maintain one traffic lane with no ponded water.
- For Arterials, maintain one traffic lane in each direction with no ponded water.

The drainage area for streets shall include at least 150 feet each side of the street right of way, unless such areas drain through independent systems.

No cross flow will be permitted for the 2-year flow. No flow through intersections will be permitted for the 2-year flow.

The 100-year flow must be contained within the street right of way.

Storm Drains: Storm drains shall be designed to convey the 2-year flow with the hydraulic grade line no higher than 6 inches below the gutter line.

Storm drains shall be designed based on a tailwater elevation equal to the 10-

year water surface in the receiving channel.

The terminal (most downstream) section of storm drains shall be sized to convey the 100-year flow.

From drainage cul-de-sacs and downstream to the outfall, storm drains shall be sized to convey the 50-year flow.

Inlets: Inlets shall be placed as needed to reduce the roadway ponded width to the minimums prescribed for streets, with a maximum spacing of 300 feet between inlets.

Inlets at drainage cul-de-sacs shall be sized to accept the 50-year flow. Terminal inlets shall be sized to accept the 100-year flow.

All inlets within public right of way shall be capable of carrying an HS 20 wheel load.

Open Channels: Open channels shall be designed with physically stable side slopes and bottoms that are completely protected from erosion.

The minimum capacity for channels shall be the design flow for cross drainage structures or flow for the required tailwater elevation for storm drains outfalling into the channel.

Cross-Drainage Structures: For local streets, culverts shall convey the 5-year flow and bridges shall convey the 10-year flow.

For collectors and minor arterials, culverts shall convey the 10-year flow and bridges shall convey the 25-year flow.

The design flows shall assume fully developed conditions upstream of the cross-drainage structure.

Culverts shall convey the design flow with the upstream water surface no higher than the lowest point on any travel lane or gutter of the roadway crossed. Bridges shall convey the design flow with the water surface no higher than 1 foot below the lowest point on the low chord of the bridge.

For local streets, the 100-year flow over a bridge or culvert shall produce a water surface no higher than 12 inches above the crown of the roadway. For collectors and arterials, the 100-year flow over a bridge or culvert shall produce a water surface no higher than 6 inches above the crown of the roadway.

Detention: When adequate downstream conveyance is not provided, all flows for the 2, 5, 10, 25, and 100-year events shall be detained as required to reduce peak flows to required levels.

Check Floods: Without regard to the required design flow for channels, roadways, storm drains, cross-drainage structures, and / or detention systems, the behavior of designed drainage systems and components shall be checked for the 100-year flow. No adverse effects are permitted for the 100-year flow from the proposed installation of any development, improvement, or renovation.

100-Year Flood Plain 1.2

Development in Flood Plain: All development in existing flood risk areas (as mapped by the Federal Emergency Management Agency {FEMA} on the current Flood Insurance Rate Maps for Burnet County, Texas), as well as in flood risk areas that must be defined for new development as specified in the current City of Marble Flood Plain ordinance, must meet all current requirements for development in flood plains as interpreted by the Flood Plain Administrator of the City of Marble Falls, Texas.

Any requested revisions to existing mapped flood plains or floodways must be accomplished at the sole expense of the developer through established processes for map revisions. All proposed physical modifications of existing flood hazard areas (through detention, channel construction or improvements, or other means) must be approved in advance by the Flood Plain Administrator of the City of Marble Falls, Texas. Any proposed development that is based on the developer's revisions of existing flood hazard areas must include a Conditional Letter of Map Revision (CLOMR) that has been reviewed and accepted by FEMA before construction can begin. The proposed development and all facilities must be constructed in accordance with the plan submitted for the CLOMR, and upon completion of construction, the developer must obtain from FEMA an approved Letter of Map Revision (LOMR). All engineering data, models, results, and revised mapping data (for the CLOMR and LOMR) must be submitted to the City of Marble Falls Flood Plain Administrator for transmittal to and coordination with FEMA.

Peak Flows / Conveyance / Easements 1.2

Peak Flows: It is impermissible for any development, improvement, renovation or, construction to cause an increase in peak flow rates of runoff from a site for any of the 2, 5, 10, 25, 50, or 100-year events, unless all flows are conveyed completely within a dedicated easement downstream to the Colorado River.

Alteration of Watercourses: It is impermissible for any development, improvement, renovation, or construction to alter an existing watercourse or to discharge storm flows at the downstream boundary of the property in a location different from that in the predevelopment condition.

Conveyance: At the time of development of any property, the owner or developer of the property is responsible for conveyance of all storm water flowing through the property (up to and including the 100-year event) including storm water that is directed to the property by other developed property, or that naturally flows to the property.

The owner or developer of a site or property may elect to provide conveyance capacity by constructing storm drains or channels to carry all or a portion of the required flow. The owner or developer shall retain full responsibility for any flooding or property damages that may occur due to inadequate conveyance of storm water flow across the site or property.

If the upstream property has been developed and detention ponds exist to reduce upstream contributing flows, the actual operational parameters of the upstream ponds must be included in the analysis of flows to be conveyed across the downstream property.

Easements: The owner or developer of a site or property shall dedicate a public easement sufficient to convey the full flow for existing conditions from all sources reaching his property. The minimum easement width shall be that which is required to convey 110% of the 100-year flow for watershed conditions current at the time of development. The easement width shall be sufficient to contain the required flow in an unlined earthen channel with side slopes no steeper than 3 horizontal to 1 vertical with a 10-foot wide maintenance berm on each side of the channel.

Erosion Protection: The owner or developer shall be required to return the accumulated site runoff plus flow through the site or property to an existing downstream channel or storm drain. Otherwise the owner or developer shall return all such flow at the downstream right of way to a sheet flow condition with adequate erosion protection.

Specific Variances: In extreme cases, if it can be shown that full compliance with the above criteria is not possible, the developer or designer may seek a specific variance; however, the granting of such variances shall be at the sole discretion of the City of Marble Falls.

Rainfall Runoff / Hydrology 1.3

General: Drainage design generally consists of two basic steps. First, the actual amount of flow passing through a given system (storm drain pipe, culvert, bridge, channel, etc.) must be calculated (hydrologic analysis). The second step is to calculate the water surface elevation within the drainage system that will result when the calculated flow actually is applied to that system; or alternatively, to calculate the conveyance capacity of the proposed drainage system when the calculated flow must be carried at or below a given critical elevation (hydraulic analysis).

Records of runoff from urban areas are not readily available, and therefore are generally not useable as design data for predictions of storm water runoff. Therefore, engineering design normally relies upon the relation between rainfall and runoff to determine design flows for storm drains and other hydraulic systems.

Method Selection: Numerous methods are available to the designer for calculating rainfall runoff relationships. However, the methods described in this Manual should be followed for projects in the geographic area covered by this Criteria Manual unless prior approval has been obtained from the City of Marble Falls.

Guidelines: The following general guidelines should be followed for best results:

- Compare results from several different methods.
- Use the discharge that appears to best reflect local project conditions. Averaging of results of several methods is not appropriate.
- Provide adequate documentation of the reasons supporting the selection of the results.
- Calculation of peak discharge is normally sufficient for design of roadways, storm drain systems, and bridge and culvert crossings, provided that there are no water control facilities contained within the system.
- If flood control systems, such as water quality facilities, pump systems, reservoirs, or detention ponds can potentially affect the design of the proposed system, then development of a full runoff hydrograph will be required. Design of open channel drainage systems of significant length may also require development of a runoff hydrograph. The analysis and design will also require the inclusion of storage routing calculations for such flood control systems.
- Numerous software packages exist for implementation of the recommended methods. Use of up-to-date computer software packages is highly recommended for all but the simplest drainage projects.
- In all cases, engineering judgment must be applied to select and use the appropriate methods and techniques.

Hydrologic Methods: The hydrologic methods recommended for use in this Criteria Manual are:

- The Rational Method
- NRCS Runoff Curve Number Methods
- Statistical analysis of stream gauge data
- Regional Regression equations

Rational Method: The Rational Method provides estimates of peak runoff rates for small urban and rural watersheds. Within the geographic area addressed by this Manual, use of the Rational Method will be limited to areas of 200 acres or less, within which natural or manmade storage is small. It is best suited to the design of urban storm drain systems, small side ditches, median ditches, and driveway pipes. The Rational Method is the preferred method for design of roadway storm drain systems within the geographic area addressed by this Manual.

Use of the Rational Method requires equations for rainfall intensity as described in later Divisions of this Manual. From isohyetal maps developed by Fredrick, Meyers, and Auciello in 1977 and published by the National Weather Service [1], and using the methodology described by Chow, Maidment, and Mays, Chapter 14 [2], intensity-duration-frequency (i-d-f) constants were developed for use with the Rational Method in the Marble Falls area. These i-d-f constants, along with complete explanations of their derivation are contained in Exhibit A-1, in Appendix A, and are valid for the 1, 2, 5, 10, 25, 50, and 100-year events for storm durations from 5 minutes to 3 hours.

Use of the Rational Method also requires runoff coefficients for various types of land use and percentages of impervious cover. Exhibit A-2 in Appendix A contains values of runoff coefficients taken from the *City of Austin Drainage Criteria Manual* [3].

Use of the values in Exhibits A-1 and A-2 are required for use with the Rational Method in the geographic areas addressed by this Criteria Manual.

NRCS Runoff Curve Number Methods: The Natural Resources Conservation Service (formerly Soil Conservation Service) developed the runoff curve number method as a means of estimating the amount of rainfall appearing as runoff. The Natural Resources Conservation Service (NRCS) publication: *Urban Hydrology for Small Watersheds, TR-55* [4] describes the Runoff Curve Number Method and provides graphical solutions for peak runoff volumes and rates. The methods described are ideally suited for computer based analysis using HEC-HMS [5] or other comparable software. The procedure easily accounts for the effects of urbanization, channel storage, flood control storage, and multiple tributaries. Apply NRCS methods to the design of culverts, bridges, detention ponds, channel modification, and to the analysis of flood control reservoirs. *TR 55* [4] includes a hydrograph development procedure for Manual calculations; however, where hydrograph determination is necessary, computer based methods are recommended.

The NRCS Runoff Curve Number Method becomes inaccurate for runoff values less than 0.5 inch. Additionally, this procedure requires experience and engineering judgment in determining appropriate soil types and curve numbers (*CN*).

Statistical Analysis of Stream Gauge Data: Statistical analysis of stream gauge data provides peak discharge estimates using annual peak stream flow data. The method is particularly useful where long records (in excess of 25 years) of stream gauge data are available at, near, or on the same stream as the structure site.

Regional Regression Equations: Regional Regression Equations provide estimates of peak discharge for watersheds in specific geographic regions.

Comparison of Methods: There are advantages and disadvantages to each of the previously listed hydrologic methods. There are also practical limitations on the use of each method. Some of the limitations include:

- The familiarity and skill of the designer with each method.
- The complexity of the drainage system to be designed or analyzed.
- The availability of appropriate software systems for use with each specific method.
- The specific needs of the designer for the project at hand: for instance, if only peak flows are required, then any of the methods mentioned

previously will provide the required information. On the other hand, if complete hydrograph development is required, the Rational Method, Stream Gauge Analysis, and Regional Regression Equations provide only peak flow data, which must then be manipulated with subjective data and methods to provide volume-of-flow calculations. The NRCS Curve Number method is likely to be more useful for hydrograph development than the other methods listed. In addition, the NRCS method uses data that can be readily entered into modern hydrologic analysis packages, such as HEC-HMS.

- Lack of available data can make use of Stream Gauge Analysis impossible. Stream Gauge data in the local area may be available for major streams only, and will have limited usefulness within smaller watersheds associated with planned development. For Stream Gauge analysis within the local area, it will be the designer's responsibility to obtain the required data, and to document all data, methodologies used, and results to the satisfaction of the City of Marble Falls.
- Regional Regression Equations are primarily developed through the use of Stream Gage Analysis for large streams; therefore, Regional Regression Equations are limited in usefulness similarly to Stream Gage Analysis. Additionally, Regional Regression Equations are normally focused on natural watersheds and their use may not allow sufficient flexibility to properly account for complex patterns of development within a watershed.
- The size and complexity of the watershed can have a significant impact on the choice of the hydrologic method to be used. For instance, the underlying assumptions of the Rational Method are normally not reasonable for watershed sizes of greater than 200 acres, and as the size of the drainage area approaches 200 acres, Rational Method results are more uncertain. Therefore, use of the Rational Method is limited to watersheds of 200 acres or less within the geographic area addressed by this Manual. On the other hand, the original analyses used to develop the Regional Regression Equations were limited to watersheds larger than 1 square mile, with the result that calculated results using the Regional Regression Equations are highly unreliable for watersheds 1 square mile or less in area. The NRCS method has no inherent limitations on maximum or minimum size of watershed.

Required Methods: In most cases, the hydrologic analyses required by this Manual will involve calculations and models of multiple alternatives for each site. Often, at least two analyses are required: one for existing conditions, and another for proposed conditions. The results of the existing analysis and the proposed analysis will be compared directly in order to accurately quantify the results of the proposed development or improvements. In many cases, a third analysis will be required in order to demonstrate that any proposed mitigation measures, such as detention ponds, will have the required effect.

Once the designer has chosen an appropriate hydrologic method, he must use the same method for all analyses that are required to obviate any differences in results that may be inherent between different methods.

For instance, it is completely inappropriate to use the Rational Method to calculate peak flows from an undeveloped site, and then use NRCS methods to calculate peak flows from the same site after development. Comparison of the results from these two analyses would not provide a basis for sound conclusions.

The simplest analysis that will produce the required result will usually be the most desirable. For instance, if only peak flows are required, the methods above can be readily applied through Manual calculations. Additionally, the NRCS methods allow computation of peak flows and simple triangular hydrographs that can be used for detention pond sizing, again using Manual calculations if desired. These simplified methods, using Manual calculations, are acceptable for simple watersheds where no flood routing calculations are required and where there is only one watershed sub area contributing to flow.

For watershed analyses where multiple hydrographs must be combined or where flood routing techniques are required, **computer based analysis will be required**. There are numerous commercial and private software packages that are available for use. For review of any complex hydraulic or hydrologic analysis required by this Manual, the City of Marble Falls will accept the use of the latest versions of HEC-HMS and HEC-RAS [5]. These software packages were written by the US Army Corps of Engineers, Hydrologic Engineering Center, and the software packages, along with full user documentation and technical Manuals, are available on the internet free of charge

Other software packages may be acceptable. The designer should receive approval from the City of Marble Falls for the use of different software before beginning any analysis required by this Manual.

Drainage Area Maps: The size and shape of the watershed or study area must be determined. In some instances, such as street drainage design, this Manual dictates minimum drainage areas to be included in analysis and design. For use of any of the analysis methods discussed in this Manual, a map of all drainage areas that contribute flow to the area under analysis is required. The map(s) must be clearly legible and contain contours at elevation intervals of 2 feet or less. All elevations must be in the same datum, and the vertical datum upon which elevations are based must be clearly described on each sheet of the map(s).

The boundaries of the drainage areas addressed must be plotted and clearly labeled, along with calculations of the areas in acres, or square feet. The boundaries of each drainage area, or each sub area, must be clearly shown and labeled, along with the area for each, surface type, and percentage of impervious cover. The overland slope for each sub area should be shown and labeled, along with the slope of each drainage channel or swale used for drainage calculations. If underground storm drains are included in drainage computations, the type, size, and slope of each underground storm drain must be labeled. For each sub area, the total outflow for each of the storm events analyzed must be shown.

For any proposed projects that may result in a change in existing land slope, impervious cover, area, or drainage conditions, map(s) of existing conditions and map(s) of proposed conditions are required. Both existing and proposed maps must contain the information listed above. Existing and proposed maps may be superimposed on common sheets, provided that the results are clearly legible.

The data used to compile the map(s) may be a combination of field survey, aerial survey, and existing contour maps. The source of all data (field survey, USGS maps, etc.) on the map(s) must be clearly noted. Each map sheet must be signed, sealed, and dated by all Licensed or Registered Professionals (surveyors and / or engineers) responsible for creation of the drainage area map(s).

Rainfall Data: In order to perform any meaningful rainfall runoff analysis, some sort of rainfall data is required that allows a calculation of peak rainfall amounts, as well as a determination of the fraction of the total rainfall that falls during each increment of time. Using previously collected data on typical rainfall events, along with statistical methods, the NRCS has developed hypothetical rainfall distribution tables for use in lieu of actual rainfall events. Since rainfall intensity varies significantly across geographic regions, the NRCS has developed four distinct rainfall distributions: Types I and IA for the Pacific maritime climate, Type III for Gulf of Mexico and Atlantic coastal regions, and Type II for the remainder of the Continental United States. These rainfall distributions were developed for a storm duration of 24 hours. Appendix B of *TR-55* [4] contains a full discussion of NRCS synthetic rainfall distributions and their development. As shown in Appendix B of *TR-55* [4], the City of Marble Falls is located just west of the boundary between the area covered by Type III and Type II rainfall; therefore, Type II rainfall data applies to all geographic areas addressed by this Manual.

Appendix B of *TR-55* [4] also contains maps of the Continental United States with plotted rainfall iso-lines for each of the 2-year, 5-year, 10-year, 25-year, 50-year, and 100-year rainfall events. By plotting the location of Marble Falls on the map and interpolating between the plotted rainfall iso-lines, a maximum precipitation amount for each of the mapped events can be determined. By using the NRCS Type II rainfall distribution and the maximum rainfall for each of the mapped rainfall events, the NRCS Design Storms can be tabulated. Exhibit A-3 in Appendix A contains a tabulation of the NRCS Type II Design Storms for the Marble Falls area.

After significant research and analysis using local rainfall-runoff data, the City of Austin developed 3-hour Design Storms for the Austin area for six separate rainfall events: 2-year, 5-year, 10-year, 25-year, 50-year, and 100-year. Exhibit A-4 in appendix A is a tabulation of cumulative precipitation values (in inches) for the six City of Austin Design Storms.

A comparison of the NRCS Type II rainfall distribution vs. the City of Austin Design Storms indicates that the distributions are significantly similar. Exhibit A-5 in Appendix A is a plot of the NRCS Type II rainfall distribution vs. the City of Austin Design Storm distribution. As can be seen in Exhibit A-5, the rainfall distributions are virtually identical in the center portion of the distributions which account for the rainfall between 20% and 80% of the rainfall totals. The significant differences between the two rainfall distributions are twofold:

- The NRCS distribution covers a 24 hour period, whereas, the City of Austin distribution covers a 3 hour period.
- The total precipitation for the City of Austin Design Storms is approximately 70% to 75% of the total precipitation for NRCS storms.

The City of Austin Design Storms were developed from data gathered from the Austin area, and confirm the validity of the NRCS Type II rainfall distributions for the Marble Falls area. The NRCS Type II rainfall distribution is the basis for a great deal of hydrologic methodology and may be directly applied to NRCS flow calculations and computations without modification. Therefore, for any required hydrologic analysis in the geographic area addressed by this Manual, **the NRCS Type II storms as tabulated in this Manual shall be used**. The total precipitation values for each storm are those corresponding to 24 hours (100% of rainfall) from Exhibit A-3 for each tabulated Design Storm and listed in the top table of Exhibit A-3.

Streets
1.4

General: Streets within urban areas often act as drainage channels, and can, in many instances, convey a substantial amount of runoff into storm drains and outfall channels. Additionally, street right of way is often used to include drainage ditches beside the roadway, or underground storm drain systems. A common design practice is to accumulate and collect all overland drainage from neighboring property into the bordering street. The street drainage system is then sized to convey the accumulated drainage at a water surface elevation that allows a portion of the travel lanes of the roadway to remain open and accessible under extreme conditions to allow for evacuation routes and for access by emergency vehicles.

Proposed new streets shall be designed to meet the criteria contained within this Manual. Storm flows and the associated water surface elevations from proposed development that will drain to existing streets must meet the criteria contained within this Manual; otherwise, detention or other measures will be required to reduce the proposed flows to acceptable levels.

Runoff Calculations: For street drainage systems, storm runoff and design flows should be calculated using the Rational Method as contained within this Manual. The design storm for street drainage systems (ditches, underground storm drains, and hydraulic grade line calculations) shall be the 2-year rainfall event. For calculations involving the ultimate conveyance capacity of the street right of way, the flow to be used is the flow calculated from the 100-year rainfall event.

Drainage Area: Street drainage systems are commonly designed to accept flow from neighboring property. Street drainage systems designed under the criteria addressed by this Manual should be sized to accept the flow from a strip of property a minimum of 150 feet in width measured at right angles to the street right of way, unless it is established that such property currently drains to another outfall through an independent drainage system. If the street drainage system is to be designed to accept flow from a strip of land greater than 150 feet in width, it must be established that the additional drainage area cannot be practically drained through an independent system. For design purposes, all flows shall be calculated as if all land draining to the roadway is fully developed.

Hydraulic Grade Line: In the case of roadway storm drains within the geographic area covered by this Manual, street drainage systems shall be designed to meet the conditions of the 2-year rainfall event. Therefore, the **design storm** for street drainage systems shall be the 2-year event as calculated by the Rational Method.

For the design storm, the hydraulic grade line (potential water surface) of an underground storm drain system or roadside ditch **shall be no higher** than the lowest of:

- In the case of roadways with roadside ditches, an elevation equal to 6 inches below the lowest edge of the lowest travel lane of the roadway

or

- In the case of curb and gutter roadway or roadway with ribbon curb, 6 inches below the elevation of the lowest gutter.

Ponded Width: Streets and associated drainage systems (roadside ditches or underground storm drains) are often designed to function in tandem. For depressed streets with concrete curbs and gutters, a significant portion of the design storm flow can be conveyed in the roadway gutter. As more flow accumulates in the gutter, the width of the flow area, or ponded width, becomes larger and larger and encroaches on a correspondingly larger portion of the available traffic lanes.

In order for a street to remain passable and accessible under conditions of an extreme rainfall event, the street must be designed to include a minimum clear width of roadway that is not inundated under the conditions of the design storm.

Streets with a typical barrier curb may be designed to allow drainage flow to accumulate along the lower edge of the roadway next to the curb. For the 2-year design storm, the maximum depth of ponded flow shall not exceed the curb height, and all other ponded width criteria for specific street classifications must be maintained. **Streets with no curb or with only ribbon curb shall be designed to allow no water to be ponded within the travel lanes under the conditions of the 2-year design storm. Streets with roadside ditches shall be designed such that the maximum water surface elevation in the roadside ditch shall not exceed an elevation of 6 inches below the nearest edge of paved roadway under conditions of the 2-year design storm regardless of whether the roadway has curb and gutter. These requirements apply regardless of the roadway classifications discussed**

below.

Curb Breaks: Breaks in roadway curbs will not be permitted. Storm water flow being removed from the roadway shall be collected in inlets and conveyed to the outfall through underground conduits. For concentrated flows coming to the roadway, open backed inlets shall be used to intercept and transfer the collected off-site flow directly into the underground conduit system.

Street Elevations: The elevation of proposed streets, curbs, and associated drainage systems shall be set in relation to the ground elevation at the street right of way such that the 100-year flow will be conveyed completely within the street right of way, and that the above restrictions on ponded width are met.

Driveways must be sloped upward away from the roadway to the right-of-way line as required in order to ensure containment of the 100-year flow within the roadway right of way.

Roadway Classifications: Municipal roadways can be divided into three common classifications with separate clear width requirements for each, as follows:

- **Local Streets** are streets within neighborhoods that generally terminate at a residence or place of business. These streets generally carry one lane of traffic in each direction. For Local Streets, there must be a minimum of one traffic lane (11 feet in width) with water ponded no deeper than 6 inches.

- **Collector Streets** serve to carry traffic from a series of Local Streets to a larger thoroughfare. Collector Streets generally carry two or more lanes of traffic. For Collector Streets with three lanes or less, there must be at least one traffic lane (11 feet in width) with no ponded water. For Collector Streets with more than three lanes, there must be at least one traffic lane (11 feet in width) in each direction with no ponded water.

- **Arterials** receive traffic from Collector Streets and provide a traffic route through urban areas with minimal access and parking for adjoining business. These roadways generally carry multiple lanes of traffic in each direction. For Arterial Roadways, the criteria for both Local Streets and Collector Streets apply; in addition, there must be no more than one traffic lane in each direction that is inundated.

Other roadway classifications are often used by TxDOT to include: Principal Arterials, Minor Freeways, and Major Freeways. For drainage criteria for these advanced roadway classifications, TxDOT documentation and criteria shall be used.

Street Flow Calculations: The amount of flow that can be tolerated in concrete curb and gutter streets shall be limited to the conditions discussed previously in the **Summary of Requirements**.

To ensure scouring velocities, the gutter line of curb and gutter streets shall have a minimum slope of 0.004 feet per foot.

Roadside ditches shall be designed as required for open channels as described elsewhere in this Division. The minimum design flow for roadside ditches shall be the flow from the 2-year event, with the maximum allowable water surface elevation in the ditch limited as stated in the preceding discussion of the hydraulic grade line.

Street Cross Flow: Whenever storm runoff moves across a traffic lane, an impediment to traffic flow occurs. Cross flow is allowed only in cases of superelevation of a curve or overflow from the higher gutter on a street with cross fall. For Collector and Arterial Roadways for the 2-year storm event, no collected flow will be allowed to cross from the higher elevation to the lower elevation.

Flow Through Intersections: Cross flow at street intersections has a potential for impediment to traffic flow similar to street cross flow. For the 2-year storm event, no flow across intersections will be permitted.

For the 25-year storm event, flow across street intersections is limited. as follows:

- At the intersection of two streets of equal classification (such as Collector and Collector) a maximum flow of three cubic feet per second for each gutter across either or both streets will be permissible.
- At the intersection of two streets of dissimilar classification, flow of three cubic feet per second for each gutter will be permissible across the street of lower classification only. No flow will be permitted in the intersection across the higher classified street.
- For Arterial Streets, no cross flow at intersections will be permissible.

Drainage Cul-de-Sacs: At low points, such as at the bottom of sag vertical curves, carryover flow from inlets has no outlet path along the roadway and must flow over the top of the roadway and right-of-way drainage divides in order to reach the outfall. Such conditions are likely to create deep ponds of water in the roadway which will pose an obstruction to traffic operations. Such conditions are not permitted. For drainage cul-de-sacs in low points, the design flow for inlets placed in the cul-de-sac shall be no less than the 50-year event, while the maximum ponded width requirements remain the same as for other portions of the roadway. If necessary to contain the 100-year flow within the roadway right of way at such drainage cul-de-sacs the inlets shall be sized to intercept the 100-year flow and the underground conduit sized to transmit the 100-year flow to the outfall.

Storm Drains 1.5

Storm drainage flow captured by inlets must be transported through an underground conduit system to an outfall channel or larger storm drain system.

In some cases, short sections of underground conduits will be needed to carry captured flow to roadside ditches that parallel the roadway. In other cases, an underground conduit system must be installed for virtually the full length of the roadway to carry captured flow to one or more natural or man made channels that cross the roadway. In all cases, the underground storm drain system is largely, if not wholly contained within the roadway right of way, and in most instances, the underground conduit is installed beneath the travel lanes of the roadway. For underground installations, such as these, a repair or upgrade of any portion of the system that lies beneath the roadway cannot be changed or repaired without incurring significant costs and causing serious disruptions to traffic operations on the roadway. Therefore, underground storm drains and / or underground conduit systems must be carefully planned and designed with long-term consequences considered.

The required design criteria and methodology included in this Manual is generally taken from The Texas Department of Transportation's (TxDOT) *Hydraulic Design Manual* [6] and the Federal Highway Administration's (FHWA) *Urban Drainage Design Manual* [7].

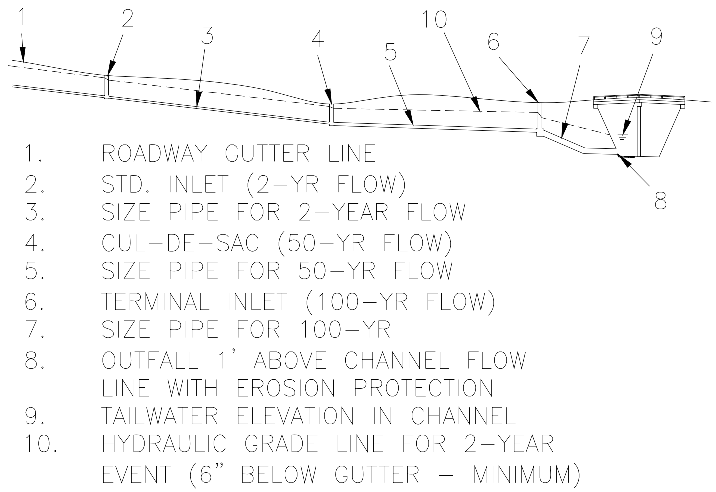
General Design Criteria: The following design requirements should be followed for all underground storm drain systems in public right of way in the geographic area addressed by this Manual:

- Use standard size and type pipe and boxes wherever possible. It is seldom cost effective to specify non-standard sizes or materials.
- The designer is completely responsible for determining and specifying the appropriate structural requirements for all underground drainage facilities and conduit.
- The minimum allowable size of pipe is 18-inch diameter, unless physical requirements dictate otherwise.
- Conduit sizes and slopes should be used to maintain a minimum velocity of 3 feet per second in the conduit to avoid siltation and plugging.
- All drainage conduit should be installed beneath the bottom of the roadway subgrade layer.
- Storm drain systems should be designed using Manning's equation, and should preferably be designed to function as non-pressurized systems.
- Gasketed joints should be used for underground storm drains in all cases.
- The slope of the underground storm drain system will generally follow the slope of the roadway.
- The capacity of underground storm drains shall be determined using the Rational Method with the 2-year Design Storm as for inlets.
- For all underground storm drain systems associated with a roadway system, the elevation of the hydraulic grade line of the system for the design storm flow should be no higher than six inches beneath the gutter or lowest point of any travel lane at any point along the roadway.
- No conduit will be allowed to discharge into a smaller sized conduit

downstream, even though the smaller conduit may have excess capacity because of greater slope. Debris moving through the larger pipe may become lodged at the junction to the smaller pipe, resulting in a blockage of the system.

- At changes in conduit sizes, the elevations of the soffits of the conduits being joined should match. If flow lines are matched instead, the smaller upstream pipe will probably discharge against a head, creating unwanted system losses. Exceptions to this requirement are allowed only if the preferred solution is physically impractical.
- When allowing a proposed conduit system to outfall into an open channel, the conduit should be installed with the flow line of the conduit at an elevation one foot higher than the flow line elevation of the receiving channel.
- The outfall section of conduit should be oriented to discharge at an angle that is 45 degrees or less between the center line of the conduit and the flow direction of the receiving stream.
- If the conduit discharges into a culvert structure, the conduit may be installed to discharge an angle of 90 degrees to the flow direction through the culvert.
- In no instance shall a storm drain conduit be installed to discharge at a direction that is upstream to the flow line of the receiving stream or culvert.
- At drainage cul-de-sacs which develop at sag points in the roadway, the total inlet capacity for inlets in the sag should provide for collection of the 50-year flow without exceeding the allowable roadway ponded width. Underground conduit from this point downstream should be sized to carry the 50-year flow downstream to the outfall. At drainage cul-de-sacs, the inlet and underground system must be sized to allow one open traffic lane under conditions of the 100-year flow.
- The downstream most inlet in the system should be sized to accept the 100-year flow and transmit that flow into the receiving channel through an oversized section of underground conduit, with the 100-year water surface at the roadway completely contained within the roadway right of way. Additionally, the inlet and underground conduit must be sized to maintain one open traffic lane during the 100-year event. This arrangement prevents the 100-year flow from cascading over the bank of the receiving channel, which could possibly cause destructive erosion.

The following illustration shows a typical drainage system in profile, along with significant items noted.



Typical Storm Drain Profile

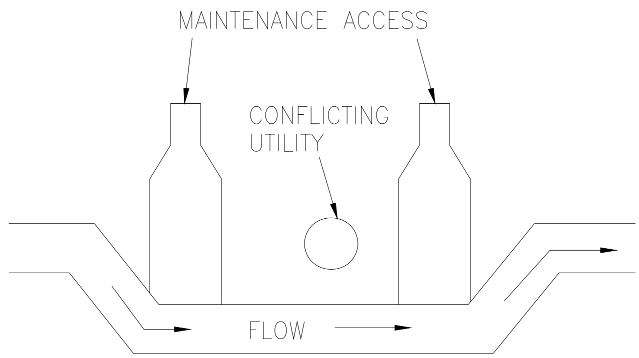
Bends: Bends and / or changes in direction of 45 degrees or less may be accomplished without the use of manholes or junction boxes. However, only one such bend is allowed between manholes.

Manholes: Manholes or combination manholes and inlets should be placed where necessary for clean-out or inspection. Manholes should be placed at changes in direction greater than 45 degrees, junctions of pipe runs where the junction angle is greater than 45 degrees, and at maximum intervals based upon pipe size according to the table in Exhibit A-6.

Bottoms of manholes should be rounded to match the inverts of the pipes attached to the manholes to minimize eddying and resultant head losses. Manholes intended to include multiple facilities or functions should be completely detailed, and all junction losses should be considered in detail.

Junctions: At junctions of conduit runs, right angle intersections should be avoided where junction losses in the system may be significant. Junctions should be accomplished at acute angles of 45 degree angles or less measured between the joined flow paths wherever practical. Such acute angle junctions may be installed without manholes or junction boxes; however, only one such junction with a single connection is allowed between manholes.

Inverted Siphons: It is good practice to locate and avoid all underground facilities, both horizontally and vertically, that may cross the path of a proposed underground storm drain system. In most cases, pressure lines, such as water lines, small gas lines, and pressurized sanitary sewer lines can be relocated vertically to provide clearance for the proposed storm drain, and such relocation is the preferred solution. However, for potential conflicts that cannot be avoided, inverted siphons may be used, provided prior approval has been obtained from the City of Marble Falls. Inverted siphons carry flow under obstructions such as sanitary sewers, water mains, or any other structure that cannot be relocated to clear the storm drain, as illustrated in the figure below.



Inverted Siphon

When no other practical alternative is available and an inverted siphon must be used, the following requirements apply:

- For the design storm, an absolute minimum velocity of 3 feet per second must be used to maintain scouring and avoid siltation and plugging.
- The conduit size through the inverted siphon should be the same size as the approaching conduit. In no case should the conduit size through the siphon be reduced.
- A complete and detailed hydraulic grade line analysis through the siphon must be performed (including bends and junctions), and all siphon losses must be included in the overall system analysis.
- Provide manholes for maintenance access at both ends of the siphon as shown in the illustration above.

Inlets
1.6

General: For roadways with any type of associated drainage system (with the possible exception of open roadside ditch drainage) inlets are an integral part of the completed roadway. For curb and gutter roadways, or depressed roadways, the road or street itself acts as a drainage channel and carries a significant amount of storm flow. When the accumulated storm flow in the roadway creates a water surface width that exceeds the allowable ponding width for the roadway, inlets are installed to remove a portion of the flow from the roadway and transfer the removed flow to the underground storm drain system.

Design Guidelines: The following guidelines should be followed in calculating ponded width and placement of inlets, as published by the FHWA [7].

- For each section of roadway with common characteristics (common longitudinal slope, roadway cross slope, and friction factor), determine the allowable ponded width and calculate the flow capacity of the gutter with the ponded width at the maximum allowable amount.

- Using the hydrologic methods described in this Manual (normally the Rational Method for roadway storm drain design), calculate the amount of flow coming to the roadway from overland drainage areas. Beginning at high points in the roadway, cumulatively total the flows coming to the roadway such that at any given point along the roadway, the flow in the gutter is equal to the sum of all flows entering the gutter upstream of that point, less any flow that has been removed from the gutter through placement of upstream inlets. Flows coming to the roadway from point sources should be added at the specific points where they enter the gutter. Sheet flow should be added to gutter flow continuously; for instance, if 30 cubic feet per second of sheet flow enters the gutter over the length of a city block that is 300 feet long, the flow should be added to the gutter at the rate of 0.1 cubic foot per second of flow for each linear foot of gutter. In this case, the flow in the gutter at mid block would be 15 cubic feet per second ($150\ feet \times 0.1\, cfs/ft = 15\, cfs$).

- When the accumulated flow in the gutter approaches or equals the flow capacity of the gutter at the allowable ponded width, an inlet must be added to remove flow from the gutter and thereby reduce the ponded width. If an inlet cannot be added at the exact point desired, move upstream in the gutter and add the inlet at the closest feasible point. Any flow intercepted by the inlet should be subtracted from the flow in the gutter, and any flow not intercepted by the inlet (carryover or bypass flow) must be included in the gutter flow in continuing calculations downstream.

Inlet Placement: Inlet location may be dictated by physical demands, such as roadway geometry or potential utility conflicts. For any inlet, there must be an available underground path to allow connection of the inlet to the underground storm drain system, and any existing underground facilities that may interfere with gravity flow of the storm drain connection must either be avoided with proper inlet placement, or some provision must be made to relocate the conflicting item.

Within the bounds set by physical requirements, inlets should be installed based upon hydraulic demand. Inlets in the curb and gutter roadways should be placed at strategic locations to allow the inlets to remove storm flow from the roadway whenever the allowable ponding width is reached. Inlets should be placed in sag points or drainage cul-de-sacs to remove storm flow as needed to avoid exceeding allowable ponding depth or ponding width.

Inlets shall be placed immediately upstream of any superelevated roadway section to reduce potential ponding and cross flow within the superelevated section.

Inlets should be installed upstream of intersections to reduce the storm flow crossing the intersected street to an acceptable level. Inlets that are placed in pedestrian crosswalks can significantly interfere with pedestrian traffic; therefore, unless there is absolutely no feasible alternative, **no inlets of any type will be allowed in pedestrian crosswalks**. Additionally, inlets should not be placed closer than 10 feet from the end of the inlet to the nearest end of radius at intersections.

Inlets may be placed in median ditches or roadside ditches as required to intercept roadside flow and transfer the flow to an underground drainage system. All inlets placed within public right of way, particularly street or road right of way, must be designed and constructed to present no traffic hazards. **No inlets with raised structures of any kind will be permitted in street or roadway right of way.**

Generally, roadway **inlets should be spaced no further than 300 feet apart**, with inlets upstream of intersecting roadways, on grade as needed, and at downstream sags or drainage cul-de-sacs. As discussed in the Division on street design, inlets at cul-de-sacs and terminal inlets should be oversized to accept increased flow amounts. It is generally advisable to place additional inlets upstream of a sag point to remove a significant portion of roadway flow above the sag point. This also provides a measure of redundancy in case of clogging of an inlet in a sag point.

Inlet Structural Capacity: All inlet structures within public right of way shall be designed to support HS 20 loading.

Carryover Flow: In order for a drainage system consisting of a combination of street flow and flow through an underground or roadside ditch system to function efficiently, the roadway must be utilized to carry the maximum allowable flow subject to roadway ponding limitations. It then follows that roadway inlets should be designed and placed such that each inlet removes only a portion of the storm flow from the roadway. This makes more efficient use of the available area at each inlet, and also ensures that a significant portion of the storm flow is carried by the roadway gutter, consistent with roadway ponding limitations.

When only a portion of existing storm flow is intercepted by an inlet, the remainder of the flow bypasses the inlet, and must be accounted for in any downstream flow calculations. This flow is termed bypass flow, or carryover flow.

Inlet Types: The following types of inlets are acceptable for use within the geographic area addressed by this Manual:

- Curb Inlets
- Grate Inlets
- Slotted Drains
- Combination Curb and Grate Inlets

Grate inlets, slotted drains, and combination inlets have a tendency to collect debris and clog, thereby increasing the associated flooding potential.

Therefore, the preferred inlet type for streets with concrete curb and gutter (not ribbon curb) is the curb inlet with extensions as needed to develop the required inlet length. Grate inlets, slotted drains, and combination inlets will be acceptable; however, the designer will be required to provide documentation to substantiate the assertion that a curb inlet is not appropriate for the given application.

Grate inlets and slotted drains may be used in swales and medians, provided that an eight-inch thick reinforced concrete apron is installed to extend at least two (2) feet beyond the edge of the inlet or slot in all directions.

If slotted drains are used in roadways or driveways within the public right of way, the entire underground portion of the slotted drain interception system must be encased in low strength non-reinforced concrete.

All grate inlets installed in public roadways must be installed with a grate type and grate orientation that is bicycle safe.

For design purposes, **in all cases, the net capacity of the grated portion of grate inlets or combination inlets shall be taken as one-half of the total capacity.** If the designer relies upon manufacturer's data to determine flows through specific grate types, the design flow shall be reduced to 50% of the manufacturer's total flow, unless the manufacturer's flow data specifically includes a 50% flow reduction. This ensures a 50% safety factor for these inlet types to allow for potential clogging.

The application of a 50% clogging factor for grate inlets provides a conservative design method for selection of grate sizes; however, for design of the associated underground storm drain system, it is advisable to assume that the grate inlet does not clog and that the full amount of the calculated intercepted flow is transmitted into the underground storm drain. Therefore, at grate inlets, the underground conduit system must be designed to carry a flow based upon the grate inlet intercepting full capacity without clogging. To do otherwise, might possibly result in a non conservative design of the underground system.

Open Channels 1.7

General: The most downstream section of a drainage system normally consists of an open channel, either natural or man made. Additionally open channels may often be much more economical than underground storm drain systems, and may comprise a significant portion of local drainage systems, including roadside ditches. The major issues to be resolved during analysis or design are discussed below.

Water Surface Elevation: For a channel that is designed to convey a specific flow, the cross sectional geometry and slope of the channel should be sufficient to convey the design flow at an elevation that allows the flow to be completely contained within the channel right of way or easement. If necessary, the channel section must be improved, the bed slope increased, the right of way or easement expanded, or the flow reduced in order to meet this requirement.

Freeboard: In addition to containing the design flow within the existing right of way or easement, the water surface must also be conveyed at an elevation low enough to account for turbulence, hydraulic jumps, and super elevation at bends. The vertical distance that is required between the water surface and the elevation at the right-of-way or easement line is called freeboard.

For straight reaches of channel flowing in the subcritical mode, the freeboard requirement is equal to the velocity head; alternately stated, the calculated energy grade line for the design flow must be no higher than the elevation at the right-of-way or easement line.

For reaches of channels at bends, the energy grade line must be equal to or lower than the elevation at the right-of-way or easement line by an amount equal to the super elevation at the outside of the bend.

For channels flowing in supercritical mode, the normal depth of the design flow plus any super elevation at bends must be at or lower than the elevation at the right-of-way or easement line.

See Division 4 of this Manual for energy grade line, velocity head, and super elevation calculations.

The channel freeboard requirements are in addition to any other freeboard requirements for specific infrastructure items, such as roadways and bridges. For instance, for a roadside ditch flowing in subcritical mode, and assuming that there is a roadway requirement that the ditch flow be maintained 6 inches below the lowest travel lane, then the energy grade line elevation for the ditch design flow must be 6 inches below the lowest travel lane.

It is worth noting that the freeboard distances discussed above are measured below the lowest right of way point along the channel. However, for bridges, the freeboard required below the bridge is measured from the low chord of the bridge superstructure.

Design Flow: The specific design flow for an open channel will often be determined from the requirements of the storm drain system of which the channel is a part. For instance, if a channel is required to convey flows from the outfall of an underground storm drain system to a larger channel downstream, the minimum design flow for the channel shall be at least the design flow of the required tailwater event for the system outfalling into the channel. For channel sections associated with cross drainage structures, within the roadway right-of-way limits, the channel shall be sized to convey at least the design flow for the associated cross drainage structure (culvert or bridge).

Channel Capacity: In all cases, a channel that is being modified, relocated, or reconstructed must not have the channel conveyance reduced in any way for any storm event, up to and including the 100-year event.

No channel may be altered in any way if such alteration produces potential adverse effects on any portion of the watershed outside the channel right of way or easements. No channel may be altered in any way that will adversely affect any mapped floodplain or floodway.

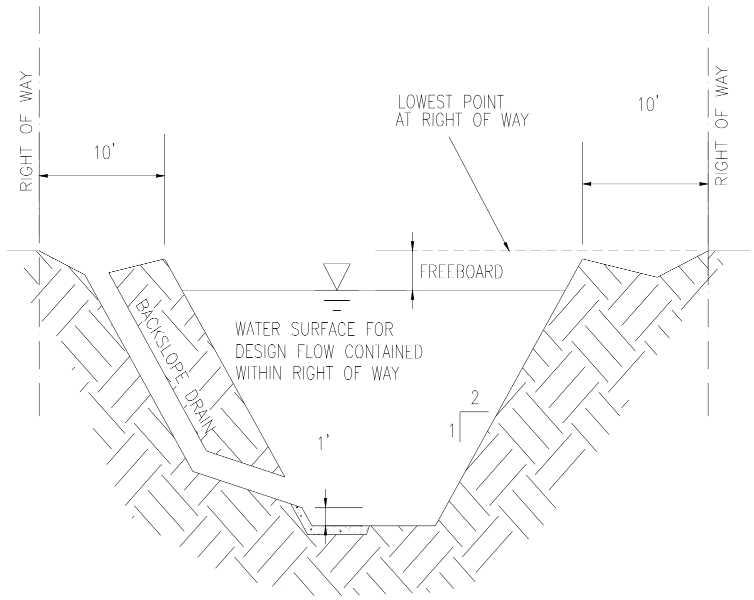
Flow Velocity: The average flow velocity in the channel must be computed for each of the 2, 5, 10, 25, 50, and 100-year events. If the average flow velocity for any of these events exceeds the erosive velocity of the surface soil in the channel banks or bed, then appropriate erosion countermeasures, such as concrete lining, gabions, concrete riprap, or synthetic liners must be installed. Additional vegetation with robust root systems may also be applied. Alternatively, the channel may be redesigned to reduce flow velocities, the design flow may be reduced through the use of upstream detention facilities, or any combination of these.

Slope Stability: If the side slopes of the channel are allowed to collapse, the collapsed portion of the slope will impede conveyance of the required flow and likely cause additional erosion or flooding. Additionally, collapsing slopes may undermine or reduce foundation support for structures constructed in proximity to the channel. Therefore, the side slopes of the channel must be stable against collapse or sloughing. A geotechnical investigation may be required to determine and / or verify that the proposed channel slopes are stable under conditions of the flows from each of the 2, 5, 10, 25, 50, and 100-year events. The channel side slopes must also be analyzed and proven to be stable under conditions of rapid drawdown, that is, during periods immediately following flooding when the side slopes are completely saturated and the water surface in the channel has dropped significantly. Under these conditions, the hydraulic pore pressure in the saturated side slopes may be sufficient to induce instability in slopes that are completely stable under dry conditions.

For open channels deeper than six feet, backslope drains or collector systems must be provided to prevent overland flow from eroding the channel banks. Alternatively, the channel banks may be completely protected from the effects of such potential erosion, making the use of backslope drains unnecessary.

Concrete lining does not normally improve the stability of side slopes; therefore, even with concrete lining, the channel side slopes must be stable based upon geotechnical engineering principles.

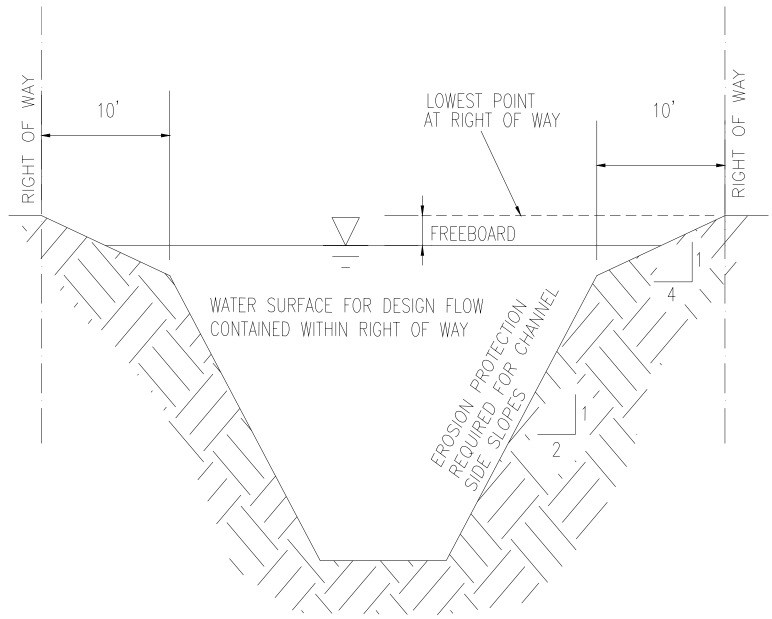
The following illustration is a typical cross section of a channel with a backslope drain installed. As shown in the illustration below, the outfall end of the backslope drain must be installed 1 foot above the channel flow line, with appropriate erosion protection beneath the outfall. The end of the outfall pipe must be cut to match the side slope of the channel.



Channel Section with Backslope Drain

Maintenance Requirements: Channels with unprotected side slopes require periodic maintenance to repair erosion and slope failures, clear brush and undergrowth, and mow excess grass and undergrowth in order to sustain the design capacity of the channel. Unlined channels should be constructed with side slopes no steeper than 2 horizontal to 1 vertical to allow access by mowers and maintenance machinery. Poor soil conditions may require flatter slopes for maintenance access.

Provided that the channel side slopes include erosion protection from overland flow that enters the channel over the bank, the maintenance berms may be sloped to drain to the channel at no steeper than 4 horizontal to 1 vertical as shown in the illustration below:



Typical Channel Section with Lined Slopes

Unlined channels must be designed to include a 10-foot wide maintenance berm parallel to the top of bank on each side of the channel to allow regular maintenance access. Channels with side slopes that are protected by reinforced concrete lining may be exempt from minimum slope requirements for the side slopes, provided that a variance has been provided by the City of Marble Falls.

For proposed channels, the minimum right of way or easement width shall be the width required to include at least the minimum width of maintenance berm on each side of the channel.

Safety Fencing: Channels with very steep side slopes may represent a safety hazard, and the designer is fully responsible for providing adequate protection, and ingress and egress facilities (such as stairways and steps), if required.

For facilities that are to be privately owned and maintained, the owner is completely responsible for all safety issues throughout the life of the facility.

For all facilities that are to be taken over by the City of Marble Falls, adequate safety fencing will be required, as well as adequate ingress and egress facilities. Dual use facilities without fencing will be reviewed and accepted by the City of Marble Falls on a case by case basis.

Water Surface Profiles: The flows used to calculate water surface elevations in open channels must be developed by a complete hydrologic analysis. For depth calculations, simple normal depth calculations will suffice for determination of channel capacity for very short sections of channel. Typically, these short design sections are at roadway crossings and extend from right of way to right of way across the roadway. In any case, for channel lengths over 200 feet, the design for a channel that is to convey ultimate flows for the 10-year event must be based upon a hydrologic analysis including all the contributing drainage areas and must include a water surface profile based upon a complete backwater analysis. The backwater analysis must begin downstream at the nearest control section and extend to the upstream limits of the project (but not drainage areas upstream of the project).

Roadside drainage ditches shall also be designed and analyzed as open channels, with the exception that the design flows for the roadside ditches shall be determined as specified in the Division of this Manual on street drainage. Using the appropriate design flows, water surface profiles for roadside ditches should be computed from a backwater analysis and must include the hydraulic effect of all culverts or structures within the ditch. Water surface profiles for roadside ditches should be plotted in profile on the associated roadway design plans to allow comparison of elevations of water surfaces in relation to the roadway structure.

The required hydrologic and hydraulic analysis should be performed with state of the art computer packages, such as HEC-HMS and HEC-RAS [5]; however, other software packages may be acceptable. The designer should receive approval for the use of different software from the City of Marble Falls before beginning any analysis required by this Manual.

Cross-Drainage Structures 1.8

Design Frequency: Cross drainage structures are bridges, culverts, and low-water crossings that allow flow from a stream to cross the roadway. The hydraulic capacity of the structure determines how much of the stream flow can be carried beneath the roadway without overtopping and interfering with traffic operations. Design of appropriately sized cross drainage structures involves accepting compromises between the cost of the cross drainage structure versus the cost of interference with traffic operations on the affected roadway. As discussed under the Division on streets, municipal roadways can be classified as Local Streets, Collector Streets, or Arterials, with different required levels of service for each classification. It naturally follows that roadways with greater required levels of service should be subject to overtopping from stream crossings less frequently than roadways with lesser required levels of service.

The majority of roadways in the geographic area covered by this Manual consist of Local Streets and Collectors, with a few minor Arterials included. The cross drainage structures associated with those roads will consist of culverts, low-flow crossings, and small bridges, which are addressed by the criteria specified in this Division of this Manual. Design criteria for roadways with greater classifications, or for major bridges and river crossings, should be developed on a case by case basis through discussions with the City of Marble Falls.

Current criteria published by the Texas Department of Transportation suggests that for local roads and streets, a 2-year, 5-year, or 10-year design frequency is appropriate for culverts and small bridges. However, street design criteria specified in this Manual requires that gutters and storm drain systems be designed to allow roadways to be fully functional (although at possibly lower speeds and lower capacities) for the 2-year event. Under heavy flooding conditions (greater than the 2-year event) roadways that have been designed as specified for the 2-year event will have at least one lane that is not flooded to a depth greater than six inches; therefore, these roadways will continue to be marginally passable, either as routes for emergency vehicles or as escape routes. Under these conditions, it would be completely inappropriate to allow such an escape route to be blocked by overtopping flow from an undersized culvert or bridge.

For the geographic area addressed by this Manual, the following criteria should be applied:

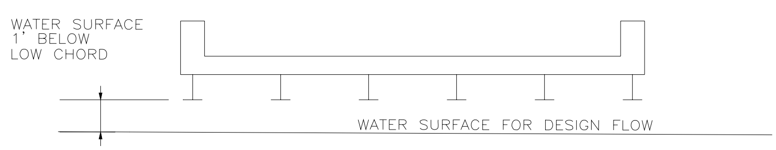
- **For Local streets, culverts should be designed to convey the 5 year flow. Small bridges should be designed to convey the 10 year flow.**

- **For Collectors and Minor Arterials, culverts should be designed to convey the 10 year flow, while small bridges should be designed to pass the 25 year flow.**

- **These design flows must include all upstream contributing drainage areas, with the calculated value of the associated flow rates based on conditions upstream of the culvert or bridge that are expected to apply within the lifetime of the bridge.**

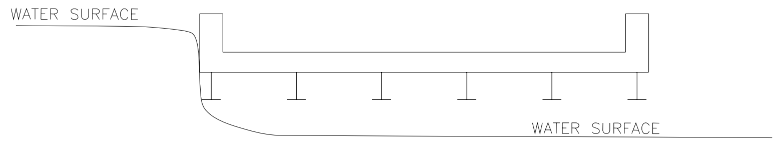
For all of the above structures and conditions, the required flow must be conveyed through the structure with the upstream water surface being no higher than the lowest point of any travel lane or gutter of the crossed roadway. Additionally, bridges should be designed to convey the design flow while keeping the low chord of the bridge superstructure a minimum of one foot above the water surface elevation for the design flow.

The minimum freeboard requirement for bridges is shown in the illustration below:



One-Foot Freeboard Below Low Chord

When the water surface rises to touch the low chord of the bridge structure, the flow through the bridge opening begins to function as orifice flow, and the upstream water surface rises rapidly to the point of overtopping of the roadway. This condition is shown graphically below, and should be avoided for conditions of the design flow.



Superstructure Partially Inundated

When a bridge superstructure becomes partially inundated as shown above, the combination of forces of uplift, buoyancy, and horizontal force must be countered by structural means. All bridges shall be designed to include tie downs or anchorages for the superstructure to ensure that no movement or displacement of the superstructure takes place under conditions of inundation.

Numerous culverts are installed in roadside ditches to provide driveway crossings for neighboring property owners. The cumulative hydraulic effect of such installations can greatly reduce the conveyance capacity of drainage ditches. Therefore, for design purposes, driveway culverts should designed, analyzed, and installed exactly as roadway cross drainage structures.

The primary design constraint for culverts is the elevation of the water surface upstream of the inlet end of the culvert. It is not permissible for the backwater effects of a proposed culvert to cause upstream water surface elevations that flood neighboring structures. At the same time, the water backed upstream of the culvert may have an adverse effect on the roadway itself by saturating base or subgrade material beneath the roadway. In some events the water surface upstream of the culvert rises to the point where water will flow over (or overtop) the roadway. The roadway profile and roadway structure must be designed to accommodate such overtopping without damage.

A proposed culvert structure will be deemed hydraulically sufficient if the elevation of the water surface upstream of the culvert is at or below the criteria discussed in earlier Divisions of this Manual.

For all culvert analyses, the water surface elevation at the downstream end of the culvert must be based upon proposed channel configuration and the flow from the required design storm.

The minimum acceptable pipe diameter or box depth for installation within the public right of way shall be 18 inches for driveway culverts, and 24 inches for pipes crossing a public street. Additionally, when a culvert installation consists of a single circular conduit of 48 inch diameter or less installed in an earthen ditch or channel, the drainage conduit shall be installed with the invert at an elevation 6 inches below the nominal flow line of the ditch or channel. For such a circular culvert installation, the conveyance capacity calculations must allow for siltation of the bottom 6 inches of the conduit. If sloped safety ends and / or concrete lining is installed immediately upstream and downstream of the conduit, no extra depth of installation is required, and no conveyance reduction for siltation is required. Rectangular conduits shall be installed with the invert matching the ditch or channel flow line.

The above criteria are based upon roadway operational requirements. However, for economic reasons, it is important to design cross drainage structures to pass the required flows **repeatedly**, meaning that the structure and associated sections of roadway must include protection from scour, erosion, and debris load that may be expected for each of the design flows. It is important to make an estimate of the amount of debris that can be expected to accumulate on the upstream side of the cross drainage structure. For watersheds in which the portion upstream of the cross drainage structure is largely undeveloped with active streams, the expected debris load may be extremely heavy, while if the area upstream of the structure is fully developed and well maintained, there may be little or no debris load expected.

Debris that has accumulated on the upstream edge of a structure (bridge or culvert) significantly reduces the hydraulic capacity of the structure. Existing debris already collected on the structure tends to collect additional debris at an accelerating rate, with the result that overtopping of the structure can occur rapidly. Therefore, cross drainage structures should be designed conservatively to allow for at least minimal debris accumulation.

Culverts are commonly designed to back a significant amount of water on the upstream side of the culvert and take advantage of the additional head that is developed to move water through the culvert at an accelerated rate. To allow for possible debris accumulation as previously discussed, a culvert should be designed to pass the design flow with a headwater elevation that is at least six inches below the lowest point on the travel lanes or gutter line of the roadway. At flows above the design flow, culverts become low-water crossings, sometimes with significant flow crossing the roadway. Such a low-water crossing should include erosion protection measures to guard the roadway and culvert structures from potential scour and destructive erosion during overtopping.

Backwater Effects: The elevated water surface immediately upstream of obstructions in flowing streams is generally referred to as backwater. The addition of any cross drainage structure to an existing channel may result in an impediment to flow, with a resulting increase in water surface elevations upstream of the structure. For any proposed cross drainage structure, the new structure shall be designed to produce no adverse effects on the watershed or on any adjacent property. This requirement includes, but is not limited to the following restrictions:

- **When a new structure replaces an existing structure, the water surface upstream of the new structure shall not be higher than the water surface associated with the existing structure unless all backwater effects are contained within the drainage right of way or easement.**

- **When a proposed structure is to be placed where none existed previously, the water surface upstream of the structure shall be no higher than the water surface that exists without the structure unless all backwater effects are contained within the drainage**

right of way or easement.

Exceptions to the above restrictions may be permitted if all the areas of increased water surface elevations are fully contained in the existing channel or existing drainage easement.

In order to comply with the above restrictions, several options may be pursued. Those options include:

- Improving or expanding the channel such that the improved channel capacity is sufficient to accept the backwater effects without an increase in water surface elevation.
- Purchasing additional right of way or easements in the upstream area to include the areas of water surface increases.
- Adding detention upstream of the structure to reduce the flows through the structure as needed.

Coordination with and acceptance from the City of Marble Falls should be obtained before beginning any design that relies upon the above listed exceptions or options.

In no case, will installation of a cross drainage structure be acceptable if such installation creates any effect that is contrary to existing FEMA or flood insurance requirements.

A complete hydraulic analysis of each proposed cross drainage structure will be required to confirm compliance with the restrictions listed in this Division for each of the 2, 5, 10, 25, 50, and 100-year events. Unless detention or storm water storage is an integral portion of the design, the required analysis can be based only on peak flows that are generated by methods discussed elsewhere in this Manual.

Roadway Profile: The roadway profile across the proposed cross drainage structure should be evaluated in terms of potential traffic safety hazards under flooding conditions. Most cross drainage structures are susceptible to overtopping in some circumstances. Many culvert structures are also intended to function as low-water crossings, with traffic operations continuing as a portion of the channel flow crosses the roadway. For relatively shallow crossings (less than 4 feet from roadway to channel bottom), the low-water crossing profile is acceptable; however, for deeper channels the possibility of a vehicle being swept off the roadway by overtopping flow represents a significant safety hazard, and should be avoided. In almost all instances, the channel depth below bridges is such that a sag roadway profile across the bridge should not be used if there is any feasible alternative. The safest alternative for a bridge crossing is a crest profile. In this configuration, the portion of the roadway that is actually on the bridge is above the roadway elevation at the approaches. When overtopped, the approaches are flooded before the bridge deck is submerged and tend to block traffic from actually reaching a point on the bridge where vehicles could be swept off the deck into a relatively deep channel.

Roadway Protection: In most cases, when the design flow for a cross-drainage structure is exceeded, the roadway is likely to be overtopped. The designer must include adequate protection measures for the roadway and associated infrastructure for all overtopping events, such that the roadway will be protected from erosion and scour.

Overtopping: For bridges and culverts in local streets the flow from the 100-year event shall not produce a water surface elevation at the roadway that is more than 12 inches above the crown of the roadway.

For bridges and culverts in collector streets or arterials, the flow from the 100-yeasr event shall not produce a water surface elevation at the roadway that is more than 6 inches above the crown of the roadway.

Traffic Safety: There are significant traffic safety issues associated with the installation of bridges and culverts. Numerous end treatment configurations can be used for culverts, along with combinations of traffic and bridge rails to prevent errant vehicles from leaving the roadway in the vicinity of the cross drainage structure. Traffic safety issues are not addressed in this Manual, but must be addressed for each structure. For the purposes of the guidelines established in this Manual, any traffic safety features must be properly accounted for in the hydraulic and scour analyses that are required. For instance, culvert entrance coefficients may depend upon the type of headwall or end treatment used, while flow over a roadway can be significantly affected by the type of traffic or bridge rail used.

Detention 1.9

During any given rainfall event, a portion of the precipitation falling on the ground surface soaks into the ground (infiltration) and a portion of the rainfall runs off the surface to the receiving stream or storm drain system. During the urbanization (or development) process, large portions of vegetated land are covered by building roofs, parking lots, and / or streets. These manmade structures are designed to be waterproof or impervious to infiltration. When rainfall impacts impervious areas, a relatively small portion of the rainfall infiltrates into the surface, and a correspondingly large portion of the rainfall runs off the surface to the receiving storm drain or stream. Additionally, the portion of the flow that runs off impervious areas moves at a higher velocity than when flowing over partially pervious areas. The result of combining increased runoff volume with increased runoff velocity is an outflow to the receiving storm drain or channel that has a much higher peak flow which takes place much more quickly than under undeveloped conditions. Such increased outflows can quickly overload existing drainage facilities and greatly increase the possibility of flooding in areas downstream of the newly developed areas.

Proposed new development and any improvement of existing facilities in the geographic area covered by this Manual shall be designed and constructed to ensure that **the proposed development or improvement will have no adverse impact on the existing watershed(s) or adjoining properties.** Adverse impact includes, but is not limited to: increases in water surface elevation compared to existing conditions, redirection of existing drainage patterns outside the right of way or easement lines of the developed property, obstruction of a defined floodway, placement of fill in a defined flood plain, obstruction of flow resulting in increased backwater effects, or any increase in flooding potential. Adverse impacts can generally be avoided by adhering to one of the following restrictions:

- The runoff from the proposed site shall flow into an existing storm drain or channel that has sufficient capacity to carry the total flow (including proposed increase) completely within the existing drainage right of way or easement. The total flows must be conveyed downstream to the Colorado River or to the nearest regional detention system, if available, with no increases in water surface elevation outside the existing drainage system right of way or easement.

or

- The proposed development shall include the construction of additional channel or storm drain conveyance sufficient to convey the total flows downstream to the Colorado River or nearest regional detention system, if available, with no increases in water surface elevation outside the existing drainage right of way or easement.

or

- The runoff rate from the developed site must not exceed the runoff rate from the existing site. On-site detention system(s) may be required to comply with this requirement.

These requirements must be independently met under conditions for each of the 2-year, 5-year, 10-year, 25-year, 50-year, and 100-year rainfall events. These rainfall events are defined in later Divisions of this Manual.

The above requirements may be met through combinations of the use of existing drainage facilities, storm drain and channel improvements, or on-site detention facilities. However, if the developer relies upon the use of regional detention systems by conveying increased flows to the regional systems for mitigation (if such systems are available), **payment of the appropriate usage fees as established by the City of Marble Falls will be required**.

Any project or development must include internal drainage systems sufficient for the requirements of the project itself. However, channels, underground storm drain systems, and cross drainage structures that accept flow from areas upstream of a plat, subdivision, or project must be capable of conveying the design flows specified in this Manual within the channel right of way or easement. The design flows must be determined by assuming that the contributing upstream drainage areas have been fully developed. It will be acceptable to delay actual construction of the drainage systems until such time that the upstream development actually takes place, with the upstream developer bearing the actual construction cost to upgrade the channel to the required capacity. However, right of way and easements must be established and maintained in the downstream areas.

Drainage into existing Texas Department of Transportation (TxDOT) facilities will require separate coordination and approval from TxDOT, as well as approval from the City of Marble Falls as described in other Divisions of this Manual.

Erosion control, environmental and water quality requirements, along with limits on the total amount of impervious cover are addressed in separate documentation and are not included in this Manual.

Check Floods 1.10

Regardless of the design flow that is used to size any given drainage facility, the performance of the proposed facility must be analyzed and checked under conditions of the 2, 5, 10, 25, 50, and 100-year events independently. For all flows listed above, the proposed drainage facility must be designed and constructed to produce no potential adverse effects to the watershed outside the existing drainage right of way or easement.

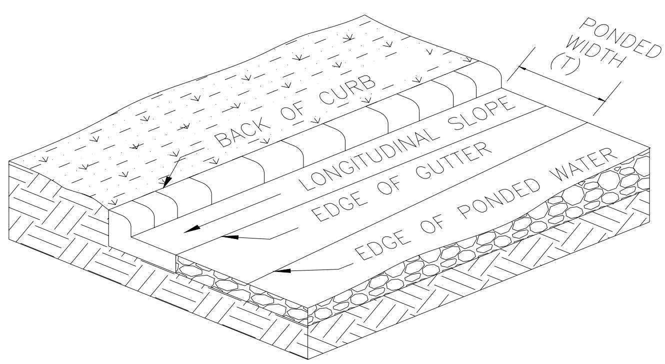
Erosion, scour, and backwater effects must be included in the analysis and design of all proposed drainage structures for all the flows listed above without regard to the flow required to size the structure. Additionally, the structural aspects of all drainage structures must be adequately addressed by the proposed design for all the flows listed above.

Ponded Width 2.1

For roadways with raised curb, the amount of storm flow conveyed in the gutter can constitute a significant portion of the design flow that must be conveyed as a part of the overall roadway drainage system. As the amount of water carried in the gutter increases, the depth of flow increases, and so does the width of the ponded area. In order to limit and control the potential traffic obstructions and hazards that may develop because of ponded water on the roadway, inlets are placed at strategic locations to transfer large quantities of storm flow from the gutter into the underground storm drain system or outfall.

Conventional gutters begin at the inside base of the curb and usually extend from the curb face toward the roadway centerline a distance of 1 to 3 feet. Gutters can have uniform, composite, or curved sections. Uniform gutter sections have a cross-slope which is equal to the cross-slope of the shoulder or travel lane adjacent to the gutter. Gutters having composite sections are depressed in relation to the adjacent pavement slope. That is, the paved gutter has a cross-slope which is steeper than that of the adjacent pavement. Curved gutter sections are sometimes found along older city streets or highways with curved pavement sections.

The primary factors affecting the capacity of flow in the gutter and the associated ponded width are discussed below. See the Curb and Gutter Layout, below, for reference.



Curb and Gutter Layout

Longitudinal slope of the gutter: The following general guidelines should be followed during design:

- A minimum longitudinal gradient is more important for a curbed pavement than for an uncurbed pavement since the water is constrained by the curb. However, flat gradients on uncurbed pavements can lead to unacceptable ponded widths if vegetation is allowed to build up along the pavement edge.

- Desirable gutter grades should not be less than 0.5 percent for curbed pavements with an absolute minimum of 0.4 percent. Minimum grades can be maintained in very flat terrain by use of a rolling profile, or by warping the cross slope to achieve rolling gutter profiles.

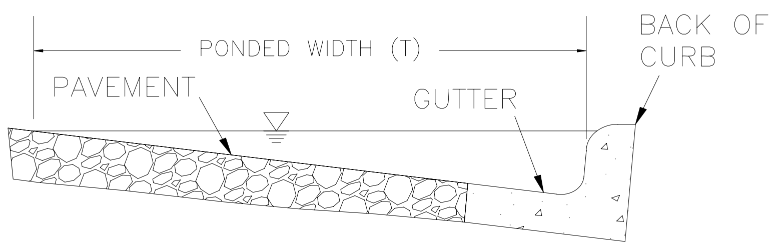
- To provide adequate drainage in sag vertical curves, a minimum slope of 0.3 percent should be maintained within 50 feet of the low point of the curve. This is accomplished where the length of the curve in feet divided by the algebraic difference in grades in percent (K) is equal to or less than 167. This is represented as:

$$K = \frac{L}{G_2 - G_1}$$

Equation 2.1
[7, pg. 4-5]

Where:
K = vertical curve constant
L = length of vertical curve measured horizontally (ft)
G1, G2 = grade 1 and grade 2 in percent

Cross slope of the roadway: Conventional curb and gutter sections usually have a triangular shape with the curb forming the near-vertical leg of the triangle. Conventional gutters may have a straight cross slope, a composite cross slope where the gutter slope varies from the pavement cross slope, or a parabolic section. Shallow swale gutters typically have V-shaped or circular sections and are often used in paved median areas on roadways with inverted crowns. As can be seen from the Curb and Gutter Section, below, for a flatter roadway cross slope, the ponded width (T) will be correspondingly greater for a given water depth.



Curb and Gutter Section

The roadway cross slope that is chosen for use is typically a compromise between two requirements:

- The need to have the roadway shed water as rapidly as possible.

and

- The need to provide a traffic-friendly driving surface.

A steeper cross slope will drain water more rapidly and reduce the potential for traffic interference and obstruction, while a flatter cross slope will have less effect on driver effort and friction demand for vehicle stability. Acceptable roadway cross slopes vary from approximately 0.015 feet per foot of roadway width to 0.05 feet per foot of roadway width. Typically, the flatter slopes are used in the interior lanes of multi-lane roadways, while the steeper slopes are appropriate for outside lanes. Shoulders are typically sloped from 0.02 feet per foot to 0.06 feet per foot of width, and should always be sloped to drain away from the pavement.

For divided roadways, the inside lanes can be sloped to drain to the median, if appropriate; however, the median area should never be drained across travel lanes.

The number and length of flat pavement sections in cross slope transition areas should be minimized. Cross slopes in sag vertical curves, crest vertical curves, and in sections of flat longitudinal grades should be increased to avoid the possibility of flat spots (bird baths) that are poorly drained.

Hydraulic friction factor of the gutter material: A variation of Manning's equation is normally used to calculate the capacity of gutter sections. Because of the relatively shallow flow in gutters, values used for Manning's *n* may differ slightly from those used in calculations for open channel flow or sheet flow. Exhibit A-7.a in Appendix A contains a list of appropriate values of Manning's *n* for gutter flow calculations.

Gutter Flow Calculations: Engineering calculations are necessary in order to establish the ponded width (spread of water) on the shoulder, parking lane, or pavement section. A modification of Manning's equation can be used for computing flow in triangular channels. The modification is necessary because the hydraulic radius in the equation does not adequately describe the gutter cross section, particularly where the top width of the water surface may be more than 40 times the depth at the curb. The resulting equations are:

$$Q = \frac{0.56}{n} \times S_x^{\,1.67} \times S^{0.5} \times T^{2.67}$$

Equation 2.2
[7, pg. 4-9]

or solving for ponded width (*T*):

$$T = \left\{ \frac{Q \times n}{0.56 \times S_x^{\,1.67} \times S^{0.5}} \right\}^{0.375}$$

Equation 2.3
[7, pg. 4-9]

Where:

n = Manning's n for gutter flow
Q = flow rate, cubic feet per second
T = width of flow (ponded width), feet
S_x *= roadway cross slope, feet per foot*
S = longitudinal slope of gutter, feet per foot

The equations above neglect the resistance of the curb face since this resistance is negligible in comparison to the resistance produced by the roadway and gutter surfaces.

Ponded width, *T*, and flow depth at the curb are often used as criteria for spacing pavement drainage inlets. Depth at the curb can be determined with the following relationship:

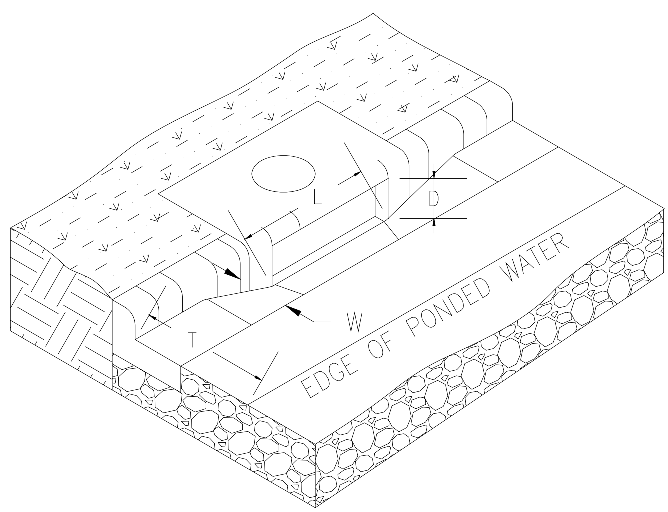
$$d = T \times S_x$$

Equation 2.4
[7, pg. 4-10]

Where:
d = depth of flow, feet

Inlets 2.2

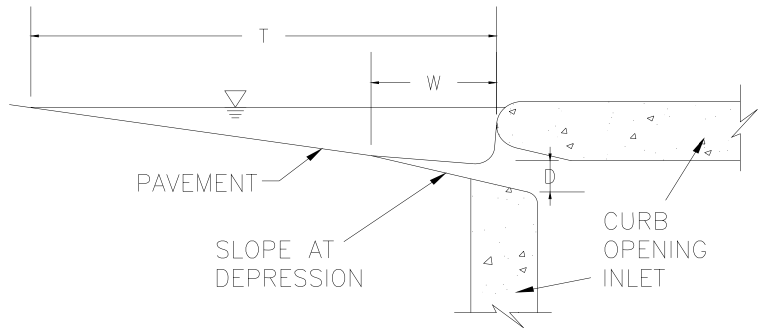
Curb Inlets on grade: The Curb Inlet Plan below illustrates the layout of a typical curb inlet.



Curb Inlet Plan

The design of curb inlets on grade involves determination of lengths required for total flow interception, subjective decisions about actual lengths to be provided, and determination of any resulting carryover rates.

For some inlets, carryover flow may not be acceptable; in instances where carryover flow can be accepted, there must be a convenient location that can accept the bypass flow. For the following discussion, refer to the Curb Inlet Section below.



Curb Inlet Section

Use the following procedure to design curb inlets on-grade.

- Compute depth of flow and ponded width (*T*) in the gutter section at the inlet, using the procedure outlined in the discussion of ponded widths.
- Determine the ratio of the width of flow in the depressed section (*W*) to the width of total gutter flow (*T*):

$$E_0 = \frac{K_w}{K_w + K_0}$$

Equation 2.5
[6, pg. 10-35]

Where:
E_0 = ratio of depression flow to total flow
K_w = conveyance of the depressed gutter section (cfs)
K_0 = conveyance of the gutter section beyond the depression (cfs)

- Use the following form of Manning's equation to calculate conveyance, K_w and K_0:

$$K = \frac{1.486 \times A^{5/3}}{n \times P^{2/3}}$$

Equation 2.6
[6, pg. 10-36]

Where:
K = conveyance of cross section (cfs)
A = area of cross section (square feet)
n = Manning's n, for gutter flow
P = wetted perimeter (ft)

- Roadway cross slopes, S, typically range from ¼" to ½" per foot; therefore, in lieu of more detailed calculations, the wetted perimeter of the gutter section beyond the depression may be approximated as the horizontal distance: (T-W). In the depressed area, the cross slope, S_x, is slightly greater than the roadway slope, and the wetted perimeter in the depressed area can be approximated as: 1.03 times the horizontal distance, W. In either case, that portion of the wetted perimeter represented by the vertical curb face is ignored, as in the calculations of roadway ponded width.

- Calculate the area of the cross section in the depressed gutter section:

$$A_W = W \times S_d \times \left(T - \frac{W}{2}\right) + 0.5 \times D \times W$$

Equation 2.7
[6, pg. 10-36]

Where:
A_W = area of depressed gutter section (sq. ft.)
W = depression width, in feet (see Curb Inlet Section)
S_d = cross slope of depressed section (ft/ft)
T = calculated ponded width (ft)
D = curb opening in feet (see Curb Inlet Section)

- Calculate the area of the cross section of the gutter beyond the depressed area:

$$A_o = \frac{S_x}{2} \times (T - W)^2$$

Equation 2.8
[6, pg. 10-37]

Where:
A_o = area of gutter section beyond depression (sq. ft.)
W = depression width, in feet (see Curb Inlet Section)
S_x = roadway cross slope (ft/ft)
T = calculated ponded width (ft)

- Determine the equivalent cross slope of a depressed curb inlet:

$$S_e = S_x + \left(\frac{D}{W}\right) \times E_0$$

Equation 2.9
[6, pg. 10-37]

Where:
S_e = equivalent cross slope (ft/ft)
S_x = cross slope of the road (ft/ft)
D = gutter depression depth (ft)
W = gutter depression width (ft)
E_0 = ratio of depression flow to total flow

- Calculate the length of curb inlet required for total interception:

$$L_r = 0.6 \times Q^{0.42} \times S^{0.3} \times \left(\frac{1}{n \times S_e}\right)^{0.6}$$

Equation 2.10
[6, pg. 10-37]

Where:
L_r = length of curb inlet required (ft)
Q = total flow in gutter (cfs)
S = longitudinal slope of the gutter (ft/ft)
n = Manning's roughness coefficient for channel flow
S_e = equivalent cross slope (ft/ft)

- If no carryover flow is allowed, the inlet length is assigned a nominal dimension of at least L_r. Use a nominal length available as a standard length for curb opening inlets. Do not use a special inlet design for this purpose, unless justified for other reasons. If carryover flow is to be allowed, round the curb opening inlet length down to the next available (nominal) standard curb opening length and compute the carryover flow. Determine an inlet design length, L_a, such that $L_a > 0.7 \times L_r$, and compute carryover flow:

$$Q_{co} = Q \times \left(1 - \frac{L_a}{L_r}\right)^{1.8}$$

Equation 2.11
[6, pg. 10-38]

Where:
Q_{co} = carryover flow (cfs)
Q = total flow (cfs)
L_a =inlet design length (ft)
L_r = length of curb inlet opening required to intercept the total flow as calculated above (ft)

- Carryover flow rates should not exceed 0.5 cfs, or about 30% of the original discharge. Greater rates can be troublesome and will cause a significant departure from the principles of the Rational Method. In all cases, the carryover flow must be intercepted at some other point in the drainage system.

- Calculate the intercepted flow, which is the total discharge minus the carryover flow:

$$Q_i = Q - Q_{co}$$

Equation 2.12
[6, pg. 10-44]

Where:
Q_i = intercepted flow (cfs)
Q = total flow (cfs)
Q_{co} = carryover flow (cfs)

Curb Inlets in Sag: The capacity of a curb inlet in a sag depends upon the water depth at the curb opening and the height of the curb opening. The inlet operates as a weir to depths equal to the curb opening height and as an orifice at depths greater than 1.4 times the opening height. At depths between 1.0 and 1.4 times the opening height, flow is in a transition stage and the capacity should be based upon the lesser of the computed weir and orifice capacity. Generally, for design purposes, this ratio should be less than 1.4, such that the inlet operates as a weir. Calculate the curb inlet capacity in a sag as follows:

- If the depth of flow in the gutter , d_f, is less than or equal to 1.4 times the inlet opening height, D ($d_f \leq 1.4 \times D$), determine the length of inlet using weir control; otherwise, skip this step:

$$L = \frac{Q}{2.3 \times d_f^{1.5}}$$

Equation 2.13
[6, pg. 10-38]

Where:
L = length of inlet
Q = total flow reaching inlet (cfs)
d_f =effective depth of flow at inlet (cfs)

- If the depth of flow in the gutter is greater than the inlet opening height ($d_f > D$), determine the length of inlet required considering orifice control using the following equation:

$$L = \frac{Q}{0.67 \times D \times \sqrt{64.4 \times (d_f - D/2)}}$$

Equation 2.14
[6, pg. 10-39]

Where:
L = length of inlet
Q = total flow reaching inlet (cfs)
d_f = depth of flow at inlet (cfs)
D = depth of curb opening (feet)

- If both the preceding steps were performed, choose the larger of the two values for *L*; then select a standard inlet length that is larger than the calculated value of *L*.

Grate Inlets on Grade: Parallel bar grates are highly efficient types of gutter inlets; however, when crossbars are added for bicycle safety, the efficiency is reduced. Where bicycle traffic is a design consideration, the curved vane grate and the tilt bar grate are recommended for both their hydraulic capacity and bicycle safety features. In certain locations where leaves or trash may create constant maintenance problems, it may be desirable to prohibit bicycle traffic in order to allow the use of the more efficient parallel bar grates.

Use the following design procedure for grate inlets on grade:

- Compute the ponded width of flow, *T*. Follow the procedure specified in the Division on ponded width of street flow.
- Choose a grate inlet type and size.
- Find the ratio of frontal flow to total gutter flow, E_0, for a straight cross slope as for curb inlets on grade. No depression is applied to grate inlets on grade.
- Find the ratio of frontal flow intercepted to total frontal flow as follows:

$$R_f = 1 - 0.3 \times (v - v_0) \; if \; v > v_0$$
$$else$$
$$R_f = 1 \; if \; v < v_0$$

Equation 2.15
[6, pg. 10-42]

Where:

R_f = *ratio of frontal flow intercepted to total frontal flow*
v = approach velocity of flow in gutter (ft/sec)
v_0 = *minimum velocity that will cause splash over grate (ft/sec)*

- For triangular sections, calculate the approach velocity of flow in the gutter (v) as follows:

$$\begin{aligned} &If\ \text{section is triangular}: \\ &v = \frac{2 \times Q}{T^2 \times S_x} \\ &else: \\ &v = \frac{Q}{A} \end{aligned}$$

Equation 2.16
[6, pg. 10-42]

Where:
T, Q, S_x *as previously defined*

- Calculate the minimum velocity (v_0) that will cause splash over the grate using the Splash-over Velocity Equation below with the appropriate coefficients chosen from the table in Exhibit A-8 in Appendix A.

$$v_0 = k_0 + k_1 \times L_g + k_2 \times {L_g}^2 + k_3 \times {L_g}^3$$

Equation 2.17
[6, pg. 10-43]

Where:
L_g = *length of grate*
k_0, k_1, k_2, k_3 *from Appendix A*

- Find the ratio of side flow intercepted to total side flow, R_s:

$$R_s = \left[1 + \frac{0.15 \times v^{1.8}}{S_x \times {L_g}^{2.3}}\right]^{-1}$$

Equation 2.18
[6, pg. 10-43]

Where:
R_s = *ratio of side flow intercepted to total flow*
L_g = *length of grate*
S_x = *roadway cross slope*
v = approach velocity of flow in gutter (ft/sec)

- Determine the efficiency of the grate:

$$E_f = R_f \times E_0 + R_s \times (1 - E_0)$$

Equation 2.19
[6, pg. 10-43]

Where:
E_f = *efficiency of grate*

R_f, R_s, E_0 as previously defined

- Calculate the interception capacity of the grate:

$$Q_c = E_f \times Q = Q \times \left[R_f \times E_0 + R_s \times (1 - E_0)\right]$$

$$Q_i = Q_c \times 50\% \text{ (allow for clogging)}$$

Equation 2.20 [6, pg. 10-43]

Where:
Q = total flow to inlet
Q_c = calculated intercepted flow
Q_i = intercepted flow after clogging allowance
E_f , R_f, R_s, E_0 as previously defined

- Calculate the carryover flow:

$$Q_{co} = Q - Q_i$$

Equation 2.21 [6, pg. 10-44]

Where:
Q_{co} = carryover flow
Q,Q_i as previously defined

Grate Inlets in Sag: A grate inlet in sag operates as weir flow at low ponding depths, and as an orifice as depth increases. For design, follow the following procedure:

- Determine the allowable depth (*A_d*) at the inlet based upon allowable ponding width as previously discussed.
- Using the allowable depth, *A_d,* calculate the grate capacity as a weir and also as an orifice, and take the lower of the two values as the design flow.

- For weir flow:

$$Q_w = 3 \times p \times A_d^{3/2}$$

Equation 2.22 [6, pg. 10-44]

Where:
Q_w = intercepted flow as weir flow
A_d = allowable depth
p = the perimeter of the actual openings in the grate

- For orifice flow:

$$Q_o = 0.67 \times a \times \sqrt{64.4 \times A_d}$$

Equation 2.23 [6, pg. 10-45]

Where:
Q_o = intercepted flow as orifice flow
a = total area of all grate openings
A_d = allowable depth

- For final grate inlet size selection, choose the lowest of Q_0 or Q_w as calculated above. Calculate the intercepted flow by applying a 50% clogging factor:

$$Q_c = lowest\ of\ Q_w\ or\ Q_o$$

$$Q_i = Q_c \times 50\%\ \text{(allow for clogging)}$$

Equation 2.24
[6, pg. 10-45]

Where:
Q_o = intercepted flow as weir flow
a = total area of all grate openings
A_d = allowable depth

- For storm events greater than the 50-year event, there will be a carryover flow calculated as before:

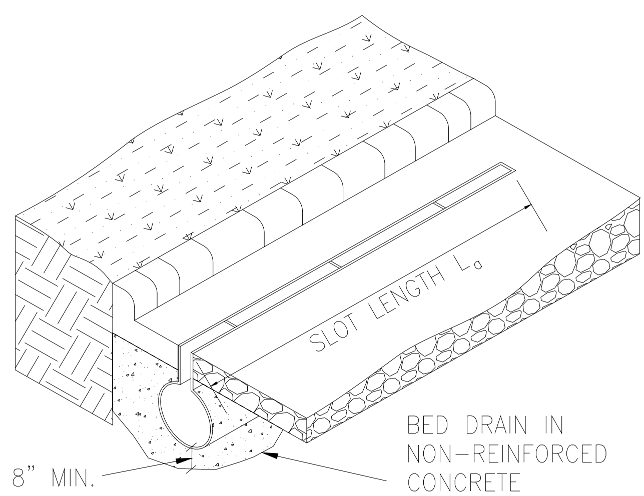
$$Q_{co} = Q - Q_i$$

Equation 2.25

Where:
Q_{co} = carryover flow
Q,Q_i as previously defined

The carryover flow in excess of the 50-year event must be analyzed to ensure that the excess flow is conveyed to the system outfall in a controlled manner.

Slotted Drains: Slotted drains have relatively long, narrow openings and as such, most of any slotted drain should be considered to be an inlet on grade, and not in a sag. Therefore, slotted drains should be exclusively designed as inlets on grade. See the Slotted Drain Plan below for a typical slotted drain layout.



Slotted Drain Plan

Use the following procedure for slotted drains:

- Determine the length of slotted drain required for interception of all of the water in the curb and gutter:

$$L_r = \frac{0.706 \times Q^{0.442} \times S^E \times S_x^{-0.849}}{n^{0.384}}$$

Equation 2.26
[6, pg. 10-40]

Where:
L_r = length of slotted drain required to intercept total flow
Q = total flow in curb and gutter
S = longitudinal slope of gutter
S_x = transverse slope of roadway
n = Manning's roughness coefficient for channel flow
E = function of S and S_x as defined in the following equation:

$$E = 0.207 - 19.084 \times S^2 + 2.613 \times S - 0.0001 \times S_x^{-2}$$
$$+ 0.007 \times S_x^{-1} - 0.049 \times S \times S_x^{-1}$$

The above set of equations are limited to the following ranges of variables:

$$Q \le 5.5\,cfs$$
$$S \le 0.09\ ft/ft$$
$$0.011 \le n \le 0.017$$

These equations are empirical, and extrapolation outside the above listed range of variables is not recommended.

- Select a slotted drain length based upon standard sizes and calculate the carryover flow. For optimal economy, the actual length of slotted drain, L_a should be about 0.65 times the required length, L_r.

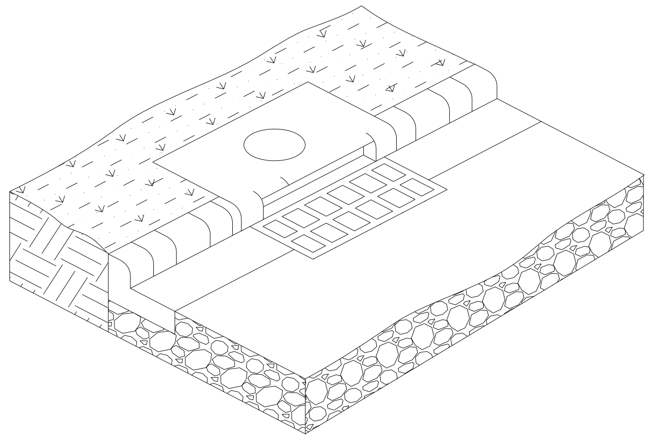
$$Q_{co} = 0.918 \times Q \times \left(1 - \frac{L_a}{L_r}\right)^{1.769}$$

Equation 2.27
[6, pg. 10-40]

Where:
Qco = carryover flow (cfs)
Q = total discharge (cfs)
La = design length of slotted drain (ft)
Lr = length of slotted drain required to intercept total flow (ft)

As previously discussed, slotted drains should not be considered to function as sag inlets, and should not be the downstream or terminal inlet in any drainage system.

Combination Curb and Grate Inlets: In instances where standard size inlets have insufficient capacity or where additional inlets cannot be feasibly installed, combination curb and grate inlets may be installed. See the Combination Inlet Plan, below for a typical layout.



Combination Inlet Plan

For combination curb and grate inlets, no depression is allowed in the inlet area. For design, the inlet capacity (on grade or sag) should be determined by assuming that there are two separate inlets. The capacity and carryover flow of the grate should be calculated first, then the curb inlet should be analyzed by assuming that it receives the carryover flow from the grate inlet. The design procedures previously outlined should be followed.

DIVISION 3 – STORM DRAIN DESIGN

Storm Drains 3.1

Conduit Design Procedure: Roadway storm drain systems are often represented as systems of links and nodes. Nodes consist of inlets or of junctions that are linked together by conduit runs, all leading to the outlet node. Flows through the system are usually computed using the Rational Method, as described elsewhere in this Manual. Normal design procedure entails beginning at the most remote upstream node and proceeding downstream to the outfall. The peak discharge is recomputed at each node based upon cumulative drainage area, runoff coefficient, and longest time of concentration contributing to that particular node. For conduit design, use the following steps:

- Identify the outfall to be used for the proposed storm drain system. As discussed elsewhere in this Manual, the selected outfall must be hydraulically capable of carrying the flows from the proposed storm drainage system. At the outfall, determine the tail water elevation as discussed in following paragraphs.
- Identify and map the drainage area associated with the drainage system. Include in the drainage area map a schematic of the conduit runs, inlet locations, and outfall location, along with the elevations of inlets, flow lines at junctions, and the tail water elevation at the outfall. Identify and delineate the sub area that drains to each inlet in the system.
- For each sub area, calculate the flow to the associated inlet using the Rational Method, where Quantity of flow equals runoff Coefficient times rainfall intensity times drainage Area ($Q = C \times i \times A$). Use of the Rational Method is discussed in detail elsewhere in this Manual. If the calculated time of concentration to any inlet is less than 5 minutes, use a minimum value of 5 minutes to size that specific inlet; however, use the calculated time of concentration at each inlet when accumulating times for downstream calculations.
- Label each sub area on the drainage area map with the area (acres), actual time of concentration (minutes), runoff coefficient, and design flows (cubic feet per second) to each inlet. For use in downstream calculations, also label each sub area with the value of runoff coefficient times area (*CA*).
- Size each inlet based upon the design flow coming to the inlet from its associated watershed sub area, plus any carryover flow coming to that inlet from other sub areas or other inlets. Ignore any carryover flows leaving the inlet.
- Size the conduit for non-pressure flow using Manning's equation, rearranged as shown below for circular pipe:

$$D = \frac{4}{3} \times \left(\frac{Q \times n}{\sqrt{S}} \right)^{\frac{3}{8}}$$

Equation 3.1
[6, pg. 10-50]

Where:
D = pipe diameter (feet)
Q = discharge (cfs)
n = Manning's roughness coefficient
S = slope of conduit (ft/ft)

For other shapes, apply Manning's equation and select a size of conduit with a capacity that is slightly higher than the required discharge to ensure non-pressure flow conditions.

- Estimate the flow velocity through the conduit. Assume uniform flow based upon an average depth of flow and determine the flow area based upon the average depth. Then use the continuity equation to calculate velocity:

$$v = \frac{Q}{A} \times 60$$

Equation 3.2
[6, pg. 10-50]

Where:
v = velocity (ft/min)
Q = discharge (cfs)
A = cross sectional area of flow (sq ft)

- Estimate the travel time through the conduit to the next downstream node by dividing the conduit length by the velocity:

$$t = \frac{L}{v}$$

Equation 3.3

Where:
t = travel time (minutes)
v = velocity (ft/minute)
L = conduit length (feet)

- Add this travel time to the time of concentration at the upstream end of the conduit run to represent the time of concentration at the downstream end of the run. When accumulating times, use the actual individually calculated times, even when the 5 minute minimum was used for inlet sizing.
- Determine the total drainage area, cumulative runoff coefficient times area (*CA*) and respective time of concentration for all conduits coming into a particular node. Based upon the accumulated value of CA and the longest time of concentration for all paths leading to the node, calculate the rainfall intensity and corresponding design flow to be used in sizing the next downstream conduit run.
- In some instances, the calculated discharge may decrease as the calculations progress downstream. This can happen when the time of concentration increases much more rapidly than the cumulative value of runoff coefficient times area. In such cases, use the previously calculated intensity and the accumulated value of *CA* to avoid designing for reduced discharge.

Tail Water Elevation: At each outfall, the elevation of the water surface at the outlet end of the proposed storm drain system may be a controlling factor in the performance of the completed storm drain system.

For a proposed storm drain system that outfalls into another buried conduit system, the water surface elevation at the outfall (the point where the proposed system joins the existing system) must be calculated by analyzing the entire system (existing plus proposed) as a single system.

For a proposed storm drain system that outfalls into an open channel, the tail water elevation at the outfall point should be the water surface elevation in the open channel corresponding to the 10-year storm event. If the 10-year water surface is unknown, a hydrologic analysis will be required to determine the flow in the channel at the outfall point. Using the 10-year flow in the outfall channel, the corresponding water surface elevation may be determined using Manning's equation, the existing channel cross section geometry, and the bed slope of the existing channel as the slope of the energy grade line.

Hydraulic Grade Line 3.2

The hydraulic grade line (hgl) is the elevation to which the water surface will rise in a manhole or inlet. The elevation of the hydraulic grade line can be a significant design limitation in sizing proposed underground storm drain systems. For proposed roadway drainage systems, the elevation of the hydraulic grade line cannot be allowed to exceed the limit described in the street design Division of this Manual.

The energy grade line (egl) in a storm buried conduit system may be determined by plotting the elevation of the total head in the system at critical points and then connecting the plotted points with a straight line. The total head at any point along the system may be determined by the following equation (assuming relatively flat slopes for the conduit):

$$h = z + d + \left(\frac{v^2}{2 \times 32.2}\right)$$

Equation 3.4
[7, pg. 5-1]

Where:
h = total head (feet)
z = elevation of the conduit flow line (feet)
d = depth of flow (feet)
v = average velocity in conduit (feet per second)

Mathematically, the hgl at any point along the length of the conduit is equal to the elevation of the energy grade line (egl) minus the velocity head at that point.

$$h_v = \left(\frac{v^2}{2 \times 32.2}\right)$$

Equation 3.5

Where:

h_v = velocity head (feet)
v = flow velocity (feet per second)

$$hgl = egl - h_v$$ *Equation 3.6*

Where:
hgl = elevation of the hydraulic grade line (ft)
egl = elevation of the energy grade line (ft)
h_v = velocity head (ft)

After completion of the preliminary conduit design, and after the tail water elevation in the outfall has been determined, the hydraulic grade line must be determined along the conduit and compared to the maximum allowable elevation. At any point where the hgl is above the allowable maximum, the conduit must be redesigned to provide for a lower hgl.

If the storm drain system functions in supercritical flow mode, the egl can be calculated by assuming critical depth at control points and working downstream from control points to calculate the egl. In order to determine the hgl for subcritical flow, a detailed analysis of the proposed system is required. In order to determine the hgl, the following procedure is acceptable:

- Plot the roadway gutter line(s) in profile, along with the proposed storm drain system; plot the tail water elevation at the outfall.
- Begin the analysis at the downstream end of the conduit system (outfall) and work upstream through the entire conduit system.
- At the outfall, the control point (c_p) is the elevation halfway between the critical water surface elevation (d_c) and the top inside surface of the conduit and is calculated as:

$$c_p = (d_c + D) \div 2$$ *Equation 3.7*

Where:
cp = elevation of the control point
d_c = elevation of flow at critical depth in conduit
D = diameter of pipe or height of conduit

- If the tail water at the conduit outlet is higher than c_p the egl will be the tail water elevation plus the velocity head for the conduit. The hgl will will be equal to the velocity head subtracted from the egl:
- If the tail water at the conduit outlet is lower than or equal to c_p, the hgl will be at c_p, and the egl will be the hgl plus the velocity head.
- Proceed upstream from the outfall and determine the egl at the downstream edge of the next junction, manhole, or control structure (node) upstream of the outfall by calculating the slope of the energy grade line with Manning's equation rearranged as follows:

$$s = \left[\frac{Q \times n}{1.486 \times a \times r^{2/3}} \right]^2$$

Equation 3.8

Where:
s = slope of the energy grade line
Q = quantity of flow (cfs)
n = Manning's n (see Exhibit A-7.b in Appendix A for typical values of n for common conduit types)
a = cross sectional area of flow
r = hydraulic radius

- The egl at the upstream node can then be calculated mathematically. The rise in egl along the conduit reach is equal to the slope of the energy grade line multiplied by the length of the conduit run. The egl at the upstream point is then equal to the rise along the conduit plus the egl at the outfall.

$$egl_1 = egl_0 + l \times \left[\frac{Q \times n}{1.486 \times a \times r^{2/3}} \right]^2$$

Equation 3.9

Where:
egl_1 = egl at upstream junction, etc.
egl_0 = egl at outfall
l = length of conduit run
Q, n, a, r as above

- Once the egl at the downstream edge of the node has been determined, apply appropriate head loss calculations to determine the egl at the upstream edge of the node. Proceed upstream to the next node. Continue upstream with the same procedure until the egl has been determined for the entire length of the conduit system.
- At the upstream and downstream edges of all nodes in the system, including the outfall, determine the hgl by subtracting the velocity head from the egl. Plot the hgl on the previously plotted profile of the storm drain system.
- Compare the plotted hgl vs. the roadway gutter lines. If, at any point along the length of the system, the hgl is higher than the allowable value, appropriate sections of the drainage system must be redesigned to provide for the hgl at a lower elevation.

Minor Losses 3.3

The major head loss in a drainage system consists of the friction loss within the conduit itself, and depends upon the type and length of the conduit as calculated in the previous discussion. Losses at manholes, junctions, bends, transitions, and other points are normally referred to as minor losses. These minor losses are usually insignificant in themselves; however, the cumulative effect of numerous small losses can have a detrimental effect on the performance of larger drainage systems.

As discussed previously, the egl is determined at the downstream edge of each node, and the energy losses at the node are then calculated and added to determine the egl at the upstream edge of the node. The following methods for estimating minor losses are recommended by the FHWA [7], and should be used for conduits designed as required by this Manual.

Exit Loss from a storm drain outlet is a function of the velocity change at the outlet of the pipe. For a sudden expansion such as at an endwall, the exit loss is:

$$H_0 = \left[\frac{v_o^2 - v_d^2}{2 \times 32.2}\right]$$

Equation 3.10
[7, pg. 7-11]

Where:
H_o = head loss at outlet
v_o = average outlet velocity
v_d = average velocity downstream of outlet

Bend Loss for storm drains is generally minor, but can be estimated as follows:

$$H_b = 0.0033 \times \Delta \times \left(\frac{v^2}{2 \times 32.2}\right)$$

Equation 3.11
[7, pg. 7-11]

Where:
H_b = head loss at bend
v = average velocity through the bend
Δ = angle of curvature in degrees

Transition Loss takes place where a conduit changes size. Transitions should be avoided and manholes should be installed at changes in pipe sizes. Energy losses at transitions take place both at expansions and at contractions, and can be expressed in terms of kinetic energy at the two ends of the transition. Contraction and expansion losses can be evaluated for pipes operating under non-pressure flow conditions as follows:

$$H_e = K_e \times \left[\frac{v_1^2 - v_2^2}{2 \times 32.2}\right]$$

Equation 3.12
[7, pg. 7-12]

Where:
H_e = head loss at contraction
K_e = expansion coefficient
v_2 = velocity downstream of transition
v_1 = velocity upstream of transition

For gradual contractions, $K_c = 0.5 \times K_e$.

$$H_c = K_c \times \left[\frac{v_2^2 - v_1^2}{2 \times 32.2} \right]$$

Equation 3.13
[7, pg. 7-12]

Where:
H_c = head loss at contraction
K_c = contraction coefficient (= 0.5 x K_e)
v_2 = velocity downstream of transition
v_1 = velocity upstream of transition

Typical values of K_e for gradual expansions are tabulated in Exhibit A-9.a in Appendix A. For gradual contractions, the values of K_c are 50% of the values for K_e shown in Exhibit A-9.a.

Typical values of K_c for sudden contractions are tabulated in Exhibit A-9.b in Appendix A.

For conduits operating under pressure flow conditions, the following equations apply:

$$H_e = K_e \times \left[\frac{v_1^2}{2 \times 32.2} \right]$$

Equation 3.14
[7, pg. 7-13]

Where:
H_e = head loss at expansion
K_e = expansion coefficient
v_1 = velocity upstream of transition

$$H_c = K_c \times \left[\frac{v_2^2}{2 \times 32.2} \right]$$

Equation 3.15
[7, pg. 7-13]

Where:
H_c = head loss at expansion
K_c = expansion coefficient
v_2 = velocity downstream of transition

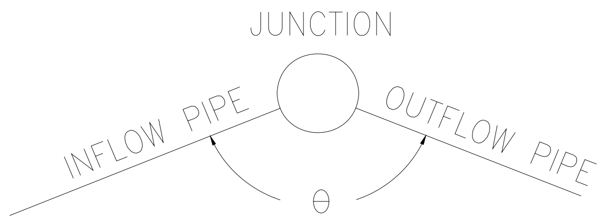
For conduits operating under pressure flow, the values of K_e for sudden and gradual enlargement, respectively, can be obtained from Exhibits A-9.c and A-9.d in Appendix A. For sudden contractions in pressure flow, the value of K_c can be obtained from Exhibit A-9.e in Appendix A.

Junction Loss: A pipe junction is the connection of a lateral pipe to a larger trunk pipe without the use of an access hole structure. The minor loss equation for a pipe junction is a form of the momentum equation as follows:

$$H_j = \frac{(Q_o \times V_o) - (Q_i \times V_i) - (Q_l \times V_l \times \cos\theta)}{0.5 \times 32.2 \times (A_o + A_i)} + h_i - h_o$$

Equation 3.16
[7, pg. 7-16]

Where:
H_j = head loss at junction (feet)
Q_o, Q_i, Q_l = outlet, inlet, and lateral flows (cfs)
V_o, V_i, V_l = outlet, inlet, and lateral velocities, (cfs)
h_o, h_i = outlet and inlet velocity heads (ft)
A_o, A_i = outlet and inlet cross-sectional areas (sq. ft.)
θ = angle between inflow and outflow pipes



Loss at Inlets, Access Holes, and Manholes: At access holes, manholes, or inlets (for simplicity, all are referenced as access holes) there may be multiple pipes flowing into the structure, typically but not necessarily, with a single outflow pipe. Each inflow pipe will represent a separate branch of the buried conduit system requiring its own analysis. From the access hole and proceeding upstream, each inflow pipe represents a separate buried conduit system with its own hydraulic grade line requiring separate analysis and determination.

In order to proceed systematically with the analysis and depiction of the system hgl through such an access hole, the following procedure is recommended:

Determine the water surface in the access hole by assuming that the downstream section of drainage conduit functions as a culvert, and calculate the water surface in the access hole as if it were the headwater elevation determined through culvert calculations (see the Division of this Manual devoted to cross-drainage structures for culvert calculations).

For inflow pipes with inverts that are above the access hole water surface elevation, determine the control point (c_p) for that pipe, then proceed with hydraulic grade line calculations moving upstream in the inflow pipe as was done for the main branch.

For other inflow pipes with inverts below the access hole water surface elevation, the energy loss encountered going from one pipe to another through the access hole should be calculated as a function of the velocity head of the outlet pipe. Each individual inflow pipe will then be associated with a specific elevation of the energy grade line, which can be used in determining the hydraulic grade line for that individual inflow pipe and all associated upstream sections on that branch.

The total energy loss through the structure of an access hole may be represented as a coefficient multiplied by the velocity head:

$$H_{ah} = K \times \left[\frac{v_o^2}{2 \times 32.2} \right]$$

Equation 3.17
[7, pg. 7-17]

Where:
H_{ah} = head loss at structure
K = coefficient
v_o = velocity of outlet pipe

The value of K can be determined as the product of specific individual coefficients as follows:

$$K = K_o \times C_D \times C_d \times C_Q \times C_p \times C_B$$

Equation 3.18
[7, pg. 7-17]

Where:
K = adjusted loss coefficient
K_o = coefficient for relative access hole size
C_D =correction factor for pipe diameter (pressure flow only)
C_d = correction factor for flow depth
C_Q = correction factor for relative flow
C_p = correction factor for plunging flow
C_B = correction factor for benching
all as determined in following equations

K_o is estimated as a function of the relative access hole size and angle of deflection between the inflow and outflow pipes:

$$K_o = 0.1 \times \left(\frac{b}{D_o} \right) \times (1 - \sin\theta) + 1.4 \times \left(\frac{b}{D_o} \right)^{0.15} \times \sin\theta$$

Equation 3.19
[7, pg. 7-18]

Where:
K_o =coefficient for access hole size
b = access hole diameter
D_o = outlet pipe diameter
θ = angle between inflow and outflow pipes

A change in head loss due to differences in pipe diameter is only significant in pressure flow situations when the ratio of the outflow pipe diameter to the inflow pipe diameter is greater than 3.2; otherwise C_D is set to 1.

$$if\ \left(\frac{d_{aho}}{D_o}\right) > 3.2\ then$$

Equation 3.20
[7, pg. 7-18]

$$C_D = \left(\frac{D_o}{D_i}\right)^3$$

$$else\ C_D = 1$$

Where:
C_D =correction factor for pipe diameter (pressure flow only)
d_{aho} = depth of water in access hole
D_o = outlet pipe diameter
D_i = inflow pipe diameter

Calculate d_{aho} as the depth from the elevation of the water surface elevation in the access hole (as previously determined). The correction factor for flow depth, C_d, is significant only in cases of free surface flow or low pressures, when the d_{aho} to D_o ratio is less than 3.2. In cases where this ratio is greater than 3.2, C_d is set to 1.

$$if\ \left(\frac{d_{aho}}{D_o}\right) < 3.2\ then$$

Equation 3.21
[7, pg. 7-20]

$$C_d = 0.5 \times \left(\frac{d_{aho}}{D_o}\right)^{0.6}$$

$$else\ C_d = 1$$

Where:
C_d =correction factor for flow depth (free surface flow only)
d_{aho} = depth of water in access hole
D_o = outflow pipe diameter

If three or more pipes enter the access hole structure at approximately the same elevation, a correction for relative flow (C_Q) is required. If less than three inflow pipes are installed at the same elevation, set C_Q to 1.0.

$$if\ \ pipes \geq 3\ then$$

Equation 3.22
[7, pg. 7-20]

$$C_Q = 1 - (2 \times \sin\theta) \times \left(1 - \frac{Q_i}{Q_o}\right)^{0.75} + 1$$

$$else\ C_Q = 1$$

Where:
C_Q =correction factor for relative flow
θ = angle between inflow and outflow pipes

Q_o = flow in outlet pipe
Q_i = flow in inflow pipe

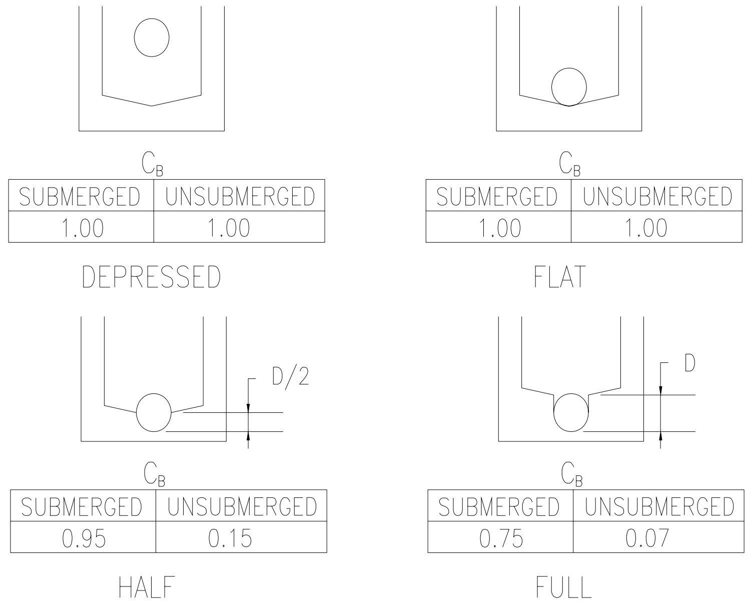
The correction factor for plunging flow, C_p, is required when a higher elevation flow plunges into an access hole that has both an inflow line and an outflow line in the bottom of the access hole. Otherwise, C_p is set to 1. This correction factor corresponds to the effect another inflow pipe, plunging into that access hole, has on the inflow pipe for which the head loss is being calculated. That is, the correction is applied for pipe #1 when pipe #2 discharges plunging flow. Flows from a grate inlet or a curb opening inlet are considered to be plunging flow and the losses should be computed using C_p.

$$
\begin{aligned}
&if\ \ h > d_{aho}\ then \\
&C_p = 1 + 0.2 \times \left(\frac{h}{D_o}\right) \times \left(\frac{h - d_{aho}}{D_o}\right) \\
&else\ C_p = 1
\end{aligned}
$$

Equation 3.23
[7, pg. 7-21]

Where:
C_p =correction factor for plunging flow
h = vertical distance of plunging flow measured from the flow line of the higher elevation inlet pipe to the center of the outflow pipe
D_o = outlet pipe diameter
d_{aho} = depth of water in access hole

Benching consists of constructing the bottom of an access hole to direct the flow through the access hole and reduce the associated head loss. The significant types of benching are shown in the following illustration. The correction factor for benching, C_B, is obtained directly from the illustration, for either a submerged or unsubmerged condition [7, pg. 7-22]. For flow depths between fully submerged and unsubmerged, a linear interpolation of C_B is appropriate.



Benching

In summary, the head loss through an access hole from the outflow pipe through any given inflow pipe should be estimated by accumulating all the individual correction coefficients and multiplying them together to obtain a head loss coefficient for the inflow pipe. This head loss coefficient should then be multiplied by the velocity head to obtain the head loss for the inflow pipe.

Channel Capacity 4.1

Conservation of Energy: Flow in open channels is analyzed using basic principles of conservation of energy. The total energy at any given point can be represented as:

$$E_t = Z + y + \frac{v^2}{2 \times 32.2}$$

Equation 4.1
[7 pg. 5-1]

Where:
E_t =total energy (ft)
Z = elevation above a given datum (ft)
y= flow depth measured above bottom of channel (ft)
v = mean velocity in channel (ft per sec)

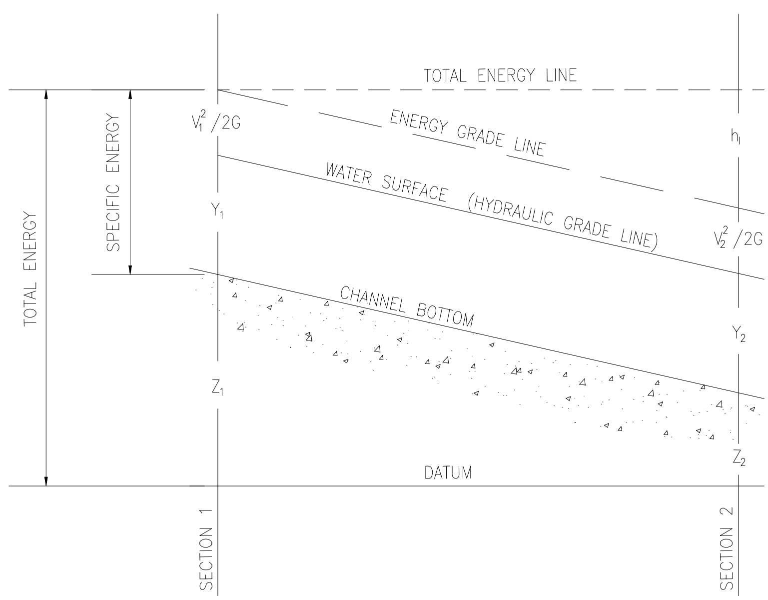
The change in total head between two channel cross sections (section 1 and section 2) may be described as the difference in energy between the two sections:

$$Z_1 + y_1 + \frac{v_1^2}{2 \times 32.2} = Z_2 + y_2 + \frac{v_2^2}{2 \times 32.2} + h_l$$

Equation 4.2
[7 pg. 5-2]

Where:
Subscripts 1 and 2 refer to data for section 1 and 2, respectively
h_l = head loss between the two sections

The terms in the energy equation are illustrated below:



Total Energy in Open Channels

Specific Energy: Specific energy is the energy head relative to the channel bottom, and is the sum of the flow depth and the velocity head;

$$E = y + \frac{v^2}{2 \times 32.2}$$

Equation 4.3
[4 pg. 5-2]

Where:
E, y, v =as previously defined

Flow Classification: A **steady** flow is one in which the discharge passing a given cross-section remains constant in time. When the discharge varies with time, the flow is **unsteady**. A **uniform flow** is one in which the flow rate and depth remain constant along the length of the channel. When the flow rate and depth vary along the channel, the flow is considered **varied**.

Most natural flow conditions are neither steady nor uniform; however, in most cases, it can be assumed that the flow will vary gradually in time and space, and can be described as **steady**, **uniform flow** for short periods and distances.

The Froude number (F_r) represents the ratio of inertial forces to gravitational forces and is defined for rectangular channels by the following equation:

$$F_r = \frac{v}{\sqrt{(32.2 \times y)}}$$

Equation 4.4
[7 pg. 5-3]

Where:
y, v =as previously defined

Critical Flow occurs when the Froude number has a value of one. The flow depth at critical flow is referred to as critical depth, and represents the minimum specific energy for any given discharge.

Subcritical Flow occurs when the Froude number is less than one. In this state of flow, depths are greater than critical depth, small water surface disturbances travel both upstream and downstream, and the control of the flow depth is always located downstream.

For analysis or determination of a water surface profile for a channel reach that is in subcritical mode, the analysis must begin at the downstream control section and proceed upstream.

Supercritical Flow occurs when the Foude number is greater than one. In this state of flow, depths are below critical depth, small water surface disturbances are always swept downstream, and the location of the flow control is always upstream.

For analysis or determination of a water surface profile for a channel reach that is in supercritical mode, the analysis must begin at the upstream control point and proceed downstream.

A Hydraulic Jump occurs as an abrupt transition from supercritical flow to subcritical flow. As the jump energy is dissipated, there are significant changes in depth and velocity, with the potential for significant turbulence in the flow. Where the channel slope changes from steep to mild, and the Froude number approaches a value of one, a hydraulic jump should be expected.

Normal Depth: The depth of flow in a channel of constant cross section and slope is primarily a function of the channel's resistance to flow or roughness. This depth is called the normal depth of the channel and is computed using Manning's equation:

$$Q = \frac{1.486}{n} \times A \times R^{2/3} \times \sqrt{s}$$

Equation 4.5
[7, pg. 5-5]

and

$$R = \frac{A}{wp}$$

Where:
Q =quantity of flow in the channel (cubic feet / second)
A = cross sectional area of flow
wp = wetted perimeter of flow area
R = hydraulic radius
s = slope of the energy grade line (for steady, uniform flow, s may be taken as the slope of the channel bottom)
n = Manning's roughness coefficient

Data for Manning's *n* is tabulated in Appendix A. Exhibit A-7.d contains *n* values for drainage channels where the depth of flow is relatively great in relation to the depth of surface roughness elements, such as grass or cobbles. Exhibit A-7.e contains data to allow calculation of an *n* value depending on material types and channel configuration factors. Exhibit A-7.f provides *n* values for relatively shallow depths of flow, while Exhibits A-7.g and A-7.h provide a means of calculating *n* values as a function of vegetal retardance and hydraulic radius for shallow flow where the height of grass is large in relation to flow depth. Exhibit A-7.i contains methodology for calculation of Manning's *n* as a function of vegetal retardance and shear stress.

Once Manning's equation has been solved for normal depth, flow velocities can be obtained simply through the use of the continuity equation:

$$v = \frac{Q}{A}$$

Equation 4.6

Where:
Q =quantity of flow in the channel (cubic feet / second)
A = cross sectional area of flow
v = average velocity in the channel

Water surface profiles and velocities may be determined by normal depth methods for relatively short sections of channel where channel geometry is constant, and where there are no obstructions, transitions in channel width, or cross-drainage structures in the channel. For complex channels with transitions, cross drainage structures, etc. included in the channel, a complete backwater analysis is required to determine the correct water surface profile and flow velocity. The backwater analysis should be performed using methods outlined in the documentation for the current version of HEC-RAS [5], or approved equivalent. The completed water surface profile must include detailed effects of all transitions, bridge and culvert backwater, and other obstructions in the channel. This detailed analysis is especially important for roadside ditches. The cumulative backwater effects of multiple driveway culverts has the potential to completely destroy the effective conveyance capacity of a roadside ditch, and all such culvert effects must be taken into account during design.

Flow in Bends: The change in flow direction around a bend in an open channel creates a centrifugal force that tends to move flow to the outside of the bend, with a resulting elevation increase at the outside of the bend. The water surface becomes super elevated similar to the surface of high-speed turns in racetracks. The amount of super elevation may be calculated with the following equation, which is valid for subcritical flow conditions:

$$\Delta d = \frac{v^2 \times t}{32.2 \times r_c}$$

Equation 4.7
[7 pg. 5-10]

Where:
Δd =difference in water surface elevation between the inner and outer bank of the channel (feet)
v = average channel velocity (ft / sec)
t = top width of the channel (feet)
r_c = radius of bend at the centerline of the channel (ft)

The water surface elevation at the outside of the bend will be *Δd/2* greater than the water surface elevation at the channel centerline. This increase in water surface elevation at the outside of bends must be taken into account when determining the amount of freeboard required for the channel.

Erosion Protection 4.2

Flow in a channel imposes forces on the elements that make up the lining of the channel bottom and sides. If the forces imposed by the channel flow exceed the forces required to dislodge or move the lining elements, then erosion occurs. The basic method for analyzing and designing erosion protection systems for channels is to determine the erosive forces imposed by the flow (shear stress or tractive force), then determine the maximum force that can be tolerated by the channel lining elements before erosion begins. If the existing channel lining elements cannot withstand the forces generated by the design flow, then the designer must choose a suitable channel lining that will resist the applied forces, and that will result in an erosion proof system.

The FHWA's publication entitled: *Design of Roadside Channels with Flexible Linings* [8] provides detailed data on stable channel design and erosion protection. The following discussion is taken largely from that document.

Shear Stress: The hydrodynamic force created by water flowing in a channel causes a shear stress on the channel bottom, which is also known as the tractive force. The average shear stress is equal to:

$$\tau = 62.4 \times R \times s$$

Equation 4.8
[7 pg. 5-11]

Where:
τ =average shear stress (lbs per square foot)
R = hydraulic radius (feet)
s = average energy slope (ft / ft)

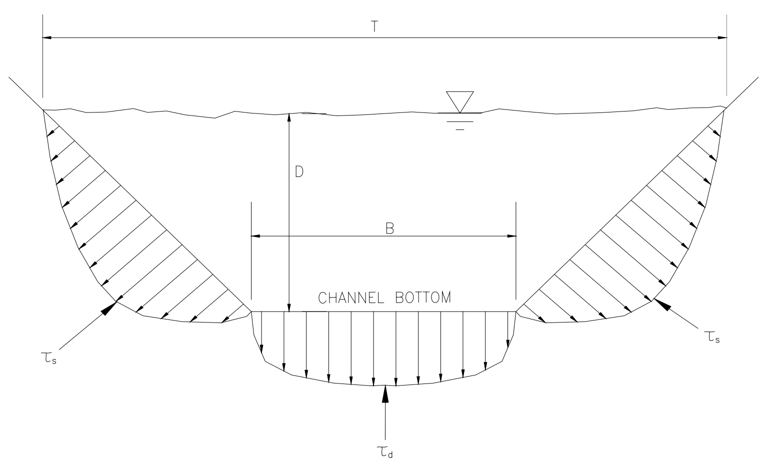
The maximum shear stress for a straight channel occurs on the channel bed, and is computed as:

$$\tau_d = 62.4 \times D \times s$$

Equation 4.9
[7 pg. 5-11]

Where:
τ_d =maximum shear stress (lbs per square foot)
D = maximum depth of flow (feet)
s = average energy slope (or bed slope) (ft / ft)

Shear stress in channels is not uniformly distributed along the wetted perimeter of the channel. The typical shear stress in a trapezoidal channel approaches zero at the toes of slopes with the maximum stress on the bed of the channel at its centerline. The maximum stress for the side slopes occurs near the lower third of the slope as illustrated below:



Typical Stress Distribution

For trapezoidal channels where the ratio of bottom width to flow depth is greater than 4, Equation 4.9 provides an appropriate design value for shear stress on a channel bottom. For narrower channels, Equation 4.9 overestimates shear stress by as much as 35% in very narrow channels with steep side slopes. However, for most major drainage channels, Equation 4.9 provides an adequate solution without excessive over design.

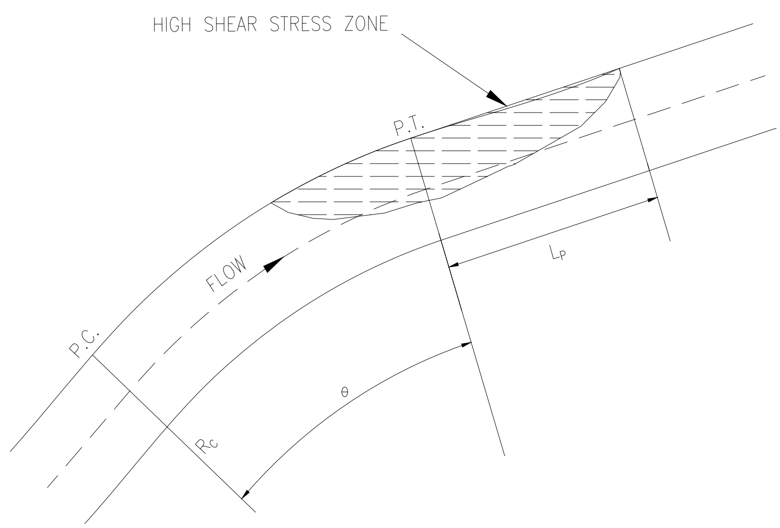
Shear stress on channel sides is generally reduced compared to the maximum stress on the channel bottom. The maximum shear stress on the side of a channel is given by:

$$\tau_s = K \times \tau_d$$
$$for\ z \leq 1.5\ K = 0.77$$
$$for\ 1.5 < z < 5\ K = 0.066 \times z + 0.67$$
$$for\ z \geq 5\ K = 1.0$$

Equation 4.10
[8, pg. 3-6]

Where:
τ_s =shear stress on channel side (lbs per square foot)
τ_d = maximum shear stress as before
z = horizontal dimension for slopes (z:1) for instance, with a slope of 3 horizontal to 1 vertical, z = 3

Flow around bends imposes higher shear stresses on the channel sides and bottom compared to straight reaches, as shown in the following illustration:



Stress in Bends

The bend shear stress can be calculated using the following equation:

$$\tau_b = K_b \times \tau_d$$

$$if\ \frac{R_c}{T} \leq 2\ then\ K_b = 2.00$$

Equation 4.11
[8 pg. 3-12]

$$for\ 2 < \frac{R_c}{T} < 10\ then$$

$$K_b = 2.38 - 0.206 \times \left(\frac{R_c}{T}\right) + 0.0073 \times \left(\frac{R_c}{T}\right)^2$$

$$else\ \ for\ \frac{R_c}{T} \geq 10\ K_b = 1.05$$

Where:
τ_b =bend stress (lbs per square foot)
R_c = radius to channel centerline (feet)
T = top width at channel surface (feet)

The increased shear stress persists downstream from the bend a distance L_p as determined below:

$$L_p = \frac{0.604 \times R^{7/6}}{n_b}$$

Equation 4.12
[7 5-13]

Where:
L_p =length of protection downstream of point of tangency (feet)
R = hydraulic radius
n_b = Manning's roughness in bend

Permissible Shear Stress: The permissible shear stress (τ_p) in a channel defines the force required to initiate movement of the channel lining material. For stone channel linings, the permissible shear stress is relatively independent of the properties of the underlying soil. The permissible shear stress indicates the force required to move the stone particles themselves, not the underlying soil. However, once the stone particles begin to move, the underlying soil is exposed to erosive forces, and can be swept away.

The value used for permissible shear stress should contain a safety factor, as expressed mathematically below:

$$\tau_p \geq SF \times \tau_d$$

Equation 4.13
[8 pg. 3-1]

Where:
τ_p = permissible shear stress

SF = safety factor
τ_d *= maximum shear stress, as previously defined*

For design, calculate the maximum shear stress at the point of interest as described previously and multiply by the appropriate safety factor to determine the permissible shear stress. Choose an appropriate lining such that the permissible shear stress for the proposed lining equals or exceeds the calculated value for permitted shear stress. The safety factor to be used depends on the nature of the application, the cost of lining failure, and engineering judgment.

Exhibit A-10.a in Appendix A contains a listing of typical values of permissible shear stresses for various bare soil and stone linings. Recommended maximum permissible shear stress values for various synthetic linings are listed in Exhibit A-10.b in Appendix A, while Exhibit A-10.c contains a listing of permissible shear stresses for vegetative linings. **In all cases, the value of *SF* to be used for vegetative linings shall be no less than 1.5.** This value of *SF* is intended to provide some measure of protection against the variability of vegetative cover in the local area.

In all cases, the type of vegetative lining to be used shall be acceptable to the City of Marble Falls. For instance, although Kudzu is commonly listed as a vegetative lining in publications and analyses from the Federal Highway Administration, Kudzu is deemed to be an undesirable plant for use in the Marble Falls area and its use should be avoided entirely.

For larger stone sizes not included in the Exhibits, the permissible shear stress may be calculated as:

$$\tau_p = 4 \times D_{50}$$

Equation 4.14
[7 pg. 5-18]

Where:
τ_p *=maximum permissible shear stress (lbs per square foot)*
D_{50} *= mean riprap size (feet)*

Riprap stability on a steep slope depends on forces acting on an individual stone making up the riprap. On a steep slope, the weight of a stone has a significant component in the direction of flow, and the stone will tend to move on the flow direction more easily than on a milder gradient. For channels with bed slopes flatter than 5%, use the following equation for riprap size:

$$D_{50} \geq \frac{RSF \times d \times S}{F^* \times (SG - 1)}$$

Equation 4.15
[8, pg. 6-3]

$$SG = wt \div 62.4$$

Where:
D_{50} = mean riprap size (feet)
RSF = safety factor for riprap (replaces SF in previous calculations)
d = maximum channel depth (ft)
S = channel slope (ft / ft)
F^* = Shield's parameter, dimensionless
SG = specific gravity of stone riprap
wt = weight of stone (use 165 lbs per cubic foot if weight is unknown)
Shield's parameter and RSF are functions of the Reynolds number as tabulated below:

Reynolds number (Rn)	F^*	*RSF*
$Rn \leq 4x10^4$	*0.047*	*1.0*
$4x10^4 < Rn < 2x10^5$	*linear interpolation*	*linear interpolation*
$Rn \geq 2x10^5$	*0.15*	*1.5*

Note that the above equation contains a safety factor (*RSF*); therefore, the application of other safety factors is unnecessary for this calculation. For channels with bed slopes greater than 10%, see reference [8], Appendix D.

In addition to the typical values for maximum allowable shear stress for selected linings listed in Appendix A, many manufacturers of channel lining materials routinely provide maximum allowable shear stresses for their products that have been determined through field and laboratory tests.

Required Erosion Protection: Proposed channel bottoms and sides must be protected from erosion by determining the maximum shear stress developed for the design flow, then selecting an appropriate lining material to be installed in the proposed channel based upon the maximum permissible shear stress for the proposed lining material.

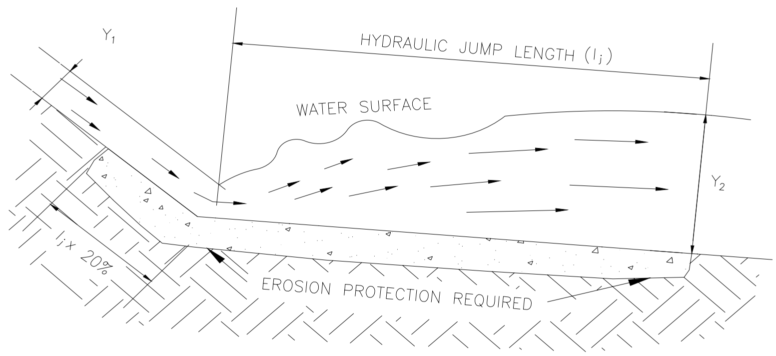
Alternatively, the designer may elect to determine the maximum allowable average channel velocity for a selected type of channel lining. By combining the continuity equation, Manning's equation, and Equation 4.9, the maximum permissible average channel velocity based upon maximum allowable shear stress for a given channel lining can be determined:

$$v_p = \left(\frac{1.486}{n \times \sqrt{62.4 \times D}} \right) \times R^{1/6} \times \sqrt{\tau_p \times SF}$$

Equation 4.16
[8 pg. 2-6]

Where:
τ_p =maximum shear stress (lbs per square foot)
SF, n, D, R as previously defined

Where hydraulic jumps occur in open channels, erosion protection for channel bottom and sides is required for the full length of the hydraulic jump, plus 20% of the length upstream of the face as shown in the illustration below:



Erosion Protection at Hydraulic Jumps

For typical hydraulic jumps in trapezoidal channels, the length of a hydraulic jump may be estimated from the table in Exhibit A-11.

Once a proposed channel lining has been chosen and all erosion protection devices have been determined, the capacity of the proposed channel must be analyzed using the channel roughness factor (*n* value) for the proposed lining and all included erosion protection measures. Exhibits A-7.d, e, and f in Appendix A contain listings of values of Manning's *n* for various channel linings and conditions. Exhibits A-7.g, h, and i in Appendix A contain data and equations for calculation of Manning's *n* for various types of grass linings.

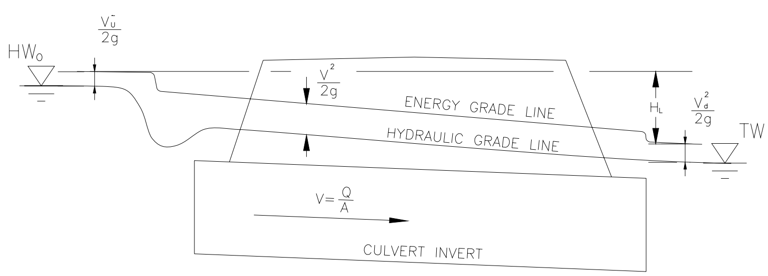
If the channel capacity with the proposed erosion protection is inadequate for the design flow and freeboard requirements, the size and geometry of the proposed channel must be modified through an iterative process to completely design the proposed channel with adequate conveyance capacity, freeboard, and erosion protection.

Design of Roadside Channels with Flexible Linings [8] contains additional details, along with methods and examples of design for channel linings for complex situations.

Culvert Analysis 5.1

Cross drainage structures consisting of culverts can be very economical, because the basic design takes advantage of increased head upstream of the culvert inlet to move water at increased velocities through a relatively small diameter structure. Additionally, a properly designed and installed culvert crossing often functions as a detention pond by impounding significant volumes of storm water flow upstream of the culvert inlet.

A culvert is a buried section of storm drain conduit and may be analyzed exactly as described in the Division of this Manual on storm drains. The water surface upstream of the culvert can be determined by beginning with the tail water elevation downstream of the culvert barrel and calculating the hydraulic grade line through the structure to determine the water surface at the upstream end of the culvert. A section view of a typical culvert is shown below, including a graphical representation of terms used in following discussions.



Culvert Profile

Culverts may operate hydraulically under any one of several different modes, and for a complete analysis, the culvert must be analyzed under each operational mode and must produce an acceptable upstream water surface elevation under each mode. The following required methods and analysis were taken from data published by the FHWA in *Hydraulic Design of Highway Culverts* [9].

For multiple barrel (or multiple box) structures, divide the design flow by the number of barrels and perform the analysis for one barrel using the appropriate fraction of the design flow.

- Calculate average velocity through the barrel:

$$v = \frac{Q}{a}$$

Equation 5.1
[9, pg. 34]

Where:
v = average velocity through the culvert (ft/sec)
Q = design flow per barrel
a = flow area (sq ft)

- Calculate the velocity head in the culvert barrel:

$$h_v = \left(\frac{v^2}{2 \times 32.2} \right)$$

Equation 5.2
[9, pg. 34]

Where:
h_v = velocity head (ft)

Inlet Control: A culvert operates under inlet control when the barrel size, shape, and slope will allow more flow to be conveyed than can enter the upstream, or inlet, end of the barrel. For inlet control, the inlet is either unsubmerged or submerged, and the water surface should be determined as follows:

$$if \left(\frac{Q}{A \times D^{0.5}} \right) < 3.5\,(unsubmerged)$$

Equation 5.3
[9, pg. 192]

$$then\ \frac{HW_i}{D} = K \times \left(\frac{Q}{A \times D^{0.5}} \right)^M$$

$$if \left(\frac{Q}{A \times D^{0.5}} \right) > 4.0\,(submerged)$$

$$then\ \frac{HW_i}{D} = c \times \left(\frac{Q}{A \times D^{0.5}} \right)^2 + Y + k_s$$

$$\text{for mitered inlets}: k_s = 0.7 \times s$$

$$\text{otherwise}: k_s = -0.5 \times s^2$$

Where:
HW_i = water depth above inlet invert (ft)
D = interior height of culvert barrel (ft)
Q = flow for design storm (cubic feet per second)
A = cross sectional area of culvert barrel (sq. ft.)
s = culvert barrel slope (ft. per ft.)
K, M, c, Y = constants from Exhibit A-12.a in Appendix A
If necessary, interpolate between submerged and unsubmerged values

Outlet Control: A culvert operates under outlet control when the inlet end of the barrel allows more flow to enter the barrel than can be conveyed by the barrel. The upstream water surface elevation is calculated as the total head loss through the culvert added to the tail water elevation at the downstream end.

- The entrance loss at the inlet end can be determined by multiplying the velocity head by an experimentally determined coefficient:

$$h_e = K_e \times h_v$$

Equation 5.4
[9, pg. 34]

Where:

h_e = head loss at entrance (ft)
K_e = entrance loss coefficient
h_v = velocity head (ft)

- Typical values of K_e are listed in the table in Exhibit A-12.b in Appendix A.

- The head loss through the culvert barrel is calculated using a rearranged version of Manning's equation:

$$h_b = \left(\frac{29 \times n^2 \times l}{r^{4/3}} \right) \times h_v$$

Equation 5.5
[9, pg. 34]

Where:
h_b = friction loss through the culvert barrel
n = Manning's n for the culvert barrel
l = length of culvert barrel
r = hydraulic radius of the culvert barrel
h_v = velocity head (ft)

- See the table in Exhibit A-7.b in Appendix A for tabulated values of Manning's *n* for culvert barrels.

- Calculate the exit loss as a coefficient times the change in velocity head at the outlet:

$$h_o = K_o \times \left(\frac{v^2 - v_d^2}{2 \times 32.2} \right)$$

Equation 5.6
[9, pg. 35]

Where:
h_o = exit loss (ft)
K_o = exit loss coefficient (normally 1, unless otherwise justified)
v = velocity inside the culvert barrel (ft/sec)
v_d = velocity in the channel downstream of the outlet (ft/sec)

- The value of K_o is normally taken as 1, unless a detailed analysis of the outlet condition shows that a different value is justified. Also, the downstream velocity, v_d, is often neglected. If both conditions are true, then the exit loss is simply the value of the velocity head in the barrel.

- Calculate the minor loss, h_m, as the sum of bend losses, junction losses, grate losses and other losses using methods discussed in other Divisions of this Manual.

- Calculate the total loss as the sum of the individual losses calculated immediately above:

$$H_L = h_e + h_b + h_o + h_m$$

Equation 5.7
[9, pg. 34]

- The elevation of the water surface upstream of the culvert then can be calculated from the tail water elevation, *TW*, as follows:

$$HW_o + \frac{v_u^2}{2 \times 32.2} = TW + \frac{v_d^2}{2 \times 32.2} + h_l$$

Equation 5.8
[9, pg. 36]

Where:
HW_o = water surface elevation upstream of culvert (ft)
v_u = velocity upstream of culvert (ft/sec)
v_d = velocity downstream of culvert (ft/sec)
TW = elevation of water surface downstream of culvert

Compare the values of HW_i and HW_o as calculated above, and use the higher of the two as the upstream water surface. If the calculated water surface elevation is lower than or equal to the maximum acceptable value, the proposed culvert design is sufficient; otherwise, revise the culvert design and repeat the anlalysis.

Once the culvert design is complete with an acceptable value of *HW*, the outlet velocity should be calculated. If the outlet velocity is greater than the minimum erosive velocity, erosion or scour protection measures must be added at the outlet as required. Similarly, velocities at the culvert inlet must be evaluated, and erosion protection added upstream of the culvert inlet as required.

The actual design of a culvert structure may require numerous iterations. A designer may take advantage of other detailed design methods and procedures, such as listed by TxDOT [6].

Alternatively, a culvert analysis may be performed using nomographs from the FHWA [8] and / or culvert analysis software, such as the FHWA's *Culvert Analysis Program*, HY-8 [10].

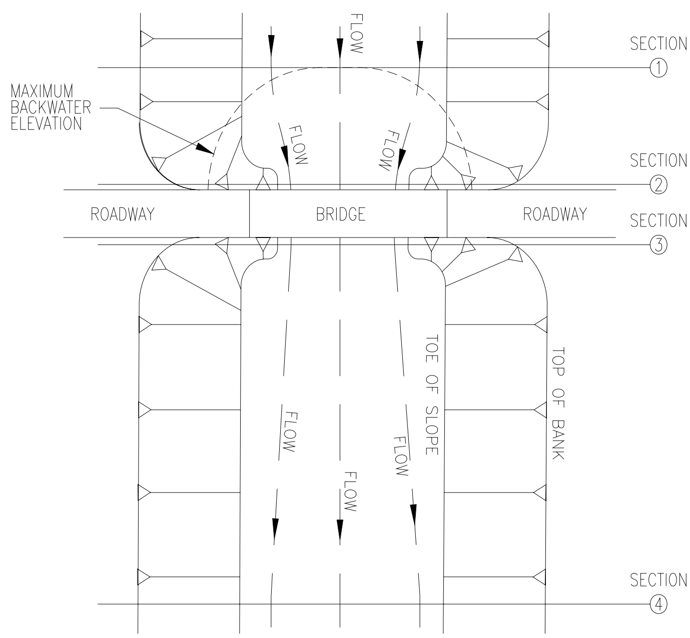
Bridge Analysis 5.2

Waterway bridges are structures that carry a primary infrastructure item (roadway, pipeline, pedestrian walkway, etc.) across a channel or chasm that is infeasible to cross by other means. The choice to span a given waterway with a culvert or with a bridge is usually based upon economic analysis, with bridge structures becoming more economically feasible as required span lengths and channel depths increase.

As discussed earlier in this Manual, bridge structures are normally designed with the low chord (the bottom surface of the bridge deck and supporting beams) to be above the water surface for the design flow and to provide a minimum one foot of freeboard. Obviously, if a bridge structure were to be constructed completely above the water surface with no portion of the structure in the channel flow, the bridge would have no effect on the channel flow and no backwater effects. However, for most bridge installations, the cost of spanning a given channel at an elevation completely above the flow for all reasonable storm events is prohibitive, and bridge structures include an extension of the roadway embankment into the channel to reduce the length of bridge and the corresponding structure costs. These extended embankments tend to force channel flow to contract and accelerate in order to pass through the reduced channel area, with a corresponding head loss through the structure. Additionally, bridge span lengths are normally limited in length based on cost, requiring intermediate supports that must be placed in the main portion of the channel flow. Obstruction effects of these interior bents also add to the accumulated head loss through the bridge structure.

The following discussion is based upon information developed through extensive studies of bridge flow patterns, velocities, head loss, and scour effects that have been performed by the Civil Engineering Section of Colorado State University and included in the FHWA publication: *Hydraulics of Bridge Waterways* [11].

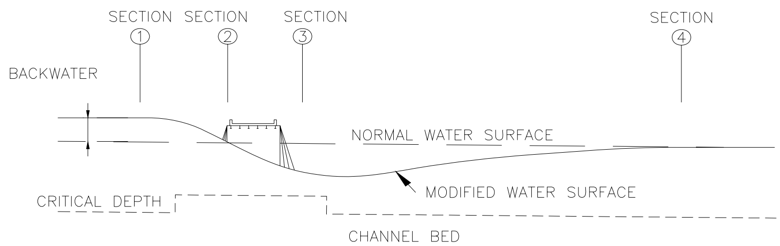
The illustration below shows a plan view of the typical flow pattern for a bridge crossing with extended abutments:



Typical Bridge Flow Pattern

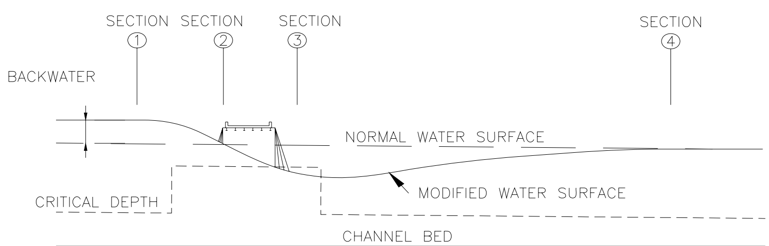
As shown above, the channel flow begins to contract at Section 1, and at Section 2, the flow has contracted sufficiently to pass through the narrowed opening beneath the bridge to Section 3. From Section 3 to Section 4, the flow expands to the normal flow pattern. Obviously, the flow must be accelerated to a higher velocity from Section 1 to Section 2 to pass the same flow volume through a smaller opening. The acceleration requires additional energy, which requires more head loss and a correspondingly higher water surface elevation upstream of the bridge than for conditions without the bridge.

There are three distinct flow conditions that may take place between Sections 1 and 3 [11, sec. 1.5]. First, the flow may remain subcritical throughout the entire region, with increased velocity (and slightly depressed water surface) beneath the bridge. This condition is referenced as Type I flow and is the type most commonly encountered. For Type I flow, the backwater upstream of the bridge can be determined by applying conservation of energy methods between Sections 1 and 4. Type I flow is illustrated below:



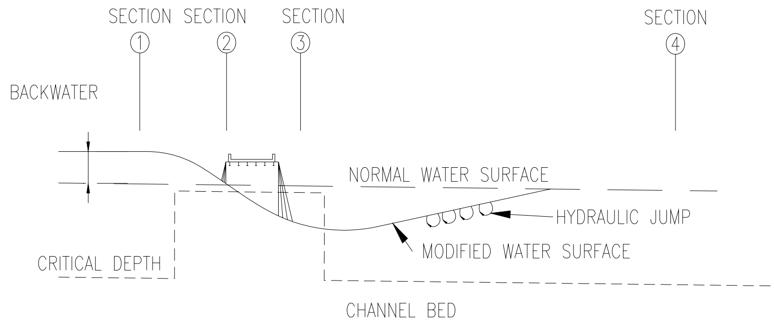
Type I Flow (subcritical)

The condition in which the flow beneath the bridge passes from above critical depth to below critical depth, and back again above critical depth is classified as Type II flow. For Type II flow, the water surface upstream of the bridge depends upon the critical depth beneath the bridge. Type II flow is further subdivided into Type IIA and Type IIB. Type IIA flow is illustrated below:



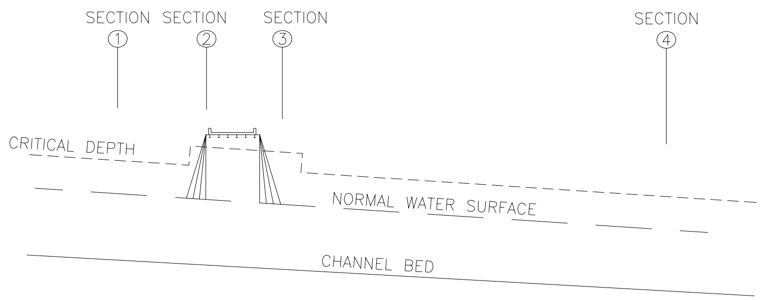
Type IIA Flow (passes through critical depth)

Type IIB flow is shown in the following illustration. As shown below, for Type IIB flow, there is a hydraulic jump a short distance downstream of the bridge.



Type IIB Flow (passes through critical depth)

Type III flow takes place when the flow depth is below critical depth throughout the entire bridge region. The water surface upstream of the bridge depends upon critical depth. This condition is encountered rarely, and is illustrated below:



Type III Flow (supercritical)

As described above, Type I flow is the most commonly encountered condition, and through extensive laboratory analysis, the following expression for computation of backwater for Type I flow has been developed in the laboratory.

Equation 5.9
[11, sec. 2.1]

$$h_1^* = K^* \times \alpha_2 \times \frac{v_{n2}^2}{2 \times 32.2} +$$

$$\alpha_1 \times \left[\left(\frac{A_{n2}}{A_4} \right)^2 - \left(\frac{A_{n2}}{A_1} \right)^2 \right] \times \frac{v_{n2}^2}{2 \times 32.2}$$

$$and \quad \alpha_1 = \frac{\sum (q \times v^2)}{Q \times V_r^2}$$

$$and \quad \alpha_2 = \frac{\sum (q \times v^2)}{Q \times V_c^2}$$

Where:
h_1^* = total backwater (ft)
K^*= total backwater coefficient
A_{n2} = gross water area in constriction measured below

normal stage
$v_{n2} = Q/A_{n2}$ *(ft / sec)*
v = velocity in a subsection
q = discharge in a subsection
V_r *= average velocity in the river*
V_c *= average velocity in the constriction*
Q = discharge in the river

The determination of the total backwater coefficient depends upon empirical relations to the number, size, shape, and orientation of piers, eccentricity of the bridge with respect to the channel cross section, and skew, and should be tabulated using the data in *Hydraulics of Bridge Waterways* [11]. The above equation is presented here primarily to demonstrate the controlling factors for bridge backwater; however, the equation was developed in the laboratory using flumes and models with consistent shapes and geometry. Therefore, use of this equation for field analysis should be limited to preliminary analysis only. **For final analysis of bridge backwater effects, a detailed backwater analysis will be required**.

Each of the flow types described above requires a different analysis technique. A great deal of physical data must be included for a proper analysis, including flow data for each of the required storm events, geometric data for the channel sections, and bridge geometry data. The water surface at Section 4 must be obtained accurately in order for any of the upstream analysis to be valid, and the determination of an accurate water surface at Section 4 must often be made through a complete channel analysis beginning at the nearest control point downstream of the bridge.

For proposed bridges, the required hydraulic analysis shall include a computer model of the channel and the bridge based upon the methods described in the Users' Manual for the current version of HEC-RAS [5].

The methods used in HEC-RAS [5] are patterned very closely to the recommendations in *Hydraulics of Bridge Waterways* [11], and the HEC-RAS [5] documentation contains all the technical data and discussion required to allow the user to complete a thorough analysis of the effects of any proposed bridge structure.

Flow Velocities 5.3

As discussed previously, culverts are normally designed to take advantage of increased head upstream of the culvert to increase quantity of flow through the culvert, resulting in a smaller and more hydraulically efficient structure. However, this increased capacity exists because the through-culvert velocity is increased over that of the channel outside the culvert. For small bridges, the bridge may be designed with sloping abutments that constrict the flow and also increase the water velocity below the bridge. The complete design of any cross drainage structure must include measures to reduce the velocity of flow downstream of the structure to a velocity that will not erode or scour the channel bottom or banks. If the velocity in the channel is greater than an erosive velocity, the cross drainage structure must include preventive measures that protect the channel bed, channel banks, and roadway embankment from erosion and scour within the limits of the roadway right of way.

Bridge scour is a process of accelerated erosion caused by the installation of obstructions to flow in channels that increase velocities in the locality of the installed structure. Culverts have solid bottoms and are, therefore, inherently protected from the effects of scour, and generally need no further analysis. Bridge structures, on the other hand, are highly susceptible to scour near piers and abutments. For any bridge with a foundation bedded on solid rock and independent of overburden effects of the erodible soil above the rock, the potential effects of scour are negligible, and no further analysis is required.

For all bridges not bedded on solid rock as noted above, a complete scour analysis is required, based upon the flows developed for the 100-year design storm. The analysis shall follow the recommendations and methodology described by the FHWA in *Evaluating Scour at Bridges* [12]. If the bridge backwater analysis described earlier was performed using HEC-RAS [5], the bridge scour analysis can be completed very simply by using the same HEC-RAS [5] model. See the HEC-RAS *Users' Manual* [5] for details.

Using the results of the scour envelope determined by the 100-year analysis, the bridge must either have a foundation that is designed to be completely stable under the expected scour conditions, or adequate scour protection measures must be installed to protect the bridge, abutments, and roadway from the effects of the expected scour. These countermeasures shall be designed and installed as recommended by FHWA in HEC-23 [13].

DIVISION 6 – THE RATIONAL METHOD

Underlying Assumptions 6.1

The following underlying assumptions form the theoretical basis for the Rational Method:

- The rate of runoff resulting from constant rainfall intensity is assumed to be a maximum when the duration of rainfall equals the time of concentration. That is, if the rainfall intensity is constant, the entire drainage area contributes to the peak discharge when the time of concentration has elapsed. As the size of the drainage area increases, this assumption becomes less valid. For large drainage areas, the time of concentration can be so large that the assumption of constant rainfall intensities for such long periods is not valid, and shorter more intense rainfalls can produce larger peak flows. Additionally, rainfall intensities usually vary during a storm. In semi-arid and arid regions, storm cells are relatively small with extreme intensity variations.
- The frequency of peak discharge is assumed to be the same as the frequency of the rainfall intensity for the given time of concentration. For small watersheds with a large percentage of impervious area, this assumption usually holds true. However, for larger watersheds, the response characteristics of the watershed and drainage system dominate other factors. In watersheds with little or no impervious area, antecedent moisture conditions tend to dominate other response factors.
- The rainfall intensity is assumed to be uniformly distributed over the entire drainage area. Actually, rainfall intensity varies spatially and temporally during a storm. As the size of the drainage area increases, the likelihood increases that rainfall intensity will vary significantly over space and time.
- The fraction of rainfall that becomes runoff is assumed to be independent of rainfall intensity or volume. This assumption is more often valid for impervious areas than for other areas.

By limiting the use of the Rational Method to 200 acres or less, the above assumptions are more likely to prove reasonable.

Applicability 6.2

The Rational Method is largely limited to calculation of peak flow values. Calculations of runoff volumes are not directly addressed by the Rational Method; therefore, design items such as detention ponds and water quality storage facilities often must be completed by other means of analysis. When the design or analysis of a proposed or existing system requires storm water storage features of any kind, the Rational Method should not be used.

Time of Concentration 6.3

The time of concentration, t_c, is the time measured from beginning of rainfall to the time at which flow from all portions of the watershed are contributing to flow at the point of interest. Typically, this is the time required for a drop of water that falls on the most hydraulically remote portion of the watershed to flow overland to the point of interest. This drop of water is the last to reach the point of interest; therefore, flow from all other portions of the watershed will have reached the point of interest and will be contributing to the flow at the point of interest. The flow at the point of interest is then the accumulated sum of all the flows. The flow at this time (t_c) is considered to be the maximum outflow from the watershed.

Several hydrologic methods (particularly the Rational Method) require the use of the time of concentration for each design point within the drainage basin. The duration of rainfall is then set equal to the time of concentration and is used to estimate the design average rainfall intensity (i). There may be multiple flow paths to consider in calculating t_c, and each path must be analyzed separately (using trial and error procedures) in order to determine the flow path with the greatest travel time and the corresponding time required for flow to traverse that path.

For purposes of calculating t_c, it is convenient to divide flow through a watershed into three separate elements:

- Sheet flow
- Shallow concentrated flow
- Channel or storm sewer flow

Sheet Flow: Sheet flow is shallow flow over land surfaces, which usually occurs in the headwaters of streams. The designer should realize that sheet flow occurs for only very short distances in urbanized conditions. Urbanized areas are assumed to have sheet flow of 300 feet or less. The following equation has been developed for sheet flow of less than 300 feet:

$$t_{sf} = L \times n \div (42 \times s^{0.5})$$

Equation 6.1
[3, pg. 2-7]

Where:
t_{sf} = travel time for sheet flow in minutes
L = length of the reach in feet
n = Manning's n (see Exhibit A-7.c)
s = slope of the ground surface in feet per foot

Shallow Concentrated Flow: After a maximum of 300 feet, sheet flow becomes shallow concentrated flow. The travel time for Shallow Concentrated Flow can be computed as follows:

$$t_{cf} = L \times n \div (60 \times s^{0.5})$$

Equation 6.2
[3, pg. 2-7]

Where:
t_{cf} = travel time for shallow concentrated flow in minutes
L = length of the reach in feet

n = Manning's n (see Exhibit A-7.c)
s = slope of the ground surface in feet per foot

The value of Manning's *n* represents a friction factor in conveyance equations, and has been developed by experimental means. The values of *n* to be used in Equations 6.1 and 6.2 above will be substantially higher than the values used in calculations of open channel flow, since friction between runoff surfaces and flowing water has a much more pronounced effect on shallow flow than for flow in relatively deep channels. The values of *n* listed in Exhibit A-7.c in Appendix A should be used in Equations 6.1 and 6.2.

Equations 6.1 and 6.2, above, were developed by the City of Austin for calculations in relatively small urbanized watersheds, and are appropriate for use with the Rational Method as described in this Manual.

Channel or Storm Sewer Flow: As soon as runoff flow begins to collect, that is, to be forced into a narrower flow pattern, the concentration and depth of flow is normally sufficient to warrant analysis as Channel or Storm Sewer Flow. For very poorly defined or shallow channels, it may be necessary to assume that the channel is triangular in cross section with side slopes that are the same as the overland slopes of the land planes that intersect to form the channel. The velocity in an open channel or storm drain can be determined by using Manning's equation:

$$V = (1.49 \times r^{2/3} \times s^{1/2}) / n$$

Equation 6.3
[14, pg. 99]

Where:
V = velocity of flow in feet per second
r = hydraulic radius = area / wetter perimeter
s = slope of the hydraulic grade line (normally assumed to be equal to channel bed slope)
n = Manning's n for open channel flow (See Exhibit A-7 in Appendix A for appropriate values)

The value of Manning's *n* used in Equation 6.3 is the appropriate friction factor for open channel flow and is substantially different from the value used in Equations 6.1 and 6.2. Exhibit A-7.d is a tabulation of commonly used values of *n* for regular channels and conduits; these values are most appropriate for man-made channels that are fairly uniform and regular. Exhibit A-7.e contains a listing of values that may be used to compute an overall friction factor for channels with irregular characteristics.

The travel time through the open channel or storm sewer reach can then be calculated as follows;

$$t_{ss} = L \div (V \times 60)$$

Equation 6.4

Where:
t_{ss} = travel time for channel or storm sewer in minutes
L = length of the reach in feet
V = velocity in feet per second

The Total Time for each individual flow path to be analyzed consists of the sum of the time required for each of the three flow elements above. Therefore, the time of concentration for a given watershed is the time required to travel the flow path which has the greatest total travel time calculated as follows:

$$t_c = t_{sf} + t_{cf} + t_{ss}$$ *Equation 6.5*

Where:
t_c = time of concentration in minutes
t_{sf} , t_{cf} , t_{ss} as defined previously

Since the quantity t_c normally appears as a part of the divisor in runoff calculations, the calculated value of intensity or runoff will approach infinity as t_c approaches zero. Therefore, if the value of t_c as calculated above is less than 5 minutes, **a minimum value of 5 minutes should be used for t_c** to avoid unreasonably large values of calculated runoff.

Runoff Calculations 6.4

The essence of the Rational Method is that storm water runoff is proportional to the area of land surface that receives a specified rainfall.

Flow from any given area of land is calculated as a product of the runoff coefficient, the rainfall intensity, and the land area, as follows:

$$Q = C \times i \times a$$ *Equation 6.6* [3, pg. 2-5]

Where:
Q = rate of runoff in cubic feet per second
C = runoff coefficient
i = intensity of rainfall in inches per hour
a = land area producing runoff (as shown on the Drainage Area Map)

Runoff Coefficient: The value of *C* in Equation 6.6 represents the fraction of rainfall that is converted to excess runoff from rainfall as it strikes any particular area. In the use of the Rational Method, different types of land areas are represented by different runoff coefficients to account for different runoff rates from variations in impervious cover. For instance, a *C* value of 0.95 for an area with asphaltic surface indicates that 95% of the rainfall striking the surface is converted into excess runoff.

Exhibit A-2, in Appendix A, contains values of runoff coefficients to be used in the Rational Method as described in this Manual. As shown in Exhibit A-2, the values of *C* tend to increase for storms of longer return periods. This increase in *C* is appropriate for larger storms because as rainfall amounts increase, the underlying soil can become saturated with a corresponding decrease in infiltration. Additionally, depression storage in the watershed becomes filled, decreasing storm water storage in the watershed, with corresponding increases in runoff.

Rainfall Intensity: The second variable in Equation 6.6 is the intensity of rainfall in inches per hour (*i*). Statistical studies have shown that rainfall events with very high intensities tend to last for very short periods of time; in other words, rainfall intensity tends to vary inversely with time. Based upon statistical studies, the following equation has been developed for rainfall intensity:

$$i = k \div (t_c + b)^d$$

Equation 6.7
[3, pg. 3-12]

Where:
i = intensity of rainfall in inches per hour
t_c = time of concentration (as previously defined)
k, b, d = intensity-duration-frequency coefficients (see Exhibit A-1 in Appendix A)

Generally, the intensity equation above for the Marble Falls area is more accurate for storm durations of 60 minutes or less than for longer durations. If the time of concentration calculated for the watershed being analyzed is greater than three hours, a different hydrologic method should be used.

DIVISION 7 – NRCS RUNOFF CURVE NUMBER METHODS

Introduction 7.1

The Natural Resource Conservation Service (NRCS), formerly known as the Soil Conservation Service (SCS) developed the runoff curve number method as a means of estimating the amount of rainfall that is converted to runoff. The procedures presented in this Division are presented in detail by the NRCS in *TR-55* [4].

The NRCS techniques require basic data similar to that used in the Rational Method. However, the NRCS approach is more sophisticated in that it considers the following:

- Total volume of rainfall and runoff.
- Initial rainfall losses to interception and depression storage.
- Infiltration rates that decrease during the course of the storm.

NRCS methods produce the direct runoff for a storm by subtracting infiltration and other losses from the total rainfall using a method called the Runoff Curve Number Method.

The primary input variables for the NRCS methods are as follows:

- Drainage area in square miles (*a*).
- Time of concentration (t_c) in **hours**
- Weighted runoff curve number (*CN*).
- Rainfall distribution (NRCS Type II).
- Total design rainfall (P) in inches.

NRCS Curves and Runoff 7.2

NRCS Rainfall-Runoff Equation: The NRCS curve numbers are an index representing the combined hydrologic effect of soil type, land use, agricultural land treatment class, hydrologic conditions, and antecedent soil moisture. Use of the NRCS curve numbers allows direct conversion of inches of rainfall to an equivalent number of inches of water that are actually converted to runoff.

Through the use of experimental data, the NRCS (formerly SCS) developed the following equations:

$$Q = (P - I_a)^2 \div [(P - I_a) + S]$$

Equation 7.1
[4, pg. 2-1]

Where:
Q = accumulated runoff in inches
P = accumulated rainfall in inches
I_a = initial abstraction, including surface storage, interception, and infiltration prior to runoff; in inches
S = potential maximum retention in inches

$$S = (1000 \div CN) - 10$$

Equation 7.2
[4, pg. 2-1]

Where:
S = potential maximum retention in inches
CN = NRCS runoff curve number

$$I_a = 0.2 \times S$$

Equation 7.3
[4, pg. 2-1]

Where:
I_a = initial abstraction, including surface storage, interception, and infiltration prior to runoff; in inches
S = potential maximum retention in inches

Substituting quantities

$$Q = (P - 0.2 \times S)^2 \div (P + 0.8 \times S)$$

Equation 7.4
[4, pg. 2-1]

Where:
Q = inches of water that can actually produce runoff

As can be seen in the equations above, the depth of water retained in the watershed, *S*, can be calculated as a function of the NRCS curve number, *CN*. Using the calculated value of *S*, the value of *Q*, which represents the inches of water that will actually be converted to runoff, can be calculated. Exhibit A-13 in Appendix A presents a direct graphical solution for *Q*, based upon *P*.

The curve numbers are index numbers that have been derived experimentally and account for the combined hydrologic effect of soil type, land use, agricultural land treatment class, hydrologic condition, and Antecedent Moisture Condition.

Hydrologic Soil Group: Soils are classified into four Hydrologic Soil Groups (HSGs): Group A, Group B, Group C, and Group D. The classifications are based upon minimum infiltration rates, which have been obtained through tests on bare soil after prolonged wetting. The most readily available and useful soil classification guides are included in soil surveys that have been compiled and published by NRCS [15] and are available on the internet. The NRCS [15] site contains a PDF file with a full tutorial on use of the facilities. Paper copies of local soils data may be available at local NRCS offices.

Antecedent Soil Moisture Condition: Antecedent Moisture Condition refers to the condition of the soil in the watershed prior to the beginning of rainfall. Antecedent Moisture Condition I represents a dry soil condition, Antecedent Moisture Condition III represents a wet soil condition, while Antecedent Moisture Condition II is normally considered average. For analyses using NRCS methods in the geographic area addressed by this Manual, **Antecedent Moisture Condition II should be assumed.**

The tables in Exhibits A-13 through A-16 in Appendix A include listings of NRCS curve numbers for typical land use and soil conditions. These curve number listings are based upon Antecedent Moisture Condition II and should be used for geographic areas addressed by this Manual.

Composite Curve Numbers: An impervious area is considered to be connected if runoff from it flows directly into the drainage system. It is also considered connected if runoff from it occurs as concentrated shallow flow that runs over a pervious area and then into the drainage system.

If all of the impervious area is directly connected to the drainage system, but the impervious area percentages or the pervious land use assumptions in Exhibit A-14 are not applicable, then read the *CN* for the pervious area from the tables and use Exhibit A-18 to compute a composite *CN*.

For a watershed in which runoff from unconnected impervious areas is spread over pervious areas as sheet flow, a composite *CN* must be computed as follows:

- If the impervious area is less than 30%, then use Exhibit A-19 to calculate a composite *CN*. Enter the right half of Exhibit A-19 with the percentage of total impervious area and the ratio of total unconnected impervious area to total impervious area. Then move left to the appropriate pervious *CN* and read down to find the composite *CN*.
- If the impervious area is greater than 30%, then read the *CN* for the pervious area from the tables and use Exhibit A-18 to calculate a composite *CN*.

Exhibit A-20 in Appendix A is a flow chart detailing the decision process for computation of composite curve numbers.

When the weighted *CN* is less than 40, a hydrologic method other than NRCS curve numbers should be applied.

NRCS Peak Flow Calculations 7.3

The value *Q*, is the depth of water, in inches, that will actually produce runoff from the watershed. In order to determine the rate at which the runoff, Q, will actually flow off the watershed, additional calculations are required. The NRCS methodology includes a solution for calculation of the peak runoff rate, q_p as follows:

- Using the curve number, CN, for the watershed, calculate the initial abstraction I_a from the equations below:

$$S = (1000 \div CN) - 10$$ *Equation 7.2* [4, pg. 2-1]

$$I_a = 0.2 \times S$$ *Equation 7.3* [4, pg. 2-1]

- Alternatively, the initial abstraction, I_a, can be read as a function of *CN* directly from Exhibit A-21 in Appendix A.

- Determine the ratio, R_a, of initial abstraction, I_a, to total rainfall, P:

$$R_a = I_a \div P$$

Equation 7.5
[4, pg. 4-1]

Where:
R_a = ratio of initial abstraction to total precipitation
I_a = initial abstraction, including surface storage, interception, and infiltration prior to runoff; in inches
P = total precipitation from design storm

- If the value of R_a is less than 0.1, then use R_a = 0.1; if the value of R_a is greater than 0.5, then use R_a = 0.5
- Calculate the value of q_u using the following equation:

$$\log(q_u) = C_0 + C_1 \times \log(T_c) + C_2 \times [\log(T_c)]^{\wedge 2}$$

Equation 7.6
[16]

Where:
q_u = runoff rate per square mile per inch; in cubic feet per second
C_0, C_1, C_2 = constants from Exhibit A-22 in Appendix A
T_c = time of concentration in hours

- The value q_u, represents the rate of runoff in cubic feet per second per square mile per inch of rainfall. As an alternative to Equation 7.6 above, q_u can be obtained graphically from Exhibit A-23 in Appendix a. Using the chart in Exhibit A-23, enter the X axis of the chart with the value of the time of concentration, T_c, in hours; move upward to intersect the appropriate curve, based upon the previously calculated value of R_a; move left and read the value of q_u.

- To calculate actual peak runoff rate, q_p, in cubic feet per second, apply Equation 7.7, below:

$$q_p = q_u \times a \times Q \times F$$

Equation 7.7
[4, pg. 4-1]

Where:
q_p = ratio of initial abstraction to total precipitation
q_u = runoff rate per square mile per inch; in cubic feet per second
a = drainage area in square miles
Q = direct runoff in inches
F = ponding or swamp factor from Exhibit A-24 in Appendix A (normally F = 1 for Marble Falls)

The value of q_p as calculated above represents the peak rate of runoff from the watershed for the chosen design storm. Although several computation steps are involved in the calculation of q_p, the calculations are well suited for entry into computer spreadsheets, and provide a ready means of comparing the expected results in changes in runoff from numerous development alternatives for a selected site.

NRCS Hydrograph Development 7.4

HEC-HMS Required: The methodology described in *TR-55* [4] includes means for calculating unit hydrographs and triangular hydrographs using Manual methods or spreadsheet calculations. However, for any but the most simple watershed analysis, the compilation of hydrographs becomes lengthy and tedious. Additionally, there are currently several high-quality software packages available that use NRCS methods for hydrograph compilation, combination, and routing. Therefore, for the geographic area addressed by this Manual, any analysis requiring the production of multiple or combined hydrographs or flood routing, including detention ponds, **shall be computer generated using the current version of HEC-HMS [5] or an equivalent approved software package**. The designer should obtain approval from the City of Marble Falls for use of other software packages before beginning any analysis required by this Manual.

Data Entry into HEC-HMS: For use of the HEC-HMS system, the user should consult the *User's Manual* [5] and *Technical Reference Manual* [5], both of which are downloaded and installed with the software.

HEC-HMS [5] allows the use of NRCS [4] standard rainfall distribution types as discussed previously in this Manual. To use NRCS Type II rainfall distribution as discussed in this Manual, the HMS user should enter the following data:

- Under the Subbasin Tab for each subbasin, select **SCS Curve Number** for the loss method, and **SCS Unit Hydrograph** as the transform method.
- Under the Loss Tab, the User can input the initial abstraction; however, if the value is left blank, HMS will calculate the value as demonstrated previously in this Manual. Unless the User has performed detailed calculations to determine the value of the initial abstraction, this value should be left blank.
- Also under the Loss Tab, the Curve Number should be entered. This value is the NRCS Curve Number and should be calculated as previously demonstrated in this Manual.
- There is also an option to enter % impervious cover under the Loss Tab. Any value in this field will be used by HMS to calculate runoff without infiltration. If the Curve Number is a composite curve number that has been calculated by including impervious area as described in this Manual, the user should leave the % impervious field blank. However, if the Curve Number that is entered represents only the pervious portion of the watershed, then the User may enter the percentage of the watershed that has unconnected impervious cover in lieu of calculating a composite Curve Number.
- Under the Meteorology Model Tab for each Meteorological Model, the User should choose **SCS Storm** for the precipitation type.
- Under the Precipitation Tab for the SCS Storm for each Meteorological Model the precipitation method should be set to **Type 2** (corresponding to Type II).
- Under the same tab, the precipitation depth in inches should be entered. This value is the total accumulated value for each of the respective Design Storm as tabulated in Exhibit A-3.
- Each Control Specification must be slightly longer than 24 hours in

duration to allow application of the full 24-hour SCS Storm.

- The data from Exhibit A-3 can be entered as cumulative, not incremental, values in a rainfall-runoff model. The total amount of precipitation associated with each storm is the cumulative precipitation value for 24 hours, for example, the total precipitation for the 10-year Design Storm is: 6.50 inches.
- The Transform Tab under each subbasin requires entry of a value for lag time. This value is normally taken as 60% of the previously calculated Time of Concentration (t_c). It is important to note that t_c for several of the NRCS methods is input in hours; however, the value for lag time in HMS should be entered in minutes.
- The SCS unit hydrograph method was originally developed using lag time as the length of time between the centroid of precipitation mass and the peak flow rate of the resulting hydrograph. Once an initial model run has been completed, the input value for Time of Concentration should be adjusted to insure that the actual lag time conforms to the actual model output time between the centroid of precipitation and the time of peak of the hydrograph. Multiple iterations should be performed, if required.

See the HEC-HMS [5] *User's Manual* for version 6.1.0, Chapters 6 and 7 for additional details regarding data entry for NRCS (SCS) Curve Number methods.

Peak flow values provided by HMS or other software packages will likely be less than peak flows calculated by Equation 7.7. The software packages perform calculations of peak flow and provide results for specified time increments. If the software time increment does not exactly match the time of peak flow, then the calculated peak values will differ. This discrepancy can be mitigated somewhat by using the smallest time increments possible in the software. However, even if peak flows do not agree, the actual volume of runoff (or depth of runoff) should be in very good agreement.

DIVISION 8 – DETENTION

Hydrologic Operation of Ponds 8.1

General Principles: In instances where existing or proposed flows are too great to be accommodated by the existing or proposed drainage systems as discussed in other Divisions of this Manual, detention ponds or systems may be required to reduce flows to an acceptable level. Detention ponds function by storing storm water flow in a reservoir and allowing the stored water to be released at a controlled rate. The outflow structure from the pond is designed and sized to maintain the outflow rate at a level that is at or below the flow rate set by the criteria described in this Manual.

Since a detention pond is normally intended to function during extreme storm events, the system must be designed to function with gravity flow. The use of pumps to move storm water into the proposed pond should be totally avoided. Such pumped systems may be susceptible to power outages or mechanical failures during the exact time when operation of the pond is critical.

The amount of flow reduction accomplished by a detention pond is dependent upon the amount of storm water flow (volume) stored by the pond. The analysis of the actual performance of a proposed detention pond is based upon the relationship between the amount of flow entering the pond (inflow), the amount of flow leaving the pond (outflow), and the amount of storm water (volume) stored in the pond. When the inflow into the pond is greater than the outflow, the relationship can be described mathematically as follows:

inflow – outflow = volume *Equation 8.1*
Where:
inflow (cubic feet per second)
outflow (cubic feet per second)
volume = the volume of water that must be stored (cubic feet per second).

Typically, the inflow rate varies with time, based upon the runoff from the design storm event, and the outflow rate varies with the water surface elevation inside the pond, which in turn depends upon the volume of water stored in the pond at any given instant. The total storage volume may be determined by subtracting the inflow from the outflow in very short time increments, then adding the incremental volumes to obtain the total stored volume.

Conversely, the performance of an existing or proposed pond can be analyzed by determining the outflow at any given point in time (incremental outflow), then subtracting the incremental storage volume from the incremental inflow. This process is known as **routing**, and requires a substantial number of calculations for inflow (as a function of the design storm runoff), outflow (based upon the water surface in the pond at a given instant) and storage (based upon inflow minus outflow).

Software Analysis 8.2

Given the tedious and voluminous nature of the routing calculations required for detention pond design and / or analysis, all detention pond routing calculations **must be performed using currently available software packages, such as HEC-HMS**. Other software packages may be acceptable, at the discretion of the City of Marble Falls.

In order to make use of detention pond design and analysis software, the designer must determine the basic relationships that control the performance of the pond:

- The type of the outflow structure(s) must be determined and the controlling flow coefficients (such as weir overflow coefficients or orifice coefficients) must be determined.

- The relationship between the volume of water stored in the pond and the elevation of the surface of the stored water must be determined. This is typically produced in table form by calculating the volume of water stored in the pond at incremental elevations (not to exceed one-foot intervals).

- The overflow coefficients for the spillway must be determined (usually consisting of overflow coefficients for a weir).

The relationships above are all dependent upon the types of outflow and spillway structures and the size, depth, and shape of the pond, all chosen by the designer. Generally accepted methods of determining the required relationships are discussed in following Divisions.

Once the appropriate data is properly entered, the software will accept (or calculate) an upstream hydrograph, route the hydrograph through the proposed pond, and produce a downstream hydrograph, with peak flows reduced by the detention pond. In order for the hydrologic design of any given pond to be accepted, the peak flow for the downstream hydrograph must be at or below the maximum allowed peak flow specified by this Manual.

Outflow Structures 8.3

Using the criteria for maximum flows as described in this Manual, the designer must determine the maximum allowable outflow from the proposed pond. The outflow structure should then be designed and sized to allow flows up to and including the maximum allowable outflow from the pond. Typical outflow structures consist of overflow weirs, circular orifices, v-notch weirs, or combinations of these or other structures; in extreme cases, pumps may be used to empty detention systems.

Pumps: Although the use of pumps to move storm water into a detention pond is prohibited, the use of pumped outflow systems may be permitted, provided the following minimum conditions are met:

- First, the detention pond must be large enough to store all required storm water if the outfall pump fails during the inflow cycle.

- Second, the amount of pumped storage must be limited to no more than 50% of the required storage volume.

- Third, there must be gravity outflow for the stored volume above the pumped storage, along with a gravity emergency outfall.

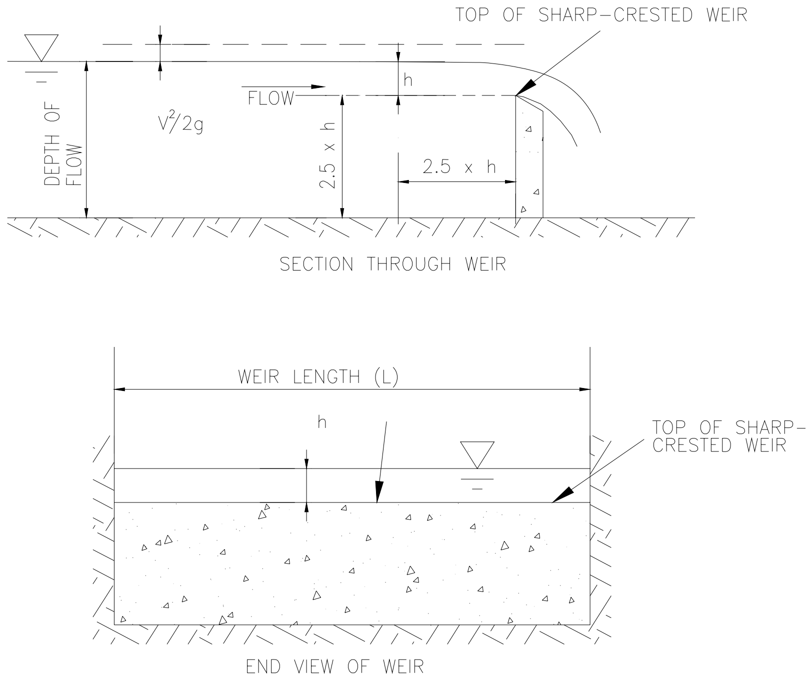
- Fourth, the proposed system must be emptied (both gravity flow and pumped portions of storage) within 48 hours of cessation of a storm event to ensure that the required storage capacity will be available for subsequent events.

- Fifth, the City of Marble Falls must be satisfied that the owner(s) of the pond will continue to maintain and operate the pump system for the lifetime of the pond.

The City of Marble Falls will review each proposed pumped detention system on a case by case basis. Specific design requirements will be determined for each case, along with any specific legal agreements that may be required before acceptance.

Weirs: Most detention pond designs include a low-flow outlet consisting of an underground pipe with a spillway or overflow consisting of a straight weir. There may also be portions of the outflow structure at intermediate elevations consisting of weirs, orifices, or any combination of these. Gravity flow through these types of structures is easily analyzed through the use of simple equations at control points. The outflow is also dependent upon the head above the control structure, which is in turn controlled by the volume of water in the pond.

An overflow weir can consist of simple or complex shapes as required for a given design. A straight overflow weir is illustrated below:



<u>Straight Overflow Weir</u>

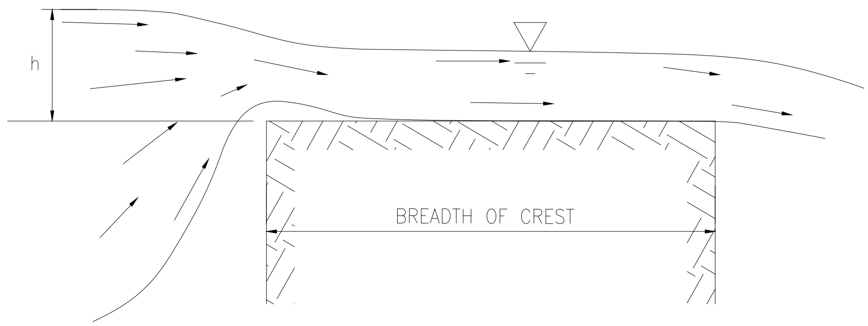
For weir flow assumptions to be valid, the height of the weir above the channel bottom must be at least 2.5 x the flow depth (*h*). The value of flow depth (*h*) must be measured at a point 2.5 x *h* upstream of the weir itself. Both conditions are shown in the illustration above. For a straight overflow weir, the flow over the top of the weir can be described as follows:

$$Q = c \times L \times h^{3/2}$$

Equation 8.2
[17 pg. 5-23]

Where:
Q = flow over the weir (cubic feet per second)
c = weir coefficient (at the accuracy required for detention pond design, c may be taken as 3.0)
L = length of overflow weir
h = head at the weir (equal to the depth from the upstream water surface to the weir surface, in feet), measured at a point 2.5 x h upstream of the weir

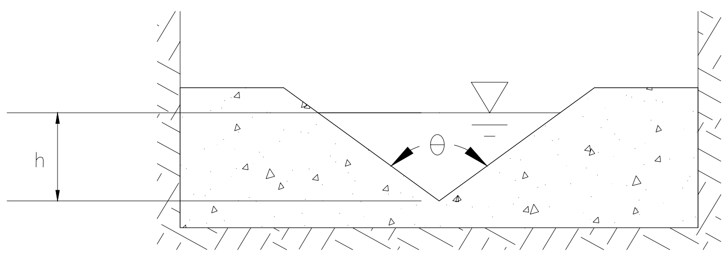
The weir illustrated above is a sharp-crested weir, which has a top edge in contact with the flow that is very thin or knife edged in relation to the depth of flow. A broad-crested weir is illustrated below, showing a very broad crest in relation to the depth of flow.



Broad-crested Weir

For practical design purposes, all overflow weirs are broad-crested to some extent. At the accuracy required for detention pond design, the weir coefficient, *c*, may be taken as: 3.0 when the breadth of the weir crest is less than 2 times the head, *h*, of the weir. For other configurations, the values of *c* listed in Exhibit A-25 should be used [17 chap. 5].

A v-notch weir, illustrated below, consists of a v-shaped notch in a vertical wall, with the angle of the notch sized as needed to provide outflow control.



V-notch Weir

For a v-notch weir, the overflow may be calculated as shown below:

$$Q = c_v \times \tan\left(\frac{\theta}{2}\right) \times h^{5/2}$$

Equation 8.3
[17 pg. 5-4]

Where:
Q = flow over the weir (cubic feet per second)
c_v = weir coefficient (at the accuracy required for detention pond design, c_v may be taken as 2.5)
h = distance from top of the water surface to the bottom of the notch; measured at a point 2.5 x h upstream of the weir.

The above weir flow equations and coefficients are greatly simplified. The designer may choose to employ more sophisticated methods of determining weir flow representations for a more efficient design. These methods will be acceptable, provided they are fully justified by current research and data.

Orifices: Low-flow structures consisting of short sections of underground pipe normally function as orifices, or as culverts flowing under inlet control. Other sections of the outflow structure may include plates with multiple openings, all of which may be designed as orifices. For calculation of orifice flow, the following equation may be applied:

$$Q = c_o \times a \times \sqrt{2 \times 32.2 \times h_o}$$

Equation 8.4
[17 pg. 4-3]

Where:
Q = flow through the orifice (cubic feet per second)
c_o = orifice coefficient
a = area of orifice opening
h_o = head above the orifice, measured from center of opening to water surface (feet).

For orifice flow, the value of the orifice coefficient depends upon the condition of the entrance and the length of the flow path downstream of the orifice. If the outflow consists of a long pipe, the accumulated downstream head loss in the pipe may force the structure to operate as a culvert under outlet control, requiring an appropriate culvert analysis. However, for short pipe sections, the orifice assumption is valid. For sections of pipe shorter than the pipe diameter, or for plates with holes where the plate material is thinner than the diameter of the circular opening, the value of c_o should be taken as: 0.6. If the entrance to the orifice is significantly rounded, the value of c_o may be taken as: 0.9. See [17 pg. 4-32] for tabulated orifice coefficients.

For orifice structures with a free outfall, the value of h_o should be taken as the vertical distance from the water surface in the pond to the center of the orifice opening. For submerged orifices discharging into a tail water pool, the value of h_o should be the difference in elevation between the water surface in the pond and the tail water elevation.

For multiple orifice openings, the flow for each orifice should be determined independently based upon each individual orifice coefficient and the head for each individual orifice.

Storage Volume 8.4

Storage Volume: As described in the preceding discussion of outflow structures, the outflow rate of a proposed detention pond is dependent upon the height of the surface of the water that is stored in the pond, which is dependent upon the volume of water stored in the pond. The relationship between the elevation of the water surface in the pond and the volume of water stored at that elevation must be determined in order to properly analyze the performance of the pond.

One of the simplest methods of tabulating the storage-volume relationship for a detention pond is to use the average-contour method as follows:

- Once the shape and depth of the pond have been determined, place elevation contours along the inside surface of the pond. The smaller the contour interval, the more accurate the volume calculations will be.

- Calculate or digitize the surface area of each contour line.

- Calculate the incremental volume between each sequential pair of contours by adding the area of the two contours, dividing by two, and multiplying by the elevation difference between the two contours.

- Cumulatively sum the incremental volumes between each pair of contours to determine the total volume stored at the elevation of each contour.

- The resulting storage-volume table can then be entered directly into most software systems for detention pond analysis.

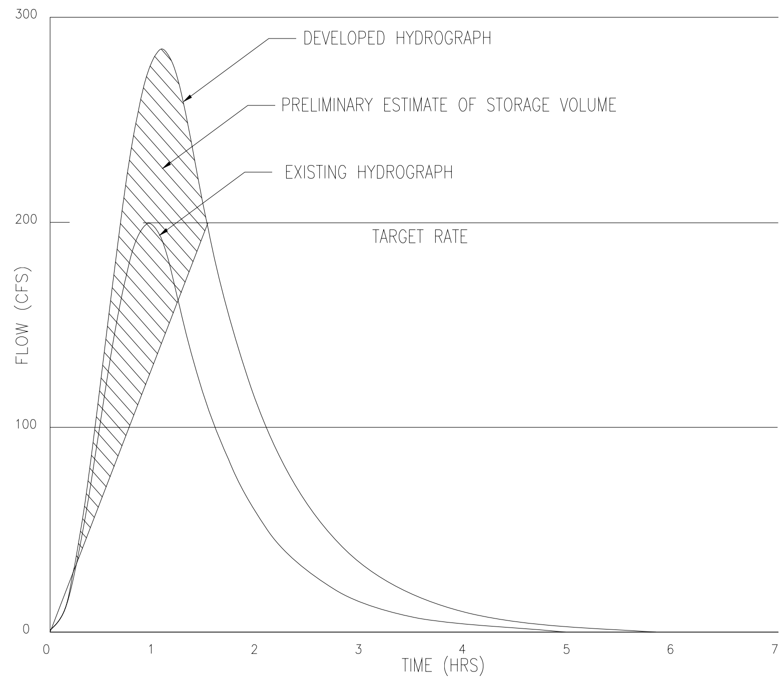
The total volume as computed above will also reflect the amount of excavation required for construction of a sunken pond.

Hydrograph Modification 8.5

In order to properly evaluate the performance of a detention pond, an inflow hydrograph must be developed and routed through the pond. Hydrograph production for proposed ponds in the Marble Falls area must be developed from a detailed hydrologic analysis that includes all contributing drainage areas. The hydrologic analysis and hydrograph production must be performed **using currently available software packages, such as HEC-HMS [5]**. Other software packages may be acceptable, at the discretion of the City of Marble Falls.

A hydrograph is simply a plot of the Quantity of flow (Q) on the vertical axis vs. time (t) on the horizontal axis. Once an accurate hydrograph has been developed, the total volume of flow can be determined by calculating the area beneath the hydrograph curve.

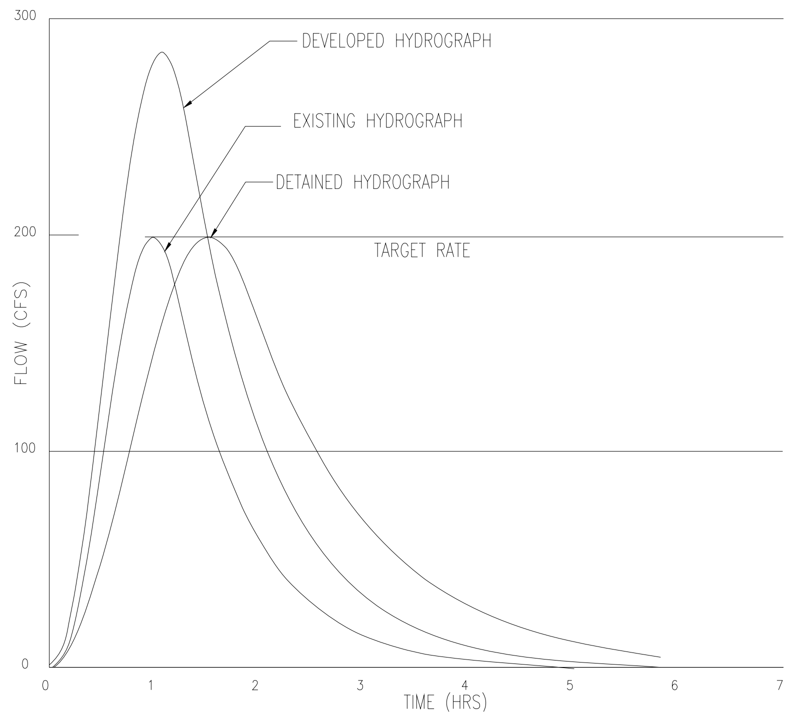
Typically, the required function for a proposed detention pond is to reduce the outflow from a proposed improvement or development to the outflow that existed prior to the improvement or development. The illustration below shows typical hydrographs for existing and developed conditions.



Existing and Developed Hydrographs

As illustrated above, the hydrograph for the proposed development peaks at a higher flow rate than for the existing condition. The detention pond must store sufficient storm water volume to reduce the peak outflow to the value of the peak flow of the existing hydrograph, which is labeled as: “Target Rate” in the illustration above. A preliminary estimate of the required storage volume may be made by drawing a reference line from the origin of the hydrograph plot to the point on the receding limb of the proposed hydrograph. The area between the reference line and the proposed hydrograph provides a good first estimate of the storage volume required, as shown above. This volume is only a preliminary estimate, and the actual volume and performance of the pond must be confirmed by detailed routing calculations. Several design iterations, using different outflow structures and pond sizes and shapes, may be required to adequately determine the required volume.

For proposed development projects with improved drainage and increased impervious cover, the peak outflow rates are typically higher, and the total volume of runoff is also greater. Even though a detention pond may reduce the peak outflow rate from a developed area, the total volume of storm water runoff is not decreased. The following illustration shows a typical hydrograph from a proposed development superimposed on the outflow hydrograph from a detention pond.



Developed and Detained Hydrographs

As shown above, the outflow hydrograph from the detention pond has an outflow rate that is less than the target rate as required; however, the outflow takes place over a longer period of time in order to release the entire stored volume.

Physical Design 8.6

Controlled Outflow: Normal detention ponds collect flow from an upstream source and control the outflow through an engineered structure. However, the outflow from the pond is normally concentrated and may be discharged at velocities that produce erosion. For all detention ponds, the erosion potential from the pond outflow must be eliminated. One or more of the following methods will provide an acceptable solution:

- The outflow from the pond must be leveled and spread to return the flow to a sheet flow condition with velocities below erosive velocities before the flow leaves the property or easement upon which the pond is constructed.
- The outflow path from the pond must be completely protected from erosion using methods described under the Division of this Manual on Open Channel Design.
- The pond must outfall into a public drainage channel, with erosion protection installed as required at the outfall point.

Perched Ponds: Detention ponds are normally constructed by excavating below natural ground to provide storage volume as required. However, it is also possible to construct a detention facility on a slope by adding embankment to the lower section of the slope to create an embanked catchment that holds the required volume. Such raised systems store storm water above the existing ground level, and are known as: "perched ponds." For perched ponds, several requirements must be adequately addressed in design:

- The embankment must be designed using acceptable engineering practices to be stable and safe against sliding, overturning, collapse, and dam breach.

- The perched pond cannot impede existing sheet flow from any neighboring or adjoining property.

- The base of the embankment must be protected from any erosion potential from sheet flow that has been redirected by addition of the embankment.

Protection of Slopes: Slopes (both inside and outside of ponds) must be designed to meet the criteria specified for open channels in other Divisions of this Manual. The side slopes of detention ponds (both inside and outside the ponds) must be permanently protected from erosion to prevent reduction of the structural stability of the pond and prevent silt accumulation in the bottom of the pond that would effectively reduce the storage capacity.

Safety Fencing: Detention ponds and their associated slopes must have adequate safety measures installed, such as protective fencing. For detention ponds and facilities that are privately owned and maintained, the owner must be fully responsible for all safety issues throughout the life of the facility.

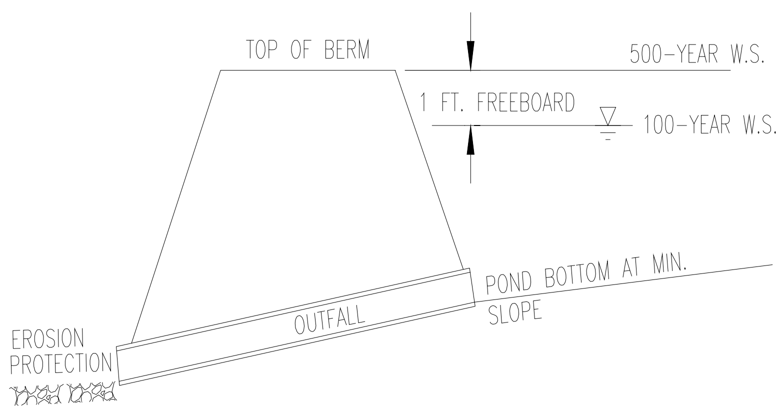
For all facilities that are to be taken over by the City of Marble Falls, adequate safety fencing must be in place. Dual use facilities without safety fencing will be reviewed and accepted by the City of Marble Falls on a case by case basis.

Maintenance: The owner or developer shall be responsible for continuing maintenance of any detention ponds and systems to insure that the entire system continues to function according to the original design parameters. This includes mowing, vegetation establishment and management, slope repair, silt removal, and any other tasks required to maintain proper operation, ensure public safety, and mitigate any nuisance issues for the lifetime of the system. Grass-lined slopes must be periodically mowed, and encroaching undergrowth must be controlled. Excessive silt and debris must be removed from the pond as required to maintain the required storage capacity, and to ensure free and unobstructed operation of the outflow structure and overflow spillway.

For all proposed detention ponds, the owner or developer must submit a maintenance plan and obtain approval from the City of Marble Falls for the proposed maintenance plan. The maintenance plan must address the following issues:

- A schedule for maintenance activities.
- Provision for access by City personnel.
- Name and contact information for the part(ies) responsible for maintenance.
- The plan must be signed and dated by the part(ies) responsible for maintenance.
- The plan must include provisions for adequate access for all equipment and personnel that may be required for maintenance and inspection activities.

Freeboard: Regardless of the design flow used to size a given detention pond, all ponds must be capable of passing the flow from the 100-year event. For ponds with a water surface perched above the elevation of adjoining natural ground, the pond must pass the 100-year flow while maintaining a freeboard (distance from the water surface to the top of the pond berm or embankment) of at least one foot, while the 500-year flow may not exceed the top of the pond slope or embankment. For purposes of this requirement, the 500-year flow may be estimated as 1.7 *x* the 100-year flow. Detention pond freeboard requirements are illustrated below:

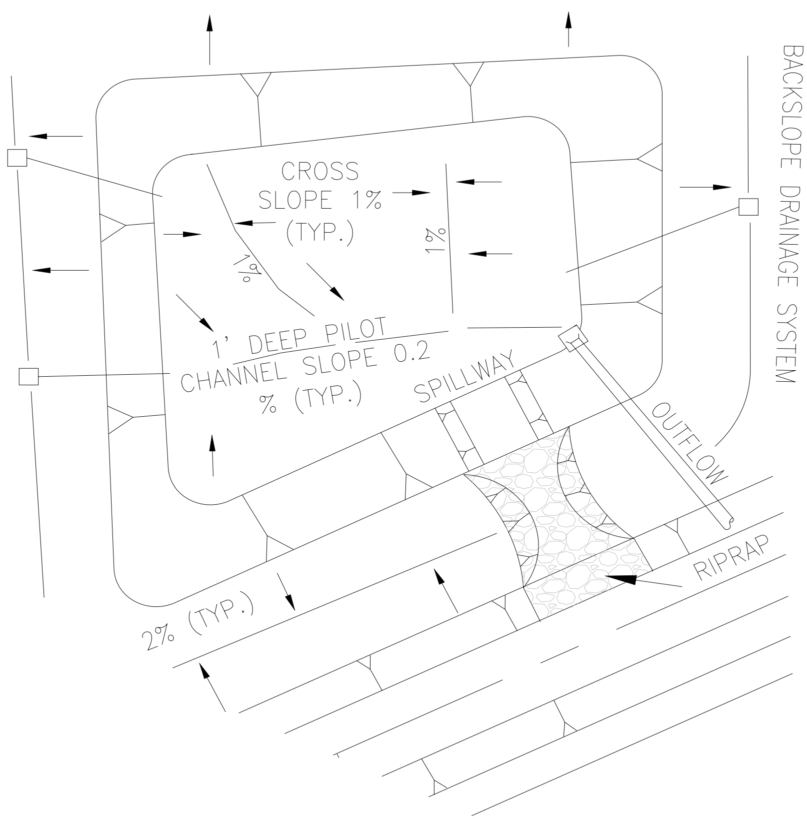


Freeboard Requirements for Detention Ponds

For detention systems that are completely excavated, there is no freeboard requirement. For detention systems that are constructed as in-line systems (an in-line system is essentially a dam across a drainage channel, with a properly sized low-flow outlet and spillway), the freeboard requirement is the same as the freeboard requirement for the channel in which the system is constructed.

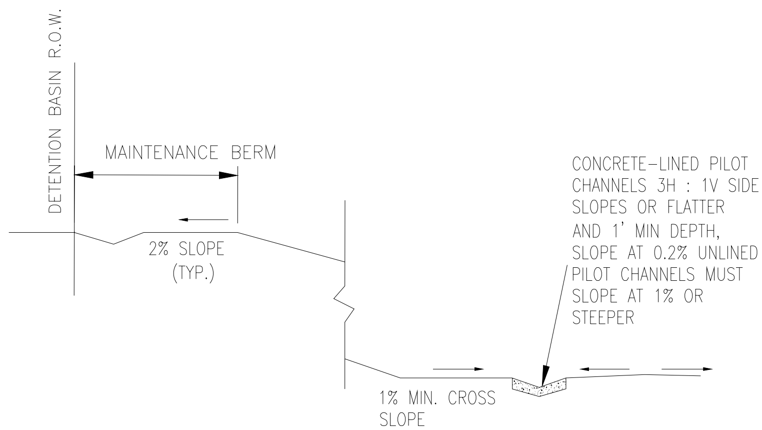
At the discretion of the City of Marble Falls, the freeboard requirements may be completely or partially waived, provided that the proposed pond design includes adequate armoring and erosion protection for the berm, slopes, and all areas downstream of the proposed pond.

Bottom Slope: Detention ponds may be designed as dry-bottom ponds or wet-bottom ponds. For a dry-bottom pond, the lowest elevation of the outflow structure must be set to allow all water to drain out of the pond during normal conditions. The bottom of the pond surface must be constructed to slope no flatter than 1%. However, if concrete-lined pilot channels are installed to intercept the flow, these concrete-lined pilot channels may be sloped as flat as 0.2% toward the outflow structure. The illustration below is a cross section of a typical dry bottom pond.



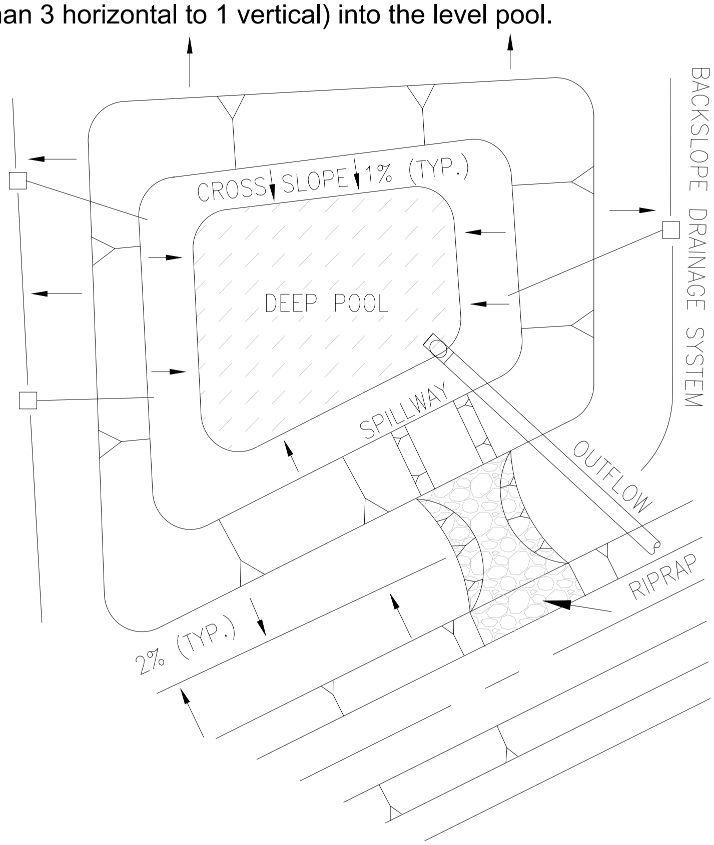
Dry-Bottom Detention Pond Layout

The illustration below is a typical cross section of a dry-bottom pond showing bottom slope requirements.



Dry-Bottom Detention Pond Cross Section

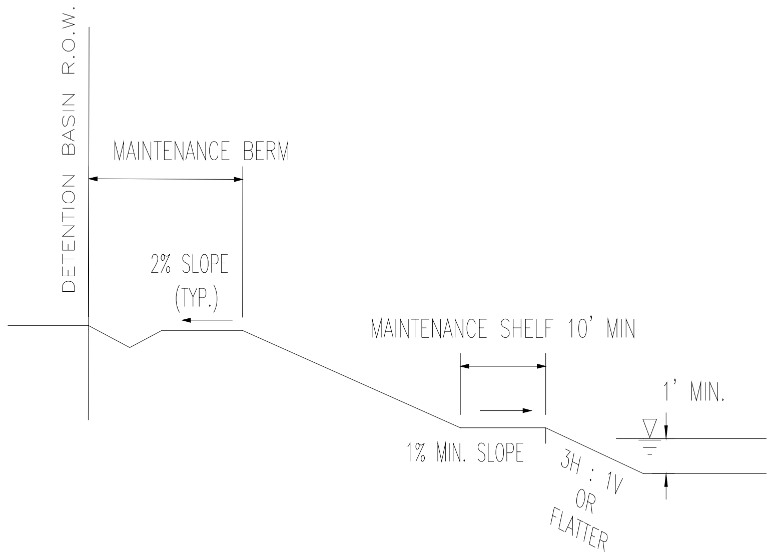
A wet-bottom pond is designed to maintain a level pool of water below the outflow structure. For wet-bottom ponds, the bottom of the excavation must be at least one foot below the elevation of the level pool. Immediately above the level pool, there must be a bench sloped no steeper than 1%, and a safety slope (no steeper than 3 horizontal to 1 vertical) into the level pool.



Wet-Bottom Detention Pond Layout

A cross section of a typical wet-bottom pond is shown in the illustration above.

The illustration below includes a typical cross section for a wet-bottom detention pond.



Wet-Bottom Detention Pond Cross Section

The storage volume available in a wet-bottom pond does not include the volume below the elevation of the water surface in the deep pool. Although more excavation is required for construction of a wet-bottom pond, the actual amount of available storage volume may be greater than for a dry-bottom pond because the required bottom slope of the dry-bottom pond may significantly impact the amount of volume available.

No Adverse Impact: Detention ponds may be installed as in-line structures in existing drainage channels, which are essentially dams across the channel with an outflow structure at the base of the dam. Ponds may be installed off line away from existing channels and discharge into the drainage system, or ponds may be installed as side-weir systems parallel to existing channels. Ponds may also be installed as perched ponds as described above.

In all cases, the installed detention ponds or systems must be designed and installed to not obstruct or impede existing flow or drainage patterns, or to produce adverse backwater effects. As installed, no pond will be allowed to produce any adverse impacts of any kind to the watershed or adjoining properties. All flow changes, impediments, obstruction, or backwater effects from any installed pond must be completely confined to the property (or easement) upon which the pond is located.

Additional Regulatory Authorities: All detention systems must fully comply with current regulations of other regulatory authorities including, but not limited to:

- The United States Army Corps of Engineers
- The Federal Emergency Management Agency
- The Texas Commission on Environmental Qualilty
- The Lower Colorado River Authority

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APPENDIX A

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EXHIBIT A-1 (sheet 1 of 4)

Total annual rainfall amounts (in inches), and the intensity of rainfall (inches of rainfall per hour) vary significantly between different localities within Texas, with both values generally being lower in areas further from the coastline of the Gulf of Mexico. The design methodology used for a local area should be based on actual local conditions as much as possible. Otherwise, the adoption of methods developed for other localities might result in unnecessarily conservative analyses and conclusions. To that end, a rainfall intensity equation and associated coefficients have been developed for the City of Marble Falls Texas as described below.

Table 1, below, contains rainfall depths for the 1, 2, 5, 10, 25, 50, and 100-year storm events tabulated at intervals of 5, 10, 15, 30, 60, 120, and 180 minutes.

Table 1
Marble Falls, Texas
Depth Duration Frequency

Return (yr)	**Rainfall Depth (inches) for Storm Duration (minutes)**						
	5 min	**10 min**	**15 min**	**30 min**	**60 min**	**120 min**	**180 min**
1	0.43	0.72	0.93	1.27	1.62	1.80	1.98
2	**0.49**	0.84*	**1.07**	1.53*	**2.00**	2.22	2.46
5	0.58	0.98	1.27	1.88	2.51	2.98	3.31
10	0.65	1.10	1.41	2.14	2.89	3.63	3.88
25	0.74	1.25	1.60	2.48	3.39	4.16	4.53
50	0.81	1.36	1.75	2.74	3.77	4.63	5.11
100	**0.87**	1.48*	**1.89**	3.00*	**4.15**	5.19	6.06

Values in bold for 2-yr and 100-yr events for 5, 15, and 60 minute durations were taken from maps by Frederick, Meyers, and Auciello in *HYDRO-35*, published by the National Weather Service in 1977. Values followed by an asterisk (*) were calculated according to the following formula:

$$P_{10\min} = \left(0.41 \times P_{5\min}\right) + \left(0.59 \times P_{15\min}\right)$$
$$P_{30\min} = \left(0.51 \times P_{15\min}\right) + \left(0.49 \times P_{60\min}\right)$$

See: Chow, Ven Te, Maidment, David R., and Mays, Larry W. in *Applied Hydrology*, by McGraw-Hill, 1988, page 452.

Where:
$P_{5\ min}$, $P_{15\ min}$, and $P_{60\ min}$ are taken from maps by Frederick, Meyers, and Auciello for the 2 and 100-year storms.

The remaining depth values for the 5 min. through 60 min. durations were interpolated (extrapolated for the 1-yr event) column by column by assuming a linear relationship between the log of the duration vs depth.

EXHIBIT A-1 (sheet 2 of 4)

Depth values for the 120 and 180 minute durations were taken from maps in *Technical Paper No. 40*, published by the National Weather Bureau in 1963.

Table 2, below, contains values of average rainfall intensity in inches per hour for the 1, 2, 5, 10, 25, 50, and 100-year storms:

Table 2 Marble Falls, Texas Intensity Duration Frequency							
Return (yr)	Rainfall Intensity (inches per hour) for Storm Duration (minutes)						
	5 min	10 min	15 min	30 min	60 min	120 min	180 min
1	5.16	4.32	3.72	2.54	1.62	0.90	0.66
2	5.88	5.04	4.28	3.06	2.00	1.11	0.82
5	6.96	5.88	5.08	3.76	2.51	1.49	1.10
10	7.80	6.60	5.64	4.28	2.89	1.81	1.29
25	8.88	7.50	6.40	4.96	3.39	2.08	1.51
50	9.72	8.16	7.00	5.48	3.77	2.31	1.70
100	10.44	8.88	7.56	6.00	4.15	2.59	2.02

The average intensity values in Table 2 were calculated by dividing each of the depth values in Table 1 by the time (hours) of the rainfall. As an example, for the 10-yr storm event after 30 minutes (1/2 hour) of rainfall, the average intensity is:

$$i_{10yr,\,30\min} = 2.14 \div (30 \div 60) = 4.28\ in.\ per\ hour$$

The chart below shows rainfall intensity from Table 2 plotted vs the duration of rainfall:

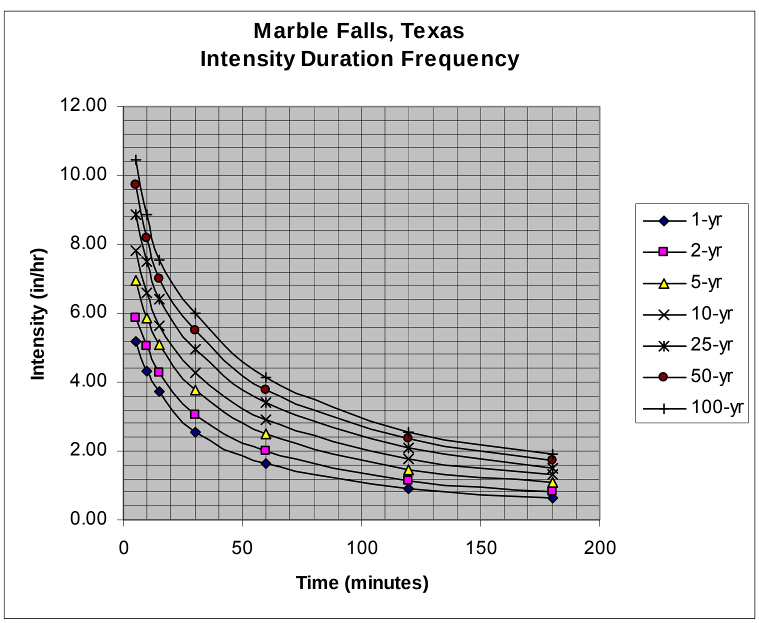


EXHIBIT A-1 (sheet 3 of 4)

Common practice is to reduce intensity-duration relationships to an intensity equation of the following form:

$$i = k \div (t_c + b)^d$$

Where:
i = intensity of rainfall in inches per hour
t_c = time of concentration in minutes
k, b, d = intensity-duration-frequency coefficients unique to each storm event.

For each storm event, multiple data points were used to create a system of equations of the form of the intensity equation above. These systems of equations were solved by computer iteration methods to determine a value of *k*, *b*, and *d* (i-d-f coefficients) for each storm event.

In order to evaluate the accuracy of the i-d-f equation and coefficients that were developed as described above, the intensity equation was used with the calculated coefficients to determine the rainfall intensity for each storm event and each duration. The calculated values of rainfall intensity were then subtracted from the intensity values in Table 2, with the resulting differences shown in Table 3, below:

Table 3							
Marble Falls, Texas							
Return (yr)	Difference Between Tabulated Intensity and Calculated Intensity for Storm Duration (minutes)						
	5 min	10 min	15 min	30 min	60 min	120 min	180 min
1	0.05	0.02	0.00	0.06	0.00	0.02	-0.01
2	0.02	-0.06	0.04	0.02	-0.04	0.03	-0.01
5	-0.02	0.00	0.04	-0.03	-0.04	0.03	0.01
10	-0.03	-0.02	0.10	-0.06	-0.05	-0.01	0.06
25	-0.04	0.01	0.17	-0.11	-0.09	0.03	0.08
50	-0.07	0.05	0.19	-0.14	-0.11	0.05	0.09
100	-0.05	-0.02	0.22	-0.16	-0.09	0.08	0.04

As shown in Table 3, above, the calculated intensity values are in good general agreement with the original data as tabulated from the rainfall maps. The largest differences are associated with the longer duration storms and greater return periods and are relatively insignificant; therefore, the use of the intensity equation and associated coefficients developed here are appropriate for use in the City of Marble Falls, Texas.

EXHIBIT A-1 (sheet 4 of 4)

The intensity equation and associated i-d-f coefficients to be used in the City of Marble Falls, Texas are tabulated below:

INTENSITY-DURATION-FREQUENCY COEFFICIENTS			
Storm Frequency	***k***	***b***	***d***
1-year	135.827	20.232	1.010
2-year	151.752	21.856	0.987
5-year	104.828	18.588	0.859
10-year	86.546	16.294	0.788
25-year	94.223	16.370	0.773
50-year	94.383	15.831	0.751
100-year	86.994	14.949	0.710
Developed for use in the following equation for the City of Marble Falls, Texas: $i = k \div (t_c + b)^d$ Where t_c = time of concentration in minutes, and *k*, *b*, *d* are from the table above.			

Analysis of the 1-year event is included here primarily for the purpose of providing i-d-f coefficients for use in water quality calculations required by various regulatory authorities in the Marble Falls area. These 1-year i-d-f coefficients will produce intensities consistent with tabulated synthetic storms included in documentation by the Lower Colorado River Authority for use in complying with that agency's water quality requirements.

For example, for the 1-year event and three hour rainfall:

$$i = k \div \left(t_c + b\right)^d \qquad \text{intensity}$$

$$i = 135.827 \div (180 + 20.232)^{1.010} = 0.643\, in/hr$$

$$depth = i \times t_c \qquad \text{total 3-hr depth}$$

$$depth = 0.643\, in/hr \times 3\, hrs = 1.93$$

Therefore, for water quality calculations for the 1-year, 3-hour event, use the i-d-f equations and constants shown here, with total rainfall depth of 1.93 inches to insure compatibility with requirements published by other agencies.

EXHIBIT A-2

RATIONAL METHOD RUNOFF COEFFICIENTS						
Character of Surface	Return Period					
	2 years	5 years	10 years	25 years	50 years	100 years
Developed						
Asphaltic	0.73	0.77	0.81	0.86	0.90	0.95
Concrete	0.75	0.80	0.83	0.88	0.92	0.97
Grass Areas (Lawns, Parks, etc.)						
<u>Poor Condition*</u>						
Flat, 0-2%	0.32	0.34	0.37	0.40	0.44	0.47
Average, 2-7%	0.37	0.40	0.43	0.46	0.49	0.53
Steep, over 7%	0.40	0.43	0.45	0.49	0.52	0.55
<u>Fair Condition**</u>						
Flat, 0-2%	0.25	0.28	0.30	0.34	0.37	0.41
Average, 2-7%	0.33	0.36	0.38	0.42	0.45	0.49
Steep, over 7%	0.37	0.40	0.42	0.46	0.49	0.53
<u>Good Condition***</u>						
Flat, 0-2%	0.21	0.23	0.25	0.29	0.32	0.36
Average, 2-7%	0.29	0.32	0.35	0.39	.042	0.46
Steep, over 7%	0.34	.037	0.40	0.44	0.47	0.51
Undeveloped						
<u>Cultivated</u>						
Flat, 0-2%	0.31	0.34	0.36	0.40	0.43	0.47
Average, 2-7%	0.35	0.38	0.41	0.44	0.48	0.51
Steep, over 7%	0.39	0.42	0.44	0.48	0.51	0.54
<u>Pasture/Range</u>						
Flat, 0-2%	0.25	0.28	0.30	0.34	0.37	0.41
Average, 2-7%	0.33	0.36	0.38	0.42	0.45	0.49
Steep, over 7%	0.37	0.40	0.42	0.46	0.49	0.53
<u>Forest/Woodlands</u>						
Flat, 0-2%	0.22	0.25	0.28	0.31	0.35	0.39
Average, 2-7%	0.31	0.34	0.36	0.40	0.43	0.47
Steep, over 7%	0.35	0.39	0.41	0.45	0.48	0.52
*	Grass cover less than 50 percent of area					
**	Grass cover on 50 to 75 percent of area					
***	Grass cover larger than 75 percent of area					
Sources: Rossmiller, R.L. *The Rational Formula Revisited* and the City of Austin Watershed Engineering Division, as presented in the *City of Austin Drainage Criteria Manual*, September 2001 Supplement						

EXHIBIT A-3

NRCS Type II Total Rainfall Amounts for Marble Falls, Texas					
2-yr	**5-yr**	**10-yr**	**25-yr**	**50-yr**	**100-yr**
4.00	5.40	6.50	7.60	8.60	9.70
Rainfall amounts tabulated for the Marble Falls, Texas area using data and methodology from Natural Resources Conservation Service, *Urban Hydrology for Small Watersheds, TR-55,* Appendix B.					

NRCS Type II Cumulative Rainfall Amounts for Marble Falls, Texas								
Time (hrs)	**NRCS Type II Rainfall Fraction**		**Cumulative Rainfall Amounts (inches)**					
			2-yr	**5-yr**	**10-yr**	**25-yr**	**50-yr**	**100-yr**
0.00		0.0000	0.00	0.00	0.00	0.00	0.00	0.00
1.00	*	0.0110	0.04	0.06	0.07	0.08	0.09	0.10
2.00		0.0220	0.09	0.12	0.14	0.17	0.19	0.21
3.00	*	0.0350	0.14	0.19	0.22	0.26	0.30	0.33
4.00		0.0480	0.19	0.26	0.31	0.36	0.41	0.47
5.00	*	0.0630	0.25	0.34	0.41	0.48	0.54	0.61
6.00		0.0800	0.32	0.43	0.52	0.61	0.69	0.78
7.00	*	0.0990	0.40	0.53	0.64	0.75	0.85	0.96
8.00		0.1200	0.48	0.65	0.78	0.91	1.03	1.16
8.50	*	0.1320	0.53	0.71	0.86	1.00	1.14	1.28
9.00		0.1470	0.59	0.79	0.96	1.12	1.26	1.43
9.50		0.1630	0.65	0.88	1.06	1.24	1.40	1.58
9.75	*	0.1720	0.69	0.93	1.11	1.30	1.47	1.66
10.00		0.1810	0.72	0.98	1.18	1.38	1.56	1.76
10.50		0.2040	0.82	1.10	1.33	1.55	1.75	1.98
11.00		0.2350	0.94	1.27	1.53	1.79	2.02	2.28
11.50		0.2830	1.13	1.53	1.84	2.15	2.43	2.75
11.75		0.3870	1.55	2.09	2.52	2.94	3.33	3.75
12.00		0.6630	2.65	3.58	4.31	5.04	5.70	6.43
12.50		0.7350	2.94	3.97	4.78	5.59	6.32	7.13
13.00		0.7720	3.09	4.17	5.02	5.87	6.64	7.49
13.50		0.7990	3.20	4.31	5.19	6.07	6.87	7.75
14.00		0.8200	3.28	4.43	5.33	6.23	7.05	7.95
15.00	*	0.8540	3.41	4.61	5.55	6.49	7.34	8.28
16.00		0.8800	3.52	4.75	5.72	6.69	7.57	8.54
17.00	*	0.9020	3.61	4.87	5.86	6.85	7.76	8.75
18.00	*	0.9210	3.68	4.97	5.99	7.00	7.92	8.93
19.00	*	0.9380	3.75	5.06	6.10	7.13	8.06	9.10
20.00		0.9520	3.81	5.14	6.19	7.24	8.19	9.23
21.00	*	0.9650	3.86	5.21	6.27	7.33	8.30	9.36
22.00	*	0.9770	3.91	5.28	6.35	7.43	8.40	9.48
23.00	*	0.9890	3.95	5.34	6.43	7.51	8.50	9.59
24.00		1.0000	4.00	5.40	6.50	7.60	8.60	9.70
Source: Fractional values from United States Department of Agriculture, Soil Conservation Service, *TP-149,* Revised April 1, 1973, page 3. (Values with asterisk are interpolated). Rainfall amounts calculated from total rainfall amounts for Marble Falls, Texas.								

EXHIBIT A-4

City of Austin (3) Hour Design Storm Distributions Cumulative Values (inches)						
Time (min)	**2-yr**	**5-yr**	**10-yr**	**25-yr**	**50-yr**	**100-yr**
5	0.01	0.03	0.03	0.04	0.05	0.06
10	0.03	0.05	0.07	0.09	0.11	0.13
15	0.04	0.08	0.11	0.14	0.17	0.19
20	0.06	0.11	0.15	0.19	0.23	0.27
25	0.08	0.15	0.19	0.25	0.30	0.34
30	0.10	0.18	0.24	0.31	0.37	0.43
35	0.12	0.22	0.29	0.38	0.44	0.52
40	0.15	0.27	0.35	0.45	0.53	0.61
45	0.17	0.32	0.41	0.53	0.62	0.72
50	0.21	0.37	0.48	0.62	0.73	0.84
55	0.25	0.44	0.56	0.72	0.84	0.98
60	0.30	0.51	0.65	0.84	0.98	1.13
65	0.36	0.60	0.76	0.98	1.14	1.31
70	0.43	0.71	0.90	1.15	1.33	1.53
75	0.54	0.86	1.07	1.36	1.57	1.80
80	0.69	1.06	1.31	1.65	1.90	2.17
85	0.94	1.39	1.67	2.19	2.40	2.72
90	1.48	2.03	2.39	3.01	3.31	3.71
95	1.84	2.47	2.89	3.53	3.96	4.43
100	2.03	2.72	3.18	3.88	4.35	4.87
105	2.16	2.89	3.38	4.13	4.63	5.18
110	2.24	3.02	3.53	4.32	4.85	5.43
115	2.31	3.12	3.65	4.47	5.03	5.63
120	2.36	3.20	3.75	4.60	5.17	5.79
125	2.41	3.27	3.84	4.71	5.30	5.94
130	2.44	3.33	3.91	4.80	5.41	6.06
135	2.47	3.38	3.98	4.89	5.51	6.17
140	2.50	3.43	4.04	4.96	5.60	6.28
145	2.52	3.47	4.09	5.03	5.68	6.37
150	2.55	3.51	4.14	5.10	5.75	6.46
155	2.56	3.54	4.19	5.16	5.82	6.54
160	2.58	3.57	4.23	5.21	5.89	6.61
165	2.60	3.60	4.27	5.26	5.95	6.68
170	2.61	3.63	4.30	5.31	6.00	6.75
175	2.63	3.66	4.34	5.36	6.06	6.81
180	2.64	3.68	4.37	5.40	6.11	6.87
Source:	City of Austin Watershed Engineering Division as presented in the *City of Austin Drainage Criteria Manual*, September 2001 Supplement					

EXHIBIT A-5

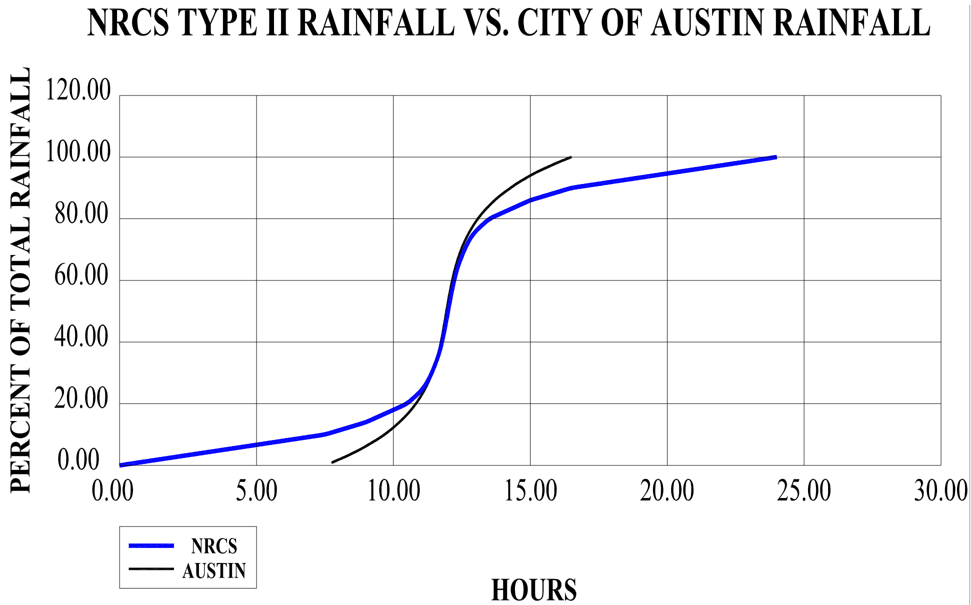


EXHIBIT A-6

Manhole Spacing	
Pipe Diameter (inches)	**Maximum Distance (feet)**
18 – 24	300
27 – 36	375
39-54	450
60 and above	900

EXHIBIT A-7.a

MANNNGS *n* FOR STREET AND PAVEMENT GUTTERS	
Mannings *n*	**Type of Gutter or Pavement**
0.012	Concrete gutter, troweled finish
	Asphalt Pavement:
0.013	Smooth texture
0.015	Rough texture
	Concrete gutter-asphalt pavement:
0.013	Smooth
0.015	Rough
	Concrete pavement:
0.014	Float finish
0.016	Broom finish
For gutters with small slope, where sediment may accumulate, increase above values of "n" by 0.02	
Source:	USDOT, FHWA, HEC-22, Urban Drainage Design Manual

EXHIBIT A-7.b

Table 4 - Manning's n Values for Culverts.*

Type of Culvert	Roughness or Corrugation	Manning's n
Concrete Pipe	Smooth	0.010-0.011
Concrete Boxes	Smooth	0.012-0.015
Spiral Rib Metal Pipe	Smooth	0.012-0.013
Corrugated Metal Pipe, Pipe Arch and box (Annular and Helical corrugations - see Figure B-3, Manning's n varies with barrel size)	2-2/3 by ½ in Annular	0.022-0.027
	2-2/3 by ½ in Helical	0.011-0.023
	6 by 1 in Helical	0.022-0.025
	5 by 1 in	0.025-0.028
	3 by 1 in	0.027-0.028
	6 by 2 in Structural Plate	0.033-0.035
	9 by 2-1/2 in Structural Plate	0.033-0.037
Corrugated Polyethylene	Smooth	0.009-0.015
Corrugated Polyethylene	Corrugated	0.018-0.025
Polyvinyl chloride (PVC)	Smooth	0.009-0.011

Source: FHWA, *Hydraulic Design of Highway Culverts*

EXHIBIT A-7.c

MANNNGS *n* FOR OVERLAND FLOW AND CONCENTRATED FLOW	
Mannings *n*	**Surface Type and Condition**
0.016	Concrete (rough of smoothed finish)
0.02	Asphalt
0.1	0-50% vegetated ground cover, remaining bare soil or rock outcrops, minimum brush or tree cover
0.2	50-90% vegetated ground cover, remaining bare soil or rock outcrops, minimum-medium brush or tree cover
0.3	100% vegetated ground cover, medium-dense grasses (lawns, grassy fields, etc.) medium brush or tree cover
0.6	100% vegetated ground cover with areas of heavy vegetation (parks, green-belts, riparian areas, etc.) dense under-growth with medium to heavy tree growth
Source:	City of Austin Watershed Engineering Division as presented in the *City of Austin Drainage Criteria Manual*, September 2001 Supplement

EXHIBIT A-7.d

For drainage conduits with relatively uniform hydraulic characteristics, the value of Manning's *n* may be taken directly from the table below:

TYPICAL VALUES FOR MANNING'S *n* for OPEN CHANNEL FLOW		
Channel		
	flow in channel	
	10,000 cfs or less	more than 10,000 cfs
Grass lined	0.040	0.035
Riprap lined	0.040	0.035
Articulated concrete block – grassed	0.040	0.035
Articulated concrete block – bare	0.030	
Concrete lined	0.015	
Natural or overgrown channel	0.050-0.080	
Overbanks		
With minor effective flow	0.080-0.150	
Ineffective flow (ponded areas, normally only addressed in hydraulic modeling)	0.99	
Conduit		
Concrete pipe	0.013	
Concrete box	0.015	
Corrugated metal pipe	0.024	
Source: Harris County Flood Control District; *Policy Criteria & Acceptance Manual,* October 2004, page 4-4		

EXHIBIT A-7.e

For open channels with varying hydraulic characteristics, Manning's *n* may be calculated from individual roughness factors follows:

$$n = (n_0 + n_1 + n_2 + n_3 + n_4) \times m_5$$

Where: n_0, n_1, n_2, n_3, n_4, *and* m_5 *are taken from the table below*

VALUES FOR COMPUTATION OF MANNING'S ROUGHNESS COEFFICIENT *n*			
Channel Conditions		Values	
Material involved	Earth	n_0	0.020
	Rock cut		0.025
	Fine gravel		0.024
	Coarse gravel		0.028
Degree of irregularity	Smooth	n_1	0.000
	Minor		0.005
	Moderate		0.010
	Severe		0.020
Variations of channel cross section	Gradual	n_2	0.000
	Alternating occasionally		0.005
	Alternating frequently		0.010-0.015
Relative effect of obstructions	Negligible	n_3	0.000
	Minor		0.010-0.015
	Appreciable		0.020-0.030
	Severe		0.040-0.060
Vegetation	Low	n_4	0.005-0.010
	Medium		0.010-0.025
	High		0.025-0.050
	Very high		0.050-0.100
Degree of meandering	Minor	m_5	1.000
	Appreciable		1.150
	Severe		1.300
Source: Chow, Ven Te, Ph.D., *Open Channel Hydraulics,* McGraw-Hill, Inc. 1988, page 109			

EXHIBIT A-7.f

Manning's Roughness Coefficients for Various Depth Ranges. **				
Lining Category	Lining Type	n-Value for Given Depth Ranges		
		(0-0.5 ft)	(0.5-2.0 ft)	>2.0 ft
Rigid	Concrete	0.015	0.013	0.013
	Grouted Riprap	0.040	0.030	0.028
	Stone Masonry	0.042	0.032	0.030
	Soil Element	0.025	0.022	0.020
	Asphalt	0.018	0.016	0.016
Unlined	Bare Soil	0.023	0.020	0.020
	Rock Cut	0.045	0.035	0.025
Temporary*	Woven Paper Net	0.016	0.015	0.015
	Jute Net	0.028	0.022	0.019
	Fiberglass Roving	0.028	0.021	0.019
	Straw with Net	0.065	0.033	0.025
	Curled Wood Mat	0.066	0.035	0.028
	Synthetic Mat	0.036	0.025	0.021
Gravel Riprap	25 mm (1 in) D_{50}	0.044	0.033	0.030
	50 mm (2 in) D_{50}	0.066	0.041	0.034
Rock Riprap	150 mm (6 in) D_{50}	0.104	0.069	0.035
	300 mm (12 in) D_{50}	--	0.078	0.040

Note: Values listed are representative values for the respective depth ranges. Manning's roughness coefficients, n, vary with the flow depth.

* Some "temporary" linings become permanent when buried.

** Table reproduced from FHWA's *Design of Roadside Channels with Flexible Linings*

EXHIBIT A-7.g

Classification of Vegetal Covers as to Degree of Retardance.**		
Retardance Class	Cover	Condition
A	Weeping lovegrass Yellow bluestem Ischaemum	Excellent stand, tall, average 0.76 m (2.5 ft) Excellent stand, tall, average 0.91 m (3.0 ft)
B	Bermuda grass Native grass mixture (Little bluestem, bluestem, blue gamma, and other long and short midwest grasses) Weeping lovegrass Lespedeza sericea Alfalfa Weeping lovegrass Blue gamma	Good stand, tall, average 0.30 m (1.0 ft) Good stand, unmowed Good stand, tall, average 0.61 m (2.0 ft) Good stand, not woody, tall, average 0.48 m (1.6 ft) Good stand, uncut, average 0.28 m (0.91ft) Good stand, unmowed, average 0.33 m (1.1 ft) Good stand, uncut, average 0.33 m (1.1 ft)
C	Crabgrass Bermuda grass Common lespedeza Grass-legume mixture-summer (orchid grass, redtop Italian ryegrass, and common lespedeza) Centipedegrass Kentucky bluegrass	Fair stand, uncut, avg. 0.25 to 1.20 m (0.8 to 4.0 ft) Good stand, mowed, average 0.15 m (0.5 ft) Good stand, uncut, average 0.28 m (0.91 ft) Good stand, uncut, average 0.15 m (0.20 ft) (0.5 to 1.5 ft) Very dense cover, average 0.15 m (0.5 ft) Good stand, headed, avg. 0.15 to 0.30 m (0.5 to 1.0 ft)
D	Bermuda grass Common lespedeza Buffalo grass Grass-legume mixture-fall, spring (orchid grass, redtop Italian ryegrass, and common lespedeza) Lespedeza sericea	Good stand, cut to 0.06 m (0.2 ft) Excellent stand, uncut, average 0.11 m (0.4 ft) Good stand, uncut, avg. .08 to 0.15 m (0.3 to 0.5 ft) Good stand, uncut, 0.10 to 0.13 m (0.3 to 0.4 ft) After cutting to 0.05 m (0.2 ft) height, very good stand before cutting
E	Bermuda grass Bermuda grass	Good stand, cut to average 0.04 m (0.1 ft) Burned stubble
Note: Covers classified have been tested in experimental channels. Covers were green and generally uniform. ** Table partially reproduced from FHWA's *Design of Roadside Channels with Flexible Linings*		

EXHIBIT A-7.h

Manning's n Relationships as a Function of Vegetal Degree of Retardance and Hydraulic Radius.	
Retardance Class	Manning's n Equation*
A	$n = R^{1/6} / \left[15.8 + 19.97 \log\left(R^{1.4} S_o^{\ 0.4}\right)\right]$
B	$n = R^{1/6} / \left[23.0 + 19.97 \log\left(R^{1.4} S_o^{\ 0.4}\right)\right]$
C	$n = R^{1/6} / \left[30.2 + 19.97 \log\left(R^{1.4} S_o^{\ 0.4}\right)\right]$
D	$n = R^{1/6} / \left[34.6 + 19.97 \log\left(R^{1.4} S_o^{\ 0.4}\right)\right]$
E	$n = R^{1/6} / \left[37.7 + 19.97 \log\left(R^{1.4} S_o^{\ 0.4}\right)\right]$
Where: *R = hydraulic radius* *S_0 = slope of energy grade line (normally channel bottom slope)*	
*Equations are valid for flows less than 50 ft^3/s. Nomograph solutions for these equations are contained in FHWA's *Design of Roadside Channels with Flexible Linings*.	

EXHIBIT A-7.i

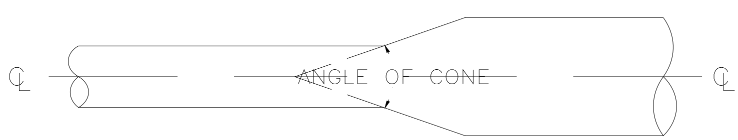
Manning's n Relationships as a Function of Vegetal Degree of Retardance and Shear Stress.	
Retardance Class	Manning's n Equation*
A	$n = 0.213 \times (0.605) \times \tau_0^{-0.4}$
B	$n = 0.213 \times (0.418) \times \tau_0^{-0.4}$
C	$n = 0.213 \times (0.220) \times \tau_0^{-0.4}$
D	$n = 0.213 \times (0.147) \times \tau_0^{-0.4}$
E	$n = 0.213 \times (0.093) \times \tau_0^{-0.4}$
Where: *τ_0 = mean boundary shear stress (lb/ft^2)*	
Source: FHWA, *Design of Roadside Channels with Flexible Linings*.	

EXHIBIT A-8

Splash-over Velocity Equation		$v_0 = k_0 + k_1 \times L_g + k_2 \times L_g{}^2 + k_3 \times L_g{}^3$ ***Where:*** ***L_g = length of grate*** ***k_0, k_1, k_2, k_3 from table below***			
Grate Configuration	**Typical Bar Spacing**	k_0	k_1	k_2	k_3
Parallel Bars	2	2.218	4.031	-0.649	0.056
Parallel Bars	1.2	1.762	3.117	-0.451	0.033
Transverse Curved Vanes	4.5	1.381	2.78	-0.300	0.020
Transverse 45^0 Tilted Vane	4	0.988	2.625	-0.359	0.029
Parallel Bars w/Transverse Rods	2 parallel 4 transverse	0.735	2.437	-0.265	0.018
Transverse 30^0 Tilted Vane	4	0.505	2.344	-0.200	0.014
Reticuline	n/a	0.030	2.278	-0.179	0.010
Source: Texas Department of Transportation: *Hydraulic Design Manual*					

EXHIBIT A-9.a

Values of K_e for determining head loss due to gradual enlargement of pipes in non-pressure flow							
D_2/D_1	Angle of Cone (degrees)						
	10	20	45	60	90	120	180
1.5	0.17	0.40	1.06	1.21	1.14	1.07	1.00
3	0.17	0.40	0.86	1.02	1.06	1.04	1.00



D_2/D_1 = ratio of diameter of larger pipe to smaller pipe
V_1 = velocity in smaller pipe (upstream of transition)
(American Society of Civil Engineers, 1992. Design and Construction of Urban Storm water Management Systems. *ASCE Manuals and Reports of Engineering Practice No. 77, WEF Manual of Practice FD-20*. New York, N.Y.

EXHIBIT A-9.b

Values of K_c for determining head loss due to sudden pipe contractions in pipes in non-pressure flow	
D_2/D_1	**K_c**
0.2	0.5
0.4	0.4
0.6	0.3
0.8	0.1
1.0	0.0

D_2/D_1 = ratio of diameter of larger pipe to smaller pipe
(American Society of Civil Engineers, 1992. Design and Construction of Urban Storm water Management Systems. *ASCE Manuals and Reports of Engineering Practice No. 77, WEF Manual of Practice FD-20*. New York, N.Y.

EXHIBIT A-9.c

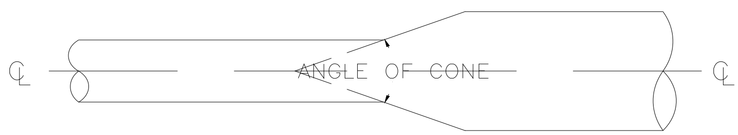
Values of K_e for determining head loss due to sudden enlargement of pipes

D_2/D_1	Velocity, V_1 in feet per second												
	2.0	3.0	4.0	5.0	6.0	7.0	8.0	10.0	12.0	15.0	20.0	30.0	40.0
1.2	0.11	0.10	0.10	0.10	0.10	0.10	0.10	0.09	0.09	0.09	0.09	0.09	0.08
1.4	0.26	0.26	0.25	0.24	0.24	0.24	0.24	0.23	0.23	0.22	0.22	0.21	0.20
1.6	0.40	0.39	0.38	0.37	0.37	0.36	0.36	035	0.35	0.34	0.33	0.32	0.32
1.8	0.51	0.49	0.48	0.47	0.47	0.46	0.46	0.45	0.44	0.43	0.42	0.41	0.40
2.0	0.60	0.58	0.56	0.55	0.55	0.54	0.53	0.52	0.52	0.51	0.50	0.48	0.47
2.5	0.74	0.72	0.70	0.69	0.68	0.67	0.66	0.65	0.64	0.63	0.62	0.60	0.58
3.0	0.83	0.80	0.78	0.77	0.76	0.75	0.74	0.73	0.72	0.70	0.69	0.67	0.65
4.0	0.92	0.89	0.87	0.85	0.84	0.83	0.82	0.80	0.79	0.78	0.76	0.74	0.72
5.0	0.96	0.93	0.91	0.89	0.88	0.87	0.86	0.84	0.83	0.82	0.80	0.77	0.75
10.0	1.00	0.99	0.96	0.95	0.93	0.92	0.91	0.89	0.88	0.86	0.84	0.82	0.80
∞	1.00	1.00	0.98	0.96	0.95	0.94	0.93	0.91	0.90	0.88	0.86	0.83	0.81

D_2/D_1 = ratio of diameter of larger pipe to smaller pipe
V_1 = velocity in smaller pipe (upstream of transition)
(American Society of Civil Engineers, 1992. Design and Construction of Urban Storm water Management Systems. *ASCE Manuals and Reports of Engineering Practice No. 77, WEF Manual of Practice FD-20*. New York, N.Y.

EXHIBIT A-9.d

Values of K_e for determining head loss due to gradual enlargement of pipes											
D_2/D_1	Angle of Cone (degrees)										
	2	6	10	15	20	25	30	35	40	50	60
1.1	0.01	0.01	0.03	0.05	0.10	0.13	0.16	0.18	0.19	0.21	0.23
1.2	0.02	0.02	0.04	0.09	0.16	0.21	0.25	0.29	0.31	0.35	0.37
1.4	0.02	0.03	0.06	0.12	0.23	0.30	0.36	0.41	0.44	0.50	0.53
1.6	0.03	0.04	0.07	0.14	0.26	0.35	0.42	0.47	0.51	0.57	0.61
1.8	0.03	0.04	0.07	0.15	0.28	0.37	0.44	0.50	0.54	0.61	0.65
2.0	0.03	0.04	0.07	0.16	0.29	0.38	0.46	0.52	0.56	0.63	0.68
2.5	0.03	0.04	0.08	0.16	0.30	0.39	0.48	0.54	0.58	0.65	0.70
3.0	0.03	0.04	0.08	0.16	0.31	0.40	0.48	0.55	0.59	0.66	0.71
∞	0.03	0.05	0.08	0.16	0.31	0.40	0.49	0.46	0.60	0.67	0.72



D_2/D_1 = ratio of diameter of larger pipe to smaller pipe
(American Society of Civil Engineers, 1992. Design and Construction of Urban Storm water Management Systems. *ASCE Manuals and Reports of Engineering Practice No. 77, WEF Manual of Practice FD-20*. New York, N.Y.

EXHIBIT A-9.e

Values of K_e for determining head loss due to sudden contraction in pressure flow pipes													
D_2/D_1	Velocity, V_1 in feet per second												
	2.0	3.0	4.0	5.0	6.0	7.0	8.0	10.0	12.0	15.0	20.0	30.0	40.0
1.1	0.03	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.05	0.05	0.06
1.2	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.08	0.08	0.08	0.09	0.10	0.11
1.4	0.17	0.17	0.17	0.17	0.17	0.17	0.17	0.18	0.18	0.18	0.18	0.19	0.20
1.6	0.26	0.26	0.26	0.26	0.26	0.26	0.26	0.26	0.26	0.26	0.25	0.25	0.24
1.8	0.34	0.34	0.34	0.34	0.34	0.34	0.33	0.33	0.32	0.32	0.32	0.29	0.27
2.0	0.38	0.38	0.37	0.37	0.37	0.37	0.36	0.36	0.35	0.34	0.33	0.31	0.29
2.2	0.40	0.40	0.40	0.39	0.39	0.39	0.39	0.38	0.37	0.37	0.35	0.33	0.30
2.5	0.42	0.42	0.42	0.41	0.41	0.41	0.40	0.40	0.39	0.38	0.37	0.34	0.31
3.0	0.44	0.44	0.44	0.43	0.43	0.43	0.42	0.42	0.41	0.40	0.39	0.36	0.33
4.0	0.47	0.46	0.46	0.46	0.45	0.45	0.45	0.44	0.43	0.42	0.41	0.37	0.34
5.0	0.48	0.48	0.47	0.47	0.47	0.46	0.46	0.45	0.45	0.44	0.42	0.38	0.35
10.0	0.49	0.48	0.48	0.48	0.48	0.47	0.47	0.46	0.46	0.45	0.43	0.40	0.36
∞	0.49	0.49	0.48	0.48	0.48	0.47	0.47	0.47	0.46	0.45	0.44	0.41	0.38

D_2/D_1 = ratio of diameter of larger pipe to smaller pipe
V_1 = velocity in smaller pipe (downstream of transition)
(American Society of Civil Engineers, 1992. Design and Construction of Urban Storm water Management Systems. *ASCE Manuals and Reports of Engineering Practice No. 77, WEF Manual of Practice FD-20*. New York, N.Y.

EXHIBIT A-10.a

Typical Permissible Shear Stresses for Bare Soil and Stone Linings		
Lining Category	Lining Type	Permissible Shear Stress lb/ft^2
Bare Soil Cohesive (PI = 10)	Clayey sands	0.037-0.095
	Inorganic silts	0.027-0.11
	Silty sands	0.024-0.072
Bare Soil Cohesive (PI $\geq$ 20)	Clayey sands	0.094
	Inorganic silts	0.083
	Silty sands	0.072
	Inorganic clays	0.14
Bare Soil Non-cohesive (PI < 10)	Finer than coarse sand D_{75}<1.3 mm (0.05 in)	0.02
	Fine gravel D_{75}=7.5 mm (0.3 in)	0.12
	Gravel D_{75}=15 mm (0.6 in)	0.24
Gravel Mulch	Coarse gravel D_{50}=25 mm (1 in)	0.4
	Very coarse gravel D_{50}=50 mm (2 in)	0.8
Reproduced from HEC-22, page 2-7		

EXHIBIT A-10.b

Permissible Shear Stresses for Lining Materials.		
Lining Category	Lining Type	Permissible Unit Shear Stress lb/ft^2
Temporary*	Woven Paper Net	0.15
	Jute Net	0.45
	Fiberglass Roving:	
	Single	0.60
	Double	0.85
	Straw with Net	1.45
	Curled Wood Mat	1.55
	Synthetic Mat	2.00
Vegetative	Class A	3.70
	Class B	2.10
	Class C	1.00
	Class D	0.60
	Class E	0.35
Gravel Riprap	25 mm (1in)	0.33
	50 mm (2 in)	0.67
Rock Riprap	150 mm (6 in)	2.00
	300 mm (12 in)	4.00
Reproduced from HEC-22, page 5-17		

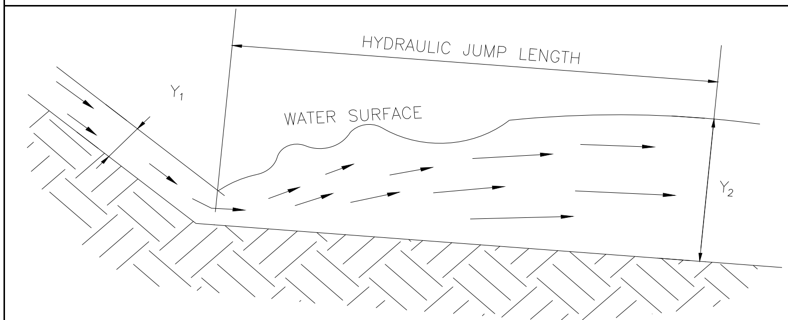
EXHIBIT A-10.c

<table>
<tr><th colspan="4">Classification of Vegetal Covers as to Degree of Retardance.**</th></tr>
<tr><th rowspan="2">Retardance Class</th><th rowspan="2">Cover</th><th rowspan="2">Condition</th><th>Permissible Unit Stress</th></tr>
<tr><th>lb/ft²</th></tr>
<tr><td rowspan="2">A</td><td>Weeping lovegrass</td><td>Excellent stand, tall, average 30 in</td><td rowspan="2">3.70</td></tr>
<tr><td>Yellow bluestem Ischaemum</td><td>Excellent stand, tall, average 36 in</td></tr>
<tr><td rowspan="7">B</td><td>Bermuda grass</td><td>Good stand, tall, average 12 in</td><td rowspan="7">2.10</td></tr>
<tr><td>Native grass mixture (Little bluestem, bluestem, blue gamma, and other long and short midwest grasses)</td><td>Good stand, unmowed</td></tr>
<tr><td>Weeping lovegrass</td><td>Good stand, tall, average 24 in</td></tr>
<tr><td>Lespedeza sericea</td><td>Good stand, not woody, tall, average 19 in</td></tr>
<tr><td>Alfalfa</td><td>Good stand, uncut, average 11 in</td></tr>
<tr><td>Weeping lovegrass</td><td>Good stand, unmowed, average 13 in</td></tr>
<tr><td>Blue gamma</td><td>Good stand, uncut, average 11 in</td></tr>
<tr><td rowspan="6">C</td><td>Crabgrass</td><td>Fair stand, uncut, avg. 10 to 48 in</td><td rowspan="6">1.00</td></tr>
<tr><td>Bermuda grass</td><td>Good stand, mowed, average 6 in</td></tr>
<tr><td>Common lespedeza</td><td>Good stand, uncut, average 11 in</td></tr>
<tr><td>Grass-legume mixture-Summer (orchid grass, redtop Italian ryegrass, and common lespedeza)</td><td>Good stand, uncut, average 6 to 8 in</td></tr>
<tr><td>Centipedegrass</td><td>Very dense cover, average 6 in</td></tr>
<tr><td>Kentucky bluegrass</td><td>Good stand, headed, avg. 6 to 12 in</td></tr>
<tr><td rowspan="5">D</td><td>Bermuda grass</td><td>Good stand, cut to 2.5 in</td><td rowspan="5">0.60</td></tr>
<tr><td>Common lespedeza</td><td>Excellent stand, uncut, average 4.5 in</td></tr>
<tr><td>Buffalo grass</td><td>Good stand, uncut, avg. 3 to 6 in</td></tr>
<tr><td>Grass-legume mixture-Summer (orchid grass, redtop Italian ryegrass, and common lespedeza)</td><td>Good stand, uncut, 4 to 5 in</td></tr>
<tr><td>Lespedeza sericea</td><td>After cutting to 2 in height, very good stand before cutting</td></tr>
<tr><td rowspan="2">E</td><td>Bermuda grass</td><td>Good stand, cut to average 1.5 in</td><td rowspan="2">0.35</td></tr>
<tr><td>Bermuda grass</td><td>Burned stubble</td></tr>
<tr><td colspan="4">Note: Covers classified have been tested in experimental channels. Covers were green and generally uniform.

**Table partially reproduced from FHWA’s Design of Roadside Channels with Flexible Linings</td></tr>
</table>

EXHIBIT A-11

Values of J_k for estimating the Length of Hydraulic Jumps	
$l_j = J_k \times y_2$	



Where:
l_j = length of hydraulic jump
y_2 = flow depth immediately downstream of jump
F_1 = Froude number for upstream channel section
J_k from table below

F_1	J_k
1.5 or less	4.0
2.0	4.4
2.5	4.8
3.0	5.3
3.5	5.6
4.0	5.8
4.5	5.9
5.0	6.0
5.5	6.2

Source:
Tabulated data is from: Chow, Ven Te, Ph.D. *Open Channel Hydraulics*, McGraw-Hill, Inc. 1988. Figure 15-4, Page 398.

EXHIBIT A-12.a

Constants for Inlet Control Design Equations.								
Chart No.	Shape and Material	Nomo-graph Scale	Inlet Edge Description	Eq'n Form	Unsubmerged		Submerged	
					K	M	c	Y
1	Circular Concrete	1 2 3	Square edge w/headwall Groove end w/headwall Groove end projecting	1	.0098 .0018 .0045	2.0 2.0 2.0	.0398 .0292 .0317	.67 .74 .69
2	Circular CMP	1 2 3	Headwall Mitered to slope Projecting	1	.0078 .0210 .0340	2.0 1.33 1.50	.0379 .0463 .0553	.69 .75 .54
3	Circular	A B	Beveled ring, 45° bevels Beveled ring, 33.7° bevels*	1	.0018 .0018	2.50 2.50	.0300 .0243	.74 .83
8	Rectangular Box	1 2 3	30° to 75° wingwall flares 90° and 15° wingwall flares 0° wingwall flares	1	.026 .061 .061	1.0 .75 .75	.0347 .0400 .0423	.81 .80 .82
9	Rectangular Box	1 2	45° wingwall flare d =.043D 18° to 33.7° wingwall flare d =.83D	2	.510 .486	.667 .667	.0309 .0249	.80 .83
10	Rectangular Box	1 2 3	90° headwall w/3/4" chamfers 90° headwall w/45° bevels 90° headwall w/33.7° bevels	2	.515 .495 .486	.667 .667 .667	.0375 .0314 .0252	.79 .82 .865
11	Rectangular Box	1 2 3 4	¾" chamfers; 45° skewed headwall ¾" chamfers; 30° skewed headwall ¾" chamfers; 15° skewed headwall 45° bevels; 10°-45° skewed headwall	2	.545 .533 .522 .498	.667 .667 .667 .667	.04505 .0425 .0402 .0327	.73 .705 .68 .75
12	Rectangular Box 3/4" chamfers	1 2 3	45° non-offset wingwall flares 18.4° non-offset wingwall flares 18.4° non-offset wingwall flares 30° skewed barrel	2	.497 .493 .495	.667 .667 .667	.0339 .0361 .0386	.803 .806 .71

EXHIBIT A-12.a (cont'd.)

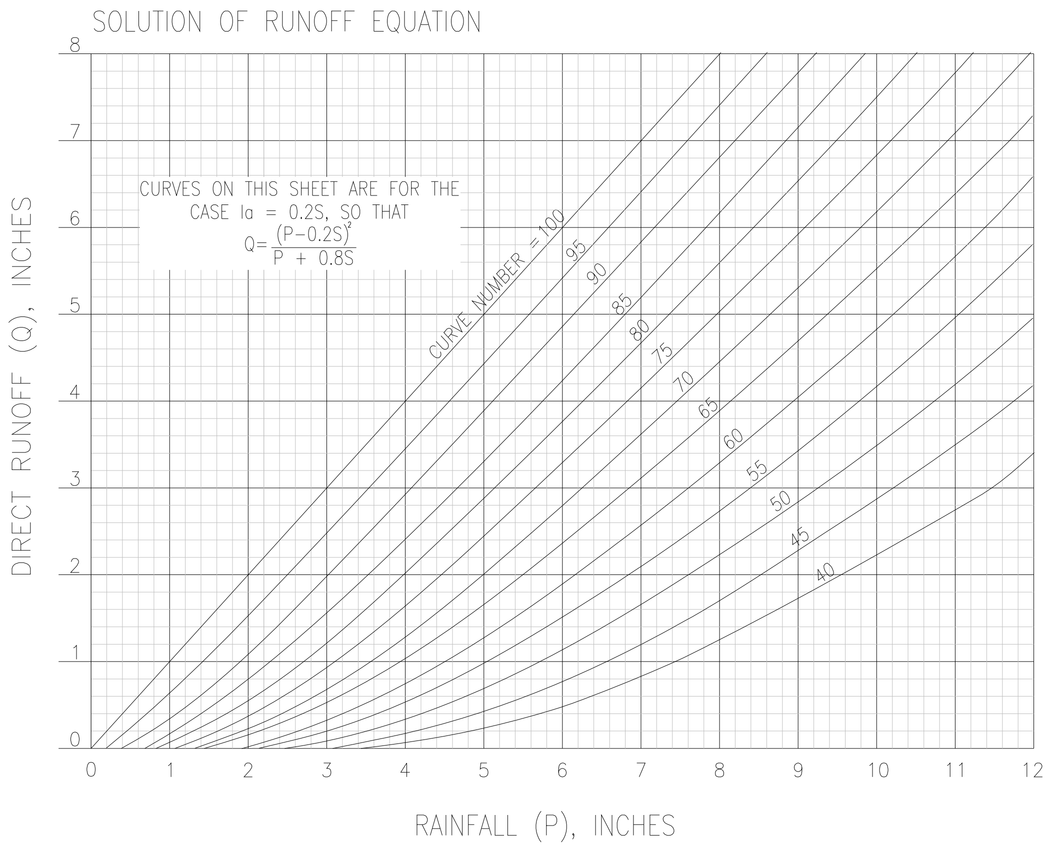
Constants for Inlet Control Design Equations - (cont'd.).								
					Unsubmerged		Submerged	
Chart No.	Shape and Material	Nomo-graph Scale	Inlet Edge Description	Eq'n Form	K	M	c	Y
13	Rectangular Box Top Bevels	1 2 3	45° wingwall flares – offset 33.7° wingwall flares – offset 18.4° wingwall flares - offset	2	.497 .495 .493	.667 .667 .667	.0302 .0252 .0227	.835 .881 .887
16-19	C M Boxes	2 3 5	90° headwall Thick wall projecting Thin wall projecting	1	.0083 .0145 .0340	2.0 1.75 1.5	.0379 .0419 .0496	.69 .64 .57
29	Horizontal Ellipse Concrete	1 2 3	Square edge w/headwall Groove end w/headwall Groove end projecting	1	.0100 .0018 .0045	2.0 2.5 2.0	.0398 .0292 .0317	.67 .74 .69
30	Vertical Ellipse Concrete	1 2 3	Square edge w/headwall Groove end w/headwall Groove end projecting	1	.0100 .0018 .0095	2.0 2.5 2.0	.0398 .0292 .0317	.67 .74 .69
34	Pipe Arch 18" Corner Radius CM	1 2 3	90° headwall Mitered to slope Thin wall projecting	1	.0083 .0300 .0340	2.0 1.0 1.5	.0379 .0463 .0496	.69 .75 .57
35	Pipe Arch 18" Corner Radius CM	1 2 3	Projecting No Bevels 33.7° Bevels	1	.0300 .0088 .0030	1.5 2.0 2.0	.0496 .0368 .0269	.57 .68 .77
36	Pipe Arch 31" Corner Radius CM	1	Projecting No Bevels 33.7° Bevels	1	.0300 .0088 .0030	1.5 2.0 2.0	.0496 .0368 .0269	.57 .68 .77
41-43	Arch CM	1 2 3	90° headwall Mitered to slope Thin wall projecting	1	.0083 .0300 .0340	2.0 1.0 1.5	.0379 .0463 .0496	.69 .75 .57
55	Circular	1 2	Smooth tapered inlet throat Rough tapered inlet throat	2	.534 .519	.555 .64	.0196 .0210	.90 .90
56	Elliptical Inlet Face	1 2 3	Tapered inlet-beveled edges Tapered inlet-square edges Tapered inlet-thin edge projecting	2	.536 .5035 .547	.622 .719 .80	.0368 .0478 .0598	.83 .80 .75
57	Rectangular	1	Tapered inlet throat	2	.475	.667	.0179	.97
58	Rectangular Concrete	1 2	Side tapered-less favorable edges Side tapered-more favorable edges	2	.56 .56	.667 .667	.0446 .0378	.85 .87
59	Rectangular Concrete	1	Slope tapered-less favorable edges Slope tapered-more favorable edges	2	.50 .50	.667 .667	.0446 .0378	.65 .71

Source: FHWA, *Hydraulic Design of Highway Culverts*

EXHIBIT A-12.b

Entrance Loss Coefficients. **Outlet Control, Full or Partly Full Entrance Head Loss** $h_e = K_e\left[\frac{V^2}{2g}\right]$	
<u>Type of Structure and Design Entrance</u>	<u>Coefficient K_e</u>
• <u>Pipe, Concrete</u>	
Projecting from fill, socket end (groove-end)	0.2
Projecting from fill, sq. cut end	0.5
Headwall or headwall and wingwalls	
Socket end of pipe (groove-end)	0.2
Square-edge	0.5
Rounded (radius = D/12)	0.2
Mitered to conform to fill slope	0.7
*End-Section conforming to fill slope	0.5
Beveled edges, 33.7^0 or 45^0 bevels	0.2
Side- or slope-tapered inlet	0.2
• <u>Pipe, or Pipe-Arch. Corrugated Metal</u>	
Projecting from fill (no headwall)	0.9
Headwall or headwall and wingwalls square-edge	0.5
Mitered to conform to fill slope, paved or unpaved slope	0.7
*End-Section conforming to fill slope	0.5
Beveled edges, 33.70 or 450 bevels	0.2
Side- or slope-tapered inlet	0.2
• <u>Box, Reinforced Concrete</u>	
Headwall parallel to embankment (no wingwalls)	
Square-edged on 3 edges	0.5
Rounded on 3 edges to radius of D/12 or B/12 or beveled edges on 3 sides	0.2
Wingwalls at 30^0 to 75^0 to barrel	
Square-edged at crown	0.4
Crown edge rounded to radius of D/12 or beveled top edge	0.2
Wingwall at 10^0 to 25^0 to barrel	
Square-edged at crown	0.5
Wingwalls parallel (extension of sides)	
Square-edged at crown	0.7
Side – or slope-tapered inlet	0.2
*Note: "End Sections conforming to fill slope," made of either metal or concrete, are the sections commonly available from manufacturers. From limited hydraulic tests they are equivalent in operation to a headwall in both <u>inlet</u> and <u>outlet</u> control. Some end sections, incorporating a <u>closed</u> taper in their design have a superior hydraulic performance. These latter sections can be designed using the information given for the beveled inlet.	
Source: FHWA, *Hydraulic Design of Highway Culverts*	

EXHIBIT A-13



Source: United States Department of Agriculture, Natural Resources Conservation Service, *Urban Hydrology for small Watersheds, TR-55,* June 1986, Figure 2-1

EXHIBIT A-14

Runoff Curves for Urban Areas

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

1 Average runoff condition, and Ia=0.2S

2 The average percent impervious area shown was used to develop the composite CN's. Other assumptions are as follows: impervious areas are directly connected to the drainage system, impervious areas have a CN of 98, and pervious areas are considered equivalent to open space in good hydrologic condition. CN's for other combinations of conditions may be computed using [Exhibit A-18] or [Exhibit A-19].

3 CN's shown are equivalent to those of pasture. Composite CN's may be computed for other combinations of open space cover type.

4 Composite CN's for natural desert landscaping should be computed using [Exhibit A-18] or [Exhibit A-19] based on the impervious area percentage (CN= 98) and the pervious area CN. The pervious area CN's are assumed equivalent to desert shrub in poor hydrologic condition.

5 Composite CN's to use for the design of temporary measures during grading and construction should be computed using [Exhibit A-18] or [Exhibit A-19] based on the degree of development (impervious area percentage) and the CN's for the newly graded pervious areas.

Source: United States Department of Agriculture, Natural Resources Conservation Service, *Urban Hydrology for small Watersheds, TR-55,* June 1986, Figure 2-2a

EXHIBIT A-15

Runoff Curve Numbers for Cultivated Agricultural Lands

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

1 Average runoff condition, and Ia=0.2S

2 Crop residue cover applies only if residue is on at least 5% of the surface throughout the year.

3 Hydraulic condition is based on combination factors that affect infiltration and runoff, including (a) density and canopy of vegetative areas, (b) amount of year-round cover, (c) amount of grass or close-seeded legumes, (d) percent of residue cover on the land surface (good $\geq$ 20%), and (e) degree of surface roughness.

Poor: Factors impair infiltration and tend to increase runoff

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

Source: United States Department of Agriculture, Natural Resources Conservation Service, *Urban Hydrology for small Watersheds, TR-55,* June 1986, Figure 2-2b

EXHIBIT A-16

Runoff Curve Numbers for Other Agricultural Lands

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

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

Source: United States Department of Agriculture, Natural Resources Conservation Service, *Urban Hydrology for small Watersheds, TR-55,* June 1986, Figure 2-2c

EXHIBIT A-17

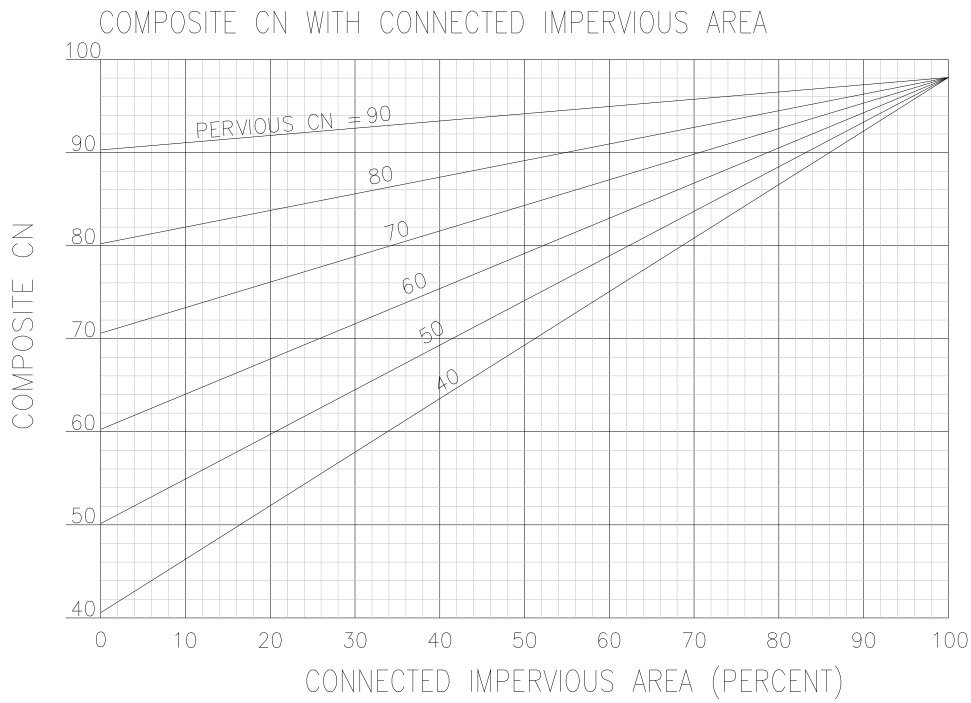
Runoff Curve Numbers for Arid and Semiarid Rangelands

-----------------------------------Cover description--------------------------------------		Curve numbers for ------------hydrologic soil group-----------			
Cover type	Hydrologic condition 2/	A3/	B	C	D
Herbaceous-mixture of grass, weeds, and low-growing brush, with brush the minor element.	Poor		80	87	93
	Fair		71	81	89
	Good		62	74	85
Oak-aspen-mountain brush mixture of oak brush, aspen, mountain mahogany, bitter brush, maple, and other brush.	Poor		66	74	79
	Fair		48	57	63
	Good		30	41	48
Pinyon-juniper-pinyon, juniper, or both; grass understory.	Poor		75	85	89
	Fair		58	73	80
	Good		41	61	71
Sagebrush with grass understory.	Poor		67	80	85
	Fair		51	63	70
	Good		35	47	55
Desert shrub-major plants include saltbush, greasewood, creosotebush, blackbrush, bursage, palo verde, mesquite, and cactus.	Poor	63	77	85	88
	Fair	55	72	81	86
	Good	49	68	79	84

1 Average runoff condition, and Ia, = 0.2S. For range in humid regions, use [Exhibit A-16].
2 Poor: <30% ground cover (litter, grass, and brush overstory).
Fair: 30 to 70%ground cover.
Good: > 70%ground cover.
3 Curve numbers for group A have been developed only for desert shrub.

Source: United States Department of Agriculture, Natural Resources Conservation Service, *Urban Hydrology for small Watersheds, TR-55,* June 1986, Figure 2-2d

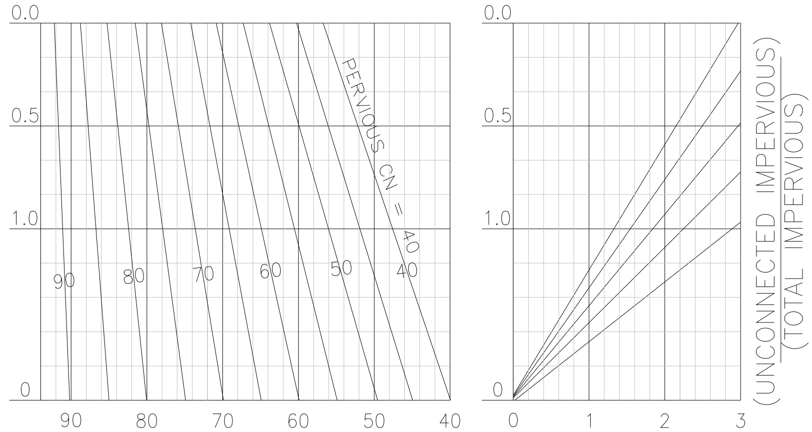
EXHIBIT A-18



Source: United States Department of Agriculture, Natural Resources Conservation Service, *Urban Hydrology for small Watersheds, TR-55,* June 1986, Figure 2-3

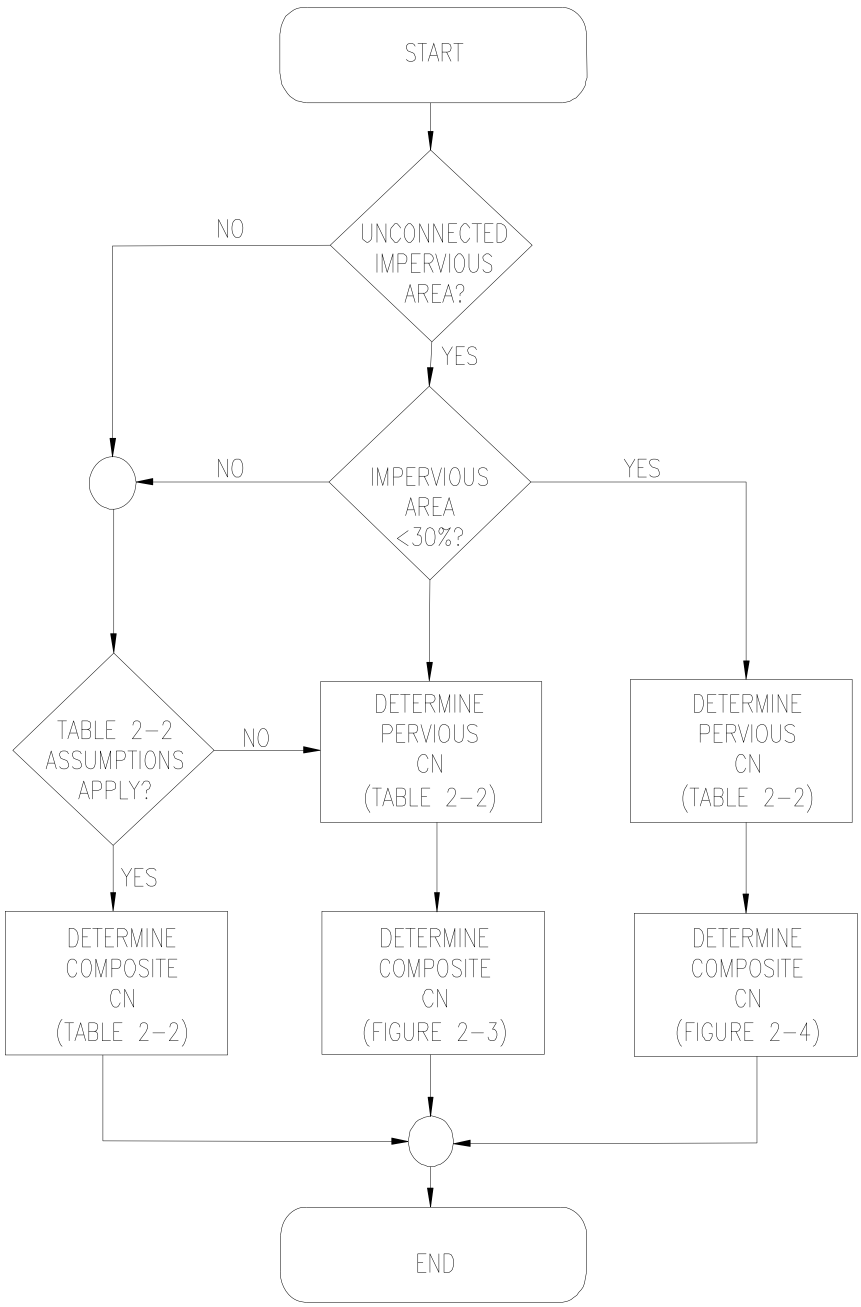
EXHIBIT A-19

COMPOSITE CN WITH UNCONNECTED IMPERVIOUS AREA AND TOTAL IMPERVIOUS AREA LESS THAN 30%



Source: United States Department of Agriculture, Natural Resources Conservation Service, *Urban Hydrology for small Watersheds, TR-55,* June 1986, Figure 2-4

EXHIBIT A-20



Source: USDA, NRCS, *Urban Hydrology for Small Watersheds; TR-55*

EXHIBIT A-21

CURVE NUMBER	I_a		CURVE NUMBER	I_a
40	3.000		70	0.857
41	2.878		71	0.817
42	2.762		72	0.778
43	2.651		73	0.740
44	2.545		74	0.703
45	2.444		75	0.667
46	2.348		76	0.632
47	2.255		77	0.597
48	2.167		78	0.564
49	2.082		79	0.532
50	2.000		80	0.500
51	1.922		81	0.469
52	1.846		82	0.439
53	1.774		83	0.410
54	1.704		84	0.381
55	1.636		85	0.353
56	1.571		86	0.326
57	1.509		87	0.299
58	1.448		88	0.273
59	1.390		89	0.247
60	1.333		90	0.222
61	1.279		91	0.198
62	1.226		92	0.174
63	1.175		93	0.151
64	1.125		94	0.128
65	1.077		95	0.105
66	1.030		96	0.083
67	0.985		97	0.062
68	0.941		98	0.041
69	0.899			

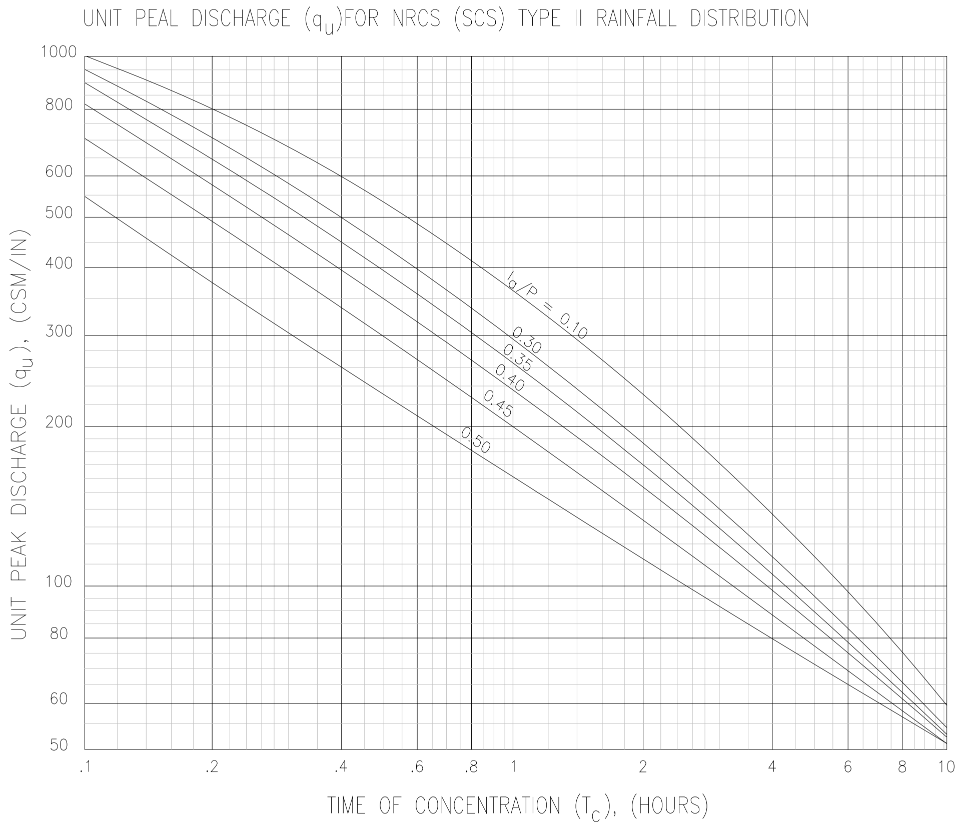
Source: USDA, NRCS, *Urban Hydrology for Small Watersheds; TR-55*

EXHIBIT A-22

Where q_u is the unit peak discharge in $^{csm}/_{in}$, T_c is the time of concentration in hours, and C_0, C_1, and C_2 are coefficients from the table below:

Rainfall Type II		**Coefficients for Eq. 4.6**		
		$\log(q_u) = C_0 + C_1 * \log(T_C) + C_2 * [\log(T_C)]^2$		
	Ia/P	C0	C1	C2
	0.10*	2.55323	-0.61512	-0.16403
	0.12	2.54444	-0.61587	-0.15928
	0.14	2.53565	-0.61661	-0.15454
	0.16	2.52686	-0.61736	-0.14979
	0.18	2.51807	-0.61810	-0.14505
	0.20	2.50928	-0.61885	-0.14030
	0.22	2.50048	-0.61959	-0.13555
	0.24	2.49169	-0.62034	-0.13081
	0.26	2.48290	-0.62108	-0.12606
	0.28	2.47411	-0.62183	-0.12132
	0.30*	2.46532	-0.62257	-0.11657
	0.32	2.44678	-0.61992	-0.10522
	0.34	2.42823	-0.61727	-0.09387
	0.36	2.40799	-0.61247	-0.08180
	0.38	2.38604	-0.60552	-0.06901
	0.40*	2.36409	-0.59857	-0.05621
	0.42	2.33541	-0.58716	-0.04285
	0.44	2.30672	-0.57575	-0.02949
	0.46	2.27447	-0.55924	-0.02077
	0.48	2.23864	-0.53761	-0.01668
	0.50*	2.20282	-0.51599	-0.01259
Source: Stevens, Michael A. in *Estimating Design Discharge for Small Ungaged Watersheds Using the SCS Method*; http://peacecorps.mtu.edu/resources/studentprojects/scs/index.html. Rows marked with * were in the Stevens table, all others were interpolated.				

EXHIBIT A-23



Source: United States Department of Agriculture, Natural Resources Conservation Service, *Urban Hydrology for small Watersheds, TR-55,* June 1986, Figure 4-II

EXHIBIT A-24

Pond Adjustment Factor F (Normally, ponding and swamp factors in Marble Falls are minimal and should be taken into account in calculating Time of Concentration, with F taken as 1)	
Percentage of pond and swamp areas	**F**
0	1.00
0.2	0.97
1.0	0.87
3.0	0.75
5.0	0.72

Source: Stevens, Michael A. in *Estimating Design Discharge for Small Ungaged Watersheds Using the SCS Method*; http://peacecorps.mtu.edu/resources/studentprojects/scs/index.html

EXHIBIT A-25

Values of *C* in *Q* = *CLH*$^{3/2}$ for Broad-crested Weirs											
Measured head *H* ft	Breadth of crest of weir, ft										
	0.50	0.75	1.00	1.50	2.00	2.50	3.00	4.00	5.00	10.00	15.00
0.2	2.80	2.75	2.69	2.62	2.54	2.48	2.44	2.38	2.34	2.49	2.68
0.4	2.92	2.80	2.72	2.64	2.61	2.60	2.58	2.54	2.50	2.56	2.70
0.6	3.08	2.89	2.75	2.64	2.61	2.60	2.68	2.69	2.70	2.70	2.70
0.8	3.30	3.04	2.85	2.68	2.60	2.60	2.67	2.68	2.68	2.69	2.64
1.0	3.32	3.14	2.98	2.75	2.66	2.64	2.65	2.67	2.68	2.68	2.63
1.2	3.32	3.20	3.08	2.86	2.70	2.65	2.64	2.67	2.66	2.69	2.64
1.4	3.32	3.26	3.20	2.92	2.77	2.68	2.64	2.65	2.65	2.67	2.64
1.6	3.32	3.29	3.28	3.07	2.89	2.75	2.68	2.66	2.65	2.64	2.63
1.8	3.32	3.32	3.31	3.07	2.88	2.74	2.68	2.66	2.65	2.64	2.63
2.0	3.32	3.31	3.30	3.03	2.85	2.76	2.72	2.68	2.65	2.64	2.63
2.5	3.32	3.32	3.31	3.28	3.07	2.89	2.81	2.72	2.67	2.64	2.63
3.0	3.32	3.32	3.32	3.32	3.20	3.05	2.92	2.73	2.66	2.64	2.63
3.5	3.32	3.32	3.32	3.32	3.32	3.19	2.97	2.76	2.68	2.64	2.63
4.0	3.32	3.32	3.32	3.32	3.32	3.32	3.07	2.79	2.70	2.64	2.63
4.5	3.32	3.32	3.32	3.32	3.32	3.32	3.32	1.88	2.74	2.64	2.63
5.0	3.32	3.32	3.32	3.32	3.32	3.32	3.32	3.07	2.79	2.64	2.63
5.5	3.32	3.32	3.32	3.32	3.32	3.32	3.32	3.32	2.88	2.64	2.63

Source: King, H. W. and Brater, E. F., *Handbook of Hydraulics,* 5th Edition, McGraw-Hill Book Company, New York, pg. 5-46