

## 1.0 Introduction

### 1.1 Purpose and Scope

- A. The purpose of this design manual is to establish standard principles and practices for the design and construction of storm drainage, flood protection, ~~and~~ erosion control, and water quality facilities within the City of New Braunfels, Texas and within its extraterritorial jurisdiction.
- B. The design factors, formulas, graphs, and procedures described in the following pages are intended to serve as guidelines for the solution of drainage problems involving the volume and rate of flow, method of collection, storage, conveyance, treatment, and disposal of stormwater and erosion protection from stormwater flows. Ultimate responsibility for actual design, however, remains with the design engineer. Any deviation from the methodology or requirements of this manual shall be approved recommended by the City Engineer and approved by the Director of Public Works of the City of New Braunfels.
- ~~C. Section 4.0 and beyond are concerned with issues related to the Drainage Plan and Drainage Report required for a Type 3 Development.~~
- ~~D. In cases where the owner has negotiated monies instead of on-site or off-site drainage improvements, the development standards will be tailored within the terms of the negotiations.~~

### 1.2 References

At certain points in the text, the reader will encounter numbers enclosed in parentheses, for example (1). These numbers correspond to the references listed in Appendix A.

### 1.3 Acknowledgments

This design manual is the result of the dedication and energy of the Drainage Advisory Committee members (for the Sep 2000 edition). For the October 2013 edition, acknowledgements to City staff, Watershed Advisory Committee, Technical Advisory Committee, Design Workshop, Lockwood Andrews & Newnam, and input from a wide variety of public stakeholders.

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**Comment [A1]:** Has City developed a tracking procedure for variances? Should a sample variance letter be included in Appendix of DCM (developer/CE/DPW signature blocks)? Does the City have a database/website where DCM-only variances could be available for public inspection?

**Comment [A2]:** Suggest deleting, implies you can buy your way out of the regs

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**A-2.0 Drainage Policy And Criteria**

**2.1 Development Categories Requirements**

In an effort to facilitate development while applying drainage rules, a tier system is established requiring different submittals and different development actions depending on the probable impact on the drainage basin. In all cases, properly sized easements shall be granted across all contiguous property owned by the applicant; and a comprehensive Drainage Plan and Drainage Report shall be provided for all property on the subject plat whether developed by this application or not. Best Management Practices (BMPs) shall be exercised in the design process.

**2.1.1 Type 1 Development**

A. A Type 1 Development is any development or redevelopment in the following categories:

- 9.0 Disturbs less than one acre of land;
- 10.0 Creates less than 1,000 square foot of additional impervious cover; or
- 11.0 Creates additions to single family or duplex residential structure.

9 Submittals for a Type 1 Development include: location and contact information (e.g. name, address, phone number, property location), site drawing for the proposed disturbance, review of applicable BMP's and temporary erosion control techniques. For a Type 1 Development:

A. Drainage easements may be required to accommodate future or existing development

**A. Type 2 Development**

1. A Type 2 Development is any development or redevelopment in the following categories:

- A. Agriculture (not including feedlots), or
- B. Single family or duplex residential not in a major subdivision (three or more lots) with more than 1,000 square feet of additional impervious cover, or
- C. Non-residential developments of less than 5,000 square foot of additional impervious cover, or
- D. Development not meeting criteria for a Type 3 Development.

9 Submittals for a Type 2 Development include: location and contact information, site drawing or sketch for the proposed disturbance (a scaled drawing (scale 1" = (50' or less)) on 11" x 17" paper showing existing drainageways, flow directions, floodplain boundaries, proposed grading and development, and proposed drainage, and erosion control facilities with a copy of the survey plat showing the lot layout, streets, and utility and drainage easements), review of applicable BMP's and temporary erosion control techniques, agreement letter specifying BMP's to be included in the project. For a Type 2 Development:

- 1. If any on site and off site stormwater structure is known to be at or above design capacity, the owner/developer shall be responsible for increasing the size of the structure to accommodate the development.
- 2. Drainage easements may be required to accommodate future or existing development.

**2.1.3 Type 3 Submittal Requirements Types of Development**

A. All A Type 3 Development is considered any development or redevelopment in the following categories:

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1. ~~Non-residential development with more than 5,000 square feet of additional impervious cover,~~
2. ~~Residential subdivision with other than single family and duplex units,~~
3. ~~Major subdivisions (three or more lots),~~
4. ~~Disturbs more than one acre of land,~~
5. ~~Development within a FEMA designated flood hazard area or adjacent to a major watercourse, or~~
6. ~~Agricultural feedlots~~

~~Submittals shall~~Submittals for a Type 3 Development include:

~~B.1. Location~~ location and contact information, Drainage Report, Erosion Control Plan, agreement letter specifying BMP's to be included in the project or other site specific requirements. ~~For a Type 3 Development:~~

~~A.2.~~ Mitigation through detention, retention, or some other technique must be designed, constructed, and maintained to reduce the post-development discharge rate to below that of pre-development for the ~~2, 5, 10, 25, 50, and 100-yr~~ 10 year and 100 year design storms. Participation in neighborhood or regional mitigation is an acceptable option.

~~B.3.~~ In cases where adequate detention is not available and an In-lieu-of negotiation is in place, all on-site and off-site stormwater structures must be sized to convey the additional stormwater below the design level of each structure encountered from the property to the first major stream. For existing structures, see paragraph ~~3-4~~ below.

~~C.4.~~ If any encountered structure is at or above design capacity, the owner/developer shall be responsible for increasing the size of the structure to accommodate the development, at their own expense or demonstrating cause why the city should partner in the project.

~~D.5.~~ On-site drainage easements may be required to accommodate future or existing development. Off-site drainage easements may also be required if the increase in water quantity impacts existing water storage capacity and increases the possibility of flooding.

~~10B.~~ The Drainage Plan and Drainage Report containing the proposed storm drainage and flood protection system must be submitted as part of the preliminary platting process or application for a Building Permit. A revised Drainage Plan and Drainage Report shall be submitted after all issues have been resolved with the City Engineer.

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## 2.2 Water Quality

Stormwater discharge from developments ~~will eventually be~~ regulated for the quality of the water discharged. Standards for water quality of discharge are ~~not currently being enforced~~ similar to those described in the TCEQ regulations for the Edwards Aquifer Recharge Zone. In addition, Low Impact Development (LID) Methods are encouraged whenever possible. ~~When~~ The responsibilities of administering EPA/~~TNRCC~~-TCEQ NPDES Phase II ~~fall to~~ regulations now rest with the City, and additional requirements ~~will~~ may be enforced developed.

- A. Permanent Structural Best Management Practices (BMPs) are required that will reduce the increase in total suspended solids (TSS) load associated with the development by at least 80%. BMP methodology and design criteria, unless otherwise specified in this manual, should follow TCEQ RG-348 procedures.
- B. At least two BMPs in series are required for every development over 75% of the developed area. The City Engineer may approve non-Structural BMPs, as the second BMP. The developer is encouraged to discuss the water quality treatment plan with the City Engineer prior to an initial submittal.
- C. Other TCEQ requirements, such as Geologic Assessments, TCEQ forms, etc., etc., are not specifically required by the City unless they directly pertain to the design of the BMPs required by the City.
- D. Innovative BMPs may be allowed if developer follows TCEQ procedures, including post-installation verification and validation requirements, and is approved by City Engineer.

These requirements apply to areas within and outside the boundaries of the TCEQ Edwards Aquifer Contributing, Transition, and Recharge zones. When developments also fall within TCEQ regulated zones, all relevant TCEQ regulations will apply as well.

## 2.3 Drainage Structure Aesthetics

- ~~7~~A. Drainage design in the urban environment must also consider appearance as an integral part of the design and structures should generally blend with the natural surroundings as much as possible to maintain the aesthetics of the natural area. The City of New Braunfels ~~strongly encourages~~ encourages requires preservation of the natural floodplains.
- ~~8~~B. The protection of existing trees and vegetation should be maximized during development of drainage plans. Whenever possible, the replacement of the trees destroyed by drainage and flood protection procedures is encouraged.

## 2.4 Drainage Design Computations

Computations to support all drainage designs shall be submitted to the City Engineer for review as part of the Drainage Report and shall be summarized in the form of the standard computation sheets contained in this manual. Computer programs used to perform computations shall be limited to those referenced in this manual unless approved by the City Engineer. On-site pre-development stormwater runoff computations shall be based upon conditions representing the existing land conditions with respect to soil type, percentage cover, and cover type as indicated ~~in the 3-20-97~~ by current aerial photos imagery and supporting documentation. ~~In the event that photography is not available for the proposed site of development, undeveloped conditions must be assumed unless other documentation is presented fixing the level of development as of March 20, 1997.~~ Design of structures shall use fully developed sub-basin conditions for the prescribed design storms.

## 2.5 Criteria for Design of Drainage Facilities

**Comment [A3]:** How would the City like to coordinate the water quality enforcement with this revision of manual?

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**Comment [A4]:** 1)How much stricter does the City want to go beyond TCEQ Edwards Aquifer regulations?

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**Comment [A5]:** Do we want stronger language than this?

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**Comment [A6]:** Do we want stronger language than this?

**Comment [A7]:** Tree ordinance only applies to commercial development.

2.5.1 General

- A. The planning and design of drainage systems should ensure that problems are not transferred from one location to another. Grading and other construction activities may not change the terrain to cause damage to public or private property from drainage or flood problems, increased runoff, or increased erosion or sediment movement.
- B. Lot to lot drainage of sheet flows should be avoided in subdivision design.
- C. The City Engineer shall not approve any Drainage Report pertaining to proposed construction, platting or other development where the proposed activity or change in the land would result in post-development discharge from the site exceeding discharge under natural conditions (prior to grading or other development), immediately downstream of the proposed site. Downstream capacity shall not be exceeded as a result of development. Exemptions from this provisions are as follows:
  - 1. Additional drainage improvements are not required if drainage improvements have been provided for the fully developed condition, which includes the proposed development.
  - 2. A fee may be utilized in place of a detention/retention system, at the request of affected persons, when it can be clearly demonstrated that detention/retention at the site does not provide off-site flood relief due to the parcel size, location, or other factors. The fees collected will be used to construct public flood control improvements, which will be designed to mitigate the potential damage of floodwaters associated with the property from which the fees are contributed. The amount of the fee shall be proportional to the cost of the otherwise required detention/retention system.

~~2-D.~~ No proposed development shall be constructed which impedes or constricts runoff from an upstream watershed based on fully developed conditions. Therefore drainage computations shall be provided to verify no adverse impact upstream or downstream. The developer and City have the option to partner on downstream improvements.

2.5.2 Stormwater retention or detention facilities

- A. Stormwater retention or detention facilities must reduce peak flows from the ~~2, 5, 10, 25, 50, and 100-yr~~ 10-year, 24-hour storm and the 100-year, 24-hour storm, such that these peak flows are no greater than under pre-development conditions. This peak flow comparison will occur at two locations for each discharge point.
  - 1. At the edge of the developing owner's property line.
  - ~~1-2.~~ At a point 3,000ft downstream from the edge of the developing owner's property line. If a stream confluence has the potential to mask a possible impact, the City Engineer may require the developer to analyze a different downstream point. The developer is encouraged to discuss the downstream analysis point with the City Engineer prior to an initial submittal.
- ~~2-B.~~ The method(s) of retention or detention shall be appropriate to the type of development, topography, and amount of control needed. Suggested measures include the following:
  - 1. basins or swales – single or multiple
  - 2. check dams in gullies to slow runoff and trap sediment

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Comment [A8]: An explicit statement of NAI. These other sentences were moved from old DCM 5.2.

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Comment [A9]: How does the City feel about the proposed requirement to evaluate downstream drainage conditions 3,000ft downstream? This was added to match what is currently in the Code, Sec 143-20.

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3. leach fields, infiltration chambers, dry wells, rain barrels, French drains
4. granular fill under permeable paving blocks
5. contour terracing, improved vegetation cover.

~~3.C.~~ Detention/retention facilities may be incorporated into parks, open space areas and landscaping designs. Parking areas may be used as detention or retention facilities provided that maximum depths of ponding do not exceed eight inches, and ponding is in the areas most remotely situated from structures.

~~4.D.~~ Stormwater infiltration systems are not permitted in any development where there is a potential for pollutants to adversely affect ground water quality (e.g. EARZ).

~~5.E.~~ No detention or retention basin shall retain standing water longer than 36 hours unless it is designed and constructed to be a permanent pond with appropriate health, safety, and water quality measures, and water rights requirements for such a body of water.

~~6.~~ Individual lot basins within subdivisions may be approved for lots of one acre or more with slopes under 5% (five percent).

~~7.F.~~ Specific requirements for retention/detention facilities are as follows:

~~A.1.~~ Facilities shall be located such that the edge of the 100-year water surface is at least 10 feet from the edge of any public road. Finished floors of adjacent structures should be a minimum of 1 foot above the 100-year water surface in the facility. Facilities should preferably be located such that the invert of the outlet structure is above the 100-year flood level in the receiving body; but in all cases facilities shall be designed to function properly during conditions where the outlet is submerged by the tailwater of the receiving stream.

~~B.2.~~ Drainage easements or open space designations may be required for retention/detention facilities. Easement boundaries shall contain the berms, inlet and outlet structures, access ramps, permanent erosion control facilities, the 100-year water surface and any additional area needed for access and maintenance.

~~C.3.~~ Ponding below natural grade (depressed storage) is encouraged.

~~D.4.~~ Detention facilities shall be designed with one or more outlet structures to allow safe passage of the 100-year post-development design storm runoff. In addition, an emergency spillway shall be provided with sufficient capacity to pass at least the 25-year design storm runoff assuming the pond is full. Spillways and outlets shall be protected from erosion with riprap, grouted riprap, or other method or erosion control to adequately protect the structure and downstream channel. Outflows shall be conveyed to an appropriate receiving drainage facility in a manner such that roadways, buildings, etc. are not damaged.

~~E.5.~~ In the event a detention facility empties into another storage facility downstream, the effect of the facility's outflow hydrograph (volume and peak flow) on that facility shall be evaluated.

~~F.6.~~ Side slopes of earthen embankments shall be designed for stability and safety, with the following minimum requirements for facilities with unrestricted access: ~~in facilities with ponding depths of 18 inches to 3 feet, side slopes of earthen banks shall be 2.53:1 or flatter; side slopes shall be 3:1 or flatter in facilities with~~

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~~maximum ponding depths over 3 feet~~; a benched configuration is required for facilities with ponding depths over 6 feet. Bench widths shall be at least 4 feet, spaced at least every 3 feet vertically. The above slope criteria may be waived if security barriers and erosion control measures are provided, with City Engineer approval. Barriers may consist of chain-link, masonry, wood, vegetation or other materials, but must not restrict the hydraulic capacity of drainage facilities. Minimum barrier height is 48". Vegetative barriers must be of a width equal to or greater than the total height, with density sufficient to restrict access. All constructed stormwater structures of earthen material shall be revegetated to mature growth.

~~G.7.~~ Maximum water depths over 6 feet shall not be allowed without prior approval from the City Engineer.

~~H.8.~~ All earthen drainage structures or facilities shall be compacted in lifts not to exceed eight inches during construction to 90% Standard Proctor.

### 2.5.3 Culvert or bridge crossings

A. Arterial streets shall meet the stricter of the most recent Texas Department of Transportation criteria for crossings on urban highways, or;

~~11-01.~~ 50-year design storm runoff, with headwater one foot below the top of the culvert structure.

~~12-02.~~ 100-year water surface shall not encroach through half of roadway lanes

~~13-03.~~ Minimum culvert size 24" circular pipe

B. All other streets shall meet the following criteria for crossings, as a minimum:

~~A.1.~~ 25-year design storm runoff, with headwater one foot below the top embankment

~~B.2.~~ 25-year water surface shall leave at least one lane open.

~~C.3.~~ 50-year design storm runoff no more than 6" over top of roadway

~~D.4.~~ Allowance shall be made for conveyance of the 100-year runoff across the road and into the downstream channel without damage to the road or adjacent property

~~E.5.~~ Minimum culvert size 18" circular pipe

C. Temporary crossings shall be designed to safely pass the 2-year design storm runoff, minimum.

D. The backwater created by a culvert or bridge during the 100-year design storm runoff shall not cause damage to public or private property.

E. Culvert outlets will be designed to minimize damage caused by erosion. Developer will provide erosion protection for design storm velocities greater than 6 fps.

F. Culverts and bridges shall be aligned with natural drainageways in grade and direction whenever practical. Minimum slopes by culvert type must ~~Culverts shall have a minimum~~

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slope of 0.5% and a minimum design storm velocity of 2.5 fps be observed to reduce sediment accumulation.

G. Larger culvert sizes, bridges and/or box culverts or smooth-walled pipe are recommended for crossings where heavy debris or sediment accumulations are anticipated. Trash racks may be required.

H. All headwalls shall be constructed of reinforced concrete.

#### 2.5.4 Surface use of streets and alleys for drainage

A. General requirements for streets are:

111. The roadway or paved alley must be able to contain the 100-year flow within the right of way. Runoff shall not enter private property from a street except in recorded drainage easements or rights-of-way, or in historic watercourses where easements or rights-of-way have not been obtained.

122. 100-year design storm depth of water shall not exceed 10" at any point within the street right-of-way and the product of maximum depth (feet) times average cross-section velocity (feet per second) at any point shall not exceed 6.5.

133. Rundowns shall be designed to convey and contain drainage carded by the roadway to ensure the 100-year event is contained within the right-of-way. If a storm drain system is present, rundowns shall be designed for the difference between the storm drain capacity and the 100-year runoff, with a 25-year minimum design assuming all of the flow bypasses the storm drain system.

144. Driveways should be constructed to allow the 25-year design storm runoff to pass under the driveway in a culvert (158" minimum) or over the driveway on a concrete apron. Concrete aprons or box culverts are preferred in areas of heavy sediment transport.

155. The side slope of a ditch or swale on the side adjacent to the road shall be no steeper then than 4:1 (6:1 TxDOT)

9C. Water Spread limits for Roadways is as indicated in Table 2-1. No lowering of the standard height of street crown shall be allowed for the purposes of obtaining additional hydraulic capacity. Where additional hydraulic capacity is required, the proposed street gradient must be increased or curb inlets and storm sewers installed to remove a portion of the flow. For non-curbed streets, the 100-year frequency flows shall be contained within available rights-of-way.

#### 2.5.5 Storm drain systems

A. General requirements for storm drain systems are:

1. Storm drain pipes, inlets, and roadside drainage swales are designed for the 25-year design storm runoff with the design HGL of the system a minimum of 2 feet below the level of the street subgrade edge of pavement.

2. Pipe. If corrugated metal pipe is used, the manufacturer's design guidelines should be followed. Concrete lining shall be used with corrugated metal pipes with diameters of 36 inches or greater. Plastic pipe can be used only if authorized by the City Engineer and in no case shall plastic be used under roadways.

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3. Connections. Concrete pipe collars or manufactured transition pieces must be used at all pipe size changes on trunk lines. For all pipe junctions other than manholes and junction boxes, manufactured wye connections should be used, and the angle of intersection shall not be greater than 45 degrees. This includes discharges into box culverts and channels. Special circumstances may require cut-ins instead of manufactured wye connections; the use of cut-ins must be approved by the City Engineer. Laterals shall be connected to trunk lines using manholes or manufactured wye connections. Special situations may require laterals to be connected to the trunk lines by a cut-in (punch-in), and such cut-ins must be approved by the City Engineer. Inlet laterals will normally connect only one inlet to the trunk line. Special circumstances requiring multiple inlets to be connected with a single lateral shall be approved by the City Engineer. Vertical curves in the conduit will not be permitted, and horizontal curves must meet manufacturer's requirements for offsetting of the joints.
4. The maximum manhole or junction box spacing for storm drain systems is shown in Table 8-2. Junction boxes must also be located at:
  - A. Pick up points having three or more laterals;
  - B. Trunk line size changes for pipes with diameter differences greater than 24 inches;
  - C. Vertical alignment changes;
  - D. Future collection points.
5. The cover over the crown of circular pipe should be at least three feet and should be based on the type of pipe used, the expected loads and the supporting strength of the pipe. Box sections should normally have a minimum of one foot of cover; however, box sections may be designed for direct traffic in special situations with the approval of the City Engineer.
6. Grates for drop inlets should be designed with grate units weighing 250 to 300 pounds to facilitate removal for maintenance, but minimize vandalism. Design shall consider traffic loading and bicycle and pedestrian safety.
7. The minimum lateral storm drainpipe diameter shall be 18 inches, except in sump areas, which shall be at least 24 inches in diameter. The minimum pipe diameter for a trunk line pipe shall be 24 inches. Manholes should be located at junctions, changes in pipe size, and at sharp changes in direction or grade, and at regular intervals of 300 feet, maximum. The requirement for manholes may be waived if the pipe size allows direct access into the pipe by maintenance personnel and equipment. More stringent criteria may be imposed by the City Engineer to reduce or facilitate maintenance.
8. A bypass flow of not more than 10% of the 103 cfs of the 25-year flow will be allowed on streets with grades of three percent or greater or through any street intersection regardless of slope. No bypass flow will be allowed for inlets on streets with grades less than three percent.
9. For storms of a 1025-year frequency or less, water flowing in arterial streets shall be intercepted by an inlet prior to super-elevated sections, to prevent water from flowing across the roadway. In critical circumstances, this requirement can be waived by the City Engineer.

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10. All storm sewer conduits to be dedicated to the City of New Braunfels shall be located in drainage easements dedicated to the City of New Braunfels at the time of final platting of the property. Storm sewer easements shall be at least 15 feet wide. Wider easements may be required for multiple box culverts, other multiple storm sewer designs or for extremely wide single-line storm sewers as outlined in the Drainage Design Manual.

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**Table 2-1**  
**Water Spread Limits for Roadways**

<b>Street Classification</b>	<b>1025-Year Permissible Water Spread</b>
Arterial Streets	One 11-foot traffic lane must remain open in each direction.
Collector Streets	One 11-foot traffic lane must remain open.
Residential Streets	Water flow must not exceed the top of either curb.

- B. Connections from Buildings to Storm Sewers. Drainage from residential areas, such as rooftops, should be allowed to flow overland before joining the storm sewer system. Seepage into basements that is pumped to ground level, seepage from springs and runoff from roof drains on nonresidential buildings that would flow onto or across driveways, sidewalks or other areas commonly crossed by pedestrians can create hazards or nuisances to pedestrians. Thus, if hazards or nuisances would be created, the basement and rooftop drains shall be tied directly to the nearest storm sewer. Pumped lines from basements shall have backflow preventers.

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2.5.6 Channels

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- ~~1.A.~~ The City of New Braunfels ~~encourages~~ requires the preservation of natural channels and drainage patterns. Concentrated drainage flows must enter and depart from a developed area in the same manner and location as under pre-development conditions.

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- ~~2.B.~~ Easements or drainage right-of-ways shall be provided for all channels (artificial and natural) and shall be labeled as drainage easements on plats for recording. For properties with existing structural development on previously platted lots, setbacks of the same dimensions may be used in place of easements. The requirement for a setback or easement may be waived by the City Engineer. Easements, setbacks and FEMA floodways shall not be encroached upon with fill materials or structures, which would reduce the channel's ability to carry the 100-year flood.

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- Easement width shall be at least the width of the water surface from the 100-year design storm runoff under post-development conditions. In addition, an additional ~~12-14~~ feet, minimum, shall be allowed for access.
- Additional easement width should be provided to allow for channel meandering near bends of channels.
- Easement width should be measured outward from the centerline of the watercourse, 1/2 of the dimension to the right and 1/2 to the left of ~~center, center~~; additional access easement shall be 10 feet on one side and ~~two~~ four feet on the other.

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- ~~3.C.~~ Artificial channels and swales:

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1. Artificial channels and swales shall be designed to contain the 100-year design storm runoff with the water surface at the top of the structure or within the easement whichever is more restrictive. Freeboard along the outside of channel bends shall include the increased water surface due to superelevation (refer to Section 9.6).

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2. For large channels where exposure to the wind may cause wave action, additional freeboard must be included to accommodate 75 mph winds without washover.

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3. Fencing and/or warning signs shall be required to prevent public access where flowing water would pose a safety hazard as determined by the City Engineer.

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4. Unlined improved channels that contain bends shall be designed such that erosion at the bends is minimized. Erosion protection at bends shall be determined based on the velocity along the outside of the channel bend (refer to Section 9.5).

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4.D. Flumes. Sidewalks crossing times shall be A.D.A. compatible so as to minimize danger to pedestrians (e.g. covered, flared, or bridged times; handrails on sidewalks). Applicants shall dedicate drainage easements for times.

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5.E. Channels shall be designed to be stable and to not create safety hazards. Lined slopes should be no steeper than 1:1. Side slopes of artificial earthen channels should be 3:1 or flatter in channels with depths greater than 2 feet. Recommended maximum water velocities for earthen channels are given in Section 9. Erosion control or energy dissipation devices should be used to control velocities such that channel degradation does not occur. Bank stabilization measures shall not reduce channel capacity and shall follow sound engineering practices.

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6.F. Should diversion of a natural drainageway be required, sufficient work shall be done upstream and/or downstream to provide all affected properties at least the same level of flood protection and erosion control that existed prior to the diversion. The time length of a diversion channel must be at least as long as the segment of natural channel being replaced so that velocity is not increased.

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7.G. Maintenance Access Requirements. Access roads and/or ramps shall be provided for all channels to allow vehicular access for maintenance. The location and design of access roads and ramps shall be approved by the City Engineer. Access roads shall have a width of at least twelve feet and a cross slope no greater than two percent. Ramps on access roads shall have a vertical grade no steeper than ten percent.

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## 2.6 Freeboard

Table 2-3 provides the required freeboard for fully developed watersheds.

**Table 2-3**  
**Freeboard**

<u>Storm Drainage Facility</u>	<u>Frequency</u>	<u>Freeboard</u>
<u>Street right-of-way</u>	<u>100-year</u>	<u>None</u>
<u>Channels and Creek Improvements</u>	<u>100-year</u>	<u>1 foot</u>
<u>Swales and Ditches*</u>	<u>25-year</u>	<u>0.5 foot</u>

<u>Detention Ponds and Reservoirs</u>	<u>100-year</u>	<u>1 foot</u>
<u>Culverts and Bridges</u>	<u>25-year</u>	<u>**</u>
<u>Floodways and Floodplains</u>	<u>100-year</u>	<u>2 feet (or in accordance with City Floodplain Ordinance, whichever is greater)</u>

\* Swales or ditches are considered to have drainage areas of 128 acres or less. In all cases, the 100-year event shall be contained in natural drainage channels, drainage easements, or public rights-of-way

\*\* The culverts and bridges are designed to withstand the 100-year event, but the water level may reach roadway level at the 25-year design level if no public safety issues are involved.

**2.7 Drainage Easement Requirements**

As defined in the City Code of Ordinances, Sec 143-2, g (Maintenance), any drainage improvements which accept storm water run-off from an area greater than 300-acres shall be dedicated to the City as right-of-way. In addition, other easement dedication requirements are described further in this Manual.

**Comment [A10]:** Add additional explanation of ROW vs easement and what City will do with this land.

**2.8.7 Maintenance of Drainage Facilities**

The hydraulic integrity of drainage systems dedicated to and accepted by the City of New Braunfels, will be maintained by the City of New Braunfels. The hydraulic integrity of drainage systems not dedicated with approval of the City Engineer to the City of New Braunfels, shall be maintained by the property owner. Floodplain, and drainage easements, and water quality features shall be maintained by the property owner.

**Comment [A11]:** Does the City want to include an O&M plan to as part of an initial submittal? Does a maintenance reporting requirement form need to be added?

A biennial (every 2 years) maintenance compliance form is required to be submitted to the City by property owners responsible for drainage or water quality maintenance. A template of the maintenance compliance form and submittal procedures will be available on the City website. The City may determine unresponsiveness to allow corrective action as described in City's Code of Ordinances 143-2.

**2.9 Enforcement of Drainage and Water Quality Facilities**

**Comment [A12]:** Does the City Attorney want to include a reference to the code of ordinances regarding enforcement (Part II Section 143-2(g)? Will water quality enforcement be added to the ordinance, also?

**Table 2-3  
Freeboard**

<b>Storm Drainage Facility</b>	<b>Frequency</b>	<b>Freeboard</b>
Street right-of-way	100-year	None
Channels and Creek Improvements	100-year	Wave action level***
Swales and Ditches*	25-year	None
Reservoirs	100-year	1-foot
Culverts and Bridges	25-year	**
Floodways and Floodplains	100-year	2 feet (in accordance with FEMA)
Levees	SPF***	4-feet

\* In all cases, the 100-year event shall be contained in natural drainage channels, drainage easements, or public rights of way  
 \*\* The culverts and bridges are designed to withstand the 100-year event, but the water level may reach roadway level at the 25-year design level if no public safety issues are involved.  
 \*\*\* SPF—Standard Project Flood  
 \*\*\*\* There are currently no known conditions in the City where wave action level is a consideration.

2.810 Drainage and Water Quality Report Requirements

a.A. An electronic media copy of drainage construction and topography in AutoCAD 14 is required in addition to the paper file copy. Specific digital file requirements, if any, will be specified on the application forms.

Comment [A13]: Add in WQ reporting items on this list

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b.B. The construction of all improvements shall be in accordance with the current specifications and regulations adopted by the City of New Braunfels.

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c.C. The applicant shall submit a preliminary Drainage and Water Quality Report with the submittal of any preliminary plat of a proposed ~~Type 3-D~~ development. A preliminary Drainage Report may also be required by the City when reviewing the merits of a change in zoning. Approval of the preliminary plat or zoning change may be contingent on the acceptability of the solutions proposed by the Drainage Report.

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d.D. The applicant shall submit a final Drainage Report with the submittal of any Final Plat, Plat Revision, or Plat Showing of a proposed development. Approval of the above mentioned plats shall be contingent on the acceptability of the solutions proposed by the final Drainage Report.

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e.E. Drainage Reports shall be prepared by a Professional Engineer licensed in the State of Texas, experienced in Civil Engineering, and having a thorough knowledge of the study of drainage issues. Drainage Reports shall be signed, sealed, and dated by the person responsible for the study.

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f.F. The City Engineer may waive the requirement of the Drainage Report or may limit certain requirements where the City Engineer determines that such requirements are not necessary for a proper review of the development.

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g.G. Requirements for Drainage Report submittals: Drainage Area Map

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1. Use a scale of one-inch equals 200 feet for the development and a scale of up to one-inch equals 2,000 feet for creeks and off-site areas, provided that the scale is adequate for review, and show match lines between any two or more maps.

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2. Show existing and proposed storm sewers and inlets.

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3. Indicate subareas for each alley, street, inlets, off-site areas, etc.

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4. Indicate contours on map for on- and off-site areas.

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5. Indicate zoning on drainage area.

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6. Show points of concentration of design points.

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7. Indicate runoff at all inlets, dead-end streets and alleys or to adjacent additions or acreage.

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8. Provide runoff calculations for all areas showing acreage, runoff coefficient, inlet time and storm frequency.

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9. Indicate all crests, sags and street and alley intersections with flow arrows.

## Requirements for Drainage Report submittals: Drainage Report

### \* Calculations

1. Show hydraulic grade lines with computations.
2. Provide table with input parameters for all models and formulas
3. Indicate all assumptions.

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### \* Storm Drain Plan and Profile Sheets (or spec page)

~~12-01.~~ Show plan and profile of all drainage elements on separate sheets from paving plans.

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~~13-02.~~ Indicate concrete cushions where applicable.

3. Specify the type of storm drainpipe to be used.
4. Indicate property lines along storm sewer and show easements with dimensions.
5. Show all existing utilities in plan view, and show those existing utilities in profile where possible conflicts may occur with the storm sewer.
6. Indicate existing and proposed ground line and improvements on all street, alley and storm sewer profiles.

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7. Show laterals on trunk profile with stations.
8. Number inlets according to the number designation given for the area or subarea contributing runoff to the inlet.
9. Indicate size and type of inlet on plan view, lateral size and flow line, paving station and top-of-curb elevation.
10. Indicate quantity and direction of flows at all inlets, stubouts, pipes and intakes.
11. Show future streets and grades, where applicable.
12. Show water surface at outfall of storm sewer, velocity and typical section of receiving water body.
13. Where fill is proposed for trench cut in creeks or outfall ditches, specify compacted fill and compaction criteria.
14. Show size of pipe, length of each pipe size, and stationing at 100-foot intervals in the plan
15. Begin and end each sheet with even or 50-foot stationing.
16. Show diameter of pipes, physical grade, discharge, capacity of pipe, slope of hydraulic grade line and velocity in the pipe in the profile view.
17. Show elevation of flow lines at 100-foot intervals on the profile.
18. Give benchmark information.
19. Show capacities, flows, velocities, etc., of the existing system into which the proposed system is being connected.
20. Show details of all connection boxes, headwalls on storm sewer, times or any other item not in a standard detail sheet.
21. Provide profile where existing utility is crossed.
22. Show headwalls and specify type for all storm sewers at outfall.
23. Show if curbing in alleys is needed to add extra capacity.
24. Runoff from alleys and other paved areas are not to cause street capacity to be exceeded.
25. Show horizontal and vertical curve data for all drainage elements.
26. Tie storm sewer stationing with paving stations.
27. On all dead-end streets and alleys, show grades for drainage overflow path on the plan and profile sheets, and show erosion controls.
28. Specify concrete strength for all structures.
29. Provide sections for road, railroad and other ditches with profiles and hydraulic computations. Show design water surface on profile.

I. Requirements for Drainage Report submittals: Bridge Plans

- A-1. Show the elevation of the lowest member of the bridge and 100-year water surface elevation.
- B-2. Indicate borings on plans.
- C-3. Provide soils report.
- D-4. Show a section at the bridge.
- E-5. Provide hydraulic calculations on all sections.
- F-6. Provide structural details and calculations with dead load deflection diagram.
- G-7. Provide vertical and horizontal alignment.
- H-8. Provide calculations and details for all erosion protection.

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\* Creek Alteration and Channel Plans

- A-1. Show stationing in plan and profile.
- B-2. Indicate flow line, banks, design water surface. Show hydraulic computations.
- C-3. Indicate the nature of banks, such as rock, earth, etc.
- D-4. Provide cross sections with ties to property lines and easements.
- E-5. Show side slopes of creeks, channels, etc.
- F-6. Specify compacted fill, where fill is proposed.
- G-7. Indicate any adjacent alley or street elevations on creek profile.
- H-8. Show any temporary or permanent erosion controls.
- I-9. Indicate existing and proposed velocities.
- J-10. Show access and/or maintenance easements.
- K-11. Identify the datum, benchmarks and date of re-leveling the benchmarks to which the flood and ground elevations are referenced.
- L-12. Show existing Finished Floor (F.F.), or proposed minimum F.F. of all structures, existing or proposed adjacent to creek or channel alternations.

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\* Environmental Effects and Required Regulatory Permits Report

- 4-01. The preliminary submittal of plans is to identify all permits that, in the design engineer's opinion, will or may be required by regulatory agencies. Such permits and agencies include, but are not limited to, NPDES (addressed in this manual in chapter 13), Section 404 permit from the U.S. Army Corps of Engineers, the Environmental Protection Agency (EPA), and Texas ~~Natural Resources Conservation Commission~~ [Commission on Environmental Quality \(TNRECEQ\)](#).
- 5-02. The final submittal of plans is to provide a list of all required permits necessary to construct the project and a copy of the approved permits.

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\* Detention and Retention Facilities

- 121. Show plan view of detention/retention area and outlet structure.
- 132. Delineate limits of conservation pool, sediment storage area, flood storage pool and/or freeboard.
- 143. Indicate size, dimensions, total capacity and design discharge velocity of the outlet structure.
- 154. Show erosion control features at the discharge point of the outlet structure.
- 165. Specify side slopes of facility and outlet structure.
- 176. Show existing or proposed structures or other facilities downstream of the outlet structure and emergency spillway, and provide information sufficient to show that the adjacent facilities will not be subjected to inundation (or increased inundation) or otherwise affected by the discharge from the facility.
- 187. Indicate locations and quantities of all inflows to the facility.
- 198. State the design time to empty the facility.

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\* Levees

- A. ~~Show location, extent, nature, dimensions, etc., of levee embankments and associated interior and exterior drainage facilities.~~

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- B. Provide engineering analysis addressing potential erosion of the levee embankments during flood events.
- C. Provide engineering analysis of levee embankment stability and seepage through the levee during flood events.
- D. Compaction of fill material should be performed in accordance with standard engineering practices.
- E. Analyze interior drainage concerns. Identify sources of interior flooding and extent and depth of such flooding. Consider capacity of pumps and other drainage devices for evacuating interior waters.
- F. Submit an operations manual which discusses the flood warning system to trigger closures; closure operations, procedures and personnel; operation plans for Interior drainage facilities; at least an annual inspection program; and maintenance plans, procedures and frequency.
- G. Provide all other information requested or required by the City Engineer and/or the Federal Emergency Management Agency.

### 2.9 Erosion Hazard Setback Regulation

Erosion hazard setback zone determination is necessary for the banks of streams in which the natural channel is to be preserved. The purpose of the setbacks is to reduce the amount of structural damage and stream degradation caused by the erosion of the bank. With the application of streambank erosion hazard setbacks, an easement is dedicated to the city such that no structure can be located, constructed, or maintained in the area encompassing the erosion hazard setback. The City of New Braunfels allows for streambank stabilization as an alternative to dedicating the erosion hazard setback zone. Streambank erosion hazard setbacks may extend beyond the limits of the regulatory floodplain. Unless different recommendations by a qualified geotechnical engineer or geologist should be presented to the City Engineer for review and approval, the following setbacks will apply on each side of the stream.

Drainage Basin Size (sq. mi)	Setback Distance from Stream Centerline (ft./ftt.)
0 - 1	0
1 - 5	50
5 or more	100

### 2.10 Finished Floor Elevations

The elevation of the lowest floor shall be at least 10 inches above the finished grade of the surrounding ground, which shall be sloped in a fashion so as to direct stormwater away from the structure. Properties adjacent to stormwater conveyance structures must have floor slab elevation or bottom of floor joists a minimum of one foot above the 100-year water flow elevation in the structure. Driveway serving houses on the downhill side of the street shall have properly sized swale before entering the garage. As defined in the building regulations and Code of Ordinances.

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**4.3.0 City of New Braunfels Comprehensive Storm Drainage Flood Protection And Erosion Control Ordinance**

The City of New Braunfels' Code of Ordinances contains requirements for the design of storm drainage, flood protection, water quality, and erosion control facilities. Where there is any conflict between this manual and the current Ordinance, the Ordinance shall take precedence. ~~Although a copy of the current Ordinance has been included in Appendix D of this Manual for reference,~~ the design engineer is responsible for complying with the latest version of the Ordinance on record with the City. It is the intent of this Manual, in concert with ~~the Drainage and Erosion Control~~applicable Ordinances, to provide all development under its jurisdiction the options of: 1) mitigation; 2) demonstrating that no mitigation is in the best interest of the watershed; or, 3) paying a share of the cost of ~~increased regional detention~~regionally scaled mitigation required ~~because due of to~~ the development.

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**A-4.0 Design Rainfall**

**4.1 Rainfall Intensity-Duration-Frequency—Rational Method**

The City evaluated area rainfall data in 2011 following procedures outlined in USGS Water Resources Investigation Report 98-4044, and which included several significant rain events that occurred after and thus were not part of the USGS analysis. The City determined the following rainfall data as most appropriate for use in this area. Rainfall intensities will be computed using the IDF equations and appropriate design frequency coefficients provided in Table 4-1. This equation is to be used only with time of concentrations between 10 minutes and 3 hours.

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1. rates for drainage design purposes shall be estimated in accordance with standard technical information provided by U.S.G.S. Water Resources Investigations Report 98-4044(2). The information, guidelines and procedures contained in these publications should be utilized by the design engineer. Rainfall information from these sources is provided in this manual for the convenience of the Engineer. If there are any discrepancies between the data in this manual and these references, the City Engineer should be contacted for clarification.
2. Point rainfall intensities can be calculated utilizing the equations listed in Tables 4-1 and 4-2. The design of storm drainage facilities within the City of New Braunfels and Comal County shall be based on rainfall information from either of the tables depending on the location.

**A. Probable Maximum Precipitation (PMP)—Dams or Impoundments**

The design rainfall for NRCS dams or impoundments is based on a percentage of the Probable Maximum Precipitation (PMP), as specified in Section 12 of this manual. PMP rainfall depths for various durations and storm sizes can be obtained from Hydro-Meteorological Reports Nos. 51(3) and 52(4), respectively. The computer calculating the basin average precipitation for each time step:

**Table 4-1  
New Braunfels Rainfall Intensity Constants**

Intensity for Durations of 10 Mmin to 3 hours			
$I=b/(Tc+d)^e$			
Year	b	d	e
2	69.7	12.03	0.857
5	61.2	9.61	0.762
10	59.8	7.69	0.720
25	64.6	7.14	0.691
50	68.4	6.40	0.673
100	74.9	5.95	0.663

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Intensity for Durations of 5 Min to 7 Days			
$I=b/(Tc+d)^e$			
Year	b	d	e
2	71.5	13.09	0.850
5	72.9	11.14	0.800
10	71.9	8.69	0.769

25	79.5	8.01	0.751
50	86.6	7.56	0.740
100	95.1	7.17	0.731
500	119.4	6.23	0.714

Note: I is rainfall intensity in inches per hour.

4.2 Rainfall Depth-Duration-Frequency

B. Standard Project Precipitation (SPP)

The design rainfall for projects which require the Corps of Engineers' Standard Project Flood (SPF) shall be obtained by applying 50 percent of the PMF, or the runoff developed from the PMF, as described in Section 4.2.

C. Hydrographs TR 20, TR 55 or HEC 1

The model of choice for the City is HEC 1 because of its compatibility with existing City studies and FEMA. When models such as in TR 20, TR 55 or HEC 1 are used to evaluate drainage, the design storm distribution shall be the standard Type II storm defined with the data from Table 4-3.

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**Table 4-2  
Comal County Rainfall Intensity Constants**

Intensity for Durations of 5 Min to 7 Days $I=b/(Tc+d)^e$			
Year	b	d	e
2	71.5	13.09	0.850
5	80.8	12.70	0.800
10	71.9	8.69	0.769
25	79.5	8.01	0.751
50	86.6	7.56	0.740
100	95.1	7.17	0.731
500	119.4	6.23	0.714

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Note: I is rainfall intensity in inches per hour.

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**Table 4-32  
New Braunfels Area Depth-Duration Values (in)**

Year	5-Min	15-Min	1-Hr	2-Hr	3-Hr	6-Hr	12-Hr	24-Hr	2-day	3-day
2	0.51	1.05	1.86	2.24	2.45	2.80	3.15	3.52	3.92	4.17
5	0.66	1.34	2.40	2.95	3.27	3.85	4.47	5.17	5.96	6.47
10	0.80	1.58	2.78	3.43	3.84	4.58	5.43	6.40	7.53	8.27
25	0.96	1.89	3.34	4.16	4.67	5.64	6.76	8.07	9.61	10.64
50	1.11	2.16	3.83	4.79	5.40	6.57	7.92	9.52	11.43	12.71
100	1.28	2.47	4.39	5.51	6.23	7.61	9.24	10.17	13.48	15.05
Year	5-Min	15-Min	1-Hr	2-Hr	3-Hr	6-Hr	12-Hr	24-Hr	2-day	3-day
2-00	0.51	1.05	1.65	2.19	2.44	2.87	3.10	3.34	3.98	4.31
5-00	0.66	1.34	2.33	3.08	3.48	4.17	4.45	4.83	5.66	6.20
10-00	0.80	1.58	2.84	3.72	4.23	5.11	5.49	6.06	7.00	7.72
25-00	0.96	1.89	3.56	4.61	5.26	6.41	7.07	8.06	9.08	10.11
50-00	1.11	2.16	4.17	5.32	6.10	7.47	8.50	9.96	10.99	12.34
100-00	1.28	2.47	4.84	6.08	6.99	8.61	10.18	12.30	13.30	15.04

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D: Rainfall Loss Rate

The method used to calculate the rainfall losses will depend on the method used to compute the design discharge. The Rational Method accounts for rainfall losses with the C coefficient, as description in Section 5.3.2. For the unit hydrograph methods described in Section 5.4, the method used is described in the respective user manual with SCS values for selected conditions are provided for standardization.

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## A-5.0 Determination of Design Discharge

### 5.1 General

The selection of thean appropriate method for calculating runoff depends upon the size of the drainage area, and time of concentration, and detention mitigation, of the drainage area contributing runoff to the most downstream point of a project. Flows are to be developed for both existing and proposed conditions at all locations where runoff leaves a proposed project for the 2, 5, 10, 25, 50, and 100-yr frequencies.

Design discharges are to be developed by the either two methodologies: Rational Method, or unit HydrographUnit Hydrograph.

### 5.2 Impact of Runoff on Downstream Facilities

No proposed development shall be constructed which impedes or constricts runoff from an upstream watershed based on fully developed conditions. The developer and City have the option to partner on downstream improvements.

### 5.3 Procedure for ~~t~~The Rational Method

Rational Method equation is based on the following assumptions:

- A.a. Rainfall intensity is constant over the time it takes to drain the watershed (time of concentration)
- B.b. The runoff coefficient remains constant during the time of concentration.
- C.c. The watershed area does not change.

The Rational Method shall be used to generate peak flows for drainage basins less than 150 acres that do not require detention or timing considerations, with a time of concentration of 20 minutes or less, may be by the Rational Method. The discharge computed by the Rational Method is the peak discharge for a given frequency on the watershed in question, and is given by the following relationship:

$$Q = K * C * I * A \quad (\text{Eq. 5-1})$$

where:	Q	is the peak design <u>discharge in cubic feet per second</u> for a given frequency on the watershed at the desired design point <u>(cfs).</u>
	C	<u>is a dimensionless weighted runoff coefficient, representing ground cover conditions and/or land use within the watershed area. (See Table 5-2.)</u> <u>is a dimensionless coefficient to account for storms with recurrence intervals greater than 10 years. (See Table 5-1.)</u>
	I	<u>is the average rainfall intensity in inches per hour at a rainfall duration equal to the time of concentration, associated with the desired design frequency. (See Table 4-1) (in/hr)</u> <u>is a dimensionless weighted runoff coefficient, representing ground cover conditions and/or land use within the watershed area. (See Table 5-2.)</u>
	A	<u>is the drainage area in acres contributing runoff to the desired design point. (ACRES)</u> <u>is the average rainfall intensity in inches per hour at a rainfall duration equal to the time of concentration, associated with the desired design frequency. (See Tables 4-1 and 4-2.)</u>
	A	<u>is the drainage area in acres contributing runoff to the desired design point.</u>

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### 5.3.1—Antecedent Precipitation Coefficient

The runoff computations should include the antecedent precipitation coefficient "K", as identified in Table 5-1. This coefficient is intended to reflect the additional runoff that results from saturated ground conditions. In no case should the product of the runoff coefficient and the antecedent precipitation coefficient exceed 1.0.

5.2.1 Runoff Coefficient

A. Runoff Coefficient

A. The American Society of Civil Engineers (ASCE) has compiled average runoff coefficients used in the Rational Method for various surface conditions (6). For the pre-development case, runoff coefficients shown in Table 5-2 are available. For the post-developed case, runoff coefficients shown in Table 5-7 are recommended.

Table 5-1  
Values of Antecedent Precipitation Coefficient "K"

Frequency	Value of K
10 year or less	1.0
25 year	1.1
100 year	1.25

Table 5-2  
Runoff Coefficients

Area (Developed)	"C"	Area (Undeveloped)	"C"
Grass (Lawns, Parks) Poor <50% cover		Cultivated	
Flat 0-2%	0.37	Flat 0-2%	0.36
Average 2-7%	0.43	Average 2-7%	0.41
Steep, >7%	0.45	Steep, >7%	0.41
Grass (Lawns, Parks) Fair 50%-75% cover		Pasture/Range	
Flat 0-2%	0.30	Flat 0-2%	0.30
Average 2-7%	0.38	Average 2-7%	0.38
Steep, >7%	0.42	Steep, >7%	0.42
Grass (Lawns, Parks) Good >50% cover		Forest/Woodlands	
Flat 0-2%	0.25	Flat 0-2%	0.28
Average 2-7%	0.35	Average 2-7%	0.36
Steep, >7%	0.40	Steep, >7%	0.41
Asphaltic	0.81		
Concrete/Roof	0.83		

1 B. The drainage area under investigation may consist of several different drainage surfaces or zoning classifications. In such cases, an average coefficient weighted in accordance with the respective areas ~~should~~shall be used, as outlined in equation 5-2.

$$C_w = (A_1 \cdot C_1 + A_2 \cdot C_2 + \dots + A_n \cdot C_n) / (A_1 + A_2 + A_3 + \dots + A_n) \quad (\text{Eq. 5-2})$$

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5.2.2 Time of Concentration

A. The time of concentration ( $t_c$ ) is the amount of time required for surface runoff to travel from the most hydraulically remote point within the drainage basin to the drainage point under consideration. The most hydraulically remote drainage point refers to the route requiring the longest drainage travel time and not necessarily the greatest linear distance. Furthermore, the most hydraulically remote point must be taken from a location that best represents majority of the contributing area.

1. The flow routes used in determining the time of concentration must take into consideration fully developed conditions as proposed by thoroughfare plans, zoning maps, etc.

2. When computing the  $t_c$ , the maximum length of overland flow shall be 300 feet, at which point flow shall be considered shallow concentrated flow or channelized flow. Table 5-3 contains initial  $t_c$  at all inlets (i.e., prior to entering a pipe systems or channel). These values are derived from initial times of concentration typically used in Rational Method calculations, and are to be used for the first 300 feet of travel distance.

**Table 5-3**  
**Initial Times of Concentration**

Type of Area	Minimum Inlet Time Minutes
Parks & Open Areas	20
Residential	
— Single Family	15
— Multi Family	10
Commercial/Business	10
Roofs & Paved Areas	5

The City accepts either Kerby-Kirpich or TR-55 as the primary methods for computing time of concentration. Refer to Section 4-11 of the TxDOT Hydraulic Design Manual (HDM, Dated October 2012/2011 or later) for methods of computing either method. Other methodologies can be used but must be approved by the City Engineer.

The minimum time of concentration is to not be less than 10 minutes nor exceed 3-hours when using the Rational Method. It should be noted that when determining the time of concentration, the flow route may consist of several segments. The total time of concentration is determined as follows:

$$t_c = \text{Initial } t_c + T_{t1} + \dots + T_{tn} \quad \text{(Eq. 5-3)}$$

where:

$t_c$	Time of concentration (min)
Initial $t_c$	The initial time of concentration from Table 5-3 (min)
$T_{t1}$	The travel time of Segment I (min)

Overland flow velocity can be calculated by using Table 5-4 in the following equations:

Sheet Flow

$$T_t = (60 * L * n) / (288.6 * S^{0.4}) \quad \text{Eq. 5-4a}$$

Shallow Concentrated Flow

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$$T_c = (L*n)/(60*S^{0.5}) \quad \text{Eq. 5-4b}$$

Channel or Sewer Flow

$$T_c = (L*A)/(60*Q) \quad \text{Eq. 5-4c}$$

where:

$T_c$	Segment time of concentration (min)
$n$	Manning "n" from Table 5-4
$S$	Slope of the land over which the runoff will flow (ft/ft)
$V$	Average velocity (ft/s)
$Q$	Design discharge (cfs)
$A$	Cross-sectional area (ft)

a. Alternative Procedure Hydrographs

D.

~~1-A.~~ For drainage areas in excess of 150 acres, ~~times of concentration greater than 20 minutes, or in instances of sizing large open channels, reclaiming floodplains, creating lakes or building other types of drainage-related facilities on major drainage courses where the use of the Rational Method does not provide reliable results, a unit hydrograph method shall be used.~~

~~2.~~ FEMA's flows shall not be used.

~~3-B.~~ The preferred unit hydrograph in general is the Natural Resource Conservation Service (SCS/NRCS) Dimensionless Unit Hydrograph. Other hydrograph methodologies may be used upon the approval of the City Engineer. Details of the Snyder's Unit Hydrograph are provided for contrast.

5.4.1 SCS/NRCS Unit Hydrograph

~~1-A.~~ The procedures for the Soil Conservation Service (SCS) method are outlined in Section 4 of the National Engineering Handbook (13) and in Section 13 of the TxDOT Hydraulic Design Manual HDM released October 2011, numerous hydrology textbooks. The designer is responsible for obtaining a copy of the user manual for the SCS method. The SCS method uses a dimensionless unit hydrograph applied to the peak discharge computed for a given watershed.

B. Runoff Coefficient. The runoff curve number used in developing the pre-development discharge shall be documented. The runoff curve number and contributing area shall reflect the fully developed sub-basin or watershed drainage area.

C. NRCS curve numbers are to be selected from Tables 4-19 through 4-22 of the TxDOT hydraulic design manual dated October 2011 HDM. CN's CN's in Table 5-3, Table XX are to shall be used when performing an analysis of fully developed conditions.

D. Curve numbers can be reduced by either using a climatic adjustment as identified on Figure 4-21 of the TxDOT HDM or calibrating to historical storms. If -CN's are calibrated from historical storms, the Engineer must provide documented data for rainfall, stream flow data, or detention pond stage storage data used to determine the historical CN.

E. Time of concentration. Time of concentration shall be computer computed using the same techniques as for the Rational Method.

F. The NRCS unit hydrograph shall be analyzed using 24-hr rainfall depths provided in Table 4-2 of this criteria manual. The 24-hr rainfall depths are to be distributed temporally with the NRCS Type III rainfall distribution. No other hyetographs or durations shall be used unless approved by the City Engineer.

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2. Table 5-5 contains runoff curve numbers. Antecedent Moisture Condition III (AMC III) shall be used for the 100-year even (9, 14). For a listing of applicable soil types, refer to the United States Department of Agriculture, Soil Conservation Service, Soil Survey of Fom Green County, Texas (15).

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**Table 5-4**  
**Manning's "n" for Overland Flow and Shallow Concentrated Flow**

Condition	"n"
Concrete (rough or smoothed finish)	0.016
Asphalt	0.02
0-50% vegetated ground cover, remaining bare soil or rock outcrops, minimum brush or tree cover	0.1
50-90% vegetated ground cover, remaining bare soil or rock outcrops, minimum medium brush or tree cover	0.2
100% vegetated ground cover, medium dense grasses (lawns, grassy fields, etc) medium brush or tree cover	0.3
100% vegetated ground cover with areas of heavy vegetation (parks, greenbelts, riparian areas, etc) dense undergrowth with medium to heavy tree growth	0.619

**Table 5-5A**  
**General SCS Runoff Curve Numbers**

Cover Type*		CN (AMC III)	
		C	D
Open space—lawns, parks, golf courses	Poor condition (grass cover <50%)	86	89
	Fair condition (grass cover 50-75%)	79	84
	Good condition (grass cover >75%)	74	80
Impervious	Paved	98	98
	Gravel	89	91
	Dirt	87	89
Urban	Commercial and business	94	95
	Industrial	91	93
Residential	1/6 acre lot size	90	92
	1/4 acre lot size	83	87
	1/2 acre lot size	80	85
	1 acre lot size	79	84
	2 acre lot size	77	82
Pasture, grassland, or range continuous Forage	Poor	86	89
	Fair	79	84

	Good	74	80
Meadow continuous generally mowed for hay		71	78
Brush-brush, weed, grass mix	Poor	77	83
	Fair	70	77
	Good	65	73
Woods grass combination (orchard, tree Farm)	Poor	82	86
	Fair	76	82
	Good	72	79
Woods	Poor	77	83
	Fair	73	79
	Good	70	77
Farmstead		82	86

**Table 5-3  
Fully Developed Runoff Coefficients**

Zone	CN (AMCII)				
	"C"	A	B	C	D
R-1/R-1A Single family	0.53	61	75	83	87
R-2/R-2A Single and two family	0.59	77	85	90	92
R-3/R-3L Multi family high density	0.67	77	89	92	94
R-3/R-3H Multi family low density	0.55	77	85	90	92
B-1/B-1A Convent & mobile homes	0.53	61	75	83	87
TH/TH-A Townhouse	0.67	77	89	92	92
ZH/ZH-A Zero lot line homes	0.55	68	79	87	90
C-1/C1A Neighborhood business	0.67	83	89	92	93
C-2/C-1B General Business	0.68	77	86	93	94
C-3 Commercial	0.80	89	92	94	95
C-4/C-4A Resort commercial / PUD*					
M-1/M1A Light industry	0.72	68	79	87	90
M-2/M-2A Heavy industry	0.78	89	92	94	95

\*must use composite values based on % impervious.

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**Table 5-5B**  
**Farming-SCS Runoff Curve Numbers**

Cover Type*		CN (AMC-III)	
		C	D
Fallow	Base soil Crop residue cover (CR)	Poor	91 94
		Good	90 93
Row crops	Straight row (SR)	Poor	88 91
		Good	85 89
	SR + CR	Poor	87 90
		Good	82 85
	Contoured (C)	Poor	84 88
		Good	82 86
	C + CR	Poor	83 87
		Good	81 85
Small grain	SR	Poor	84 88
		Good	85 89
	SR + CR	Poor	83 86
		Good	80 84
	C	Poor	82 85
		Good	81 84
	C + CR	Poor	81 84
		Good	80 83
Close-seeded or broadcast Or rotation meadow	SR	Poor	85 89
		Good	81 85
	C	Poor	83 85
		Good	78 83

\*From 210 VI TR 55, Second Ed., June 1986.

1. ~~Time of concentration. Time of concentration shall be computer using the same techniques as for the Rational Method.~~

1. ~~Snyder's Unit Hydrograph~~

1. ~~This method, detailed in the U.S. Army Corps of Engineers Engineering Manual (EM-1110-2-1405), Flood Hydrograph Analysis and Computations (10) and Paul Rodman's paper, "The Effects of Urbanization on Various Frequency Peak Discharges" (22), utilizes the following equations:~~

$$t_p = C_t (L L_{ca})^{0.34} / S^{1/2} \quad \text{(Eq. 5-5)}$$

$$t_r = t_p / 5.5 \quad \text{(Eq. 5-6)}$$

$$q_p = C_p 640 / t_p \quad \text{(Eq. 5-7)}$$

$$t_{pR} = t_p + 0.25 (t_r - t_r) \quad \text{(Eq. 5-8)}$$

$$q_{pR} = C_p 640 / t_{pR} \quad \text{(Eq. 5-9)}$$

$$q_{pR} = q_p t_p / t_{pR} \quad \text{(Eq. 5-10)}$$

$$Q_p = q_p A \quad \text{(Eq. 5-11)}$$

Where:

- $t_r$  Is the unit rainfall duration in hours other than standard unit,  $t_r$ , adopted in specific study.
- $t_p$  Is the lag time from midpoint of unit rainfall duration,  $t_r$ , to peak of unit hydrograph in hours
- $t_{pR}$  Is the lag time from midpoint of unit rainfall duration,  $t_r$ , peak of unit hydrograph in hours

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$q_p$	Is the peak rate of discharge of unit hydrograph for unit rainfall duration, $t_r$ , in cfs/sq mi
$q_{pR}$	Is the peak rate of discharge in cfs/sq mi of unit in hydrograph for unit rainfall duration, $t_r$
$Q_D$	Is the peak rate of discharge of unit hydrograph in cfs
A	Is the drainage area in square miles
$L_{ca}$	Is the river mileage from the design point to the centroid of gravity of the drainage area
L	Is the river mileage from the given station to the upstream limits of the drainage area.
S	Is the average slope of the streambed in ft/mi

2. The coefficient  $C_c$  is a regional coefficient for variations in slopes within the watershed. Typical values of  $C_c$  range from about 0.5 to 2.0 and a representative value for the New Braunfels region 0.8 (11).  $C_c$  for a watershed can be estimated if the lag time,  $t_p$ , stream length, L, distance to the basin central,  $L_{ca}$ , and the streambed slope (ft/mi) are known. These values reflect no significant urbanization of the watershed.

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3. The coefficient  $C_p$  is the peaking coefficient, which typically ranges from 0.3 to 1.2 with a representative value for the New Braunfels area of 0.63 (11), and is related to the flood wave and storage conditions of the watershed. Larger values of  $C_p$  are generally associated with smaller values of  $C_c$ . Typical values of  $C_p$  are listed in Table 5-6.

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4. Urbanization Curves. To account for the effects of urbanization, another method was developed by the Corps of Engineers to adjust the  $t_p$  coefficient. Urbanization curves allow for the determination of  $t_p$  based on the percent urbanization and the type of soil in the study area. They were determined from the equation below (12):

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$$T_p = 10^{((0.3833 * \log(L * L_{ca} / (S_{st})^{0.5}) + \log(I_p) - (BW * \%Urb) / 100)} \quad (\text{Eq. 5-12})$$

$$S_{st} = (el_{85\%} - el_{15\%}) / (0.7 * L) \quad (\text{Eq. 5-13})$$

Where:

$t_p$	Is the lag time from midpoint of unit rainfall duration, $t_r$ , to peak of unit hydrograph in hours.
$L_{ca}$	Is the river mileage from the design point to the centroid of the drainage area
L	Is the river mileage from the given station to the upstream limits of the drainage area.
$S_{st}$	Is the weighted slope of the flow path (ft/mi)
$I_p$	Is the calibration point, defined as the log of $t_p$ where $(L * L_{ca} / S_{st}^{0.5}) = 1$ and urbanization = 0%
BW	Is the bandwidth, equal to the log of the width between each 20% urbanization line.
%Urb	Is a value representative of the degree to which urbanization has occurred in the basin, in percent
$el_{85\%}$	Is the elevation at a location 85% upstream of the given station
$el_{15\%}$	Is the elevation at a location 15% upstream of the given station

For the New Braunfels area, the  $I_p$  values used are 1.03 for the Comfort Soils Group, 1.12 for the Rumpel/Echran Soils Group, and 0.94 for the Hieden/Houston Black Soils Group. The bandwidth value (BW) for both of the soil types is 0.266. For a study area that is composed of more than one type of soil group, a weighted average can be used.

**Table 5-6**  
**Typical Values of  $C_p$**

<u>Drainage Area Characteristics</u>	<u>Value of <math>C_p</math></u>
<u>Undeveloped Areas with Storm Drains</u>	
Flat Basin Slope (less than 0.50%)	0.55
Moderate Basin Slope (0.50% to 0.80%)	0.58
Steep Basin Slope (greater than 0.80%)	0.61
<u>Moderately Developed Area</u>	
Flat Basin Slope (less than 0.50%)	0.63
Moderate Basin Slope (0.50% to 0.80%)	0.66
Steep Basin Slope (greater than 0.80%)	0.69
<u>Highly Developed/Commercial Area</u>	
Flat Basin Slope (less than 0.50%)	0.70
Moderate Basin Slope (0.50% to 0.80%)	0.73
Steep Basin Slope (greater than 0.80%)	0.77

5. ~~Design runoff may be determined for a given watershed by applying the intensity duration frequency relationships from Tables 4-1 and 4-2 to the unit hydrograph by multiplying each ordinate of the unit hydrograph by the rainfall intensity.~~

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~~1. Fully Developed Runoff Conditions~~

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~~Table 5-7 provides the runoff coefficients the fully developed watershed case by zoning category.~~

~~1.5.5 Hydrologic Computer Programs~~

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~~1.A. The preferred hydrologic model for the City is HEC-HMS. The use of other computer modeling software is to be approved by the City Engineer.The recognized computer models are T-20, TR-55 and HEC-1. When using any model use the procedures outlined in the respective user's manual. Data generated with the model and the results of the program shall be summarized on the drainage plans. Hydrologic data obtained from the Corps of Engineers in the NUDALLAS format shall be converted to the HEC-1 format.~~

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~~2. In using the Soil Conservation Service's TR-20 program for hydrograph analysis use the procedures outlined in Technical Release No. 20 (17).~~

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**Table 5-7**  
**Fully Developed Runoff Coefficients**

Zone	"C"	CN (AMCII)	
		€	Ð
R-1/R-1A Single family	0.53	83	87
R-2/R-2A Single and two family	0.59	90	92
R-3/R-3L Multi-family high density	0.67	92	94
R-3/R-3H Multi-family low density	0.55	90	92
B-1/B-1A Convent & mobile homes	0.53	83	87

TH/TH A Townhouse	0.67	92	92
ZH/ZH A Zero-lot-line homes	0.55	87	90
C-1/C1A Neighborhood business	0.67	92	93
C-2/C-1B General Business	0.68	93	94
C-3 Commercial	0.80	94	95
C-4/C-4A Resort commercial / PUD*			
M-1/M1A Light industry	0.72	87	90
M-2/M-2A Heavy industry	0.78	94	95

\*must use composite values based on % impervious.

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## 4-6.0 Street Flow

### 6.1 Lot-to-Lot Drainage

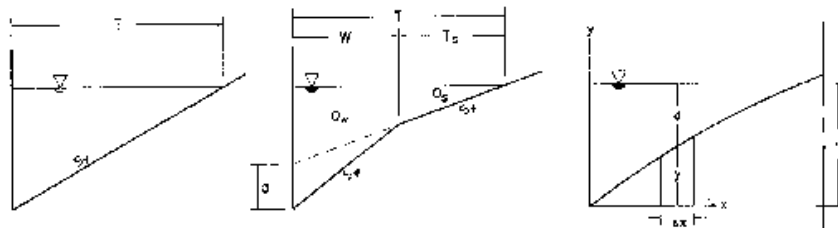
Existing drainage between developed lots will remain the responsibility of the affected property owners. Developments are required to drain surface runoff from an individual lot to a public right-of-way or to a drainage system contained in an an-public easement.

### 4-6.2 Positive Overflow

The approved drainage system shall provide for positive overflow at all low points. The term "positive overflow" means that, when the inlets do not function properly or when the design capacity of the conduit or roadway ditch is exceeded, the excess flow can be conveyed overland along an open course. Normally, this would mean along a street or alley, but it can be constructed on private property within the dedication of a special drainage easement.

### 2-6.3 Street Flow Calculations

Evaluation of street flow is based upon open channel hydraulics theory, with the Manning's equation modified to allow direct solution, based on the street cross section. Generally, the street will have a straight or parabolic section. Refer to Chapter 10, Section 4 of the TxDOT HDM. All proposed projects must meet the ponding criteria defined in sectionSection 2-2 of this drainage criteria manual. Provided below are Table 6-1 lists the typical dimensions for streets used in the City. Figure 6-1 has been prepared for gutter flow capacity as divided into two types, triangular and parabolic. These street flow calculations are dependent on the shape of the street. Flow calculations for inverted streets shall use the methods outlined in Section 6.6 of this manual.



a. Triangular — b. Triangular with Gutter — c. Parabolic

Figure 6-1. Typical Gutter Cross Sections

### 1. Flow Calculations for Straight Street Sections

The direct solution for gutter flow depth for a given flow in straight sections (triangular channel) is based upon the following formula:

$$Q = 0.56 * S^{0.5} * y^{8/3} / (n * S_x) \quad \text{(Eq.6-1)}$$

$$y = [(Q * n * S_x) / (0.56 * S^{1/2})]^{3/8} \quad \text{(Eq.6-2)}$$

$$T = y / S_x \quad \text{(Eq.6-3)}$$

$$V = (2 * Q) / (T^2 * S_x) \quad \text{(Eq.6-4)}$$

Where:  $Q$  Is the gutter discharge (ft<sup>3</sup>/sec)  
 $y$  Is the flow depth in the gutter (ft)

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- $S_x$  Is the crown slope (ft/ft)
- $n$  Is Manning's coefficient of roughness, usually 0.016 for streets
- $S$  Is the slope of the gutter (ft/ft)
- $T$  Is the spread of flow or ponding width (ft)
- $V$  Is the velocity of flow (ft/sec)

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## 2. Flow Calculations for Parabolic Street Sections

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When possible, the 100-yr flow shall be contained within the City's Right of Way or easements.

The flow capacity in parabolic street section is less than that of a triangular street section for the same slope and spread width but parabolic street sections provide a flatter driving surface for traffic toward the center of the street and are commonly used for urban streets. Depth of flow for a given flow quantity, in only one side of the street, for parabolic street sections of 26 through 100 feet of width may be determined from the equations for constants in Table 6-4 and in used in equations 6-5 through 6-9.

$$y = ((Q / S^{0.5})^{0.62}) / C1 \quad \text{(Eq.6-5)}$$

$$Q = (y * C1)^{1.62} * S^{0.5} \quad \text{(Eq.6-6)}$$

$$T = B - ((B^2 - (D^2 / c_h) * y)^{0.5}) \quad \text{(Eq.6-7)}$$

$$A = (T * y) / 3 \quad \text{(Eq.6-8)}$$

$$V = Q / A \quad \text{(Eq.6-9)}$$

- Where:
- $y$  Is the flow depth in the gutter for one side of the street (ft)
  - $Q$  Is the gutter discharge for one side of the street
  - $S$  Is the slope of the gutter (ft/ft)
  - $C1$  Is a constant for which an equation is given in table 6-4
  - $C2$  Is a constant for which an equation is given in table 6-4
  - $T$  Is the spread of flow for one side of the street (ft)
  - $B$  Is the one half of the street width (ft)
  - $c_h$  Is the crown height of the street (ft)
  - $A$  Is the cross-section area of flow (ft<sup>2</sup>)
  - $V$  Is the velocity of the flow (ft/sec)

**Table 6-1**  
**Street Types and Abbreviations**

Street Type	Pavement Width (Ft)	Crown (in)
Arterial	65	6
	64 (4ft median)	6
	48	4
Collector	45	4
Local	42	4
	40	4

**Table 6-2**  
**Equations for Spread Constant C1 and C2**

Constant and Crown	Equation
C1 (4" crown)	$3.676 * 10^{-05} * B^3 - 5.594 * 10^{-03} * B^2 + 0.4215 * B + 6.3775$
C2 (5" crown)	$3.321 * 10^{-05} * B^3 - 5.077 * 10^{-03} * B^2 + 0.3842 * B + 5.7688$
C2 (6" crown)	$3.056 * 10^{-05} * B^3 - 4.690 * 10^{-03} * B^2 + 0.3562 * B + 5.3162$
C2 (7" crown)	$2.848 * 10^{-05} * B^3 - 4.386 * 10^{-03} * B^2 + 0.3341 * B + 4.9615$
C2 (8" crown)	$2.679 * 10^{-05} * B^3 - 4.138 * 10^{-03} * B^2 + 0.3161 * B + 4.6734$
C2 (9" crown)	$2.546 * 10^{-05} * B^3 - 3.938 * 10^{-03} * B^2 + 0.3012 * B + 4.4320$
C2 (10" crown)	$2.420 * 10^{-05} * B^3 - 3.755 * 10^{-03} * B^2 + 0.2881 * B + 4.2287$
C2 (11" crown)	$2.315 * 10^{-05} * B^3 - 3.601 * 10^{-03} * B^2 + 0.2769 * B + 4.0521$
C2 (12" crown)	$2.224 * 10^{-05} * B^3 - 3.467 * 10^{-03} * B^2 + 0.2671 * B + 3.8973$
C2 (4-12" crown)	$6.99 * 10^{-08} * B^3 - 1.12 * 10^{-05} * B^2 + 6.50 * B + 0.3337$

Note: These equations/constants were derived for a Manning's n of 0.016, for total street widths of 26 through 100 feet.

In cases where the crown of a parabolic is not centered, street flows for each side of the crown will be determined independently using a distance from the curb to the crown as street half width B. This method will also allow calculation of flow in instances where a curb and gutter one side of the street is a higher elevation than the curb and gutter on the other side of the street.

**Table 6-3**  
**Minimum Crown Heights and various C1 and C2 Values**

Crown Height	Min Street Width (ft)	Max Street Width (ft)	Typical Street Width (ft)	Half Street Width (ft)	C1	C2
4"		33.33'	26'	13'	10.9924	0.3404
5"	33.34'	41.67'	36'	18'	11.2331	0.3422
6"	41.68'	50.00'	44'	22'	11.2080	0.3433
7"	50.01'	58.33'	52'	26'	11.1837	0.3443
8"	58.34'	66.67'	60'	30'	11.1555	0.3450
9"	66.68'	75.00'	-	-	-	-
10"	75.01'	83.33'	80'	40'	11.1783	0.3463
11"	83.34'	91.67'	-	-	-	-
12"	91.67'	100.00'	100'	50'	11.3648	0.3469

### 3-6.4 Alley Flow Limitations

Figure 6-2 shows the various typical sections. Alley capacities shall be checked at all alley turns and "T" intersections to determine if curbing is needed or grades should be flattened. Curbing shall be required for at least 10 feet on either side of an inlet in an alley and on the other side of the alley so that the top of the inlet is even with the high edge of the alley pavement. Alleys adjacent to drainage channel shall be required to have curbs for the full length of the channel.

### 4-6.5 Alley Flow Calculations

Flow in alleys is also based upon open channel hydraulic theory, with the Manning equation modified to allow direct solution, with regard to the alley cross section.

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The depth for the triangular cross section in Figure 6-3 a can be calculated by the following equation:

$$y = (Q * n * (1 + z^2)^{1/2} / (0.936 * z^{5/3} * S^{1/2}))^{3/8} \quad \text{(Eq. 6-20)}$$

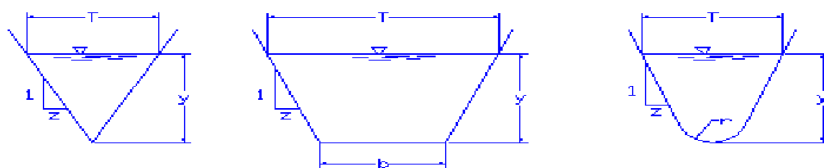
$$T = 2 * z * y \quad \text{(Eq. 6-21)}$$

$$A = z * y^2 \quad \text{(Eq. 6-22)}$$

$$V = 0.936 * z^{2/3} * y^{2/3} * S^{1/2} / (n * (1 + z^2)^{1/2}) \quad \text{(Eq. 6-23)}$$

$$Q = 0.936 * z^{2/3} * y^{8/3} * S^{1/2} / (n * (1 + z^2)^{1/2}) \quad \text{(Eq. 6-24)}$$

Where:	Q	Is the discharge (ft <sup>3</sup> /sec)
	y	Is the depth of flow (ft)
	z	Is the inverse of the slope of the crown slope (ft/ft)
	n	Is Manning's coefficient of roughness, usually 0.016 for streets
	S	Is the slope of the gutter (ft/ft)
	T	Is the spread of flow or ponding width (ft)
	V	Is the velocity of flow (ft/sec)



a. Triangular

b. Trapezoidal

c. round

Figure 6-2 Typical Alley Cross Sections

Trapezoidal cross sections like the one shown in Figure 6-3b require an iterative process of depth using the Manning's equation for discharge. Once the depth of flow is determined, it can be used to calculate top width, area, and velocity using the following equations:

$$T = b + 2 * z * y \quad \text{(Eq. 6-25)}$$

$$A = (b + 2 * z * y) * y \quad \text{(Eq. 6-26)}$$

$$V = Q / A \quad \text{(Eq. 6-28)}$$

$$Q = (1.486 / n) * S^{1/2} * ((b + z * y) * y)^{5/3} / ((b + z * y) * (1 + z^2)^{1/2}) \quad \text{(Eq. 6-29)}$$

Note: y must be solved interactively using equation 6-24)

Where:	Y	Is the depth of flow (ft)
	Z	Is the inverse of the slope of the crown slope (ft/ft)
	T	Is the spread of flow or ponding width (ft)
	b	Is the bottom width of the trapezoid (ft)
	n	Is Manning's coefficient of roughness, usually 0.016 for streets
	S	Is the slope of the gutter (ft/ft)
	V	Is the velocity of flow (ft/sec)
	Q	Is the discharge (ft <sup>3</sup> /sec)

The normal depth for a round-bottomed, triangular cross section shown in Figure 6-3c can be calculated if the top width is known. To determine the top width, a trial-and-error approach must be taken using the Manning's equation:

$$Q = (1.486 / n) * S^{1/2} * ((T / (1 + z)) * (1 - z * \cot^{-1} z))^{5/3} \quad \text{(Eq. 6-30)}$$

$$\left( \frac{T}{2} * (1+z^2)^{1/6} - (2 * r / z) * (1 - z * \cot^{-1} z) \right)^{2/3}$$

$$y = \frac{(T/2) - r * (1 + z^2) + r}{z} \quad \text{(Eq. 6-31)}$$

$$A = (T / (1 * z)) - (r^2 / z) * (1 - z * \cot^{-1} z) \quad \text{(Eq. 6-32)}$$

$$V = Q / A \quad \text{(Eq. 6-33)}$$

Where:	T	Is the spread of flow or ponding width (ft)
	b	Is the bottom width of the trapezoid (ft)
	n	Is Manning's coefficient of roughness, usually 0.016 for streets
	S	Is the slope of the gutter (ft/ft)
	V	Is the velocity of flow (ft/sec)
	Q	Is the discharge (ft <sup>3</sup> /sec)
	r	Is the radius of curvature in the bottom of the alley (ft)

5. Computer Models

6.6

Computer models such as Haestad FlowMaster may be used when appropriate to the situation.

- A. The use of computer modeling software is to be approved by the City Engineer. When using any model use the procedures outlined in the respective user's manual. Data generated with the model and the results of the program shall be summarized on the drainage plans.

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## 7.0 Inlet Design

### 7.1 Inlet Design Considerations

**1.A.** Inlets shall be located as necessary to remove the flow based on the 1025-year storm and accommodate ponding widths in streets as defined in Table 2-1 located in Section 2. The hydraulic efficiency of storm drain inlets varies with the amount of gutter flow, street grade, street crown and the geometry of the inlet opening. The following are some considerations which must be given attention during inlet design:

1. Inlets must be located where the allowable street flow capacities are exceeded, at low points (sumps or sags) and upstream of transition between normal and super-elevated street sections.
2. In super-elevated sections of divided arterial streets, inlets placed against the center medians shall have no gutter depression. Interior gutter flow (flow along the median) shall be intercepted at the point of super-elevation transition, to prevent pavement cross flow.
3. At bridges with curbed approaches, gutter flow shall be intercepted prior to flowing onto the bridge, to prevent ice from developing during cold weather.
4. The maximum approved vertical inlet opening is six inches. Openings larger than six inches require approval of the City Engineer and, if approved must contain a bar or other form of restraint to prevent entry by a child.
5. The design and location of all inlets must take into consideration pedestrian and bicycle traffic.
6. Combination curb inlets (with opening in curb and grate opening in gutter) may be used only where space behind the curb prohibits the use of other inlet types.
7. Where recessed inlets are required, they shall not decrease the width of the sidewalk or interfere with utilities.
8. Recessed inlets must also be depressed, unless otherwise approved by the City Engineer. The maximum allowable inlet depression for recessed inlets shall be seven inches.
9. Non-recessed, depressed inlets shall have a maximum allowable inlet depression of five inches.
10. The use of slotted drains is not allowed except in instances where there is no alternative, in which case approval must be obtained from the City Engineer. If slotted drains are used, the inlet capacity shall be the lesser of the calculated capacity from this manual or the manufacturer's design guidelines.

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## 7.2 Inlet Types and Descriptions

Stormwater inlets are used to remove surface runoff and convey it to a storm drainage system. For the purposes of this manual, inlets are divided into four classes:

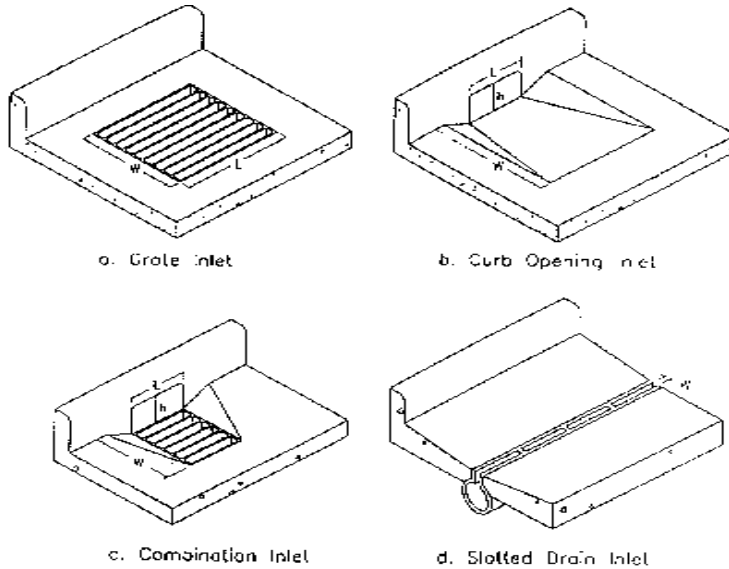


Figure 7-1. Inlet Types

- 1-a. \_\_\_\_\_ grate inlets
- 2-b. \_\_\_\_\_ curb-opening inlets and type y-inlets
- 3-c. \_\_\_\_\_ combination inlets
- 4-d. \_\_\_\_\_ slotted inlets

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1-A. **Grate Inlets:** Although grate inlets may be designed to operate satisfactorily in a range of conditions, they may become clogged by floating debris during storm events. In addition, they can produce a hazard to wheel chair and bicycle traffic and must be designed to be safe for both. For these reasons, they may be used only at locations where space restriction prohibit the use of other types of inlets, should be designed to be twice as large as the theoretical required area, to compensate for clogging, and must be approved by the City Engineer.

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2-B. **Curb Inlets and Type-Y Inlets.** Curb inlets are the most effective type of inlet on slopes flatter than 3%, in sag locations, and with flows, which typically carry large amounts of debris. Similar to grate inlets, curb inlets also tend to lose capacity as street grades increase, but to a lesser degree than grate inlets, curb inlets also tend to lose capacity as street grades increase, but to a lesser degree than grate inlets. Curb inlets are recommended in grades less than 3% and in sag locations. Type-Y inlets are most often used in drainage of swales and sags. Both curb and Type-Y inlets are bicycle safe.

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3-C. **Combination Inlets.** A combination inlet consists of both the grate inlet and the curb inlet. This configuration provides many of the advantages of both inlet types. The combination inlet also reduces the chance of clogging by debris with flow into the curb

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portion of the inlet. If a curb opening is extended on the upstream side of the combination inlet it will act as a "Sweeper", and remove debris before it reaches the grate portion of the inlet.

~~D. Slot inlets. Although slotted drains can be used to intercept sheet flow, or flow in wide sections, they are not recommended for use in the City of New Braunfels since they are very susceptible to clogging from debris. Slot inlets may only be used with the permission of the City Engineer.~~

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~~4. If slot inlets are allowed, the inlet capacity shall be calculated by both equations for a curb inlet, Grate Inlets on Grade, and the manufacturer's design guidelines, and the lesser inlet capacity, or more conservative method, shall be used for design.~~

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### 7.3 Inlet Capacity Calculations

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~~A. Refer to Chapter 10 Section 5 of TxDOT's hydraulic design manual October 2011HDM for inlet capacity equations.~~

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~~1. Stormwater inlets can be further classified into three groups: sump inlets, un-depressed inlets on grade, and depressed inlets on grade. Calculations of the capacity for each inlet type and group which pertains to it are discussed in this section. Many of the equations used for the calculation of inlet capacity came directly or are modified forms of equations found in Hydraulic Engineering Circular No.22 (10). The types of inlets discussed below are:~~

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~~1.B. Sump inlets.~~

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~~1. Grate inlets in a sump are subject to clogging by debris during storm events, and are not recommended for use in sumps or sags. A combination inlet may be more efficient than a grate with "Sweeper" curb inlets. A grate inlet in a sump or a sag operates under either weir or orifice flow. Capacity calculations for both conditions will be performed and the lesser of the two capacities will be the design capacity of the grate inlet. Due to the fact that grate inlets in a sump are prone to clog, only 50% of the perimeter shall be used for the weir calculations and 50% of the surface area shall be used for the orifice calculations. The only grate types that are acceptable in a sump location are the P-50, P-30, P-50 x 100, and Rectiline grates. Capacity of a grate inlet in a sump under weir conditions shall be calculated by the following equation:~~

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$$Q_i = C_w * P * y^{1.5} \quad (\text{Eq. 7-1})$$

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Where:  $Q_i$  Is the capacity in cfs of grate inlet under weir conditions (ft<sup>3</sup>/sec)  
 $C_w$  Is the weir coefficient of 3.0  
 $P$  Is the perimeter of the grate inlet, = 2 \* (width + length) (ft)  
 $y$  Is the head at the inlet, which is the sum of the approach gutter flow depth and the inlet depression and is derived in equations 6-2, 6-5, 6-10, and 6-15 or it is the adjusted head required to accept the 100-year flood (ft), whichever is greater

Capacity of a grate inlet in a sump location under orifice flow conditions shall be calculated by the following equation or by use of Figure 7-9:

$$Q_i = C_o * A * (2 * g * y)^{0.5} \quad (\text{Eq. 7-2})$$

Where:  $Q_i$  Is the capacity in cfs of grate inlet under orifice conditions (ft<sup>3</sup>/sec)  
 $C_o$  Is the orifice coefficient of 0.67

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- A Is 50% of the open surface area of the grate inlet opening (ft<sup>2</sup>/sec). Effective area, 50% of total open area, for the different grate inlet types can be calculated from the equations 7-3 through 7-6 in Table 7-2.
- g Is the acceleration due to gravity, = 32.2 (ft/sec<sup>2</sup>)
- y Is the head at the inlet, which is the sum of the approach gutter flow depth and the inlet depression and is derived in equations 6-2, 6-5, 6-10 and 6-15 or it is the adjusted head required to accept the 100-year flood (ft), whichever is greater

**Table 7-2  
Grate Effective Area**

Inlet	Area	
Sag-Grate P-50	$L * W * 0.729 * 50\%$	(Eq. 7-3)
Sag-Grate P-30	$(L * 0.948) * (W * 0.655) * 50\%$	(Eq. 7-4)
Sag-Grate 50x100	$(L * 0.910) * (W * 0.729) * 50\%$	(Eq. 7-5)
Sag-Recticuline	$L * W * 0.800 * 50\%$	(Eq. 7-6)

1.2. Curb inlets in a Sump are to be used along paved streets and Type-Y inlets (drop inlets) in a Sump are to be used in unpaved areas and drainage ditches. Curb inlets, recessed curb inlets, and Type-Y inlets located in a sump or a low point can generally be considered to function as rectangular broad-crested weirs. The capacity of an inlet in a sump should be based on the following weir equation:

$$Q_i = C_w (L + 1.8 W) y^{3/2} \quad \text{(Eq. 7-7)}$$

- Where:
- $Q_i$  Is the intercept capacity in cfs of curb opening inlet or drop inlet, (ft<sup>3</sup>/sec)
  - $C_w$  Is the weir coefficient, 2.3, for on-grade curb gutters, and 3.0 for depressed curb gutters and Type-Y inlets.
  - L Is the length of curb opening, or the portion of perimeter of inlet opening through which water enters the drop inlet (ft)
  - W Is the width of the depression or the gutter. This is zero if there is no depression, or if the inlet length is greater than 12.0 feet (ft)
  - y Is the head at the inlet, which is the sum of the approach gutter flow depth and the inlet depression and is derived in equations 6-2, 6-5, 6-10, and 6-15, or it is the adjusted head required to accept the 100-year flood (ft), whichever is greater

Inlets should be located frequently enough along the street that the inlet openings do not become submerged. When the depth of flow is more than 1.4 times the height of the opening of the inlet, the inlet operates under completely submerged conditions and the orifice equation should be used to compute the inlet capacity. The capacity of a completely submerged inlet is derived from following orifice equation:

$$Q_i = C_o * A * [2 * g * (y - (h / 2))]^{0.5} \quad \text{(Eq. 7-8)}$$

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Where:  $Q_i$  Is the capacity in cfs of curb opening inlet or drop inlet under submerged conditions (ft<sup>3</sup>/sec)  
 $C_o$  Is the orifice coefficient of 0.67  
 $h$  Is the height of the curb opening (ft)  
 $A$  Is the area of inlet opening,  $L \times h$  (ft<sup>2</sup>)  
 $g$  Is the acceleration due to gravity, = 32.2 (ft/sec<sup>2</sup>)  
 $y$  Is the head at the inlet, which is the sum of the approach gutter flow depth and the inlet depression and is derived in equations 6-2, 6-5, 6-10 and 6-15, or it is the adjusted head required to accept the 100-year flood (ft), whichever is greater

**2.3.** Combination Inlets in a Sump shall have a minimum of a five-foot "Sweeper" curb inlet on both sides. The "Sweeper" curb inlet capacity, that portion of inlet, which does not have a grate, shall be calculated as if it was operating alone. The flow which bypasses the "Sweeper" inlets will then be used for the sizing of the grate inlet in the sump. The capacity of the combination inlet is equivalent to the sum of 50% of the capacity of the grate inlet in a sump as determined in Section 7.4, Grate Inlets in a Sump, and 50% of the capacity of the curb inlet in a sump as determined in Section 7.4, Curb Inlets and Type-Y Inlets in a Sump.

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**3.4.** Slot Inlets are not allowed in a sump due to their susceptibility to clogging.

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**4.C.** Un-depressed inlets on grade are to be placed to provide sufficient capacity to capture the flow from a 1025-year event as outlined in Table 2-2. Inlets on grade generally do not suffer diminished capacity due to floating debris. They do, however, suffer from diminished capacity from excessive street grades. In general, more inlet length will be required to remove the same flow from a steeper roadway than from a flatter roadway.

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**B.1.** Grate Inlets on Grade are an effective means of conveying flow from the roadway to the drainage system. Grate inlets are to only be used in areas where floating debris will not be a problem and with the approval of the City Engineer. Each grate inlet type has a splash over velocity,  $V_0$ , which is used to determine the amount of the flow which will be intercepted by the front of the inlet, based on the spacing of the bars, length of the inlet, and longitudinal slope of the road. The equations for  $V_0$  can be found in Table 7-3. If the velocity of the flow in the gutter is less than  $V_0$  then essentially all of the frontal flow will be intercepted by the grate. If the velocity of flow is greater than  $V_0$  then only a portion of the flow will be intercepted.

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The frontal flow or the gutter flow is the portion of the total gutter flow that is found between the curb and the outer edge of the grate, or between the curb and the point where the depression/gutter begins. The ratio of frontal flow or gutter flow to the total flow,  $E_0$ , is found by the following equation:

$$E_0 = 1 / \{ (1 + ((S_w / S_x) / (1 + (S_w / S_x) / ((T / W) - 1)))^{2.67} - 1) \} \quad \text{(Eq. 7-16)}$$

For non-depressed inlets, the equation can be simplified to:

$$E_0 = 1 - (1 - W / T)^{2.67} \quad \text{(Eq. 7-17)}$$

$$Q_b / Q = 1 - (Q_w / Q) = 1 - E_0 \quad \text{(Eq. 7-18)}$$

Where:  $E_0$  Is the ratio of frontal flow or gutter flow to total flow  
 $S_x$  Is the cross slope of the roadway (ft/ft)

- W Is the width of the grate or the depression (ft)
- $S_w$  Is  $S_x + [a * 12 \text{ (ft/in)}] / W$ . (ft/ft)
- a Is the amount of depression (in)
- T Is the spread width of the flow in the roadway,  $y/S_x$ , (ft) for straight streets
- $Q_w$  Is the flow in the gutter or depressed section (ft<sup>3</sup>/sec)
- $Q_b$  Is the side flow that does not flow in the gutter or depressed section and will flow into or along the side of the grate (ft<sup>3</sup>/sec)
- Q Is the total flow (ft<sup>3</sup>/sec)

**Table 7-3**  
**Splash-over Velocity,  $V_0$ , for Various Grate Inlets**

Grate Type	Splash-over Velocity, $V_0$	
P-50	$2.218 + 4.031 * L - 0.65 * L^2 + 0.06 * L^3$	Eq. 7-9
P-30	$1.762 + 3.117 * L - 0.45 * L^2 + 0.03 * L^3$	Eq. 7-10
Curved Vane	$1.381 + 2.780 * L - 0.30 * L^2 + 0.02 * L^3$	Eq. 7-11
45° Tilt Bar	$0.988 + 2.625 * L - 0.26 * L^2 + 0.03 * L^3$	Eq. 7-12
30° Tilt Bar	$0.505 + 2.344 * L - 0.20 * L^2 + 0.01 * L^3$	Eq. 7-13
P-50x100	$0.735 + 2.437 * L - 0.26 * L^2 + 0.02 * L^3$	Eq. 7-14
Recticuline	$0.030 + 2.278 * L - 0.18 * L^2 + 0.01 * L^3$	Eq. 7-15

Note: L is the length of the grate

The ratio of flow intercepted by the grate to frontal flow,  $R_f$ , is given by the following equation:

$$R_f = 1 - K_c (V - V_0) \quad \text{(Eq. 7-20)}$$

- Where:
- $K_c$  Is the coefficient of 0.09
  - V Is the velocity of flow in the gutter,  $2Q / (T^2 * S_x)$ , (ft/sec) (Eq. 7-21)
  - $V_0$  Is the splash-over velocity which can be determined from the equations 7-9 through 7-15 found in Table 7-3, (ft/sec) Note:  $R_f$  can not exceed 1.0.

The ratio of side-flow intercepted to total flow,  $R_s$ , is given by the equation:

$$R_s = 1 / [1 + ((K_s * V^{1.8}) / (S_x * L^{2.3}))] \quad \text{(Eq. 7-22)}$$

- Where:  $K_s$  Is the coefficient of 0.15

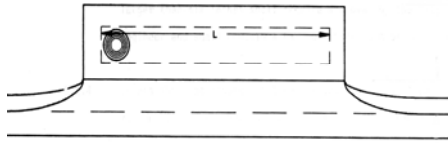
The efficiency,  $E$ , of a grate inlet is given by the equation:

$$E = R_f * E_0 + R_s * (1 - E_0) \quad \text{(Eq. 7-23)}$$

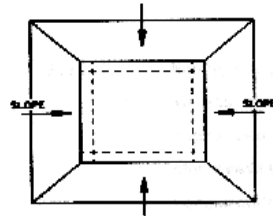
Therefore the total interception (capacity) of grate inlet on-grade is given by the equation:

$$Q_i = E * Q = Q * [R_f * E_0 + R_s * (1 - E_0)] \quad \text{(Eq. 7-24)}$$

6-2. Curb inlets and Type-Y inlets On Grade. The recessed curb inlet is recommended due to its superior interception efficiency. In areas where there is insufficient room to construct the recessed inlet, or it poses the possibility of a traffic hazard, other inlet types may be used with the permission of the City Engineer.



a. Recessed Inlet



b. Type-Y Inlet

Figure 7-2. Curb Inlet Types

The calculation of the amount of flow intercepted by a curb inlet on grade requires the calculation of the length of inlet required to intercept the entire flow,  $L_T$ , which is given by the following equation:

$$L_T = K_e * Q^{0.42} * S_L^{0.3} * (1 / (n * S_e))^{0.9} \quad (\text{Eq. 7-25})$$

Where:

$K_e$	Is the coefficient 0.6
$Q$	Is the total flow ( $\text{ft}^3/\text{sec}$ )
$S_L$	Is the longitudinal slope of the roadway (ft/ft)
$n$	Is Manning's roughness coefficient, usually = 0.016 for streets
$S_e$	Is the equivalent cross slope in cross sections with a depression, this is $S_x$ if there is no depression (ft/ft)
$S_e$	Is $S_x + S_w * E_0$
$S_x$	Is the cross slope of the roadway (ft/ft)
$S_w$	Is the cross slope of the gutter measured from the cross slope of the pavement, $S_{x'}$ (ft/ft)
$S_w$	Is $a / 12$ (in/ft), (ft)
$a$	Is the gutter depression (in)
$E_0$	Is the ratio of frontal or gutter flow to total flow from equation 7-16 or 7-17

The amount of flow that a curb inlet on grade will intercept is equivalent to the product of the total flow and the efficiency of the inlet,  $E$ . The inlet efficiency,  $E$ , is dependent on the actual inlet length,  $L$ , and the required inlet length to intercept the entire flow,  $L_T$ , and is determined by the equation:

$$E = 1 - (1 - (L / L_T))^{1.8} \quad (\text{Eq. 7-28})$$

Therefore the total amount of flow intercepted by a curb inlet on grade is:

$$Q_i = Q * E = Q * [1 - (1 - (L / L_T))^{1.8}] \quad (\text{Eq. 7-29})$$

6-3. Combination Inlets On Grade inlets may be used, with the permission of the City Engineer, in areas where a depressed inlet cannot be constructed. The "Sweeper" curb inlet capacity, that portion of inlet, which does not have a grate, will be calculated as if it was operating along ge. The flow which bypasses the

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"Sweeper" inlets will then be used for the sizing of the combination ~~inlet grate with -on grade a 50% clogging factor.-~~ ~~The capacity of the combination inlet on grade is equivalent to the sum of 50% of the capacity of the grate of the combination inlet on grade is equivalent to the sum of 50% of the capacity of the grate inlet on grade as determined above and 50% of the capacity of the curb inlet on graded as determined above.~~

E.4. Slot Inlets On Grade may only be used with the permission of the City Engineer. If slot inlets are allowed, the inlet capacity shall be calculated as described in chapter 10 section 5 of the HDM or by equations for a curb inlet found as discussed above and the manufacturer's design guidelines. The more conservative method of the two shall be used.

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## 4-8.0 Storm Drain Design

### 8.1 Storm Drain Design Standards

~~1.A.~~ Storm sewer conduit shall be ~~sized~~ sized to flow full, and with Manning's Equation ~~shall be used to determine the conduit size and standard step backwater methodology outlined in TxDOT's HDM.~~ The coefficients of roughness listed in Table 8-1 are for use in Manning's Equation to estimate friction losses.

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~~2.B.~~ The minimum velocity in a conduit shall be 2.5 feet per second (~~1025~~-year design storm). This minimum velocity is required to minimize or prevent the accumulation of sediment in the system. Such sediment accumulation can severely reduce to ability of the system to convey the design flow. ~~The minimum slopes for various pipe sizes required to maintain this minimum velocity and the recommended maximum velocities of flow in a conduit.~~

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~~3.C.~~ Maximum velocities in conduits are important because of the possibility of excessive erosion of the storm drain pipe material. Table 8-3 lists the maximum velocities allowed. Maximum flow velocities at the downstream end of pipe systems shall be consistent with the maximum allowable velocities for the receiving channel (refer to Section 9, Open Channels). Erosion protection is required for disturbed banks of natural channels.

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~~4.D.~~ The maximum discharge velocities in the pipe shall not exceed the design velocity of the receiving channel or conduit at the outfall. The maximum outfall velocity of a conduit in partial flow shall be computed for partial depth and shall not exceed the maximum permissible velocity of the receiving channel unless controlled by an appropriate energy ~~dissipator~~ dissipator.

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~~5.E.~~ When establishing the hydraulic gradient of a storm sewer, entrance and exit losses, expansion losses, manhole and bend losses, junction losses, and minor head losses at points of turbulence shall be calculated and included in the computation of the hydraulic gradient. Refer to HEC-22 for minor headloss loss coefficients.

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~~6.F.~~ Outfalls to Open Channels and Lakes. The flow lines of storm sewer conduits that discharge into open channels shall be higher than or equal to match the flow line of the channel. Storm sewer outfall pipes shall not be at sump with the receiving channel.

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~~7.G.~~ Pipe diameters shall increase downstream, unless otherwise approved by the City Engineer. Select pipe size and slope so that the velocity of flow will increase progressively down the system or at least will not appreciably decrease at inlets, bends or other changes in geometry or configuration.

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~~H.~~ At points of change in storm drain size, pipe crowns (soffits) shall be set at the same elevation.

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~~8.I.~~ All facilities are to be assessed with the 100-yr storm.

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### 4-8.2 Calculation of the Hydraulic Grade Line

The ~~1025~~-year and 100-year frequency hydraulic grade lines (HGL) shall be computed and plotted for all storm drain systems. The ~~1025~~-year frequency hydraulic grade line shall be calculated throughout the system and shall be at least two feet below the street subgrade edge of pavement at the entrance to the inlet. For designs that contain sumps, the 100-year hydraulic grade line is required from the system outfall to the most upstream sump. The determination of friction losses and minor losses are required for these calculations.

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**8.2.1 Starting Tailwater Conditions**

A. The designer must determine the tailwater conditions at the downstream end of the proposed storm drain system when calculating the hydraulic performance of the system. When proposed storm drains are to discharge into existing watercourses, the tailwater elevation used in hydraulic calculations of the proposed storm drain system will be determined by the design engineer and approved by the City Engineer. The tailwater elevation shall be the greater of the water surface of the receiving stream and the minimum outlet water surface,  $y_m$ , both in feet above mean sea level (ft msl),

The minimum water surface,  $y_m$ , is derived from the following equations:

$$y_m = (D_0 + y_c) / 2 + FL \quad (\text{Eq. 8-1})$$

- Where:
- $y_m$  Is the minimum water surface elevation of the pipe in question (ft msl)
  - $D_0$  Is the pipe outlet diameter (ft)
  - $y_c$  Is the critical depth of the channel for a given flow and geometric conditions (ft)
  - FL Is the flow line of the pipe, lateral, trunk, or channel in question (ft msl)

The critical depth,  $y_c$ , is determined by the following equation for Froude Number, which is set equal to 1.0 and solved for depth:

$$1.0 = (Q / A) / (g * D)^{0.5} \quad (\text{Eq. 8-2})$$

- Where:
- Q Is the flow in the inlet pipe (ft<sup>3</sup>/sec)
  - A Is the cross-sectional area of the flow as determined from the equations in Table 8-5, in section 8.3B, (ft<sup>2</sup>)
  - D Is the diameter of the inlet pipe (ft)
  - g Is the acceleration due to gravity (32.2 ft/ft./sec<sup>2</sup>)

**Table 8-1  
Roughness Coefficient "N" for Storm Drains**

Materials of Construction	Minimum Roughness Coefficient "n"
Concrete Pipe	0.013
Corrugated-metal pipe*	
Plain or coated	0.024
Concrete Lined	0.013
Plastic pipe	
Smooth	0.011
Corrugated	0.024

\*Requires approval

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**Table 8-2**  
**Maximum Spacing of Manholes and Junction Boxes**

Pipe Diameter (in)	Max. Spacing (ft)
18-24	400
27-39	800
42-60	1,000
Larger than 60	1,200

Refer to ~~pppp~~pg. 10-45 of the TxDOT HDM.

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**Table 8-3**  
**Maximum Velocity in Storm Drains**

Storm Drain Type	Maximum Velocity
Inlet Laterals (shorter than 30 feet)	No Limit
Inlet Laterals (longer than 30 feet)	20-15 fps
Trunk Lines	20-15 fps

~~1-B.~~ The water surface of the receiving ~~stream,~~ stream shall be based on the "Coincidence Occurrence Frequency, ~~Table~~, Table 8-4, obtained from the Federal Highway Administration HEC-22, (8). The methodology of coincidental occurrence shall be performed as described in HEC-22 If there is a difference in the drainage area of the receiving stream and the contributing stream there will most likely be a difference in the time the peak flow from both systems will reach the point where the contributing tributary discharges into the receiving stream. In an effort to determine what the water surface will be in the receiving stream a statistical analysis has been compiled to show, given the ratio of drainage areas, what storm frequency should be used in receiving stream to determine its water surface elevation.

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~~2.~~ As an example, given a contributing tributary of with a drainage area of 10 acres and a receiving stream with a drainage area of 1,000 acres, the ratio would be 10 to 1,000, or 1 to 100. For a 10-year storm design the design engineer would use the 5-year water surface of the receiving stream.

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~~3-C.~~ It is also important to ensure that when the receiving stream is at its high flow condition, the back water will not cause the low flow in the contributing tributary to either 1) flow at velocities less than 2.5 ft/sec causing sedimentation, or 2) raise the HGL above the natural ground or inlets. Therefore the design engineer must also calculate the velocities in the conduit and HGL of the contributing tributary at the higher water surface. Using the same example, the design engineer would use the 10-year water surface of the receiving stream, and the 5-year flows in the contributing tributary to check sufficient velocities and HGL. The designer must also perform a "Normal Depth" outfall analysis to determine the maximum outlet velocities of the facility. This analysis includes solving the downstream boundary condition using Manning's equation for Normal depth.

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The minimum water surface,  $y_m$ , is derived from the following equations:

$$y_m = (D_0 + y_e) / 2 + FL \quad \text{(Eq. 8-1)}$$



- Where:
- $y_m$  Is the minimum water surface elevation of the pipe in question (ft msl)
  - $D_o$  Is the pipe outlet diameter (ft)
  - $y_c$  Is the critical depth of the channel for a given flow and geometric conditions (ft)
  - FL Is the flow line of the pipe, lateral, trunk, or channel in question (ft msl)

The critical depth,  $y_c$ , is determined by the following equation for Froude Number, which is set equal to 1.0 and solved for depth:

$$1.0 = (Q / A) / (g * D)^{0.5} \text{ (Eq. 8-2)}$$

- Where:
- Q Is the flow in the inlet pipe (ft<sup>3</sup>/sec)
  - A Is the cross-sectional area of the flow as determined from the equations in Table 8-5, in section 8.3B, (ft<sup>2</sup>)
  - D Is the diameter of the inlet pipe (ft)
  - g Is the acceleration due to gravity (32.2 ft/sec<sup>2</sup>)

#### 8.2.2 Friction Losses (Major Losses)

Friction losses (Major Losses) shall be computed using Manning's equation as outlined in Chapter 6, Section 3 of TxDOT's HDM, below, with the Manning's "n" values consistent with Table 8-1.

$$Q = (1.486 / n) * A^{(5/3)} / P^{(2/3)} * S^{0.5} \text{ (Eq. 8-3)}$$

$$V = (1.486 / n) * A^{(5/3)} / P^{(2/3)} * S^{0.5} \text{ (Eq. 8-4)}$$

$$S = ((Q * n * P^{(2/3)}) / (1.486 * A^{(5/3)}))^2 \text{ (Eq. 8-5)}$$

- Where:
- Q Is the flow in the conduit (cfs)
  - V Is the velocity of the flow in the conduit (ft/sec)
  - S Is the slope of the conduit in the direction of flow (percent)
  - n Is the Manning's roughness coefficient from Table 8-1
  - A Is the cross-sectional area of the flow from the equations from the equations found in Table 8-5 (ft<sup>2</sup>)
  - P Is the wetted perimeter of the flow from the equations found in Table 8-5 (ft)

**Table 8-4**  
**Frequencies for Coincidental Occurrence**

**Table 8-5**  
**Hydraulic Geometric Elements of Storm Drain Conduits**

Conduit Shape	Flow Area A	Wetted Perimeter P	Top Width T	Hydraulic Depth D
Circular	$\frac{1}{8} (q - \sin \theta) d_o^3$	$\frac{1}{2} \theta d_o$	$(\sin \frac{1}{2} \theta) d_o$	$\frac{1}{8} * d_o * \frac{(\theta - \sin \theta)}{\sin \frac{1}{2} \theta}$
Box, $y < 0.99 * d_o$	$y * b$	$y * 2 + b$	b	y
$y \geq 0.99 * d_o$	$y * b$	$(y + b) * 2$	b	y

Junction Losses Note:  $\theta = \pi + 2 * (\arcsin((y * d_o / 2) / (d_o / 2)))$

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2-8.2.3 Junction Losses, Without a Manhole or Junction Box (Minor Box Minor Losses)

Minor losses are to be described as all forms of energy loss to a system attributed from junctions, exits, bends, manholes or inlets, and expansions and contractions. Refer to Chapter 10, Section 7 of TxDOT's HDM on methods for computing minor losses.

8.2.4 Hydraulic Grade Line (HGL)

The hydraulic grade line shall account for the summation of major and minor losses within a system and be analyzed using standard step backwater method as described in Chapters 6 and 7 of TxDOT's HDM.

Junction losses, losses incurred when a lateral or trunk line flow into a trunk line, without the use of a manhole or junction box, shown in Figure 8-1, shall be computed using the following equation:

$$H_j = \frac{(Q_o * V_o) - (Q_i * V_i) - (Q_l * V_l \cos \theta) + h_i - h_o}{0.5 * g * (A_o + A_i)} \quad \text{(Eq. 8-6)}$$

Where:	$H_j$	Is the head loss incurred in junction (ft)
	$Q_o, Q_l$ and $Q_i$	Is the outlet, inlet and lateral flows respectively (ft <sup>3</sup> /sec)
	$V_o, V_l$ and $V_i$	Is the outlet, inlet and lateral velocities respectively (ft/sec)
	$h_i, h_o$	Is the outlet and inlet velocity heads respectively (ft)
	$A_o, A_i$	Is the outlet and inlet cross sectional areas respectively (ft <sup>2</sup> )
	$\theta$	Is the angle between the inflow and outflow pipes
	$g$	Is the acceleration due to gravity (32.2 ft/sec <sup>2</sup> )

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3- Junction Losses, With a Manhole or Junction Box

Junction losses incurred when a lateral or trunk line flows into a trunk line, concurrent with the use of a manhole or junction box, shown in Figure 8-2, shall be computed using the following equations:

$$H_{ah} = K_j (V_o^2 / 2g) \quad \text{(Eq. 8-7)}$$

$$K_j = K_o * C_D * C_Q \quad \text{(Eq. 8-8)}$$

Where:	$H_{ah}$	Is the head loss incurred in the junction (ft)
	$V_o$	Is the velocity of the flow in the outlet pipe (ft/sec)
	$g$	Is the acceleration due to gravity (32.2 ft/sec <sup>2</sup> )
	$K_j$	Is the adjusted loss coefficient
	$K_o$	Is the initial head loss coefficient based on relative access hole size
	$C_D$	Is the correction factor for pipe diameter and pipe depth
	$C_Q$	Is the correction factor for relative flow

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The coefficient of head loss due to relative access hole size,  $K_o$ , is determined by the following equation:

$$K_o = 0.1 (b / D_o)^2 (1 - \sin \theta) + 1.4 (b / D_o) 0.15 \sin \theta \quad \text{(Eq. 8-9)}$$

Where:	$K_o$	Is the coefficient of head loss due to relative access hole size
	$b$	Is the manhole or junction box diameter
	$D_o$	Is the outlet pipe diameter (ft)
	$\theta$	Is the angle between the inflow and outflow pipes

A head loss due to pipe diameter is only significant in pressure flow, likewise head loss due to flow depth is only significant in non-pressure flow. Therefore, the coefficient of head loss for pipe diameter and pipe depth,  $C_D$ , is determined by the following equations:

$$\text{If } d_{\text{ahc}} / D_o \Rightarrow 3.2; \text{ then } C_D = (D_o / D_i)^3 \quad (\text{Eq. 8-10})$$

$$\text{If } d_{\text{ahc}} / D_o < 3.2; \text{ then } C_D = 0.5 * (d_{\text{ahc}} / D_o)^{0.6} \quad (\text{Eq. 8-11})$$

Where:

$C_D$	Is the coefficient of head loss due to for pipe diameter and pipe depth
$d_{\text{ahc}}$	Is the water depth in the access hole above the outlet pipe invert (ft)
$D_o$	Is the outlet pipe diameter (ft)
$D_i$	Is the inflowing pipe diameter (ft)

A coefficient of head loss due to the flow relative to an incoming pipe,  $C_Q$ , is a function of the angle of the incoming flow as well as the ratio of the flow from the inflow pipe to the total outflow. The coefficient is given by the following equation:

$$C_Q = (1 - 2 \sin \theta) * (1 - (Q_i / Q_o))^{0.75} + 1 \quad (\text{Eq. 8-12})$$

Where:

$C_Q$	Is the coefficient of head loss due to the flow relative to an incoming pipe
$Q_i$ and $Q_o$	Is the inlet and outlet flows respectively ( $\text{ft}^3/\text{sec}$ )
$\theta$	Is the angle between the inflow and outflow pipes

If the flow line of the inflow lateral or trunk line is above the hydraulic grade line of the outflow pipe, then the initial depth is equivalent to the minimum depth,  $y_{\text{min}}$ , of the inflow pipe and is determined by the Equation 8-2.

#### 4. Losses in a Bend

The head loss at pipe bends is related to the velocity head and can be computed using the following equation:

$$h_b = K_a * K_r * V^2 / 2g \quad (\text{Eq. 8-13})$$

Where:

$h_b$	Is the head loss at the bend (ft)
$K_a$	Is the bend loss coefficient due to the angle of the bend, Table 8-6
$K_r$	Is the bend loss coefficient due to the diameter of the radius the bend is pulled, Table 8-6
$V^2 / 2g$	Is the velocity head using the velocity at the downstream end of the bend

The coefficients  $K_a$  and  $K_r$  vary with the angle of the bend. Table 8-6 and Figure 8-3 contain different  $K_r$  coefficients used in bend losses calculations.

#### 5. Losses Due to Transitions (Sudden Expansion or Contraction)

The head losses due to sudden enlargements and contractions are calculated using the same equation as for junction losses, Equation 8-7. The values for  $K_j$  for sudden enlargements and contractions are in Table 8-7.

### 2-8.3 Hydraulic Grade Line Computation Sheet

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In order to facilitate computations required in determining the hydraulics of a drainage system, Computation Sheet No. 8-1 has been prepared and may be used. Each line in the computation sheet represents a junction or structure and its associated pipe. The calculations begin at the outfall and work upstream with each junction. Computation Sheet 8-2 is used to calculate HGL and EGL elevations and pipe and structure losses. In subcritical flow, the losses are summed to determine upstream HGL levels. In the case of supercritical flow, pipe and manhole losses are not carried upstream. Should supercritical flow occur, the designer should advance to the next section upstream to determine to flow regime at that point. This process continues until the system returns to a subcritical flow regime. Before filling out the columns in Sheet 8-2, the designer should determine a Hydraulic grade line (HGL) at the outlet of the structure. Section 8.4, Starting Tailwater Conditions, describes the method for determining the starting tailwater elevation is described above. The Designer shall provide a HGL computation sheet that depicts all forms or energy loss for each junction and pipe connection and identifies the upstream and downstream HGL and EGL elevations. These computations shall be provided for the design storm and 100-yr assessment.

**Table 8-6**  
**Coefficients of Loss Due to a Bend  $K_a$  and  $K_r$**

Bend Angel Degrees	Angle Bend Loss Coefficient, $K_a$	Radius / Diameter	Radius Bend Loss Coefficient, $K_r$
$0 \leq 15^\circ$		$R/D < 2$	1.000
$15^\circ < \theta \leq 22.5^\circ$	0.20	$2 \leq R/D < 4$	0.500
$22.5^\circ < \theta \leq 45^\circ$	0.35	$4 \leq R/D < 6$	0.433
$45^\circ < \theta \leq 60^\circ$	0.43	$6 \leq R/D < 8$	0.367
$60^\circ < \theta$	0.50	$8 \leq R/D < 20$ $20 \leq R/D$	0.300 -

Note: Minimum radius of bend shall be specified by the manufacture.

**Table 8-7**  
**Head Loss Coefficients Due to Sudden Expansion and Contraction**

$D_2/D_1^*$	Sudden Expansion $K_e$	Sudden Contractions $K_c$
1.2	0.10	0.08
1.4	0.23	0.18
1.6	0.35	0.25
1.8	0.44	0.33
2.0	0.52	0.36
2.5	0.65	0.40
3.0	0.72	0.42
4.0	0.80	0.44
5.0	0.84	0.45
10.0	0.89	0.46
	0.91	0.47

\* $D_2/D_1$  = Ratio of larger to smaller diameter

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## 4-9.0 Open Channels

### 9.1 Hydraulics

#### 9.1.1 Gradually Varied Flow

Calculations of water surface profiles can be accomplished by using ~~the standard backwater methods, Standard Step Method, or acceptable computer routines.~~ Water surface profiles for the design frequency floods shall be computed for all channels and shown on all final drawings. The Corps of ~~Engineers~~ ~~HEC 2 or~~ HEC-RAS Water Surface Profile Programs may also be used to perform standard step backwater calculations, and if used, a summary table shall be submitted to the City. Losses due to changes in velocity, drops, bridge openings, and other obstructions shall be considered in the backwater computations, as described in the ~~HEC 2 and~~ HEC-RAS User's Manuals. ~~The Standard Step Method for determining water surface profiles can be performed using Computation Sheet 9-1.~~

$$S_f = \frac{n^2 V^2}{2.22 R^{4/3}} = S_c \quad (\text{Eq. 9-1})$$

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#### 4-9.1.2 Normal Flow

~~Any of several computer programs or nomograms are acceptable. Manning's "n" for various conditions are in Table 9-1 and 9-2.~~

~~Calculations for normal flow shall be done using Manning's open channel flow equation. Refer to Chapter 6 of TxDOT's HDM.~~

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## 2-9.2 Design Parameters

~~1-A.~~ Where possible, channels should have sufficient gradient, depending upon the type of soil or channel lining material, to provide velocities that will be self-cleaning (greater than 2 feet per second) but will not be so great as to create erosion. Maximum permissible velocities are shown in Table 9-3. ~~Refer to Chapter 7, Section 3 of the TxDOT HDM and Chapter 2, Section 6 of the TxDOT roadway Design Manual for additional design criteria pertaining to ditches.~~

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~~2-B.~~ Appropriate energy dissipating structures may be used to control erosion due to high velocities at pipe system outfalls and steep grades and shall be designed in accordance with accepted design practices such as outlined by the Soil Conservation Service, the Corps of Engineers, the Bureau of Land Reclamation, or TxDOT.

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## 4-9.3 Supercritical Flow

~~1-A.~~ The Froude Number provides a relationship between flow velocity and the hydraulic depth of flow, and gravitational action, and shall be calculated for all channel improvements designs. Subcritical flow conditions occur when the Froude Number is less than 1.0 and supercritical flow conditions exist in lined channels when the Froude Number exceeds 1.0. ~~The Froude Number may be calculated by the following equations:~~

$$Fr = V / (g * D)^{0.5} \quad (\text{Eq. 9-2})$$

Where:  $V$  Is velocity of flow (ft/sec)  
 $g$  Is the acceleration due to gravity (ft/sec<sup>2</sup>)  
 $D$  Is the hydraulic depth (ft)

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And

$$D = A / T \quad \text{(Eq. 9-3)}$$

Where: A Is the cross-sectional area of the flow (ft<sup>2</sup>)  
 T Is the top of the flow (ft)

2.B. ~~Each channel cross section has two flow depths, the normal depth and the alternate depth. Although the depths, velocities, and Froude Number differ, the specific energy of the two depths are equivalent. Figure 9-1 shows the relationship of specific energy to depth. If a channel's normal depth is supercritical, its alternate depth is a deeper subcritical depth. Obstructions that may enter a stream during a storm event may cause supercritical flows to experience a hydraulic jump and become subcritical flows. Due to this fact, Therefore, channels that are designed for supercritical conditions must have freeboard equal to the alternate depth. Supercritical flow must be contained in straight reinforced concrete lined sections of the channel with no bends.~~

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**Table 9-1**  
**Roughness Coefficients of New or Altered Channels**

Type of Channel	Manning's "n"	
Grass-lined		
Bermuda	0.04	
St. Augustine	0.045	
Soils		Velocity (fps)
Cobbles	0.035	5.5
Coarse gravel	0.025	6
Graded silts to gravel	0.030	5.5
Graded loam to cobbles	0.030	5
Fine gravel	0.020	5
Shale & hardpan	0.025	6
Alluvial silts, colloidal	0.025	3.75
Stiff clay, very colloidal	0.025	3.75
Firm loam	0.020	2.5
Alluvial silts, non colloidal	0.020	2
Silt loam, non colloidal	0.020	2
Sandy loam, non colloidal	0.020	1.75
Fine sand, colloidal	0.020	1.5
Concrete		
Rough finish	0.02	
Smooth finish	0.015	
Exposed rubble	0.025	
Gabion	0.035	
Rock cut	0.025	

**Table 9-2**  
**Computation of Composite Roughness Coefficient for Excavated and Natural Channels**

For use with  $n = (n_0 + n_1 + n_2 + n_3 + n_4) * m$

	Channel-Conditions	Qualifiers	Values
$n_0$	Material	Earth	0.02
		Fine-Gravel	0.024
		Course-Gravel	0.028
$n_1$	Degree of Irregularity	Smooth	0.0
		Minor	0.005
		Moderate	0.010
		Severe	0.02
$n_2$	Relative-Effect of Channel-Cross-Section	Gradual	0.0
		Alternating Occasionally	0.005
		Alternating-Frequently	0.013
$n_3$	Relative-Effect of Obstructions	Negligible	0.0
		Minor	0.013
		Appreciable	0.025
		Severe	0.05
$n_4$	Vegetation	Low	0.008
		Medium	0.018
		High	0.038
		Very-High	0.075
$m$	Degree of Member	Minor	1.0-1.02
		Appreciable	1.2-1.5
		Severe	1.5 or greater

In selecting the value of  $n_1$ , the degree of irregularity is considered smooth for surfaces comparable to the best attainable for the materials involved; minor for good dredged channels, slightly eroded or scoured side slopes of canals or drainage channels; moderate for fair to poor dredged channels, moderately sloughed or eroded side slopes of canals or drainage channels, and unshaped, jagged and irregular surfaces of channels excavated in rock.

In selecting the value of  $n_2$ , the character of variations in size and shape of cross-section is considered gradual when the change in size or shape occurs gradually; alternating occasionally when large and small sections alternate occasionally or when shape changes cause occasional shifting of main flow from side to side; and alternating frequently when large and small sections alternate frequently or when shape changes cause frequent shifting of main flow from side to side.

In selection of the value of  $n_3$  is based on the presence and characteristics of obstructions such as debris deposits, stumps, exposed roots, boulders and fallen and

lodged logs. One should recall that conditions considered in other steps must not be re-evaluated or double-counted in this selection. In judging the relative effect of obstructions, consider the following: the extent to which the obstruction occupy or reduce the average water area, the obstruction characteristics (sharp edged or angular objects induce greater turbulence than curved, smooth-surfaced objects) and the position and spacing of obstructions transversely and longitudinally in the reach under consideration.

In selecting the value of  $n_4$ , the degree of effect of vegetation is considered in the following way:

1. Low for conditions comparable to: 1) dense growths of flexible turf grasses or weeds, of which Bermuda and blue grasses are examples; where the average depth of flow is two to three time the height of vegetation and 2) supple seeding tree switches, such as willow, cottonwood or salt cedar where the average depth of flow is three to four times the height of the vegetation.
2. Medium for conditions comparable to: 1) turf grasses where the averaged depth of flow is one to two times the height of vegetation, 2) stemmy grasses, weed or tree seedlings with moderate cover where the average depth of flow is two to three times the height of vegetation and 3) brush growth, moderately dense, similar to willows one to two years old, dormant season, along side of a channel with no significant vegetation along the channel bottom, where the hydraulic radius is greater that two feet.
3. High for conditions comparable to: 1) turf grasses where the average depth of flow is about equal to the height of vegetation, 2) dormant season willow or cottonwood trees eight to ten years old, inter grown with some weeds and brush, where none of the vegetation is in foliage along side slopes, no significant vegetation along channel bottom, where hydraulic radius is greater that two feet.
4. Very high for conditions comparable to: 1) turf grasses where the average depth of flow is less than one half the height of vegetation, 2) growing season bushy willows about one year old, inter grown with weeds in full foliage along side slope, or dense growth of cattails along channel bottom, with any value of hydraulic radius up to ten or 15 feet and 3) growing season trees inter grown with weed and brush, all in full foliage, with any value of hydraulic radius up to ten or 15 feet.

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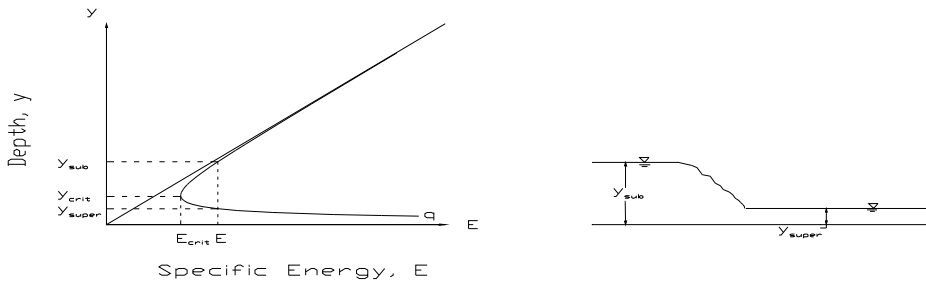
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In selecting the value of  $m$ , the degree of meandering depends on the ratio of the meander length to the straight length of the channel reach. The meandering is considered minor for ratios of 1.0 to 1.2, appreciable for ratios of 1.2 to 1.5, and severe for ratios of 1.5 and greater.

- BC.** Subcritical flow conditions are recommended for all channel designs, as supercritical flow tends to have high velocities and high potential for channel erosion. Supercritical flow conditions will not be allowed in unlined channels. Subcritical flow conditions may be achieved by using energy dissipators in unlined channels in areas where the existing topography will not allow subcritical flow conditions to occur naturally. In all cases, the channel improvements shall be designed to avoid the unstable transitional flow conditions that occur when the Froude Number is between 0.9 and 1.1.





$$E = V_{sub}^2 / 2g + y_{sub} = V_{super}^2 / 2g + y_{super}$$

Figure 9-1. Alternate Depths on the Specific Energy Curve.

#### 9.4 Flow in Bends

1.A. When a channel changes direction, the depth of flow along the outside edge of the curve is higher than the average channel flow depth, or the water surface is super-elevated. Therefore, additional freeboard must be provided to prevent the channel bank from being overtopped. The amount of super-elevation along the outside of the bend can be estimated using (18):

$$\Delta H = \frac{C^2}{2gr_o^2r_i^2} (r_o^2 - r_i^2) \quad (\text{Eq. 9-4})$$

Where:  $\Delta H$  Is the increase in water surface elevation along the outside of the channel bend due to super-elevated in feet  
 $C$  Is the circulation constant (ft<sup>2</sup>/sec)  
 $r_o$  Is the outside radius of the channel bend in feet  
 $r_i$  Is the inside radius of the channel bend in feet  
 $g$  Is the acceleration due to gravity (32.2 ft/ft./sec)

2.B. If the discharge, depth of flow at the approach to the bend, average flow velocity in the approach to the bend, and the inner and outer radii of the bend are know, the value of the circulation constant can be approximately approximated by solving for C:

$$Q = C \left[ y_a + \frac{V_a^2}{2g} - \frac{C^2}{2gr_o r_i} \right] \ln(r_o / r_i) \quad (\text{Eq. 9-5})$$

Where:  $Q$  Is the total flow in the channel (cfs)  
 $V_a$  Is the average velocity in the bend approach (fps)  
 $y_a$  Is the depth of flow in the approach to the bend in feet.

Table 9-3  
**Maximum Velocity in Open Channels**

Channel Material*	Channel	Velocity (fps)	Velocity (fps) erosion resistant soil
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Comment [A20]: Table to be replaced with range of permissible vel and shear columns.

	Slope (%)	erosion resistant soil	
Grass Line Earth			
Bermuda	0-5	8	6
	5-10	7	5
	>10	6	4
Buffalo grass, Kentucky bluegrass, smooth brome, blue grama	0-5	7	5
	5-10	6	4
	>10	5	3
Weeping lovegrass, alfalfa, annuals	0-5**	3.5	2.5
Rock (Native)***			10
Gabion lined			12
Reinforced concrete lining			20
Rock riprap (placed rock)			Use U.S. Army Corps of Engineers Guidelines
Prefabricated lining products			Use 90% of Manufacturer's Recommended Velocity Limits

\* Uniform, in well-maintained condition

\*\* Not recommended for channels >5% except for channel side slopes

\*\*\* Depends on size ( $d_{50}$ ) and velocity C.

The flow velocity along the outside of the bend,  $V_o$  (in feet per second), can then be approximated:

$$V_o = C / r_o \quad (\text{Eq. 9-6})$$

$V_o$  shall not exceed the maximum values established in Table 9-1.

3.C. Super-elevation may also be determined using:

$$\Delta H = \frac{V^2 T_w}{g r_c} \quad (\text{Eq. 9-7})$$

Where:  $T_w$  Top width of channel (ft)(ft.)  
 $r_c$  Centerline radius of curvature (ft)(ft.)

### 9.5 Drop Structures

1.A. The function of a drop structure is to reduce flow velocities by dissipating some of the kinetic energy of the flow at the drop structure, and also providing flatter channel slopes upstream and downstream of the drop structure. Sloping channel drops and vertical channel drops are two commonly used drop structure types.

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2-B. An apron shall be constructed immediately upstream and downstream of a drop structure to protect against turbulence. The upstream apron shall extend at least ten feet upstream from the point where flow becomes supercritical and shall include a concrete toe into the ground. The downstream apron shall be extended extend to a minimum length as identified in Table 9-4 of twenty feet beyond the anticipated location of the jump and shall include a concrete toe into the ground. The toe at each end shall extend a minimum of twenty-four inches into the ground to minimize scour at the transition to natural ground.

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**Table 9-4  
Length of Downstream Apron**

Maximum Unit Discharge (cfs/ft <sup>2</sup> /ft.)	Length of Downstream Apron (ft.)
0-14	10
15	15
20	20
25	23
30	25

3-C. All drop structures shall be constructed of reinforced concrete, and the bottom and walls (if any) shall have a minimum thickness of six inches. To facilitate maintenance, drop structures should be located near bridges or culverts if possible.

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#### 4-9.5.1 Vertical Drop Structures

The drop length and the hydraulic jump length of the drop structure should be calculated to determine the length of the downstream apron required to prevent erosion (18, 19). In order to utilize a vertical drop structure vehicular access must be provided to both the upstream and downstream ends of the structures.

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#### 2-9.5.2 Sloping Drop Structures

The location of the hydraulic jump should be determined based on the upstream and downstream flow depths and channel slopes (18, 19). When utilizing a sloping drop structure, a minimum slope of 6:1 shall be used to allow vehicular access from one end across the structure. If the slope of the drop structure is less than 6:1, vehicular access must be provided to both the upstream and downstream ends of the structures.

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#### 9.6 Shear stress

Shear stress and channeling shall be computed for all open channels and adequate protection provided in accordance to FHWA FHWA HEC-15.

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#### 9.7- Energy Dissipators Dissipators

Although hydraulic jumps can be used as energy dissipators, impact dissipators are recommended for their predictability, efficiency, and economy. The Baffled Apron is used to dissipate the energy in the flow at a drop. It requires no initial tailwater to be effective, although scour is reduced with tailwater. The chute of the Baffle Apron is constructed on a 2:1 or flatter slope extending below the channel bottom. See Examples. Refer to FHWA HEC-14 and FHWA's HY-8 software help menu for methods to designing energy dissipators.

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2. **Energy Dissipaters**

~~Although hydraulic jumps can be used as energy dissipaters, impact dissipaters are recommended for their predictability, efficiency, and economy. The Baffled Apron is used to dissipate the energy in the flow at a drop. It requires no initial tailwater to be effective, although scour is reduced with tailwater. The chute of the Baffle Apron is constructed on a 2:1 or flatter slope extending below the channel bottom. See Examples.~~

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9-510 **Bridge and Culvert design**

10.1 **Applicable Design Criteria**

All bridge design shall be on a case-by-case basis to establish design requirements set by the City Engineer.

Headwalls and necessary erosion protection shall be provided at all culverts and shall comply with TxDOT standard, or modified standard, details. All culverts and bridges are to be analyzed at both the design flow and 100-yr check flow.

10.2 **Design Parameters**

1.A. Alignment, location and grade of proposed culverts must be consistent with planned development of the drainage system for that watershed. In the event the particular watershed or waterway is not covered by a planned storm drainage system, the designer should proceed with the design from the nearest downstream control (i.e. bridge, culvert dam, etc.) and design the proposed drainage system improvements anticipating future system expansion due to fully developed watershed conditions.

2. Wingwalls, if used, may be either straight parallel, flared, or tapered. Approach and discharge aprons shall be provided for all culvert headwall designs. The guidelines listed in Table 10-1 are intended to aid in determining when to use various types of wingwalls. Precast headwalls and endwalls may be used if all other criteria are satisfied; generally precast headwalls/endwalls are available for smaller culverts (18 inches to 24 inches diameter).

B.

**Table 10-1  
Guidelines for Wingwall Use**

Conditions	Wingwall Type
Small culverts with flat slopes	Straight (parallel), flared or tapered.
Abrupt change in flow direction is necessary	Straight with one perpendicular wingwall (not recommended for large culverts) or flared.
Approach velocities below 6 fps, approach channel undefined, formation of backwater pools acceptable	Straight, flared or tapered
Approach velocities 6-10 fps, approach channel well defined	Flared (wingwalls located with respect to axis of the approach channel)

10.3 **Culvert Outlet Protection**

High discharge velocities from culverts can cause eddies or other turbulence which could damage unprotected downstream channel banks and roadway embankments. To prevent damage from scour and erosion in these conditions, culvert outlet protection is needed. The outlet protection should extend downstream to a point where non-erosive channel velocities or shear stress are established in accordance with section 9.6 of this manual. The outlet protection should be placed sufficiently high on the adjacent banks to extend 1' above the design WSEL, provide protection from wave wash under design flow conditions. All outlet protection shall be designed with an appropriate toe depth. All toes shall be no less than 1'.

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**Table 10-2  
Culvert Entrance Loss Coefficients**

Type of Culvert	$K_e$
Reinforced concrete pipe:	
Projecting from fill, socket end (groove end)	0.2
Projecting from fill, square cut end headwall or headwall with wingwalls	0.5
Socket end of pipe (groove end)	0.2
Square edge	0.5
Rounded edge (radius $\geq 0.0833D$ )	0.2
Mitered to conform to fill slope	0.7
Beveled edges, 33.7° or 45° bevels	0.2
Side or slope tapered inlet	0.2
Corrugated metal pipe or arch pipe:	
Projecting from fill (no headwall)	0.9
Headwall or headwall with wingwalls, square edge	0.5
Mitered to conform to fill slope, paved or unpaved slope	0.7
Beveled edges, 33.7° or 45° bevels	0.2
Side or slope tapered inlet	0.2
Reinforced concrete box:	
Headwall parallel to embankment (no wingwalls)	
Square edged on three sides	0.5
Rounded on three sides to radius of 1/12 barrel dimension, or beveled edges on three sides	0.2
Wingwalls at 30°-70° to barrel square edged at crown	0.4
Crown edge rounded to radius of 1/12 barrel dimension, or beveled top edge	0.2
Wingwall at 10°-25° to barrel, square edged at crown	0.5
Wingwalls parallel (extension of sides), square edged at crown	0.7
Side or slope tapered inlet	0.2

**2:10.4 Culvert Hydraulics**

~~1.A.~~ The hydraulic design of culverts shall be based upon design guidelines set forth by ~~TxDOT,~~ the U.S. Department of Transportation, or other suitable material as approved by the City Engineer. ~~Figures and Computation Sheets are available as guides.~~ Computer programs ~~are available~~ such as FHWA's "HY-8" model ~~model may be used, provided that the design Engineer provides output tables showing models results and input data (FHWA, 1985).~~ Table 10-2 contains the culvert entrance loss coefficients ( $K_e$ ).

~~B.~~ In general, there are two categories of flow through culverts: inlet control and outlet control.

~~2.~~

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1. Inlet Control. The flow is controlled by the cross-sectional area of the culvert, inlet configuration, and headwater depth. Slope, roughness and length of culvert are of no importance. ~~Inlet control can generally be assumed when the slope is greater than the critical slope as defined below with culvert diameter (D) in feet:~~

$$S_o\% = 2.04 / D^{0.333} \quad \text{(Eq. 10-1)}$$

Nomographs are available for inlet control estimations as proved in FHWA's Hydraulic Design Series Number 5 (HDS 5). ~~Note that there is a choice of three axes on the right hand side, depending on the entrance type. Also note that the box culvert nomograph give discharge in cfs per foot of box width.~~

2. Outlet control. The flow is controlled by the cross-section area of the culvert, inlet configuration, and headwater depth and, slope, roughness and length of culvert. Culverts will be outlet controlled if the culvert slope is relatively flat, the tailwater sufficiently deep or the culvert is quite long. It is also possible, where the water enters the culvert under inlet control, but the culvert slope or tailwater conditions cause a hydraulic jump near the outlet. This situation should be avoided because damage can occur ~~tot-heto~~ the culvert pipe. Unstable conditions are most likely when the culvert is placed at a near-critical slope.

3. ~~The Design Engineer shall typical procedure is to calculate for both outlet and inlet control conditions -and use the more conservative of the two as the design condition.~~

3. ~~use the small flow rates to design the system:~~

4. ~~Terms shown on the nomographs are defined as:~~

1. ~~HW = Headwater depth, the vertical distance from culvert at inlet to surface of ponded water at inlet.~~
2. ~~h<sub>o</sub> = tailwater depth, the vertical distance from culvert invert at outlet to water surface at outlet. Actually, h<sub>o</sub> is a reference depth for calculations and can differ from the actual tailwater depth, TW.~~
3. ~~S<sub>o</sub> = barrel slope~~
4. ~~k<sub>e</sub> = entrance loss coefficient (e.g. wingwalls, see table)~~
5. ~~H = head, the energy needed to pass a given quantity of water through a culvert under outlet control. Equal to sum of velocity head, entrance loss and friction loss in the pipe. Note that the nomographs provide an estimate of "H" only, and headwater "HW" must be calculated from:~~

$$HW = H + h_o + LS_o \quad \text{(Eq. 10-2)}$$

6. ~~For the case of outlet control where both end are submerged and velocity (V) is discharge (cfs)/cross sectional area (SF) (Q/A) "H" can be calculated directly:~~

$$h = (1 + k_e + (29n^2L) / (R^{1.33})) (V^2 / 2g) \quad \text{(Eq. 10-3)}$$

7. ~~For the case of outlet control where the tailwater is at or below the top of the culvert critical depth (d<sub>c</sub>) must be determined.~~

1. ~~If critical depth is greater than culvert height, pipe will flow full at outlet, and H is measured from the top of the pipe at the outlet.~~
2. ~~If the culvert flows full over part of its length, H is calculated from a point slightly higher than critical depth:~~

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$$h_o = \max((d_e + D)/2, TW) \quad (\text{Eq. 10-4})$$

3. A free water surface will extend through the culvert barrel if:

$$HW < D + (1 + k_e)(V^2 / 2g) \quad (\text{Eq. 10-5})$$

The tailwater depth in this case is less than critical depth. A precise solution requires a backwater flow computation through the culvert. However, capacity can be approximated using the method in 2 above and will give satisfactory results when  $HW > 0.75D$ .

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### 10.5 Debris Fins

1.A. For conditions where more than one box culvert is required, the upstream face of the structure shall incorporate debris deflector fins to prevent debris buildup. For multiple-pipe or single box in critical situations, installations of debris fins may be used but are not required. The debris fin is an extension of the interior walls of a multiple-box culvert. The wall thickness shall be designed to satisfy structural requirements and reduce impact and turbulence to the flow.

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2.B. A debris fin is constructed to the height of the culvert with a fin length of one and one-half times the height of the box culvert. Since the debris fins are subject of the same erosive forces as bridge piers, care must be taken in the design of the footing. A toewall at the upstream end of the debris fin and the apron is required, recommended.

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### 10.6 Energy Dissipation

A. Material requirements for outlet protection are indicated in Table 10-13.

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**Table 10-13  
Material Requirements**

Culvert Outlet Velocity (fps)	Outlet Protection
<4	None, grass
4-10	Dumped rock-Riprap stone protection
>10-15	Energy dissipation structure Wire-enclosed or grouted rock riprap

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2. Riprap run out apron length for pipe diameter (D) and water depth at outlet (d) can be determined using:

$$\text{Riprap length (ft)} = 1.7d * Q / D^{2.5+8} \quad (\text{Eq. 10-6})$$

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1.B. Design of riprap stone protection shall be done in accordance to FHWA HEC-22 Vol 1, section 5.2. Median stone diameter ( $d_{50}$ ) can be determined (ASCE 1992) as:

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C. The design engineer is to design concrete baffles or stilling basins in accordance to FHWA HEC-14.



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$$d_{50} = 0.02 * Q^{(4/3)} / TW * D \text{ or by HEC 15 (FHWA, 1988)}$$

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## 4-11.0 Detention Basin Design

### 11.1 Design Criteria

- ~~1.A.~~ Stormwater detention basins are used to temporarily impound (detain) excess stormwater, thereby reducing peak discharge rates.
- ~~2.~~ Detention basins without upstream detention areas where the design rainfall was determined using the Rational Method can be designed using the Modified Rational Method to determine the critical duration. The detention volume for a given duration in this method is calculated as the difference between the trapezoidal inflow hydrograph and the triangular outflow hydrograph. A range of storm durations is required to determine the critical duration for the detention basin. See Appendix F for example of this calculation method. The "C" for the Rational Method shall be a minimum of 0.60. Time of concentration shall provide a conservative choice.
- ~~B.~~ All detention ponds are to be designed to prevent an increase in flow to the existing 2, 5, 10, 25, 50, and 100-yr peak runoff leaving a proposed site.
- ~~3.C.~~ Detention ponds will be sized using the NRCS synthetic hydrograph as outlined in section 5 of this manual. Computations performed for detention pond sizing using a unit hydrograph shall use a minimum curve number of 67, 79, 86 and 89 for soil types A, B, C and D, respectively. Time of concentration shall provide a conservative choice.

### 4-11.2 Minimum Requirements for Detention Pond Design

- ~~1.A.~~ The following criteria shall serve as minimum requirements for detention pond design:
1. Detention basins to be excavated shall provide positive drainage through the pond with a minimum slope of ~~0.25~~1.0%.
  2. When the outflow structure discharges flow into a natural stream or unlined channels, it shall do so at a non-erosive rate in accordance with Section 9 of this Manual.
  3. Earthen embankments used to impound a required detention volume must have a minimum top-width of 4 feet, shall contain a non-permeable core, and shall be based on a geotechnical investigation for the site. Compaction of ~~at all~~ earthen drainage structures shall be to 90% standard proctor. Earthen embankments higher than 6 feet shall conform to Permanent Rule 31, Texas Administrative Code (TAC) Chapter 299 and other applicable dam safety requirements.
  4. Security fencing with a minimum height of six feet may be required to encompass the detention storage area if the location, velocity, depth, or slopes justify restricted access to the general public as determined by the City Engineer. ~~Ferret~~ fence shall be designed to allow access for maintenance as well as not to restrict stormwater flow into or out of the detention basin.
  5. A maintenance ramp shall be provided for vehicular access in detention basin design for periodic desilting and debris removal. The slope of the ramp shall not exceed 6:1 and the minimum width shall be 12 feet.
  6. Basins with permanent storage must include dewatering facilities to provide for maintenance.

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7. The design of detention facilities shall include provisions for collecting and removing sediment deposited after collecting and releasing stormwater.
8. A non-erodible pilot channel shall be provided to convey runoff from entry points of concentrated flow into the pond to the outlet structure of the pond during low flow conditions. Erosion protection must be provided adjacent to the pilot channel to prevent undermining of the pilot channel due to scour.
9. The basin shall have 1 foot of freeboard from top of berm to the 100-yr WSEL of the pond.
10. All ponds are to have an emergency spillway set to engage at the 100-yr WSEL. The emergency spillway shall be designed to convey overflow within proposed property limits leading up to the pond outfall.

8:

### 11.3 Outlet Structure Design

- 1.A. Design and construction information for outlet works is contained in the publication "Stormwater Detention Outlet Control Structures (ASCE, 1985). Multi-level outlet structures may be necessary to restrict ~~due~~ both the 2, 5, 10, 25, 50, ~~10~~-year and 100-year design storm runoff to pre-development levels, and provided for an emergency spillway.
2. For small slick pipes flush with the upstream embankment and a free outfall, the orifice equation can be used ( $H$  = height of water above center of pipe, ft):
 
$$\text{Outflow (cfs)} = 4.8 * (\text{area of opening, sq ft}) * H^{0.5}$$
- 3.B. Standard culvert design techniques are appropriate for larger pipes. Outlet structures are to be analyzed as culverts unless ~~multi~~multi-level outlet structures are to be used. Downstream boundary conditions are to be applied in the same manner as discussed in ~~section~~Section 10 of this manual.
- 4.C. Spillway size should generally be estimated using a broad-crested weir equation such as:
 
$$\text{Outflow (cfs)} = 2.7 * L * H^{1.5}$$
- 5.D. The spillway should be heavily protected from erosion with concrete, rock, grouted riprap, gabions, geotextile, geogrids, concrete block or other material, as selected by the designer.
- 6.E. Documentation on retention or detention structures should include design hydrographs, calculation of stage-storage-discharge tables, drawings of the basin, spillway and outlet size and location, and erosion control measures.

### 1. Playa Lakes

Playa Lakes are natural formations which retain runoff but lack the ability to sufficiently drain after a storm event. A designer may use a playa as part of a runoff control plan if there is suitable area capacity information describing storage availability. Excavation and fill are allowed, if necessary, but under no circumstances may the storage capacity be reduced without the appropriate compensation of storage loss within the immediate are of the development. In creating a hydraulic model of the playa, the lake shall be assumed to be full to the lowest natural outlet point prior to rainfall.

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## 4.12.0 Lakes, Dams and Levees

### 12.1 Lakes and Dams

#### 12.1.1 General

In the event that a property owner or developer desires to modify an existing pond or lake or desires to impound stormwater by filling or constructing an aboveground dam, thereby creating a lake, pond, lagoon or basin as part of the planned development of that property, the criteria listed below shall be met before city approval of the impoundment can be given. Ponds or lakes created by excavation of a channel area without erecting a dam above natural ground elevation or instream low water check dams are also subject to the criteria listed below, with the exception of spillway capacity requirements. The City Engineer has the final authority to determine the design criteria for a proposed dam, check dam or excavated lake. The dam safety requirements of the State of Texas and FEMA must also be met for the construction of dams, lakes, and other improvements.

#### 4.12.1.2 Applicable Design Criteria

4.A. The design criteria for a dam are dependent on the size and hazard classification of the dam. The size and hazard classification will be based on the recommended guidelines adopted by the Texas Natural Resource Conservation Commission (TCEQ) under V.T.C.A, Water Code 12.052, which provides for the safe construction, maintenance, repair and removal of dams located in the State of Texas, and will be determined by the City Engineer based on information furnished by the owner. The following criteria will be used to classify a dam:

1. Size. The classification for size is based on the height of the dam and storage capacity, whichever gives the larger size category. "Height" is defined as the distance between the top of the dam (minus the freeboard) and the existing streambed at the downstream toe. Storage is defined as the maximum water volume impounded at the top of the dam (minus the freeboard).
2. Hazard potential. The hazard potential for a dam is based on the potential for loss of human life and property damage downstream from a dam in the event of failure. Hazard Potential Classifications are based on the potential for loss of life and for the extent of economic loss based on existing and potential development downstream of the dam.
3. Spillway Design Flood. The classification of a dam based on the above criteria will be used to determine the Spillway Design Flood (SDF). The total capacity of a dam structure, including principal and emergency spillways, shall be adequate to pass the SDF without exceeding the top dam elevation. The SDF's for various dam classifications are shown in this Drainage Design Manual described by TCEQ Dam Safety Guidelines.

4.B. All design will be for the fully developed watershed contributing to the structure.

5.C. In all cases, the minimum principal spillway design capacity is a minimum of the 100-year design flood. In certain cases, a dam breach analysis may be required to determine the proper classification of the structure. For all structures requiring a spillway design flood equal to the Probable Maximum Flood (PMF), a dam breach analysis is required to determine the downstream consequences of a failure. All dams shall be constructed with a minimum freeboard of two feet above SDF elevation and upstream development within the contour line determined by the emergency spillway crest elevation plus 2-feet, or the

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100-year flood elevation (based on fully developed watershed conditions) plus 2-feet, whichever is greater is prohibited.

**12.1.3** Maintenance and Liability Criteria

The owner or developer shall retain their private ownership of the constructed lake, pond or lagoon or basin and shall assume full responsibility for the protection of the general public from any health or safety hazards related to the lake, pond or lagoon constructed. The owner or developer shall assume full responsibility for the maintenance of the lake, pond or lagoon or basin constructed. The owner or developer shall keep the City Engineer advised of the currently responsible agent for this maintenance.

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**12.1.4** Natural Resource Conservation Service Lakes

**A.** There are a number of NRCS (previously Soil Conservation Service) lakes within the city limits and extraterritorial jurisdiction of the City of New Braunfels. These lakes present complex issues of flood control, erosion control, maintenance and floodplain management. These lakes were constructed to NRCS standards. The lakes are in private ownership, with maintenance provided by Comal County. Operation of the lakes is the responsibility of Comal County. The City of New Braunfels is responsible for floodplain management of those areas upstream, downstream and adjacent to the lakes. Operation and maintenance of the NRCS lakes shall remain the responsibility of others.

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**B.** The City of New Braunfels shall control future development upstream, downstream and adjacent to all NRCS lakes. Planning for future development which impacts on, or is impacted by, NRCS lakes shall require that a detailed engineering study be performed to provide a technical basis for development and that the dam be upgraded as follows:

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1. Provide principal spillway capacity adequate to discharge the 100-year flood event based on fully developed watershed conditions.

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2. Provide total capacity of the dam structure, including principal and emergency spillways to accommodate the Probable Maximum Flood (PMF).

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3. Maintain existing flood storage capacity.

4. Prohibit upstream development within the contour line determine by the emergency spillway crest elevation plus 2-feet, or the 100-year flood elevation (based on fully developed watershed conditions) plus 2-feet, whichever is greater.

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5. Restrict development and improvements within the floodplain established by a breach flow analysis from the dam to the downstream limits of the dam breach impact.

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**Table 12-1  
Size Classifications Impoundment**

Category	Storage (acre-foot)	Height (feet)
Small	<1,000	<40
Intermediate	≥1,000 and <50,000	≥40 and <100
Large	≥50,000	≥100

**1. Design Frequencies for Dam or Impoundments**

1. the design criteria for a dam is dependent on the size and hazard classification of the dam, as described by the Texas Natural Resource Conservation Commission (TNRCC) under 30 TAC §299.11-§299.14 (5), which provides for the safe construction, maintenance, repair and removal of dams located in the State of Texas. The following criteria from current TNRCC regulations will be used to classify a dam. If the TNRCC criteria are updated, then the revised TNRCC regulations will apply.

1. Size. The classification for size is based on the height of the dam and storage capacity, whichever gives the larger size category. "Height" is defined as the distance between the top of the dam and the existing streambed at the downstream toe. Storage is defined as the maximum water volume impounded at the top of the dam.

2. Hazard Potential. The hazard potential for a dam is based on the potential for loss of human life and property damage downstream from a dam in the event of failure. The categories from Table 12-2 will be used.

3. Spillway Design Flood. The classification of a dam based on the above criteria will be used to determine the Spillway Design Flood (SDF). The total capacity of a dam structure, including principal and emergency spillways, shall be adequate to pass the SDF without a failure of the dam. The SDF is computed as a percentage of the PMF hydrograph for various dam classifications according to the following table:

**Table 12-2  
Hazard Potential Classification**

Category	Loss of Life	Economic Loss
Low	None expected (no permanent structures for human habitation)	Minimal (undeveloped to occasional structures of agriculture)
Significant	Few (no urban Appreciable developments and no more than a small number of inhabitable structures)	(notable agricultural, industry structures)
High	More than few	Excessive (extensive industry or agriculture)

**1.12.1.5 Additional Design Requirements**

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~~1.A.~~ An engineering plan for such construction accomplished by complete drainage design information and sealed by a licensed professional engineer, shall be approved by the City of New Braunfels.

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~~2.B.~~ The spillway and any emergency overflow areas shall be located so that floodwaters will not inundate any permanent habitable structures.

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~~3.C.~~ The minimum SPDF should be the 100-year, 24-hour storm regardless of critical inflow design storm peaks.

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~~4.D.~~ The design shall comply with all federal, state and county laws pertaining to the impoundment of surface water, including the design, construction, and safety of the impounding structure. Copies of any federal, state or county permits issued for proposed impoundments shall be submitted to the City Engineer.

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~~5.E.~~ Any existing NRCS structure or other dams which are included in the project drainage area shall comply with the applicable federal, state, county and city safety requirements for structures. Improvements may be required to upgrade the structure to the currently adopted guidelines. Before removing, enlarging or altering any existing lake, the applicant will furnish a study of the effects of the alteration upon flooding conditions both upstream and downstream. The study shall be prepared by a professional engineer and submitted to the City Engineer for approval prior to making the proposed alteration.

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~~6.F.~~ Any improvements to existing dams or lakes or construction of new impoundments shall be made at the expense of the developer, prior to completion of the adjacent street, utilities and drainage improvements, as provided for under the subdivision regulations.

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#### ~~12.2~~ Levees

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In the event that developers or owners wish to build levees to protect an area from flooding, all applicable FEMA guidelines, State of Texas dam safety guidelines, and the following criteria apply:

~~1.A.~~ Levees shall be designed to have ~~four feet of freeboard above the Standard Project Flood for the fully developed watershed flows~~ freeboard requirements as specified by FEMA.

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~~2.B.~~ Levees shall be designed according to the Corps of Engineers' design criteria used for federally authorized levees, whether or not they are federally authorized.

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~~3.C.~~ Ring levees ~~protecting individual structures proposed for construction after the enactment date of this Ordinance~~ shall not be permitted.

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~~4.D.~~ If possible, provision shall be made to provide the permanent maintenance of levees either by a flood control district or similar governmental organization or by the existing property owner and all future owners, heirs or assigns, through the use of a maintenance agreement.

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~~5.E.~~ Levee systems shall be designed with interior drainage system to prevent flooding from local runoff contained within the system for the 100-year design flood.

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~~6.F.~~ Levee system shall have written operation procedures that address gate-closure conditions and an emergency warning plan. A copy of these procedures shall be furnished to the City Engineer.

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~~7.G.~~ Automated gate-closure systems shall have power from two independent sources and shall be capable of being operated manually.

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8.H. All new levee systems shall have permanent positive closures to the required design elevation. Temporary closures involving sandbagging or other procedures requiring manual operations shall not be permitted.

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9.I. Additional plan requirements including water surface profiles for the design flood and standard project flood; the top of the levee profile, definition of interior drainage facilities, including pump station and ponding areas; location of gravity outlets, gatewells and closure structures; and elevation-duration data on the receiving system.

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**Table 12-3**  
**Spillway Design**

Hazard Potential*	Size	SDF (Flood Hydrograph)
Low	Small	1/4 PMF
	Intermediate	1/4 PMF to 1/2 PMF**
	Large	PMF
Significant	Small	1/4 PMF to 1/2 PMF**
	Intermediate	1/2 PMF to PMF**
	Large	PMF
High	Small	PMF
	Intermediate	PMF
	Large	PMF

\*—In certain cases, a dam breach analysis may be required to determine the proper classification of the structure. For all structures requiring a spillway design flood equal to the PMF, and emergency action plan, prepared according to TNRCC guidelines, is required prior to completion of construction.

\*\*—Where a range is given, the minimum flood hydrograph will be determined by straight line interpolation with the given range. Interpolation shall be based on either hydraulic height or impoundment size, whichever is greater. In all cases, the minimum spillway design capacity is the 100-year design flood.

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**4-13.0 Site Erosion Control During Construction**

**Comment [A25]:** Need City input on how to improve this section. Plan requirements? Enforcement/inspection?

**13.1 Applicable Properties or Construction Sites**

All proposed projects are to submit the TCEQ's TXR 150000 permit for approval by the City for any amounts of disturbance greater than 0.01 acres and implement a storm water pollution prevention plan. If the TXR 150000 permit is required by TCEQ for development, a copy of the approved document will suffice.

Private property owners, developers or builders shall be accountable for any erosion of their property or construction site which results in measurable accumulation of sedimentation in dedicated streets, alleys, any waterway or other private properties. Any accumulation or deposit of soil material beyond the limits of the property or in City streets, alleys or drainage facilities in an amount sufficient to constitute a threat to public safety and comfort as determined by the City Engineer shall constitute a violation. Sediment carried by stormwater runoff through these acres shall be prevented from entering storm drain systems and natural watercourses.

**4-13.2 General Guidelines for Erosion Control Plan**

- ~~1~~A. Maximum use shall be made of vegetation to minimize soil loss.
- ~~2~~B. Natural vegetation should be retained wherever possible.
- ~~3~~C. Where inadequate natural vegetation exists, or where it becomes necessary to remove existing natural vegetation, temporary controls should be installed promptly to minimize soil loss and ensure that erosion and sedimentation does not occur. The developer is responsible for maintenance of site erosion control devices until a sufficient vegetation cover has been provided or replaced as determined by the City Engineer. Periodic maintenance shall be performed by the developer to remove accumulated sediment that would otherwise inhibit the proper functioning of the erosion control devices.
- ~~4~~D. Wherever possible during construction, erosion controls shall be used on hillsides to slow drainage flow rate.
- ~~5~~E. Erosion control elements should be implemented as soon as practical in the development process.
- ~~6~~F. Waste or disposal areas and construction roads should be located and constructed in a manner that will minimize the amount of sediment entering streams.
- ~~7~~G. Frequent fording of live streams will not be permitted; therefore, temporary bridges or other structures shall be used wherever an appreciable number of crossings of a stream are necessary.
- ~~8~~H. When work areas or material sources are located in or adjacent to live streams, such areas shall be separated from the stream by a dike or other barrier to keep sediment from entering a flowing stream. Care shall be taken during the construction and removal of such barriers to minimize the sediment transport into a stream.
- ~~9~~I. Should preventative measures fail to function effectively, the applicant shall act immediately to bring the erosion and/or siltation under control by whatever additional means are necessary.
- ~~10~~J. Erosion control devices shall be placed to trap any losses from stockpiled topsoil. Some acceptable forms of site erosion control devices include, but are not limited to, silt fences, silt traps, hay bale barriers, geonetting and geotextiles. ~~Hay bale barriers, if~~

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~~used, shall be replaced with new hay bales as a part of the regular maintenance program.~~

~~11.K.~~ The selection and timing of the installation of erosion controls shall be based upon weather and seasonal conditions that could make certain controls not practicable.

~~12.L.~~ Vegetation used for vegetative cover shall be suitable for local soil and weather conditions. Ground cover plants shall comply with listings from the Texas Agricultural Extension Service.

~~13.M.~~ Runoff shall be diverted away from construction areas as much as possible.

~~14.N.~~ Stripping of vegetation from project sites shall be phased so as to expose the minimum amount of area to soil erosion for the shortest possible period of time. Phasing shall also consider the varying requirements of an erosion control plan at different stages of construction.

~~O.~~ Developers, builders, or owners of property shall install all utilities, including franchise utilities, before final acceptance of a subdivision, property and/or structure. Final acceptance will also be contingent upon having all necessary erosion control measures installed to minimize off-site sediment. At the discretion of the City Engineer; a site may be accepted without erosion control measures if perennial vegetative cover is actively growing.

~~15.P.~~ SW3P shall follow TCEQ and TxDOT guidelines and standard details unless approved by the City Engineer.

### 13.3 Applicable Best Management Practices

Published Best Management Practices for erosion control for many different activities are available through libraries, Internet, and through the City Engineer.

### 13.4 Stream Bank Erosion

Erosion control will be provided along streams and drainage channels. Where bank stabilization or other erosion protection measures are required to protect streams and channels, the stream bank protection and erosion damage mitigation measures provided in this Manual shall be utilized as approved by the City Engineer.

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14.0 Storm Water Quality

**Comment [A26]:** This is where the LID chapter could be placed

**APPENDIX A  
LIST OF REFERENCES**

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- (2) USGS, "Depth-Direction Frequency of Precipitation for Texas", Water Resources Investigations Report 98-4044, 1998.
- (3) National Oceanic and Atmospheric Administration, "Probable Maximum Precipitation Estimates, United States East of the 105th Meridian", Hydro-meteorological Report No. 51, 1978.
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- (5) Texas, Secretary of State. "Official Texas Administrative Code. Title 30-Environmental Quality." §299.12- §299.14, West Group, St. Paul, Minn. 1997.
- (6) ASCE and Water Pollution Control Federation, "Quantity of Stormwater, Manuals and Reports of Engineering Practice - No. 37," New York, N.Y., 1969.
- ~~(7) City of Fort Worth, "Storm Drainage Criteria and Design Manual," Fort Worth, Texas, 1967.~~
- (8) Highway Administration, "Urban Drainage Design Manual, Hydraulic Engineering Circular No. 22", Washington, D.C., 1996.
- (9) U.S. Soil Conservation Service, "Urban Hydrology for Small Water Sheds", Technical Release No. 55, January 1975.
- (10) U.S. Army Corps of Engineers, "Flood Hydrograph Analysis and Computations", EM 1110-2- 1405, 1959.
- (11) Viessman, Warren, "Introduction to Hydrology", Harper and Row, Publishers, New York, 1996.
- ~~(12) U.S. Army Corps of Engineers, "NUDALLAS, Documentation and Supporting Appendices", 1986.~~
- (13) U.S. Soil Conservation Service, "National Engineering Handbook", 1972.
- (14) U.S. Soil Conservation Service, Engineering Technical Note NO. 210-1 8-TX-I, "Emergency Spillway and Freeboard Hydrograph Development", August, 1982.
- ~~(15) U.S. Soil Conservation Service, "Soil Survey of Tom Green County, Texas", October 1976.~~
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- ~~(17) U.S. Soil Conservation Service, Technical Release No. 20, "TR-20", Project Formulation Hydrology," August 1972.~~
- (18) Chow, V.T., "Open Channel Hydraulics." McGraw-Hill Book Co., Inc., New York, 1959.
- (19) Merritt, Frederick S., "Standard Handbook for Engineers", McGraw-Hill Book Co., Inc., New York, 1986.
- (20) U.S. Army Corps of Engineers, "HEC-2 Water Surface Profile User's Manual", 1982. (21) U.S. Army Corps of Engineers, "HEC-RAS River Analysis System User's Manual", 1997. (22) Rodman, Paul K., "Effects of Urbanization on Various Frequency Peak Discharges," 1977.

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**— APPENDIX B  
LIST OF ABBREVIATIONS**

$\theta$	Slope of inlet gutter depression
a	Gutter depression
ac	Acres
A	Area
AMC	Antecedent moisture condition
ASCE	American Society of Civil Engineers
b	Width of partial flow in circular conduit
C	Dimension less weighted runoff coefficient used in the Rational Method to account for ground cover and/or land use within the watershed
CA	Product of runoff coefficient and drainage area used in Rational Method
cfs	Cubic feet per second
CLOMR	Conditional letter of map revision
Cp	Coefficient of peak discharge used in Snyder's unit hydrograph method to account for flood wave and storage conditions
Ct	Dimension less coefficient used in Snyder's unit hydrograph method related to the watershed slopes and storage
D	Diameter
$d_c$	Critical depth
FEMA	Federal Emergency Management Agency
fps	Feet per second
Ft	Feet
g	Acceleration due to gravity (32.2 fps)
SAS MMP	San Angelo Stormwater Management Master Plan
$h, H$	Head
$h_b$	Headloss at a bend
$H_j$	Headloss at a junction
$H_L$	Total headloss
HW	Headwater depth
I	Rainfall intensity
K	Dimension less coefficient used in the Ration Method to account for antecedent precipitation
$K_b$	Headloss coefficient at a bend
$K_e$	Entrance loss coefficient
$K_j$	Headloss coefficient at a junction
L	Length
$L_a$	Length of curb inlet require for 100% interception
$L_{ca}$	River mileage from design point to center of gravity of drainage area
$L_g$	Gutter flow length
$L_o$	Overland flow length
Min.	Minimum or minutes
MSL	Mean sea level
$n, N$	Roughness coefficient used in Manning's formula
P	Wetted perimeter of flow
PMF	Probable maximum flood
q	Peak design discharge per unit area
$Q, Q_p$	Peak design discharge
$Q_a$	Approach flow in gutter upstream of curb inlet
$q_p$	Peak rate of discharge of unit hydrograph for unit rainfall duration, $t_r$
$q_{pR}$	Peak rate of discharge of unit hydrograph for unit rainfall duration, $t_R$

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ROW	Right-of-way
$S, S_o, S_g$	Ground slope, overland ground slope, or gutter flow ground slope.
SCS	Soil Conservation Service
$S_e$	Slope of energy gradient
$S_f$	Slope of frictional gradient
$S_p$	Spread of water from curb toward the street centerline for peak flow
$S_w$	Slope of hydraulic gradient
$t_c$	Time of concentration
$t_p$	Hydrograph lag time from midpoint of rainfall duration, $t_r$ , to peak of unit hydrograph
$t_{pR}$	Lag time from midpoint of unit rainfall duration, $t_r$ , to peak of unit hydrograph
$t_r$	Standard unit rainfall duration
$t_R$	Unit rainfall duration in hours other than the standard unit $t_r$
T	Top width of flow
$v, V$	Velocity
$w, W$	Width
$y, Y$	Flow depth
z	Reciprocal of street cross slope

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## APPENDIX C DEFINITIONS OF TERMS

Abstractions.	The fractions of precipitation lost to evaporation, transpiration, interception, depression storage and infiltration.
Abutment.	A wall supporting the end of a bridge or span, and sustaining the pressure of the abutting earth.
Apron.	A floor or lining of concrete, timber, or other suitable material at the toe of a dam, entrance or discharge side of spillway, a chute, or other discharge structure, to protect the waterway from erosion from falling water or turbulent flow.
Arterial street.	Any major street defined as an arterial street in the City of New Braunfels Comprehensive Traffic Plan, or as designated on official maps of the City of New Braunfels.
Backwater.	The rise of the water level upstream due to an obstruction or constriction in the channel.
Backwater Curve.	The term applied to the longitudinal profile of the water surface in an open channel when flow is steady but non-uniform.
Baffle Chute.	A drop structure in a channel with baffles for energy dissipation to permit the lowering of the hydraulic energy gradient in a short distance to accommodate topography.
Baffles.	Deflector vanes, guides, grids, gratings, or similar devices constructed or placed in flowing water, to: (a) check or effect a more uniform distribution of velocities; (b) absorb energy; (c) divert, guide, or agitate the liquids; and (d) check eddy currents.
Base Flood.	The base flood for the City of San Angelo is defined as the 100-year frequency flood based on fully developed watershed conditions. The base flood elevation is the water surface elevation developed using the base flood as defined in part II of this manual. The City of San Angelo base flood elevation will not necessarily correspond <u>with</u> FEMA base flood elevation.
Calibration.	Process of checking, adjusting, or standardizing operating characteristics of instruments and model appurtenances on a physical model or coefficients in a mathematical model. The process of evaluating the scale readings of an instrument in terms of the physical quantity to be measured.
Channel.	Any <u>open to air</u> arroyo, stream, wash, swale, gully, ditch, diversion, or watercourse that conveys storm runoff, including manmade facilities.
Channel Roughness.	Irregularities in channel configuration which retard the flow of water and dissipate its energy.
Channel stability.	A condition in which a channel neither degrades to the degree that structures, utilities or private property are endangered, nor aggrades to the degree that flow

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capacity is significantly diminished as a result of one or more storm runoff events or moves laterally to the degree that adjacent property is endangered.

Channel treatment measure.	A physical alteration of a channel for any purpose.
Chute.	An inclined conduit or structure used for conveying water to a lower level.
City Engineer.	City of New Braunfels staff individual responsible for administration, processing and compliance with provisions of this ordinance or his/her designated representative.
Comprehensive plan.	The City of New Braunfels Comprehensive Plan and amendments thereto.
Conduit.	Any open or closed device for conveying flowing water.
Contaminants.	Contaminating agents such as oil wastes, sewage, chemicals, etc.
Continuity.	Continuity of flow exists between two sections of a pipe or channel when the same quantity of water passes the two cross sections and all intermediate cross sections at any one instant.
Critical Flow.	The state of flow which exhibits the following characteristics: (a) the specific energy is a minimum for a given discharge; (b) the discharge is a maximum for a given specific energy; (c) the specific force is a minimum for a given discharge; (d) the velocity head is equal to half the hydraulic depth in a channel of small slope; (e) the Froude Number is equal to unity.
Crown.	(a) The highest point on a transverse section of conduit; (b) the highest point of a roadway cross section.
Culvert.	Large pipe or other conduit through which a stormwater flows under a road or street.
Curb.	A vertical or sloping rim along the edge of a roadway, normally constructed integrally with the gutter, which strengthens and protects the pavement edge and clearly defines the pavement edge to vehicle operators.
Curb Inlet.	A vertical opening in a curb through which the gutter flow passes. The gutter may be undepressed or depressed in the area of the curb opening.
Curb Split Dam.	The elevation difference between curbs on opposite sides of a street.
Dam.	A barrier constructed across a watercourse for the purposes of (a) creating a reservoir; (b) diverting water from a conduit or channel.
Degradation.	The progressive general lowering of a stream channel by erosion.
Depression Storage.	Collection and storage of rainfall in natural depressions (small puddles) after exceeding infiltration capacity of the soil.
Design Storm or Flood.	The storm or flood which is used as the basis for design, i.e., against which the structure is designed to provide a stated degree of protection or other specified result.

Detention. The storage of storm runoff for a controlled release during or immediately following the design storm.

1-a. Off-site detention - A detention pond located outside the boundary of the area it serves.

2-b. On-site detention - A detention pond which is located within and serves only a specific site or subdivision.

3-c. On-stream detention - Detention facilities provided to control excess runoff based on a watershed-wide hydrologic analysis.

Developed land. Any lot or parcel of land occupied by any structure or manmade change intended for human occupation or use, including structures intended for commercial or industrial enterprise.

Developer. Any individual, estate, trust, receiver, cooperative association, club, corporation, company, firm, partnership, joint venture, syndicate or other entity engaging in platting, subdivision, filling, grading, excavating, or construction of structures.

Downstream capacity. The ability of downstream drainage facilities to accept and safely convey runoff generated upstream.

Drainage way. Any path of concentrated flow which drains more than 1/4 acre. Watercourse is typically used for larger drainage ways. Channel is a more general term.

Drainage basin. The storm water catchment area above a point on a channel to which waters drain and collect. Watershed has the same meaning.

Drainage control. The treatment and/or management of surface runoff.

Drainage easement. A platted area reserved for the primary purpose of stormwater drainage and maintenance.

Drainage System. Drainage systems shall include streets, alleys, storm drains, drainage channels, culverts, bridges, overflow swales and any other facility through which or over which storm water flows.

Drop Inlet. A storm drain intake structure typically located in unpaved areas. The inlet may extend above the ground level with openings on one or more sides or it may be flush with the ground with a grated cover.

Drop Structures. A sloping or vertical section of a channel designed to reduce the elevation of flowing water without increasing its velocity.

Entrance Head. The head required to cause flow into a conduit or other structure; it includes both entrance loss and velocity head.

Entrance Loss. Head lost in eddies or friction at the inlet to a conduit, headwall or structure.

Erosion control. Treatment measures for the prevention of damages due to soil movement and to deposition.

Evaporation. Process by which water is transferred from land and water masses to the atmosphere.

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Excavation.	Digging and removal of earth by mechanical means.
Exceedance Probability.	The statistical probability that an event will equal or exceed a specific magnitude.
Fill.	The placement of material such as soil or rock to replace existing material, or to create an elevated embankment. Fill also refers to the material which is placed.
Flash Flood.	A flood of short duration with a relatively high peak rate of flow, usually resulting from high intensity rainfall over a small area.
Flexible Pipe.	Any corrugated metal pipe, pipe-arch, sectional plate pipe, sectional plate pipe-arch or plastic (polyethylene) pipe.
Flood or Flooding.	A general and temporary condition of inundation of normally dry land areas by surface runoff. The 100-year flood is the flow rate with a 1% probability of being equaled or exceeded in any one year.
Flood Control.	The elimination or reduction of flood damage by the construction of flood storage reservoirs, channel improvements, dikes and levees, bypass channels, or other engineering works.
Flood Hazard Area.	Area subject to flooding by 100-year frequency floods.
Flood Hazard Mitigation.	See Stormwater Management.
Flood Management.	See Stormwater Management.
Floodplain.	Geographically the entire area subject to flooding. In usual practice, it is the area subject to flooding by the 100-year frequency flood. In this manual, the "100-year floodplain" refers to the floodplain resulting from a 100-year flood based on ultimate watershed development conditions. The "FEMA floodplain" shall refer to the area subject to flooding resulting from the 100-year flood for current watershed development conditions.
Floodway.	The channel of a river or other watercourse and the adjacent land areas that must be reserved in order to discharge the base flood without cumulatively increasing the water surface elevation by more than a designated height. In this manual, the floodway refers to the floodway resulting from a 100-year flood event based on ultimate development conditions with a cumulative increase of no more than one foot.
Floodway Fringe.	Part of the flood hazard area within the floodplain but outside of the floodway.
Freeboard.	The distance between the normal operating level and the top of the side of an open conduit left to allow for wave action, floating debris, or any other condition or emergency without overtopping the structure.
Frequency.	Average recurrence interval of a given flood event over long periods of time. Mathematically, frequency is the reciprocal of the exceedance probability.
Froude Number.	A flow parameter which is a measure of the extent to which gravitational action affects the flow. A Froude number greater than one indicates supercritical flow

and a value less than one indicates subcritical flow. The simplest form of the Froude number is given by the equation:  $F = V/(gD)^{0.5}$

where: V=Velocity in ft./ft./sec  
g = the acceleration due to gravity (32.2ft t/see2)  
D= depth in ft.

- Fully developed watershed. A hydrologic condition in which all areas upstream and downstream of a point in question are assumed completely developed, including any undeveloped areas which are assumed to be developed in accordance with development densities established in the Comprehensive Zoning Map of the City of New Braunfels.
- Gabion. A wire basket containing earth or stones, deposited with others to provide protection against erosion.
- Grade. (a) The inclination or slope of a channel, canal, conduit, etc., or natural ground surface, usually expressed in terms of the percentage of number of units of vertical rise (or fall) per unit of horizontal distance. (b) The elevation of the bottom of a conduit, canal, culvert, sewer, etc. and etc. (c) The finished surface of a canal bed, road bed, top of an embankment, or bottom of excavation.
- Grading. Any movement of soil, rock or vegetation by artificial means, to include any or all of the following acts: clearing, grubbing, excavating, placement of fill material, or grading of land.
- Grate Inlet. An opening in the gutter covered by one or more grates through which the water falls. As with all inlets, grated inlets may be either depressed or undepressed and may be located either on a continuous grade or in a sump.
- Gutter. A generally shallow waterway adjacent to a curb, used or suitable for drainage of water.
- Head. The amount of energy per pound of fluid.
- Headwater. (a) The upper reaches of a stream near its sources; (b) the region where ground waters emerge to form a surface stream; (c) the water upstream from a structure.
- High Intensity Node. Areas of existing or proposed development that contain a large concentration of buildings and large amounts of pavement. High Intensity nodes typically generate large volumes of storm water runoff.
- Histogram. Representation of statistical data by means of rectangles whose widths represent rainfall, runoff, etc. and whose height represents frequency.
- Hydraulic Control. The hydraulic characteristic which determines the stage- discharge relationship in a flowing stream or conduit. The control is usually critical depth, tailwater depth or uniform depth.
- Hydraulic Grade Line. A line representing the pressure head available at any given point within the system.

Hydraulic Gradient.	A hydraulic profile of the piezometric level of the water, representing the sum of the depth of flow and the pressure head. In open channel flow it is the water surface.
Hydraulic Jump.	The hydraulic jump is an abrupt rise in the water surface which occurs in an open channel when water flowing at supercritical velocity is retarded by water flowing at subcritical velocity. The transition through the jump results in a marked loss of energy, evidenced by turbulence of the flow within the area of the jump. The hydraulic jump is sometimes used as a means of energy dissipation.
Hydraulics.	A branch of science that deals with practical applications of the mechanics of water movement.
Hydrograph.	A graph showing flow (or sometimes stage, velocity or other properties of water) versus time at a given point on a stream or conduit.
Hydrology.	The science that deals with the processes governing the depletion and replenishment of the water resources of the land areas of the earth.
Hyetograph.	A histogram or graph of rainfall intensity versus time usually during a storm.
Impervious.	A term applied to a material through which water cannot pass, or through which water passes with great difficulty.
Infiltration.	(a) The entering of water through the interstices or pores of a soil or other porous medium; (b) the quantity of groundwater which leaks into a sanitary or combined sewer or drain through defective joints, breaks or porous walls; (c) The absorption of water by soil, either as it falls as precipitation or from a stream flowing over the surface.
Inlet.	(a) An opening into a storm sewer system for the entrance of surface storm runoff, more completely described as a storm sewer inlet; (b) a structure at the diversion end of a conduit; (c) the upstream connection between the surface of the ground and a drain or sewer, for the admission of surface or storm water.
Intensity.	As applied to rainfall, a rate usually expressed in inches per hour.
Interception.	As applied to hydrology, refers to the process by which precipitation is caught and held by foliage, twigs, and branches of trees, shrubs and buildings, never reaching the surface of the ground, and then lost by evaporation.
Invert.	The floor, bottom, or lowest portion of the internal cross-section of a conduit. Used particularly with reference to storm drains, sewers, tunnels, channels and swales.
Lag Time.	The time difference between two occurrences, such as between rainfall and runoff or pumping of a well and effect on the stream. See Time of Concentration.
Lining.	Impervious material such as concrete, clay, grass, plastic, etc., placed on the sides and bottom of a ditch, channel, and reservoir to prevent or reduce seepage of water through the sides and bottom and/or to prevent erosion.
Lip.	A small wall on the downstream end of an apron, to break the flow from the apron.

Maintenance.	The cleaning, shaping, grading, repair and minor replacement of drainage, flood control and erosion facilities, but not including the cost of power consumed in the normal operation of pump stations.
Major facility.	Any facility, including a street or alley, which would collect, divert, or convey a peak discharge of more than 50 cubic feet per second (50 cfs) or store more than 2.0 acre-feet of runoff in the event of a 50-year design storm.
<u>Major stream.</u>	<u>Any stream which has a drainage basin of 10 square miles (mi<sup>2</sup>) or greater.</u>
<u>Manning Coefficient.</u>	<u>The coefficient of roughness used in the Manning Equation for flow in open channels.</u>
<del>Manning Coefficient.</del>	<del>The coefficient of roughness used in the Manning Equation for flow in open channels.</del>
Manning Equation.	A uniform flow equation used to relate velocity, hydraulic radius and the energy gradient.
Minor facility.	Any facility which would collect, divert or convey a peak discharge of 50 cubic feet per second (50 cfs) or less, or store 2-acre feet of water or less in the event of the 50-year design storm.
Model.	Mathematical systems analysis by computer, applied to evaluate rainfall-runoff relationships; simulate watershed characteristics; predict flood and reservoir routings; or for other aspects of planning.
Multiple use facility.	A drainage control, flood control or erosion control facility in which other secondary uses are planned or allowed, including but not limited to recreation, open space, transportation, and utility location.
Nappe.	The sheet or curtain of water overflowing a weir or dam. When freely overflowing any given structure, it has a well- defined upper and lower surface.
100-year Event.	Event (rainfall or flood) that statistically has a one percent chance of being equaled or exceeded in any given year.
Open Channel.	A conduit in which water flows with a free surface.
Orifice.	(a) An opening with closed perimeter and regular form in a plate, wall, or partition, through which water may flow; (b) the end of a small tube, such as a Pitot tube, piezometer, etc.
Peak Flow.	The maximum rate of runoff during a given runoff event.
Percolation.	To pass through a permeable substance such as ground water flowing through an aquifer.
Permeability.	The property of a material which permits movement of water through it when saturated and actuated by hydrostatic pressure.
Pervious.	Applied to a material through which water passes relatively freely.

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Porosity.	(a) An index of the void characteristics of a soil or stratum as pertaining to percolation; degree of perviousness; (b) the ratio, usually expressed as a percentage, of the volume of the interstices in a quantity of material to the total volume of the material.
Post-development.	The condition of the given site and drainage area after the anticipated development has <del>taketaketaken</del> place.
Precipitation.	Any moisture that falls from the atmosphere, including snow, sleet, rain and hail.
Pre-development.	The condition of the given site and drainage area prior to development.
Prismatic Channel.	A channel with unvarying cross-section and constant bottom slope.
Probable Maximum Flood (PMF).	The flood that may be expected from the most severe meteorological and hydrologic conditions that are reasonably possible in the region.
Probable Maximum Precipitation (PMP).	The critical depth-duration-area rainfall relationship for a given area during a storm containing the most critical meteorological conditions considered probable of occurring.
Rainfall Duration.	The length of time over which a single rainfall event occurs.
Rainfall Frequency.	The average recurrence interval of rainfall events, averaged over long periods of time.
Rainfall Intensity.	The rate of accumulation of rainfall, usually in inches or millimeters per hour.
Rational Formula.	A traditional means of relating runoff from a drainage basin to the intensity of the storm rainfall, the size of the basin, and the characteristics of the basin (such as land use, impervious cover).
Reach.	Any length of river or channel. Normally refers to sections which are uniform with respect to discharge, depth, area or slope, or sections between gaging stations.
Recurrence Interval.	The average interval of time within which a given event is statistically predicted to be equaled or exceeded once. For an annual series (as opposed to a partial duration series), it is the probability of occurrence interval. Thus a flood having a recurrence interval of 100 years has a one percent probability of being equaled or exceeded.
Return Period.	See Recurrence Interval
Reynold's Number.	A flow parameter which is a measure of the viscous effects on the flow. Typically defined as: $Re = VD/\nu$ <p>where, V = Velocity  D = Depth  <math>\nu</math> = Kinematic viscosity of the fluid</p>
Rigid Pipe.	Any concrete, clay or cast iron pipe.

Riprap (Revetment).	Forms of bank channel protection, usually using rock or concrete. Riprap is a term sometimes applied to stone which is dumped rather than placed more carefully.
Routing.	Routing is a technique used to predict the temporal and spatial variations of a flood wave as it traverses a river reach or reservoir. Generally, routing technique may be classified into two categories - hydrologic routing and hydraulic routing.
ROW (Right-of-Way).	A strip of land dedicated for public streets and/or related facilities, including utilities, drainage systems and other transportation uses.
ROW Width.	The shortest horizontal distance between the lines which delineate the limits of right-of-way of a street.
Runoff.	That part of the precipitation that exceeds abstractions and reaches a stream or storm drain.
Runoff Coefficient.	A decimal number used in the Rational Formula, which defines the runoff characteristics (i.e., land use impervious cover) of the drainage area under consideration. It may be applied to an entire drainage basin as a composite representation or it may be applied to a small individual area such as one residential lot.
Runoff Total.	The total volume of flow from a drainage area for a definite period of time such as a day, month, year, or for the duration of a particular storm.
Scour.	The erosive action of running water in streams or channels in excavating and carrying away material from the bed and banks.
SCS Runoff Curve Number.	Index number used by the Soil Conservation Service as a measure of the tendency of rainfall to run off into streams rather than evaporate or infiltrate.
Sediment.	Material of soil and rock origin transported, carried, or deposited by flowing water.
Sidewalk.	A paved area within the street right-of-way specifically designed for pedestrians and/or bicyclists.
Slope, Critical.	(a) The slope or grade of a channel that is exactly equal to the loss of head per foot resulting from flow at a depth that will give uniform flow at critical depth; (b) the slope of a conduit which will produce critical flow.
Slope, Friction.	The friction head or loss per unit length of channel or conduit. For uniform flow the friction slope coincides with the energy gradient. Where a distinction is made between energy losses due to bends, expansions, impacts, etc., a distinction must also be made between the friction slope and the energy gradient. The friction slope is equal to the bed or surface slope only for uniform flow in uniform open channels.
Soffit.	The top of the inside of a pipe. In a pipe, the uppermost point on the inside of the structure.

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Spillway.	A waterway in or about a dam or other hydraulic structure for the escape of excess water.
Steady Flow.	Open channel flow is said to be steady if the depth of flow does not change and can be assumed to be constant during the time interval under consideration.
Stilling Basin.	Pool of water conventionally used, as part of a drop structure or other structure, to dissipate energy.
Storm Hydrology.	The branch of hydrology that concentrates on the calculation of runoff from storm rainfall.
Stormwater Management.	The control of storm runoff by means of land use restrictions, detention storage, erosion control, and/or drainage systems
Stormwater Model.	Mathematical method of solving stormwater problems by computer technology.

**Street Classifications:**

Alley-	An alley is a passageway designed primarily to provide access to or from the rear or side of property otherwise abutting on a public street.
Arterial Street-	Arterial streets are designed to carry high volumes of through traffic. Arterial streets serve as a link between major activity centers within the urban area.
Collector Street-	The primary function of a collector street is to serve abutting traffic from intersecting local streets and expedite the movement of this traffic in the most direct route to an arterial street or other collector street.
Freeway-	Freeways are divided arterial highways designed with full control of access and grade separations at all intersections. Freeways provide movement of high volumes of traffic at relatively high speeds. This system carries most of the trips entering and leaving the urban area, as well as most of the through movements bypassing the central city.
Local-	The primary function of a local street is to serve abutting land use and traffic within a neighborhood or limited residential district. A local street is not generally continuous through several districts.
Parkway-	(a) a freeway which does not have continuous frontage roads; (b) greenspace buffer between the roadway and adjacent development.

**Comment [A27]:** to be removed and referenced elsewhere, per Octavio

Subcritical Flow.	Relatively deep, tranquil flow with low flow velocities. The Froude Number is less than 1.0 for subcritical flow conditions.
Supercritical Flow.	Relatively shallow, turbulent flow with high velocities. The Froude Number is greater than 1.0 for supercritical flow conditions.
Tailwater.	The depth of flow in the stream directly downstream of a drainage facility or other man-made control structure.

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Temporary drainage facility.	A non-permanent drainage control, flood control or erosion control facility constructed as part of a phased project or to serve until such time that a permanent facility is in place, including but not limited to desilting ponds, berms, diversions, channels, detention ponds, erosion control measures, bank protection and channel stabilization measures.
Time of Concentration.	The estimated time in minutes required for runoff to flow from the most hydraulically remote section of the drainage area to the point at which the flow is to be determined. Hydraulically remote refer to the travel path with the longest flow travel time, not necessarily the longest linear distance.
Total Head Line.	A line representing the energy in flowing water. The elevation of the energy line is equal to the elevation of the flow line plus the depth plus the velocity head plus the pressure head.
Trash Rack.	Racks, gratings, or mesh designed so as to prevent tree limbs, water-borne debris and rubbish from plugging the outlets from a dam or detention basin.
Trunk Line.	The main line of a storm drain system, extending from manhole to manhole or from manhole to outlet structure.
Ultimate Development.	The condition of the watershed after the entire watershed has undergone development.
Uniform Channel.	A channel with a constant cross section, slope and roughness.
Uniform Flow.	Open channel flow is said to be uniform if the depth of flow is the same at every section of the channel.
Unit Hydrograph.	The direct runoff hydrograph resulting from one inch of precipitation excess, distributed uniformly over a watershed for a specified duration.
Velocity Head.	The energy per unit weight of water due to its velocity. The velocity head also represents the vertical distance water must all freely under gravity to reach its velocity. The velocity head can be computed from: $H_v = V^2/2g$
	where, $H_v$ = Velocity head in $\text{ft/ft}$ $V$ = Velocity in $\text{ft/ft}/\text{sec}$ $g$ = acceleration due to gravity $32,2 \text{ ft/sec}^2$ )
Warped Headwall.	The wingwalls are tapered from vertical at the abutment to a stable bank slope at the end of the wall.
Water Year.	The water year commonly used in the United States is the period from October 1 of the previous calendar year to September 30 of the numbered calendar year.
Watershed.	The area contributing storm runoff to a stream or drainage system. Other terms are drainage area, drainage basin and catchment area.

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APPENDIX D  
EXAMPLES

Comment [A28]: others?

1. Sizing drainage basin using Rational Method
2. Input file for TR-20 with basin
3. Input file for HEC-1 run

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### SIZING DRAINAGE BASIN USING RATIONAL METHOD

**GIVEN:** A 10-acre site, currently agricultural use, is to be developed for townhouses. The entire area is the drainage area of the proposed detention basin.

**DETERMINE:** Maximum release rate and required detention storage.

**SOLUTION:**

1. Determine 100-year peak run-off rate prior to site development. This is the maximum release rate from site after development.
2. Determine inflow hydrograph for storms of various durations in order to determine maximum volume required with release rate determined in Step 1. NOTE: Incrementally increase durations by 10- minutes to determine maximum required volume. The duration with a peak inflow less than maximum release rate or where required storage is less than storage for the prior duration is the last increment.

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STEP 1. Calculate Peak Discharge for Present Conditions

$$\begin{aligned} Q &= KCIA \\ K &= 1.25 \\ C &= 0.30 \\ T_c &= 20 \text{ minutes} \\ i_{100} &= 7.0 \text{ in./hr} \\ Q_{100} &= 1.25 * 0.30 * (7.0) * 10 = 26.25 \text{ cfs (Maximum release rate)} \end{aligned}$$

STEP 2. Future Conditions (Townhouses)

$$\begin{aligned} K &= 1.25 \\ C &= 0.80 \\ T_c &= 15 \text{ minutes} \\ i_{100} &= 7.7 \text{ in./hr.} \\ Q_{100} &= 1.25 * 0.80 * ( ) * 10 = \text{ cfs} \end{aligned}$$

Check various duration storms:

$$\begin{aligned} 15 \text{ minutes} & i = Q = 1.25 * 0.80 * ( ) * 10 = \text{ cfs} \\ 20 \text{ minutes} & i = Q = 1.25 * 0.80 * ( ) * 10 = \text{ cfs} \\ 30 \text{ minutes} & i = Q = 1.25 * 0.80 * ( ) * 10 = \text{ cfs} \\ 40 \text{ minutes} & i = Q = 1.25 * 0.80 * ( ) * 10 = \text{ cfs} \\ 50 \text{ minutes} & i = Q = 1.25 * 0.80 * ( ) * 10 = \text{ cfs} \\ 60 \text{ minutes} & i = Q = 1.25 * 0.80 * ( ) * 10 = \text{ cfs} \\ 70 \text{ minutes} & i = Q = 1.25 * 0.80 * ( ) * 10 = \text{ cfs} \\ 80 \text{ minutes} & i = Q = 1.25 * 0.80 * ( ) * 10 = \text{ cfs} \\ 90 \text{ minutes} & i = Q = 1.25 * 0.80 * ( ) * 10 = \text{ cfs} \end{aligned}$$

Maximum Storage Volume is determined by deducting the volume of runoff released during the time of inflow from the total inflow for each storm duration.

$$\begin{aligned} \text{Inflow} &= T_c * Q * 60 \text{ sec/min} \\ \text{Outflow} &= 0.5 * (T_i + T_o) * Q_0 * 60 \text{ sec/min} \\ \text{Storage} &= \text{Inflow} - \text{Outflow} \\ \text{where: } T_c &= \text{Time of concentration, (min) for that duration} \\ Q &= \text{Flow for that } T_o, \text{ (cfs)} \\ T_i &= \text{Time of concentration of the basin} \\ Q_0 &= \text{Original flow, pre-development conditions} \end{aligned}$$

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15 min. Storm	Inflow $15 * (61.6) * 60 \text{ sec/min}$	=	55,440 cf
	Outflow $(0.5) * (15+15) * (21.0) * 60 \text{ sec/min}$	=	<u>18,900 cf</u>
	Storage	=	36,540 cf
20 min. Storm	Inflow $20 * (56.0) * 60 \text{ sec/min}$	=	67,200 cf
	Outflow $(0.5) * (15+20) * (21.0) * 60 \text{ sec/min}$	=	<u>22,050 cf</u>
	Storage	=	45,150 cf
30 min. Storm	Inflow $30 * (46.4) * 60 \text{ sec/min}$	=	83,520 cf
	Outflow $(0.5) * (15+30) * (21.0) * 60 \text{ sec/rain}$	=	<u>28,350 cf</u>
	Storage	=	55,170 cf
40 min. Storm	Inflow $40 * (40.0) * 60 \text{ sec/min}$	=	96,000 cf
	Outflow $(0.5) * (15+40) * (21.0) * 60 \text{ sec/min}$	=	<u>34,650 cf</u>
	Storage	=	61,350 cf
50 min. Storm	Inflow $50 * (35.2) * 60 \text{ sec/min}$	=	105,600 cf
	Outflow $(0.5) * (15+50) * (21.0) * 60 \text{ sec/min}$	=	<u>40,950 cf</u>
	Storage	=	64,650 cf
60 min. Storm	Inflow $60 * (32.0) * 60 \text{ sec/min}$	=	115,200 cf
	Outflow $(0.5) * (15+60) * (21.0) * 60 \text{ sec/min}$	=	<u>47,250 cf</u>
	Storage	=	67,950 cf
70 min. Storm	Inflow $70 * (29.6) * 60 \text{ sec/min}$	=	124,320 cf
	Outflow $(0.5) * (15+60) * (21.0) * 60 \text{ sec/rain}$	=	<u>53,550 cf</u>
	Storage	=	70,770 cf
80 min. Storm	Inflow $80 * (27.2) * 60 \text{ sec/min}$	=	130,560 cf
	Outflow $(0.5) * (15+80) * (21.0) * 60 \text{ sec/rain}$	=	<u>59,850 cf</u>
	Storage	=	70,710 cf
90 min. Storm	Inflow $90 * (24.8) * 60 \text{ sec/min}$	=	133,920 cf
	Outflow $(0.5) * (15+90) * (21.0) * 60 \text{ sec/min}$	=	<u>66,150 cf</u>
	Storage	=	67,770 cf

Maximum volume required is 70,770 cfs at the 70 min. storm duration.

INPUT FILE FOR TR-20 WITH BASIN

```

JOB TR-20 COTTON          FULLPRINT          10
TITLE  JOB USES  24HR TYPE II STORM, FROM STR 1 TO XSEC 1          20
TITLE  WITH FULLPRINT, CROSS SECTION DATA, & OUTPUT OPTIONS ON.  30
-2 XSECTN 001          1.00          508.50          40
-8          503.50          0.0          0.0          50
-8          505.21          100.00          57.25          60
-8          506.74          300.00          118.20          70
-8          508.28          600.00          189.25          80
-8          508.50          624.56          200.00          90
-8          509.51          1000.00          364.35          100
-8          510.37          1500.00          662.99          110
-8          511.07          2100.00          963.82          120
-8          511.44          2500.00          1134.18          130
-9 ENDTBL          140
-2 XSECTN 002          1.00          508.50          40
-8          503.50          0.0          0.0          50
-8          505.21          100.00          57.25          60
-8          506.74          300.00          118.20          70
-8          508.28          600.00          189.25          80
-8          508.50          624.56          200.00          90
-8          509.51          1000.00          364.35          100
-8          510.37          1500.00          662.99          110
-8          511.07          2100.00          963.82          120
-8          511.44          2500.00          1134.18          130
-9 ENDTBL          140
-3 STRUCT 01          150
-8          667.1          0.0          0.0          160
-8          667.2          2.2          0.2          170
-8          667.6          10.8          1.2          180
-8          668.1          21.6          2.4          190
-8          668.6          41.4          3.7          200
-8          669.1          61.2          5.4          210
-8          669.6          86.8          7.6          220
-8          670.1          112.4          9.9          230
-8          670.6          142.8          12.3          240
-8          671.1          173.1          14.8          250
-8          671.25          199.1          15.6          260
-9 ENDTBL          270
-6 RUNOFF 1 001          1 0.157          87.          1.193          1          310
-6 RUNOFF 1 001          2 0.042          93.          0.547          1          320
-6 ADDHYD 4 001          1 2 3          1          330
-6 RUNOFF 1 01          1 0.179          90.          1.21          1          280
-6 RESVOR 2 01 1          2 667.1          1          290
-6 REACH 3 002 2          4 550.          1          300
-6 RUNOFF 1 002          5 0.013          90.          0.461          1          280
-6 ADDHYD 4 002          4 5 6          1          340
-6 RUNOFF 1 002          1 0.013          87.          0.431          1          280
-6 ADDHYD 4 002          6 1 4          1          340
-6 ADDHYD 4 002          4 3 7          1          340
- ENDTBL          350
-7 INCREM 6          0.1          360
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ID New Braunfels Hydrology Study  
ID DA A for existing conditions and the 100-year frequency storm  
ID The USGS New Braunfels precipitation data is being used for Alligator Ck  
ID Note that TR20 side slope data not available from DS of Reach 3, so assumed z=0.053  
ID Alligator Creek, 1999  
\*  
ID FNI JOB:NEB99163 DATE: 12/1/99  
ID BY: MHP FILE: 100H1E.PRN  
IT 20 01JUL99 0 216  
IO 4 0  
\*  
KK E A1  
KM Calculation under existing conditions,100-year frequency storm, and Comal Ct. rainfall data  
BA 7.427  
PH 1 1.04 2.19 4.11 5.12 5.73 6.82 8.01 9.35  
PH 10.87 12.56  
UD 0.91  
LS 70  
\*  
KK Route 1  
KM Route A1 Hydrograph  
RD 9200 0.00126 0.03 TRAP 10 8.33  
\*  
KKE A10  
KM Local Runoff from A10  
BA 1.29  
UD 0.97  
LS 74  
\*  
KK CP10  
KM Combine Local A10 and Routed A1 Hydrographs  
HC 2  
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**APPENDIX E  
COMPUTATIONS SHEETS**

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1. Standard Step Backwater Calculations

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### STANDARD STEP BACKWATER CALCULATIONS

Sta	Inv (ft,msl)	Q (cfs)	Depth (ft)	HGL (ft)	Flow Area (ft <sup>2</sup> )	Flow Vel (ft/s)	Total Head Elev (ft)	Wetted Perimeter (ft)	Hyd Rad (ft)	R <sup>4/3</sup>	Energy Slope	Ave Energy Slope	Length (ft)	Head loss (ft)	Total Head Elev (ft)

$$S_e = n^2 V^2 / 2.22R^{4/3} = S_f$$

- Column 1            Cross-section identified by number.
- Column 2            Invert flowline elevation of channel or stream, (ft,msl)
- Column 3            Design Discharge, (cfs)
- Column 4            Assumed depth of flow, Y (ft)
- Column 5            Water surface elevation, Col2 + Col 4, (ft, msl)
- Column 6            Area of flow at depth Y, (ft<sup>2</sup>)
- Column 7            Mean Velocity, Col 3 / Col 6, (fps)
- Column 8            Velocity Head, (Col 6)<sup>2</sup>/2g, (ft)
- Column 9            Elevation of total head, Col 5 + Col 8, (ft)
- Column 10            Wetted Perimeter of flow at depth Y, (ft)
- Column 11            Hydraulic radius, Col 6/Col 10, (ft)
- Column 12            Hydraulic radius to the 4/3 power
- Column 13            Energy slope at section, (ft/ft), equation above
- Column 14            Average energy slope, approximately equal to half of Col 13 plus the energy slope of the previous section.
- Column 15            Length between cross sections, L, (ft)
- Column 16            Friction loss between sections, Col 14 times Col 15
- Column 17            Elevation of total energy head in feet. This step equates the total energy head (H) from Col 9 to the total energy head found by adding Col 16 with Col 17 of the previous section. If the elevation obtained does not closely agree with the elevation in Col 9, then a new flow depth (col 4) must be assumed and the procedure repeated until agreement is obtained. Then proceed to next section.

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# New Braunfels LID Design Guidelines

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## 1. Non-Structural LID Practices

Non-structural practices are often the first step in implementing Low Impact Development. These involve many planning and site design practices such as keeping existing trees onsite, minimizing compaction of soil that inhibits water infiltration, and planting trees and other vegetation in areas where none exists. Once developing on a given site, non-structural practices include simple tools such as disconnection of impervious cover, all of which are discussed in this section.

These non-structural practices can:

- Lower a project cost, by reducing elements such as street length and width;
- Increase project yield, by creating more space for development compared with conventional designs; and
- Often require no-cost strategies such as disconnecting a downspout.

One effect of these practices is to reduce the volume of runoff, thereby reducing the size of conveyance systems as well as flood control structures. Consequently, these practices should be implemented to the maximum extent possible consistent with local code. This section will present several non-structural processes that sites, whether new development or re-development, should examine first. These include items such as a conducting a site assessment or strategies for site layout (Section 1.1). It will then follow with guidelines to reduce the impact of the development, including reductions in impervious cover (Section 1.2) or two different disconnection strategies (Section 1.3).

### 1.1.Sustainable Site Design

Sustainable site design incorporates approaches to new and redevelopment projects which reduce impacts on watersheds by conserving natural areas, and better integrating stormwater treatment. The aim of sustainable site design is to reduce the environmental “footprint” of the site while retaining and enhancing the owner/developer’s purpose and vision for the site. Many of the sustainable site design concepts employ non-structural on-site treatment that can reduce the cost of infrastructure while maintaining or even increasing the value of the property relative to conventional designed developments.

The goals of sustainable site design include:

- Prevent stormwater impacts rather than having to mitigate for them;
- Manage stormwater (quantity and quality) as close to the source as possible and minimize the use of large or regional collection and conveyance;
- Preserve natural areas, healthy soils, native vegetation and reduce the impact on watershed hydrology;
- Use natural drainage pathways as a framework for site design;
- Reduce soil compaction during construction to maintain infiltration capacities of the soil;
- Minimize the amount of disturbance to existing, mature stands of vegetation;

- Utilize simple, non-structural methods for stormwater management that are lower cost and lower maintenance than structural controls;
- Create a multifunctional landscape which considers construction and maintenance implications; and
- Use appropriate plant species and communities for the eco-region and the designed media.

The first series of stormwater site design practices and techniques can be grouped into Preservation of Natural Features and Conservation Design. Discussion of nonstructural techniques on site and lot, such as reductions in impervious surface and disconnection, will follow in Sections 1.2 and 1.3. For more in-depth guidance on sustainable site design, please see the Sustainable Sites Initiative [www.sustainable-sites.org](http://www.sustainable-sites.org).

### **1.1.1. Preservation of Natural Features**

Preservation of natural features includes techniques to foster the identification and preservation of natural areas that can be used in the protection of water resources. Whether a large contiguous area is set aside as a preservation zone or certain smaller areas have been identified as appropriate for preservation, protecting established vegetation (existing trees, shrubs, grasses, and other flora) can help reduce revegetation requirements, reduce long-term erosion, preserve habitat, protect water and land resources, and maintain a healthy ecosystem.

Other benefits include:

- An immediate finished “aesthetic” that does not require time to establish;
- Increased stormwater infiltration due to the ability of mature vegetation to process higher quantities of stormwater runoff than newly seeded areas;
- Reduced runoff velocity, quantity, and erosion rates (by intercepting rainfall, promoting infiltration, and lowering the water table through transpiration among others);
- Provides a buffer against noise and visual disturbance during construction;
- Provides fully developed habitat for wildlife;
- Reduced construction costs; and
- Usually requires less maintenance (e.g., irrigation, fertilizer) and land clearing labor and costs than planting new vegetation.

In order to reach these benefits, it is important to first identify and preserve sensitive areas that affect hydrology. A site assessment is the process whereby the design team conducts an in-depth evaluation of the overall environmental conditions of the proposed development or redevelopment prior to detailed site design. Natural conservation areas are typically identified using mapping and field reconnaissance assessments. Areas proposed for protection shall be delineated early in the planning stage, long before any site design, clearing or construction begins.

The goal is to broadly identify and evaluate the ecological systems influencing the area to reduce cost and time impacts from a design, construction and maintenance prospective. Achieving cost reductions is a direct result of a solid understanding of environmental

characteristics and integrating the most appropriate construction. The initial design and planning phase is the most appropriate time to conduct the site inventory. For a project which will include LID in New Braunfels, items to examine during a site assessment shall include:

- Soil types and infiltration rates;
- Health and types of existing vegetation (trees, grasses, shrubs and forbs);
- Land use history and historical vegetation pre-settlement;
- Riparian areas and significant waterways;
- Prominent landforms;
- Site drainage patterns;
- Potential pollution sources;
- Floodplains; and aquifer zones; and
- Any and all karst features including caves or sinkholes.

Identifying these areas can help inform later development as sites shall be located to avoid sensitive resource areas such as floodplains, steep slopes, erodible soils, wetlands, mature forests and critical habitat areas. Buildings, roadways, and parking areas shall be located to fit the terrain and in areas that will create the least impact.

#### **Floodplains and Floodways**

Development in floodplains and floodways can reduce the ability of the floodplain and floodway to convey stormwater, potentially causing safety problems or significant damage to the site in question, as well as to both upstream and downstream properties. Ideally, the entire 100-year full build out floodplain and FEMA-approved floodway should be avoided for clearing or building activities, and should be preserved in a natural undisturbed state. Future development should stay out of all floodplain and the floodway, including areas where development has already occurred.

Once identified, preservation areas shall then be incorporated into site development plans and clearly marked on all construction and grading plans to ensure that construction activities are kept out of these areas and that native vegetation is kept in an undisturbed state. The boundaries of each conservation area shall be mapped by carefully determining the limit that shall not be crossed by construction activity.

#### **Slopes**

Development on slopes with a grade of 15% (7:1) or greater shall be avoided to limit soil loss, erosion, excessive stormwater runoff, and the degradation of surface water. Excessive grading shall be avoided on all slopes, as shall the flattening of hills and ridges. Steep slopes shall be kept in an undisturbed natural condition to help stabilize hillsides and soils. On slopes greater than 25% (4:1), no development, re-grading, or stripping of vegetation shall be allowed. Developer shall prepare a slope map as part of the site development plans showing >25% slopes and restricting that area from development.

## **Soils**

Areas of a site with hydrologic soil group A and B soils, such as sands and sandy loam soils should be conserved and these areas are ideal to be incorporated into undisturbed natural or open space areas. Conversely, buildings and other impervious surfaces should be located on those portions of the site with the least permeable soils. Similarly, areas on a site with highly erodible or unstable soils should be avoided for land disturbing activities and buildings to prevent erosion and sedimentation problems as well as potential future structural problems. These areas should be left in an undisturbed and vegetated condition.

## **Buffers**

A riparian buffer is a special type of natural conservation area along a stream, wetland or shoreline where development is restricted or prohibited. The primary function of buffers is to protect and physically separate these water bodies from future disturbance or encroachment. If properly designed, a buffer can provide stormwater management functions, can act as a right-of-way during floods, and can sustain the integrity of water resource ecosystems and habitats.

Riparian buffers should be continuous and not interrupted by impervious areas that would allow stormwater to concentrate and flow into the stream without first flowing through the buffer. Existing forested riparian buffers shall be maintained. Where no wooded buffer exists, reforestation shall be required. Proper restoration shall include all layers of the forest plant community, including trees, understory, shrubs and groundcover.

The buffer width needed to perform properly will depend on the size of the stream and the surrounding conditions, but a minimum 25-foot undisturbed vegetative buffer is needed for water bodies draining less than 40 acres but 5 or more acres. This first 25-foot section shall be a zero development zone and shall contain restrictions on the types of the uses and vegetation in this zone. Beyond the 25-foot section, an additional 50-foot or larger undisturbed buffer is ideal. Additional zones can be added to extend the total buffer to at least 100 feet from the edge of the stream. Some streams and watersheds may benefit from additional measures to ensure adequate protection. In some areas, specific state laws or local ordinances already require stricter buffers than are described here. The following required buffer sizes are determined by catchment area:

- Streams draining 640 acres (one square mile) or greater shall have a minimum buffer of 300 feet from the centerline on each side of the stream.
- Streams draining less than 640 acres but 320 or more acres shall have a minimum buffer of 200 feet from the centerline on each side of the stream.



- Streams draining less than 320 acres but 128 or more acres shall have a minimum buffer of 100 feet from the centerline on each side of the stream.
- Streams or swales draining less than 128 acres but 40 or more acres shall have a minimum buffer of 50 feet from the centerline on each side of the drainage.
- Streams or swales draining less than 40 acres but 5 or more acres shall have a minimum buffer of 25 feet from the centerline on each side of the drainage.

The development of LID features within the buffer zone shall be allowed within the 25-75 foot buffer zone as long as the development does not adversely impact the riparian area.

Development within the larger riparian buffer (beyond 50') shall be limited only to those structures and facilities that are absolutely necessary. Such limited development shall be specifically identified in any codes or ordinances enabling the buffers. When construction activities do occur within the riparian corridor, specific mitigation measures shall be required, such as deeper buffers or riparian buffer improvements.

Buffers shall remain in their natural state. However, some maintenance is periodically necessary, such as:

- Planting to minimize concentrated flow;
- Removal of invasive or exotic plant species when these species are detrimental to the vegetated buffer; and
- Removal of diseased or damaged trees.

### **Construction and Maintenance Considerations**

Once a site is under construction, minimal disturbance methods shall be used to limit the amount of clearing and grading that takes place on a development site, preserving the undisturbed vegetation and natural hydrology of a site. A limit of disturbance (LOD) shall be established based on the maximum disturbance zone. These maximum distances shall reflect reasonable construction techniques and equipment needs together with the physical situation of the development site such as slopes or soils. LOD distances may vary by type of development, size of lot or site, and by the specific development feature involved. A Limit of Development line shall be shown on the site development plans.

Not only shall these natural conservation areas be protected during construction, but they shall also be managed after occupancy by a responsible party able to maintain the areas in a natural state in perpetuity. Typically, conservation areas are protected by legally enforceable deed restrictions, conservation easements, and a maintenance agreement.

#### **1.1.2. Conservation Design**

For the purposes of this document, conventional design can be viewed as the style of suburban development that has evolved over the past 50 years and generally involves larger lot

development, clearing and grading of significant portions of a site, wider streets and larger cul-de-sacs, enclosed drainage systems for stormwater conveyance, and large “hole-in-the-ground” detention basins (see left image of Figure 1-1).

[insert image of conservation design from Phase III Report]

Conservation design, also known as open space design or cluster development, includes laying out the elements of a development project in such a way that the site design takes advantage of a site’s natural features, preserves the more sensitive areas, and identifies any site constraints and opportunities to prevent or reduce impacts. Techniques include:

- Preservation of undisturbed areas;
- Preservation of stream buffers;
- Reduction in clearing and grading;
- Locating projects in less sensitive areas; and
- Clustering development.

As mentioned in Section 1.1.1, these natural conservation areas are typically identified through a site assessment. Depending on the site, an assessment can be performed by professionals on the project development team (engineers, landscape architects or planners for example); however, to fully examine a site and its ecological conditions which will influence BMP design, more in-depth site analysis shall be done by hydrologists, ecologists, biologists or others professionals with site assessment experience in order to test infiltration rates, asses soil type and quality, and be able to properly identify existing vegetation. In many cases, a geotechnical report may also be required to assess depth to groundwater among other factors. When done before the concept plan phase, the planned conservation areas, and identification of other sensitive features also outlined above, can then be used to guide the layout of a project. For more guidance on conducting a site assessment, visit the Sustainable Sites Initiative™ guidelines.

Conservation subdivisions typically incorporate smaller lot sizes to reduce overall impervious over while providing more undisturbed open space and protection of water resources. This approach concentrates structures and impervious surfaces in a compact area in one portion of the development site in exchange for providing open space and natural areas elsewhere on the site. Typically smaller lots and/or nontraditional lot designs are used to cluster development and create more conservation areas on the site.

Conservation developments have many benefits compared with conventional commercial developments or residential subdivisions. They can reduce:

- Impervious cover;
- Stormwater pollution;
- Construction costs; and
- Reduce the need for grading and landscaping, while providing for the conservation of natural areas.

Along with reduced imperviousness, which carries multiple ancillary benefits as mentioned above, conservation designs provide a host of other environmental benefits lacking in most conventional designs. They can prevent encroachment on conservation and buffer areas. They create community-wide interconnected network of protected meadows, fields and woodlands. They can help to provide habitat, and protect farmland and other natural resources while allowing for the maximum number of residences under current community zoning. As less land is cleared during the construction process, alteration of the natural hydrology and the potential for soil erosion are also greatly diminished. Perhaps most importantly, open space design reserves 25 to 50% of the development site in conservation areas that would not otherwise be protected.

Conservation developments can also be significantly less expensive to build than conventional projects. Most of the cost savings are due to reduced infrastructure cost for roads and stormwater management controls and conveyances. Further, developers find that these properties often command higher prices than those in more conventional developments. Several studies estimate that residential properties in open space developments garner premiums that are higher than conventional subdivisions and moreover, sell or lease at increased rates.

Once established, common open space and natural conservation areas must be managed by a responsible party able to maintain the areas in a natural state in perpetuity. Typically, the conservation areas are protected by legally enforceable deed restrictions, conservation easements, and maintenance agreements.

Preservation of natural areas and conservation designs can help to preserve predevelopment hydrology of the site and aid in reducing stormwater runoff and pollutant load. Undisturbed vegetated areas also promote soil stabilization and provide for filtering and infiltration of runoff. Maintaining existing vegetation can be particularly beneficial to sites with floodplains, wetlands, stream banks, steep slopes, critical environmental features, or where erosion controls are difficult to establish, install, or maintain.

## **1.2.Reduction of Impervious Cover**

Once a development or redevelopment site has completed a site assessment to identify all the features mentioned above and the initial planning and design phase has begun, there are several additional non-structural LID tools to implement: reduce total impervious cover and disconnect.

Reduction of impervious cover includes methods to reduce the amount of rooftops, parking lots, roadways, sidewalks and other surfaces that do not allow rainfall to infiltrate into the soil, in order to reduce the volume of stormwater runoff, increase groundwater recharge, and reduce pollutant loadings that are generated from a site.

Most municipalities agree that an increase in impervious cover will increase runoff. However, the degree to which this is true is a function of several factors such as soil type, rainfall intensity, flow path and the amount of connected impervious cover among others. Thus, the effectiveness of disconnection practices – directing gutter downspouts into vegetated areas or disconnecting pavement – can be difficult to quantify. Therefore, many municipalities may not give any credit for these types of activities, even though there is obviously some benefit. The following section describes techniques to reduce overall impervious cover, and methods to disconnect existing or proposed impervious areas to maximize the benefit of LID. If at any time the City requires a maximum impervious coverage percentage by land use, then this BMP shall apply if the impervious coverage percentage is at least 5 percent below the maximum. OR Per City of Austin zoning code the maximum impervious cover for residential land use is 45 percent and commercial is 65 percent. If there is any further reduction in these percentages, then this BMP be effective.

### **1.2.1. Streets, Sidewalks, Driveways and Parking Lots**

#### **Streets**

The first step in achieving a reduction in impervious cover for streets is by examining street lengths and widths. The use of alternative road layouts that reduce the total linear length of roadways can significantly reduce overall imperviousness of a development site. Site designers are encouraged to analyze different site and roadway layouts to see if they can reduce overall street length. Streets shall be designed for the minimum required pavement width needed to support travel lanes, on-street parking, and emergency access. Several design options exist to reduce the total length and width of streets:

- One-way single-lane loop roads can reduce the width of lower traffic streets;
- On-street parking can be reduced to one lane or eliminated on local access roads with less than 200 average daily trips (ADT), and on short cul-de-sac streets;
- Reducing side yard setbacks and using narrower frontages can reduce total street length, which is especially important in Conservation Designs (Section 1.1.2).

Another large opportunity to reduce impervious cover on streets is with alternative turnaround areas, such as cul-de-sac design. Many of these cul-de-sacs can have a radius of more than 40 feet. From a stormwater perspective, cul-de-sacs create a huge bulb of impervious cover, increasing the amount of runoff. For this reason, reducing the size of cul-de-sacs through the use of alternative turnarounds or eliminating them altogether can reduce the amount of impervious cover created at a site. Alternative design options include:

- Reducing cul-de-sacs to a 30-foot radius;
- Allowing hammerheads as an alternative cul-de-sac form;
- Creating pervious islands in the center of the cul-de-sac;
- Including LID features in the center of the cul-de-sac such as bioretention areas to capture and treat runoff from the circular pavement; or
- Eliminating turnarounds altogether and building loop roads.

Sufficient turnaround area is a significant factor to consider in the design of these cul-de-sacs. For example, fire trucks, service vehicles and school buses are often cited as needing large turning radii. However, some fire trucks are designed for smaller turning radii. In addition, many newer large service vehicles are designed with a tri-axle (requiring a smaller turning radius) and many school buses usually do not enter individual cul-de-sacs.

Another option for designing cul-de-sacs involves the placement of a pervious island in the center. Vehicles only travel along the outside of the cul-de-sac when turning, leaving an unused "island" of pavement in the center. These islands can be attractively landscaped and also designed as bioretention areas to treat stormwater.

### **Sidewalks**

Most codes require that sidewalks be constructed of impervious concrete or asphalt. These codes are enforced to provide sidewalks as a safety measure. However, New Braunfels shall consider several alternative sidewalk designs.

- Allow sidewalks to be graded to drain to front yards, or vegetated areas between the sidewalk and the street, rather than the street.
- Allow alternative surfaces for sidewalks and walkways, such as pervious pavements, to reduce total impervious cover.
- Building and home setbacks shall be shortened to reduce the amount of impervious cover from entry walks.

Providing a landscaped area between sidewalks and the streets will also provide substantial opportunity for stormwater infiltration.

### **Driveways and Setbacks**

Typical residential driveways range from 12 feet wide for one car to 20 feet wide for two. There are several alternative driveway designs developers shall be allowed to implement which help reduce impervious cover and these include:

- Shared driveways: can reduce impervious cover and should be encouraged with enforceable maintenance agreements and easements;
- Narrower driveway widths and lengths: the typical 400-800 square feet of impervious cover per driveway can be minimized by using narrower driveway widths or reducing the length of driveways;
- Alternative design such as double-tracks; or
- Alternative surfaces such as reinforced grass, or permeable paving materials.

Building and home setbacks shall be shortened to reduce the amount of impervious cover from driveways and entry walks. A setback of 20 feet is more than sufficient to allow a car to park in a driveway without encroaching into the public right of way, and reduces driveway and walk pavement by more than 30% compared with a setback of 30 feet.

## **Parking**

Many parking lot designs result in far more spaces than actually required. This problem is exacerbated by a common practice of setting parking ratios to accommodate the highest hourly parking during the peak season. This is often the case with minimum parking standards which are often set to accommodate the highest hourly parking demand for the particular site and use. However, these minimum parking standards often result in more spaces than are required to meet demand as the language provides flexibility for the designer and developer to provide additional parking spaces beyond the minimum resulting in excess levels of parking. Setting parking standards as a maximum can ensure that sufficient parking is established to meet the demand without creating excess spaces.

There are many options available to reduce the overall parking footprint and site imperviousness. First steps include determining average parking demand and the lot location. A lower maximum number of parking spaces can be set to accommodate most of the demand. The number of parking spaces needed may be reduced by a site's accessibility to public transportation. Additional design strategies include:

- Setting maximums for parking spaces rather than minimums;
- Minimizing stall dimensions (by reducing both the length and width of the parking stall);
- Requiring a certain number of spaces be sized for compact vehicles;
- Using structured parking (which can reduce the conversion of land to impervious cover);
- Incorporate efficient parking lanes such as utilizing one-way drive aisles with angled parking rather than the traditional two-way aisles;
- Encouraging shared parking, particularly in mixed-use areas and for noncompeting parking lot users; and
- Using alternative porous surfaces.

Utilizing alternative surfaces such as porous pavers or porous concrete is an effective way to reduce the amount of runoff generated by parking lots. They can replace conventional asphalt or concrete in both new development and redevelopment projects.

### **1.3.Disconnection**

Disconnection of impervious surfaces and downspouts is encouraged to maximize the function of the LID practices. Disconnection is a low-cost, effective non-structural control which can reduce total runoff volume, increase the time of concentration and promote infiltration. The first step in disconnection is to identify the source of runoff and understand how it will be managed once disconnection occurs. In addition, well-conceived use of disconnection methods can reduce overall project costs by reducing or eliminating the need for more expensive structural practices. If at any time the City requires a disconnection of impervious surfaces and downspouts, then this BMP shall apply if the points of disconnection exceed the minimum by 10 percent.

### **1.3.1. Impervious Cover Disconnection**

Although the amount of impervious cover on a site can be minimized, it is unrealistic to think it can be eliminated completely. Despite this, impervious areas do not necessarily have to contribute to the runoff leaving the site. The amount of runoff and associated pollutants from a project can be reduced by disconnecting impervious surfaces. By disconnecting impervious areas and directing the flow to infiltration basins or designated buffer areas, a portion of additional runoff that would contribute to stormwater runoff is infiltrated close to the source instead. Further, the runoff that would potentially carry pollutants from the site to surface water instead gets treated and helps recharge groundwater.

Disconnection methods shall be incorporated at the planning and design level. However, the designer and reviewer should note that these methods must be used in concert with the design of other stormwater conveyance and treatment practices. The use of these disconnection methods does not relieve the designer or reviewer from following the standard engineering practices associated with safe conveyance of stormwater runoff and good drainage design.

### **1.3.2. Downspout Disconnection**

Rooftops with exterior drains for the gutter (the normal configuration for most residential structures) are one of the easiest disconnection practices to implement. These downspouts shall be directed to landscaped portions of the site rather than driveways or sidewalks unless the driveway is constructed of pervious paving materials as shown in Figure 1-2. It is not common, but driveways can be crowned so that a portion of the runoff is directed to vegetated areas, rather the street.

In addition to directing downspouts to vegetated areas, roof runoff may also be directed to cisterns and other rain barrels for later consumption, or even to depressed storage or other underground storage areas. Further, this runoff may be directed through a treatment train system as described below and demonstrated in Figure 1-3. Some design considerations include:

- Slowing down the water after it leaves the downspout if the volume and velocity is high (as shown with a splash pad configuration in Figure 1-4);
- Keeping the disconnected runoff away (10' minimum) from other impervious surfaces to reduce the chance for re-connection;
- Not placing the disconnected runoff into a steep slope area which could cause erosion and concentrate flows; and
- Directing the runoff into features specifically designed to receive (and either store, soak, treat, or convey) this runoff.

## **2. Structural LID Practices**

The following sections describe a variety of structural LID practices that can be used to convey, treat, and infiltrate stormwater runoff. This section will describe the following practices in detail:

- Rain Gardens & Bioretention
- Vegetated Swales
- Vegetated Filter Strips
- Porous Pavement
- Rainwater Harvesting

In order for LID practices to perform effectively basic guidelines need to be followed in their design, construction, and maintenance. This section is written to guide professionals through the design process. Detailed requirements are provided for the full suite of LID practices appropriate for the New Braunfels area. However, technical guidance will not be provided for rainwater harvesting as current local guidance is already available and resources for this material are provided in Section 2.7. Lastly, this section discusses several other structural controls in brief – green roofs, constructed wetlands, and proprietary systems – and provides links to resources for more information if projects in the New Braunfels region wish to implement these practices.

## 2.1. Descriptions and Selection Criteria

Though the LID toolbox is unlimited, this manual focuses on the above structural tools as they are most appropriate for the New Braunfels region. Further, many of these practices are most effective at reducing both runoff volume and pollutant loads. A quick summary of the selection criteria, described in detail throughout this section, is provided in Table 1.

[insert table like in p. 8 of San Marcos LID]

## 2.2. Rain Gardens and Bioretention

The rain garden and bioretention best management practices function as a soil and plant-based filtration device that removes pollutants through a variety of physical, biological, and chemical treatment processes. These facilities normally consist of a filtration bed, ponding area, organic or mulch layer, and plants. Figure 2-1 illustrates the basic components of the system.

Rain gardens and bioretention systems are very similar BMPs in their design and function. Both systems can be used in any land use type or for any site. For the purposes of this manual, the main difference between the two systems is that a bioretention system uses engineered soils. Engineered soil refers to the bioretention planting media which has been modified from its original condition or is a specialized, pre-determined mix of materials. While rain gardens do not include engineered soils, they can include slightly modified soils. Both systems can be designed with or without underdrains or liners. Circumstances where underdrains or liners would be required are described in Section 2.2.

[insert figure of rain garden section]



### 2.2.1. Description

A rain garden is a landscaped area in a basin shape designed to capture runoff and settle and filter out sediment and pollutants, primarily from rooftops, driveways, sidewalks, parking lots, and streets. Swales with check dams or berms that allow water to back up behind them function like rain gardens, and flow-through planters have also been described as a series, or treatment train, of rain gardens. What they all have in common is that they allow water to be retained in an area with plants and soil where the water is allowed to pass through the plant roots and the soil column.

In general, there are two kinds of rain gardens. Filtration rain gardens cleanse and detain stormwater runoff. Because they are specifically lined or have an underdrain to prevent infiltration in some areas (in areas where the influent is deemed too pollutant-heavy or for other site constraints) they do not significantly reduce stormwater volumes. However, these systems still provide substantial pollutant removal and increase the time of concentration. Infiltration rain gardens cleanse, detain and reduce runoff volumes by allowing water to seep into the surrounding soils.

Rain gardens are constructed with native soils (often with amendments, described in Section 2.2.4, Media Properties), rather than the engineered media used in bioretention systems. They are designed with shallow ponding depths that are adjusted to the infiltration capacity of the soil to ensure timely absorption of the water and, as mentioned above, do not always include underdrains in their design. Otherwise, the selection criteria and limitations are the same as bioretention systems.

Bioretention areas are similar to rain gardens except that they contain an engineered media mix. A picture of a bioretention system located in a parking lot island is presented in Figure 2-2.

In areas of New Braunfels with thin layers of topsoil over shallow limestone, bioretention could be a lined, more linear filtration system to remove pollutants before being released to streams. This alternative design would be a shallower system with a larger surface area.

Infiltration of the stored water in the bioretention system into the underlying soils occurs over a period of days when installed without an underdrain system. Figure 2-3 shows rain gardens or bioretention systems with no liner or underdrain, where filtered stormwater runoff infiltrates into the surrounding native soils.

### 2.2.2. Selection Criteria

#### Benefits

- Good choice of an on-site system serving a relatively small drainage area, since it can be incorporated into the site landscaping;

- Provides stormwater treatment that enhances the quality of downstream water bodies by infiltrating runoff, or when designed with liner or underdrain, temporarily storing runoff and releasing it over a period of days to the receiving water;
- The vegetation provides shade and wind breaks, and absorbs noise;
- Improves an area's landscape and has many aesthetic benefits;
- Easily integrated into site landscaping; and
- Their design can be formal or informal in character.

One advantage to rain gardens is that these are slightly simpler systems that can be easily implemented without the need for special training making them a good BMP for homeowners or property owners to implement on their own property.

### **Limitations**

- Can be difficult in areas with slopes greater than 20% (5:1). Bioretention systems need to be level for optimal filtration so in locations with slopes greater than 20% (5:1), they should be terraced. One option of an alternative design for these areas would be to construct a series of bioretention systems in a terraced design with check dams at regular intervals;
- Can be difficult where mature tree removal would be required since clogging may result, particularly if the facility receives runoff with high sediment loads;
- Unlined systems are not suitable at locations where the water table is within 6 feet of the ground surface and where the surrounding soil stratum is unstable; and
- Inclusion of substantial amounts of compost in the filter media can substantially increase nutrients in the discharge. Organic matter needs to be used in limited amounts and of high-quality, low-nutrient composition (see Section 2.2.4).

### **Cost Considerations**

The major costs associated with bioretention systems are the soil mixture and plants. The costs are greater than those for landscaping alone; however, the water quality benefits are substantial. The use of underdrains and liners will add to the cost versus bioretention systems designed for infiltration. An additional cost-benefit is received when bioretention or rain garden features double as traffic calming devices, such as a street bump-out.

### **2.2.3. Rain Garden and Design Specifications**

Rain gardens provide both sedimentation and filtration of stormwater, and infiltration when no underdrain is used. While sedimentation and filtration will occur throughout the entire surface area of the rain garden, the majority of sedimentation occurs in the immediate area of runoff entry. For design specifications of rain garden systems, reference Section 1.6.7.H of the Austin Environmental Criteria Manual.

#### **2.2.4. Bioretention Design Specifications**

Bioretention facilities are effectively sand filters that include additional organic and soil material in the filtration media to support vegetation. This allows these facilities to be integrated into the site landscaping where they can provide unobtrusive treatment of stormwater runoff.

For bioretention design specifications, plant selection considerations and recommended maintenance, refer to Section 3-63 of the TCEQ manual "Technical Guidance on Best Management Practices".

### **2.3. Vegetated Swales**

#### **2.3.1. Description**

Grassy swales are vegetated channels that convey stormwater and remove pollutants by sedimentation and infiltration through soil. They require shallow slopes and soils that drain well. Pollutant removal capability is related to channel dimensions, longitudinal slope, and amount of vegetation. Optimum design of these components will increase contact time of runoff through the swale and improve pollutant removal rates.

Grassy swales are primarily stormwater conveyance systems. They can provide sufficient control under light to moderate runoff conditions, but their ability to control large storms is limited. Therefore, they are most applicable in low to moderate sloped areas or along highway medians as an alternative to ditches and curb and gutter drainage. Grassy swales can be used as a pretreatment measure for other downstream facilities, such as bioretention areas. Enhanced grassy swales utilize engineered soils and an underdrain to provide filtration of pollutants. A picture of a grassy swale is presented in Figure 2-4.

Swales can be more aesthetically pleasing than concrete or rock-lined drainage systems and are generally less expensive to construct and maintain. Swales can slightly reduce impervious area and reduce the pollutant accumulation and delivery associated with curbs and gutters. The disadvantages of this technique include the possibility of erosion and channelization over time, and the need for more ROW than a storm drain system.

#### **2.3.2. Selection Criteria**

When choosing to implement swales, it is important to note that swales are limited to treating only a few acres and there must be sufficient land area available in order for these features to function properly.

#### **Benefits**

- Pretreatment for other LID practices; and
- Availability of water during dry periods to maintain vegetation.

The suitability of a swale at a site will depend on land use, size of the area serviced, soil type, slope, and dimensions and slope of the swale system. In general, swales can be used to convey runoff from areas of less than 2 acres, with slopes no greater than 5% (20:1). Research in the Central Texas area indicates that vegetated controls are effective at removing pollutants even when dormant. Therefore, irrigation is not required to maintain growth during dry periods, but may be necessary only to prevent the vegetation from dying. An example of a parking lot swale is shown in Figure 2-5.

### **Limitations**

- Can be difficult to avoid channelization;
- Cannot be placed on steep slopes; and
- Area required may make infeasible on intensely developed areas.

The topography of the site shall permit the design of a channel with appropriate slope and cross-sectional area. Site topography may also dictate the need for additional structural controls since the maximum recommended longitudinal slope is about 2.5% (40:1). Flatter slopes can be used, if sufficient to provide adequate conveyance. Steep slopes increase flow velocity, decrease detention time, and may require energy dissipation and grade check. Steep slopes also can be managed using a series of check dams to terrace the swale and reduce the slope to within acceptable limits. The use of check dams with swales also promotes infiltration.

### **Cost Considerations**

A swale is a lower-cost, alternate form of conveyance that provides some treatment and so has both a cost benefit and a water quality benefit when compared with curbs and gutter or pipe drainage systems. Enhanced swale systems will cost more than a grassy swale due to the addition of certain components for enhanced treatment capacity. Both types of swales are described in more detail in Section 2.3.3.

#### **2.3.3. Design Specifications**

A grassy swale is a sloped, vegetated channel or ditch that provides both conveyance and water quality treatment of stormwater runoff. Pollutant removal occurs through the processes of particle settling, adsorption, and biological uptake that occur when runoff flows over and through vegetated areas. There are two options available for swale design, the basic swale and the enhanced swale. For vegetated swale design specifications, plant selection and maintenance, refer to Section 3-51 of the TCEQ manual "Technical Guidance on Best Management Practices".

## 2.4. Vegetated Filter Strips (VFS)

### 2.4.1. Description

Vegetated Filter Strips (VFS), also known as filter strips or vegetated buffer strips, are a moderate to low-cost method for improving the quality of storm water runoff by using biological and chemical processes in soil and vegetation to filter out pollutants from runoff flowing through it as sheet flow. VFS are similar to grassy swales except that they are essentially flat with low, even slopes, and are designed to accept runoff as overland sheet flow only. Formerly common agricultural practices, VFS have now become common practice for treating runoff from roads, highways, and other pervious surfaces.

A photograph of a vegetated buffer strip is shown in Figure 2-6. The dense vegetative cover facilitates conventional pollutant removal through sedimentation and infiltration.

Filter strips cannot treat high velocity flows, and do not provide enough storage or infiltration to effectively reduce peak discharges to predevelopment levels for design storms. This lack of quantity control restricts their use to relatively small tributary areas. While VFS are applicable in many different areas, there are two primary applications for vegetative filter strips. Roadways and small parking lots are ideal locations where runoff that would otherwise discharge directly to a receiving water body, passes through the filter strip before entering a conveyance system. Properly designed roadway medians and shoulders make effective vegetated filter strips. The second application is land maintained in the natural condition adjacent to perimeter lots in subdivisions that will not drain via gravity to other stormwater treatment systems. The catchment area must have sheet flow to the filter strips without the use of a level spreader. VFS often require a large amount of space relative to other BMPs so they can be restricted in some areas beyond those two examples mentioned above.

Successful performance of filter strips relies heavily on maintaining shallow dispersed flow. If runoff is flowing over the VFS too fast, or in a concentrated manner, it will likely lead to rill erosion or scouring. To avoid flow channelization and maintain performance, a filter strip shall:

- Contain dense vegetation with a mix of erosion resistant, soil binding species;
- Engineered vegetated filter strips shall be graded to a uniform, even and a slope of less than 20% (5:1);
- Natural vegetated filter strip slopes shall not exceed 10% (10:1) on average, providing that there are no flow concentrating areas on the strip; and
- Laterally traverse the contributing runoff area.

Filter strips can be used upgradient from watercourses, wetlands, or other water bodies, along toes and tops of slopes, and at outlets of other stormwater management structures. The most important criteria for selection and use of this BMP are space and slope.

### 2.4.2. Selection Criteria

When implementing VFS, it is important to make sure that sufficient space is available and that the slope of the VFS is less than 20% (5:1).

#### Benefits

- Soils and moisture are adequate to grow relatively dense vegetative stands; and
- Comparable performance to more expensive structural controls.

#### Limitations

- Can be difficult to maintain sheet flow (there is a tendency to form rills or gullies);
- Cannot be placed on steep slopes;
- Area required may make infeasible on some sites; and
- Poor soils which cannot sustain a grass cover crop.

#### Cost Considerations

Filter strips are one of the least expensive stormwater treatment options and cost less to construct than curb and gutter drainage systems. This is one reason why they are often used in conjunction with other stormwater management practices in a treatment train approach.

### 2.4.3. Design Specifications

Filter strips may be natural or engineered. The use of natural filter strips is limited to perimeter lots and other areas that will not drain by gravity to other BMPs on the site. Engineered filter strips achieve an 85% TSS removal efficiency in the first 15% of the area, and no concentration reduction after that. For vegetated swale design specifications, plant selection and maintenance, refer to Section 3-51 of the TCEQ manual "Technical Guidance on Best Management Practices".

## 2.5. Porous Pavement

### 2.5.1. Description

Porous pavements are a special type of pavement that allows rain to pass through it. They can be used on both permeable and impermeable soils and in the latter case are designed with an underdrain system. Where soils are sufficiently permeable all the runoff will infiltrate and no discharge of stormwater or associated pollutants will occur. Systems designed with an underdrain provide substantial pollutant removal and increase the time of concentration, which are substantial benefits even when the volume of runoff is not substantially reduced.

There are several types of porous pavement, including porous asphalt, pervious concrete, pavers, and grid type systems. Porous asphalt pavement consists of an open-graded coarse

aggregate, bonded together by asphalt cement, with sufficient interconnected voids to make it highly permeable to water. Pervious concrete consists of specially formulated mixtures of Portland cement, uniform, open-graded coarse aggregate, and water. Pervious concrete has enough void space to allow rapid percolation of liquids through the pavement. Pavers themselves are typically impermeable; however, infiltration occurs either in the gaps between the pavers or within openings cast as part of the geometry of the paver. The use of pavers in a portion of a parking lot is presented in Figure 2-7.

The porous pavement surface is typically placed over a highly permeable layer of open-graded gravel and crushed stone. The void spaces in the aggregate layers act as a storage reservoir for runoff. A filter fabric is placed beneath the gravel and stone layers to screen out fine soil particles. Figure 2-8 illustrates a common porous paver installation and demonstrates the use of the filter fabric between gravel and stone layers.

Two common modifications made in designing porous pavement systems are:

- 1) Varying the amount of storage in the stone reservoir beneath the pavement;
- 2) Adding perforated pipes near the top of the reservoir to discharge excess stormwater after the reservoir has been filled.

Some municipalities have also added stormwater reservoirs (in addition to stone reservoirs) beneath the pavement. These reservoirs shall be designed to accommodate runoff from a design storm and should provide for infiltration through the underlying subsoil if an underdrain is not provided.

### **2.5.2. Selection Criteria**

Porous pavement may substitute for conventional pavement on parking areas, areas with light traffic, sidewalks, and patios. Slopes shall be flat or very gentle. For systems installed without underdrains, soils shall have field-verified permeability rates of greater than 0.5 in/hour, and there shall be a 4-foot minimum clearance from the bottom of the system to bedrock or the water table.

### **Benefits**

The advantages of using porous pavement include:

- Substantial pollutant reduction, even in systems with underdrains with surface discharge;
- Increased time of concentration;
- Less need for curbing and storm sewers; and
- Potential for groundwater recharge.

### **Limitations**

The use of porous pavement is constrained, requiring deep permeable soils (in systems without underdrains), low traffic loads, and consideration of impacts to adjacent buildings. Some specific disadvantages of porous pavement include the following:

- Many pavement engineers and contractors lack expertise with this technology;
- Porous pavement has a tendency to become clogged if improperly installed or maintained;
- Porous pavement has a high rate of failure;
- There is some risk of contaminating groundwater, depending on soil conditions and aquifer susceptibility; and
- Fuel may leak from vehicles and toxic chemicals may leach from asphalt and/or binder surface. Porous pavement systems are not designed to treat these pollutants.

### **Cost Considerations**

Estimated costs for an average annual maintenance program of a porous pavement parking lot are approximately \$200 per acre per year. This cost assumes four inspections each year with appropriate jet hosing and vacuum sweeping treatments.

### **2.5.3. Design Specifications**

Porous pavement systems consist of a pervious surface on top of a stone base, often referred to as the stone reservoir, which stores runoff before it infiltrates into the underlying soil (Figure 2-9). The use of permeable pavement techniques will be dictated by local or regional regulations but are often allowed in pedestrian areas (sidewalks, patios, plazas) and in some cases, for certain parking areas such as in stalls or overflow areas. For porous pavement design specifications, refer to the Addendum of the TCEQ manual "Technical Guidance on Best Management Practices" ..

## **2.6. Rainwater Harvesting**

Rainwater harvesting - collecting rainwater from impervious surfaces and storing it for later use - is a technique used for millennia. In drought stricken Central Texas and other areas around the country with limited water resources and stormwater pollution concerns, the role that rainwater harvesting can play for water supply is being reassessed for both residential and commercial buildings. Thus, it is important to note that there are current changes being made to local rainwater harvesting laws,



and design criteria are often modified, so it is best to check the most current regulations and incentives before implementing this practice.

Rainwater can be stored in a variety of structures. These include small 55 gallon barrels, the most common sizes for residential applications, to large underground cisterns. A photo of a rainwater collection system with a large above-ground storage tank is provided in Figure 2-10. Further, cisterns and barrels can be constructed of many different materials including wood, metal, plastic, glass, or synthetic compounds.

While potable use is possible for harvested rainwater, necessary on-site treatment and perceived public health concerns will likely limit the quantity of rainwater used for potable demands. Irrigation and the non-potable uses of toilets, urinals and HVAC make-up are currently the most common end uses for harvested rainwater. These are all beneficial uses individually, and when combined, they constitute a significant portion of residential and commercial water demand.

Focusing harvested rainwater on irrigation and selected non-potable indoor uses can significantly lower demand while allowing a balance and public comfort level between municipal potable water and reused rainwater. For harvesting systems to be efficient stormwater retention systems, the collected rainwater needs to be used in a timely manner to ensure maximum storage capacity for subsequent rain events. Cistern systems generally supply uses with significant demands to ensure timely usage of the collected water.

Outreach and education is a critical component of rain barrel programs, because of the more episodic and less structured use of this collected water. Homeowners shall be informed of the steps needed to maximize the effectiveness of their rain barrels.

### **2.6.1. Selection Criteria**

#### **Benefits**

- Contributes to water conservation;
- Augments drinking water supplies;
- Reduces stormwater runoff and pollution;
- Reduces erosion in urban environments; and
- Provides water that needs little treatment for irrigation or non-potable indoor uses.

#### **Limitations**

- Limited standards or guidelines for rainwater harvesting, especially its use indoors (however, please see the Texas Water Development Board or Comal and Guadalupe Counties for guidance);
- Sufficient storage needs to be available to capture subsequent rain events, so the stored water needs to be used relatively rapidly;
- Storage takes up space on small lots; and

- It is difficult for regulators to ensure that these small, dispersed systems are being operated in a way to significantly reduce stormwater runoff.

### **Cost Considerations**

The average cost of water delivered by municipal distribution systems is very low, which generally puts rainwater harvesting at a disadvantage compared to potable water when only the economics of water supply are considered. However, when these systems are sufficiently large, they may reduce the size of downstream detention facilities. In areas without water distribution systems and poor groundwater quality, rainwater harvesting may provide the best option for providing high quality water for indoor use.

### **2.6.2. Design Guidance**

This manual does not provide design guidance for rainwater harvesting as there are a variety of design options available. Further design guidance can be found in the following resources:

- Texas Water Development Board:  
<http://www.twdb.state.tx.us/innovativewater/rainwater/>  
[http://www.twdb.state.tx.us/publications/reports/RainwaterHarvestingManual\\_3rd edition.pdf](http://www.twdb.state.tx.us/publications/reports/RainwaterHarvestingManual_3rd%20edition.pdf)
- Brad Lancaster. Rainwater Harvesting for Drylands and Beyond.
- Texas A&M University Rainwater Harvesting Calculator:  
<http://rainwaterharvesting.tamu.edu/2011/05/31/calculator/>

### **2.7. Treatment Trains**

A treatment train consists of a series of stormwater practices installed in series. There are a number of reasons why this type of configuration is preferred. First, implementing a number of practices provides the opportunity to include a variety of unit processes (sedimentation, filtration, biological uptake, etc.) to treat the runoff, which optimizes the pollutant removal. Secondly, the use of multiple systems provides a level of redundancy so that at least partial treatment is being achieved even if one system is not functioning properly.

Probably the biggest benefit is the reduction in maintenance costs that can be achieved by using a dry system, such as a swale, upstream of ponds or other permanently wet facilities. Removal of accumulated sediment, trash, and debris from a dry swale is far easier and less expensive than removal of the same material once it enters a pond.

The configuration for a treatment train can take many different forms. Common applications include the use of a vegetated swale to convey stormwater to or from other treatment systems, such as bioretention cells. Swales can provide some level of pretreatment when installed upstream of other facilities and can provide the opportunity for some ancillary infiltration even when that is not the primary goal of implementing this practice. Other applications include the treatment train system

described in Section 2.7 where disconnected downspouts are directed through a series of additional BMPs. If there is excess runoff at the end of a treatment train system, the treated stormwater could then be connected to the storm sewer or other area. Figure 2-11 provides an example of a treatment train installation. Similar to the guidance provided in Section 2.7, treatment train systems shall be designed with maintenance considerations in mind. This includes items such as reducing velocity and erosion, or potentially adding a litter trap that is shovel width, at the point of entry.

## **2.8.Additional LID BMPs**

Thus far, Section 2 has described five structural LID practices that can be used to convey, treat, and infiltrate stormwater runoff in New Braunfels. Though the LID toolbox is unlimited, this chapter focuses on the above structural tools as they are most appropriate for the New Braunfels region. This last section discusses several other structural controls in brief, including a description of each technique and some important design considerations and limitations to each practice. Other TCEQ-approved BMPs may be used on a case-by-case basis, upon approval of the City engineer.

### **2.8.1. Green Roofs**

Green roofs, also known as vegetated- or eco-roofs, are roofs with a vegetated surface and growing media substrate. Green roofs are typically grouped into two distinct categories: extensive and intensive. Extensive green roofs have a shallower soil media, typically 6" or less, and thus support mainly low-growing ground cover. Intensive green roofs have a deeper amount of substrate (6" or more) and can include a variety of uses and vegetation, including trees. These intensive green roofs also have the appearance of a ground level garden, and thus can require additional investments in plant maintenance. Whether intensive or extensive in design, all green roofs contain, in their simplest design, an insulation layer, a waterproof membrane, a root barrier, a layer of growing medium and vegetation.

#### **Benefits**

Research has shown that green roofs can, if adequately designed, exhibit many benefits, and that these benefits are even more substantial in urban areas such as noise reduction, heating / cooling benefits, improved water quality, habitat provision, and runoff volume reductions. With regard to stormwater management, green roofs can prevent or reduce runoff from the lot by capturing it on the rooftop via plants, growing media and other green roof structural features (Oberndorfer et al., 2007). Rainfall soaks into the green roof's media layer, detaining runoff until after peak rainfall, and plants help return this moisture to the atmosphere through evapotranspiration. Of course, depth of media, plant type and regional climatic factors including rainfall patterns all directly affect the amount of runoff delay and reduction. However, studies have consistently shown that there is potential benefit in terms of stormwater management when compared with a conventional roof.

#### **Limitations**

There are several limitations to green roof implementation and most of these limitations depend on the system's design, including each of the components. First, green roofs are expensive LID tools, because their implementation can require specific media mixes or structural modifications to support the added weight on retrofit projects, and liability concerns among other items. Second, although there has been demonstrated sequestration of potential water pollutants such as nitrates and heavy metals, some research has demonstrated that runoff from green roofs can include increased levels of organic carbon, nitrogen and phosphorus due to leaching from the substrate, particularly if the green roof substrate includes high-nutrient organic matter or fertilizers. Additional research is needed to investigate growing mediums which do not contribute pollutants to runoff. Part of this needed research is to examine regionally appropriate plants that might optimize the uptake of nutrients or contaminants or conversely, not require any fertilizer or high-nutrient compost. Third, in certain Texas regions, with its extended periods of intense heat and drought, it can be difficult to keep green roof vegetation alive without regular irrigation. It is important to choose regionally-appropriate plant species that can withstand drought and high air and soil temperatures found in this sub-tropical region. It is important to note that many appropriate green roof species may go dormant during the summer months and that aesthetic does not always match the desired goals of the project. Lastly, it is essential to specify the performance objectives of the roof upfront to optimize success and efficacy of the green roof system. Specifying performance goals helps to ensure that the manufacturer supplied system suits the design needs and is not simply an unspecified green roof for its own sake.

### **2.8.2. Proprietary Systems**

Currently, there are many proprietary systems on the market designed to meet stormwater management goals. Suppliers of these systems all have specific design and maintenance criteria available if this a desired option for a project site. Other TCEQ-approved proprietary systems may be used on a case-by-case basis, upon approval of the City engineer.

#### **Benefits**

There are many benefits to these and other systems currently on the market. First, many of these systems, like the other structural systems described above, can be custom designed for a specific project with regard to media mix and vegetation. Secondly, they can be a good choice for highly urban areas where space is limited or where retrofits to existing storm drains are desired. Lastly, they can be efficient to implement and often offer guarantees against performance and structural failure.

#### **Limitations**

While proprietary systems have certain advantages, they also have several limitations. First, under current regulations, several proprietary systems are not allowed in certain jurisdictions or over aquifer recharge or contributing zones. Thus, it is important to investigate any local or regional regulatory obstacles that may exist which prohibit or prescribe their application.

Second, these systems can be limited in their ability to address site performance goals and the regional ecological conditions to the fullest degree. Many of the proprietary systems are designed to reach certain performance targets, such volume reductions or solely filtration purposes, or a combination. If this approach is chosen, it is important to understand the various performance goals for each system to assess whether these match the performance goals of a project in New Braunfels.

### **2.8.3. Constructed Wetland and Wet Ponds**

A constructed wetland is a constructed basin that has a permanent pool of water throughout the year (or at least throughout the wet season). They differ from wet ponds primarily in that they are shallower and have greater vegetation coverage. Constructed wetlands are now used to remove point and nonpoint water pollutants from stormwater runoff as well as from domestic wastewater, agricultural wastewater, landfill leachates, and coal mine drainage among other industries. For some wastewaters, constructed wetlands are the sole treatment; for others, they are one component in a sequence of treatment processes. Constructed wetlands can be highly effective systems; however, to be effective, they must be carefully designed, constructed, operated, and maintained. A distinction shall be made between using a constructed wetland for stormwater management and diverting stormwater into a natural wetland. The latter practice is not recommended and in all circumstances, natural wetlands shall be protected from the adverse effects of development, including impacts from increased stormwater runoff. This is especially important because natural wetlands provide stormwater and flood control benefits on a regional scale.

Wet ponds, also called stormwater ponds, retention ponds, wet extended detention ponds, differ from constructed wetlands primarily in having a greater average depth. Wet ponds treat incoming stormwater runoff by settling and biological uptake. The primary removal mechanism is settling as stormwater runoff resides in this pool, but pollutant uptake, particularly of nutrients, also occurs to some degree through biological activity in the pond. Wet ponds are among the most widely used stormwater practices. While there are several different versions of the wet pond design, the most common modification is the extended detention wet pond, where storage is provided above the permanent pool in order to detain stormwater runoff and promote settling.

These two stormwater management tools may be appropriate options for certain projects or circumstances. Design criteria and specifications can be found in the City of Austin's Environmental Criteria Manual.

#### **Benefits**

- Effective pollutant removal (as mentioned in Section 2, wet ponds and constructed wetlands are two BMPs capable of achieving high TSS removal efficiencies);
- High aesthetic value;

- Habitat; and
- Recreational / amenity value if integrated into park setting (wet ponds).

### **Limitations**

One reason these two BMPs are not included in this manual is because these systems are end-of-pipe stormwater management treatment systems and LID practices focus on onsite, distributed, at the source controls. Additionally, some issues may arise with maintaining a certain level of water within constructed wetlands and wet ponds during drought periods which can be costly to maintain or render the BMPs ineffective if the level is reduced. Please note that alternative methods to keep ponds wet from sources other than potable water are required in the City of New Braunfels. Potable water cannot be used as make-up water..