VOLUME I – DESIGN CRITERIA
VOLUME II – CONSTRUCTION STANDARDS

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April 2010
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PART A

GENERAL STANDARDS
PART A – GENERAL STANDARDS

I – Submittals for SmartGrowth Management Plan Criteria

All Development applications shall be submitted to the Town’s Planning Services Department and shall comply with the SmartGrowth criteria adopted with the SmartGrowth Management Plan. Applications will be evaluated in order of receipt by the Engineering Department. Applications will be placed on the priority list for review only upon submittal of all the required information.

A. Water Demand

Residential developments shall submit the number of units and total acres for the proposed development. Non-residential developments shall submit historical water use records showing average day demand, maximum day usage and peak usage, building square footage and total acres for the proposed use. The projected maximum day demand and peak hour demand, in million gallons per day (MGD) shall be calculated in accordance with the Town’s most recently adopted Water System Master Plan.

B. Wastewater Discharge

Residential developments shall submit the number of units and total acres for the proposed development. Non-residential developments shall submit historical wastewater discharge records, building square footage and total acres for the proposed use. The projected wastewater discharge flows in million gallons per day shall be calculated in accordance with the Town’s most recently adopted Wastewater System Master Plan.

C. Transportation

Residential developments shall submit the number of units and total acres for the proposed development. Non-residential developments shall submit evidence of trip generation based on studies sealed by a registered engineer. The use of this evidence will be determined at the discretion of the Town Manager or his designee. Non-residential developments shall also submit building square footage and total acres for each proposed use. Zoning amendment applications not requiring a site plan shall provide point of access locations to existing roads. A determination regarding the necessity of a Traffic Impact Analysis will be made in accordance with Section 82-181, 82-182, and 82-183 of the Town’s Code of Ordinances.
II – CONSTRUCTION STANDARDS

The October 2009 Construction Standards and any revisions there to are declared to be a part of this document and are included by reference as Volume II of II. The Town Manager shall be authorized to amend the Construction Standards.
III – Construction Plan Submittals


A. Review of Construction Plans:

1. Submit five (5) initial submittal, two (2) thereafter or complete 24”x 36”, bound sets of prints of the plans to the Town Engineering Department. Plans that do not include all required sheets, will be returned without a review and marked “Incomplete” for requirements.

Construction Plans shall include:

- Cover Page
- Record Plat
- Site Plan (non-residential development)
- Quantity Summary Sheet
- Construction Phase sequencing
- Street Layout
- Street Plan/Layout
- Paving plan (non-residential development)
- Water/Sanitary Sewer Layout
- Water/Sanitary Sewer Plan/Profiles
- Pre-Development Drainage Area Map
- Post-Development Drainage Area map
- Erosion Control Grading Plan (w/trees overlay/retaining walls/building pad elevation
- Erosion Control Plan
- Storm Sewer Layout
- Storm Sewer Plan/Profile
- Detention Pond/Retention Pond Plans
• Tree Survey
• Drainage Calculations
• Traffic Control Plan
• Traffic Sign Location
• Traffic Sign Details
• Plan for utility crossings, conduits, including Franchise
• Paving Details
• Pavement Marking Detail
• Pavement Marking Layout
• Street Light Locations (Residential)
• Landscape Plan
• Screening & Retaining Wall Plans
• Water Line Details
• Sanitary Sewer Details
• Storm Sewer Details
• Listing of infrastructure quantities associated with the proposed projects. Quantities shall be provided by Street section. Infrastructure quantities shall be provided for:
  o Right-Of-Ways
  o Street Width and Thickness
  o Waterline Length and Size
  o Storm Water Length and Width

2. Project plans that are resubmitted for review must include the original markups and/or comments and any subsequently resubmitted plans. All submitted plans and markups are to be retained by the Infrastructure Department.

3. The firm submitting (or resubmitting) project plans will be notified upon completion of project plan review. Plan submittals and re-submittals will be reviewed in order of receipt unless authorized by the Town Manager or his designee.

4. Plans submitted for review include a preliminary engineer’s stamp. Plans submitted for final review shall bear the seal of an engineer licensed in the State of Texas.

5. Upon final approval the design engineer shall submit reproducible drawings for approval. Any subsequent changes thereafter require resubmittal and approval.

6. Where a construction project requires interruption of traffic on any existing roadway, a detailed traffic control plan in accordance with the latest addition of Texas Manual on Uniform Traffic Control Devices shall be included in the construction plans.
7. Each sheet of the plans shall include the following statement: **The Town of Flower Mound Construction Standards apply, whether indicated on these plans or not.**

8. All plans shall be tied to the Town’s Geodetic Control network.

9. Any request for exceptions to the criteria set forth in the Town’s Design Criteria and Construction Standards shall be submitted in writing and is subject to approval by the Town Manager or his designee.

**B. Submittals After Approval of Construction Plans:**

1. The design Engineer is to:
   
   a. Supply four (4) 24” x 36” or 22” x 34” bound blue line or black line sets, and three (3) 11” x 17” bound sets of prints to the Infrastructure Services Department for construction purposes 3-5 business days prior to the pre-construction meeting.

   b. Approved plans shall be active for 120 days from date of release on extension may be given if submitted in writing to the Town Manager or his designee. If the date expires, the developer/owner shall be subject to re-submittal and all associated fees.

   c. If the project includes sidewalks, the plans shall be submitted to the Texas Department of Licensing and Regulation in compliance with the Texas Architectural Barrier’s Act. Provide the Infrastructure Services Department with a copy of the application and a copy of the approval letter.

2. The General Contractor is to:

   a. Furnish the drawings as required by Texas State Laws HB 662 and HB 665 regarding the safety systems to be used during trench excavation (as stated in the Occupational Safety and Health Administration’s Standards). Compliance with any and all safety rules and regulations is the sole responsibility of the contractor. A responsible party contact shall be provided.
IV – Construction Requirements

A. Requirements and Conditions prior to commencing Construction:

1. Construction plans must be reviewed and released for construction by the Town of Flower Mound and all other agencies and/or municipalities affected by this project. For projects that consist of grading only, that are located in floodplain areas as defined by the Town Wide Hydrologic Study, a Floodplain Development permit shall be presented to the Floodplain Administrator that is in compliance with the Flood Damage Prevention Ordinance included by reference in this document as Part B, Section IV, Subsection D, item 2. The permit is referenced in the ordinance under Section 2, part d, Administration, item iii, Permit Procedures.

2. Approval of all permits.

3. Completion of a pre-construction conference conducted by the Town Manager or his designee or a designated representative of the Town’s Engineering Department.

   NOTE: A written request to schedule a pre-construction conference must be made to the Town’s Infrastructure Services Department a minimum of five (5) working days in advance of the requested meeting date.

4. Submittal of two (2) copies of concrete mix design (as determined by an approved independent laboratory or certified testing company) to the Town’s Infrastructure Services Department.

5. Notification to the assigned Engineering/Construction Inspector two (2) working days prior to beginning any construction.

6. Submittal to the Infrastructure Services Department of original unrecorded instrument or instruments for any additional Rights of Way or Easements dedicated to the Town by separate instrument (legal description and an exhibit showing the metes and bounds on a plan view detail) on a form acceptable to the Town along with a check made payable to the Town of Flower Mound for the applicable recording fee. Submittal of one (1) copy of the SWPPP for disturbed areas of one (1) acre or more – following the currently adopted requirements of TCEQ.

7. File Notice of Intent as required by the U.S. Environmental Protection Agency for Storm Water Pollution Control and provide a copy to the Town’s Infrastructure Services Department.
8. All applicable fees must be paid prior to the pre-construction meeting. Fees shall include design review and inspection fees, escrow fees, pro-rata fees, and any other applicable fees.

9. Capital Improvement Projects:

1) Performance Bond
A good and sufficient bond in an amount not less than 100% of the total amount of the contract, guaranteeing satisfactory completion of the project by a contractor in accordance with the terms of the written contract, including price and time. This guarantees that public funds are protected should the bonded contractor fail to perform the terms of the contract for any reason, including default or insolvency.

2) Payment Bond
A good and sufficient bond in the amount not less than 100% of the total value of the contract, guaranteeing the full and proper protection of all claimants, supplying labor and material in the prosecution of the work provided for in the contract and for the use of each claimant.

B. Requirements during Construction:

1. Working hours:

   a. Standard daylight hours shall be defined as the hours between 7:00 a.m. to 7:00 p.m. Monday through Friday and 9:00 a.m. to 5:00 p.m. Saturday. Any work, including starting or operating equipment or machinery, delivery of materials, outside of these hours must be approved in writing by the Town. No construction equipment or machinery shall be operated before or after standard daylight hours and within one thousand (1,000) feet of any occupied residential structure without prior written approval. Saturday work must be requested in writing Thursday by 12:00pm and subset to availability of inspectors. Work on Sundays or legal holidays shall not be done without the written consent of the Town Manager or his designee and shall be restricted to work done in connection with the care, maintenance or protection of equipment or already completed work or to correct conditions that are unsafe to the public.

   EXCEPTIONS:

   1) Concrete placement work shall be scheduled so that all pouring and finishing shall be completed during standard daylight hours, except as approved by Town Manager or his designee. When working under emergency conditions, or when work must be concluded under artificial lighting, lighting shall be erected and directed so that it shall not shine upon any residence or create a visual traffic hazard.
2) Certain traffic congestion areas will require that modified standard work hours be enforced where street blockage, traffic flow, channeling of traffic and/or flagmen are required. The contractor will be notified of these areas during the pre-construction conference.

b. All work requiring inspection that is performed outside of weekdays, 7:00 am to 4:00 pm will require payment of overtime fees. There will be a two-hour minimum charge on weekdays and a four-hour minimum charge on weekends. In the event that Public Works staff is required to respond, invoices will be prepared and shall be paid for staff time, equipment and associated materials. Requests for work on Saturday must be submitted by noon on Thursday.

c. Holiday work days: The following holidays are to be observed and construction is not to be undertaken unless prior approval is received from the Town Manager or his designee:

1) New Year’s Day
2) Martin Luther King Day
3) Good Friday
4) Memorial Day
5) Independence Day
6) Labor Day
7) Thanksgiving Day and the following Friday
8) Christmas Eve Day and Christmas Day

Requests may be submitted for work on Town scheduled holidays, however, approval for the work is subject to inspector availability and if approved shall require payment of overtime fees.

2. All inspections must be called in 24 hrs prior to inspection.

3. The contractor shall possess one (1) copy of each approved construction permit pertaining to the project, and one (1) set of approved construction drawings, and have one set available on the project site.

4. A job superintendent must be on site at all times during any site activities.

5. Each contractor or subcontractor must possess a stamped approved set of engineering plans pertaining to that contractor’s phase of work at the project site.

6. One (1) copy of the pre-construction conference form must be in possession on the project site for emergency contacts, and must include at least one (1) 24 hour project representative contact.

7. One (1) copy of the Town water meter deposit receipt must be on file when construction of the project requires metered water.

8. Sanitary facilities shall be provided on the project site.
9. The contractor shall comply with all Town ordinances and State laws.

10. No tracked equipment shall be used on the streets or sidewalks of Flower Mound. Vehicles with steel lugs and/or plates shall not be operated on the streets of the Town of Flower Mound. Where such machinery must be used for construction, prior approval must be granted and the contractor shall use timbers, tires, or mounded earth over the paving surface to protect the pavement. Where such machinery must be loaded or unloaded from proper carrier vehicles, timbers, tires or mounded earth shall be used to protect paving and curbs. The general contractor shall be responsible for any damage from operation of a tracked vehicle on the project with the damage being repaired to the satisfaction of the Town Manager or his designee before acceptance of the project.

11. No construction shall continue on property that is or becomes the subject of a Town initiated condemnation proceeding unless the construction is continued in conformance with approved engineering plans reflecting the post-condemnation condition of the property as set forth in the condemnation petition.

12. All contractors working within Town right-of-way must comply with the latest edition of the Texas Manual of Uniform Traffic Control Devices (TMUTCD) with a plan submitted to the Traffic Engineer.

C. Requirements and Conditions for Final Acceptance of the Project

1. All sidewalks, established vegetation (per TCEQ requirements, by sodding or seeding), screening walls, fences, and all required appurtenances must be constructed as required by the approved construction plans before scheduling any final inspections. In addition, replacement of any traffic (lane divider) buttons and/or paving striping and any repairs of pavement damaged during construction activities must be completed before scheduling any final inspections. Requests shall be submitted in writing five (5) days prior to the requested date, to the engineering department, for final inspection and re-inspection of any corrections of construction.

2. All fees and assessments due to the Town of Flower Mound must be paid in full prior to acceptance, including but not limited to the following:
   
   a. Water bacteriological testing fees
   b. Construction inspection fees (if value of work exceeds original estimate)
   c. Construction inspection overtime fees
   d. Metered Town water fees

3. a) Original two year maintenance bond(s) shall be furnished by the developer and/or contractor on the insuring company’s form. The bond shall be dated to begin from the date of the acceptance of the project by the Town. The maintenance bond shall come from an approved surety company holding a permit from the State of
Texas to act as surety and acceptable according to the latest list of companies holding certificates of approval from the State Board of Insurance under 7.19-1 of the Texas Insurance Code. Maintenance bonds for projects are to be ten percent (10%) of the total contract amount for the facilities to be accepted and maintained by the Town. These facilities include water, sanitary sewer, paving and drainage improvements within rights-of-way and easements dedicated to the Town. A two-year maintenance walk-through will be required.

b) Capital Improvement Projects:

Original two year maintenance bond(s) shall be furnished by the contractor on the insuring company’s form. The bond shall be dated to begin from the date of the acceptance of the project by the Town. The maintenance bond shall come from an approved surety company holding a permit from the State of Texas to act as surety and acceptable according to the latest list of companies holding certificates of approval from the State Board of Insurance under 7.19-1 of the Texas Insurance Code. Maintenance bonds for projects are to be one hundred percent (100%) of the total contract amount for the facilities to be accepted and maintained by the Town. These facilities include water, sanitary sewer, paving and drainage improvements within rights-of-way and easements dedicated to the Town. A two-year maintenance walk-through will be required.

4. “As-Built” Drawings:

The developer, engineer or other responsible party shall be responsible for providing “as-built” drawings to the Town of Flower Mound Engineer for the project. “As-built” drawings shall be 22”x 34” in size and shall consist of one (1) mylar set (three (3) mils in thickness or greater), one (1) blueline/blackline set, and (1) CD with PDF, .tif, and either .dwg or .dxf 2004 or later. All information on the submitted mylar prints shall be legible.

5. The As-Built drawings shall be provided for all projects and shall be completed by the Engineer responsible for the design of the project. Hand written revisions shall only be accepted if they are drafting quality and legible. As Built drawings shall contain the signature on each page of the Engineer who prepared the As-Built and/or Record Drawings.

6. Final acceptance of development will be completed within 10 business days after receipt of all required submittals.

D. Repairs and Adjustments necessitated during the effective Maintenance Bond Period

1. The Town will notify the owner, developer, or contractor when a repair is required within the two (2) year period.
2. Repairs or adjustments shall commence within ten (10) calendar days following notification from the Town Engineering Department. Emergency situations shall be responded to immediately.

3. If the contractor does not start repair within the designated time frame, the Town will facilitate the repairs at the contractor’s expense.

   a) Town Forces will make repairs in emergency situations if the Contractor cannot respond in an acceptable period of time. The Contractor/Bonding company will be charged for all costs associated with such repairs made by the Town Forces.

   b) If contractor does not make payment or make arrangement of payment within thirty (30) calendar days, the Town will notify the bonding company.

4. Repairs or adjustments will be inspected, and inspection overtime provisions will apply, if applicable.

5. Repairs or adjustments will meet the specifications in force during the initial construction period.

END OF SECTION
V – Construction Standards References

1. Materials

The specifications for Materials shall comply with the North Central Texas Standard Specifications for Public Works Construction, 2004 version, or any updates there to.

2. Construction Specifications

The specifications for construction shall comply with the North Central Texas Standard Specifications for Public Works Construction, 2004 version, or any updates there to.

The latest version of TxDOT specifications shall be used for the following:

a. Lime Stabilizations
   - Lime installations shall meet TxDOT item 260
   - Type I Lime shall be used
   - All applications of Lime shall be installed by slurry form; no dry applications allowed.
   - All Lime placed to final grade shall be sealed with an asphalt emulsions material at a rate of .20 gallon per sq that meets TxDOT Item 300 Asphalt Oils and Emulsions.

b. Cement Stabilizations
   - Cement installations shall meet TxDOT Item 275
   - Type I cement shall be used
   - All applications of cement shall be installed by slurry form; no dry applications allowed.
   - All cement placed to final grade shall be sealed with an asphalt emulsions material at a rate of .20 gallon per sq that meets TxDOT Item 300 Asphalt Oils and Emulsions.
V – Geodetic Control Network Monument Installation Requirements

1. **General**

   All developments of over five acres shall be required to install a geodetic control network station (monument) within the boundaries of the development. The location of the station (monument) shall be selected and approved by the Town. The entire cost of the survey and installation of the station (monument) shall be the responsibility of the developers.

2. **Survey Technical Specifications**

   The geodetic control station (monument) shall be established utilizing Global Positioning Real Time Kinematic methods. The published results of the survey shall be in accordance with the following specifications: horizontal positions will have a positional tolerance of 0.2 feet and a vertical position shall have a positional tolerance of 0.3 feet.

3. **Monument Installation Specifications**

   The monuments shall be constructed using Berntsen CD2 Concrete Markers (2" diameter) or equivalent. The 2" diameter domed aluminum cap shall have “THE TOWN OF FLOWER MOUND” stamped on its face in 1/8" size letters around the outside edge. The monuments typically shall be placed on drainage structures or in concrete paving utilizing Berntsen DRYLOK Fast Plug anchoring epoxy or equivalent.

4. **Geodetic Control Station Data Sheet Specifications**

   The Town shall be furnished with both an electronic file and an 8 ½” by 11” hard copy of the monument description form. The monument description form shall have the minimum data shown and follow the format of the example monument description contained in the Standard Details.

END OF SECTION
PART B

TECHNICAL STANDARDS
PART B – TECHNICAL STANDARDS

I. Street Improvements

The Town of Flower Mound Design Criteria and Construction Standards shall govern and shall constitute the Town of Flower Mound’s technical specifications. Any reference to the Standard Specifications for public works of the North Central Texas Council of Governments (NCTCOG) shall be to the latest version. They will be referred to as the Standard Specifications and will not be physically bound with the Town’s Design Criteria.

A. Street Classification

1. Definitions

   a. Major (Principal) Arterial

      Major Arterials carry traffic long distances and connect the Town to the regional expressway system and other major regional activity centers. These roads serve Flower Mound traffic and traffic traveling through Flower Mound. They are the most heavily traveled roadways and require some degree of access control. Access standards are more stringent than those applied to minor arterials. Roadway drainage is provided by curb and gutter and storm sewer system.

   b. Greenway Urban Arterial

      Greenway Urban Arterials are Major Arterials with a large (42’) landscaped median. These roads have four lanes but can be widened to six lanes without the need for additional right-of-way. 2-12” travel lanes are provided in each direction.

   c. Greenway Rural Arterial

      Greenway Rural Arterials are Major Arterials with a large (40’) grassy swale in the median. 2-11’ travel lanes are provided in each direction. The minimum ROW = 120.

   d. Urban Minor Arterial

      Urban Minor Arterials serve traffic originating in or destined for locations within Flower Mound and are not intended to serve traffic traveling through the Town. Roadway drainage is provided by curb and gutter and storm sewer.

   e. Urban Minor with Bike Lane
Urban Minor Arterials with Bike Lane serve traffic originating in or destined for locations within Flower Mound and are not intended to serve traffic traveling through the Town. Roadway drainage is provided with curb and gutter. The outside 5’ of pavement is to be designated as a 5’ marked bike lane for bicycle traffic.

f. **Urban Minor Arterial with On-Street Parking**

Street, arterial (minor with on-street parking), means an urban minor arterial with on-street parking. Minor arterial streets are designed to serve traffic originating in or destined for locations within town and are intended to be used in Mixed Use type environments. It will have 10’ lanes with a 16 foot median and 18 foot angled parking on the outside curb. Curbs would extend to limit parking near intersections to provide better operations and visibility at these locations and reduce pedestrian crossing distance to approximately 56 feet. These sections of roadway will typically have large sidewalks to accommodate pedestrians.

g. **Urban Minor Arterial Undivided**

Urban Minor Arterial Undivided roadways are generally located in neighborhoods where the ROW is limited and traffic volumes require more than two lanes. Urban Minor Arterial Undivided roadways are intended to serve as the bridge between major arterials and collector streets.

h. **Urban Collector**

Urban Collector roadways connect residential and commercial areas to the arterial system and collect and distribute traffic from these areas. They carry less traffic for shorter distances than arterials and carry traffic to and from areas rather than through them. Roadway drainage is provided by curb and gutter, and storm sewer.

i. **Rural Collector**

Rural Collector roadways connect residential and commercial areas to the arterial system and collect and distribute traffic from these areas. They carry less traffic for shorter distances than arterials and carry traffic to and from areas rather than through them. Roadway drainage is provided with swales, ditches and culverts.

j. **Local Residential Urban**

Local Residential Urban roadways distribute traffic to and from residential areas. They are short in length and non-continuous to discourage through traffic. Roadway drainage is provided by curb and gutter and storm sewer.
k. Local Residential Estate

Local Residential Estate roadways distribute traffic to and from residential areas. They are short in length, and non-continuous to discourage through traffic. In order to retain a more rural character swales are used to convey drainage.

l. Local Commercial

Local Commercial roadways distribute traffic to and from commercial areas. They are short in length and non-continuous to discourage through traffic. Roadway drainage is provided by curb and gutter and storm sewer.

m. Local Conservation or Rural Development Street

Local Cluster Development roadways distribute traffic to and from residential areas in cluster development. Roadway drainage is provided by swales in order to retain a more rural character.

2. Standard Design Criteria

Pavement width for major arterials and urban streets shall be measured from face of curb to face of curb. Pavement standards for rural streets, where no curb is required, shall be measured from edge of pavement to edge of pavement.

<table>
<thead>
<tr>
<th>Street Classification</th>
<th>Minimum ROW</th>
<th>No. of Lanes and Width</th>
<th>Pavement Width</th>
<th>Median Width</th>
<th>Parkway Width</th>
</tr>
</thead>
<tbody>
<tr>
<td>Major (Principal) Arterial</td>
<td>120'</td>
<td>6-12' lanes</td>
<td>(2) 36'</td>
<td>18'</td>
<td>13'</td>
</tr>
<tr>
<td>Greenway Urban Arterial</td>
<td>120'</td>
<td>4 - 12' lanes</td>
<td>(2) 24'</td>
<td>42'</td>
<td>13'</td>
</tr>
<tr>
<td>Greenway Rural Arterial</td>
<td>120'</td>
<td>4 – 11’ lanes</td>
<td>(2) 29'</td>
<td>40'</td>
<td>11'</td>
</tr>
<tr>
<td>Urban Minor Arterial Divided</td>
<td>90’</td>
<td>4 – 12’ lanes</td>
<td>(2) 24’</td>
<td>16’</td>
<td>11’</td>
</tr>
<tr>
<td>Urban Minor Arterial with Bike Lane</td>
<td>100’</td>
<td>4 – 12’ lanes</td>
<td>(2) 29’</td>
<td>16’</td>
<td>11’</td>
</tr>
<tr>
<td>Urban Minor Arterial with On-Street Parking</td>
<td>120’</td>
<td>2 – 10’ lanes with 2 – 18’ parking lanes</td>
<td>(2) 38’</td>
<td>16’</td>
<td>14’</td>
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<tr>
<td>Urban Minor Arterial Undivided</td>
<td>70’</td>
<td>4-12’</td>
<td>48</td>
<td>None</td>
<td>10’</td>
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<tr>
<td>Urban Collector</td>
<td>60’</td>
<td>2 – 12’</td>
<td>36’</td>
<td>None</td>
<td>12’</td>
</tr>
</tbody>
</table>
Street Classification | Minimum ROW | No. of Lanes and Width | Pavement Width | Median Width | Parkway Width
---|---|---|---|---|---
Rural Collector | 80' | 2 – 12' lanes | 34' | None | 23'
Local Residential Urban | 50' | 2 – 15' lanes | 30' | None | 10'
Local Residential Estate | *26’ | 2 – 13’ lanes | 26’ | None | None
Local Commercial | 60’ | 2 – 12’ lanes | 36’ | None | 12’
Local Conservation or Rural Development | *26’ | 2 - 12’ lanes | 24’ | None | None
Alley | Varies | Varies | Varies | None | None

*With a fire hydrant minimum is 26’ wide, with 2-13’ lanes.

Additional right-of-way will be required at most Arterial and Collector intersections and may be required at high-volume residential street intersections or driveways to provide left and right turn lanes to maintain traffic volume capacities through the intersections. Requirements for turn lanes shall be determined by the Town Manager or his designee. Also, additional utility easements may be required beyond the right-of-way.

B. Street Pattern
   1. Conformance with Thoroughfare Master Plan
      a. The arrangement, character, extent, pavement width, right-of-way width, grade and location of each street shall conform to the Thoroughfare Master Plan. Each street shall be considered in its relation to existing and planned streets, topographical conditions, drainage, public safety, convenience and its relationship to proposed land uses to be served by such streets.
      
      b. Whenever a tract to be subdivided abuts any part of any street so designated on the Thoroughfare Master Plan, or where a street designated on the Thoroughfare Master Plan crosses any part of the tract to be subdivided, such part of the proposed public street shall be platted. The right-of-way shall be dedicated, and the street shall be constructed by the subdivider, generally consistent with the location as indicated on the Thoroughfare Master Plan, and to a width consistent with the Thoroughfare Master Plan and the requirements of these Design Standards. Where the tract to be subdivided abuts an arterial street, the subdivider shall be responsible for the construction of the one-half of the pavement section.
2. General

a. **Street layout.**

Provisions shall be made for the extension of arterial streets in accordance with the Thoroughfare Master Plan. Collector streets shall be provided for the circulation of traffic through residential and commercial areas and the connection thereof to arterial streets. Adequate local streets shall be provided to accommodate access to homes and businesses. Local streets shall be laid out so that their use by through traffic will be discouraged and grid patterns avoided. Where the layout of streets is not shown in the Thoroughfare Plan, the arrangement of streets shall either:

i) Provide for the continuation or appropriate projection to existing principal streets in surrounding areas; or

b. Conform to a plan for a neighborhood or planned development approved or adopted by the Planning and Zoning Commission to meet a particular situation where topographical or other conditions make continuance or conformance to existing streets impracticable or where neighborhood design makes a varied plan appropriate.

c. **Street connections.**

The systems of streets designated for a development must:

i) Connect with streets already dedicated in an adjacent subdivision.

ii) Be a reasonable projection of streets from the nearest subdivided tract if no adjacent development is platted.

iii) Be continued to the boundaries of the tract subdivided so that other subdivisions may connect.

iv) Streets with 30 or more residential units require separate access.

Reserve strips of land controlling access to or egress from other property or to or from any street or alley, or having the effect of restricting or damaging the adjoining property for subdivision purposes, or that will not be taxable or accessible for special improvements shall not be permitted in any subdivision.

c. **Half streets** shall be prohibited except where essential to the reasonable development of the subdivision and where the Planning and Zoning Commission finds it will be practicable to require the dictation of the other half of a street when the adjoining property is subdivided. Wherever a half street is adjacent to a tract to be subdivided, the other half of the street shall be platted within such tract.
Half streets on bounding property lines shall be permitted for arterial streets and one-half of the street shall be required from each property.

Streets should be platted to allow two tiers of lots between streets when possible.

**Street names.** Street names shall be subject to the approval of the Planning and Zoning Commission and shall be in accordance with the Town's street naming plan. No street names shall be used that will duplicate or be confused with the names of existing streets.

3. **Block Lengths**
   
a. Block lengths shall not exceed one thousand two hundred feet (1,200').
   
b. In cases where physical barriers, property ownership or land use create conditions where it is appropriate to increase the allowable block length the Planning and Zoning Commission may grant special approval after considering street connections, traffic circulation and public safety.

4. **Cul-de-sacs**
   
a. Cul-de-sacs shall provide proper access to all lots and shall not exceed six hundred feet (600') in length, except for local residential estate streets which shall not exceed one thousand two hundred feet (1,200') in length. Streets that exceed seven hundred fifty feet (750') in length require an emergency access and special permission from the fire department.
   
b. A turnaround shall be provided at the closed end that has a minimum right-of-way radius of sixty feet (60') and a minimum paved surface radius of fifty feet (50') measured from face of curbs.
   
c. Spot elevations shall be provided around the radius with the paving plan.

5. **Partial Cul-de-sacs (Eyebrow)**
   
a. Partial cul-de-sacs are only permitted on local residential type streets.
   
b. They shall have a minimum depth of no less than 55 feet, measured from the curb line of the intersected street to back curb line of the partial cul-de-sac. When the depth exceeds 80-feet, the facility shall be constructed as a conventional cul-de-sac.
   
c. They shall be no less than 80 feet, and no more than 100 feet in width.
   
d. They shall not be located on the crest of a hill where the street turns abruptly.
e. They shall be permitted only when the property is zoned and used for residential.

f. Valley gutters are required where a local street intersects with a partial cul-de-sac and shall have a minimum slope of 1/4" per foot towards the valley gutter.

g. Spot elevations shall be provided around the radius with the paving plan.

6. Intersections

a. Skewed Intersections
   i) For maximum traffic safety and ease of traffic operations, all streets should intersect at 90-degree angles.
   ii) No street intersecting an Arterial street shall vary from a 90-degree angle of intersection by more than 5-degrees.
   iii) No street intersecting a Collector street shall vary from a 90-degree angle of intersection by more than 5-degrees.
   iv) Intersecting Local streets shall not vary from a 90-degree angle of intersection by more than 20-degrees.

b. Offset Intersections
   i) Offset intersections shall be avoided whenever possible. Streets shall be designed to align with existing streets in adjoining subdivisions.
   ii) Non-intersecting local streets shall have a centerline offset of at least one hundred fifty feet (150').
   iii) Non-intersecting collectors or arterials shall have a centerline offset of at least three hundred feet (300').

7. Driveway Locations

All access points shall be designed in accordance with the Town of Flower Mound's Access Management Policy and Criteria. Residential subdivisions shall be designed as such that no residential driveways will intersect any collector or arterial. Any driveways to be constructed where there is an existing gutter shall be constructed with a horizontal curb cut, unless approved otherwise by the Town Manager or his designee.

Grade breaks shall not exceed 8% for crest conditions and 10% for sag conditions at the intersection of the driveway with the roadway.

C. Geometric Design Urban Collector
1. Roadway Design Parameters

<table>
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<td>30</td>
<td>30</td>
<td>30</td>
<td>15</td>
<td></td>
</tr>
<tr>
<td>Minimum Stopping Sight</td>
<td>360</td>
<td>360</td>
<td>305</td>
<td>305</td>
<td>200</td>
<td>250</td>
<td>305</td>
<td>305</td>
<td>200</td>
<td>200</td>
<td>200</td>
<td>305</td>
<td>200</td>
<td>80</td>
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<tr>
<td>Minimum Centerline Radius</td>
<td>800</td>
<td>800</td>
<td>640</td>
<td>640</td>
<td>275</td>
<td>475</td>
<td>640</td>
<td>640</td>
<td>275</td>
<td>275</td>
<td>640</td>
<td>275</td>
<td>50</td>
<td></td>
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<tr>
<td>Minimum Grade</td>
<td>0.7%</td>
<td>0.7%</td>
<td>0.7%</td>
<td>0.7%</td>
<td>0.7%</td>
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<td>0.7%</td>
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<tr>
<td>Maximum Grade</td>
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<td>7.5%</td>
<td>7.5%</td>
<td>7.5%</td>
<td>7.5%</td>
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<td>7.5%</td>
<td>8%</td>
<td>8%</td>
<td>7.5%</td>
<td>8%</td>
<td>8%</td>
<td>8%</td>
</tr>
</tbody>
</table>

*0.5% is acceptable for residential streets for a distance less than 200’

2. Vertical curves are required where algebraic grade difference is one-percent (1%) or greater.

3. Intersections
   a. The gradients of intersecting roadways should be held as near horizontal as practicable for those pavements surfaces where vehicles must stop and wait as in left turn storage lanes.
   b. Grades in excess of three-percent (3%) at intersections shall be avoided. In addition, ADA regulations must be met.
   c. Swales or valleys shall not be used to convey drainage across streets intersecting arterials or collectors.
   d. The profile and cross section of a minor road shall be adjusted to fit the principal road it intersects. The minor road crowned section should be transitioned to match the slope of the principal road profile.

4. Turning Lanes
a. All turning lanes shall be 12 feet (12') in width.

b. All turning lanes shall be of sufficient length to provide for storage of queued turning vehicles, tapering of turning lane to full width and allow vehicles to decelerate from normal speeds to a stopped position in advance of the intersection.

c. Storage Length

i) Shall be long enough so that the entrance to the storage lane is not blocked by vehicles stopped in the through lanes waiting for a signal change.

ii) Shall be long enough to avoid the possibility of vehicles in the storage lane backing up into through lanes.

iii) Required storage lengths are based on the classification of the street into which turning vehicles will enter. Minimum required storage lengths are as follows:

<table>
<thead>
<tr>
<th>Cross Street</th>
<th>Minimum Storage Length</th>
</tr>
</thead>
<tbody>
<tr>
<td>Major Arterial</td>
<td>200'</td>
</tr>
<tr>
<td>Minor Arterial</td>
<td>150'</td>
</tr>
<tr>
<td>Collector</td>
<td>100'</td>
</tr>
<tr>
<td>Residential</td>
<td>60'</td>
</tr>
</tbody>
</table>

d. Deceleration Length

i) The required deceleration length is the length needed for a comfortable stop from a typical average running speed. Deceleration lengths that include taper length are given below:

<table>
<thead>
<tr>
<th>Average Running Speed (mph)</th>
<th>Deceleration Length Including Taper (feet)</th>
</tr>
</thead>
<tbody>
<tr>
<td>20</td>
<td>160</td>
</tr>
<tr>
<td>30</td>
<td>250</td>
</tr>
<tr>
<td>40</td>
<td>370</td>
</tr>
<tr>
<td>45</td>
<td>435</td>
</tr>
</tbody>
</table>

ii) When intersections occur as frequently as 4 per mile, deceleration length is not required, only storage and taper lengths shall be provided.

5. Median Openings
a. Existing Medians
Widening, relocating, cutting or other alterations proposed to change or modify an existing median opening must be approved by the Town Manager or his designee.

b. New Construction
   i) Nose of the median shall be at a 10 foot setback from the intersecting curb-line.
   ii) The width of a median opening shall be the width of the intersecting street plus 20 feet (10 feet back of each curb-line).
   iii) Median spacing shall be in conformance with the Access Management Policy and Criteria.

D. Signs and Pavement Marking

1. All Traffic Signs shall be required and installed in accordance with the Texas Manual On Uniform Traffic Control Devices For Streets And Highways; and in accordance with the latest edition of the Texas Manual on Uniform Traffic Control Devices (TMUTCD); and Town of Flower Mound Standards.

2. Center lines and lane lines are required for all collector and arterial roadways.
   a. Divided arterials
      i) The 4” yellow left edge marker will be for divided arterials, as described in the TMUTCD. The right edge marker will be 4” white.
      ii) Lane lines shall be delineated using a combination of reflective raised pavement markers and non-reflective raised markers as described in the TMUTCD.
   b. Undivided arterials and collector streets
      i) Centerlines shall be a combination of striping and reflective and non-reflective raised pavement markings.
      ii) Lane lines shall be delineated using a combination of reflective and non-reflective raised markers as described in the TMUTCD.
      ii) The right lane marker will be white where there is a shoulder indicate, as described in the TMUTCD.
c. Intersections for arterial and collector streets shall include both striping and reflective and non-reflective raised pavement markings.

3. Traffic controls for school areas shall follow the TMUTCD. Pavement marking in the ensuring areas are specified as follows:

   a. Crosswalks

   Crosswalks shall be designed in accordance with the latest version of the TMUTCD.

   Stop Bars

   Stop Bars shall be designed as described in the TMUTCD with a minimum 24-inch wide bar, located 4-ft from the edge of the crosswalk stripes.

4. Materials

   a. Pavement Markings

      i) For all permanent striping, a 120 mil Thermo-Plastic shall be used.

   b. Raised Pavement Markers/Reflective Pavement Markers/Traffic Buttons/Jiggle Bar Tiles

      i) All markers, buttons, and tiles shall be ceramic. The use of any other material shall be approved by the Town Manager or his designee.

      ii) Supplemental traffic buttons for yellow marking applications shall be 4” square double reflective yellow.

      iii) Supplemental traffic buttons for white marking applications shall be 4” square double reflective white/red.
c. Adhesive

i) In areas of permanent construction, an epoxy adhesive shall be used for adhering markers, buttons, and tiles to paved surfaces. A two part resin based epoxy and catalysts mixed at a ratio of 1:1 creating a grey epoxy shall be used to install traffic buttons.

ii) In areas of temporary construction, a bituminous adhesive shall be used allowing for easier removal of the markers when permanent construction takes place.

d. Reference Town of Flower Mound sign and marking specifications for all items not listed.

E. Sidewalks

1. Sidewalks shall be a minimum of 4’ to the face of curb and 1’ from the right-of-way line. Exceptions shall be approved by the Town Manager or his designee.

2. If the sidewalk crosses private property, a pedestrian access easement is required to be dedicated to the public.

3. The minimum radius for curvilinear sidewalks is 200 feet.

4. All Sidewalks and Handicap Curb Ramps shall comply with all state and federal access regulations. If required, plans shall be submitted to the Texas Department of Licensing and Regulation in compliance with the Texas Architectural Barrier’s Act. Copies of the application and the approval letter shall be provided to the Engineering Department. TDLR letter shall be required prior to project acceptance.

5. Ramps shall be Armor Tile, Armor Cast, or approved equal. Color shall be red or approved equal unless shown on Town Hall system.

6. Sidewalk widths shall conform to the thoroughfare plan unless show on Town Trail System.

F. Trails

a. Multi-purpose trails shall be a minimum of 8’ in width.
b. Auxiliary trails shall be a minimum of 6’ in width.
c. Access walkways shall be a minimum of 6’ in width.
d. Equestrian trails shall be a minimum of 15’ in width.
G. Pavement Design

A geotechnical report sealed by a Professional Engineer licensed in the State of Texas shall provide recommendations for the total pavement design including subgrade treatment thickness, lime or cement content, base type and thickness, and surface type and thickness. The geotechnical investigation shall be submitted with the construction plans. The minimum pavement design should be for a 20 year life loading.

1. For Residential Streets serving primarily residential areas, the minimum pavement and subgrade requirement is 6 inches of reinforced concrete pavement on 8 inches of lime or cement stabilized subgrade.

2. For Collector streets, the minimum pavement and subgrade requirement is 8 inches of reinforced concrete pavement on 8 inches of lime or cement stabilized subgrade.

3. For Arterial Streets, the minimum pavement and subgrade requirement is 8 inches of reinforced concrete pavement on 10 inches of lime or cement stabilized subgrade.

4. For Local Conservation or Rural Development Streets, the minimum pavement and subgrade requirement is 2 inches of Type D asphalt on 5 inches of Type B asphalt on 8 inches of lime or cement stabilized subgrade, if using the asphalt pavement option.

5. For fire lanes, the minimum pavement and subgrade requirements is 6" of reinforced concrete pavement on 6" of lime or cement stabilized subgrade as recommended by a geotechnical engineer, or 8 " of reinforced concrete pavement on 6" of untreated subgrade compacted to 95 percent density. Fire lanes must support a vehicle load of 85,000 pounds.

6. Pavement designs should be included for all streets & fire lanes. All pavement subgrade shall be treated so that the treated soil liquid limit does not exceed 35 and the plasticity index does not exceed 15.

7. Driveway approaches shall be of the same thickness as the road it abuts to.

8. All concrete pavement that is to be installed abutting existing concrete pavement, shall require headers.

H. Pavement Construction

1. All utility surface features within the pavement area shall be poured monolithic with in paving operations. Block-outs except for inlets, shall not be allowed unless approved in writing by the Town Manager or his designee.

2. Subgrade Preparation
All franchise utilities shall be installed prior to stabilization. Once rough grading is complete, the Geotechnical Engineer responsible for preparing the design report shall be required to reexamine the soil types to confirm the original report. He shall be required to submit findings in writing to the Town Manager or his designee. Should soil types differ, a revision to the report shall be required considering all data that affects design.

Regardless of the pavement type to be constructed, all subbase materials shall be modified by either lime stabilization or cement stabilization, as specified herein. Exceptions or modifications to subbase treatment must be approved in writing by the Town Manager or his designee.

All materials placed shall be placed in slurry form.

All completed subbase shall be proof rolled as specified in TxDOT Item 215, and shall be considered incidental to the project.

a. Lime Treatment for Subbase shall be specified in TxDOT Item 260.

The geotechnical report shall state the percentage of lime required to modify subbase materials such that the plasticity index is not greater than 15.

b. Portland Cement Treatment for Subbase

The geotechnical report shall state the percentage of cement content required to produce a minimum design compressive strength of 250 psi.

Cement treatment of subbase shall be as specified in TxDOT Item 275 or 276.
3. Concrete Pavement
   a. Materials
      i) Concrete
         (1) All concrete paving design mixes shall be 4200 psi for arterials and collectors streets and 3600 psi for all other street sections in accordance with the COG Standard Specifications. Fly ash may be used in concrete design mixes in accordance with the COG Standard Specifications. Class C fly ash as described in ASTM Designation C618 is the only acceptable fly ash classification.
         (2) All concrete for paving shall be air entrained with a total air content of 5% plus or minus 1%. Air entrained mixtures shall conform to A.S.T.M. Designation C-260.
         (3) All concrete for pavement which includes the use of water reducing admixtures shall conform to A.S.T.M. Designation C-494, Types A, D, F and G.
         (4) All concrete patch work shall be full panel replacement.
      ii) Reinforcing
         (1) Reinforcing bars shall be #3 bars and conform to A.S.T.M. Designation A-615, and placed at a max spacing of 18” on center each way. The bend-test shall be provided by the contractor prior to installation.
         (2) Reinforcing bars for headers, abutting existing concrete pavement shall tie-in with #4 bars.
         (3) Only new billet steel will be acceptable for field bending. Rust or oil contamination is cause for rejection.
   b. Installation
      i) Concrete
         (1) All concrete placed shall be in conformance with the Town’s Standard Specifications.
         (2) Concrete pavement surfaces shall be thoroughly finished and straight prior to applying the final finish.
         (3) The final finish shall be a rough broom finish parallel with the curb line.
      ii) Reinforcing
(1) All reinforcing shall be supported on bar chairs or supports designed for the specific purpose of reinforcement support.

(2) All bars, laps, and splices shall be secured with wire ties and 50% of mat steel and 100% at all ends. All reinforcing in concrete which is in contact with the ground shall have a minimum clearance of three (3) inches; and two (2) inches from any formed surface. All dimensions are clear dimensions.

(3) Concrete headers are required abutting existing concrete and #4 deformed rebar dowels.

3. Asphalt Pavement
   a. Materials
      i) Hot Mix Asphalitic Concrete (HMAC)
         (1) All HMAC material shall comply with TxDOT Item 340. The addition of modifier and additives will only be allowed with the specific approval of the Town Manager or his designee.

         (2) HMAC Base Course shall be of thickness shown on the drawings, and at least 4” thick, and be Type B.

         (3) HMAC Surface Course shall be of the thickness shown on the drawings, and at least 2” thick, and be Type D.

   b. Installation
      i) Installation shall comply with TxDOT Item 340. The application of Prime Coats and Tack Coats shall be as specified in Item 340. Where reference is made to other TxDOT Items, the provisions included in such referenced Items shall apply.

END OF SECTION
## Streets Construction Standards

*(Provided for Reference – Included in Volume II)*

<table>
<thead>
<tr>
<th>Title</th>
<th>Detail No.</th>
<th>Sheet No.</th>
<th>Revision Date</th>
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<td>1</td>
<td>August, 2002</td>
</tr>
<tr>
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<td>ST-1</td>
<td>2</td>
<td>August, 2002</td>
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<tr>
<td>Greenway Rural Arterial</td>
<td>ST-1</td>
<td>3</td>
<td>April, 2012</td>
</tr>
<tr>
<td>Urban Minor Arterial Divided</td>
<td>ST-1</td>
<td>4</td>
<td>February, 1999</td>
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II. Water System Improvements

A. General

This section pertains to general design requirements for water distribution system construction in the Town of Flower Mound. All water lines shall be sized and designed in accordance with the most recent Town of Flower Mound Water Distribution System Master Plan or as determined by the Town Manager or his designee. Other pertinent requirements for water connections, over sizing, main extension, backflow, cross connections, water restrictions, conservation, impact fee and other water related fees are included in Chapter 11, Utilities and Solid Waste, of the Town Code. In the absence of specific standards, all water supply, distribution, pumping, and storage improvements shall be designed in accordance with the most current standards of the American Water Works Association, the Standard Specifications for Public Works Construction of the North Central Texas Council of Governments, and criteria adopted by the Texas Natural Resource Conservation Commission, Chapter 290, "Water Hygiene".

B. Water Lines

1. Standard water line sizes are 8", 12", 16", 20", 24", and 30" diameter. Other sizes must be approved by the Town Manager or his designee.

2. All water lines shall be a minimum of 8" in diameter. Water lines 12" or greater shall require a profile with the plan. All water lines shall be looped; no dead end except for dead ends that will be connected to future development unless approved by the Town Manager or his designee. If temporary or permanent dead ends are allowed, a hydrant shall be installed at the dead end for flushing purposes.

3. 8" and 12" water lines shall be located in the parkway, with a minimum cover of 42" over the 8" line and 48" of cover over the 12" line. 16" and larger water lines shall be located in the parkway, with a minimum cover of 60". Lines that are installed at least 2' below the minimum depths shall have a profile in the design plans reflecting the actual depth. Water lines shall be located on the north and east side of the roadway. Along State Highways, water lines are required to be constructed on both sides of the roadway. All water lines shall be located 5½ feet behind the curb, or 2 feet from the edge of pavement for streets without curb and gutter, unless approved by the Town Manager or his designee. All water lines, whether main lines or service lines, crossing existing streets shall be placed by dry boring within an encasement. Open cut excavation will not be allowed to cross existing streets.

4. Water lines must be extended to neighboring lot(s) to allow future looping of water system on future lots. Town Manager or his designee has final authority on location of extensions and stub outs for future use.

5. Easements for water line construction shall meet the following requirements:
a. The easement width shall be a minimum of 15 feet.

b. Water easements must be dedicated solely as ‘water’ easements. Other utility easements may cross perpendicular to Town water easement. No overlapping easements are allowed. Exceptions must be approved by the Town Manager or his designee.

c. If the water line is less than 12 feet deep, the outside diameter of the water line shall be located a minimum distance of 6 feet from the edge of the easement, and if other utilities are located in the same easement, the outside diameter of the water line shall be located a minimum distance of 3 feet from the outside diameter of the other utilities.

d. If the water line is greater than 12 feet deep, the outside diameter of the water line shall be located a minimum distance of 9 feet from the edge of the easement, and if other utilities are located in the same easement, the outside diameter of the water line shall be located a minimum distance of 6 feet from the outside diameter of the other utilities.

e. Fire hydrants are required to be located within a utility easement or public Right-of-Way.

6. Water lines for multi-family, commercial and industrial fire protection lines shall be in an easement dedicated to the Town, unless the system is isolated from the Town system by detector check valves. All water lines shall be 8” minimum diameter. Dead end lines for hydrant leads shall not exceed 50 feet on multi-family, commercial, or industrial sites. All public water lines located on private property shall be centered in a 15-foot minimum easement. Larger easements may be required by the Town Manager or his designee to provide adequate space for maintenance. Water lines shall not be located under paved surfaces where possible, or under structures.

7. All piping with mechanical couplings, push-on, or similar joints subject to internal pressure shall be designed with blocking and anchors or restraining harnesses to preclude separation of joints. All bends and fittings shall be constructed with blocking and anchors.

8. GIS Markers
In accordance with Town Standards, GIS Marker shall be placed for each valve and hydrant located on or connecting to a main that is 16” or larger. The Town of Flower Mound will provide the identification number and the contractor and/or developer will be responsible for providing the markers and installation. The markers shall be placed immediately adjacent to the valve or hydrant. If the marker is to be set within a paved surface, the installation shall be monolithic with the paving installation.
9. The following data will be furnished by the manufacturer for all 16 inch diameter and larger water mains. The manufacturer shall prepare drawings and schedules showing full details of the pipe, all outlets, and all fittings. Any change in pipe class or type will be clearly indicated on the drawings. The drawings shall identify water line stationing. These drawings and schedules shall be submitted to the Town Engineer or his designee for comment and approval prior to the shipment of any pipe to the project site. In the event the manufacturer recommends any grade or alignment change from the plans, this information must be furnished on drawings and schedules that clearly identify the recommended change.

The manufacturer shall submit design calculations for approval to the Project Engineer prior to the shipment of any pipe to the project. As an alternative, the manufacturer may submit a certification by a Professional Engineer licensed in the State of Texas, that the design calculations used to manufacture the pipe meet the criteria shown on the plans.

10. Materials

a. Polyvinyl Chloride (PVC) Pipe
   i) PVC pipe shall be designed, manufactured, and tested in accordance with the applicable requirements of AWWA C-900 (6”-12” water pipe) AWWA C-905 (16” and larger water pipe), and AWWA M-23.
   ii) All PVC water pipe shall be blue in color.
   iii) 8” through 16” water pipe shall be pressure class 150, DR 18. Pressure class 200, DR 14 pipe may be required by the Town Manager or his designee in areas of high distribution system pressure. Areas bounded by F.M. 3040 on the north side and east of Simmons Road shall be DR 14.
   iv) PVC pipe in sizes 20” and larger will only be allowed when approved by the Town Manager or his designee. Water mains 20” and larger shall be RCCP, or Ductile Iron Pipe Class 150, unless other materials are approved by the Town Manager or his designee.
   v) Fittings shall be ductile iron in accordance with AWWA C110 or AWWA C153.
   vi) Fittings: ANSI/AWWA C111/A21.11, except gaskets shall be neoprene or other synthetic rubber. Natural rubber will not be accepted.
   vii) All buried metal shall be wrapped in polyethylene Tube Wrap: ANSI/AWWA C105/A21.5

b. Reinforced Concrete Cylinder Pipe (RCCP)
i) RCCP shall be designed, manufactured, and tested in accordance with the applicable requirements of AWWA C-303 and AWWA M-9, and the following minimum design parameters.

(a) Unit Weight of Fill (w) = 130 pcf
(b) Live Load = AASHTO HS 20
(c) Trench Depth = 5 feet minimum, or as indicated on construction plans
(d) Coefficient Ku' = 0.150
(e) Minimum Trench Width (Bd) = 36 inch minimum
(f) Bedding Conditions
(g) Minimum Soil Reaction Modulus (E') = 700 maximum
(h) Coefficient k = 0.090
(i) Minimum Pressure Class = 150 psi min. working pressure
(j) Minimum Surge Pressure = 150 psi plus working pressure

ii) Trench depth indicated shall be verified after existing utilities are located. Vertical alignment changes required because of existing utility or other conflicts shall be accommodated by an appropriate change in pipe design depth. In no case shall pipe be installed deeper than its design allows.

iii) Joint Wrappers: Joint wrappers shall be similar and equal to those manufactured by Mar-Mac Manufacturing Company.

iv) Flexible Joint Couplings: Flexible joint couplings shall be equal to Smith-Blair Style 441, or approved equal.

v) Pipe Ends: The standard pipe end shall include steel joint ring and a continuous solid rubber ring gasket as per AWWA Manual M-9.

vi) Flanges: Flanges shall conform with AWWA Standard C-207 with laying dimensions and drilling in accordance with ANSI B 16.1, Class 125, unless otherwise specified. Drilling shall match class of valves or appurtenances which are attached. Nuts and bolts shall conform to ASTM A-307, Grade B.

c. Ductile Iron Pipe

i) Ductile iron pipe will only be allowed in non-corrosive soils, (resistivity of <1000), when approved by the Town Manager or his designee.


iii) Fittings: ANSI/AWWA C111/A21.11, except gaskets shall be neoprene or other synthetic rubber. Natural rubber will not be acceptable.

iv) Fittings shall be ductile iron in accordance with AWWA C110.

vi) All bolts and nuts shall be ASTM A325 Type III Enhanced Corrosion Resistant steel, or stainless steel Grade 304 or 316.

vii) All buried metal shall be wrapped in polyethylene Tube Wrap: ANSI/AWWA C105/A21.5

11. Installation

a. General

i) All installations shall conform to the latest TCEQ Specifications. Separation distances from sanitary sewer and storm sewer shall be as specified by TCEQ.

ii) All 8” water pipe shall be installed with a minimum of 42 inches of cover over top of pipe, 12” water pipe shall be installed with a minimum of 48 inches cover, and pipe 16” and larger water pipe shall be installed with a minimum of 60 inches of cover over top of pipe.

iii) Reference Part B Section IV for trench testing.

iv) The amount of trench excavation shall not exceed 200 (two hundred) feet from the end of the pipe laying operations, and no more than 300 (three hundred) feet of total open trench will be allowed. At the end of each work day, all trench excavation shall be backfilled to the end of the pipe laying operation. Barricades, safety fencing, and lights will be required around any open trench left overnight.

v) A waterproof plug shall be installed in the exposed end of line at the end of each working day. If contractor fails to install the plug, the Town Manager or his designee may require that the entire water line be swabbed prior to hydrostatic testing.

vi) All connections to existing water mains shall be made under pressure unless dry connections will not cause any loss of service. Under special conditions connections that cause an interruption of service may be performed with approval of the Town Manager or his designee.

vii) Water jetting, jacking, or missile type bores shall not be permitted under any conditions.

b. PVC Water Pipe

i) PVC water pipe, and appurtenances shall be installed as specified in AWWA Manual M-23 and in accordance with the pipe manufacturer’s recommendations.
ii) When PVC pipe is used, blue tracer tape shall be installed in the backfill material no more than twelve-inches (12") above the top of the pipe in accordance with the manufacturer's recommendations and located as specified by the Town.

iii) Blue detector pad is required at all service connections to main and all main line valves.

c. Reinforced Concrete Cylinder Water Pipe (RCCP)

Install RCCP, fittings, specials, and appurtenances as specified in AWWA Manual M-9, and in accordance with the pipe manufacturer's recommendations.

d. Fittings

i) Fittings shall be installed in accordance with AWWA C-600.

ii) All mechanical joint bends, tees, and reducers which require blocking shall be additionally restrained with EBAA mega lug retainer gland or approved equal.

iii) All RCCP fittings shall be restrained by welding the joints in accordance with AWWA M-9. Concrete blocking may be used in lieu of welding if approved by the Town Manager or his designee.
C. Fire Hydrants

1. The centerline of the hydrants shall be located in the parkway, 3’6” behind the back of curb with a splash pad provided from edge of pavement to the fire hydrant.

2. Fire hydrant lead shall have a gate valve at the connection to the main. Fire hydrants in commercial and industrial areas shall generally be at street intersections and so located that there will be a fire hydrant every three hundred (300) feet. Fire hydrants in a residential area shall be generally located on street intersections (or on property line at locations other than street intersections) and not over five hundred (500) feet apart. Fire hydrants in a multi-family complex shall be generally located on street intersections and not over three hundred (300) feet apart.

3. Materials
   a. Fire hydrants shall be manufactured in accordance with AWWA C-502, dry-barrel fire hydrants.
   b. Hydrants shall be manufactured such that all maintenance and adjustments can be performed without excavation and such that hydrants may be faced in any direction in relation to base.
   c. Each fire hydrant shall have one (1) 4 1/2” pumper connection and two (2) 2 1/2” hose connections. Threads for hose connections shall be National Standard Threads. The hydrant shall open counter clockwise.
   d. Painting
      i) The fire hydrant bonnet and body shall be powder coated silver from the manufacturer.
4. Manufacturers
   i) Mueller (Super Centerlon 200)
   ii) American Darling (8-84-B)
   iii) M&H Valve Company

5. Installation
   a. General
      i) Installation shall be in accordance with AWWA M-17. The use of a 90° anchor or flange fitting shall be required unless otherwise approved by the Town Manager or his designee.
      ii) Fire hydrants shall have a maximum buried depth of 7 feet including extension.

   b. Location Markers
      i) The location marker shall be placed in the center of the roadway opposite the fire hydrant. The installation of this reflector shall be in accordance with the manufacturer's recommendation. Location markers shall be Stemsonite 1-88-55A or approved equal (reflective and blue in color).

D. Valves

1. Resilient seated gate valves shall be used for all sizes of water lines. Butterfly valves shall only be used when approved by the Town Manager or his designee.

2. Valves of approved design shall be installed at the intersections of all water mains so as to provide for proper maintenance and operation of the system and to provide a means of shutting off the supply to portions of the system for repairs. Valves shall be spaced such that only one fire hydrant is out of service at any one time. Four valves shall be used on a four way water line intersection and three valves shall be used on a three way intersection. Main line valves shall also be placed on each side of main where fire vault is connected.

3. In-line valves shall be added to each side of main where fire vault is connected.

4. Materials
   a. Resilient Seated Gate Valves
      i) Resilient seated gate valves 6” through 20” shall meet or exceed the latest revisions of AWWA C509 and shall meet or exceed the requirements of these standards.
ii) The valve body shall be cast iron or ductile iron. The valve body for valves 16" and larger shall be constructed of ductile iron. Flanged ends shall be furnished in accordance with ANSI/AWWA C115/A21.15 Standard Flanged Drilling. Mechanical Joints furnished shall be furnished with outlets which conform to ANSI/AWWA C111/A21.11 mechanical joint requirements.

iii) The disc shall be fully encapsulated in rubber. No iron shall be exposed on the disc.

iv) Bolt, hex head and nut shall be Steel ASTM A307 Gr. B, Zinc Plated per ASTM B633, SC3 for non-buried service and for 4" through 12" valves. Bolt, hex head and nut shall be 316 Stainless Steel for buried service valves.

v) T-Bolts shall be high strength low alloy Cor-Ten or approved equal.

vi) Resilient seated gate valves for buried service shall be furnished with a square 2" operating nut. The valve box shall be by American Flow Control series or approved equal. The valve box lid shall be painted safety blue. The paint shall be Glidden or approved equal.

vii) 20" and 24" valves shall be furnished with gear reduction.

viii) Valves larger than 24" shall be approved by the Town Manager or his designee.

b. Butterfly Valves

i) Butterfly valves 20" and larger shall meet or exceed the latest revision of AWWA Standard C504 for Class 150B butterfly valves and shall meet or exceed the requirements of this specification. All valve components shall conform to Underwriters Laboratories classification in accordance with ANSI/NSF Standard 61.

ii) Valve bodies shall be of cast iron per ASTM A 126 Class B. Flanged end valves shall be of the short body design with 125 lb. flanged ends faced and drilled per ANSI B16.1 standard for cast iron flanges. Mechanical joint end valves shall meet the requirements of AWWA C111/ANSI21.11.

iii) Discs shall be cast iron per ASTM A48 Class 40C. The disc seating edge shall be solid 316 stainless steel. The disc shall be securely attached to the valve shaft utilizing a field removable/replaceable 304 stainless steel torque screw or a tangential pin locked in place with a set screw.

iv) Valve shaft shall be type 304 stainless steel. Valve shaft seals shall be self-compensating V-type packing with a minimum of 4 sealing rings. One-piece molded shaft seals and O-ring shaft seals are not acceptable.

v) The seat shall be a Buna-N for water to 180°F, or EPDM for air to 29°F, and shall be molded in and vulcanized to the valve body. The seat shall contain
an integral shaft seal protecting the valve bearings and packing from any line debris. Seats vulcanized to cartridge inserts in the valve body and seats on the disc are not acceptable.

vi) Valve shaft bearings shall be non-metallic and permanently lubricated.

vii) Unless otherwise specified, exterior and interior metallic surfaces of each valve shall be shop painted per the latest revision of AWWA C504.

viii) Bolt, hex head and nut shall be Steel ASTM A307 Gr. B, Zinc Plat per ASTM B633, SC3 for non-buried service and for 4" through 12" valves. Bolt, hex head and nut shall be 316 Stainless Steel for buried service valves. If a standard valve box is used with buried service, the valve box lid shall be painted safety blue. The paint shall be Glidden or approved equal.

ix) T-Bolts shall be high strength low alloy Cor-Ten or approved equal.

x) Valves larger than 24" shall be approved by the Town Manager or his designee.

5. Manufacturers
   i) American Flow Control
   ii) Clow
   iii) Mueller
   iv) M&H Valve Company

6. Installation
   a. General
      i) Valves shall be furnished with extensions, such that the working nut is a maximum of 48" below grade.

      ii) Adjustable valve boxes shall be furnished and set on each valve in accordance with these standards. Valves that are deeper than 48", AWWA C900 PVC pipe shall be used for stacks, as long as the adjustable valve box is used at the top.

      iii) After the final clean-up and alignment has been complete, the contractor shall cast in place a concrete block, 24" x 24" x 6" around all valve box tops at the finish grade, with geodetic marker.

      iv) Valves located within a right-of-way shall be indicated on the face of the curb, or where curbs do not exist, on a conspicuous location adjacent to the valve location. Markings are to be the saw cutting of a four (4) inch high letter "V" with the point of the "V" pointing towards the valve location.
v) Valve box covers shall be painted safety blue.

vi) Valve markers shall be provided in undeveloped areas and as required by the Town Manager or his designee.

vii) Detector pads embedded in sand shall be installed beside all valves.

E. **Air Release and Flushing Valves**

1. Adequate fire hydrants, air release, and flushing valves shall be provided for flushing, disinfection, daily operation requirements, and repairs when required by the Town Manager or his designee. Air release valves shall be required on 16” and larger water lines. Water lines shall be designed so that each section of the water line can be flushed at its lowest and highest points.

2. All dead end lines shall have a fire hydrant installed for flushing purposes. If installation of a fire hydrant is not possible, a flushing valve is required.

3. A fire hydrant shall be required at high points on 12” and smaller water lines for air relief and flushing.
4. Materials
Air release valves and air/vacuum shall meet or exceed the latest revision of AWWA C512.

5. Manufacturers
VENT-O-MAT or town approved equivalent.

F. Tapping Sleeve
A tapping sleeve and valve shall be used when connecting a new water line to an existing line. A resilient seated gate shall be flanged to the tapping sleeve. Coupon must be furnished to the inspector. Tapping of size on size is not permitted.

G. Water Service
1. All water service installations shall be in conformance with the current Water Meter Installation Policy.

H. Flushing Valves
1. Materials
   a. Corporation stop shall be 2” ball type with compression outlet fitting, designed for a minimum working pressure of 200 psi.
   b. 2” curb stop shall be ball type with compression inlet fitting with tee head shut off.
   c. Pipe shall be 2-inch diameter, Type K copper as specified in ASTM B88.

I. Test Procedures
1. All Water Mains
   a. The test pressure shall not exceed 150 psi.
   b. The test pressure shall be reduced by 0.43 pounds per vertical foot measured between the pump-in point and the lowest point in the test section.
   c. The test pressure shall not vary by more than +/- 5 psi for the duration of the test.
   d. The test pressure shall not exceed the rated pressure of the valve when the pressure boundary of the test section includes closed resilient-seated gate valves or butterfly valves.
d. During the pressure test, all exposed joints, valves, etc. are checked for leaks and repairs made. Then the allowable leakage test is made.

2. Flushing

Foreign material left in the pipelines during installation often results in valve leakage during pressure tests. Thorough flushing is recommended prior to the pressure test.

3. Pressurization

Each valved section of the pipe shall be slowly filled with water, and the specified test pressure shall be applied by means of a pump connected to the pipe. The system should be allowed to stabilize at the test pressure before conducting the test.

4. Air Removal

Before applying the specified test pressure, air shall be expelled completely from the section of piping under test. After all the air is expelled, the corporation cocks at the blow-offs shall be closed.

5. Examination

All exposed pipe, fittings, valves, hydrants, and joints shall be examined during the test. Visual leaks shall be repaired and the test repeated.

6. Leakage defined

Leakage shall be defined as the quantity of water that must be supplied into the newly laid pipe to maintain pressure within 5 psi of the specified test pressure. Leakage shall not be measured by a drop in pressure in the test section over a period of time.

7. Acceptance of installation

Acceptance shall be determined on the basis of allowable leakage.


Pressure test at 150 psi

Pressure test duration -4 hours
Allowable Leakage:
No pipe installation will be accepted if the leakage is greater than that determined by the following formula:

\[
L = \frac{SD(P)^{1/2}}{133,200}
\]

- \(L\) = allowable leakage in gallons per hour
- \(S\) = length of pipe tested in feet
- \(D\) = nominal diameter of the pipe in inches
- \(P\) = average test pressure during the test in pounds per square inch

The formula is based on allowable leakage of 11.65 gpd/mi/in of nominal diameter at a pressure of 150 psi.
When testing against closed metal seat valves, an additional leakage per closed valve of 0.0078 gal/h/in of nominal valve size shall be allowed.

\textbf{Note:} The AWWA 600 Standard has tables of allowable leakage for size and length of pipe, and duration of test that may be used which are calculated by the formula shown above.

9. Polyvinyl Chloride (PVC) pipe [AWWA C605]
- Pressure test at 150 psi
- Pressure test duration – four hours
- Allowable leakage:
  \[
  L = \frac{ND(P)^{1/2}}{7,400}
  \]
  - \(L\) = allowable leakage in gallons per hour
  - \(N\) = number of joints in the length of pipe tested
  - \(D\) = nominal diameter of the pipe in inches
  - \(P\) = average test pressure during the test in psi

Concrete Pressure Pipe, Bar-Wrapped, Steel-Cylinder Type [AWWA C303]
Prestressed Concrete Lined Cylinder Pipe [AWWA C301]
Prestressed Concrete Embedded Cylinder Pipe [AWWA C304]
- Pressure test at 150 psi
- Pressure test duration – four hours
- Allowable leakage:
  \[
  L = \frac{ND(P)^{1/2}}{7,400}
  \]
  - \(L\) = allowable leakage in gallons per hour
  - \(N\) = number of joints in the length of pipe tested
  - \(D\) = nominal diameter of the pipe in inches
  - \(P\) = average test pressure during the test in psi

10. Disinfecting Water Mains

1. Meters shall be utilized for all Town water usage associated with the project.
2. Purpose

The purpose of this specification is to define the minimum requirements for the disinfection of water mains, including the preparation of water mains, hydrostatic tests, flushing, application of chlorine, and sampling for the presence of coliform bacteria.

3. Chlorine for Disinfection

Calcium Hypochlorite in granular form conforming to ANSI/AWWA B300 must be used and must contain approximately 65 percent available chlorine by weight. The material should be stored in a cool, dry, and dark environment to minimize deterioration.

4. Basic Procedure

a. Inspecting materials to be used to ensure their integrity.

b. Preventing contaminating materials from entering the water main during storage and construction.

c. Remove, by flushing or other means, those materials that may have entered the water main.

d. Perform a hydrostatic test.

e. Chlorinating any residual contamination that may remain, and flushing the chlorinated water from the main.

f. Protecting the existing distribution system from backflow caused by Hydrostatic test and disinfection procedure.

5. Preventive and Corrective Measures During Construction

a. General. Heavy particulates generally contain bacteria and prevent even very high chlorine concentrations from contacting and killing these organisms.

b. Keeping pipe dry and clean.

   (1) Openings in the pipeline shall be closed with watertight plugs when pipe...
laying is stopped.

(2) The lubricant used in the installation of sealing gaskets shall be suitable for use in potable water and shall not contribute odors.

(3) If dirt enters the pipe during storage or installation, it shall be removed and the interior surface swabbed with a 1 to 5 percent hypochlorite disinfecting solution.

c. Connection to the existing distribution system. Water required to fill the new main for hydrostatic pressure testing, disinfection, and flushing shall be supplied through a temporary connection between the distribution system and the new main. The temporary connection shall include an appropriate cross-connection control device and shall be disconnected during the hydrostatic pressure test. As an alternate, a connection to the existing distribution system is permitted provided a new valve is placed at the connection point. Do not test against an existing valve in the existing system.

6. Purging. Purging may be accomplished by passing an appropriate sized “poly-pig(s)” through the pipe or by flushing. A “poly-pig” must be used on all mains 12-inch and larger.

a. Flushing Method

Before the main is chlorinated, it shall be filled to eliminate air pockets and flushed to remove particulates. The flushing velocity in the main shall not be less than 2.5 ft/sec.

Required flow and openings to flush pipelines with a pressure of 40 psi

<table>
<thead>
<tr>
<th>Pipe Inch</th>
<th>Flow gpm</th>
<th>1” Tap</th>
<th>1-1/2” Tap</th>
<th>2” Tap</th>
<th>2-1/2” Hydrant Outlets</th>
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</thead>
<tbody>
<tr>
<td>4</td>
<td>100</td>
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<tr>
<td>16</td>
<td>1600</td>
<td>4</td>
<td>2</td>
<td>2</td>
<td></td>
</tr>
</tbody>
</table>

b. Poly-Pig Method

(1) The “poly-pig” shall be inserted in the new conduit at the location where the new conduit is connected to the active distribution system.

(2) Where expulsion of the “poly-pig” is required through a dead-ended conduit, the Contractor shall make every effort to prevent back flow of the purged water into the conduit after passage of the pig. Backwater re-entry into the pipe can be prevented by the temporary installation of mechanical joint bends and pipe joints to provide a riser out of the trench.
(3) After passage of the “poly-pig”, flushing of all backwater from the pipe, and satisfactory test results, the Contractor shall secure the test location openings and then proceed with disinfection.

7. Hydrostatic Test

All ductile iron and plastic pipelines shall be tested with a hydraulic test pressure of not less than 150 psi over a period of not less than 4 hours. The rate of leakage of all pipe tested shall not exceed 11.65-gallons-per-inch of nominal diameter of the pipe per mile. If the tests indicate a leakage in excess of the acceptable rate, the Contractor shall be required to find and repair the leak. Even if the test requirements are met, all apparent leaks shall be stopped.

The hydrostatic pump shall be connected to a system where the amount of leakage can be determined by measurement or gauge. The 200-psi pressure shall be maintained over the entire 2-hour test period. The leakage shall be determined by comparing the quantity of water in the measuring system at the beginning of the test and quantity of water at the end of the test. The difference in these quantities shall be the leakage. An alternate method is to add water to the measuring system during the test. At the end of the 2-hour test, the quantity of water added shall be the leakage.

8. Disinfection (Chlorination)

a. Two methods of chlorination are used: Continuous feed and slug.

b. The slug method gives a three hour exposure of not less than 50mg/L free chlorine.

c. The Continuous-feed method must be used unless it is stated otherwise in the Contract Specifications.

d. The Contractor shall install and remove all pump-in, blow-off and sampling points.

e. Water from the existing system or other approved source shall be made to flow at a constant rate in the new main.

f. At a point no more than 10-ft downstream of the beginning of the new conduit, water entering the new conduit shall receive a dose of chlorine such that the water shall have not less than 100-mg/L (ppm) free chlorine. Chlorine application shall not cease until the entire conduit is filled with heavily chlorinated water. 125 lbs of Calcium Hypochlorite (65% available chlorine) is required in 100,000 gal of water to produce 100 mg/L (ppm) Chlorine concentration.

g. The chlorinated water shall be retained in the conduit for at least 24 hours, during which time all valves and hydrants in the section treated shall be operated in order to disinfect the appurtenances. Every effort shall be made to prevent the...
flow of chlorinated water into conduits in active service. At the end of the 24-hour period, the treated water in all portions of the conduit shall have a residual of at least 10-mg/L (ppm) free chlorine.

h. The heavily chlorinated water shall then be flushed from the conduit and disposed in a manner meeting the requirements set out below.

i. The chlorine residual shall be tested prior to flushing operations.

9. Heavily Chlorinated Water

If the chlorine residual exceeds 4-mg/L (ppm) the water shall remain in the new water conduit until the chlorine residual is less than 4-mg/L (ppm). As an alternate, the Contractor may choose to evacuate the water into water trucks, or discharge into an existing sanitary sewer system, or an approved storage facility (such as a detention pond until the chlorine residual is 4-mg/L (ppm) or less), or treat the water with Sodium Bisulfite or another dechlorination chemical (Sulfur Dioxide, Sodium Sulfite, Sodium Thiosulfate, or Ascorbic Acid) or method appropriate for potable water and approved by the Owner until the chlorine residual is reduced to 4-mg/L (ppm) or less. **The heavily chlorinated water shall not be disposed of into the storm sewer system.** After the specified chlorine residual is obtained, less than 4-mg/L (ppm), the water may then be discharged into the storm sewer system or utilized by the Contractor.

The requirement for discharge of heavily chlorinated water is found in the TPDES General Permit To Authorize the Discharge of Storm Water And Certain Non-Storm Water Discharges from Regulated Construction Activities Within the State of Texas.

10. Contractor Requirements

The Contractor shall prepare the conduit for disinfection activities and secure same after chlorination is complete.

a. This shall consist of furnishing all equipment, material and labor to satisfactorily prepare the conduit for disinfection. The Contractor shall also be required to provide adequate provisions for sampling.

b. The Contractor shall make all necessary taps into the pipe to accomplish chlorination of a new line.

c. After satisfactory completion of the disinfection operation, the Contractor shall remove surplus pipe at the chlorination and sampling points, plug the remaining pipe, backfill, and complete all appurtenant work necessary to secure the conduit.

11. Sampling
a. Unless otherwise specified, the Contractor shall inject chlorine disinfectant into the conduit and monitor the solution.

b. The Owner’s inspector shall take water samples from a suitable tap (not through a fire hydrant) for analysis by the North Texas Municipal Water District laboratory. The sample(s) shall be transported by City staff to the laboratory at 9:00 AM on Tuesdays and Thursdays. Samples may not be taken earlier than 3:00 PM on the day prior to delivery. The Owner’s inspector shall notify the Contractor of the results.

c. Microbiological sampling shall be done prior to connecting the new conduit into the existing distribution system in accordance with AWWA C651 Disinfecting Water Mains. Samples shall be tested in accordance with Standard Methods for the Examination of Water and Wastewater. Samples for bacteriological analysis shall be collected in sterile bottles treated with sodium thiosulfate. At least one sample shall be collected from every 1,000-linear-feet of new water conduit, plus one set from the end of the line and at least one set from each branch. If trench water has entered the new conduit during construction or, if in the opinion of the City inspector, excessive quantities of dirt or debris have entered the new conduit, samples shall be taken at intervals of approximately 200-linear-feet. Samples shall be taken of water that has stood in the new conduit for at least 16-hours.

d. Unsatisfactory test results shall require a repeat of the disinfection process and resampling as required above until a satisfactory sample is obtained.

e. In the event there are three unsatisfactory test results from the same sampling point, the Contractor must “poly-pig” the new water main and samples taken again until a satisfactory sample is obtained.

12. Measurement and Payment

Measurement and payment for hydrostatic testing and disinfection shall not be applicable under this Technical Specification unless specified otherwise. The Contractor shall provide materials, equipment and services in support of both purging and disinfection of the conduit. All costs for these services shall be included in the unit bid price in the proposal per foot of pipe complete in place, and no other compensation shall be allowed.

13. Water Samples

Contractor shall pick up sampling bottles and paper work from a Town approved lab. Samples shall be taken by the contractor with the Inspector present and shall be released to the Inspector to take to the lab. The water line shall not be in the Town’s system until a copy of the passing tests are provided to and approved by the Chief Construction Inspector or his designee. Testing intervals shall not exceed 1000 ft and will be taken at each dead-end.
END OF SECTION
# Water Construction Standards Details

*(Provided for Reference – Included in Volume II)*

<table>
<thead>
<tr>
<th>Title</th>
<th>Detail No.</th>
<th>Sheet</th>
<th>Revision Date</th>
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<td>1</td>
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III. Wastewater System Improvements

A. General
This section pertains to general design requirements for wastewater collection system construction in the Town of Flower Mound. All wastewater flow calculations and all sewer lines shall be sized and designed in accordance with the Town of Flower Mound Wastewater System Master Plan or as determined by the Town Engineering Department. No development shall require the use of lift stations, other than the existing lift stations, for the conveyance of wastewater flows without the approval of the Town Council. Other pertinent requirements for sewer connections, over-sizing, main extension, wastewater restrictions, conservation, impact fee and other sewer related fees are included in the Town Code. In the absence of specific standards, all collection, treatment, and disposal systems shall be designed in accordance with the most current criteria adopted by the Texas Commission on Environmental Quality (TCEQ), Chapter 317, "Design Criteria for Sewerage Systems". Drawings must be submitted to TCEQ for review and approval. Approval letter from TCEQ must be submitted to the Town.

B. Sewer Lines
1. Standard sewer line sizes are 8”, 12”, 15”, 18”, 21”, 24”, 27”, and 30” diameter. Other sizes must be approved by the Town Manager or his designee.

2. Sewer lines shall be a minimum of 8 inches in diameter.

3. Gravity sewer lines shall be constructed at a minimum depth of 5’. Pressure lines shall be constructed at a minimum depth of four (4) feet. Sewer lines shall be located in the parkway on the south and west side of the roadway, and are required to be constructed on both sides of a State Highway. No service lines will be allowed to cross a State Highway. Deviations of these requirements may be approved by the Town Manager or his designee in circumstances where compliance is not physically feasible. All sewer lines, whether main lines or service lines, crossing existing streets shall be placed by dry boring within an encasement. Open cut excavation will not be allowed to cross existing streets.

4. Easements for sewer line construction shall meet the following requirements:
   a. The easement width shall be a minimum of 15 feet.
   b. If the sewer line is less than 12 feet deep, the outside diameter of the sewer line shall be located a minimum distance of 6 feet from the edge of the easement, and if other utilities are located in the same easement, the outside diameter of the sewer line shall be located a minimum distance of 3 feet from the outside diameter of the other utilities.
   c. If the sewer line is greater than 12 feet deep, the outside diameter of the sewer line shall be located a minimum distance of 9 feet from the edge of the
easement, and if other utilities are located in the same easement, the outside diameter of the sewer line shall be located a minimum distance of 6 feet from the outside diameter of the other utilities.

d. Parallel lines will require an additional 5’ easement width, a minimum of 6’ from deeper line.

5. Sewer line shall be located 3 feet from back of curb or 3’ feet from edge of pavement, opposite side from water line, if there is no curb and gutter.

6. All sewers shall be designed with consideration for serving the full drainage area subject to collection by the sewer in question.

7. Sewers should be designed with straight alignment whenever possible. When horizontal curvatures must be used, the maximum joint deflection should be in accordance with the pipe manufacturer’s recommendations and comply with TCEQ requirements.

8. All sewer line installations must extend to the borders of the subdivision or property as required for future extensions of the collection system, regardless of whether such extensions are required for service within the subdivision or property. Must end with a manhole. The amount of trench excavation shall not exceed 200 (two hundred) feet from the end of the pipe laying operations, and no more than 300 (three hundred) feet of total open trench will be allowed. At the end of each work day, all trench excavation shall be backfilled to the end of the pipe laying operation. Barricades, safety fencing, and lights will be required around any open trench left overnight.

9. All sewers shall be designed with hydraulic slopes sufficient to give mean velocities, when flowing full or half full, of no less than two feet (2’) per second and no more than ten feet (10’) per second on Kutter’s or Manning’s formulas using an "n" value of 0.013, at design flow. Design flow, slopes, and velocities shall also conform to (TAC 30, Chapter 317, Design Criteria for Sewage System).

10. Materials

a. Polyvinyl Chloride (PVC) Pipe

i) All sanitary sewer pipes shall be PVC pipe type SDR-35 for gravity sewer lines constructed less than twelve feet (12’) deep. Type SDR-26 shall be provided where gravity sewer lines exceeds twelve feet (12’). C900 DR18 PVC pipe is required for depths greater than 24’. If service connections are needed on sewer pipe constructed below fifteen feet (15’) in depth for a minimum of 500’, a parallel line shall be constructed at a shallower depth, specifically for service connections. Ribbed pipe will be allowed on deeper pipe if no service lines are connected to that line. Pressure sewer pipe shall be C900 DR18 PVC.
ii) All gravity PVC sanitary sewer pipe shall be green in color. Pressure sewer pipe shall be white in color.

iii) PVC sewer pipe and fittings shall conform to the current ASTM Designation D 3034 for 8”-15” and ASTM Designation F 679 for greater than 15”.

11. Installation

a. General

i) Spacing of pipes shall comply with latest TCEQ standards.

ii) All installations shall conform to ASTM Designation D2321, and the latest NCTCOG Specifications as amended by these standards.

iii) Construction shall begin at downstream end of project and continue upstream with bell facing upstream. No upstream piping shall be installed before downstream piping unless approved by the Town Manager or his designee.

iv) When PVC pipe is used for, green marker tape with the wording “Buried Sanitary Sewer” shall be installed in the backfill material no more than twelve inches (12”) above the top of the pipe.

v) The amount of trench excavation shall not exceed 200 (two hundred) feet from the end of the pipe laying operations, and no more than 300 (three hundred) feet of total open trench will be allowed. At the end of each workday, all trench excavation shall be backfilled to the end of the pipe laying operation. Barricades and lights will be required around any open trench left overnight, for any trench within right-of-way or public access easement.

vi) Approved plugs shall be installed at the open ends of the line at the end of each working day. All joints shall be assembled free of dirt and any foreign matter.

vii) Water jetting, jacking or missile type bores shall not be permitted under any condition.

viii) Reference Part B Section V for trench testing.

ix) When a 150 psi rated sewer line is required due to its proximity to a water line, the 150 psi rated pipe shall terminate at a manhole on each end. The pipe shall be extended to the interior wall of the manhole. No external boot connection will be allowed.
C. Manholes

1. Manholes shall be located at all intersections of sewer lines and at intermediate spacing along the line. Generally the maximum spacing should not exceed 500 feet. Manholes should be located at all changes in grade and at the ends of all sewer lines that will be extended.

2. A manhole is required at the junction of sewer lines with different inside pipe diameters.

3. A tenth foot (.1') of fall is required through the manhole when a change in flow direction occurs.

4. The flow line into a manhole must not be greater than 6" above the flow line out of the manhole. Where the flow line in is greater than two feet (2') above the flow line out, a drop inlet is required.

5. Minimum manhole inside diameter is four feet (4').
   a. If depth is greater than 12' the minimum diameter shall be 5 feet.

6. Installation
   a. Use the following table to determine sanitary sewer manhole sizes:

<table>
<thead>
<tr>
<th>Pipe Sizes</th>
<th>Diameter of Manhole</th>
</tr>
</thead>
<tbody>
<tr>
<td>8&quot; through 18&quot;</td>
<td>4'</td>
</tr>
<tr>
<td>21&quot; through 30&quot;</td>
<td>5'</td>
</tr>
<tr>
<td>33&quot; through 48&quot;</td>
<td>6'</td>
</tr>
<tr>
<td>&gt;48</td>
<td>Special Design</td>
</tr>
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</table>

Note:
1. If the proposed design requires the sewer line to be placed at depths greater than shown above, the design will require approval by the Town Manager or his designee.

2. The clear distance between the outside of adjacent pipes shall not be less than two feet.
7. Drop-connection manholes shall have a minimum inside diameter of five feet (5'), with a single interior drop connection. Drop MHS for lines over 12' in depth or drop MHS with more than 1 interior drop. Must be a minimum of 6' in diameter.

8. Materials
   a. Concrete
      i) All manholes shall be constructed of cast-in-place or precast concrete.
   b. Cast-in-place Manhole
      v) Minimum cast in place manhole wall thickness is eight inches (8”). For depth's greater than 20 feet a special design will be required.
      ii) The manhole foundation shall be poured on undisturbed soil and shall have a minimum thickness of eight inches (8”).
      iii) The inlet and outlet pipes shall be poured into the foundation of the manhole. The pipe shall extend one-and-one-half inches (1 1/2”) into the manhole.
      iv) The invert shall be shaped and smoothed so that no projections will exist and the invert shall be self cleaning. The invert floor shall have a minimum slope of one-inch (1”) per foot.
      v) Concrete work shall conform to all requirements of ACI 301, Standard Specification for Structural Concrete, published by the American Concrete Institute, except as modified herein.
      vi) Detailing of concrete reinforcement and accessories shall be in accordance with ACI Publication 315.
      vii) Portland Cement shall be Type II, low-alkali and conform to ASTM Designation C-150.
      viii) The manhole shall not be backfilled within 12 hours after the concrete placement.
      ix) The face of curb shall be sawed with an "MH" to mark the location of all manholes. The location of the stamp shall be a line that intersects the center of the manhole cover and the curb perpendicular to the centerline of the street. For manholes located in intersections, the curb shall be stamped at the closest location to the manhole.
   c. Precast Manhole
      i) Minimum pre-cast wall thickness is 5".
ii) Precast manholes shall be constructed in accordance to ASTM Designation C-478.

iii) Manhole base shall have a spread footing and be placed on a minimum of twelve-inches (12") of ¾" crushed rock.

D. Manhole Frame and Cover

1. Cover
   a. Materials
      All manhole covers shall include Town approved brass ID marker.
      All manhole covers shall conform to the Standard Specifications for Ductile Iron Castings, ASTM A536.
   b. Installation
      i) All manhole covers shall be 32-inches in diameter.
      ii) Manhole covers shall indicate “Sanitary Sewer”.
   c. Manufacturers
      i) Certain Teed PAMREX
      ii) Power Seal

2. Frames
   a. Materials
      All manhole frames shall conform to the Standard Specifications for Ductile Iron Castings, ASTM A536.
   b. Installation

3. Extension Ring
   a. Materials
      All precast reinforced concrete extension rings shall conform to ASTM C-478.
   b. Installation
      i) The number of extension ring sections shall be kept to a minimum (i.e. use 1-12” extension ring instead of 2-6” extension rings).
      ii) A 1” x 3 ½” bitumastic gasket shall be used to seal the extension ring at both joints.
iii) Reference latest NCTCOG specs for max height of neck and minimum opening diameter.

E. Sewer Service

1. No sewer service line (lateral) shall be less than 4" in nominal diameter. Commercial sewer laterals shall be 6" minimum diameter.

2. Sewer laterals shall be located at the center of the lot and extended to the property line and be a minimum of 10 feet downstream of the water service. The end of the lateral shall have a green detector pad with tape extending up to final grade.

3. Sewer service laterals shall have no more than 6’ of cover at the property line.

4. Materials

   a. Polyvinyl Chloride (PVC) Pipe

      i) All lateral sewer service lines shall be PVC pipe type SDR-35.

      ii) All PVC sanitary sewer pipe used for lateral services shall be green in color.

5. Installation

   a. Polyvinyl Chloride (PVC) Pipe

      i) All service laterals shall be installed in accordance with the sanitary sewer embedment and backfill standards.

      ii) All service laterals below proposed area to be paved shall be installed and properly backfilled prior to the subgrade preparation and pavement construction.

      iii) All lateral locations shall be saw-cut into the curb with an "II" at the point the lateral crosses the curb with 4” high lettering painted green. The lateral indicator mark shall be placed at the edge of pavement when there is no curb and gutter.

F. Cleanouts

1. Residential Service Line cleanouts shall be placed at property line, and to final grade prior to acceptance of subdivisions. Cleanouts shall not be placed in future sidewalk location.

2. Materials

   a. Polyvinyl Chloride (PVC) Pipe

      i) All cleanouts are to be constructed of PVC pipe type SDR-35.

      ii) All PVC sanitary sewer pipe shall be green in color.
iii) PVC sewer pipe and fittings shall conform to the current ASTM Designation D 3034 for 4"-15" and ASTM Designation F 679 for greater than 15".

G. Main Line Cleanouts

*Main line cleanouts are not allowed.*

H. Aerial Sewer

1. The piers for the aerial crossing shall be designed in accordance with the guidelines of the Ductile Iron Pipe Research Association.

2. Aerial sewer crossing shall be located in areas where the sewer line can not be constructed with the appropriate minimum cover. The design engineer shall design the aerial crossing in accordance with these standards and as approved by the Town Manager or his designee.

3. Pier placement and spacing shall be determined according to soils analysis performed by a geotechnical engineer. Piers shall be placed at a maximum span distance as indicated by the design engineer’s calculations.

4. Pier placement and spacing along with a soils report shall be submitted to the Town Manager or his designee.

5. Materials
   a. Pipe
      i) All above ground sewer installations shall be ductile iron, minimum Class 150, utilizing restrained joints and shall have a wall thickness required for the size and span as designed or approved alternate. The pipe shall have an internal polyurethane coating.

      ii) The aerial pipe shall be connected to the sanitary sewer pipe by means of a manhole on each side of the aerial crossing.

   b. Piers
      Piers to be constructed with a minimum of Class A 3,000 psi reinforced concrete.

6. Installation
   a. Pipe
      The design engineer shall submit a pipe design for approval by the Town Manager or his designee.

   b. Piers
The design engineer shall submit a pier design for approval by the Town Manager or his designee.

I. Lift Stations

1. Instrumentation and Control
   a. The voltage supplied for pump operation shall be 3 phase, 480 volts. Converting single phase power to three phase power using additional mechanical equipment shall not be allowed.

   b. Wet well level indication shall be accomplished through use of an ultra sonic level sensing device. The Town standard for this item is the Milltronics HydroRanger.

   c. A Motorola MOSCAD Remote Terminal Unit (RTU) shall be installed at all lift stations. Programming of the RTU is the responsibility of the contractor, and shall be coordinated with the Town. The Motorola RTU shall be a “smart” RTU, utilizing a micro-compressor for communications, calculations, local control, and data storage. The RTU shall be modular capable of receiving and transmitting control messages to and from the MASTER STATION via the existing repeater. An integral radio transceiver, shall be provided for direct communication with the MASTER STATION via the existing repeater. The RTU shall be the MOTOROLA ACE 3600. Substitutions will not be considered.

   d. Submersible pumps shall be provided with moisture and motor over temperature sensors.

   e. Discharge flow from the lift station shall be measured by using a magnetic flow meter. The meter manufacturer installation requirements shall be followed to ensure accuracy of flow measurements. The meter shall be placed in a concrete vault with an aluminum access door rated for the anticipated traffic load. Combining the meter vault with the valve vault is acceptable. The vault will be designed to drain or pump water accumulation in the vault to the wet well. The Magnetoflow® Mag Meter manufactured by Badger Meter Inc., is an approved model.

2. Pumps, Piping, Valves and Wells
   a. Pumps shall be sized to operate at optimum efficiency. Minimum acceptable efficiency at the operating point will be sixty percent (60%).

   b. Each pump discharge must have a cushioned swing check valve and isolation valve.

   c. Inlet piping shall be designed to minimize turbulence.

   d. Valves shall be located in a separate vault from the wet-well.
e. The valve vault shall have a concrete floor with a drain line connecting the vault to the wet-well. The drain line shall have a gasketed flap valve at the discharge into the wet-well to prevent wet-well contents from entering the valve vault.

f. Wet-well working volume shall be sized to allow for the recommended pump cycle of 6 minutes for each pump, with no more than 10 starts per hour.

g. Lift station piping shall be designed with an additional emergency pump connection, allowing the station to be operated with the primary pump(s) out of service for an extended period of time.

3. Site Requirements

a. All lift station sites are required to have a minimum 6 foot chain link fence. Fencing specifications are as follows:
   i) Fence fabric shall be hot dip galvanized 9 gauge steel.
   ii) Three strands of hot dip galvanized barbed wire are required above the top rail and must terminate at the corner posts with brace bands.
   iii) A 12' double gate is required for vehicle traffic.
   iv) Posts shall be schedule 40 hot dip galvanized steel. Post shall be placed in concrete.

b. A concrete pad will be required at the front of the control cabinet. The pad shall provide a 3' working area away from the face of the cabinet and extend the width of the enclosure mounting structure. Pad depth shall be a typical 4".

c. Crushed stone will be required inside the fenced area of the station. This requirement includes a water penetrating weed barrier covered with a minimum crushed stone in accordance with NCTCOG Item 2.1.8.(d).

d. 12" x 6" concrete perimeter curb is required to contain the crushed stone. The curb shall be 6" in width and extend approximately 6" below finished grade.

e. A potable water service shall be provided at the station site. A one inch service with a one inch angle stop and a RPZ back-flow preventer shall be installed in an appropriately sized meter box. The meter box shall meet the Town's current standard for water service connections.

f. The site shall be graded to drain away from the station to prevent stormwater inflow or infiltration into the wet-well.

g. The site shall be located outside of the 100-year flood plain.
h. The site shall not be located within 100 feet of an existing or proposed residence.

i. The lift station site shall include a driveway area for maintenance vehicles to park off public roadway while performing maintenance. The minimum driveway length shall be 15 feet.

j. A concrete driveway turning area is required where access drives extend more than 20 feet from main roads. The driveway area shall be "T" shaped with the applicable turning radius. The minimum driveway width shall be 15 feet.

k. The lift station site shall be property owned by or dedicated to the Town. Lift stations are prohibited from placement within public right-of-way or easements.

4. Materials

a. Instrumentation and Control
   i) All enclosures, with the exception of the metering base, shall be NEMA 4X stainless steel.
   ii) All electric conduit will be epoxy coated rigid steel or aluminum rigid.

b. Pump Accessories, Piping, Valves and Wells
   i) Wet-well interior piping shall be fabricated of flanged Class 150 Ductile Iron.
   ii) Fasteners used for pipe connection shall be 318 stainless steel.
   iii) Pump guide bars, guide bar brackets, cable/chain hooks and pump lifting chains shall all be fabricated of stainless steel.
   iv) Pump access door, and door frames must be fabricated from aluminum.
   v) Isolation valves shall be resilient seated gate valves in accordance with the water standards.
   vi) Swing check valves shall be in accordance with AWWA C-508. Eight inch (8") and larger check valves shall be equipped with a bottom mounted oil dash pot.
   vii) Wet wells shall be constructed of concrete. Fiberglass or steel wet wells are not acceptable.
   viii) The wet well interior concrete surface shall be coated with TNEMEC series 218 MotarClad surfacing material, followed by TNEMEC series 436 Perma-
Sheild FR at 125 mills dry film thickness, or Town approved equivalent. Application shall be per manufacturer recommendation.

ix) All submersible wet well pumps shall be equipped with an automatic mixing/flushing valve attached to the pump volute. This accessory item will direct a water jet across the floor of the wet well to temporarily suspend settled materials. This valve will operate by using the hydraulic energy created by the operation of a pump. An additional mixer needed to suspend settled material will not be accepted.

5. Installation

a. Instrumentation and Control

i) All stations shall be equipped with a 12 volt/DC flashing strobe placed at an elevation visible by passing traffic.

ii) Stations shall be equipped with radio equipment compatible with the Town's existing SCADA equipment. Antenna mountings, masts, and cables shall be provided for continuous and accurate telemetry and control.

iii) All stations will be equipped with a magnetic flow meter located on the discharge pipe. The flow meter shall be contained in a concrete vault with an aluminum access door. Installation of the meter shall be in accordance with manufacturer’s installation recommendations to ensure accuracy of flow valves.

iv) Modifications to the existing SCADA system will be required with the addition of any new station. The installing contractor shall provide the following information to the Town prior to beginning this modification process:

(a) The general contractor shall provide evidence that the instrumentation subcontractor has maintained a continuous business operation for at least five (5) years.

(b) The general contractor shall provide evidence that the instrumentation subcontractor maintains a staff of competent technicians and Licensed installers who are Motorola factory trained.

(c) The general contractor shall provide evidence that the instrumentation subcontractor maintains a fully staffed service shop for supplying demand service calls on systems and maintains an office of operation within a reasonable response distance (generally in the Texas Counties of Denton, Tarrant, or Dallas) to the Town of Flower Mound Wastewater Treatment Plant.
(d) The creation of new Human-Machine Interface (HMI) SCADA display screens, and modifications of existing HMF displays, must be proposed and approved prior to installation.

v) Enclosures shall be mounted on an appropriately sized mounting structure. Mounting structures shall be constructed of 6” x 2” x 0.25” hot dip galvanized steel channel stock. Intersections shall be bolted, not welded, with stainless steel fasteners. Aluminum or epoxy coated steel unistrut may be attached to the mounting structure to facilitate placement of enclosures. The legs of the mounting structure shall be set at least 24” below grade and be encased in concrete.

b. Pumps, Piping Valves and Wells

i) Pump access doors and door frames must be fabricated from aluminum with a recessed lifting handle, locking lever to hold the door in the open position, and a method of placing a No. 5 Master padlock on the door for safety and security.

ii) All submersible wet well pumps shall be equipped with an automatic mixing/flushing valve attached to the pump volute. This accessory item will direct a water jet across the floor of the wet well to temporarily suspend settled materials. This valve will operate by using the hydraulic energy created by the operation of a pump. An additional mixer needed to suspend settled material will not be accepted.

J. Testing Procedures

1. Sewer Lines

a. A Town inspector shall be in attendance for each testing procedure. Testing shall be performed by a company certified by the pipe manufacturer.

b. Deflection Testing. Upon completion of sanitary sewer pipe installation, the contractor shall pull a mandrel through the pipe to test for a maximum 5% deflection, unless otherwise specified.

c. Video Inspection. Before acceptance of a subdivision or project by the Town, the contractor will be required to retain a qualified company to perform a video inspection of the sewer mains in the subdivision at the contractor’s expense. Prior to video inspection, sewer mains shall be flushed. The video inspection shall be done no sooner than ten days prior to final acceptance of the project.

d. Criteria for Repair and Reinspection:

i) Pulled or slipped joints

ii) Water infiltration
iii) No standing water will be permitted in sewer lines with a slope greater than or equal to 1%. Standing water shall be permitted in sewer lines with a slope less than 1% to a maximum depth of 20% of the nominal inside diameter of the pipe.

iv) Structural damage to the pipe

v) The Town will make the final determination if repairs are required. A final set of tapes and logs shall be given to the designated Town Inspector of the project. Furnish two copies of audio/video inspection in DVD format, plus one copy in VHS format. By audio on tape, the operator must note the following:

(a) Date and time of recording
(b) Developer’s or contractor’s name
(c) Project name and contract number
(d) Name of company performing the inspection
(e) The location of line, designation, main size, direction of run, identify every 50-foot station, and identify the station of each manhole.

e. Air Testing

Air Testing shall be in accordance with NCTCOG requirements.

2. Manholes

A Town inspector shall be in attendance for each testing procedure. Vacuum Testing shall be in accordance with latest NCTCOG requirements.

END OF SECTION
## Wastewater Construction Standards

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IV. Storm Water System Improvements

A. Runoff Methods

1. Town Wide Hydrologic Study

The Town Wide Hydrologic Study flows shall be utilized for all areas where flows are provided by the study.

2. Rational Method

a. Methodology

For floodplain areas, the Town Wide Hydrology Study flow values shall be used.

Storm water runoff for drainage areas less than 200 acres may be calculated using the Rational Method. The Rational Method is based on the principle that the maximum rate of runoff from a given drainage area occurs when all parts of the area are contributing to the flow at the point of discharge. The formula for the Rational Method is:

\[ Q = CIA \]

Where
- \( Q \) = maximum rate of discharge (cfs).
- \( C \) = runoff coefficient based on topography, soil, land use and moisture content of the soil at the time the rainfall producing runoff occurs.
- \( I \) = rainfall intensity in inches/hour for the time period it takes flow from the farthest point of the drainage area to reach the point of design.
- \( A \) = the drainage area contributing to the runoff at the specified concentration point/outfall (acres).

b. Runoff Coefficient

The runoff coefficient “\( C \)” for existing conditions shall be based on the current land use and the coefficient for proposed conditions shall be based on the proposed land use. A weighted “\( C \)” may be used for both existing and proposed conditions upon approval of the Town Manager or his designee.

i) Runoff coefficient "C" values are based on topography and land use. The table entitled "Runoff Coefficient Values" included in Appendix A – Hydrology Table and Figures shall be used to determine "C" values.
c. Time of Concentration

Time of concentration ""Tc"" is the longest time, without interruption of flow by detention devices, which a drop of water takes to flow from the farthest point of the drainage area to the point of concentration (i.e. the point of design).

The SCS methodology is recommended to determine the time of concentration (Tc), which is based on three types of flow: sheet flow, shallow concentrated flow, and open channel flow.

i) Sheet Flow. Sheet flow is flow over plane surfaces. It occurs in the uppermost area of the defined basin.

The time of concentration in minutes for sheet flow is determined using the following equation:

\[
T_{c \text{ sheet flow}} = 60DV
\]

where,
\[
D = \text{distance along the flow path (feet)}
\]
\[
V = \text{velocity (feet per second)}
\]
\[
T_c = \text{time of concentration (minutes)}
\]

The longest flow path for the water is determined for the basin, which begins at the uppermost part of the basin and ends at the concentration point for the basin. Sheet flow length is measured along this flow path from the uppermost part of the basin for a length no more than 300-feet (At a maximum distance of 300-feet sheet flow changes to shallow concentrated flow).

A slope should be estimated along this length of flow. A velocity corresponding to this slope can be determined using the upland method graph found in Appendix A – Hydrology Tables and Figures.

ii) Shallow Concentrated Flow. Shallow concentrated flow begins where sheet flow ends. A projected slope should be established along the flow line for the shallow concentrated flow length. A velocity corresponding to this slope can be determined from the shallow concentrated flow graph in Appendix A – Hydrology Tables and Figures. The time of concentration in minutes for shallow concentrated flow is determined by the following equation:

\[
T_{c \text{ shallowconcentrated flow}} = 60DV
\]

where,
iii) **Open Channel Flow.** Open channel flow is applicable for large channel sections. In most cases, it is not applicable for the rational methods (since large channels indicate large watersheds where a unit hydrograph method is applied in the Town of Flower Mound). The time of concentration for open channel flow is determined using the following equations:

\[
T_c = \frac{60DV}{D}
\]

where,
- \(D\) = distance along the flow path (feet)
- \(V\) = velocity (feet per second)
- \(T_c\) = time of concentration (minutes)

\(V\) is determined using a combination of the continuity equation and Manning’s equation:

\[
V = 1.486 \left( \frac{R}{n} \right)^{\frac{2}{3}} S^{\frac{1}{2}}
\]

where,  
- \(n\) = Manning’s \(n\)
- \(R = A/P\) = hydraulic radius (feet)
- \(A\) = area (square feet)
- \(P\) = wetted perimeter (feet)
- \(S\) = slope (ft/ft)

iv) **Resulting Time of Concentration.** The time of concentration is the sum of the sheet flow, shallow flow, and open channel flow segments.

\[
T_c = T_{c\text{, sheet flow}} + T_{c\text{, shallow concentrated flow}} + T_{c\text{, channel flow}}
\]

In no case shall a longer time of concentration be used than:

(a) 10 minutes for property zoned multiple family, high-density single family (lots smaller than 7,500 square feet), churches, schools, local business, central business, mixed use, commercial, or industrial.
(b) 15 minutes for property zoned for parks, cemeteries, agricultural, and single family residential (7,500 square foot lots or larger).

v) **Resulting Time of Concentration for Inlets.** When designing inlets the time of concentration is equal to the inlet time. The design engineer may use the actual calculated or specified inlet time. In no case shall a longer inlet time be used than as specified in items (a) and (b) below.

(a) 10 minutes for property zoned multiple family, high-density single family (lots smaller than 7,500 square feet), churches, schools, local business, central business, mixed use, commercial, or industrial.

(b) 15 minutes for property zoned for parks, cemeteries, agricultural, and single family residential (7,500 square foot lots or larger).

c. **Rainfall Intensity**

Rainfall depths can be found from the Town of Flower Mound IntensityDuration-Frequency (IDF) curve. A copy of the IDF curve is provided in Appendix A – Hydrology Tables and Figures.

Design storm frequencies for various drainage structures are listed in Appendix A – Hydrologic Tables and Figures.

3. **Unit Hydrograph Method**

a. **Methodology**

A unit hydrograph method is required for drainage areas greater than 200 acres. The recommended unit hydrograph model is HEC-1 and HEC-HMS. The Town Manager or his designee must approve other unit hydrograph methods.

b. **Rainfall**

Design storm frequency requirements are listed in Appendix A – Hydrology Tables and Figures. Rainfall values can be determined using the Town of Flower Mound IDF curve and corresponding table included in the same appendix.

c. **Loss Rate**
i) **Methodology**  
The loss Rate shall be determined using SCS Curve Numbers. A table of Curve Numbers corresponding to land use can be found in Appendix A – Hydrology Tables and Figures.

ii) **HEC-1**  
For the HEC-1 hydrology model, the LS card should be used to model the loss rate.

d. **Time of Concentration**

i) **Methodology**  
The time of concentration shall be developed using SCS methods. The methodology is listed in section A. Rational Method, 3. Time of Concentration. The basic principal of the SCS methodology for estimating the time of concentration is where the total time of concentration is equal to the sum of the time of concentrations along the flow path for sheet flow, shallow concentrated flow, and open channel flow.

\[ T_{C \text{ total}} = T_{C \text{ sheet flow}} + T_{C \text{ shallow concentrated flow}} + T_{C \text{ open channel flow}} \]

ii) **Methodology**  
The Clark Unit Hydrograph Method (UC card, time of concentration in hours) shall be used in the HEC-1 hydrology model.

e. **Routing**

i) **Methodology**  
Routing shall be accomplished using normal depth storage (normal depth channel routing).

ii) **HEC-1**  
The RS, RC, RX, and RY records shall be used for routing in the HEC-1 hydrology model.
B. FEMA Criteria

The developer or owner is required to meet all FEMA regulations. When a submittal to FEMA is required to adjust the FEMA Flood Boundary and Floodway Maps, the submittal must be submitted to and approved by the Town Manager or his designee prior to submitting to FEMA.

Conditional Letter of Map Revisions (CLOMRs) and Letter of Map Revisions (LOMRs) are required for any modifications to a floodplain or floodway. The developer will be required to pay all fees and costs associated with this review.

A Building Permit will not be issued by the Town until a copy of the FEMA approval letter for the LOMRs and CLOMRs is sent to the Town (if required for the site).

C. Town Design Criteria

1. General Requirements

   a. Property shall be developed so that the velocity and rate of runoff created by the development as it leaves the property does not exceed the velocity that existed under pre-developed conditions and rate of runoff that would exist under single family residential land use. This requirement may be satisfied by discharging to a Town approved regional facility. If the downstream system was not designed to carry the single family rate of runoff, detention to predeveloped runoff conditions is required.

   b. Mitigation measures to prevent increases in the velocity and rate of runoff include detention and/or retention facilities. Such mitigation measures shall be constructed in a manner that does not increase the tail water elevation at the upstream property line thereby decreasing the velocity or rate of runoff from the upstream property.

   c. Non-residential developments with a gross acreage of less than 1 1/2 acres are exempt from detention requirements.

   d. Single-family residential developments and small (1 1/2 acres or less) non-residential developments shall incorporate a minimum of three storm water quality best management practices into a storm water management plan, to be submitted with the development plan.

   e. Runoff calculations are required for development of property one acre (1 ac.) or greater, or any property that is platted as a part of an overall tract which is greater than one acre (1 ac.) in size, or where major streams are proposed to be modified.
f. The Rational Method may be used to calculate runoff for situations in which the drainage area is less than 200 acres. A unit hydrograph method is required for drainage areas 200 acres and larger.

g. Runoff calculations must be based on the proposed lot grading for the development. Runoff contributed from off-site sources must be considered in the calculations. All off-site drainage systems must be evaluated to ensure adequate capacity for the proposed post-development discharge. All undeveloped off-site property must be considered as single-family residential land use as future development will be required to detain to this runoff level.

h. All drainage systems shall provide for positive overflow at all low points. Positive overflow means that when the inlets do not function properly or when the design capacity of the conduit is exceeded, the excess flow can be conveyed overland.

i. Residential lots shall not be platted to contain floodplain. HOA lots with drainage easements shall be dedicated for containing the floodplain area.

j. No lot to lot drainage is allowed without drainage easements.

k. Property shall be developed in accordance with the Town’s Flood Damage Prevention Ordinance as amended.

2. Open Channels

a. Design Criteria
   Unless proven that there is no alternative for compliance with the drainage requirements, existing creeks, streams and natural channels shall not be channelized or improved such that there is degradation of the natural vegetation along the drainage way. Channelization of existing creeks, streams and natural channels shall require the approval of Town Council. The following criteria apply to approved channelization projects or other channels:

   i) General Criteria

   (a) In cases where the entire channel section is contained within the limits of the developer/owner property boundaries, the developer/owner shall:

   1. Provide for an improved stabilized channel cross-section which reduces all velocities to 6.0 fps or below for vegetated channels. The channel improvements must meet all requirements of these standards. Stabilization method must be designed by a Geo-technical Engineer, based on in-situ soil types.
2. For vegetated channel sections with overbank velocities exceeding 6 fps, grade control structures must be constructed within the channel and overbank areas to prevent erosion.

(b) In cases where the property boundary follows the centerline of the channel or incorporates only a portion of the channel cross-section, the developer/owner shall:

1. Determine the design section required to provide for an improved stabilized channel cross-section which reduces all velocities to 6.0 fps or below for vegetated channels. The design channel section must meet all requirements of this ordinance.

2. The design section may include vegetated channel sections with overbank velocities exceeding 6 fps, provided that grade control structures are included within the channel and overbank areas to prevent erosion. The developer/owner shall construct all improvements required on their property for the ultimate channel design.

3. If grade control structures are incorporated into the design, the developer/owner shall coordinate with adjacent owners in order to construct these features in their entirety at the time of the channel improvements.

4. The developer/owner shall provide for a drainage easement and access/maintenance easement consistent with the improvements provided.

(c) In cases where the developer/owner owns property adjacent to channel or floodplain areas but does not own a portion of the channel or floodplain area, the developer/owner shall (at the discretion of the Town Manager or his designee):

1. Determine the channel improvement configuration necessary to reduce all velocities to 6.0 fps or below for vegetated channels.

2. Shall provide a dedicated easement to the portion of this future improvement configuration, including necessary maintenance and access easement, which will include the developer/owner property.

ii) Methodology

(a) Models. Major channels or channels where backwater effects occur must be modeled using a standard backwater model. The U.S. Army Corps of Engineer’s Hydraulic Engineering Center’s model HEC-RAS is preferred. Other models that are
acceptable are HEC-2 and WSPRO. Note that the model must meet FEMA and standard engineering criteria (in some cases other models must be used).

(b) **Manning’s Equation.** For collector channels and swales, Manning’s equation can be used to determine water surface elevations and velocities since backwater effects are negligible. Manning’s equation is included in *Appendix B – Hydraulic Equations.*

### iii) Hydraulic Criteria

(a) **Freeboard.** Two feet of freeboard is required on all major channels, and one foot of freeboard on collector channels. In bends in the channel, the super elevation of the water must be estimated and added to the freeboard of the channel on the outside of the bend. An equation that can be used to determine the height of the super elevation of the water is included in *Appendix B – Hydraulic Equations.*

(b) **Flow Regime.** All channels are to be designed to have subcritical flow with a Froude Number less than 0.86. Hydraulic jumps are to be avoided. An exception to this is at bridges and culverts. When a hydraulic jump is predicted at a bridge or culvert the channel bottom and side slopes must be protected from erosion. In addition, the worst case water surface elevation must be used for determining floodway impacts.

(c) **Channel/Ditch Shape.** It is preferred that channel/ditches and swales have a trapezoidal section. The channel/ditches section should have adequate area to take care of the uncertainties in runoff estimates, changes in channel/ditch roughness coefficients, channel/ditch obstructions, and silt accumulations.

(d) **Bends.** All channel/ditch radii shall be a minimum of three times the top width of flow. An alignment with few horizontal curves or changes is desirable. If the natural channel/ditch radius is smaller than three times the top width, care should be provided in the design to protect any structures from channel/ditch migration or flooding.

(e) **Channel/ditch Side Slopes.** Unless shown to be feasible in a soils report sealed by a licensed Professional Engineer in the State of Texas, and approved by the Town Manager or his designee, improved channel/ditches shall have side slopes no steeper than:

1. 4 feet horizontal to 1-foot vertical for side slopes with grass or other ground cover
2. 1.5 feet horizontal to 1-foot vertical for side slopes protected by rock or gabions.

(f) **Channel/ditch Slopes and Velocities.**

1. **Minimum Slopes.** All vegetated channel and ditches shall have sufficient gradient to avoid ponding in low flow conditions. A minimum slope of 1 1/2% is required for all vegetated channel/ditches and swales except those used as part of a wetlands area. Channels, ditches and swales that have been stabilized by erosion matting or rip-rap shall have a minimum of grade of 1.0%.

2. **Maximum Velocity for Earthen Channel/ditch.** Channels and ditches are to be left in their natural state provided that the velocities are 6.0 or less. The maximum velocity allowed in vegetated channel, ditches and swales is 6 feet per second.

3. **Maximum Velocity for Improved Stabilized Channel/ditch.** The maximum velocity allowed in channels, ditches or swales that have been stabilized by erosion matting or rip-rap is 10 feet per second.

(g) **Channel/Ditch Vegetation.** The developer/owner shall use low maintenance vegetation cover, as approved by the Town Manager or his designee prior to planting. The selection of materials shall comply with the current ground cover listing for North Central Texas furnished through the Texas Agricultural Extension Service. The vegetation shall be established prior to final acceptance of the project.

(h) **Erosion Prevention and Channel/Ditch Maintenance.** All channel/ditch sections must consider and account for channel/ditch stabilization in their design. This requirement pertains to all sections whether they are left in their natural condition or are modified in any manner. The design of all drainage channel/ditch and swales shall assure adequate capacity and minimum maintenance to overcome the result of erosion, silting, sloughing of banks or similar occurrences. Drop structures, ditch checks or paved spillways may be required to control erosion that results from the high velocities of large volumes of water on steep grades.

Lined channel/ditches are strictly prohibited unless approved by Town Council. If the design velocity is greater than 6.0 feet per second, for earthen channel/ditches, measures shall be used to reduce velocities. Erosion countermeasures such as stabilization, rock walls, and gabions may be used upon approval of the Town Manager or his designee.
3. Bridges

a. Design Criteria

i) Freeboard. Two feet of freeboard is required between the 100-year water surface elevation and the low chord of the bridge. Exceptions to this requirement must be approved by the Town Manager or his designee in writing.

ii) Skew. The skew of the bridge piers and abutments shall be oriented as close to perpendicular to the flood direction of flow resulting in an angle of attack as close to 90 degrees as possible.

iii) Model. Bridges shall be designed using standard methods. If the HEC-2 backwater model is used, the normal bridge option shall be used for bridges without interior bents (interior piers). The special bridge option in HEC-2 shall be used for bridges with interior bents and for approved situations where pressure flow or weir flow occurs.

b. Erosion Protection

Stream stability shall be assessed when designing the abutments and interior bents of the bridge. Scour shall be accounted for in the design. Refer to references 3 and 4 for more information.

c. Floodplain Vegetation Connection

Bridges designed to cross floodplain shall incorporate a vegetation connection design to ensure the continuity of the natural habitat upstream and downstream of the structure.

4. Culverts

a. Design Criteria.

i) Freeboard. One foot of freeboard is required between the 100-year water surface elevation and the top of road elevation. Exceptions must be approved in writing by the Town Manager or his designee.

ii) Methods. Culverts must be designed using standard methods and engineering judgment. Standard charts are provided in Appendix D--Hydraulic Figures. In addition, numerous computer models are available. Culverts shall be designed in accordance with the latest edition of the Texas Department of Transportation (TxDOT) Hydraulic Design Manual. Standards of the Town of Flower Mound will take precedence over the TxDOT Manual in cases of conflict.
iii) **Skew.** The culvert shall be skewed such that impacts due to the flood and normal flow angles of attack on the structure are minimized.

iv) **Erosion.** Culverts can be designed in supercritical flow for hydraulic efficiency. However, the outlet of the culvert needs to be evaluated for erosion concerns, and the hydraulic jump must be forced at the outlet of the culvert, whether by design grades or an energy dissipater.

v) **Bends.** For long culverts, bends may sometimes be necessary. Culverts are recommended to be designed with maximum 15 degree bends on 50-foot intervals (Reference 5). When modeling culverts with bends, backwater models may need to be calibrated with other models to accurately estimate the losses in the culvert. The Town Manager or his designee can provide guidance on these methods as necessary.

vi) **Velocity.** The maximum velocity of flow through a culvert shall be 15 feet per second.

b. Erosion Protection

**Vegetation Connection.** Culverts designed to cross floodplains shall incorporate a vegetation connection design to ensure the continuity of the natural habitat upstream and downstream of the structure.

ii) **Energy Dissipaters:** Energy dissipaters shall be designed, where required, at exits to culverts, bridges, and in channel/ditches to convey flow safely. Design of these structures shall be according to standard methods (see References for additional sources). The Town discourages the use of energy dissipaters except where hydraulically constrained (sub critical flow throughout the drainage system is preferred).

c. Materials

i) **Floodplain Crossings.** Culverts crossing floodplain will have an arched top section and all exposed concrete surfaces (culvert, headwall, erosion protection, energy dissipaters, etc.) shall be clad in stone to complement the adjacent development; the height of the exposed face above the culvert top shall be no greater than one-half the height of the culvert opening.

5. **Detention Facilities**

a. General

i) Detention facilities may be used to prevent increases in the velocity and rate of runoff, for aesthetics and to control flooding, soil erosion, sedimentation and pollution at or downstream of the developed site.
ii) Detention facilities will be in the form of wet ponds in residential, mixed-use, office and retail land uses and either wet or dry facilities for commercial or industrial land uses. Institutional uses will adhere to the form of the pond for the predominant surrounding land use, but will at a minimum provide detention in the form of a dry pond.

iii) A maintenance plan, approved by the Town Manager or his designee, is required for all detention facilities and shall meet the standards given in this section.

iv) All detention facilities must be designed to drain within 36 hours of the storm event.

v) Detention in underground facilities is permitted.

vi) The use of regional detention facilities is encouraged.

b. Design Criteria

i) Design Storm Frequency

Detention facilities shall be designed for the 100-year event under fully developed conditions in the water shed.

ii) Maximum Discharge Rates

The discharge velocity and runoff rate for all developments shall be limited to the pre-developed velocity and single-family residential runoff rate with the following exceptions:

(a) The runoff from the site can be directly carried to a major channel/ditch or natural drainage way that has adequate capacity to carry the expected runoff to a major drainage outfall. This must be approved by the Town Manager or his designee.

(b) The runoff characteristics are such that the post project flows are at or below the flows prior to the development.

iii) Minimum Storage Volumes

A detention facility shall be designed to provide adequate storage to detain the required volume for all storm events up to a 100 year storm as indicated by a routing analysis. The post development runoff rate leaving the detention pond shall not exceed a single family residential discharge rate, for any storm event. If the downstream system was not designed to carry the single family rate of runoff, detention to pre-developed runoff conditions is required.

The equations and methodology to determine detention facility storage can be found in Appendix E-Detention Equations and Sample Calculations.
iv) Minimum Slope in Detention Facilities

A detention facility shall have enough gradient to ensure positive drainage to the outlet structures so as to avoid nuisance conditions such as standing water, odors, insects, and weeds. A minimum slope of 1.5% towards the outlet structure is required for all detention facilities.

v) Freeboard

All detention facilities shall have minimum of one foot (1 ft.) of freeboard above the water surface elevation of the required runoff storage.

vi) Sediment Control Facilities

Detention facilities used as a sediment control device shall meet the following requirements:

(a) The sediment control facility shall be designed with minimal velocities such that sediment is dropped and not picked up by flows at any time during the storm event.

(b) The basin shall be designed with adequate sediment storage area so that sediment removal is not required more than twice a year. Expected removal periods greater than twice a year must be specified in the maintenance plan and approved by the Town Manager or his designee.

(c) Sediment control facilities cannot be used to meet detention requirements unless the volume of sediment is included in the calculations for the detention basin design.

vii) Flood Peak Consideration

All detention facilities designs shall consider the timing of the flood peak in the main channel/ditch into which the detention facility drains. Delaying the peak from a site in lower portions of a watershed may result in a higher peak on the main channel/ditch.

c. Pond Form

i) Wet (retention) ponds are required for residential, mixed-use, retail and office land uses and shall be designed utilizing the following guidelines:

(a) Grouted stone or loose stone rip rap will be used to prevent mechanical erosion of the pond banks.

(b) Spray fountains, bubblers or stone water features shall be used for aeration.
(c) A normal water depth will be selected to discourage the growth of algae and other undesirable plant life as well as minimize risk to human safety.

(d) The detention volume will be in the form of vegetation established on a maximum 4:1 slope.

(e) Inflow and discharge structures will be hidden from view to the extent practical and clad with stone to complement the development.

(f) The use of perimeter fences is discouraged, but where required, shall be painted tubular steel with a height no greater than 42 inches.

ii) Dry detention ponds will only be allowed in commercial and industrial land uses, and shall be designed utilizing the following guidelines:

(a) Detention storage volume will be in the form of vegetation established on a maximum 4:1 slope; or a combination of stone walls and vegetated slopes, alternating 3'-high walls (maximum exposed face) and 4:1 slopes with a minimum rise of 4' per tier.

(b) Walls, if used, will be of a stone material to complement the development.

(c) A strip of permeable paving product may be used in the flow line of the pond to facilitate pond maintenance.

(d) Inflow and discharge structures will be hidden from view to the extent practical and clad with stone to complement the development.

(e) The use of perimeter fences is discouraged, but where required, shall be painted tubular steel with a height no greater than 42 inches.

(f) Screening of the pond, if desired, shall be achieved through the use of vegetated berms, landscaping, or a combination of both.

6. Storm Sewer Systems and Appurtenances

a. Storm Sewer Design

i) Storm Frequency
   The design storm event for storm sewers shall be a minimum of 100-year, plus 100-year positive overflow at inlets in street low points.
ii) Velocities and Grades in Storm Sewers

(a) **Minimum Velocity.** The minimum velocity in a storm sewer conduit shall be 2.5 feet per second.

(b) **Recommended Slopes.** The minimum slopes for various pipe sizes that will maintain the minimum velocity are given in Appendix C: – *Hydraulic Tables.*

(c) **Maximum Velocity**

1. **Storm Sewer.** The maximum velocity allowed within a storm sewer conduit is 12.5 feet per second.

2. **Outfall.** The maximum discharge velocities in the pipe shall also not exceed the permitted velocity of the receiving channel/ditch or conduit at the outfall to prevent erosive conditions. 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/ditch unless controlled by an appropriate energy dissipater (e.g. stilling basins, impact basins, riprap protection).

iii) Hydraulic Gradient of Storm Sewers

Conduits must be sized and slopes set such that runoff flows smoothly down the drainage system. To ensure this smooth passage, the hydraulic gradient must be at the proper elevations. In storm drain systems flowing full, all losses of energy through resistance with flow in pipes, by changes of momentum or by interference with flow patterns at junctions, must be accounted for by accumulative head losses along the system from its initial upstream inlet to its outlet. The purpose of accurate determinations of head losses at junctions is to include these values in a progressive calculation of the hydraulic gradient along the storm drain system. In this way, it is possible to determine the water surface elevation, which will exist at each structure. The hydraulic grade line shall represent the rate of energy loss through the storm drain system.

(a) The hydraulic grade line (hgl) shall be established for all storm drainage design and be included in the profile of the storm sewer. In open channel/ditches, the hgl is often controlled by the conditions of the storm sewer outfall. Therefore, the tail water elevation must be known. The hydraulic gradient is constructed downstream from the upstream end, taking into account all of the head losses that may occur along the
line. The gradient can then be adjusted to intersect at or above the gradient of the sewer outfall.

(b) The friction head loss shall be determined by direct application of Manning's Equation. Minor losses due to turbulence at structures shall be determined.

(c) The hydraulic grade line shall in no case be closer to the surface of the ground or street than 1 foot.

iv) Minor Losses

When establishing the hydraulic gradient of a storm sewer, minor head losses at points of turbulence shall be calculated and included in the computation of the hydraulic gradient. The following minor losses shall be accounted for in the storm sewer system design. Equations to determine these head losses are in Appendix B: Hydraulic Equations.

(a) Entrance Losses. Entrance losses to a closed storm sewer system from an open channel/ditch or lake shall be calculated. The resulting hydraulic grade line shall be compared to inlet control conditions for the storm sewer. The higher of the two values will be used as the controlling upstream hydraulic grade line.

(b) Expansion Losses. For locations within the storm sewer system where the pipe size increases, expansion head loss shall be calculated.

(c) Manhole and Bend Losses. Head losses associated with manholes used for pipe direction changes and bends in pipes of equal diameter shall be calculated.

(d) Junction Losses. Head losses associated with wye connections or manholes with branch laterals entering the main line shall be calculated.

b. Storm Sewer Laterals

Laterals for storm sewer systems shall be sized to control the flooding depth at the inlets. The depth shall not be closer to the surface of the ground or street than 1 foot. Calculation of the flooding depth shall be determined based on the addition of the velocity head of the lateral to the computed hydraulic grade line. See Appendix B: Hydraulic Equations for flooding depth equation for laterals.

c. Inlets

i) Inlet Placement
(a) Storm sewer inlets shall be built along paved streets at frequent enough intervals so that the 100-year storm does not exceed the top of curb.

(b) Inlets shall be generally placed upstream of intersections. Surface drainage will be allowed to cross intersections of residential streets. However, only one street shall be crossed with surface drainage at any one intersection and this street shall be the lower classified street. Surface drainage must be intercepted prior to an intersection of collector or arterial streets. No surface drainage will be permitted to cross a collector or arterial street. Valley gutters will not be permitted in collector or arterial streets.

(c) When an alley intersects a street, inlets shall be placed in the alley whenever flow down that alley would cause the capacity of the intersecting street to be exceeded.

(d) When a driveway intersects a street, inlets shall be placed in the driveway whenever flow down that driveway would cause the capacity of the intersecting street to be exceeded.

(e) All storm pipe shall be placed in the inside wall of any structure. No sonotube will be allowed.

ii) Capacity and Size

(a) The minimum inlet size shall be five feet (5'). Pre-cast inlets are not allowed.

(b) A maximum of 20 feet of inlets shall be at any location.

(c) Minimum lateral pipe size shall be eighteen-inches (18").

(d) Where laterals tie into trunk lines, place the laterals on a sixty-degree (60') angle with the trunk line. Inlet sizing charts are provided in Appendix D – Hydraulic Figures.

iii) Design

(a) All storm drain inlets shall be designed in accordance with the State of Texas Department of Highways and Public Transportation Bridge Division Hydraulic Manual.

(b) Recessed inlets shall be required on arterial and collector streets.

(c) A curb line inlet is to be used on all streets except arterial and collectors unless otherwise approved by the Town Manager or his designee.
(d) Slotted drains, grate inlets or combination inlets will not be allowed unless approval is obtained from the Town Manager or his designee.

iv) Labeling inlets constructed in conjunction with all new developments shall include signs that prohibit dumping in storm drains.

d. Outfall

i) Flow line Elevations
   The flow lines of storm sewer conduits that discharge into open channels/ditches shall be designed to prevent erosive conditions.

ii) Flumes. Flumes to bring the discharge down to the flow line of earthen creeks shall not be permitted.

iii) Drop Structures. Drop structures shall be allowed only upon the written approval of the Town Manager or his designee.

iv) Intersections with Creeks.

(a) The discharge pipe shall intersect minor creeks at an angle not to exceed 60 degrees.

(b) Pipes may intersect major creeks at an angle of 90 degrees.

(c) The Town Manager or his designee may require that pipes intersect major creeks at an angle not to exceed 60 degrees, when a 90-degree angle would result in an erosive condition.

e. Manholes

i) Location

(a) Manholes or junction boxes shall be located at intervals not to exceed 500 feet for pipe 48 inches in diameter or smaller and shall not be less than 4’ x 4’ interior dimension.

(b) Manholes shall preferably be located at street intersections, sewer junctions, changes of grade-changes of alignment, changes in pipe size, and as a designation of the point between public and private systems.
(c) Manholes for sewers greater than 48 inches in diameter shall be located at points where design indicates entrance into the sewer is desirable; however, in no case should the distance between openings or entrances be greater than 1,100 feet.

ii) **Shape**

Manholes shall be rectangular and as specified in the construction details of these standards.

**f. Materials**

In general, Storm water shall be carried in reinforced concrete pipe. Other materials are not permitted unless prior approval is obtained from the Town Manager or his designee. Appendix C-Hydraulic Tables shows recommended roughness coefficients for various types of conduits. If, in the opinion of the design engineer, other values for the roughness coefficient should be used, the different value can be used with the permission of the Town Manager or his designee in writing. Appropriate notes showing the roughness coefficient shall be provided on the plans. When rock rip-rap is utilized, a filter fabric underlayment shall be included in the design.

**g. Installation**

i) **Storm Sewer Installation**

(a) Construction shall begin at downstream end of project and continue upstream with the pipe bell facing upstream. No upstream piping shall be installed before downstream piping unless approved by the Town Manager or his designee. All sewer lines, whether main lines or service lines, crossing existing streets shall be placed by dry boring within an encasement. Open cut excavation will not be allowed to cross existing streets.

(b) Sonotubes to connect to structures in lieu of pipe shall not be allowed.

(c) The amount of trench excavation shall not exceed 200 (two hundred) feet from the end of the pipe laying operations, and no more than 300 (three hundred) feet of total open trench will be allowed. At the end of each work day, all trench excavation shall be backfilled to the end of the pipe laying operation. Barricades and lights will be required around any open trench left overnight.

(d) Approved manufactured plugs shall be installed at the open ends of the line at the end of each working day. All joints shall be assembled free of dirt and any foreign matter.

(e) Water jetting of storm sewer trenches shall not be allowed.
(f) Reference part B Section V for trench testing.

(g) Concrete work for all structures shall conform to all requirements of ACI 301, Standard Specification for Structural Concrete, published by the American Concrete Institute, except as modified herein.

ii) Extension Ring Installation

(a) The number of concrete extension ring sections shall be kept to a minimum (i.e. use 1-12" extension ring instead of 2-6" extension rings). [Total depth of manhole throat shall not exceed 24”]

(b) A 1" x 3-1/2" bitumastic gasket shall be used to seal the extension ring at both joints.

iii) Storm Drainage Wyes and Bends

All storm sewer wyes and bends shall be factory made unless authorized by the Town Manager or his designee or his representative.

7. Roads

a. Design Storm Frequencies

i) Urban Street Sections

Urban street sections must be designed to contain the 100-year storm event within the curb.

ii) Rural Street Sections

Rural street sections must be designed to contain the 100-year event within the right-of-way or parallel drainage easements.

a. Permissible Spread of Water

As water collects in the gutter and flows downhill, a certain portion of the roadway will be encroached by this flow. This spread of water on the roadway shall be limited to prevent the street from losing its effectiveness as a traffic carrier, which is an important concern in the case of emergency vehicles which may not be able to traverse an inundated roadway. The following table lists the allowable encroachment limits.
b. **Calculation of Flow in Gutters**

The flow of storm water in curb and gutter sections is classified as open channel/ditch flow. As such, the design calculations are based on a modified form of Manning’s equation. A modification to the hydraulic radius term is required because the hydraulic radius is not suitable for describing the cross section. The modified Manning’s equation to determine gutter discharge is included in *Appendix B: Hydraulic Equations*.

c. **Alleys**

i) The flow created by a 100-year storm shall be contained within the limits of pavement of all paved alleys.

ii) Alleys shall be super-elevated as required at corners and curves to insure that flow remains in the paved alley section.

iii) Curbs are required for at least ten-feet (10') on either side of an inlet in an alley and on the other side of the alley. Grate inlets are not allowed in alleys, accept upon approval of the Town Manager or his designee.

iv) Design flow in alleys shall be calculated by using the same equation to calculate gutter flow for a straight crown calculated in two triangular sections.
8. Easements
a. Easements Required for Open Channel/Ditches

i) General

(a) Drainage and/or floodway easements shall be provided for all open channel/ditches, creeks and flumes and may be dedicated to the Town of Flower Mound.

(b) No fences, buildings, or other structures which could impede flow shall be placed within these dedicated drainage easements, unless approved by the Town Manager or his designee.

ii) Minimum Widths

(a) The easement shall also include at least a 10-foot wide maintenance strip along both sides of the channel/ditch or, if the Town Manager or his designee so allows, at least a 20-foot wide maintenance strip along one side of the channel/ditch. Streets, alleys, bike paths, etc., alongside the channel/ditch can serve as all or part of the maintenance easement.

(b) Drainage easements for flumes shall be located with sufficient width to permit future maintenance accessibility, and in no case shall be less than 15 feet wide.

b. Easements for Enclosed Storm Sewers and Positive Overflow Areas

i) General

All storm sewer conduits to be dedicated to the Town of Flower Mound shall be located in an easement dedicated to the Town at the time of final platting of the property.

ii) Minimum Easement Widths

(a) The easement shall be at least 15 feet wide for storm sewers or meet the following requirements:

1. If the storm sewer line has 12-feet or less of fill between the pipe invert and finished grade, the outside diameter of the storm sewer line shall be located a minimum distance of 6-feet from the edge of the easement. If other utilities are located in the same easement, the outside diameter of the storm sewer line shall be located a minimum distance of 2-feet from the outside diameter of the other utilities.
2. If the storm sewer line has more than 12-feet of fill between the pipe invert and finished grade, the outside diameter of the storm line shall be located a minimum distance of 9-feet from the edge of the easement. If other utilities are located in the same easement, the outside diameter of the storm sewer line shall be located a minimum distance of 6-feet from the outside diameter of the other utilities.

(b) Special drainage easements on private property shall be a minimum of 10 feet wide or wider if the Town Manager or his designee requires it for maintenance or other purposes. No buildings or other structures and improvements shall be placed within these dedicated easements.

9. Utility Conflicts

With the hydraulic gradient established for a particular line, considerable latitude is available for the physical placement of the pipe flow line elevations. The inside top of the pipe must be on or below the hydraulic gradient, thus allowing the pipe to be lowered where necessary to maintain proper cover and to minimize grade conflicts with existing utilities.

a. General. In the design of a storm drainage system, the engineer is frequently confronted with the problem of intersections between the proposed storm drain and existing utilities such as water, gas and sanitary sewer lines.

b. Water Lines. All existing water lines in the immediate vicinity of the proposed storm drains shall be clearly indicated on both the plan and profile sheets. When design clearly indicates that an intersection of the storm drain line and the water main exists and the proposed storm drain cannot be economically relocated, then the existing water line shall be adjusted.

c. Gas Lines and Other Utilities. All existing gas lines and other utilities in the immediate vicinity of the proposed storm drain shall be clearly indicated on both the plan and profile sheets. Gas lines and other utilities, not controlled by elevation, shall be adjusted when the design clearly indicates that an intersection of the storm drain line and the utility exists and the proposed storm drain cannot be economically relocated.

10. Private Storm Water Systems


i) These standards apply to storm water systems that are located within private drainage easements or X-Lots that are dedicated to an HOA or BOA for maintenance.
ii) Private storm water systems may be in the form of an open channel (swale) or an underground storm sewer system.

iii) Private storm water systems must be designed to convey the 100 year storm event. A complete design must be provided in the construction plans and the design must include the HGL for all storm lines, laterals and inlets.

iv) The velocities in private storm drainage systems must conform to the Town’s design standards for public storm water systems.

v) Private storm water systems must be located in drainage easements that are a minimum of 10 feet wide. The drainage easements must be accessible for maintenance purposes from an X-Lot or the ROW.

vi) Private storm water systems will be inspected by the Town during installation and are subject to the Town’s 3% inspection fee.

b. Private Open Channel Storm Water Systems.

i) These standards do not apply to areas within the flood plain or to existing creeks or streams even if they are contained completely on a private lot. Creeks and streams must conform to the Town’s public design criteria.

ii) With the exception of freeboard, private open channel systems must meet all of the design requirements as public systems. This includes, but is not limited to, velocity, slopes, side slopes and bends. Private open channel systems require a minimum of 6-inches of freeboard.

iii) Private open channel systems may be vegetated or mechanically stabilized (lined). While lined channels are permitted, the material used to line the channel must be “enhanced” or decorative in nature; plain concrete or grouted rip-rap is not permitted.

iv) Drainage easements for open channel systems may only be crossed by tubular steel type fencing; no solid style fences are permitted across an open channel type system. The fence restriction must be noted on the subdivision plat as well as in the construction plans. There must be a minimum of 4” of clearance between the lowest part of a fence and the top of a private swale.

c. Private Storm Sewer Systems and Appurtenances

i) These standards do not apply to private streets; private streets must meet the same standards as public streets.

ii) Private storm sewer systems must meet the same velocity and hydraulic gradient requirements as public storm sewer systems.
iii) Curb inlet size shall be a minimum of three feet (3’).

iv) Drop inlets, including those in residential systems, shall be a minimum of 18”x18”. Inlets located in unpaved areas must have a minimum of a 2’ wide concrete apron.

v) Inlet structures must be constructed from concrete and may be either pre-cast or cast in place. Cast in place concrete inlets must be designed by an engineer and must have a detail provided in the construction plans. Grate type inlet structures are permitted provided that the grate is cast or ductile iron and can be removed to allow for access and cleaning.

vi) Storm pipes may be either corrugated smooth interior polypropylene (PP) pipe or reinforced concrete pipe (RCP) and must be a minimum of 12” in diameter. Private storm lines must be placed in embedment and must be installed in compliance with the manufacturer’s specification. All pipe intersections require a concrete junction box or a manufactured fitting.
D. **Other Criteria**

**General**

In addition to the requirements given in this standard, all other state and federal agency regulations, such as U.S. Army Corps of Engineers and TCEQ requirements must be met. In some cases where channel/ditch areas are disturbed, mitigation may also be required.

2. **Flood Damage Prevention Standards**

   The Flood Damage Prevention Ordinance No. 42-09 and any revisions there to are declared to be a part of this document and are included by reference.


   a. **General.** Proposed development shall include Best Management Practices (BMP’s) for watershed protection in order to mitigate the ill effects of increases in velocity, volume and pollution of surface run-off on downstream property owners and to protect the overall water quality of the Town as a result of rapid and intense urbanization.

   b. **Potential Best Management Practices.** Appendix H in the Drainage Appendices provides specifications for various BMP’s that may be used for watershed protection. Other BMP’s may be considered for approval by the Town Engineer if demonstrated to provide adequate watershed protection.

4. **Flood Damage Prevention Standards**

   The Flood Damage Prevention Ordinance No. 42-09 and any revisions there to are declared to be a part of this document and are included by reference.


   a. **General.** Proposed development shall include Best Management Practices (BMP’s) for watershed protection in order to mitigate the ill effects of increases in velocity, volume and pollution of surface run-off on downstream property owners and to protect the overall water quality of the Town as a result of rapid and intense urbanization.

   b. **Potential Best Management Practices.** Appendix H in the Drainage Appendices provides specifications for various BMP’s that may be used for watershed protection. Other BMP’s may be considered for approval by the Town Engineer if demonstrated to provide adequate watershed protection.
E. Drainage Report Submittal

General

A Drainage Report is required to be submitted to the Town Manager or his designee for all non-residential projects and projects greater than five acres when all of the hydrologic and hydraulic calculations are not provided on the Construction Plans.

1. Report Text

The report text shall include all methodology and references for all hydrologic and hydraulic calculations.

2. Report Appendix

The Appendix shall include all tables, figures, spreadsheets, and forms used in the analysis. The following are required appendices when the required information is not provided on the construction plans.

Appendix A - General

- Vicinity Map
- Site Boundary

Appendix B - Hydrology

- Pre-Development Drainage Area Map
- Post-Development Drainage Area Map
- Supporting Calculations and Figures for Existing Conditions.
- Supporting Calculations and Figures for Post Project Conditions Hydrology.

Appendix C - Existing Hydraulics

- 100-year floodplain delineation
- Model Input
- Model Output
- Profile Data
- Cross-Section Data
Appendix D - Post Project Hydraulics

- 100-year floodplain delineation
- Model Input
- Detention Pond Cross Sections
- Model Output
- Detention Pond Routing Analysis
- Profile Data
- Cross-Section Data
- Detention Pond Grading

Appendix E – FEMA Information / Other

- FEMA Forms (if required)
- Other

Additional Appendices may be required, depending on site models required by FEMA (Duplicate Effective Models, Corrected Effective Models, Post Project Models)
F. Plan Submittals for Drainage Facilities

Plans submitted to the Town Engineering Department shall include the following information for drainage facilities:

1. Drainage Area Map

   a. General

      i) A separate Drainage Area Map must be included in the plan set for existing conditions and for proposed conditions.

      ii) The Drainage Area Map must show the delineation of on-site and off-site basins, concentration points, and corresponding flows.

2. Drainage Calculations

   A table containing the existing, post-development, controlled release rates for each outfall location shall be provided for each storm event demonstrating that post-development runoff rates do not exceed pre-development runoff rates.

   a. Rational Method Calculations

      i) If the Rational Method is used to determine the flows, the plan sheet shall include a table that shows the following (or a note that references the Drainage Report name and date):

         (a) Drainage area designation (name) for each delineated basin
         (b) Basin area, A
         (c) Runoff coefficient, C₁ for fully developed basin
         (d) Time of Concentration, Tₖ, for fully developed basin
         (e) Rainfall Intensity, I, for fully developed basin
         (f) Discharge at each concentration point, Q, for fully developed basin
         (g) Discharge velocity at each concentration point, V, for fully developed basin

      ii) The plan sheet must show a concentration point for each basin and a flow arrow that depicts the direction of flow.

3. Culverts

   The following information shall be submitted in either the Construction Plans or the Drainage Report for all culverts:

   a. Profile of culvert
   b. Culvert size
   c. Required discharge
d. Design discharge  
e. Headwater elevation  
f. Tail water surface elevation  
g. Upstream invert  
h. Downstream invert  

4. Bridges  

The following information shall be submitted in either the Construction Plans or the Drainage Report for all bridges:  

a. Profile of bridge  
b. Bridge size  
c. Required discharge  
d. Design discharge  
e. Headwater elevation  
f. Tail water surface elevation  
g. Upstream invert  
h. Downstream invert  

5. Open Channel/Ditches  

The following information shall be provided for each section of open channel/ditch in the Construction Plans:  

a. Length of segment  
b. Geometry of segment  
c. Required discharge  
d. Design discharge  
e. Slope of hydraulic gradient  
f. Upstream hydraulic gradient elevation  
g. Downstream hydraulic gradient elevation  
h. Velocity  
i. Upstream invert  
j. Downstream invert  

6. Storm Sewer  

The following information shall be submitted for storm sewers on the plan set:  

a. Description of pipe location (i.e., from which manhole or inlet to which manhole or inlet)  
b. Length of pipe  
c. Required discharge  
   10. flow capacity (Qcap)  
   11. design flow (Q100)  
d. Pipe size  
e. Slope of hydraulic gradient
f. Upstream hydraulic gradient elevation  
g. Downstream hydraulic gradient elevation  
h. Inflow velocity  
i. Outflow velocity  
j. Sum of minor losses through the pipe section  
k. Hydraulic gradient elevation at design point  
l. Incoming pipe invert elevation at design point  
m. Outgoing pipe invert elevation at design point  
n. Profile of pipe  

7. Laterals

The following information shall be submitted for storm sewer laterals on the plan set:

a. Description of pipe location (to what inlet)  
b. Length of pipe  
c. Required discharge  
d. Pipe size  
e. Slope of hydraulic gradient  
f. Upstream hydraulic gradient elevation  
g. Downstream hydraulic gradient elevation  
h. Upstream pipe invert elevation of lateral  
i. Downstream pipe invert elevation of lateral  

8. Inlets

The following information shall be submitted for inlets on the plan set or with a note referencing the drainage report by name and date:

a. Description of inlet location (where along street)  
b. Length of inlet  
c. Discharge at inlet  
d. Flow captured by inlet  
e. Flow that passes by inlet  
f. Pipe sizes in and out of inlet  
g. Hydraulic gradient at inlet  
h. Elevation of gutter at inlet  
i. Elevation of top of curb  
j. Upstream pipe invert elevation at inlet  
k. Downstream pipe invert elevation at inlet  

9. Outfall
The following information shall be submitted for storm sewer outfalls on the plan set or with a note referencing the drainage report by name and date:

a. Description of outfall location
b. 100 year water surface elevation at outfall
c. Description of outfall stream dimensions
d. Maximum allowable velocity of outfall stream
e. Maximum capacity of outfall stream

9. Materials

All material to be used in construction of the storm drain system shall be specified either in the General Notes of the Construction Plans or on the plan and profile sheets.

10. Easements

a. All utility easements and right-of-way must be clearly identified on the plans. The width of all easements and right-of-way shall be clearly shown on the plans.

11. Detention Facilities

Construction Plans shall include the following information:

a. location
b. dimensions
c. depth
d. landscaping
e. grading
f. maintenance plan

12. Roadway Ditch Sections

The following information shall be provided for each roadway ditch section:

a. typical cross section
b. depth at each property line not to exceed 200 feet.
c. Slope (minimum 1%) 
d. culvert size for each lot
e. design discharge
f. 100-year water surface elevation

12. Erosion Control

The following information shall be provided in the construction plans:

a. typical erosion control devices
b. location of erosion control devices
c. maintenance plan
END OF SECTION
G. References


I. Drainage Appendices

Appendix A: Hydrology Tables and Figures

Design Storm Frequencies
Rational Method, Runoff Coefficient “C”
Tc Graph for Sheet Flow (Upland Method)
Tc Graph for Shallow Concentrated Flow
Flower Mound IDF Curve
SCS Curve Numbers
Appendix A

Design Storm Frequencies

The design storm frequencies for various drainage structures are given below:

<table>
<thead>
<tr>
<th>Drainage Facility</th>
<th>Minimum Design Recurrence</th>
</tr>
</thead>
<tbody>
<tr>
<td>Street - Urban sections</td>
<td>100-year within the curb</td>
</tr>
<tr>
<td>Street - Rural sections</td>
<td>5-year in the borrow ditch and 100-year within the right-of-way</td>
</tr>
<tr>
<td>Closed storm sewer systems</td>
<td>100-year, plus 100-year positive overflow at inlets in street low points.</td>
</tr>
<tr>
<td>Culverts and bridges</td>
<td>100-year minimum</td>
</tr>
<tr>
<td>Channels</td>
<td>100-year</td>
</tr>
</tbody>
</table>

The Town requires that sites be developed to the 100-year ultimate conditions. FEMA requires that models be submitted that represent existing or post-project conditions.
## Appendix A: Rational Method, Runoff Coefficient "C"

<table>
<thead>
<tr>
<th>Land Use</th>
<th>Runoff Coefficient</th>
<th>Zoning Districts</th>
</tr>
</thead>
<tbody>
<tr>
<td>Park areas – No developed land</td>
<td>0.30</td>
<td>A, WR, REC</td>
</tr>
<tr>
<td>Developed park sites</td>
<td>0.40</td>
<td>WR, REC</td>
</tr>
<tr>
<td>2 acre Residential or Single Family Estate</td>
<td>0.45</td>
<td>AG or SF-E</td>
</tr>
<tr>
<td>Single family residential (7,500 SF or larger)</td>
<td>0.55</td>
<td>SF-10, SF-15, SF-30,</td>
</tr>
<tr>
<td>Duplex</td>
<td>0.60</td>
<td>2F</td>
</tr>
<tr>
<td>Single family residential (below 7,500 SF)</td>
<td>0.65</td>
<td>CBD, MU, SF-5</td>
</tr>
<tr>
<td>Multi-family, Townhouse or SFA</td>
<td>0.70</td>
<td>MF, MH</td>
</tr>
<tr>
<td>Schools</td>
<td>0.70</td>
<td>SF-E, SF-30, SF-15, SF-10, SF-A, 2F, MF, MH, O, C-1, C-2, I-1</td>
</tr>
<tr>
<td>Churches</td>
<td>0.75</td>
<td>SF-E, SF-30, SF-15, SF-10, SF-A, 2F, MF, MH, O, R-1, R-2, I-1, I-2</td>
</tr>
<tr>
<td>Local Business</td>
<td>0.75</td>
<td>O, R-1</td>
</tr>
<tr>
<td>Central Business or Mixed Use</td>
<td>0.85</td>
<td>O, R-2, MU</td>
</tr>
<tr>
<td>Commercial</td>
<td>0.85</td>
<td>C-1, C-2</td>
</tr>
<tr>
<td>Industrial</td>
<td>0.85</td>
<td>I-1, I-2</td>
</tr>
<tr>
<td>ROW</td>
<td>0.95</td>
<td></td>
</tr>
</tbody>
</table>
A. Forest with heavy ground litter & hay meadow (overland flow)
B. Trash fallow or minimum tillage cultivation; contour or strip cropped & woodland (overland flow)
C. Short grass pasture (overland flow)
D. Cultivated, straight row (overland flow)
E. Nearly bare and untitled (overland flow); alluvial fans western mountain regions
F. Grassed waterway
G. Paved area (sheet flow); small upland gullies

Overland Flow Velocities for Upland Method of Estimating $T_c$
- Average velocities for estimating travel time for shallow concentrated flow.
### Appendix A: Flower Mound IDF Curve

Flower Mound, NOAA Atlas 14

<table>
<thead>
<tr>
<th>Storm Frequency (Years)</th>
<th>Rainfall Intensity (in/hr)</th>
<th>Storm Duration (min)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>5</td>
<td>10</td>
</tr>
<tr>
<td>1</td>
<td>4.99</td>
<td>4.00</td>
</tr>
<tr>
<td>2</td>
<td>5.80</td>
<td>4.65</td>
</tr>
<tr>
<td>5</td>
<td>7.14</td>
<td>5.73</td>
</tr>
<tr>
<td>10</td>
<td>8.24</td>
<td>6.61</td>
</tr>
<tr>
<td>25</td>
<td>9.73</td>
<td>7.81</td>
</tr>
<tr>
<td>50</td>
<td>10.90</td>
<td>8.73</td>
</tr>
<tr>
<td>100</td>
<td>12.00</td>
<td>9.63</td>
</tr>
</tbody>
</table>
# SCS Curve Numbers

## Table 2.2a.—Runoff curve numbers for urban areas

<table>
<thead>
<tr>
<th>Cover description</th>
<th>Average percent impervious area(^2)</th>
<th>Curve numbers for hydrologic soil group—</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>A</td>
</tr>
<tr>
<td><strong>Fully developed urban areas (vegetation established)</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Open space (lawns, parks, golf courses, cemeteries, etc.(^3))</td>
<td>Poor condition (grass cover &lt; 50%)</td>
<td>68</td>
</tr>
<tr>
<td></td>
<td>Fair condition (grass cover 50% to 75%)</td>
<td>49</td>
</tr>
<tr>
<td></td>
<td>Good condition (grass cover &gt; 75%)</td>
<td>39</td>
</tr>
<tr>
<td><strong>Impervious areas:</strong></td>
<td>Paved parking lots, roofs, driveways, etc. (excluding right-of-way).</td>
<td>98</td>
</tr>
<tr>
<td>Streets and roads:</td>
<td>Paved; curbs and storm sewers (excluding right-of-way).</td>
<td>98</td>
</tr>
<tr>
<td></td>
<td>Paved; open ditches (including right-of-way)</td>
<td>83</td>
</tr>
<tr>
<td></td>
<td>Gravel (including right-of-way)</td>
<td>76</td>
</tr>
<tr>
<td></td>
<td>Dirt (including right-of-way)</td>
<td>72</td>
</tr>
<tr>
<td><strong>Western desert urban areas:</strong></td>
<td>Natural desert landscaping (pervious areas only)(^4)</td>
<td>63</td>
</tr>
<tr>
<td></td>
<td>Artificial desert landscaping (impervious weed barrier, desert shrub with 1- to 2-inch sand or gravel mulch and basin borders)</td>
<td>96</td>
</tr>
<tr>
<td><strong>Urban districts:</strong></td>
<td>Commercial and business</td>
<td>85</td>
</tr>
<tr>
<td></td>
<td>Industrial</td>
<td>72</td>
</tr>
<tr>
<td><strong>Residential districts by average lot size:</strong></td>
<td>1/8 acre or less (town houses)</td>
<td>65</td>
</tr>
<tr>
<td></td>
<td>1/4 acre</td>
<td>38</td>
</tr>
<tr>
<td></td>
<td>1/3 acre</td>
<td>30</td>
</tr>
<tr>
<td></td>
<td>1/2 acre</td>
<td>25</td>
</tr>
<tr>
<td></td>
<td>1 acre</td>
<td>20</td>
</tr>
<tr>
<td></td>
<td>2 acres</td>
<td>12</td>
</tr>
<tr>
<td><strong>Developing urban areas:</strong></td>
<td>Newly graded areas (pervious areas only, no vegetation)(^5)</td>
<td>77</td>
</tr>
</tbody>
</table>

---

\(^1\)Average runoff condition, and \(I_n = 0.25\).

\(^2\)The average percent impervious area shown was used to develop the composite CN's. Other assumptions are as follows: impervious areas are directly connected to the drainage system, impervious areas have a CN of 98, and pervious areas are considered equivalent to open space in good hydrologic condition. CN's for other combinations of conditions may be computed using figure 2.3 or 2.4.

\(^3\)CN's shown are equivalent to those of pasture. Composite CN's may be computed for other combinations of open space cover type.

\(^4\)Composite CN's for natural desert landscaping should be computed using figures 2.3 or 2.4 based on the impervious area percentage (CN = 98) and the pervious area CN. The pervious area CN's are assumed equivalent to desert shrub in poor hydrologic condition.

\(^5\)Composite CN's to use for the design of temporary measures during grading and construction should be computed using figure 2.3 or 2.4, based on the degree of development (impervious area percentage) and the CN's for the newly graded pervious areas.
## SCS Curve Numbers

### Table 2-2b.—Runoff curve numbers for cultivated agricultural lands

<table>
<thead>
<tr>
<th>Cover type</th>
<th>Treatment</th>
<th>Hydrologic condition</th>
<th>A</th>
<th>B</th>
<th>C</th>
<th>D</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fallow</td>
<td>Bare soil</td>
<td>—</td>
<td>77</td>
<td>86</td>
<td>91</td>
<td>94</td>
</tr>
<tr>
<td></td>
<td>Crop residue cover (CR)</td>
<td>Poor</td>
<td>76</td>
<td>85</td>
<td>99</td>
<td>93</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Good</td>
<td>74</td>
<td>83</td>
<td>88</td>
<td>90</td>
</tr>
<tr>
<td>Row crops</td>
<td>Straight row (SR)</td>
<td>Poor</td>
<td>72</td>
<td>81</td>
<td>88</td>
<td>91</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Good</td>
<td>67</td>
<td>78</td>
<td>85</td>
<td>89</td>
</tr>
<tr>
<td></td>
<td>SR + CR</td>
<td>Poor</td>
<td>71</td>
<td>80</td>
<td>87</td>
<td>90</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Good</td>
<td>64</td>
<td>75</td>
<td>82</td>
<td>85</td>
</tr>
<tr>
<td></td>
<td>Contoured (C)</td>
<td>Poor</td>
<td>70</td>
<td>79</td>
<td>84</td>
<td>88</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Good</td>
<td>65</td>
<td>75</td>
<td>82</td>
<td>86</td>
</tr>
<tr>
<td></td>
<td>C + CR</td>
<td>Poor</td>
<td>69</td>
<td>78</td>
<td>83</td>
<td>87</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Good</td>
<td>64</td>
<td>74</td>
<td>81</td>
<td>85</td>
</tr>
<tr>
<td></td>
<td>Contoured &amp; terraced (C&amp;T)</td>
<td>Poor</td>
<td>66</td>
<td>74</td>
<td>80</td>
<td>82</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Good</td>
<td>62</td>
<td>71</td>
<td>78</td>
<td>81</td>
</tr>
<tr>
<td></td>
<td>C&amp;T + CR</td>
<td>Poor</td>
<td>65</td>
<td>73</td>
<td>79</td>
<td>81</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Good</td>
<td>61</td>
<td>70</td>
<td>77</td>
<td>80</td>
</tr>
<tr>
<td>Small grain</td>
<td>SR</td>
<td>Poor</td>
<td>65</td>
<td>76</td>
<td>84</td>
<td>88</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Good</td>
<td>63</td>
<td>75</td>
<td>83</td>
<td>87</td>
</tr>
<tr>
<td></td>
<td>SR + CR</td>
<td>Poor</td>
<td>64</td>
<td>73</td>
<td>83</td>
<td>86</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Good</td>
<td>60</td>
<td>72</td>
<td>80</td>
<td>84</td>
</tr>
<tr>
<td></td>
<td>C</td>
<td>Poor</td>
<td>63</td>
<td>74</td>
<td>82</td>
<td>85</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Good</td>
<td>61</td>
<td>73</td>
<td>81</td>
<td>84</td>
</tr>
<tr>
<td></td>
<td>C + CR</td>
<td>Poor</td>
<td>62</td>
<td>73</td>
<td>81</td>
<td>84</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Good</td>
<td>60</td>
<td>72</td>
<td>80</td>
<td>83</td>
</tr>
<tr>
<td></td>
<td>C&amp;T</td>
<td>Poor</td>
<td>61</td>
<td>72</td>
<td>79</td>
<td>82</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Good</td>
<td>59</td>
<td>70</td>
<td>78</td>
<td>81</td>
</tr>
<tr>
<td></td>
<td>C&amp;T + CR</td>
<td>Poor</td>
<td>60</td>
<td>71</td>
<td>78</td>
<td>81</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Good</td>
<td>58</td>
<td>69</td>
<td>77</td>
<td>80</td>
</tr>
<tr>
<td>Close-seeded</td>
<td>SR</td>
<td>Poor</td>
<td>66</td>
<td>77</td>
<td>85</td>
<td>89</td>
</tr>
<tr>
<td>or broadcast</td>
<td></td>
<td>Good</td>
<td>58</td>
<td>72</td>
<td>81</td>
<td>85</td>
</tr>
<tr>
<td>legumes or</td>
<td>C</td>
<td>Poor</td>
<td>64</td>
<td>73</td>
<td>83</td>
<td>85</td>
</tr>
<tr>
<td>rotation</td>
<td></td>
<td>Good</td>
<td>55</td>
<td>69</td>
<td>78</td>
<td>83</td>
</tr>
<tr>
<td>meadow</td>
<td>C&amp;T</td>
<td>Poor</td>
<td>63</td>
<td>73</td>
<td>80</td>
<td>83</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Good</td>
<td>51</td>
<td>67</td>
<td>76</td>
<td>80</td>
</tr>
</tbody>
</table>

1Average runoff condition, and \( I_w = 0.25 \).

2Crop residue cover applies only if residue is on at least 5% of the surface throughout the year.

3Hydrologic condition is based on combination of factors that affect infiltration and runoff, including (a) density and canopy of vegetative areas, (b) amount of year-round cover, (c) amount of grass or close-seeded legumes in rotations, (d) percent of residue cover on the land surface (good \( > 20 \% \)), and (e) degree of surface roughness.

Note: Factors impair infiltration and tend to increase runoff.

Good: Factors encourage average and better than average infiltration and tend to decrease runoff.

# SCS Curve Numbers

## Table 2-2c.—Runoff curve numbers for other agricultural lands¹

<table>
<thead>
<tr>
<th>Cover description</th>
<th>Cover type</th>
<th>Hydrologic condition</th>
<th>Curve numbers for hydrologic soil group—</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>A</td>
<td>B</td>
</tr>
<tr>
<td>Pasture, grassland, or range—continuous forage for grazing.²</td>
<td>Poor</td>
<td>68</td>
<td>79</td>
</tr>
<tr>
<td></td>
<td>Fair</td>
<td>49</td>
<td>69</td>
</tr>
<tr>
<td></td>
<td>Good</td>
<td>39</td>
<td>61</td>
</tr>
<tr>
<td>Meadow—continuous grass, protected from grazing and generally mowed for hay.</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>30</td>
<td>58</td>
</tr>
<tr>
<td>Brush—brush-weed-grass mixture with brush the major element.³</td>
<td>Poor</td>
<td>48</td>
<td>67</td>
</tr>
<tr>
<td></td>
<td>Fair</td>
<td>35</td>
<td>56</td>
</tr>
<tr>
<td></td>
<td>Good</td>
<td>30</td>
<td>48</td>
</tr>
<tr>
<td>Woods—grass combination (orchard or tree farm).⁴</td>
<td>Poor</td>
<td>57</td>
<td>73</td>
</tr>
<tr>
<td></td>
<td>Fair</td>
<td>43</td>
<td>65</td>
</tr>
<tr>
<td></td>
<td>Good</td>
<td>32</td>
<td>58</td>
</tr>
<tr>
<td>Woods.⁵</td>
<td>Poor</td>
<td>45</td>
<td>60</td>
</tr>
<tr>
<td></td>
<td>Fair</td>
<td>35</td>
<td>55</td>
</tr>
<tr>
<td></td>
<td>Good</td>
<td>30</td>
<td>50</td>
</tr>
<tr>
<td>Farmsteads—buildings, lanes, driveways, and surrounding lots.</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>59</td>
<td>74</td>
</tr>
</tbody>
</table>

¹Average runoff condition, and \( I_e = 0.25 \).

²Poor: < 50% ground cover or heavily grazed with no mulch.
Fair: 50 to 75% ground cover and not heavily grazed.
Good: > 75% ground cover and lightly or only occasionally grazed.

³Poor: < 30% ground cover.
Fair: 30 to 75% ground cover.
Good: > 75% ground cover.

⁴Actual curve number is less than 30; use CN = 30 for runoff computations.

⁵CN's shown were computed for areas with 50% woods and 50% grass (pasture) cover. Other combinations of conditions may be computed from the CN's for woods and pasture.

⁶Poor: Forest litter, small trees, and brush are destroyed by heavy grazing or regular burning.
Fair: Woods are grazed but not burned, and some forest litter covers the soil.
Good: Woods are protected from grazing, and litter and brush adequately cover the soil.
Appendix B: Hydraulic Equations

Manning’s & Continuity Equations
Superelevation of Water
Gutter Discharge
Weir Equations
Minor Headloss Equations
Lateral Flooding Depth
Froude Number

Note: The following equations will assist in calculations required in storm sewer design. In addition to these equations there are numerous computer programs.
that may be used to do the calculations required for designing storm sewer systems.

Appendix B

**Manning’s Equation**

Following is Manning’s Equation:

\[ Q = 1.486 \frac{A R^{2/3}}{S_f^{1/2}} n \]  (6)

where,
- \( Q \) = Total discharge (cfs)
- \( n \) = Coefficient of roughness (Manning’s n)
- \( A \) = Cross-section area of channel (sq.ft.)
- \( R \) = \( \frac{A}{P} \) = Hydraulic radius of channel (ft)
- \( P \) = Wetted perimeter (ft)
- \( S_f \) = Slope of the frictional gradient (ft/ft)

**Continuity Equation**

Following is the Continuity Equation:

\[ Q = V A \]  (7)

where,
- \( Q \) = Total discharge (cfs)
- \( V \) = Velocity (ft/s)
- \( A \) = Cross-section area of channel (sq.ft.)

**Combined Manning’s Equation and Continuity Equation**

The two equations can be combined to solve for Velocity:

\[ Q = 1.486 \frac{A R^{2/3}}{S_f^{1/2}} n \]  (8)
Appendix B

Height of Superelevation of Water

In a Bend

An equation that can be used to determine the height of the superelevation of the water is:

\[ H_s = \frac{c \cdot V^2 \cdot w}{g \cdot r} \]

where

- \( H_s \) = height of superelevation of the water around the bend (ft)
- \( c \) = coefficient, see table below
- \( V \) = velocity around the bend (ft/s)
- \( w \) = top width of flow (ft)
- \( g \) = gravity constant, (32.2 ft/s/s)
- \( r \) = radius of the bend (ft)

Table for Coefficient, \( c \)

<table>
<thead>
<tr>
<th>Channel Flow type</th>
<th>Cross-section Type of Curve</th>
<th>Value of ( c )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tranquil</td>
<td>Rectangular</td>
<td>Simple circular</td>
</tr>
<tr>
<td>Tranquil</td>
<td>Trapezoidal</td>
<td>Simple circular</td>
</tr>
<tr>
<td>Rapid</td>
<td>Rectangular</td>
<td>Simple circular</td>
</tr>
<tr>
<td>Rapid</td>
<td>Trapezoidal</td>
<td>Simple circular</td>
</tr>
<tr>
<td>Rapid</td>
<td>Rectangular</td>
<td>Spiral transitions</td>
</tr>
<tr>
<td>Rapid</td>
<td>Trapezoidal</td>
<td>Spiral transitions</td>
</tr>
<tr>
<td>Rapid</td>
<td>Rectangular</td>
<td>Spiral banked</td>
</tr>
</tbody>
</table>
Appendix B

Gutter Discharge

For streets with straight crowns, the gutter section will resemble a triangular channel. The equation to determine gutter discharge is as follows:

\[ Q = 0.56(Z/n)(S^{1/2})(Y^{8/3}) \]

Where:
- \( Q \) = gutter discharge (cfs)
- \( Z \) = reciprocal of the crown slope (ft/ft)
- \( S \) = longitudinal street or gutter slope (ft/ft)
- \( n \) = roughness coefficient
- \( Y \) = depth of flow at curb (ft)

Manning’s n for Gutters

The following table includes various values for the roughness coefficient ("n" values) for use in the gutter discharge equation.

<table>
<thead>
<tr>
<th>Type of Gutter</th>
<th>Manning’s n</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete gutter with asphalt pavement rough</td>
<td>0.015</td>
</tr>
<tr>
<td>Concrete pavement broom finish</td>
<td>0.016</td>
</tr>
</tbody>
</table>
Appendix B

Weir Equations

Broad Crested Weirs
Discharge over broad-crested weirs can be expressed by the equation:

\[ Q = C_B L H^{3/2} \]

Where:
- \( Q \) = discharge, cfs
- \( C_B \) = coefficient, below
- \( L \) = length of weir, ft
- \( H \) = height of water over weir, ft

Broad Crested Weir Coefficient \( C_B \)

<table>
<thead>
<tr>
<th>Measured head in feet, ( H )</th>
<th>0.5</th>
<th>0.75</th>
<th>1.0</th>
<th>1.5</th>
<th>2.0</th>
<th>2.5</th>
<th>3.0</th>
<th>4.0</th>
<th>5.0</th>
<th>10</th>
<th>15</th>
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<tbody>
<tr>
<td>0.2</td>
<td>2.80</td>
<td>2.75</td>
<td>2.69</td>
<td>2.62</td>
<td>2.54</td>
<td>2.48</td>
<td>2.44</td>
<td>2.38</td>
<td>2.34</td>
<td>2.49</td>
<td>2.68</td>
</tr>
<tr>
<td>0.4</td>
<td>2.92</td>
<td>2.80</td>
<td>2.72</td>
<td>2.64</td>
<td>2.61</td>
<td>2.60</td>
<td>2.58</td>
<td>2.54</td>
<td>2.50</td>
<td>2.56</td>
<td>2.70</td>
</tr>
<tr>
<td>0.6</td>
<td>3.08</td>
<td>2.89</td>
<td>2.72</td>
<td>2.64</td>
<td>2.61</td>
<td>2.60</td>
<td>2.68</td>
<td>2.69</td>
<td>2.70</td>
<td>2.70</td>
<td>2.07</td>
</tr>
<tr>
<td>0.8</td>
<td>3.30</td>
<td>3.04</td>
<td>2.85</td>
<td>2.68</td>
<td>2.60</td>
<td>2.67</td>
<td>2.68</td>
<td>2.68</td>
<td>2.68</td>
<td>2.69</td>
<td>2.64</td>
</tr>
<tr>
<td>1.0</td>
<td>3.32</td>
<td>3.14</td>
<td>2.98</td>
<td>2.75</td>
<td>2.66</td>
<td>2.64</td>
<td>2.65</td>
<td>2.67</td>
<td>2.68</td>
<td>2.68</td>
<td>2.63</td>
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<td>3.20</td>
<td>3.08</td>
<td>2.86</td>
<td>2.70</td>
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<td>2.65</td>
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<td>3.28</td>
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<td>2.65</td>
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<td>2.63</td>
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<td>1.8</td>
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<td>3.32</td>
<td>3.31</td>
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<td>3.32</td>
<td>3.31</td>
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<td>2.63</td>
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<td>3.32</td>
<td>3.32</td>
<td>3.32</td>
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<td>2.66</td>
<td>2.64</td>
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<td>3.32</td>
<td>3.32</td>
<td>3.32</td>
<td>3.32</td>
<td>3.32</td>
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<td>3.19</td>
<td>2.97</td>
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<td>2.64</td>
</tr>
<tr>
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<td>3.32</td>
<td>3.32</td>
<td>3.32</td>
<td>3.32</td>
<td>3.32</td>
<td>3.32</td>
<td>2.79</td>
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<td>3.32</td>
<td>3.32</td>
<td>3.32</td>
<td>3.32</td>
<td>3.32</td>
<td>3.32</td>
<td>3.32</td>
<td>3.32</td>
<td>2.88</td>
<td>2.64</td>
</tr>
</tbody>
</table>
Appendix B

V-Notch Weirs

Discharge over a V-notch weir can be expressed by the equation

\[ Q = \frac{C_d}{15} \left( \frac{8}{15} \right) (2g)^{1/2} \tan \Theta \frac{H^{5/2}}{2} \]

Where:

- \( Q \) = discharge, cfs
- \( \Theta \) = Vertex angle, degrees
- \( C_d \) = coefficient, see chart below
- \( g \) = gravitational constant (32.2 ft/s/s)
- \( L \) = length of weir, ft
- \( H \) = height of water over weir
Appendix B

**Minor Head Loss Equations**

**Entrance Losses**

\[
HL = K_e \left( V_1 \right)^2 \frac{2}{2g}
\]

Where:
- \(HL\) = Head loss (feet)
- \(V_1\) = Velocity in the downstream pipe (feet per second)
- \(K_e\) = Head loss coefficient (see appendix C, entrance loss coefficients)
- \(g\) = Gravitational constant (32.2 ft/s/s)

**Expansion Losses**

\[
HL = \left(1 - \frac{D_1}{D_2} \right)^2 \frac{V_1^2}{\left( \frac{D_2}{D_1} \right)^2} \frac{2}{2g}
\]

where:
- \(HL\) = Head loss (feet)
- \(V_1\) = Upstream velocity (feet per second)
- \(D_1\) = Upstream pipe diameter (feet)
- \(D_2\) = Downstream pipe diameter (feet)
- \(g\) = Gravitational constant (32.2 ft/s/s)
Appendix B

Bend Losses

Head losses associated with bends in pipes of equal diameter shall be calculated using:

\[
HL = \frac{K_b (V_2)^2}{2g}
\]

Where:
- \( HL \) = Head loss (feet)
- \( V_2 \) = Velocity in the downstream pipe (feet per second)
- \( K_b \) = Head loss coefficient (see appendix C, junction or structure loss coefficients)
- \( g \) = Gravitational constant (32.2 ft/s/s)

Junction Losses

\[
HL = \frac{(V_2)^2 - K_j (V_1)^2}{2g}
\]

Where:
- \( HL \) = Head loss (feet)
- \( V_1 \) = Velocity in the upstream pipe (feet per second)
- \( V_2 \) = Velocity in the downstream pipe (feet per second)
- \( K_j \) = Head loss coefficient (see appendix C, junction or structure loss coefficients)
- \( g \) = Gravitational constant (32.2 ft/s/s)
Appendix B

**Lateral Flooding Depth**

The flooding depth for lateral shall be based on the following equation:

\[
ELEV = HGL + \frac{V^2}{2g} + \frac{L \left( \frac{nV}{2g} \right)^2}{(1.486)R^{2/3}}
\]

Where:
- **ELEV** = Flooding depth for storm sewer lateral (ft)
- **HGL** = Hydraulic grade line elevation for inlet at downstream end of lateral (ft)
- **V** = Velocity in lateral (ft/s)
- **g** = Gravitational constant (32.2 ft/s)
- **n** = Roughness coefficient
- **R** = Hydraulic radius (ft)
- **L** = Length of lateral (ft)
Appendix B

**Froude Number**

The Froude Number can be calculated using the following equation

\[ Fr = \frac{V}{\sqrt{gy}} \]

Where:

- \( Fr \) = Froude Number
- \( V \) = Velocity (ft/s)
- \( g \) = Gravitational constant (32.2 ft/s/s)
- \( y \) = Depth of flow (ft)

The Froude Number is used to define the flow regime. The following table gives regime type based on Froude Number and range for design purposes.

<table>
<thead>
<tr>
<th>Flow Regime</th>
<th>Fr</th>
<th>Fr for design</th>
</tr>
</thead>
<tbody>
<tr>
<td>Subcritical</td>
<td>Fr&lt;1</td>
<td>Fr&lt;0.86</td>
</tr>
<tr>
<td>Critical</td>
<td>Fr=1</td>
<td>NA</td>
</tr>
<tr>
<td>Supercritical</td>
<td>Fr&gt;1</td>
<td>NA</td>
</tr>
</tbody>
</table>
Appendix C: Hydraulic Tables

Manning’s n
Minimum Slopes for Concrete Pipes
Maximum Permissible Velocities
in Conduits Flowing Full and Channels
Entrance Loss Coefficients
Junction or Structure Loss Coefficients
Headloss Coefficients due to Sudden Expansions and Contractions
Appendix C

Manning’s n for Channels

Reference Source: 2

<table>
<thead>
<tr>
<th>Channel Description</th>
<th>Coefficient of Roughness “n”</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>MINOR NATURAL STREAMS</strong></td>
<td></td>
</tr>
<tr>
<td><strong>Fairly regular section</strong></td>
<td></td>
</tr>
<tr>
<td>Some grass and weeds; little or no brush</td>
<td>0.030</td>
</tr>
<tr>
<td>Dense growth of weeds, depth of flow</td>
<td></td>
</tr>
<tr>
<td>materially greater than weed height</td>
<td>0.035</td>
</tr>
<tr>
<td>Some weeds, light brush on banks</td>
<td>0.035</td>
</tr>
<tr>
<td>Some weeds, heavy brush on banks</td>
<td>0.050</td>
</tr>
<tr>
<td>Some weeds, dense willows on banks</td>
<td>0.060</td>
</tr>
<tr>
<td>For trees within channels with branches,</td>
<td></td>
</tr>
<tr>
<td>submerged at high stage, increase all values above by</td>
<td>0.010</td>
</tr>
<tr>
<td><strong>Irregular section with pools, slight</strong></td>
<td></td>
</tr>
<tr>
<td>channel meander, use above, increase all values by</td>
<td>0.010</td>
</tr>
<tr>
<td><strong>Flood Plain - Pasture</strong></td>
<td></td>
</tr>
<tr>
<td>Short grass</td>
<td>0.030</td>
</tr>
<tr>
<td>Tall grass</td>
<td>0.035</td>
</tr>
<tr>
<td><strong>Flood Plain - Cultivated Areas</strong></td>
<td></td>
</tr>
<tr>
<td>No crop</td>
<td>0.030</td>
</tr>
<tr>
<td>Mature row crops</td>
<td>0.035</td>
</tr>
<tr>
<td>Mature field crops</td>
<td>0.040</td>
</tr>
<tr>
<td><strong>Flood Plain - Uncleared</strong></td>
<td></td>
</tr>
<tr>
<td>Heavy weeds, scattered</td>
<td>0.050</td>
</tr>
<tr>
<td>Wooded</td>
<td>0.120</td>
</tr>
</tbody>
</table>
Appendix C

Manning's n for Channels

<table>
<thead>
<tr>
<th>Channel Description</th>
<th>Coefficient</th>
</tr>
</thead>
<tbody>
<tr>
<td>MAJOR NATURAL STREAMS</td>
<td></td>
</tr>
<tr>
<td>Roughness coefficient is usually less for major streams than for minor streams of similar description on account of less effective resistance offered by irregular banks or vegetation on banks. Values of &quot;n&quot; for larger streams of mostly regular sections, with no boulders or brush may be in the range from 0.028 to 0.033.</td>
<td></td>
</tr>
<tr>
<td>UNLINED VEGETATED CHANNELS</td>
<td></td>
</tr>
<tr>
<td>Clays (Bermuda Grass)</td>
<td>0.035</td>
</tr>
<tr>
<td>Sandy and Silty Soils (Bermuda Grass)</td>
<td>0.035</td>
</tr>
<tr>
<td>UNLINED NON-VEGETATED CHANNELS</td>
<td></td>
</tr>
<tr>
<td>Sandy Soils</td>
<td>0.030</td>
</tr>
<tr>
<td>Silts</td>
<td>0.030</td>
</tr>
<tr>
<td>Sandy Silts</td>
<td>0.030</td>
</tr>
<tr>
<td>Clays</td>
<td>0.030</td>
</tr>
<tr>
<td>Coarse Gravels</td>
<td>0.030</td>
</tr>
<tr>
<td>Shale</td>
<td>0.030</td>
</tr>
<tr>
<td>Rock</td>
<td>0.025</td>
</tr>
<tr>
<td>LINED CHANNELS</td>
<td></td>
</tr>
<tr>
<td>Neat Concrete</td>
<td>0.015</td>
</tr>
<tr>
<td>Riprap (Broken Concrete &amp; Rubble)</td>
<td>0.030</td>
</tr>
</tbody>
</table>
## Appendix C

### Manning’s n for Conduits

<table>
<thead>
<tr>
<th>Coefficient of Roughness “n”</th>
<th>Conduit Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.013</td>
<td>Concrete Pipe Storm Sewer</td>
</tr>
<tr>
<td>0.013</td>
<td>Concrete Pipe Culverts</td>
</tr>
<tr>
<td>0.013</td>
<td>Monolithic Concrete Culverts</td>
</tr>
<tr>
<td>0.024</td>
<td>Corrugated Metal</td>
</tr>
<tr>
<td>0.013</td>
<td>Corrugated Metal Pipe (Smooth Lined)</td>
</tr>
</tbody>
</table>
## Appendix C

### MINIMUM SLOPES FOR CONCRETE PIPES
(to produce a velocity of 2.5 fps or greater)

<table>
<thead>
<tr>
<th>Pipe Diameter (inches)</th>
<th>Slope (Feet/100 Feet)</th>
<th>Pipe Diameter (inches)</th>
<th>Slope (Feet/100 Feet)</th>
</tr>
</thead>
<tbody>
<tr>
<td>18</td>
<td>.180</td>
<td>42</td>
<td>.056</td>
</tr>
<tr>
<td>21</td>
<td>.150</td>
<td>45</td>
<td>.052</td>
</tr>
<tr>
<td>24</td>
<td>.120</td>
<td>48</td>
<td>.048</td>
</tr>
<tr>
<td>27</td>
<td>.110</td>
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<td>.045</td>
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<td>30</td>
<td>.090</td>
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<td>.041</td>
</tr>
<tr>
<td>33</td>
<td>.080</td>
<td>60</td>
<td>.036</td>
</tr>
<tr>
<td>36</td>
<td>.070</td>
<td>66</td>
<td>.032</td>
</tr>
<tr>
<td>39</td>
<td>.062</td>
<td>72</td>
<td>.028</td>
</tr>
</tbody>
</table>
Appendix C

MAXIMUM VELOCITIES IN CONDUITS FLOWING FULL AND CHANNELS

<table>
<thead>
<tr>
<th></th>
<th>Maximum Flow Velocity (fps)</th>
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<tbody>
<tr>
<td>Culverts</td>
<td>15</td>
</tr>
<tr>
<td>Inlet Laterals</td>
<td>10</td>
</tr>
<tr>
<td>Storm Sewers</td>
<td>12.5</td>
</tr>
<tr>
<td>Earthen Channels</td>
<td>6</td>
</tr>
<tr>
<td>Lined Channels</td>
<td>15</td>
</tr>
</tbody>
</table>
Appendix C

Entrance Loss Coefficients

Entrance head loss $HL = K_e \frac{V_1^2}{2g}$

<table>
<thead>
<tr>
<th>Type of Structure and Design of Entrance</th>
<th>Coefficient $K_e$</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Concrete Pipe</strong></td>
<td></td>
</tr>
<tr>
<td>Projecting from fill, socket end (groove-end)</td>
<td>0.2</td>
</tr>
<tr>
<td>Projecting from fill, square cut end</td>
<td>0.5</td>
</tr>
<tr>
<td>Headwall or headwall and wingwalls</td>
<td></td>
</tr>
<tr>
<td>Socket end of pipe (groove-end)</td>
<td>0.2</td>
</tr>
<tr>
<td>Square-edge</td>
<td>0.5</td>
</tr>
<tr>
<td>Rounded (radius = $1/12D$)</td>
<td>0.2</td>
</tr>
<tr>
<td>Mitered to conform to fill slope</td>
<td>0.7</td>
</tr>
<tr>
<td>End-section conforming to fill slope</td>
<td>0.5</td>
</tr>
<tr>
<td>Beveled edges, $33^\circ$ to $45^\circ$</td>
<td>0.2</td>
</tr>
<tr>
<td>Side- or slope-tapered inlet</td>
<td>0.2</td>
</tr>
<tr>
<td><strong>Pipe or Pipe-Arch Corrugated Metal</strong></td>
<td></td>
</tr>
<tr>
<td>Projecting from fill (no headwall)</td>
<td>0.9</td>
</tr>
<tr>
<td>Headwall or headwall and wingwalls</td>
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</tr>
<tr>
<td>Square-edged</td>
<td>0.5</td>
</tr>
<tr>
<td>Mitered to conform to fill slope, paved or unpaved slope</td>
<td>0.7</td>
</tr>
<tr>
<td>End-section conforming to fill slope</td>
<td>0.5</td>
</tr>
<tr>
<td>Beveled edges, $33^\circ$ to $45^\circ$</td>
<td>0.2</td>
</tr>
<tr>
<td>Side- or slope-tapered inlet</td>
<td>0.2</td>
</tr>
<tr>
<td><strong>Box, Reinforced Concrete</strong></td>
<td></td>
</tr>
<tr>
<td>Headwall parallel to embankment (no wingwalls)</td>
<td>0.5</td>
</tr>
<tr>
<td>Square-edged on 3 edges</td>
<td></td>
</tr>
<tr>
<td>Rounded on 3 edges to radius of $1/12$ barrel dimension or beveled on 3 sides</td>
<td>0.2</td>
</tr>
<tr>
<td>Wingwalls at $30^\circ$ to $75^\circ$ to barrel</td>
<td>0.2</td>
</tr>
<tr>
<td>Square-edged at crown</td>
<td>0.4</td>
</tr>
<tr>
<td>Crown edge rounded to radius of $1/12$ barrel dimension, or beveled top edge</td>
<td>0.2</td>
</tr>
<tr>
<td>Wingwall at $10^\circ$ to $25^\circ$ to barrel</td>
<td></td>
</tr>
<tr>
<td>Square-edged at crown</td>
<td>0.5</td>
</tr>
<tr>
<td>Wingwall parallel (extension of sides)</td>
<td></td>
</tr>
<tr>
<td>Square-edged at crown</td>
<td>0.7</td>
</tr>
<tr>
<td>Side- or slope-tapered inlet</td>
<td>0.2</td>
</tr>
</tbody>
</table>
### Appendix C

#### Junction or Structure Loss Coefficients

<table>
<thead>
<tr>
<th>Description of Condition</th>
<th>$K_j$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Inlet on Main Line</td>
<td>0.50</td>
</tr>
<tr>
<td>Inlet on Main Line with Lateral</td>
<td>0.25</td>
</tr>
<tr>
<td>Manhole on Main Line with 22-1/2&quot; Lateral</td>
<td>0.75</td>
</tr>
<tr>
<td>Manhole on Main Line with 45° Lateral</td>
<td>0.50</td>
</tr>
<tr>
<td>Manhole on Main Line with 60° Lateral</td>
<td>0.35</td>
</tr>
<tr>
<td>Manhole on Main Line with 90° Lateral</td>
<td>0.25</td>
</tr>
<tr>
<td>45° Wye Connection or Cut-in</td>
<td>0.75</td>
</tr>
<tr>
<td>60° Wye Connection or Cut-in</td>
<td>0.70</td>
</tr>
<tr>
<td>Inlet or Manhole at Beginning of Line</td>
<td>1.25</td>
</tr>
</tbody>
</table>

The following head loss coefficients for bends in storm sewers are for pipes with diameters of 48-inches or less. For storm sewers with diameters greater than 48-inches the momentum equation should be used to determine headloss.

<table>
<thead>
<tr>
<th>Conduit on Curves</th>
<th>Radius of Pipe Bend</th>
<th>$K_b$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>90°</td>
<td>60°</td>
</tr>
<tr>
<td>Pipe radius = diameter</td>
<td>0.50</td>
<td>0.43</td>
</tr>
<tr>
<td>Pipe radius = 2 to 8 diameter</td>
<td>0.25</td>
<td>0.21</td>
</tr>
<tr>
<td>Pipe radius = 8 to 20 diameter</td>
<td>0.40</td>
<td>0.34</td>
</tr>
</tbody>
</table>
Appendix C

Head Loss Coefficients Due To Sudden Enlargements and Contractions

<table>
<thead>
<tr>
<th>Contractions</th>
<th>$D_2/D_1^*$</th>
<th>$K_j$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.2</td>
<td></td>
<td>0.08</td>
</tr>
<tr>
<td>1.4</td>
<td></td>
<td>0.18</td>
</tr>
<tr>
<td>1.6</td>
<td></td>
<td>0.25</td>
</tr>
<tr>
<td>1.8</td>
<td></td>
<td>0.33</td>
</tr>
<tr>
<td>2.0</td>
<td></td>
<td>0.36</td>
</tr>
<tr>
<td>2.5</td>
<td></td>
<td>0.40</td>
</tr>
<tr>
<td>3.0</td>
<td></td>
<td>0.42</td>
</tr>
<tr>
<td>4.0</td>
<td></td>
<td>0.44</td>
</tr>
<tr>
<td>5.0</td>
<td></td>
<td>0.45</td>
</tr>
<tr>
<td>10.0</td>
<td></td>
<td>0.46</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.47</td>
</tr>
</tbody>
</table>

*D_2/D_1 = Ratio of larger to smaller diameter.

The values of the coefficient "$K_j$" for determining the loss of head due to obstructions in pipes are shown below and the coefficient is used in the following equation to calculate the head loss at the obstruction:

$$HL = \frac{K_j \sqrt{V^2}}{2g}$$

### Head Loss Coefficients Due To Obstructions

<table>
<thead>
<tr>
<th>$A/A_o^*$</th>
<th>$K_j$</th>
<th>$A/A_o^*$</th>
<th>$K_j$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.0</td>
<td>0.10</td>
<td>3.0</td>
<td>15.0</td>
</tr>
<tr>
<td>1.0</td>
<td>0.21</td>
<td>4.0</td>
<td>27.3</td>
</tr>
<tr>
<td>1.2</td>
<td>0.50</td>
<td>5.0</td>
<td>42.0</td>
</tr>
<tr>
<td>1.4</td>
<td>1.15</td>
<td>6.0</td>
<td>57.0</td>
</tr>
<tr>
<td>1.6</td>
<td>2.40</td>
<td>7.0</td>
<td>72.5</td>
</tr>
<tr>
<td>1.8</td>
<td>4.00</td>
<td>8.0</td>
<td>88.0</td>
</tr>
<tr>
<td>2.0</td>
<td>5.55</td>
<td>9.0</td>
<td>104.0</td>
</tr>
<tr>
<td>2.2</td>
<td>7.05</td>
<td>10.0</td>
<td>121.0</td>
</tr>
<tr>
<td>2.5</td>
<td>9.70</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

*A/A_o = Ratio of area of pipe to area of opening at obstruction.*
Appendix D: Hydraulic Figures

Inlet Capacity for Low Point Inlets
Inlet Capacity for On-Grade Inlets
Ratio of Intercepted to Total Flow Inlets on Grade
Culvert Design Charts
INLET CAPACITY
FOR
LOW POINT INLETS
Figure 3
Figure 10

Bridge Division Hydraulic Manual 12-85
CRITICAL FLOW
FOR BOX CULVERTS

\( n = 0.012 \)

BRIDGE DIVISION HYDRAULIC MANUAL 12-85
DHT
UNIFORM FLOW
FOR
BOX CULVERTS
n = 0.012

3 Sides Wetted

NOMOGRAPH B

BRIDGE DIVISION HYDRAULIC MANUAL 12-85
EQUATION FOR SQUARE BOX:

\[ H = \left[ \frac{1555 (1 - C_e)}{0.5} + \frac{287.64 n^2 L^2}{0.144} \right]^{1/2} \]

\( H \) = Head in feet
\( C_e \) = Entrance loss coefficient
\( D \) = Height, also span, of box in feet
\( n \) = Manning's roughness coefficient
\( L \) = Length of culvert in feet
\( Q \) = Design discharge rate in cfs

HEAD FOR CONCRETE BOX CULVERTS
FLOWING FULL
\( n = 0.012 \)

FHWA

BRIDGE DIVISION HYDRAULIC MANUAL

12-85
EXAMPLE

\[ Q = 71.5 \]

\[ \frac{a}{b} = 1.66 \]

\[ \frac{H}{D} = 8.3 \text{ FT.} \]

THROAT CONTROL OPERATION OF A TAPERED INLET OCCURS ONLY FOR DIMENSIONS AND DISCHARGE RATES FOR WHICH \( H = U S_0 \) OR \( H = \text{FALL} \) EXCEEDS \( HW \) FOR THE FACE SECTION. USE THE SPECIAL INSTRUCTIONS FOR DESIGN OF TAPERED INLETS.

TAPERED-SIDE INLETS ON BOX CULVERTS
EXAMPLE

GIVEN:  \( S = 0.02 \)
\( Q = 20 \text{ cfs} \)
\( D = 36^\circ \) (CONCRETE)

FIND:  \( d/D = \frac{d}{D} \)

SOLUTION
\[ d/D = 0.30 \]
\[ d = 0.30 \times 3' = 0.9' \]

DHT
UNIFORM FLOW
FOR
PIPE CULVERTS

BRIDGE DIVISION HYDRAULIC MANUAL 12-6
DHT
CRITICAL DEPTH OF FLOW FOR CIRCULAR CONDUITS

Nomograph G

BRIDGE DIVISION HYDRAULIC MANUAL 12-6
### Example

5-42 inches (3.3 feet)
7-120 cfs

<table>
<thead>
<tr>
<th>H</th>
<th>W</th>
<th>D</th>
<th>Scale</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>6.6</td>
<td>10</td>
<td>6.6</td>
</tr>
<tr>
<td>5</td>
<td>6.4</td>
<td>10</td>
<td>6.4</td>
</tr>
<tr>
<td>4</td>
<td>4.3</td>
<td>10</td>
<td>4.3</td>
</tr>
<tr>
<td>3</td>
<td>3.2</td>
<td>10</td>
<td>3.2</td>
</tr>
<tr>
<td>2</td>
<td>2.1</td>
<td>10</td>
<td>2.1</td>
</tr>
<tr>
<td>1</td>
<td>1.0</td>
<td>10</td>
<td>1.0</td>
</tr>
</tbody>
</table>

- H in feet
- W in feet
- D in inches
- Scale

### Entrance Type

1. Square edge with headwall
2. Groove and with headwall
3. Groove and projecting

---

**Headwater Depth for Concrete Pipe Culverts with Inlet Control**

**Nomograph 1**

*To use scale (2) or (3) project horizontally to scale (1), then use straight inclined line through D and Q scales, or reverse as illustrated.*
Appendix E: Detention Equations and Sample Calculations

Detention

Sample Calculations
Example 1

Sample Watershed

- Basin 1 is 8 acres
- Basin 2 is 18 acres
- Basin 3 is 4 acres
- Total watershed is 30 acres

Existing conditions of the land (pre-developed conditions)
- Basin 1 is Forested
- Basin 2 is Woodland
- Basin 3 is Grass

Developed conditions of the land
- Basin 1 is same (not developed)
- Basin 2 is same (not developed)
- Basin 3 is commercial

Determine Time of Concentration for (Undeveloped and Developed Conditions)

<table>
<thead>
<tr>
<th>SHEET 1/20</th>
<th>Vel from Length Elev.</th>
<th>Slope</th>
<th>Vel from Length Elev.</th>
<th>Slope</th>
<th>Avg Vel from Length Elev.</th>
<th>Slope</th>
<th>Segment 1</th>
<th>Vel from Length Elev.</th>
<th>Slope</th>
<th>Avg Vel from Length Elev.</th>
<th>Slope</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 UD</td>
<td>200 1115 1090 7.5/5</td>
<td>0.69</td>
<td>600 1160 1070 7.5/4</td>
<td>0.69</td>
<td>36 600 1160 1070 7.5/4</td>
<td>0.69</td>
<td></td>
<td>0.046</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2 UD</td>
<td>300 1095 1080 5.0/11</td>
<td>0.76</td>
<td>900 1080 1045 5.0/11</td>
<td>0.76</td>
<td>32 900 1080 1045 5.0/11</td>
<td>0.76</td>
<td></td>
<td>0.078</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3 UD</td>
<td>200 1060 1050 3.6/13</td>
<td>0.69</td>
<td>400 1050 1040 3.6/13</td>
<td>0.69</td>
<td>28 400 1050 1040 3.6/13</td>
<td>0.69</td>
<td></td>
<td>0.040</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1 D</td>
<td>200 1115 1100 7.5/5</td>
<td>0.69</td>
<td>600 1160 1070 7.5/4</td>
<td>0.69</td>
<td>36 600 1160 1070 7.5/4</td>
<td>0.69</td>
<td></td>
<td>0.046</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2 D</td>
<td>300 1095 1080 5.0/11</td>
<td>0.76</td>
<td>900 1080 1045 5.0/11</td>
<td>0.76</td>
<td>32 900 1080 1045 5.0/11</td>
<td>0.76</td>
<td></td>
<td>0.078</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3 D</td>
<td>280 1060 1050 3.6/13</td>
<td>0.69</td>
<td>400 1050 1040 3.6/13</td>
<td>0.69</td>
<td>28 400 1050 1040 3.6/13</td>
<td>0.69</td>
<td></td>
<td>0.040</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Note: Coding two "segments" of shallow concentrated flow may be appropriate in cases where there is a distinct slope change.

Example for SCS Methodology

Determine Time of Concentration for Undeveloped and Developed Conditions

1. UD = Undeveloped
2. D = Developed

Time of Concentration Graph

Flow

TOTAL

Unit:

Time (hr) Slope

Legend: Open channels

Example 1

Note: The lengths and elevations may need to be changed based on site conditions.
EXAMPLE 1

Discharge by Rational Method

**Given:**
Determined from site plan:
- Basin 1 is 8 acres
- Basin 2 is 18 acres
- Basin 3 is 4 acres
- Total watershed is 30 acres

Existing conditions of the land
(pre-developed conditions):
- Basin 1 is Forested
- Basin 2 is Woodland
- Basin 3 is Grass

Developed conditions of the land:
- Basin 1 is same (not developed)
- Basin 2 is same (not developed)
- Basin 3 is commercial

<table>
<thead>
<tr>
<th>BASIN</th>
<th>calculated</th>
<th>allowed</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>C</td>
<td>Tc (min)</td>
</tr>
<tr>
<td>1</td>
<td>pre-developed</td>
<td>0.30</td>
</tr>
<tr>
<td>2</td>
<td>pre-developed</td>
<td>0.30</td>
</tr>
<tr>
<td>3</td>
<td>pre-developed</td>
<td>0.30</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

| 1     | not developed | 0.30   | 7.6      | 15.0      | 9.6    | 8.0    | 23.0 upper watershed |
| 2     | not developed | 0.30   | 9.2      | 15.0      | 9.6    | 18.0   | 51.8 upper watershed |
| 3     | developed     | 0.85   | 3.1      | 10.0      | 11.8   | 4.0    | 39.4 site |
| **Total** | | | | | | **30.0** |

Change in flows: 27.9

There is a 28 cfs increase in discharge after the property is developed.
SAMPLE DETENTION CALCULATIONS

Given:
Existing site land use is non developed
Proposed site development is commercial
Area of the site is 4 acres (onsite watershed)
Total watershed for the site is 30 acres (onsite and offsite)
Detention basin must be designed for the 100-year, 2-hour event

Step 1: Determine parameters for the site:
For the 100-year, 2-hour storm, I = 2.6 in/hr
see Town of Flower Mound IDF Curve in Appendix A.
For the site, C = 0.85
see Runoff Coefficient Table in Appendix A

Step 2: Compute Q

<table>
<thead>
<tr>
<th></th>
<th>I</th>
<th>A</th>
<th>Q = CIA</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>(in/hr)</td>
<td>(ac)</td>
<td>(cfs)</td>
</tr>
<tr>
<td>100-yr, 2hr</td>
<td>0.85</td>
<td>2.6</td>
<td>8.8</td>
</tr>
<tr>
<td>50-yr, 2hr</td>
<td>0.85</td>
<td>2.3</td>
<td>7.8</td>
</tr>
<tr>
<td>25-yr, 2hr</td>
<td>0.85</td>
<td>2.0</td>
<td>6.8</td>
</tr>
<tr>
<td>10-yr, 2 hr</td>
<td>0.85</td>
<td>1.8</td>
<td>6.1</td>
</tr>
</tbody>
</table>

Step 3: Compute Volume required for the detention basin:

\[ V = Q \times 7200 \]

<table>
<thead>
<tr>
<th>Q (cfs)</th>
<th>V (acre-ft)</th>
<th>Site area needed for detention pond</th>
</tr>
</thead>
<tbody>
<tr>
<td>100-yr, 2hr</td>
<td>8.8</td>
<td>63,648</td>
</tr>
<tr>
<td>50-yr, 2hr</td>
<td>7.8</td>
<td>56,304</td>
</tr>
<tr>
<td>25-yr, 2hr</td>
<td>6.8</td>
<td>48,960</td>
</tr>
<tr>
<td>10-yr, 2 hr</td>
<td>6.1</td>
<td>44,064</td>
</tr>
</tbody>
</table>

Step 4: Design of Outfall Structure:
Based on Town of Flower Mound requirements, the outfall of the site must be equal to or less than the pre-developed rate of discharge.

-- Design for In-line basin (flow from the site AND the upper watershed is routed through detention basin):
- Design of the in-line basin must be such that the outfall discharge from the site is no more than existing. For example 1, this discharge was estimated as 87 cfs.
- This means the flow from the detention basin, plus flows on the site downstream of the detention basin must be less than or equal to 87 cfs.
- The overflow weir for the detention must be provided at the location of the historical flow path.

-- Off-line basin (only flows from the site enter the detention basin; the upper watershed flows are routed around the basin):
- Design of the off-line basin must be such that the total outfall discharge from the site (detention basin discharge plus upperwatershed flow) is no more than the existing flow off the site.
- The existing flows off the site are 87 cfs. The upper watershed flows (75 cfs) plus the flows from the detention basin plus the flows on the site downstream of the detention basin must be equal to or less than 87 cfs.
- The overflow weir for the detention must be provided at the location of the historical flow path.
Example 2

Sample Watershed

ITRS

Basin 1

Basin 2

Basin 3 (SITE)

Outfall

Determining Time of Concentration for Developed and Undeveloped Conditions

<table>
<thead>
<tr>
<th>DEVELOPMENT STATUS</th>
<th>UNDEVELOPED</th>
<th>DEVELOPED</th>
</tr>
</thead>
<tbody>
<tr>
<td>Basin 1</td>
<td>8 acres</td>
<td>8 acres</td>
</tr>
<tr>
<td>Basin 2</td>
<td>18 acres</td>
<td>18 acres</td>
</tr>
<tr>
<td>Basin 3</td>
<td>4 acres</td>
<td>4 acres</td>
</tr>
<tr>
<td>Total Watershed</td>
<td>30 acres</td>
<td>30 acres</td>
</tr>
</tbody>
</table>

Existing Conditions of the Land (undeveloped conditions)
- Basin 1 is Forested
- Basin 2 is Woodland
- Basin 3 is Grass

Developed Conditions of the Land
- Basin 1 is same (not developed)
- Basin 2 is same (not developed)
- Basin 3 is a master planned single family residential community

**Diagrams and Tables**

**Sheets Flow**

<table>
<thead>
<tr>
<th>Basin</th>
<th>Length (ft)</th>
<th>Elev1</th>
<th>Elev2</th>
<th>Elev3</th>
<th>Slope Graph</th>
<th>Tc (hr)</th>
<th>Avg Vel from</th>
<th>Segments 1</th>
<th>Avg Vel from</th>
<th>Segments 2</th>
<th>Avg Vel from</th>
<th>Segments 3</th>
<th>Total</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Manning's Equation

V = 1.49R^0.6S^0.5T^0.5

Note:
Coding two "segments" of shallow concentrated flow may be appropriate in cases where there is a distinct slope change.
EXAMPLE 2

Discharge by Rational Method

Given:
Determined from site plan:
- Basin 1 is 8 acres
- Basin 2 is 18 acres
- Basin 3 is 4 acres
- Total watershed is 30 acres

Existing conditions of the land:
(pre-developed conditions):
- Basin 1 is Forested
- Basin 2 is Woodland
- Basin 3 is Grass

Developed conditions of the land:
- Basin 1 is same (not developed)
- Basin 2 is same (not developed)
- Basin 3 is master planned single family residential community
  (avg lots are 1/4 acre)

<table>
<thead>
<tr>
<th>BASIN</th>
<th>C</th>
<th>Tc (min)</th>
<th>Tc (in/hr)</th>
<th>I</th>
<th>A (ac)</th>
<th>Q = CIA (cfs)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>pre-developed</td>
<td>0.30</td>
<td>7.6</td>
<td>15.0</td>
<td>9.6</td>
<td>8.0</td>
</tr>
<tr>
<td>2</td>
<td>pre-developed</td>
<td>0.30</td>
<td>9.2</td>
<td>15.0</td>
<td>9.6</td>
<td>18.0</td>
</tr>
<tr>
<td>3</td>
<td>pre-developed</td>
<td>0.30</td>
<td>6.0</td>
<td>15.0</td>
<td>9.6</td>
<td>4.0</td>
</tr>
<tr>
<td>Total</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>30.0</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>BASIN</th>
<th>C</th>
<th>Tc (min)</th>
<th>Tc (in/hr)</th>
<th>I</th>
<th>A (ac)</th>
<th>Q = CIA (cfs)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>not developed</td>
<td>0.30</td>
<td>7.6</td>
<td>15.0</td>
<td>9.6</td>
<td>9.0</td>
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<td>2</td>
<td>not developed</td>
<td>0.30</td>
<td>9.2</td>
<td>15.0</td>
<td>9.6</td>
<td>18.0</td>
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<tr>
<td>3</td>
<td>developed</td>
<td>0.85</td>
<td>5.5</td>
<td>15.0</td>
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<tr>
<td>Total</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>30.0</td>
</tr>
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</table>

Change in flows: 21.1

There is a 21 cfs increase in discharge after the property is developed.
EXAMPLE 2

SAMPLE DETENTION CALCULATIONS

Given:
- Existing site land use is non-developed
- Proposed site development is a master planned residential community
- Area of the site is 4 acres (onsite watershed)
- Total watershed for the site is 18 acres (onsite and offsite)
- Detention basin must be designed for the 100-year, 2-hour event

Step 1. Determine parameters for the site:
- For the 100-year, 2-hour storm, I = 2.6 in/hr
  - see Town of Flower Mound IDF Curve in Appendix A.
- For the site, C = 0.55
  - see Runoff Coefficient Table in Appendix A

Step 2. Compute Q:

<table>
<thead>
<tr>
<th>C</th>
<th>I (in/hr)</th>
<th>A (ac)</th>
<th>Q (cfs)</th>
</tr>
</thead>
<tbody>
<tr>
<td>100-yd, 2 hr</td>
<td>0.55</td>
<td>2.6</td>
<td>4.0</td>
</tr>
<tr>
<td>50-yd, 2 hr</td>
<td>0.55</td>
<td>2.3</td>
<td>4.0</td>
</tr>
<tr>
<td>25-yd, 2 hr</td>
<td>0.55</td>
<td>2.0</td>
<td>4.0</td>
</tr>
<tr>
<td>10-yd, 2 hr</td>
<td>0.55</td>
<td>1.8</td>
<td>4.0</td>
</tr>
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</table>

Step 3. Compute Volume required for the detention basin:

<table>
<thead>
<tr>
<th>Q (cfs)</th>
<th>V = Q * 7200</th>
<th>V (acre-ft)</th>
<th>Site area needed for detention pond assumed depth of 3 feet</th>
</tr>
</thead>
<tbody>
<tr>
<td>100-yd, 2 hr</td>
<td>5.7</td>
<td>41,184</td>
<td>0.95</td>
</tr>
<tr>
<td>50-yd, 2 hr</td>
<td>5.1</td>
<td>36,432</td>
<td>0.84</td>
</tr>
<tr>
<td>25-yd, 2 hr</td>
<td>4.4</td>
<td>31,680</td>
<td>0.73</td>
</tr>
<tr>
<td>10-yd, 2 hr</td>
<td>4.0</td>
<td>28,512</td>
<td>0.65</td>
</tr>
</tbody>
</table>

Step 4. Design of Outfall Structure:
Based on Town of Flower Mound requirements, the outfall of the site must be equal to or less than the pre-developed rate of discharge.

-- Design for In-line basin (flow from the site AND the upper watershed is routed through detention basin):
- Design of the in-line basin must be such that the outfall discharge from the site is no more than existing. For example 1, this discharge was estimated as 87 cfs.
- This means the flow from the detention basin, plus flows on the site downstream of the detention basin must be less than or equal to 87 cfs.
- The overflow weir for the detention must be provided at the location of the historical flow path.

-- Off-line basin (only flows from the site enter the detention basin; the upper watershed flows are routed around the basin):
- Design of the off-line basin must be such that the total outfall discharge from the site (detention basin discharge plus upper watershed flow) is no more than the existing flow off the site.
- The existing flows off the site are 87 cfs. The upper watershed flows (75 cfs) plus the flows from the detention basin plus the flows on the site downstream of the detention basin must be equal to or less than 87 cfs.
- The overflow weir for the detention must be provided at the location of the historical flow path.
Appendix F

This appendix provides material specification and standards for storm sewer products that will be used in the Town of Flower Mound. The Town Manager or his designee before installation must approve all materials or products that do not meet these minimum standards.

A. Culverts

1. Reinforced Concrete Box
   a. Precast. Precast reinforced concrete box shall conform to ASTM C789 or C850.
   b. Cast-in-place. Cast-in-place concrete box shall be designed by a Licensed Professional Engineer in the State of Texas.

2. Concrete Pipe. Pipe used for culverts shall be able to handle HS-20 traffic loading and conform to the standards for storm sewer conduits.

B. Storm Sewer Conduits

1. Storm Sewer (Reinforced Concrete) pipe shall confirm to the following ASTM Designations:
   a. Circular Pipe – ASTM Designation C76; Class III.
   d. Other Pipe Material – Such as metal and high density polyethylene pipe, will not be allowed unless approved by the Town Manager or his designee.

2. Fittings
   The design and manufacture of all special fittings shall be governed by the same requirements as the connecting pipe.

3. Joint Materials:
   a. Gasket Material – ASTM C443. The polymer shall be synthetic rubber; natural rubber will not be acceptable.

4. Extension Ring Materials:
C. **Manholes**

1. **Manhole Materials**

   a. **Manhole Covers** – All manhole covers shall conform to the Standard Specifications for Grey Iron Castings, ASTM A-48, Class 30 B.

   b. **Installation**

      i) All manhole covers shall be 24-inch in diameter.
      ii) All manhole covers shall have two integrally cast pick bars.
      iii) Manhole covers shall weigh approximately 134 lbs.

   c. **Manufacturers**

      i) Bass and Hays
      ii) Vulcan

2. **Manhole Frames**

   a. All manhole frames shall conform to the Standard Specifications for Grey Iron Castings, ASTM A-48, Class 30 B.

   b. **Installation**

      i) All manhole frames shall provide a 24-1/4" opening to assure proper fit of the manhole cover.
      ii) Manhole rings shall weigh approximately 170 lbs.

   c. **Manufacturers**

      i) Bass and Hays
      ii) Vulcan

3. **Extension Ring Materials**

   a. All precast reinforced concrete extension rings shall conform to ASTM C-478.
   b. The number of extension ring sections shall be kept to a minimum (i.e. use 1-12” extension ring instead of 2-6” extension rings).
   c. A 1” x 3-1/2” bitumastic gasket shall be used to seal the extension ring at both joints.
   d. **Manufacturers**

      i) Hydro-Conduit
      ii) Hansen
## Appendix G: Storm Water Construction Standards

*(Provided for reference – Included in Volume II)*

<table>
<thead>
<tr>
<th>Title</th>
<th>Detail No.</th>
<th>Sheet No.</th>
<th>Revision Date</th>
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<tr>
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<td>D-2</td>
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<tr>
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<td>Reinforced Concrete Class III Pipe Installation</td>
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<td>Curbed Flume &amp; Pilot Channels</td>
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<td>1</td>
<td>February, 1999</td>
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<tr>
<td>Sloping Headwall</td>
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<td>February, 1999</td>
</tr>
<tr>
<td>Vertical Headwall</td>
<td>D-9</td>
<td>2</td>
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Dry Extended Detention Ponds
Wet Detention Ponds
Infiltration Basin
Infiltration Trench
Infiltration Drainfields
Porous Pavement
Bioretention
Sand Filters
Storm Water Wetlands
Vegetated Swales
Grassed Filter Strip
Silt Fence
In-line Storage
Green Parking
Modular Treatment Systems
Water Quality Inlets
Dry Extended Detention Ponds
Post-Construction Storm Water Management in New Development & Redevelopment

Dry Extended Detention Pond

A dry extended detention pond is designed to temporarily detain runoff during storm events.

Description

Dry extended detention ponds (a.k.a. dry ponds, extended detention basins, detention ponds, extended detention ponds) are basins whose outlets have been designed to detain the storm water runoff from a water quality design storm for some minimum time (e.g., 24 hours) to allow particles and associated pollutants to settle. Unlike wet ponds, these facilities do not have a large permanent pool. However, they are often designed with small pools at the inlet and outlet of the basin. They can also be used to provide flood control by including additional flood detention storage.

Applicability

Dry extended detention ponds are among the most widely applicable storm water management practices. Although they have limited applicability in highly urbanized settings, they have few other restrictions.

Regional Applicability

Dry extended detention ponds can be applied in all regions of the United States. Some minor design modifications might be needed, however, in cold or arid climates or in regions with karst (i.e. limestone) topography.

Ultra-Urban Areas

Ultra-urban areas are densely developed urban areas in which little pervious surface is present. It is difficult to use dry extended detention ponds in the...
ultra-urban environment because of the land area each pond consumes. They can, however, be used in an ultra-urban environment if a relatively large area is available downstream of the pond.

Storm Water Hot Spots

Storm water hot spots are areas where land use or activities generate highly contaminated runoff, with concentrations of pollutants in excess of those typically found in storm water. Dry extended detention ponds can accept runoff from storm water hot spots, but they need significant separation from ground water if they will be used for this purpose.

Storm Water Retrofit

A storm water retrofit is a storm water management practice (usually structural) put into place after development has occurred to improve water quality, protect downstream channels, reduce flooding, or meet other specific objectives. Dry extended detention ponds are very useful storm water retrofits, and they have two primary applications as a retrofit design. In many communities in the past, detention basins have been designed for flood control. It is possible to modify these facilities to incorporate features that encourage water quality control and/or channel protection. It is also possible to construct new dry ponds in open areas of a watershed to capture existing drainage.

Cold Water (Trout) Streams

A study in Prince George's County, Maryland, found that storm water management practices can increase stream temperatures (Gall, 1990). Overall, dry extended detention ponds increased temperature by about 5°F. In cold water streams, dry ponds should be designed to detain storm water for a relatively short time (i.e., less than 12 hours) to minimize the amount of warming that occurs in the practice.

Siting and Design Considerations

Siting Considerations

Although dry extended detention ponds can be applied rather broadly, designers need to ensure that they are feasible at the site in question. This section provides basic guidelines for siting dry extended detention ponds.

Drainage Area

In general, dry extended detention ponds should be used on sites with a minimum area of 10 acres. On smaller sites, it can be challenging to provide channel or water quality control because the orifice diameter at the outlet needed to control relatively small storms becomes very small and thus prone to clogging. In addition, it is generally more cost-effective to control larger drainage areas due to the economies of scale (see Cost Considerations).

Slope

Dry extended detention basins can be used on sites with slopes up to about 15 percent. The local slope needs to be relatively flat, however, to maintain reasonably flat side slopes in the practice. There is no minimum slope requirement, but there does need to be enough elevation drop from the pond inlet to the pond outlet to ensure that flow can move through the system.

Soils / Topography

Extended detention basins can be used with almost all soils and geology, with minor design adjustments for regions of karst topography or in rapidly percolating soils such as sand. In these areas, extended detention ponds should be designed with an impermeable liner to prevent ground water contamination or sinkhole formation.

Ground Water
Except for the case of hot spot runoff, the only consideration regarding ground water is that the base of the extended detention facility should not intersect the ground water table. A permanently wet bottom may become a mosquito breeding ground. Research in Southwest Florida (Santana et al., 1994) demonstrated that intermittently flooded systems, such as dry extended detention ponds, produce more mosquitoes than other pond systems, particularly when the facilities remained wet for more than 3 days following heavy rainfall.

Design Considerations

Specific designs may vary considerably, depending on site constraints or preferences of the designer or community. Some features, however, should be incorporated into most dry extended detention pond designs. These design features can be divided into five basic categories: pretreatment, treatment, conveyance, maintenance reduction, and landscaping.

Pretreatment

Pretreatment incorporates design features that help to settle out coarse sediment particles. By removing these particles from runoff before they reach the large permanent pool, the maintenance burden of the pond is reduced. In ponds, pretreatment is achieved with a sediment forebay, which is a small pool (typically about 10 percent of the volume of water to be treated for pollutant removal).

Treatment

Treatment design features help enhance the ability of a storm water management practice to remove pollutants. Designing dry ponds with a high length-to-width ratio (i.e., at least 1.5:1) and incorporating other design features to maximize the flow path effectively increases the detention time in the system by eliminating the potential of flow to short-circuit the pond. Designing ponds with relatively flat side slopes can also help to lengthen the effective flow path. Finally, the pond should be sized to detain the volume of runoff to be treated for between 12 and 48 hours.

Conveyance

Conveyance of storm water runoff into and through a storm water management practice is a critical component of any such practice. Storm water should be conveyed to and from practices safely in a manner that minimizes erosion potential. The outfall of pond systems should always be stabilized to prevent scour. To convey low flows through the system, designers should provide a pilot channel. A pilot channel is a surface channel that should be used to convey low flows through the pond. In addition, an emergency spillway should be provided to safely convey large flood events. To help mitigate warming at the outlet channel, designers should provide shade around the channel at the pond outlet.

Maintenance Reduction

In addition to regular maintenance activities needed to maintain the function of storm water practices, some design features can be incorporated to ease the maintenance burden of each practice. In dry extended detention ponds, a "micropool" at the outlet can prevent resuspension of sediment and outlet clogging. A good design includes maintenance access to the forebay and micropool.

Another design feature that can reduce maintenance needs is a non-clogging outlet. Typical examples include a reverse-slope pipe or a weir outlet with a trash rack. A reverse slope pipe draws from below the permanent pool extending in a reverse angle up to the riser and determines the water elevation of the micropool. Because these outlets draw water from below the level of the permanent pool, they are less likely to be clogged by floating debris.

Landscaping

Designers should maintain a vegetated buffer around the pond and should select
plants within the extended detention zone (i.e., the portion of the pond up to the elevation where storm water is detained) that can withstand both wet and dry periods. The side slopes of dry ponds should be relatively flat to reduce safety risks.

**Design Variations**

**Dry Detention Ponds**

Dry detention ponds are similar in design to extended detention ponds, except that they do not incorporate features to improve water quality. In particular, these practices do not detain storm water from small-flow events. Therefore, detention ponds provide almost no pollutant removal. However, dry ponds can help to meet flood control, and sometimes channel protection, objectives in a watershed.

**Tank Storage**

Another variation of the dry detention pond design is the use of tank storage. In these designs, storm water runoff is conveyed to large storage tanks or vaults underground. This practice is most often used in the ultra-urban environment, on small sites where no other opportunity is available to provide flood control. Tank storage is provided on small areas because providing underground storage for a large drainage area would generally be cost-prohibitive. Because the drainage area contributing to tank storage is typically small, the outlet diameter needed to reduce the flow from very small storms would be very small. A very small outlet diameter, along with the underground location of the tanks, creates the potential for debris being caught in the outlet and resulting maintenance problems. Since it is necessary to control small runoff events (such as the runoff from a 1-inch storm) to improve water quality, it is generally infeasible to use tank storage for water quality and generally impractical to use it to protect stream channels.

**Regional Variations**

**Arid or Semi-Arid Climates**

In arid and semi-arid regions, some modifications might be needed to conserve scarce water resources. Any landscaping plans should prescribe drought-tolerant vegetation wherever possible. In addition, the wet forebay can be replaced with an alternative dry pretreatment, such as a detention cell. One opportunity in regions with a distinct wet and dry season, as in many arid regions, is to use regional extended detention ponds as a recreation area such as a ball field during the dry season.

**Cold Climates**

In cold climates, some additional design features can help to treat the spring snowmelt. One such modification is to increase the volume available for detention to help treat this relatively large runoff event. In some cases, dry facilities may be an option as a snow storage facility to promote some treatment of plowed snow. If a pond is used to treat road runoff or is used for snow storage, landscaping should incorporate salt-tolerant species. Finally, sediment might need to be removed from the forebay more frequently than in warmer climates (see Maintenance Considerations for guidelines) to account for sediment deposited as a result of road sanding.

**Limitations**

Although dry extended detention ponds are widely applicable, they have some limitations that might make other storm water management options preferable:

- Dry extended detention ponds have only moderate pollutant removal when compared to other structural storm water practices, and they are ineffective at removing soluble pollutants (See Efficacy).
- Dry extended detention ponds may become a nuisance due to mosquito breeding.
- Habitat destruction may occur during construction if the practice is designed in-stream or within the stream buffer.
Although wet ponds can increase property values, dry ponds can actually detract from the value of a home (see Cost Considerations).

Dry extended detention ponds on their own only provide peak flow reduction and do little to control overall runoff volume, which could result in adverse downstream impacts.

Maintenance Considerations

In addition to incorporating features into the pond design to minimize maintenance, some regular maintenance and inspection practices are needed. Table 1 outlines some of these practices.

Table 1. Typical maintenance activities for dry ponds (Source: Modified from WMI, 1997)

<table>
<thead>
<tr>
<th>Activity</th>
<th>Schedule</th>
</tr>
</thead>
<tbody>
<tr>
<td>- Note erosion of pond banks or bottom</td>
<td>Semiannual inspection</td>
</tr>
<tr>
<td>- Inspect for damage to the embankment</td>
<td>Annual inspection</td>
</tr>
<tr>
<td>- Monitor for sediment accumulation in the facility and forebay</td>
<td></td>
</tr>
<tr>
<td>- Examine to ensure that inlet and outlet devices are free of debris and operational</td>
<td></td>
</tr>
<tr>
<td>- Repair undercut or eroded areas</td>
<td>Standard maintenance</td>
</tr>
<tr>
<td>- Mow side slopes</td>
<td></td>
</tr>
<tr>
<td>- Manage pesticide and nutrients</td>
<td></td>
</tr>
<tr>
<td>- Remove litter and debris</td>
<td></td>
</tr>
<tr>
<td>- Seed or sod to restore dead or damaged ground cover</td>
<td>Annual maintenance (as needed)</td>
</tr>
<tr>
<td>- Remove sediment from the forebay</td>
<td>5- to 7-year maintenance</td>
</tr>
<tr>
<td>- Monitor sediment accumulations, and remove sediment when the pond volume has been reduced by 25 percent</td>
<td>25- to 50-year maintenance</td>
</tr>
</tbody>
</table>

Effectiveness

Structural management practices can be used to achieve four broad resource protection goals: flood control, channel protection, ground water recharge, and pollutant removal. Dry extended detention basins can provide flood control and channel protection, as well as some pollutant removal.

Flood Control

One objective of storm water management practices can be to reduce the flood hazard associated with large storm events by reducing the peak flow associated with these storms. Dry extended detention basins can easily be designed for flood control, and this is actually the primary purpose of most extended detention ponds.

Channel Protection

One result of urbanization is the geomorphic changes that occur in response to modified hydrology. Traditionally, dry extended detention basins have provided control of the 2-year storm (i.e., the storm that occurs, on average, once every 2 years) for channel protection. It appears that this control has been relatively ineffective, and recent research suggests that control of a smaller storm might be more appropriate (MacRae, 1998). Slighty modifying the design of dry extended detention basins to reduce the flow of smaller storm events might make them effective tools in reducing downstream erosion.

Pollutant Removal

Dry extended detention basins provide moderate pollutant removal, provided that the design features described in the Siting and Design Considerations section are incorporated. Although they can be effective at removing some
pollutants through settling, they are less effective at removing soluble pollutants because of the absence of a permanent pool. A few studies are available on the effectiveness of dry extended detention ponds. Typical removal rates, as reported by Schueler (1997), are as follows:

Total suspended solids: 61%
Total phosphorus: 19%
Total nitrogen: 31%
Nitrate nitrogen: 9%
Metals: 26%&ndash;54%

There is considerable variability in the effectiveness of ponds, and it is believed that properly designing and maintaining ponds may help to improve their performance. The siting and design criteria presented in this sheet reflect the best current information and experience to improve the performance of wet ponds. A recent joint project of the American Society of Civil Engineers (ASCE) and the USEPA Office of Water might help to isolate specific design features that can improve performance. The National Storm Water Best Management Practice (BMP) database is a compilation of storm water practices that includes both design information and performance data for various practices. As the database expands, inferences about the extent to which specific design criteria influence pollutant removal may be made. For more information on this database, access the BMP database web page at http://www.bmpdatabase.org.

Cost Considerations

Dry extended detention ponds are the least expensive storm water management practice, on the basis of cost per unit area treated. The construction costs associated with these facilities range considerably. One recent study evaluated the cost of all pond systems (Brown and Schueler, 1997). Adjusting for inflation, the cost of dry extended detention ponds can be estimated with the equation

\[ C = 12.4V^{0.760} \]

where:

\[ C = \text{Construction, design, and permitting cost, and} \]

\[ V = \text{Volume needed to control the 10-year storm (ft}^3)\].

Using this equation, typical construction costs are

- $41,600 for a 1 acre-foot pond
- $239,000 for a 10 acre-foot pond
- $1,380,000 for a 100 acre-foot pond

Interestingly, these costs are generally slightly higher than the cost of wet ponds on a cost per total volume basis. Dry extended detention ponds are generally less expensive on a given site, however, because they are usually smaller than a wet pond design for the same site.

Ponds do not consume a large area compared to the total area treated (typically 2 to 3 percent of the contributing drainage area). It is important to note, however, that each pond is generally large. Other practices, such as filters or swales, may be "squeezed in" on relatively unusable land, but ponds need a relatively large continuous area.

For ponds, the annual cost of routine maintenance is typically estimated at about 3 to 5 percent of the construction cost. Alternatively, a community can estimate the cost of the maintenance activities outlined in the maintenance section. Finally, ponds are long-lived facilities (typically longer than 20 years). Thus, the initial investment into pond systems can be spread over a relatively long time.
period.

Another economic concern associated with dry ponds is that they might detract slightly from the value of adjacent properties. One study found that dry ponds can actually detract from the perceived value of homes adjacent to a dry pond by between 3 and 10 percent (Emmerling-Dinovo, 1995).

References

Design References:


Other References:


Information Resources


Wet Detention Ponds
Storm Water Technology Fact Sheet

Wet Detention Ponds

DESCRIPTION

Wet detention ponds are storm water control structures providing both retention and treatment of contaminated storm water runoff. A typical wet detention pond design is shown in Figure 1. The pond consists of a permanent pool of water into which storm water runoff is directed. Runoff from each rain event is detained and treated in the pond until it is displaced by runoff from the next storm. By capturing and retaining runoff during storm events, wet detention ponds control both storm water quantity and quality. The pond’s natural physical, biological, and chemical processes then work to remove pollutants. Sedimentation processes remove particulates, organic matter, and metals, while dissolved metals and nutrients are removed through biological uptake. In general, a higher level of nutrient removal and better storm water quantity control can be achieved in wet ponds.

![Diagram of a typical wet detention pond](image)

Source: Maryland Department of the Environment, 1986.

FIGURE 1 TYPICAL LAYOUT OF A WET DETENTION POND
detention ponds than can be achieved with other Best Management Practices (BMPs), such as dry ponds, infiltration trenches, or sand filters.

There are several common modifications that can be made to the ponds to increase their pollutant removal effectiveness. The first is to increase the settling area for sediments through the addition of a sediment forebay, as shown in Figure 1. Heavier sediments will drop out of suspension as runoff passes through the sediment forebay, while lighter sediments will settle out as the runoff is retained in the permanent pool. A second common modification is the construction of shallow ledges along the edge of the permanent pool. These shallow peripheral ledges can be used to establish aquatic plants that can impede flow and trap pollutants as they enter the pond. The plants also increase biological uptake of nutrients. In addition to their function as aquatic plant habitat, the ledges also have several other functions, which can include acting as a safety precaution to prevent accidental drowning and providing easy access to the permanent pool to aid in maintenance. Finally, perimeter wetland areas can also be created around the pond to aid in pollutant removal.

APPLICABILITY

Wet detention ponds have been widely used throughout the U.S. for many years. Many of these ponds have been monitored to determine their performance. EPA Region V is currently performing a study on the effectiveness of 50 to 60 wet detention ponds. Other organizations, such as the Washington, D.C., Council of Governments (WMCOG) and the Maryland Department of the Environment, have also conducted extensive evaluations of wet detention pond performance.

ADVANTAGES AND DISADVANTAGES

Wet detention ponds provide both storm water quantity and quality benefits, and provide significant retrofit coverage for existing development. Benefits include decreased potential for downstream flooding and stream bank erosion and improved water quality due to the removal of suspended solids, metals, and dissolved nutrients.

While the positive impacts from a wet detention pond will generally exceed any negative impacts, wet detention ponds that are improperly designed, sited, or maintained, may have potential adverse affects on water quality, groundwater, cold water fisheries, or wetlands. Improperly designed or maintained ponds may result in stratification and anoxic conditions that can promote the resuspension of solids and the release of nutrients and metals from the trapped sediments. In addition, precautions should be taken to prevent damage to wetland areas during pond construction. Finally, the potential for groundwater contamination should be carefully evaluated. However, studies to date indicate that wet detention ponds do not significantly contribute to groundwater contamination (Schueler, 1992).

The following limitation should also be considered when determining the feasibility of installing a wet detention pond:

1. Wet detention ponds must be able to maintain a permanent pool of water. Therefore, ponds cannot be constructed in areas where there is insufficient precipitation to maintain the pool or in soils that are highly permeable. In wetter regions, a small drainage area may be sufficient to ensure that there is enough water to maintain a permanent pool; whereas in more arid regions, a larger drainage area may be required. In some cases, soils that are highly permeable may be compacted or overlaid with clay blankets to make the bottom less permeable.

2. Land constraints, such as small sites or highly developed areas, may preclude the installation of a pond.

3. Discharges from ponds usually consist of warm water, and thus pond use may be limited in areas where warm water discharges from the pond will adversely impact a cold water fishery.

4. The local climate (i.e., temperature) may affect the biological uptake in the pond.
5. Without proper maintenance, the performance of the pond will drop off sharply. Regular cleaning of the forebays is particularly important. Maintaining the permanent pool is also important in preventing the resuspension of trapped sediments. The accumulation of sediments in the pond will reduce the pond's storage capacity and cause a decline in its performance. Therefore, the bottom sediments in the permanent pool should be removed about every 2 to 5 years. In most cases, no specific limitations have been placed on disposal of sediments removed from wet detention ponds. Studies to date indicate that pond sediments are likely to meet toxicity limits and can be safely landfilled (NVPDC, 1992). Some states have allowed sediment disposal on-site, as long as the sediments are deposited away from the shoreline to prevent their re-entry into the pond.

DESIGN CRITERIA

In general, pond designs are unique for each site and application. Criteria for selecting the site for installation of the pond should include the site's ability to support the pond environment, as well as the cost effectiveness of locating a pond at that specific site. In addition, the pond should be located where the topography of the site allows for maximum storage at minimum construction costs (NVPDC, 1992). Site-specific constraints for pond construction may include wetlands impacts, existing utilities (e.g., electric or gas) that would be costly to relocate, and underlying bedrock that would require expensive blasting operations to excavate.

The site must have adequate base-flow from the groundwater or from the drainage area to maintain the permanent pool. Typically, underlying soils with permeabilities of between $10^{-4}$ and $10^{-6}$ cm/sec will be adequate to maintain a permanent pool.

All local, state and federal permit requirements should be established prior to initiating the pond design. Depending on the location of the pond, required permits and certifications may include wetland permits, water quality certifications, dam safety permits, sediment and erosion control plans, waterway permits, local grading permits, land use approvals, etc. (Schueler, 1992). Since many states and municipalities are still in the process of developing or modifying storm water permit requirements, the applicable requirements should be confirmed with the appropriate regulatory authorities.

Wet detention ponds should be designed to meet both storm water quality and quantity control requirements. Storm water quantity requirements are typically met by designing the pond to control post-development peak discharge rates to pre-development levels. Usually the pond is designed to control multiple design storms (e.g. 2- and/or 10-year storms) and safely pass the 100-year storm event. However, the design storm may vary depending on local conditions and requirements.

Storm water quality control is achieved through pollutant removal in the permanent pool. Removal efficiency is primarily dependent on the length of time that runoff remains in the pond, which is known as the pond's Hydraulic Residence Time (HRT). As discussed above, wet detention ponds remove pollutants through both sedimentation and biological uptake processes, both of which increase with the length of time runoff remains in the pond. These processes can be modeled to determine a design HRT using either the solids settling method or the eutrophication method, respectively (Hartigan, 1988).

The calculated HRT will be dependent on the method selected. HRTs calculated by the eutrophication method can be up to three times greater than HRTs calculated by the solids settling method. The longer HRTs associated with the eutrophication method appear to be due to the slower reaction rates associated with the biological removal of dissolved nutrients (Hartigan, 1988).

Once the design HRT has been determined, the actual dimensions of the pond must be calculated to achieve the design HRT. The primary factor contributing to a pond's HRT is its volume. Because many wet detention ponds are restricted in area, pond depth can be an important factor in the
pond’s overall volume. However, the depth of the pool also affects many of the pond’s removal processes, and so must be carefully controlled. It is important to maintain a sufficient permanent pool depth in order to prevent the resuspension of trapped sediments (NVAPDC, 1992). Conversely, thermal stratification and anoxic conditions in the bottom layer might develop if permanent pool depths are too great. Stratification and anoxic conditions may decrease biological activity. Anoxic conditions may also increase the potential for the release of phosphorus and heavy metals from the pond sediments (NVAPDC, 1992). These factors dictate that the permanent pool depth should not exceed 6 meters (20 feet). The optimal depth ranges between 1 and 3 meters (3 and 9 feet) for most regions, given a 2 week HRT (Hartigan, 1988).

Other key factors to be considered in the pond design are the volume and area ratios. The volume ratio, VB/VR, is the ratio of the permanent pool storage (VB) to the mean storm runoff (VR). Larger VBs and smaller VRs provide for increased retention and treatment between storm events. Low VB/VR ratios result in poor pollutant removal efficiencies.

The area ratio, A/A5, is the ratio of the contributing drainage area (A) to the permanent pool surface area (A5). The area ratio is also an indicator of pollutant removal efficiency. Data from previous studies indicates that area ratios of less than 100 typically have better pollutant removal efficiencies (MD DEQ, 1986).

The contours of the pond are also important. The pond should be constructed with adequate slopes and lengths. While a length-to-width ratio is usually not used in the design of wet detention ponds for storm water quantity management, a 2:1 length-to-width ratio is commonly used when water quality is of concern. In general, high length-to-width ratios (greater than 2:1) will decrease the possibility of short-circuiting and will enhance sedimentation within the permanent pool. Baffles or islands can also be added within the permanent pool to increase the flow path (Hartigan, 1988). Shoreline slopes between 5:1 and 10:1 are common and allow easy access for maintenance, such as mowing and sediment removal (Hartigan, 1988). In addition, wetland vegetation is difficult to establish and maintain on slopes steeper than 10:1. Ponds should be wedge-shaped so that flow enters the pond and gradually spreads out. This minimizes the potential for zones with little or no flow (Urbonas, 1993).

The design of the wet pond embankment is another key factor to be considered. Proper design and construction of the embankments will prolong the integrity of the pond structure. Subsidence and settling will likely occur after an embankment is constructed. Therefore during construction, the embankment should be overfilled by at least 5 percent (SEWRPC, 1991). Seepage through the embankment can also affect the stability of the structure. Seepage can generally be minimized by adding drains, anti-seepage collars, and core trenches. The embankment side slopes can be protected from erosion by using minimum side slopes of 2:1 and by covering the embankment with vegetation or rip-rap. The embankment should also have a minimum top width of 2 meters (6 feet) to aid in maintenance.

Finally, the internal flow control of the pond must be considered. Discharge from the pond is controlled by a riser and an inverted release pipe. Normal flows will be discharged through the wet pond outlet, which consists of a concrete or corrugated metal riser and barrel. The riser is a vertical pipe or inlet structure that is attached to the base with a watertight connection. Risers are typically placed in or adjacent to the embankment rather than in the middle of the pond. This provides easy access for maintenance and prevents the use of the riser as a recreation spot (e.g. diving platform for kids) (Schueler, 1988). The barrel is a horizontal pipe attached to the riser that conveys flow under the embankment.

Typically, flow passes through an inverted pipe attached to the riser, as shown in Figure 1, while higher flows will pass through a trash rack installed on the riser. The inverted pipe should discharge water from below the pond water surface to prevent floatables from clogging the pipe and to avoid discharging the warmer surface water. Clogging of the pipe could result in overtopping of the
embankment and damage to the embankment (NVPDC, 1992). Flow is conveyed through the near horizontal barrel and is discharged to the receiving stream. Rip-rap, plunge pools, or other energy dissipators, should be placed at the outlet to prevent scouring and to minimize erosion. Rip-rap also provides a secondary benefit of re-aeration of the pond discharges.

Planners should consider both the design storm and potential construction materials when designing and constructing the riser and barrel. Generally, the riser and barrel are sized to meet the storm water management design criteria (e.g. to pass a 2-year or a 10-year storm event). In many installations, the riser and barrel are designed to convey multiple design storms (Urbanos, 1993). To increase the life of the outlet, the riser and barrel should be constructed of reinforced concrete rather than corrugated metal pipe (Schueler, 1992). The riser, barrel, and base should also provide have sufficient weight to prevent flotation (NVPDC, 1992).

In most cases, emergency spillways should be included in the pond design. Emergency spillways should be sized to safely pass flows that exceed the design storm flows. The spillway prevents pond water levels from overtopping the embankment, which could cause structural damage to the embankment. The emergency spillway should be located so that downstream buildings and structures will not be negatively impacted by spillway discharges. The pond design should include a low flow drain, as shown in Figure 1. The drain pipe should be designed for gravity discharge and should be equipped with an adjustable gate valve.

PERFORMANCE

The primary pollutant removal mechanism in a wet detention pond is sedimentation. Significant loads of suspended pollutants, such as metals, nutrients, sediments, and organics, can be removed by sedimentation. Other pollutant removal mechanisms include algal uptake, wetland plant uptake, and bacterial decomposition (Schueler, 1992). Dissolved pollutant removal also occurs as a result of biological and chemical processes (NVPDC, 1992).

The removal rates of conventional wet detention ponds (i.e., without the sediment forebay or peripheral ledges) are well documented and are shown in Table 1. The wide range in the removal rates is a result of varying hydraulic residence times (HRTs), which is further discussed in the Design Criteria section. Increased pollutant removal by biological uptake and sedimentation is correlated with increased HRTs. Proper design and maintenance also effect pond performance.

Studies have shown that more than 90 percent of the pollutant removal occurs during the quiescent period (the period between the rainfall events) (MD DEQ, 1986). However, some removal occurs during the dynamic period (when the runoff enters the pond). Modeling results have indicated that two-thirds of the sediment, nutrients and trace metal loads are removed by sedimentation within 24

### TABLE 1 REMOVAL EFFICIENCIES FROM WET DETENTION PONDS

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Percent Removal</th>
</tr>
</thead>
<tbody>
<tr>
<td>Schueler, 1992</td>
<td>Hartigan, 1988</td>
</tr>
<tr>
<td>Total</td>
<td>50-90</td>
</tr>
<tr>
<td>Suspended Solid</td>
<td>80-90</td>
</tr>
<tr>
<td>Total Phosphorus</td>
<td>30-90</td>
</tr>
<tr>
<td>Soluble Nutrients</td>
<td>40-80</td>
</tr>
<tr>
<td>Lead</td>
<td>50-70</td>
</tr>
<tr>
<td>Zinc</td>
<td>40-50</td>
</tr>
<tr>
<td>Biochemical Oxygen Demand</td>
<td>20-40</td>
</tr>
</tbody>
</table>

1 hydraulic residence time varies
2 hydraulic residence time of 2 weeks

hours. These projections are supported by the results of the EPA's 1993 National Urban Runoff Program (NURP) studies. However, other studies indicate that an HRT of two weeks is required to achieve significant phosphorus removal (MD DEQ, 1986).

The pond's treatment efficiency can be enhanced by extending the detention time in the permanent pool to up to 40 hours. This allows for a more gradual release of collected runoff, resulting in both increased pollutant removal and control of peak flows (Hartigan, 1988).

OPERATION AND MAINTENANCE

Wet detention ponds function more effectively when they are regularly inspected and maintained. Routine maintenance of the pond includes mowing of the embankment and buffer areas and inspection for erosion and nuisance problems (e.g., burrowing animals, weeds, odors) (SEWRPC, 1991). Trash and debris should be removed routinely to maintain an attractive appearance and to prevent the outlet from becoming clogged. In general, wet detention ponds should be inspected after every storm event. The embankment and emergency spillway should also be routinely inspected for structural integrity, especially after major storm events. Embankment failure could result in severe downstream flooding. When any problems are observed during routine inspections, necessary repairs should be made immediately. Failure to correct minor problems may lead to larger and more expensive repairs or even to pond failure. Typically, maintenance includes repairs to the embankment, emergency spillway, inlet, and outlet; removal of sediment; and control of algal growth, insects, and odors (SEWRPC, 1991). Large vegetation or trees that may weaken the embankment should be removed. Periodic maintenance may also include the stabilization of the outfall area (e.g., adding rip-rap) to prevent erosive damage to the embankment and the stream bank. In most cases, sediments removed from wet detention ponds are suitable for landfill disposal. However, where available, on-site use of removed sediments for soil amendment will reduce maintenance costs.

COSTS

Typical costs for wet detention ponds range from $17.50-$35.00 per cubic meter ($0.50-$1.00 per cubic foot) of storage area (CWP, 1998). The total cost for a pond includes permitting, design and construction, and maintenance costs. Permitting costs may vary depending on state and local regulations. Typically, wet detention ponds are less costly to construct in undeveloped areas than to retrofit into developed areas. This is due to the cost of land and the difficulty in finding suitable sites in developed areas. The cost of relocating pre-existing utilities or structures is also a major concern in developed areas. Several studies have shown the construction cost of retrofitting a wet detention pond into a developed area may be 5 to 10 times the cost of constructing the same size pond in an undeveloped area. Annual maintenance costs can generally be estimated at 3 to 5 percent of the construction costs (Schueler, 1992). Maintenance costs include the costs for regular inspections of the pond embankments, grass mowing, nuisance control, debris and litter removal, inlet and outlet maintenance and inspection, and sediment removal and disposal. Sediment removal cost can be decreased by as much as 50 percent if an on-site disposal areas are available (SEWRPC, 1991).

REFERENCES


ADDITIONAL INFORMATION

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Infiltration Basin
Post-Construction Storm Water Management in New Development & Redevelopment

Infiltration Basin

Infiltration basins are designed to collect storm water from impervious areas and provide pollutant removal benefits through detention and filtration.

Description

An infiltration basin is a shallow impoundment which is designed to infiltrate storm water into the ground water. This practice is believed to have a high pollutant removal efficiency and can also help recharge the ground water, thus restoring low flows to stream systems. Infiltration basins can be challenging to apply on many sites, however, because of soils requirements. In addition, some studies have shown relatively high failure rates compared with other management practices.

Applicability

Infiltration basins have select applications. Their use is often sharply restricted by concerns over ground water contamination, soils, and clogging at the site.

Regional Applicability

Infiltration basins can be utilized in most regions of the country, with some design modifications in cold and arid climates. In regions of karst (i.e., limestone) topography, these storm water management practices may not be applied due to concerns of sink hole formation and ground water contamination.

Ultra-Urban Areas

Ultra-urban areas are densely developed urban areas in which little pervious...
surface exists. In these areas, few storm water practices can be easily applied due to space limitations. Infiltration basins can rarely be applied in the ultra-urban environment.

Two features that can restrict their use are the potential of infiltrated water to interfere with existing infrastructure, and the relatively poor infiltration capacity of most urban soils. In addition, while they consume only the space of the infiltration basin site itself, they need a continuous, relatively flat area. Thus, it is more difficult to fit them into small unusable areas on a site.

Storm Water Hot Spots

A storm water hot spot is an area where land use or activities generate highly contaminated runoff, with concentrations of pollutants in excess of those typically found in storm water. Infiltration basins should never receive runoff from storm water hot spots, unless the storm water has already been treated by another practice. This caution is due to potential ground water contamination.

Storm Water Retrofit

A storm water retrofit is a storm water practice (usually structural) put into place after development has occurred, to improve water quality, protect downstream channels, reduce flooding, or meet other specific objectives. Infiltration basins have limited applications as a storm water retrofit. Their use is restricted by three factors. First, infiltration basins should be used to treat small sites (less than 5 acres). Practices that are applied to small sites, such as infiltration basins, are generally a high-cost retrofit option in terms of construction cost and the maintenance burden associated with the large number of practices needed to retrofit a watershed. Second, it is often difficult to find areas where soils are appropriate for infiltration in an already urban or suburban environment. Finally, infiltration basins are best applied to small sites, yet need a flat, relatively continuous area. It is often difficult to find sites with this type of area available.

Cold Water (Trout) Streams

Infiltration basins are an excellent option for cold water streams because they encourage infiltration of storm water and maintain dry weather flow. Because storm water travels underground to the stream, it has little opportunity to increase in temperature.

Siting and Design Considerations

When designing infiltration basins, designers need to carefully consider both the restrictions on the site and design features to improve the long-term performance of the practice.

Siting Considerations

Infiltration practices need to be located extremely carefully. In particular, designers need to ensure that the soils on the site are appropriate for infiltration, and that designs minimize the potential for ground water contamination and long-term maintenance problems.

Drainage Area

Infiltration basins have historically been used as regional facilities, serving for both quantity and quality control. In some regions of the country, this practice is feasible, particularly if the soils are particularly sandy. In most areas, however, infiltration basins experience high rates of failure when used in this manner. In general, the practice is best applied to relatively small drainage areas (i.e., less than 10 acres).

Slope

The bottom of infiltration basins needs to be completely flat to allow infiltration throughout the entire basin bottom.

Soils/Topography
Soils and topography are strongly limiting factors when locating infiltration practices. Soils must be significantly permeable to ensure that the practice can infiltrate quickly enough to reduce the potential for clogging, and soils that infiltrate too rapidly may not provide sufficient treatment, creating the potential for ground water contamination. The infiltration rate should range between 0.5 and 3 inches per hour. In addition, the soils should have no greater than 20 percent clay content, and less than 40 percent silt/clay content (MDE, 2000). Finally, infiltration basins may not be used in regions of karst topography, due to the potential for sinkhole formation or ground water contamination.

Ground Water

Designers always need to provide significant separation distance (2 to 6 feet) from the bottom of the infiltration basin and the seasonally high ground water table, to reduce the risk of contamination. Infiltration practices should also be separated from drinking water wells.

Design Considerations

Specific designs may vary considerably, depending on site constraints or preferences of the designer or community. There are some features, however, that should be incorporated into most infiltration basin designs. These design features can be divided into five basic categories: pretreatment, treatment, conveyance, maintenance reduction, and landscaping.

Pretreatment

Pretreatment refers to design features that provide settling of large particles before runoff reaches a management practice, easing the long-term maintenance burden. Pretreatment is important for all structural management practices, but it is particularly important for infiltration practices. In order to ensure that pretreatment mechanisms are effective, designers should incorporate "multiple pretreatment," using practices such as grassed swales, sediment basins, and vegetated filter strips in series.

Treatment

Treatment design features enhance the pollutant removal of a practice. For infiltration practices, designers need to stabilize upland soils to ensure that the basin does not become clogged with sediment. In addition, the facility needs to be sized so that the volume of water to be treated infiltrates through the bottom in a given amount of time. Because infiltration basins are designed in this manner, infiltration basins designed on less permeable soils should be significantly larger than those designed on more permeable soils.

Conveyance

Storm water needs to be conveyed through storm water management practices safely and in a way that minimizes erosion. Designers need to be particularly careful in ensuring that channels leading to an infiltration practice are designed to minimize erosion. In general, infiltration basins should be designed to treat only small storms (i.e., only for water quality). Thus, these practices should be designed "off-line," using a flow separator to divert only small flows to the practice.

Maintenance Reduction

In addition to regular maintenance activities, designers also need to incorporate features into the design to ensure that the maintenance burden of a practice is reduced. These features can make regular maintenance activities easier or reduce the need to perform maintenance. In infiltration basins, designers need to provide access to the basin for regular maintenance activities. Where possible, a means to drain the basin, such as an underdrain, should be provided in case the bottom becomes clogged. This feature allows the basin to be drained and accessed for maintenance in the event that the water has ponded in the basin bottom or the soil is saturated.
Landscaping

Landscaping can enhance the aesthetic value of storm water practices or improve their function. In infiltration basins, the most important purpose of vegetation is to reduce the tendency of the practice to clog. Upland drainage needs to be properly stabilized with a thick layer of vegetation, particularly immediately following construction. In addition, providing a thick turf at the basin bottom helps encourage infiltration and prevent the formation of rills in the basin bottom.

Design Variations

Some modifications may be needed to ensure the performance of infiltration basins in arid and cold climates.

Arid or Semi-Arid Climates

In arid regions, infiltration practices are often highly recommended because of the need to recharge the ground water. In arid regions, designers need to emphasize pretreatment even more strongly to ensure that the practice does not clog, because of the high sediment concentrations associated with storm water runoff in areas such as the Southwest. In addition, the basin bottom may be planted with drought-tolerant species and/or covered with an alternative material such as sand or gravel.

Cold Climates

In extremely cold climates (i.e., regions that experience permafrost), infiltration basins may be an infeasible option. In most cold climates, infiltration basins can be a feasible practice, but there are some challenges to its use. First, the practice may become inoperable during some portions of the year when the surface of the basin becomes frozen. Other design features also may be incorporated to deal with the challenges of cold climates. One such challenge is the volume of runoff associated with the spring snowmelt event. The capacity of the infiltration basin might be increased to account for snowmelt volume.

Another option is the use of a seasonally operated facility (Oberts, 1994). A seasonally operated infiltration/detention basin combines several techniques to improve the performance of infiltration practices in cold climates. Two features, the underdrain system and level control valves, are useful in cold climates. These features are used as follows: At the beginning of the winter season, the level control valve is opened and the soil is drained. As the snow begins to melt in the spring, the underdrain and the level control valves are closed. The snowmelt is infiltrated until the capacity of the soil is reached. Then, the facility acts as a detention facility, providing storage for particles to settle.

Other design features can help to minimize problems associated with winter conditions, particularly concerns that chlorides from road salting may contaminate ground water. The basin may be disconnected during the winter to ensure that chlorides do not enter the ground water in areas where this is a problem, or if the basin is used to treat roadside runoff. Designers may also want to reconsider application of infiltration practices on parking lots or roads where deicing is used, unless it is confirmed that the practice will not cause elevated chloride levels in the ground water. If the basin is used for snow storage, or to treat roadside or parking lot runoff, the basin bottom should be planted with salt-tolerant vegetation.

Limitations

Although infiltration basins can be useful practices, they have several limitations. Infiltration basins are not generally aesthetic practices, particularly if they clog. If they clog, the soils become saturated, and the practice can be a source of mosquitoes. In addition, these practices are challenging to apply because of concerns over ground water contamination and sufficient soil infiltration. Finally, maintenance of infiltration practices can be burdensome, and they have a relatively high rate of failure.

Maintenance Considerations
Regular maintenance is critical to the successful operation of infiltration basins (see Table 1). Historically, infiltration basins have had a poor track record. In one study conducted in Prince George's County, Maryland (Gallic, 1992), all of the infiltration basins investigated clogged within 2 years. This trend may not be the same in soils with high infiltration rates, however. A study of 23 infiltration basins in the Pacific Northwest showed better long-term performance in an area with highly permeable soils (Holding, 1996). In this study, few of the infiltration basins had failed after 10 years.

Table 1. Typical maintenance activities for infiltration basins (Source: Modified from WMI, 1997)

<table>
<thead>
<tr>
<th>Activity</th>
<th>Schedule</th>
</tr>
</thead>
<tbody>
<tr>
<td>• Inspect facility for signs of wetness or damage to structures</td>
<td>Semi-annual inspection</td>
</tr>
<tr>
<td>• Note eroded areas.</td>
<td></td>
</tr>
<tr>
<td>• If dead or dying grass on the bottom is observed, check to ensure that water percolates 2–3 days following storms.</td>
<td></td>
</tr>
<tr>
<td>• Note signs of petroleum hydrocarbon contamination and handle properly.</td>
<td></td>
</tr>
<tr>
<td>• Mow and remove litter and debris.</td>
<td>Standard maintenance (as needed)</td>
</tr>
<tr>
<td>• Stabilize eroded banks.</td>
<td></td>
</tr>
<tr>
<td>• Repair undercut and eroded areas at inflow and outflow structures.</td>
<td></td>
</tr>
<tr>
<td>• Disc or otherwise aerate bottom.</td>
<td>Annual maintenance</td>
</tr>
<tr>
<td>• Delhatch basin bottom.</td>
<td></td>
</tr>
<tr>
<td>• Scrape bottom and remove sediment.</td>
<td>5-year maintenance</td>
</tr>
<tr>
<td>• Restore original cross-section and Infiltration rate.</td>
<td></td>
</tr>
<tr>
<td>• Seed or sod to restore ground cover.</td>
<td></td>
</tr>
</tbody>
</table>

Effectiveness

Structural management practices can be used to achieve four broad resource protection goals. These include flood control, channel protection, ground water recharge, and pollutant removal. Infiltration basins can provide ground water recharge and pollutant removal.

Ground Water Recharge

Infiltration basins recharge the ground water because runoff is treated for water quality by filtering through the soil and discharging to ground water.

Pollutant Removal

Very little data are available regarding the pollutant removal associated with infiltration basins. It is generally assumed that they have very high pollutant removal because none of the storm water entering the practice remains on the surface. Schueler (1987) estimated pollutant removal for infiltration basins based on data from land disposal of wastewater. The average pollutant removal, assuming the infiltration basin is sized to treat the runoff from a 1-inch storm, is:

TSS 75%
Phosphorous 60–70%
Nitrogen 55–80%
Metals 85–90%
Bacteria 90%

These removal efficiencies assume that the infiltration basin is well designed and maintained. The information in the Siting and Design Considerations and Maintenance Considerations sections represent the best available information.
on how to properly design these practices. The design references below also provide additional information.

Cost Considerations

Infiltration basins are relatively cost-effective practices because little infrastructure is needed when constructing them. One study estimated the total construction cost at about $2 per ft³ (adjusted for inflation) of storage for a 0.25-acre basin (SWRPC, 1991). Infiltration basins typically consume about 2 to 3 percent of the site draining to them, which is relatively small. Maintenance costs are estimated at 5 to 10 percent of construction costs.

One cost concern associated with infiltration practices is the maintenance burden and longevity. If improperly maintained, infiltration basins have a high failure rate (see Maintenance Considerations). Thus, it may be necessary to replace the basin after a relatively short period of time.

References


Information Resources


Infiltration Trench
Storm Water Technology Fact Sheet
Infiltration Trench

DESCRIPTION

Urban development is significantly increasing surface runoff and contamination of local watersheds. As a result, infiltration practices, such as infiltration trenches, are being employed to remove suspended solids, particulate pollutants, coliform bacteria, organics, and some soluble forms of metals and nutrients from storm water runoff. As shown in Figure 1, an infiltration trench is an excavated trench, 0.9 to 3.7 meters (3 to 12 feet) deep, backfilled with a stone aggregate, and lined with filter fabric. A small portion of the runoff, usually the first flush, is diverted to the infiltration

![Infiltration Trench Diagram]


FIGURE 1 TYPICAL INFILTRATION TRENCH
trench, which is located either underground or at grade. Pollutants are filtered out of the runoff as it infiltrates the surrounding soils. Infiltration trenches also provide groundwater recharge and preserve baseflow in nearby streams.

APPLICABILITY

Infiltration trenches are often used in place of other Best Management Practices where limited land is available. Infiltration trenches are most widely used in warmer, less arid regions of the U.S. However, recent studies conducted in Maryland and New Jersey on trench performance and operation and maintenance have demonstrated the applicability of infiltration trenches in colder climates if surface icing is avoided (Lindsey, et al, 1991).

Infiltration trenches capture and treat small amounts of runoff, but do not control peak hydraulic flows. Infiltration trenches may be used in conjunction with another Best Management Practice (BMP), such as a detention pond, to provide both water quality control and peak flow control (Harrington, 1989). Figure 2 is an example of such a combined technology. This type of infiltration trench has a concentrated input, as opposed to dispersed input (as shown in Figure 1). This system stores the entire storm water volume with the water quality (BMP) volume connected to the infiltration system. This is commonly achieved with a slow release of the storm water management volume through an orifice set at a specified level in the storage facility. As a result the BMP water quality volume will equal the storm water detention area below the orifice level which must infiltrate to exit.

Runoff that contains high levels of sediments or hydrocarbons (oil and grease) that may clog the trench are often pretreated with other BMPs. Examples of some pretreatment BMPs include grit chambers, water quality inlets, sediment traps, swales, and vegetated filter strips (SEWRPC, 1991, Harrington, 1989).

ADVANTAGES AND DISADVANTAGES

Infiltration trenches provide efficient removal of suspended solids, particulate pollutants, coliform bacteria, organics and some soluble forms of metals and nutrients from storm water runoff. The captured runoff infiltrates the surrounding soils and increases groundwater recharge and baseflow in nearby streams.

Negative impacts include the potential for groundwater contamination and a high likelihood of early failure if not properly maintained.

As with any infiltration BMP, the potential for groundwater contamination must be carefully considered, especially if the groundwater is used for human consumption or agricultural purposes. The infiltration trench is not suitable for sites that use or store chemicals or hazardous materials unless hazardous and toxic materials are prevented from entering the trench. In these areas, other BMPs that do not interact with the groundwater should be considered. The potential for spills can be minimized by aggressive pollution prevention measures. Many municipalities and industries have developed comprehensive spill prevention control and countermeasure (SPCC) plans. These plans should be modified to include the infiltration trench and the contributing drainage area. For example, diversion structures can be used to prevent spills from entering the infiltration trench.

Because of the potential to contaminate groundwater, extensive site investigation must be undertaken early in the site planning process to establish site suitability for the installation of an infiltration trench. The use of infiltration trenches may be limited by a number of factors, including type of native soils, climate, and location of groundwater tables. Site characteristics, such as excessive slope of the drainage area, fine-particled soil types, and proximate location of the water table and bedrock, may preclude the use of infiltration trenches. The slope of the surrounding area should be such that the runoff is evenly distributed in sheet flow as it enters the trench unless specifically designed for concentrated input. Generally, infiltration trenches are not suitable for areas with relatively impermeable soils containing clay and silt.
or in areas with fill. The trench should be located well above the water table so that the runoff can filter through the trench and into the surrounding soils and eventually into the groundwater. In addition, the drainage area should not convey heavy levels of sediments or hydrocarbons to the trench. For this reason, trenches serving parking lots must be preceded by appropriate pretreatment such as an oil-grit separator. This measure will make effective maintenance feasible. Generally, trenches that are constructed under parking lots must provide access for maintenance.

An additional limitation on use of infiltration trenches is the climate. In cold climates, the trench surface may freeze, thereby preventing the runoff from entering the trench and allowing the untreated runoff to enter surface water. The surrounding soils may also freeze, reducing infiltration into the soils and groundwater. However, recent studies indicate that if properly designed and maintained, infiltration trenches can operate effectively in colder climates. By keeping the trench surface free of compacted snow and ice, and by ensuring that part of the trench is constructed below the frost line, the performance of the infiltration trench during cold weather will be greatly improved.

Finally, there have been a number of concerns raised about the long term effectiveness of infiltration trench systems. In the past, infiltration trenches have demonstrated a relatively short life span, with over 50 percent of the systems checked having partially or completely failed after 5 years. A recent study of infiltration trenches in Maryland (Lindsey et al., 1991) found that 53 percent were not operating as designed, 36 percent were partially or totally clogged, and another 22 percent exhibited slow filtration. Longevity can be increased by careful geotechnical evaluation prior to construction and by designing and implementing an inspection and maintenance plan. Soil infiltration rates and the water table depth should be evaluated to ensure that conditions are satisfactory for proper operation of an infiltration trench. Pretreatment structures, such as a vegetated buffer strip or water quality inlet, can increase longevity by removing sediments, hydrocarbons, and other materials that may clog the trench. Regular maintenance, including the replacement of clogged aggregate, will also increase the effectiveness and life of the trench.

DESIGN CRITERIA

Prior to trench construction, a review of the design plans may be required by state and local governments. The design plans should include a geotechnical evaluation that determines the feasibility of using an infiltration trench at the site. Soils should have a low silt and clay content and have infiltration rates greater than 1.3 centimeters (0.5 inches) per hour. Acceptable soil texture classes include sand, loamy sand, sandy loam and loam. These soils are within the A or B hydrologic group. Soils in the C or D hydrologic groups should be avoided. Soil survey reports published by the Soil Conservation Service can be used to identify soil types and infiltration rates. However, sufficient soil borings should always be taken to verify site conditions. Feasible sites should have a minimum of 1.2 meters (4 feet) to bedrock in order to reduce excavation costs. There should also be at least 1.2 meters (4 feet) below the trench to the water table to prevent potential ground water problems. Trenches should also be located at least 30.5 meters (100 feet) upgradient from water supply wells and 30.5 meters (100 feet) from building foundations. Land availability, the depth to bedrock, and the depth to the water table will determine whether the infiltration trench is located underground or at grade. Underground trenches receive runoff through pipes or channels, whereas surface trenches collect sheet flow from the drainage area.

In general, infiltration trenches are suitable for drainage areas up to 4 hectares (10 acres) (SEWRPC, 1991, Harrington, 1989). However, when the drainage area exceeds 2 hectares (5 acres), other BMPs should be carefully considered. The drainage area must be fully developed and stabilized with vegetation before constructing an infiltration trench. High sediment loads from unstabilized areas will quickly clog the infiltration trench. Runoff from unstabilized areas should be diverted away from the trench into a construction BMP until vegetation is established.
The drainage area slope determines the velocity of the runoff and also influences the amount of pollutants entrained in the runoff. Infiltration trenches work best when the upgradient drainage area slope is less than 5 percent (SEWRPC, 1991). The downgradient slope should be no greater than 20 percent to minimize slope failure and seepage.

The trench surface may consist of stone or vegetation with inlets to evenly distribute the runoff entering the trench (SEWRPC, 1991, Harrington, 1989). Runoff can be captured by depressing the trench surface or by placing a berm at the down gradient side of the trench.

The basic infiltration trench design utilizes stone aggregate in the top of the trench to promote filtration; however, this design can be modified by substituting pea gravel for stone aggregate in the top 0.3 meter (1 foot) of the trench. The pea gravel improves sediment filtering and maximizes the pollutant removal in the top of the trench. When the modified trenches become clogged, they can generally be restored to full performance by removing and replacing only the pea gravel layer, without replacing the lower stone aggregate layers.

Infiltration trenches can also be modified by adding a layer of organic material (peat) or loam to the trench subsoil. This modification appears to enhance the removal of metals and nutrients through adsorption. The trenches are then covered with an impermeable geotextile membrane overlain with topsoil and grass (Figure 2).

A vegetated buffer strip (6.1 to 7.6 meters, or 20-25 feet, wide) should be established adjacent to the infiltration trench to capture large sediment particles in the runoff. The buffer strip should be installed immediately after trench construction using sod instead of hydroseeding (Schueler, 1987). The buffer strip should be graded with a slope between 0.5 and 15 percent so that runoff enters the trench as sheet flow. If runoff is piped or channeled to the trench, a level spreader must be installed to create sheet flow (Harrington, 1989).

During excavation and trench construction, only light equipment such as backhoes or wheel and ladder type trenchers should be used to minimize compaction of the surrounding soils. Filter fabric should be placed around the walls and bottom of the trench and 0.3 meters (1 foot) below the trench.
surface. The filter fabric should overlap each side of the trench in order to cover the top of the stone aggregate layer (see Figure 1). The filter fabric prevents sediment in the runoff and soil particles from the sides of the trench from clogging the aggregate. Filter fabric that is placed 0.3 meters (1 foot) below the trench surface will maximize pollutant removal within the top layer of the trench and decrease the pollutant loading to the trench bottom, reducing frequency of maintenance.

The required trench volume can be determined by several methods. One method calculates the volume based on capture of the first flush, which is defined as the first 1.3 centimeters (0.5 inches) of runoff from the contributing drainage area (SEWRPC, 1991). The State of Maryland (MD., 1986) also recommends sizing the trench based on the first flush, but defines first flush as the first 1.3 centimeters (0.5 inches) from the contributing impervious area. The Metropolitan Washington Council of Governments (MWCOC) suggests that the trench volume be based on the first 1.3 centimeters (0.5 inches) per impervious acre or the runoff produced from a 6.4 centimeter (2.5 inch) storm. In Washington D.C., the capture of 1.3 centimeters (0.5 inches) per impervious acre accounts for 40 to 50 percent of the annual storm runoff volume. The runoff not captured by the infiltration trench should be bypassed to another BMP (Harrington, 1989) if treatment of the entire runoff from the site is desired.

Trench depths are usually between 0.9 and 3.7 meters (3 and 12 feet) (SEWRPC, 1991, Harrington, 1989). However, a depth of 2.4 meters (8 feet) is most commonly used (Schueler, 1987). A site specific trench depth can be calculated based on the soil infiltration rate, aggregate void space, and the trench storage time (Harrington, 1989). The stone aggregate used in the trench is normally 2.5 to 7.6 centimeters (1 to 3 inches) in diameter, which provides a void space of 40 percent (SEWRPC, 1991, Harrington, 1989, Schueler, 1987).

A minimum drainage time of 6 hours should be provided to ensure satisfactory pollutant removal in the infiltration trench (Schueler, 1987, SEWRPC, 1991). Although trenches may be designed to provide temporary storage of storm water, the trench should drain prior to the next storm event. The drainage time will vary by precipitation zone. In the Washington, D.C. area, infiltration trenches are designed to drain within 72 hours.

An observation well is recommended to monitor water levels in the trench. The well can be a 10.2 to 15.2 centimeter (4 to 6 inch) diameter PVC pipe, which is anchored vertically to a foot plate at the bottom of the trench as shown in Figure 1 above. Inadequate drainage may indicate the need for maintenance.

**PERFORMANCE**

Infiltration trenches function similarly to rapid infiltration systems that are used in wastewater treatment. Estimated pollutant removal efficiencies from wastewater treatment performance and modeling studies are shown in Table 1.

Based on this data, infiltration trenches can be expected to remove up to 90 percent of sediments, metals, coliform bacteria and organic matter, and up to 60 percent of phosphorus and nitrogen in the runoff (Schueler, 1992). Biochemical oxygen demand (BOD) removal is estimated to be between 70 to 80 percent. Lower removal rates for nitrate, chlorides and soluble metals should be expected.

**TABLE 1 TYPICAL POLLUTANT REMOVAL EFFICIENCY**

<table>
<thead>
<tr>
<th>Pollutant</th>
<th>Typical Percent Removal Rates</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sediment</td>
<td>90%</td>
</tr>
<tr>
<td>Total Phosphorous</td>
<td>60%</td>
</tr>
<tr>
<td>Total Nitrogen</td>
<td>60%</td>
</tr>
<tr>
<td>Metals</td>
<td>90%</td>
</tr>
<tr>
<td>Bacteria</td>
<td>90%</td>
</tr>
<tr>
<td>Organic</td>
<td>90%</td>
</tr>
<tr>
<td>Biochemical Oxygen Demand</td>
<td>70-80%</td>
</tr>
</tbody>
</table>

especially in sandy soils (Schueler, 1992).

Pollutant removal efficiencies may be improved by using washed aggregate and adding organic matter and loam to the subsoil. The stone aggregate should be washed to remove dirt and fines before placement in the trench. The addition of organic material and loam to the trench subsoil will enhance metals and nutrient removal through adsorption.

OPERATION AND MAINTENANCE

Infiltration, as with all BMPs, must have routine inspection and maintenance designed into the life performance of the facility. Maintenance should be performed as indicated by these routine inspections. The principal maintenance objective is to prevent clogging, which may lead to trench failure. Infiltration trenches and any pretreatment BMPs should be inspected after large storm events and any accumulated debris or material removed. A more thorough inspection of the trench should be conducted at least annually. Annual inspection should include monitoring of the observation well to confirm that the trench is draining within the specified time. Trenches with filter fabric should be inspected for sediment deposits by removing a small section of the top layer. If inspection indicates that the trench is partially or completely clogged, it should be restored to its design condition.

When vegetated buffer strips are used, they should be inspected for erosion or other damage after each major storm event. The vegetated buffer strip should have healthy grass that is routinely mowed. Trash, grass clippings and other debris should be removed from the trench perimeter and should be disposed properly. Trees and other large vegetation adjacent to the trench should also be removed to prevent damage to the trench.

COSTS

Construction costs include clearing, excavation, placement of the filter fabric and stone, installation of the monitoring well, and establishment of a vegetated buffer strip. Additional costs include planning, geotechnical evaluation, engineering and permitting. The Southeastern Wisconsin Regional Planning Commission (SEWRPC, 1991) has developed cost curves and tables for infiltration trenches based on 1989 dollars. The 1993 construction cost for a relatively large infiltration trench (i.e., 1.8 meters (6 feet) deep and 1.2 meters (4 feet) wide with a 68 cubic meter (2,400 cubic feet) volume) ranges from $8,000 to $19,000. A smaller infiltration trench (i.e., 0.9 meters (3 feet) deep and 1.2 meters (4 feet) wide with a 34 cubic meter (1,200 cubic feet) volume) is estimated to cost from $3,000 to $8,500.

Maintenance costs include buffer strip maintenance and trench inspection and rehabilitation. SEWRPC (1991) has also developed maintenance costs for infiltration trenches. Based on the above examples, annual operation and maintenance costs would average $700 for the large trench and $325 for the small trench. Typically, annual maintenance costs are approximately 5 to 10 percent of the capital cost (Schueler, 1987). Trench rehabilitation, may be required every 5 to 15 years. Cost for rehabilitation will vary depending on site conditions and the degree of clogging. Estimated rehabilitation costs run from 15 to 20 percent of the original capital cost (SEWRPC, 1991).

REFERENCES


**ADDITIONAL INFORMATION**

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Infiltration Drainfields
DESCRIPTION

Infiltration drainfields are innovative technologies that are specially designed to promote storm water infiltration into subsoils. These drainfields help to control runoff and prevent the contamination of local watersheds. The system is usually composed of a pretreatment structure, a manifold system, and a drainfield. Runoff is first diverted into a storm sewer system that passes through a pretreatment structure such as an oil and grit separator. The oil and grit chamber effectively removes coarse sediment, oils, and grease from the runoff. The storm water runoff then continues through a manifold system into the infiltration drainfield. The manifold system consists of a perforated pipe which distributes the runoff evenly throughout the infiltration drainfield. The runoff then percolates through an underlying aggregate sand filter and filter fabric into the subsoils. An example of this system is provided in Figure 1.


FIGURE 1 TYPICAL INFILTRATION DRAINFIELD SCHEMATIC
Common design modifications to the infiltration drainfield best management practice (BMP) include the installation of porous pavement surrounded by a grass filter strip over the infiltration drainfield or the insertion of an emergency overflow pipe in the oil and grit pretreatment chamber. The overflow pipe allows runoff volumes exceeding design capacities to discharge directly to a trunk storm sewer system.

APPLICABILITY

Infiltration drainfields are most applicable on sites with a relatively small drainage area (less than 15 acres.) They can be used to control runoff from parking lots, rooftops, impervious storage areas, or other land uses. Infiltration drainfields should not be used in locations that receive a large sediment load that could clog the pretreatment system, which in turn would plug the infiltration drainfield and reduce its effectiveness.

Soils in areas where the installation of an infiltration drainfield is being considered should have field-verified permeability rates of greater than 0.5 inches per hour and should include a 4-foot minimum clearance between the bottom of the system and the bedrock or the water table.

ADVANTAGES AND DISADVANTAGES

The use of infiltration drainfields may be restricted in regions with colder climates, arid regions, regions with high wind erosion rates (because of increased windblown sediment loads), and areas of sole source aquifers. Some specific limitations of infiltration drainfields include:

- High maintenance when sediment loads to the drainfield are heavy.
- High costs of engineering design, excavation, fill material, and pretreatment systems.
- Short life span if not well maintained.
- Not suitable for use in regions with clay or silty soils.
- Not suitable for use in regions where groundwater is used locally for human consumption.
- Anaerobic conditions that could clog the soil and reduce the capacity and performance of the system may develop in underlying soils if there is not sufficient time between storm events to allow the soil to dry out.

One potential negative impact of infiltration drainfields is the risk of groundwater contamination. Studies to date do not indicate that this is a major risk if site suitability guidelines are observed. However, migration of nitrates and chlorides from the drainfield has been documented.

Additional questions regarding infiltration drainfields remain to be answered:

- Is the oil and grit separator the most effective pretreatment system to protect infiltration capacity?
- What are the pollutant removal capacities of infiltration drainfields with various pretreatment systems?
- Is the performance of infiltration drainfields better than the performance of infiltration basins and trenches during subfreezing weather and snow melt runoff conditions?
- What level of maintenance is required to ensure proper performance?

DESIGN CRITERIA

Infiltration drainfields, along with most other infiltration BMP structures (infiltration trenches, basins, etc.) have proved to have short life spans in the past. Failure of the systems has been attributed to poor design, inadequate construction techniques, low permeability soils, and a lack of pretreatment. Some design factors which could significantly increase the longevity of infiltration drainfields and other infiltration processes are shown in Table 1.
### TABLE 1 INFILTRATION DRAINFIELD DESIGN CRITERIA

<table>
<thead>
<tr>
<th>Design Criteria</th>
<th>Guidelines</th>
</tr>
</thead>
<tbody>
<tr>
<td>Site Evaluation</td>
<td>Take soil borings to a depth of at least 4 feet below bottom of stone reservoir to check for soil permeability, porosity, depth to seasonally high water table, and depth to bedrock. Not recommended on slopes greater than 5 percent and best when slopes are as flat as possible. Minimum infiltration rate 3 feet below bottom of stone reservoir: 0.5 inches per hour. Minimum depth to bedrock and seasonally high water table: 4 feet. Minimum setback from building foundations: 10 feet downgradient, 100 feet upgradient. Drainage area should be less than 15 acres.</td>
</tr>
<tr>
<td>Design Storm Storage Volume</td>
<td>Literature values suggest this parameter is highly variable and dependent upon regulatory requirements. One typically recommended storage volume is the stormwater runoff volume produced in the tributary watershed by the 6-month, 24-hour duration storm event.</td>
</tr>
<tr>
<td>Drainage Time for Design Storm</td>
<td>Minimum: 12 hours. Maximum: 72 hours. Recommended: 24 hours.</td>
</tr>
<tr>
<td>Construction</td>
<td>Excavate and grade with light equipment with tracks or oversized tires to prevent soil compaction. As needed, divert stormwater runoff away from site before and during construction. A typical infiltration cross-section consists of the following: 1) a stone reservoir consisting of coarse 1.5 to 3-inch diameter stone (washed); 2) 6 to 12-inch sand filter at the bottom of the drainfield; and 3) filter fabric.</td>
</tr>
<tr>
<td>Pretreatment</td>
<td>Pretreatment is recommended to treat runoff from all contributing areas.</td>
</tr>
<tr>
<td>Dispersion Manifold</td>
<td>A dispersion manifold should be placed in the upper portions of the infiltration drainfield. The purpose of this manifold is to evenly distribute stormwater runoff over the largest possible area. Two to four manifold extension pipes are recommended for most typical infiltration drainfield applications.</td>
</tr>
</tbody>
</table>


### PERFORMANCE

The effectiveness of infiltration drainfields depends upon their design. When runoff enters the drainfield, 100 percent of the pollutants are prevented from entering surface water. Any water that bypasses the pretreatment system and drainfield will not be treated. Pollutant removal mechanisms include absorption and adsorption, straining, microbial decomposition in the soil below the drainfield, and trapping of sediment, grit, and oil in the pretreatment chamber.

Currently there is little monitoring data on the performance of infiltration drainfields. However, some monitoring data is available on porous pavements. The design criteria for porous pavements is very similar to the design criteria of infiltration drainfields. An estimate of porous pavement pollutant removal efficiencies ranges between 82 and 95 percent for sediment, 65 percent for total phosphorus, and 80 to 85 percent for total nitrogen. Porous pavement works most effectively for about 6 months.

Some key factors to increase pollutant removal efficiencies include:

- Properly maintaining the system.
- Implementing good housekeeping practices in the tributary drainage area.
- Allowing sufficient drying time (approximately 24 hours) between storm events.
Choose a site with highly permeable soils and subsoils.

- Incorporating a pretreatment system.
- Ensuring that there is sufficient organic matter in subsoils.
- Using a sand layer on top of a filter fabric at the bottom of the drainfield.

OPERATION AND MAINTENANCE

Routine maintenance of infiltration drainfields is extremely important. The pretreatment grit chamber should be checked at least four times per year and after major storm events. Sediment should be cleaned out when the sediment depletes more than 10 percent of the available infiltration capacity. This can be done manually or by vacuum pump. Inlet and outlet pipes should also be inspected at this time.

The infiltration drainfield should contain an observation well that can provide information on how well the system is operating. It is recommended that the observation well be monitored daily after runoff-producing storm events. If the infiltration drainfield does not drain after three days, it usually means that the drainfield is clogged. Once the performance characteristics of the structure have been verified, the monitoring schedule can be reduced to a monthly or quarterly basis.

COSTS

There is little information on the cost of infiltration drainfields. However, the construction costs for installing an infiltration drainfield that is 30.5 meters (100 feet) long, 15 meters (50 feet) wide, 2.4 meters (8 feet) deep and with 1.2 meters (4 feet) of cover can be estimated using the general information in Table 2.

REFERENCES


ADDITIONAL INFORMATION

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TABLE 2  ESTIMATED COST FOR INSTALLING AN INFILTRATION DRAINFIELD

<table>
<thead>
<tr>
<th>Item</th>
<th>Cost</th>
</tr>
</thead>
<tbody>
<tr>
<td>Excavation Costs: 2,220 yd³ @ $5.00/yd³</td>
<td>$11,100</td>
</tr>
<tr>
<td>Stone Fill: (1,296 yd³)($20.00/yd³)</td>
<td>$25,920</td>
</tr>
<tr>
<td>Sand Fill: (185 yd³)($10.00/yd³)</td>
<td>$1,850</td>
</tr>
<tr>
<td>Filter Fabric: Top and Bottom= 10,000 ft²</td>
<td>$4,550</td>
</tr>
<tr>
<td>Slides= 1,600 + 800= 2,400 ft² +10%= 13,540 ft²</td>
<td></td>
</tr>
<tr>
<td>(13,540 ft²)(1 yd³ / 8 ft³)($3.00/yd³)</td>
<td>$4,550</td>
</tr>
<tr>
<td>Perforated Manifold and Inlet Pipe: 75 ft + (4)(40 ft)= 235 ft + 40 ft = 275 ft (275)($10.00/ft)</td>
<td>$2,750</td>
</tr>
<tr>
<td>Observation Well: 1 at $500 each</td>
<td>$500</td>
</tr>
<tr>
<td>Pretreatment Chamber: 1 at $10,000</td>
<td>$10,000</td>
</tr>
<tr>
<td>Miscellaneous (back filling, overflow pipe, sodding, etc.):</td>
<td>$1000</td>
</tr>
<tr>
<td>Subtotal:</td>
<td>$57,670</td>
</tr>
<tr>
<td>Contingencies (engineering, administration, permits, etc.)= 25%</td>
<td>$14,420</td>
</tr>
<tr>
<td>Total:</td>
<td>$72,090</td>
</tr>
</tbody>
</table>

Note: Unit price will vary greatly depending upon local market conditions


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Porous Pavement
Storm Water Technology Fact Sheet
Porous Pavement

DESCRIPTION

Porous pavement is a special type of pavement that allows rain and snowmelt to pass through it, thereby reducing the runoff from a site and surrounding areas. In addition, porous pavement filters some pollutants from the runoff if maintained.

There are two types of porous pavement: porous asphalt and pervious concrete. 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.

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 1 illustrates a common porous asphalt pavement installation.

Two common modifications made in designing porous pavement systems are (1) varying the amount of storage in the stone reservoir beneath the pavement and (2) adding perforated pipes near the top of the reservoir to discharge excess storm water after the reservoir has been filled.

Some municipalities have also added storm water reservoirs (in addition to stone reservoirs) beneath the pavement. These reservoirs should be designed to accommodate runoff from a design storm and should provide for infiltration through the underlying subsoil.

APPLICABILITY

Porous pavement may substitute for conventional pavement on parking areas, areas with light traffic, and the shoulders of airport taxiways and runways, provided that the grades, subsoils, drainage characteristics, and groundwater conditions are suitable. Slopes should be flat or very gentle. Soils should have field-verified permeability rates of greater than 1.3 centimeters (0.5 inches) per hour, and there should be a 1.2 meter (4-foot) minimum clearance from the bottom of the system to bedrock or the water table.

ADVANTAGES AND DISADVANTAGES

The advantages of using porous pavement include:

- Water treatment by pollutant removal.
- Less need for curbing and storm sewers.
- Improved road safety because of better skid resistance.
- Recharge to local aquifers.

The use of porous pavement may be restricted in cold regions, arid regions or regions with high wind erosion rates, and areas of sole-source aquifers. The use of porous pavement is highly constrained, requiring deep permeable soils, restricted traffic, and adjacent land
uses. 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.
- 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.
- Some building codes may not allow for its installation.
- Anaerobic conditions may develop in underlying soils if the soils are unable to dry out between storm events. This may impede microbiological decomposition.

As noted above, the use of porous pavement does create risk of groundwater contamination. Pollutants that are not easily trapped, adsorbed, or reduced, such as nitrates and chlorides, may continue to move through the soil profile and into the groundwater, possibly contaminating drinking water supplies. Therefore, until more scientific data is available, it is not advisable to construct porous pavement near groundwater drinking supplies.
In addition to these documented pros and cons of porous pavements, several questions remain regarding their use. These include:

- Whether porous pavement can maintain its porosity over a long period of time, particularly with resurfacing needs and snow removal.
- Whether porous pavement remains capable of removing pollutants after subfreezing weather and snow removal.
- The cost of maintenance and rehabilitation options for restoration of porosity.

**DESIGN CRITERIA**

Porous pavement - along with other infiltration technologies like infiltration basins and trenches - have demonstrated a short life span. Failures generally have been attributed to poor design, poor construction techniques, subsoils with low permeability, and lack of adequate preventive maintenance. Key design factors that can increase the performance and reduce the risk of failure of porous pavements (and other infiltration technologies) include:

- Site conditions;
- Construction materials; and
- Installation methods.

These factors are discussed further in Table 1.

**PERFORMANCE**

Porous pavement pollutant removal mechanisms include absorption, straining, and microbiological decomposition in the soil. An estimate of porous pavement pollutant removal efficiency is provided by two long-term monitoring studies conducted in Rockville, MD, and Prince William, VA. These studies indicate removal efficiencies of between 82 and 95 percent for sediment, 65 percent for total phosphorus, and between 80 and 85 percent of total nitrogen. The Rockville, MD, site also indicated high removal rates for zinc, lead, and chemical oxygen demand. Some key factors to increase pollutant removal include:

- Routine vacuum sweeping and high pressure washing (with proper disposal of removed material).
- Drainage time of at least 24 hours.
- Highly permeable soils.
- Pretreatment of runoff from site.
- Organic matter in subsoils.
- Clean-washed aggregate.

Traditionally, porous pavement sites have had a high failure rate - approximately 75 percent. Failure has been attributed to poor design, inadequate construction techniques, soils with low permeability, heavy vehicular traffic, and resurfacing with nonporous pavement materials. Factors enhancing longevity include:

- Vacuum sweeping and high-pressure washing.
- Use in low-intensity parking areas.
- Restrictions on use by heavy vehicles.
- Limited use of de-icing chemicals and sand.
- Resurfacing.
- Inspection and enforcement of specifications during construction.
- Pretreatment of runoff from offsite.
- Implementation of a stringent sediment control plan.

**OPERATION AND MAINTENANCE**

Porous pavements need to be maintained. Maintenance should include vacuum sweeping at least four times a year (with proper disposal of
<table>
<thead>
<tr>
<th>Design Criterion</th>
<th>Guidelines</th>
</tr>
</thead>
<tbody>
<tr>
<td>Site Evaluation</td>
<td>Take soil boring to a depth of at least 1.2 meters (4 feet) below bottom of stone reservoir to check for soil permeability, porosity, depth of seasonally high water table, and depth to bedrock.</td>
</tr>
<tr>
<td></td>
<td>Not recommended on slopes greater than 5 percent and best with slopes as flat as possible.</td>
</tr>
<tr>
<td></td>
<td>Minimum infiltration rate 0.9 meters (3 feet) below bottom of stone reservoir: 1.3 centimeters (0.5 inches) per hour.</td>
</tr>
<tr>
<td></td>
<td>Minimum depth to bedrock and seasonally high water table: 1.2 meters (4 feet).</td>
</tr>
<tr>
<td></td>
<td>Minimum setback from water supply wells: 30 meters (100 feet).</td>
</tr>
<tr>
<td></td>
<td>Minimum setback from building foundations: 3 meters (10 feet) downgradient, 30 meters (100 feet) upgradient.</td>
</tr>
<tr>
<td></td>
<td>Not recommended in areas where wind erosion supplies significant amounts of windblown sediment.</td>
</tr>
<tr>
<td></td>
<td>Drainage area should be less than 6.1 hectares (15 acres).</td>
</tr>
<tr>
<td>Traffic conditions</td>
<td>Use for low-volume automobile parking areas and lightly used access roads.</td>
</tr>
<tr>
<td></td>
<td>Avoid moderate to high traffic areas and significant truck traffic.</td>
</tr>
<tr>
<td></td>
<td>Avoid snow removal operations; post with signs to restrict the use of sand, salt, and other deicing chemicals typically associated with snow clearing activities.</td>
</tr>
<tr>
<td>Design Storm Storage Volume</td>
<td>Highly variable; depends upon regulatory requirements. Typically design for storm water runoff volume produced in the tributary watershed by the 5-month, 24-hour duration storm event.</td>
</tr>
<tr>
<td>Drainage Time for Design Storm</td>
<td>Minimum: 12 hours.</td>
</tr>
<tr>
<td></td>
<td>Maximum: 72 hours.</td>
</tr>
<tr>
<td></td>
<td>Recommended: 24 hours.</td>
</tr>
<tr>
<td>Construction</td>
<td>Excavate and grade with light equipment with tracks or oversized tires to prevent soil compaction.</td>
</tr>
<tr>
<td></td>
<td>As needed, divert storm water runoff away from planned pavement area before and during construction.</td>
</tr>
<tr>
<td></td>
<td>A typical porous pavement cross-section consists of the following layers: 1) porous asphalt course, 6-10 centimeters (2-4 inches) thick; 2) filter aggregate course; 3) reservoir course of 4-8 centimeters (1.5-3-inch) diameter stones; and 4) filter fabric.</td>
</tr>
<tr>
<td>Porous Pavement Placement</td>
<td>Paving temperature: 240° - 260° F.</td>
</tr>
<tr>
<td></td>
<td>Minimum air temperature: 50° F.</td>
</tr>
<tr>
<td></td>
<td>Compact with one or two passes of a 10,000-kilogram (10-ton) roller.</td>
</tr>
<tr>
<td></td>
<td>Prevent any vehicular traffic on pavement for at least two days.</td>
</tr>
<tr>
<td>Pretreatment</td>
<td>Pretreatment recommended to treat runoff from off-site areas. For example, place a 7.6-meter (25-foot) wide vegetative filter strip around the perimeter of the porous pavement where drainage flows onto the pavement surface.</td>
</tr>
</tbody>
</table>
removed material), followed by high-pressure hosing to
free pores in the top layer from clogging. Potholes and
cracks can be filled with patching mixes unless more
than 10 percent of the surface area needs repair.
Spot-clogging may be fixed by drilling 1.3 centimeter
(half-inch) holes through the porous pavement layer
every few feet.

The pavement should be inspected several times during
the first few months following installation and annually
thereafter. Annual inspections should take place after
large storms, when puddles will make any clogging
obvious. The condition of adjacent pretreatment
devices should also be inspected.

COSTS

The costs associated with developing a porous
pavement system are illustrated in Table 2.

Estimated costs for an average annual maintenance
program of a porous pavement parking lot are
approximately $4,942 per hectare per year ($200 per
acre per year). This cost assumes four inspections
each year with appropriate jet hosing and vacuum
sweeping treatments.

<table>
<thead>
<tr>
<th>Component</th>
<th>Unit Cost</th>
<th>Total</th>
</tr>
</thead>
<tbody>
<tr>
<td>Excavation Costs</td>
<td>740 cy X $5.00/cy</td>
<td>$3,700</td>
</tr>
<tr>
<td>Filter Aggregate/Stone Fill</td>
<td>740 cy X $20.00/cy</td>
<td>$14,800</td>
</tr>
<tr>
<td>Filter Fabric</td>
<td>760 sq X $3.00/sq</td>
<td>$2,280</td>
</tr>
<tr>
<td>Porous Pavement</td>
<td>555 sq X $13.00/sq</td>
<td>$7,228</td>
</tr>
<tr>
<td>Overflow Pipes</td>
<td>200 ft X $12.00/ft</td>
<td>$2,400</td>
</tr>
<tr>
<td>Observation Well</td>
<td>1 at $200 each</td>
<td>$200</td>
</tr>
<tr>
<td>Grass Buffer</td>
<td>822 sq X $1.50/sq</td>
<td>$1,250</td>
</tr>
<tr>
<td>Erosion Control</td>
<td>$1000</td>
<td>$1,000</td>
</tr>
<tr>
<td>Subtotal</td>
<td></td>
<td>$32,858</td>
</tr>
<tr>
<td>Contingencies (Engineering,</td>
<td>25%</td>
<td>$8,215</td>
</tr>
<tr>
<td>Administration, etc.)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Total</td>
<td></td>
<td>$41,073</td>
</tr>
</tbody>
</table>

REFERENCES

   Porous Pavement Research.” Water
   265-267.

2. Metropolitan Washington Council of
   Runoff: A Practical Manual for Planning
   and Designing Urban BMPs.

3. Metropolitan Washington Council of
   Governments, 1992. A Current Assessment
   of Best Management Practices: Techniques
   for Reducing Nonpoint Source Pollution in
   a Coastal Zone.

4. Southeastern Wisconsin Regional Planning
   Nonpoint Source Water Pollution Control


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Washington, DC, 20460
Post-Construction Storm Water Management in New Development & Redevelopment

Alternative Pavers

Description

Alternative pavers are permeable surfaces that can replace asphalt and concrete and can be used for driveways, parking lots, and walkways. From a storm water perspective, this is important because alternative pavers can replace impervious surfaces, creating less storm water runoff. The two broad categories of alternative pavers are paving blocks and other surfaces, including gravel, cobbles, wood, mulch, brick, and natural stone. While porous pavement is an alternative paver, as an engineered storm water management practice it is discussed in detail in the Porous Pavement fact sheet.

Paving Blocks

Paving blocks are concrete or plastic grids with gaps between them. Paving blocks make the surface more rigid and gravel or grass planted inside the holes allows for infiltration. Depending on the use and soil types, a gravel layer can be added underneath to prevent settling and allow further infiltration.

Other Alternative Surfaces

Gravel, cobbles, wood, and mulch also allow varying degrees of infiltration. Brick and natural stone arranged in a loose configuration allow for some infiltration through the gaps. Gravel and cobbles can be used as driveway material, and wood and mulch can be used to provide walking trails.

Applicability
Alternative pavers can replace conventional asphalt or concrete in parking lots, driveways, and walkways. At the same time, traffic volume and type can limit application. For this reason, alternative pavers for parking are recommended only for overflow areas. In residential areas, alternative surfaces can be used for driveways and walkways, but are not ideal for areas that require handicap accessibility.

Siteing and Design Criteria

Accessibility, climate, soil type, traffic volume, and long-term performance should be considered, along with costs and storm water quality controls, when choosing paving materials. Use of alternative pavers in cold climates will require special consideration, as snow shovels are not practical for many of these surfaces. Sand is particularly troublesome if used with paving blocks, as the sand that ends up between the blocks cannot effectively wash away or be removed. In addition, salt used to de-ice can also infiltrate directly into the soil and cause potential ground water pollution.

Soil types will affect the infiltration rates and should be considered when using alternative pavers. Clayey soils (D soils) will limit the infiltration on a site. If ground water pollution is a concern, use of alternative pavers with porous soils should be carefully considered.

The durability and maintenance cost of alternative pavers also limits use to low-traffic-volume areas. At the same time, alternative pavers can abate storm water management costs. Used in combination with other better-site-design techniques, the cumulative effect on storm water can be dramatic.

Limitations

Alternative pavers are not recommended for high-traffic volumes for durability reasons. Access for wheelchairs is limited with alternative pavers. In addition, snow removal is difficult since plows cannot be used, sand can cause the system to clog, and salt can be a potential pollutant.

Maintenance Considerations

Alternative pavers require periodic maintenance, and costs increase when the permeable surface must be restored.

Effectiveness

The most obvious benefit of utilizing alternative pavers includes reduction or elimination of other storm water management techniques. Applied in combination with other techniques such as bioretention and green parking, pollutant removal and storm water management can be further improved. (see Bioretention and Green Parking fact sheets for more information.)

Alternative pavers all provide better water quality improvement than conventional asphalt or concrete, and the range of improvement depends on the type of paver used. Table 1 provides a list of pavers and the range of water quality improvement achievable by different types of alternative pavers.

Table 1. Water quality improvement of various pavers (Source: BASMAA, 1997)

<table>
<thead>
<tr>
<th>Material</th>
<th>Water Quality Effectiveness</th>
</tr>
</thead>
<tbody>
<tr>
<td>Conventional Asphalt/Concrete</td>
<td>Low</td>
</tr>
<tr>
<td>Brick (in a loose configuration)</td>
<td>Medium</td>
</tr>
<tr>
<td>Natural Stone</td>
<td>Medium</td>
</tr>
<tr>
<td>Gravel</td>
<td>High</td>
</tr>
<tr>
<td>Wood Mulch</td>
<td>High</td>
</tr>
<tr>
<td>Cobble</td>
<td>Medium</td>
</tr>
</tbody>
</table>

Cost Considerations

The range of installation and maintenance costs of various pavers is provided in
Table 2. Depending on the material used, installation costs can be higher or lower for alternative pavers than for conventional asphalt or concrete, but maintenance costs are almost always higher.

<table>
<thead>
<tr>
<th>Material</th>
<th>Installation Cost</th>
<th>Maintenance Cost</th>
</tr>
</thead>
<tbody>
<tr>
<td>Conventional Asphalt/Concrete</td>
<td>Medium</td>
<td>Low</td>
</tr>
<tr>
<td>Brick (in a loose configuration)</td>
<td>High</td>
<td>Medium</td>
</tr>
<tr>
<td>Natural Stone</td>
<td>High</td>
<td>Medium</td>
</tr>
<tr>
<td>Gravel</td>
<td>Low</td>
<td>Medium</td>
</tr>
<tr>
<td>Wood Mulch</td>
<td>Low</td>
<td>Medium</td>
</tr>
<tr>
<td>Cobblestone</td>
<td>Low</td>
<td>Medium</td>
</tr>
</tbody>
</table>

Reference


Information Sources


Bioretention
Storm Water
Technology Fact Sheet
Bioretention

DESCRIPTION

Bioretention is a best management practice (BMP) developed in the early 1990's by the Prince George's County, MD, Department of Environmental Resources (PGDER). Bioretention utilizes soils and both woody and herbaceous plants to remove pollutants from storm water runoff. As shown in Figure 1, runoff is conveyed as sheet flow to the treatment area, which consists of a grass buffer strip, sand bed, ponding area, organic layer or mulch layer, planting soil, and plants. Runoff passes first over or through a sand bed, which slows the runoff's velocity, distributes it evenly along the length of the ponding area, which consists of a surface organic layer and/or ground cover and the underlying planting soil. The ponding area is graded, its center depressed. Water is ponded to a depth of 15 centimeters (6 inches) and gradually infiltrates the bioretention area or is

FIGURE 1 BIORETENTION AREA

evapotranspired. The bioretention area is graded to divert excess runoff away from itself. Stored water in the bioretention area planting soil exfiltrates over a period of days into the underlying soils.

The basic bioretention design shown in Figure 1 can be modified to accommodate more specific needs. The City of Alexandria, VA, has modified the bioretention BMP design to include an underdrain within the sand bed to collect the infiltrated water and discharge it to a downstream sewer system. This modification was required because impervious subsoils and marine clays prevented complete infiltration in the soil system. This modified design makes the bioretention area act more as a filter that discharges treated water than as an infiltration device. Design modifications are also being reviewed that will potentially include both aerobic and anaerobic zones in the treatment area. The anaerobic zone will promote denitrification.

APPLICABILITY

Bioretention typically treats storm water that has run over impervious surfaces at commercial, residential, and industrial areas. For example, bioretention is an ideal storm water management BMP for median strips, parking lot islands, and swales. These areas can be designed or modified so that runoff is either diverted directly into the bioretention area or conveyed into the bioretention area by a curb and gutter collection system. Bioretention is usually best used upland from inlets that receive sheet flow from graded areas and at areas that will be excavated. The site must be graded in a manner that minimizes erosive conditions as sheet flow is conveyed to the treatment area, maximizing treatment effectiveness. Construction of bioretention areas is best suited to sites where grading or excavation will occur in any case so that the bioretention area can be readily incorporated into the site plan without further environmental damage. Bioretention should be used in stabilized drainage areas to minimize sediment loading in the treatment area. As with all BMPs, a maintenance plan must be developed.

Bioretention has been used as a storm water BMP since 1992. In addition to Prince George's County and Alexandria, bioretention has been used successfully at urban and suburban areas in Montgomery County, MD; Baltimore County, MD; Chesterfield County, VA; Prince William County, VA; Smith Mountain Lake State Park, VA; and Cary, NC.

ADVANTAGES AND DISADVANTAGES

Bioretention is not an appropriate BMP at locations where the water table is within 1.8 meters (6 feet) of the ground surface and where the surrounding soil stratum is unstable. In cold climates the soil may freeze, preventing runoff from infiltrating into the planting soil. The BMP is also not recommended for areas with slopes greater than 20 percent, or where mature tree removal would be required. Clogging may be a problem, particularly if the BMP receives runoff with high sediment loads.

Bioretention provides storm water treatment that enhances the quality of downstream water bodies. Runoff is temporarily stored in the BMP and released over a period of four days to the receiving water. The BMP is also able to provide shade and wind breaks, absorb noise, and improve an area's landscape.

DESIGN CRITERIA

Design details have been specified by the Prince George's County DER in a document entitled Design Manual for the Use of Bioretention in Storm Water Management (PGDER, 1993). The specifications were developed after extensive research on soil adsorption capacities and rates, water balance, plant pollutant removal potential, plant adsorption capacities and rates, and maintenance requirements. A case study was performed using the specifications at three commercial sites and one residential site in Prince George's County, Maryland.

Each of the components of the bioretention area is designed to perform a specific function. The grass buffer strip reduces incoming runoff velocity and filters particulates from the runoff. The sand bed also reduces the velocity, filters particulates, and spreads flow over the length of the bioretention
Aeration and drainage of the planting soil are provided by the 0.5 meter (18 inch) deep sand bed. The ponding area provides a temporary storage location for runoff prior to its evaporation or infiltration. Some particulates not filtered out by the grass filter strip or the sand bed settle within the ponding area.

The organic or mulch layer also filters pollutants and provides an environment conducive to the growth of microorganisms, which degrade petroleum-based products and other organic material. This layer acts in a similar way to the leaf litter in a forest and prevents the erosion and drying of underlying soils. Planted ground cover reduces the potential for erosion as well, slightly more effectively than mulch. The maximum sheet flow velocity prior to erosive conditions is 0.3 meters per second (1 foot per second) for planted ground cover and 0.9 meters per second (3 feet per second) for mulch.

The clay in the planting soil provides adsorption sites for hydrocarbons, heavy metals, nutrients and other pollutants. Storm water storage is also provided by the voids in the planting soil. The stored water and nutrients in the water and soil are then available to the plants for uptake.

The layout of the bioretention area is determined after site constraints such as location of utilities, underlying soils, existing vegetation, and drainage are considered. Sites with loamy sand soils are especially appropriate for bioretention because the excavated soil can be backfilled and used as the planting soil, thus eliminating the cost of importing planting soil. An unstable surrounding soil stratum (e.g., Marlboro Clay) and soils with a clay content greater than 25 percent may preclude the use of bioretention, as would a site with slopes greater than 20 percent or a site with mature trees that would be removed during construction of the BMP. Bioretention can be designed to be off-line or on-line of the existing drainage system. The "first flush" of runoff is diverted to the off-line system. The first flush of runoff is the initial runoff volume that typically contains higher pollutant concentrations than those in the extended runoff period. On-line systems capture the first flush but that volume of water will likely be washed out by subsequent runoff resulting in a release of the captured pollutants. The size of the drainage area for one bioretention area should be between 0.1 and 0.4 hectares (0.25 and 1.0 acres). Multiple bioretention areas may be required for larger drainage areas. The maximum drainage area for one bioretention area is determined by the amount of sheet flow generated by a 10-year storm. Flows greater than 141 liters per second (5 cubic feet per second) may potentially erode stabilized areas. In Maryland, such a flow generally occurs with a 10-year storm at one-acre commercial or residential sites. The designer should determine the potential for erosive conditions at the site.

The size of the bioretention area is a function of the drainage area and the runoff generated from the area. The size should be 5 to 7 percent of the drainage area multiplied by the rational method runoff coefficient, "c," determined for the site. The 5 percent specification applies to a bioretention area that includes a sand bed; 7 percent to an area without one. An example of sizing a facility is shown in Figure 2. For this discussion, sizing specifications are based on 1.3 to 1.8 centimeters (0.5 to 0.7 inches) of precipitation over a 6-hour period (the mean storm event for the Baltimore-Washington area), infiltrating into the bioretention area. Other areas with different mean storm events will need to account for the difference in the design of the BMP. Recommended minimum dimensions of the bioretention area are 4.6 meters (15 feet) wide by 12.2 meters (40 feet) in length. The minimum width allows enough space for a dense, randomly-distributed area of trees and shrubs to become established that replicates a natural forest and creates a microclimate. This enables the bioretention area to tolerate the effects of heat stress, acid rain, runoff pollutants, and insect and disease infestations which landscaped areas in urban settings typically are unable to tolerate. The preferred width is 7.6 meters (25 feet), with a length of twice the width. Any facilities wider than 6.1 meters (20 feet) should be twice as long as they are wide. This length requirement promotes the distribution of flow and decreases the chances of concentrated flow.

The maximum recommended ponding depth of the bioretention area is 15 centimeters (6 inches). This
depth provides for adequate storage and prevents water from standing for excessive periods of time. Because of some plants' water intolerance, water left to stand for longer than four days restricts the type of plants that can be used. Further, mosquitoes and other insects may start to breed if water is standing for longer than four days.

The appropriate planting soil should be backfilled into the excavated bioretention area. Planting soils should be sandy loam, loamy sand, or loam texture with a clay content ranging from 10 to 25 percent. The soil should have infiltration rates greater than 1.25 centimeters (0.5 inches) per hour, which is typical of sandy loams, loamy sands, or loams. Silt loams and clay loams generally have rates of less than 0.68 centimeters (0.27 inches) per hour. The pH of the soil should be between 5.5 and 6.5. Within this pH range, pollutants (e.g., organic nitrogen and phosphorus) can be adsorbed by the

---

**FIGURE 2 BIORETENTION AREA SIZING**

<table>
<thead>
<tr>
<th>DEVELOPMENT</th>
<th>AREA (SQ. FT)</th>
<th>&quot;C&quot; FACTOR</th>
<th>C X AREA</th>
</tr>
</thead>
<tbody>
<tr>
<td>PAVEMENT</td>
<td>23,800</td>
<td>0.90</td>
<td>21,400</td>
</tr>
<tr>
<td>GRASS</td>
<td>15,100</td>
<td>0.25</td>
<td>3,800</td>
</tr>
<tr>
<td><strong>TOTALS</strong></td>
<td><strong>32,900</strong></td>
<td></td>
<td><strong>25,200</strong></td>
</tr>
</tbody>
</table>

**BIORETENTION AREA SIZE**

1. With Sand Bed (5% Sum of C x Area)
   = 93 x 23,800 = 1,195 OR SAY 1,200 sq. ft.
2. Without Sand Bed (7% Sum of C x Area)
   = 87 x 23,800 = 2,077 OR SAY 2,000 sq. ft.

* SEE CHAPTER IV, PRINCE GEORGE'S COUNTY STORMWATER MANAGEMENT MANUAL

soil and microbial activity can flourish. Other requirements for the planting soil are a 1.5 to 3 percent organic content and a maximum 500 ppm concentration of soluble salts. In addition, criteria for magnesium, phosphorus, and potassium are 39.2 kilograms per acre (35 pounds per acre), 112 kilograms per acre (100 pounds per acre), and 95.2 kilograms per acre (85 pounds per acre), respectively. Soil tests should be performed for every 382 cubic yards (500 cubic yards) of planting soil, with the exception of pH and organic content tests, which are required only once per bioretention area.

Planting soil should be 10.1 centimeters (4 inches) deeper than the bottom of the largest root ball and 1.2 meters (4 feet) altogether. This depth will provide adequate soil for the plants' root systems to become established and prevent plant damage due to severe wind. A soil depth of 1.2 meters (4 feet) also provides adequate moisture capacity. To obtain the recommended depth, most sites will require excavation. Planting soil depths of greater than 1.2 meters (4 feet) may require additional construction practices (e.g., shoring measures). Planting soil should be placed in 18 inches or greater lifts and lightly compacted until the desired depth is reached. The bioretention area should be vegetated to resemble a terrestrial forest community ecosystem, which is dominated by understory trees (high canopy trees may be destroyed during maintenance) and has discrete soil zones as well as a mature canopy and a distinct sub-canopy of understory trees, a shrub layer, and herbaceous ground covers. Three species each of both trees and shrubs are recommended to be planted at a rate of 2500 trees and shrubs per hectare (1000 per acre). For example, a 4.6 meter (15 foot) by 12.2 meter (40 foot) bioretention area (55.75 square meters or 600 square feet) would require 14 trees and shrubs. The shrub-to-tree ratio should be 2:1 to 3:1. On average, the trees should be spaced 3.65 meters (12 feet) apart and the shrubs should be spaced 2.4 meters (8 feet) apart. In the metropolitan Washington, D.C., area, trees and shrubs should be planted from mid-March through the end of June or from mid-September through mid-November. Planting periods in other areas of the U.S. will vary. Vegetation should be watered at the end of each day for fourteen days following its planting.

Native species that are tolerant to pollutant loads and varying wet and dry conditions should be used in the bioretention area. These species can be determined from several published sources, including Native Trees, Shrubs, and Vines for Urban and Rural America (Highthouse, 1988). The designer should assess aesthetics, site layout, and maintenance requirements when selecting plant species. Adjacent non-native invasive species should be identified and the designer should take measures (e.g., provide a soil breach) to eliminate the threat of these species invading the bioretention area. Regional landscaping manuals should be consulted to ensure that the planting of the bioretention area meets the landscaping requirements established by the local authorities.

The optimal placement of vegetation within the bioretention area should be evaluated by the designers. Plants should be placed at irregular intervals to replicate a natural forest. Shade and shelter from the wind will be provided to the bioretention area if the designer places the trees on the perimeter of the area. Trees and shrubs can be sheltered from damaging flows if they are placed away from the path of the incoming runoff. Species that are more tolerant to cold winds (e.g., evergreens) should be placed in windier areas of the site.

After the trees and shrubs are placed, the ground cover and/or mulch should be established. Ground cover such as grasses or legumes can be planted during the spring of the year. Mulch should be placed immediately after trees and shrubs are planted. Five to 7.6 cm (2 to 3 inches) of commercially-available fine shredded hardwood mulch or shredded hardwood chips should be applied to the bioretention area to protect from erosion. Mulch depths should be kept below 7.6 centimeters (3 inches) because more would interfere with the cycling of carbon dioxide and oxygen between the soil and the atmosphere. The mulch should be aged for at least six months (one year is optimal), and applied uniformly over the site.

**PERFORMANCE**

Bioretention removes storm water pollutants through physical and biological processes,
including adsorption, filtration, plant uptake, microbial activity, decomposition, sedimentation and volatilization. Adsorption is the process whereby particulate pollutants attach to soil (e.g., clay) or vegetation surfaces. Adequate contact time between the surface and pollutant must be provided for in the design of the system for this removal process to occur. Therefore, the infiltration rate of the soils must not exceed those specified in the design criteria or pollutant removal may decrease. Pollutants removed by adsorption include metals, phosphorus, and some hydrocarbons. Filtration occurs as runoff passes through the bioretention area media, such as the sand bed, ground cover and planting soil. The media trap particulate matter and allow water to pass through. The filtering effectiveness of the bioretention area may decrease over time. Common particulates removed from storm water include particulate organic matter, phosphorus, and suspended solids. Biological processes that occur in wetlands result in pollutant uptake by plants and microorganisms in the soil. Plant growth is sustained by the uptake of nutrients from the soils, with woody plants locking up these nutrients through the seasons. Microbial activity within the soil also contributes to the removal of nitrogen and organic matter. Nitrogen is removed by nitrifying and denitrifying bacteria, while aerobic bacteria are responsible for the decomposition of the organic matter (e.g., petroleum). Microbial processes require oxygen and can result in depleted oxygen levels if the bioretention area is not adequately aerated.

Sedimentation occurs in the swale or ponding area as the velocity slows and solids fall out of suspension.

Volatilization also plays a role in pollutant removal. Pollutants such as oils and hydrocarbons can be removed from the wetland via evaporation or by aerosol formation under windy conditions. The removal effectiveness of bioretention has been studied during field and laboratory studies conducted by the University of Maryland (Davis et al., 1998). During these experiments, synthetic storm water runoff was pumped through several laboratory and field bioretention areas to simulate typical storm events in Prince George's County, MD. Removal rates for heavy metals an nutrients are shown in Table 1. As shown, the BMP removed between 93 and 98 percent of metals, between 68 and 80 percent of TKN and between 70 and 83 percent of total phosphorus. For all of the pollutants analyzed, results of the laboratory study were similar to those of field experiments. Doubling or halving the influent pollutant levels had little effect on the effluent pollutants levels (Davis et al., 1998). For other parameters, results from the performance studies for infiltration BMPs, which are similar to bioretention, can be used to estimate bioretention's performance. These removal rates are also shown in Table 1. As shown, the BMP could potentially achieve greater than 90 percent removal rates for total suspended solids, organics, and bacteria. The microbial activity and plant uptake occurring in the bioretention area will likely result in higher removal rates than those determined for infiltration BMPs.

<table>
<thead>
<tr>
<th>Pollutant</th>
<th>Removal Rate</th>
</tr>
</thead>
<tbody>
<tr>
<td>Total Phosphorus</td>
<td>70%-83%</td>
</tr>
<tr>
<td>Metals (Cu, Zn, Pb)</td>
<td>93%-98%</td>
</tr>
<tr>
<td>TKN</td>
<td>68%-80%</td>
</tr>
<tr>
<td>Total Suspended Solids</td>
<td>90%</td>
</tr>
<tr>
<td>Organics</td>
<td>90%</td>
</tr>
<tr>
<td>Bacteria</td>
<td>90%</td>
</tr>
</tbody>
</table>

Source: ^Davis et al. (1998)
^PGDIER (1993)

OPERATION AND MAINTENANCE

Recommended maintenance for a bioretention area includes inspection and repair or replacement of the treatment area components. Trees and shrubs should be inspected twice per year to evaluate their health and remove any dead or severely diseased vegetation. Diseased vegetation should be treated as necessary using preventative and low-toxic measures to the extent possible. Pruning and weeding may also be necessary to maintain the treatment area's appearance. Mulch replacement is recommended when erosion is evident or when the site begins to look unattractive. Spot mulching may
be adequate when there are random void areas; however, once every two to three years the entire area may require mulch replacement. This should be done during the spring. The old mulch should be removed before the new mulch is distributed. Old mulch should be disposed of properly.

The application of an alkaline product, such as limestone, is recommended one to two times per year to counteract soil acidity resulting from slightly acidic precipitation and runoff. Before the limestone is applied, the soils and organic layer should be tested to determine the pH and therefore the quantity of limestone required. When levels of pollutants reach toxic levels which impair plant growth and the effectiveness of the BMP, soil replacement may be required (PGDER, 1993).

COSTS

Construction cost estimates for a bioretention area are slightly greater than those for the required landscaping for a new development. Recently-constructed 37.16 square meter (400 square foot) bioretention areas in Prince George's County, MD cost approximately $500. These units are rather small and their cost is low. The cost estimate includes the cost for excavating 0.6 to 1 meters (2 to 3 feet) and vegetating the site with 1 to 2 trees and 3 to 5 shrubs. The estimate does not include the cost for the planting soil, which increases the cost for a bioretention area. Retrofitting a site typically costs more, averaging $6,500 per bioretention area. The higher costs are attributed to the demolition of existing concrete, asphalt, and existing structures and the replacement of fill material with planting soil. The costs of retrofitting a commercial site in Maryland (Kettering Development) with 15 bioretention areas were estimated at $111,600.

The use of bioretention can decrease the cost for storm water conveyance systems at a site. A medical office building in Maryland was able to reduce the required amount of storm drain pipe from 243.8 meters (800 feet) to 70.1 meters (230 feet) with the use of bioretention. The drainage pipe costs were reduced by $24,000, or 50 percent of the total drainage cost for the site (PGDER, 1993). Landscaping costs that would be required at a development regardless of the installation of the bioretention area should also be considered when determining the net cost of the BMP.

The operation and maintenance costs for a bioretention facility will be comparable to those of typical landscaping required for a site. Costs beyond the normal landscaping fees will include the cost for testing the soils and may include costs for a sand bed and planting soil.

REFERENCES


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Sand Filters
DESCRIPTION

Sand filters have proven effective in removing several common pollutants from storm water runoff. Sand filters generally control storm water quality, providing very limited flow rate control.

A typical sand filter system consists of two or three chambers or basins. The first is the sedimentation chamber, which removes floatables and heavy sediments. The second is the filtration chamber, which removes additional pollutants by filtering the runoff through a sand bed. The third is the discharge chamber. The treated filtrate normally is then discharged through an underdrain system either to a storm drainage system or directly to surface waters. Sand filters take up little space and can be used on highly developed sites and sites with steep slopes. They can be added to retrofit existing sites. Sand filters are able to achieve high removal efficiencies for sediment, biochemical oxygen-demand (BOD), and fecal coliform bacteria. Total metal removal, however, is moderate, and nutrient removal is often low.

There are three main sand filter designs currently in common use: the Austin sand filter (Figure 1); the Washington, D.C., sand filter (Figure 2); and the...
Delaware sand filter (Figure 3). The primary differences among these designs are location (i.e., above or below ground), the drainage area served, their filter surface areas, their land requirements, and the quantity of runoff they treat.

Modifications that may improve sand filter design and performance are being tested. One modification is the addition of a peat layer in the filtration chamber. The addition of peat to the sand
filter may increase microbial growth within the sand filter and improve metals and nutrient removal rates.

APPLICABILITY

Sand filters are intended primarily for water quality enhancement. In general, sand filters are preferred over infiltration practices, such as infiltration trenches, when contamination of groundwater with conventional pollutants - BOD, suspended solids, and fecal coliform - is of concern. This usually occurs in areas where underlying soils alone cannot treat runoff adequately - or ground water tables are high. In most cases, sand filters can be constructed with impermeable basin or chamber bottoms, which help to collect, treat, and release runoff to a storm drainage system or directly to surface water with no contact between contaminated runoff and groundwater.

The selection of a sand filter design depends largely on the drainage area’s characteristics. For example, the Washington, D.C., and Delaware sand filter systems are well suited for highly impervious areas where land available for structural controls is limited, since both are installed underground. They are often used to treat runoff from parking lots, driveways, loading docks, service stations, garages, airport runways/taxiways, and storage yards. The Austin sand filtration system is more suited for large drainage areas that have both impervious and pervious surfaces. This system is located at grade and is often used at transportation facilities, in large parking areas, and in commercial developments.

In general, all three types of sand filters can be used as alternatives for water quality inlets. They are more frequently used to treat runoff contaminated with oil and grease from drainage areas with heavy vehicle usage. In regions where evaporation exceeds rainfall and a wet pond would be unlikely to maintain the required permanent pool, the Austin sand filtration system can be used.

ADVANTAGES AND DISADVANTAGES

Sand filters can be highly effective storm water best management practices (BMPs). All three types of sand filters achieve high removal rates for sediment, BOD, and fecal coliform bacteria. The filter media is periodically removed from the filter unit, thus also permanently removing trapped contaminants. Waste media from the filters does not appear to be toxic and is environmentally safe for landfill disposal. If they are designed with an impermeable basin liner, sand filters can also reduce the potential for groundwater contamination. Finally sand filters also generally require less land than other BMPs, such as ponds or wetlands.

The size and characteristics of the drainage area, as well as the pollutant loading, will greatly influence the effectiveness of the sand filter system. For example, sand filters may be of limited value in some applications because of they are designed to handle runoff from relatively small drainage areas and they have low nutrient removal and metal removal capabilities. In these cases, other BMPs, such as wet ponds, may be less costly and/or more effective. The system also requires routine maintenance to prevent sediment from clogging the filter. In some cases, filter media may need to be replaced 3 to 5 years. Lastly, sand filters generally do not control storm water flow, and consequently, they do not prevent downstream stream bank and channel erosion.

Climatic conditions may also limit the filter’s performance. For example, it is not yet known how well sand filters will operate in colder climates or in freezing conditions.

DESIGN CRITERIA

Typically the Austin sand filter system is designed to handle runoff from drainage areas up to 20 hectares (50 acres). The collected runoff is first diverted to the sedimentation basin, where heavy sediments and floatables are removed. There are two designs for the sedimentation basin: the full sedimentation system, as shown in Figure 1; and a partial sedimentation system, where only the initial flow is diverted. Both systems are located off-line and are designed to collect and treat the first 1.3 centimeters (0.5 inches) of runoff. The partial system has the capacity to hold only a portion (at least 20 percent) of the first flush volume in the sedimentation basin, whereas the full system captures and holds the entire flow volume.
Equations used to determine the sedimentation basin surface areas (A_s) in square and meters acres are shown in Table 1.

**TABLE 1 SURFACE AREA EQUATION FOR AUSTIN SAND FILTER SYSTEM**

<table>
<thead>
<tr>
<th>Partial Sedimentation</th>
<th>Full Sedimentation</th>
</tr>
</thead>
<tbody>
<tr>
<td>A_s=(AD)(H)/(1+D_s-1/10)</td>
<td>A_s=(AD)(H)/10</td>
</tr>
<tr>
<td>A_f=(AD)(H)/18</td>
<td></td>
</tr>
</tbody>
</table>

Note: Designed to collect and treat 0.5 inches of runoff.

D_s (feet) = depth of the sedimentation basin.
H (feet) = depth of rainfall, 0.042ft (0.5 in).
AD (acres) = impervious and pervious areas that provide contributing drainage.

Source: Galli, 1980.

Flow is conveyed from the sedimentation basin, through a perforated riser, a gabion wall, or a berm, to the filtration basin. The filtration basin consists of a 45-centimeter (18-inch) layer of sand particles 0.05 to 0.10 centimeters (0.02 to 0.04 inches) in diameter that may be underlain by a gravel layer. Equations used to determine the surface areas (A_f) in acres are also shown in Table 1. The filtrate is discharged from the filtration basin through underdrain piping 10 to 15 centimeters (4 to 6 inches) in diameter with 1-centimeter (0.4 inch) perforations. Filter fabric is placed around the underdrain piping to prevent sand and other particulates from being discharged.

Typically, the Washington, D.C., sand filter system is designed to handle runoff from completely impervious drainage areas of 0.4 hectares (1 acre) or less. The system, as shown in Figure 2, consists of three underground chambers: a sedimentation chamber, a filtration chamber, and a discharge chamber. The sand filter system is designed to accept the first 1.3 centimeters (0.5 inches) of runoff. Coarse sediments and floatables are removed from the runoff within the sedimentation chamber. Runoff is discharged from the sedimentation chamber through a submerged weir, into the filtration chamber, which consists of a combination of sand and gravel layers totaling 1 meter (3 feet) in depth with underdrain piping wrapped in filter fabric. The underdrain system collects the filtered water and discharges it to the third chamber, where the water is collected and discharged to a storm water channel or sewer system. An overflow weir is located between the second and third chambers to bypass excess flow. The Washington, D.C., sand filter is often constructed on-line, but can be constructed off-line. When the system is off-line, the overflow between the second and third chambers is not included.

The Delaware sand filter, shown in Figure 3, is similar to the Washington, D.C., sand filter in that both utilize underground concrete vaults. However, the Delaware sand filter has only two chambers: a sedimentation chamber and a filtration chamber. A 2.5-centimeter (1 inch) design storm was selected for sizing the sedimentation basin because it is representative of large storm events: in Delaware, 92 percent of all storms are less than 2.5 centimeters (1 inch) in depth. Runoff enters the sedimentation chamber through a grated cover and then overflows into the filtration chamber, which contains a sand layer 45 centimeters (18 inches) in depth. Gravel is not normally used in the filtration chamber although the filter can be modified to include it. Typical systems are designed to handle runoff from drainage areas of 2 hectares (5 acres) or less. A major advantage of the Delaware sand filter is its shallow structure depth of only 76 centimeters (30 inches), which reduces construction and maintenance costs.

Proper design and maintenance are also critical factors in maintaining the operating life of any filter system. The life of the filter media may be increased by a number of methods, including:

- Stabilizing the drainage area so that sediment loadings in the runoff are minimized.
- Providing adequate storm water detention times to enhance sedimentation and filtration.
- Inspecting and maintaining the sand filter frequently enough to ensure proper operation.
PERFORMANCE

Sand filters are currently in use in Delaware, Maryland, Florida, Texas, Virginia, and Washington, D.C. Studies on the systems' pollutant removal efficiencies are currently being performed in Washington, D.C., and Austin, TX. Additional evaluations are needed to evaluate alternative sand filter designs and media. Sand filters remove particulates in both the sedimentation and the filtration chambers. The City of Austin has estimated their systems' pollutant removal efficiencies based on preliminary findings of their storm water monitoring program (Austin, 1988). The estimates shown in Table 2 are average values for various sand filters serving drainage areas of several different sizes. As shown in Table 2, no removal of nitrate was observed. No other dissolved pollutants were monitored. Additional monitoring is currently being performed by the City of Austin to supplement the preliminary estimates.

OPERATION AND MAINTENANCE

All filter system designs must provide adequate access to the filter for inspection and maintenance. The sand filters should be inspected after all storm events to verify that they are working as intended. Since the Washington, D.C., and Austin sand filter systems can be deep, they may be designated as confined spaces and require compliance with confined space entry safety procedures.

Typically, sand filters begin to experience clogging problems within 3 to 5 years (NVPC, 1992). Accumulated trash, paper and debris should be removed from the sand filters every 6 months or as necessary to keep the filter clean. A record should be kept of the dewatering times for all sand filters to determine if maintenance is necessary. Corrective maintenance of the filtration chamber includes removal and replacement of the top layers of sand, gravel and/or filter fabric that has become clogged. The removed media may usually be disposed in a landfill. The City of Austin tests their waste media before disposal. Results thus far indicate that the waste media is not toxic and can be safely landfilled (Schueler, 1992). Sand filter systems may also require the periodic removal of vegetative growth.

<table>
<thead>
<tr>
<th>Pollutant</th>
<th>Percent Removal</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fecal Coliform</td>
<td>76</td>
</tr>
<tr>
<td>Biochemical Oxygen Demand</td>
<td>70</td>
</tr>
<tr>
<td>Total Suspended Solids (TSS)</td>
<td>70</td>
</tr>
<tr>
<td>Total Organic Carbon (TOC)</td>
<td>48</td>
</tr>
<tr>
<td>Total Nitrogen (TN)</td>
<td>21</td>
</tr>
<tr>
<td>Total Kjeldahl Nitrogen (TKN)</td>
<td>46</td>
</tr>
<tr>
<td>Nitrate as Nitrogen (NO₃⁻N)</td>
<td>0</td>
</tr>
<tr>
<td>Total Phosphorus (TP)</td>
<td>33</td>
</tr>
<tr>
<td>Iron (Fe)</td>
<td>45</td>
</tr>
<tr>
<td>Lead (Pb)</td>
<td>45</td>
</tr>
<tr>
<td>Zinc (Zn)</td>
<td>45</td>
</tr>
</tbody>
</table>

Source: Galli, 1990

COSTS

The construction cost for an Austin sand filtration system is approximately $18,500 (1997 dollars) for a 0.4 hectare- (1 acre-) drainage area. The cost per hectare decreases with increasing drainage area. The cost for precast Washington, D.C. sand filters, with drainage areas of less than 0.4 hectares (1 acre), ranges between $6,600 and $11,000 (1997 dollars). This is considerably less than the cost for the same size cast-in-place system. Costs for the Delaware sand filter are similar to that of the D.C. system, with the exception of the lower excavation costs due to the Delaware filters' shallowness.

Annual costs for maintaining sand filter systems average about 5 percent of the initial construction cost (Schueler, 1992). Media is replaced as needed. Currently the sand is being replaced in the D.C. filter systems about every 2 years. The cost to replace the gravel layer, filter fabric and top portion of the sand for D.C. sand filters is approximately
$1,700 (1997 dollars). Improvements in Washington, D.C.'s maintenance procedures may extend the life of the filter media and reduce the overall maintenance costs.

REFERENCES


ADDITIONAL INFORMATION

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Storm Water Wetlands
DESCRIPTION

Wetlands are those areas that are typically inundated with surface or ground water and that support plants adapted to saturated soil conditions. A typical shallow marsh wetland is shown in Figure 1. Wetlands have been described as "nature's kidneys" because the physical, chemical, and biological processes that occur in wetlands break down some compounds (e.g., nitrogen-containing compounds, sulfate) and filter others (Hammer, 1989). The natural pollutant-removal capabilities of wetlands have brought them increased attention as storm water best management practices (BMPs).

Wetlands used for storm water treatment can be incidental, natural, or constructed. Incidental wetlands are those wetlands that were created as a result of previous development or human activity. The use of natural wetlands for storm water treatment is discouraged by many experts and/or public interest groups, and may not be an option in many areas. However, some states allow wetlands to be used as storm water BMPs, but only in very restricted circumstances. For example, the State of Florida allows the use of natural wetlands that have been severely degraded or wetlands that are intermittently connected to other waters (i.e., they are connected only when groundwater rises above ground level) (Livingston, 1994). Conversion of natural wetlands to storm water wetlands is done on a case-by-case basis and requires the appropriate state and federal permits (e.g., 401 water quality certification and 404 wetland permit).

Two types of constructed wetlands have been used

![Figure 1: Shallow Marsh Wetland](image-url)

Source: MWCOG, 1992a.
successfully for wastewater treatment: the subsurface flow (SF) constructed wetland and the free water surface (FWS) constructed wetland. In the FWS wetland, runoff flows through the soil-lined basin at shallow depths. The wetland consists of a shallow pool planted with emergent vegetation (vegetation which is rooted in the sediment but with leaves at or above the water surface).

In contrast to the FWS wetland, the SF wetland basin is lined with a pre-designed amount of rock or gravel, through which the runoff is conveyed. The water level in an SF wetland remains below the top of the rock or gravel bed. Studies have indicated that the SF wetland is well suited for the diurnal flow pattern of wastewater; however, the peak flows from storm water or combined sewer overflows (CSOs) may be several orders of magnitude higher than the baseflow. The cost for a gravel bed to contain the peak storm event would be very high, which may preclude the use of SF wetlands for storm water or CSO treatment. Therefore, the remainder of this fact sheet addresses the FWS constructed wetland or natural and incidental wetlands for use in storm water applications.

There are four basic designs of FWS constructed wetlands: shallow marsh, extended detention wetland, pond/wetland system, and pocket wetland. As shown in Figure 2, these wetlands store runoff in a shallow basin vegetated with wetland plants. The selection of one design over another will depend on various factors, including land availability, level and reliability of pollutant removal, and size of the contributing drainage area.

The shallow marsh design requires the most land and a sufficient baseflow to maintain water within the wetlands. The basic shallow marsh design can be modified to store extra water above the normal pool elevation. This wetland, known as an extended detention wetland, attenuates flows and relieves downstream flooding.

The pond/wetland system has two separate cells: a wet pond and a shallow marsh. The wet pond traps sediments and reduces runoff velocities prior to entry into the wetland. Less land is required for a pond/wetland system than for the shallow marsh system.

Still less land is required for a pocket wetland. Pocket wetlands should be designed with contributing drainage areas of 0.4 to 4 hectares (1 to 10 acres) and usually require excavation down to the water table for a reliable water source. Unreliable water sources and fluctuating water levels result in low plant diversity and poor wildlife habitat value (MWCOCG, 1992b).

![FIGURE 2 COMPARATIVE PROFILES OF FOUR STORM WATER WETLAND DESIGNS](image)

Cross-sectional profiles of the four storm water wetlands not drawn to scale. In Panel A, most of the shallow marsh is shallow, supporting emergent wetland plants. In extended detention wetlands (Panel B), the runoff storage of the wetland is augmented by temporary, vertical extended detention storage. The pond/wetland system (Panel C) is composed of a deep and a shallow pool. Pocket wetlands (Panel D) are excavated to the groundwater table to keep water elevation more consistent.

Source: MWCOCG, 1992b.
APPLICABILITY

Wetlands improve the quality of storm water runoff, and can also control runoff volume (e.g., extended detention wetland). Wetlands are one of the more reliable BMPs for removing pollutants and are adaptable to most locations in the U.S. Locations with existing wetlands used for storm water treatment include Alabama, California, Colorado, Florida, Illinois, Maine, Maryland, Michigan, Minnesota, Virginia, and Washington. Wetlands have been used to treat runoff from agricultural, commercial, industrial, and residential areas.

In the past, the natural ability of wetlands to remove pollutants from water has primarily been harnessed to treat wastewater. However, the utilization of wetlands to treat storm water has gained attention in recent years, and many storm water wetlands treatment systems are now operational. Ongoing evaluations are being conducted to determine the effectiveness of wetlands in pollutant removal and to determine the level of maintenance required to sustain their performance, while other studies are evaluating the potential for design modifications to improve wetland performance.

ADVANTAGES AND DISADVANTAGES

Environmental benefits associated with storm water wetlands include improvements in downstream water and habitat quality, enhancement of diverse vegetation and wildlife habitat in urban areas, and flood attenuation. Downstream water quality is improved by the partial removal of suspended solids, metals, nutrients, and organics from urban runoff. Habitat quality is also improved as reduced sediment loads are carried downstream and the erosion of stream banks associated with peak storm water flows is reduced. Wetlands can support a diverse wildlife population, including species such as sandpipers and herons, and can attenuate runoff and alleviate downstream flooding (particularly extended detention wetlands).

Storm water wetlands can cause adverse environmental impacts upstream of the wetland, within the wetland itself, and downstream of the wetland. Storm water wetlands located in a large watershed (larger than 40 hectares (100 acres)) may degrade upstream headwaters, which receive no effective hydrologic control (MWCOG, 1992b). The wetland designer can incorporate upstream modifications to relieve this negative impact.

Possible adverse effects within the wetland itself are the potential for blocking fish passage, potential habitat by undesirable species, and potential groundwater contamination. A wetland constructed in the stream channel may block fish access to part of the stream, thereby decreasing fish diversity in the stream.

Geese and mallards may become undesirable year-round residents of the wetland if structural complexity is not included in the wetland design (i.e., features that limit deep and open water areas and open grassy areas that are favored by these birds). These animals will increase the nutrient and coliform loadings to the wetland and may also become a nuisance to local residents. The takeover of vegetation by invasive nuisance plants is also a potential negative impact. Invasive species pose a threat to native species and may adversely affect the wetland's ability to treat storm water. Maintaining and/or planting upland buffer zones can help to reduce the introduction of nuisance plant species. Planting emergent vegetation may also reduce nuisance algal blooms (Carr, 1995).

The issue of groundwater contamination resulting from the migration of polluted sediments to the groundwater has been considered a potential negative environmental impact. However, studies indicate that there is little risk of groundwater contamination (MWCOG, 1992b).

A storm water wetland can act as a heat sink, especially during the summer, and can discharge warmer waters to downstream water bodies. The increased temperatures can affect sensitive fish species (such as trout and sculpins) and aquatic insects downstream. Therefore, it is not recommended to construct storm wetlands upstream of temperature-sensitive fish populations. Regardless of the sensitivity of downstream species, the designer should always take precautions to reduce the potential warming effects of wetlands construction.

Communities may be opposed to a wetland for fear of mosquitoes and other nuisances, or because of wetlands' appearance. However, wetlands can be
designed attractively and features (e.g., fish and vegetation) can be adapted to control mosquitoes and other nuisances. The use of Gambusia fish for mosquito control has become a common practice in warmer climates, while colder climates use the black striped topminnow (*Notropus funderus*) (U.S. EPA, 1995). To minimize the protection from predators offered by taller plants, the use of low growing plants is recommended where pests are a concern (U.S. EPA, 1996).

Wetlands may remove pollutants less effectively during the non-growing season and in localities with lower temperatures. Decreases in some pollutant-removal efficiencies have been observed when wetlands are covered with ice and when they receive snow melt runoff.

Finally, because of the large land requirement for storm water wetlands systems (See Design Criteria), their use may be precluded in urban settings and established communities.

Several possible remedies to these impacts are discussed in the publication *Design of Storm Water Wetland Systems* (MWCOG, 1992).

**DESIGN CRITERIA**

Local, state and federal permit requirements should be determined prior to wetland design. Required permits and certifications may include 401 water quality certifications, 402 storm water National Pollutant Discharge Elimination System (NPDES) permits, 404 wetland permits, dam safety permits, sediment and erosion control plans, waterway disturbance permits, forest-clearing permits, local grading permits, and land use approvals.

A site appropriate for a wetland must have an adequate water flow and appropriate underlying soils. The baseflow from the drainage area or groundwater must be sufficient to maintain a shallow pool in the wetland and support the wetlands' vegetation, including species susceptible to damage during dry periods. Underlying soils that are type B, C, or R (zone of accumulation, partially altered parent material and unaltered parent material, respectively) will have only small infiltration losses. Sites with type A soils (soils rich in organic matter) may have high infiltration rates.

These sites may require geotextile liners or a 15 centimeter (6 inch) layer of clay. After any necessary excavation and grading of the wetland, at least 10 centimeters (4 inches) of soil should be applied to the site. This material, which may be the previously-excavated soil or sand and other suitable material, is needed to provide a substrate in which the vegetation can become established and to which it can become anchored. The substrate should be soft so that plants can be inserted easily.

The Metropolitan Washington Council of Governments (MWCOG, 1992b) has recommended basic sizing criteria for wetland design. The volume of the wetland is determined as the quantity of runoff generated by 90 percent of the runoff-producing storms. This volume will vary throughout the U.S. due to different rainstorm patterns. In the Mid-Atlantic Region, for example, a 1.25-inch storm is used as the sizing criterion.

Watershed imperviousness will also impact the runoff volume generated from a storm. The following equations are used to determine the treatment volume (Vt):

1. \[ V_t = 0.05 + 0.009 (I) \]
   where:
   \[ R_v = \text{storm runoff coefficient} \]
   \[ I = \% \text{ (as decimal) site imperviousness} \]

2. \[ V_t = \frac{(1.25)(R_v)(A)}{12}(43,560) \]
   where:
   \[ V_t = \text{treatment volume (cubic feet)} \]
   \[ A = \text{contributing area (acres)} \]

Sizing criteria for wetlands vary, with some states having their own methods. For example, shallow wetland basins constructed in Maryland are designed to maximize basin surface area. The surface area should be a minimum of 3 percent of the area of the watershed draining to it. Maryland recommends designing for extended detention, using 24-hour detention of the 1-year storm for design purposes. In contrast, the Washington State Department of Ecology sizes wetlands using the runoff generated from the 6-month, 24-hour rainfall event. The minimum surface area established by MWCOG for shallow marshes is 2 percent of the wetland area. The remaining three wetland designs should have wetland to watershed ratios greater than 1 percent.
TABLE 1 GUIDELINES FOR ALLOCATING WETLAND SURFACE AREA AND TREATMENT VOLUME

<table>
<thead>
<tr>
<th>Target Allocations</th>
<th>Shallow Marsh</th>
<th>Extended Detention</th>
<th>Pond/Wetland</th>
<th>Pocket Wetland</th>
</tr>
</thead>
<tbody>
<tr>
<td>Percent of Wetland Surface Area</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Forebay</td>
<td>5</td>
<td>5</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Micropool</td>
<td>5</td>
<td>5</td>
<td>5</td>
<td>0</td>
</tr>
<tr>
<td>Deepwater</td>
<td>5</td>
<td>0</td>
<td>40</td>
<td>5</td>
</tr>
<tr>
<td>Low Marsh</td>
<td>40</td>
<td>40</td>
<td>25</td>
<td>50</td>
</tr>
<tr>
<td>High Marsh</td>
<td>40</td>
<td>40</td>
<td>25</td>
<td>40</td>
</tr>
<tr>
<td>Semi-Wet</td>
<td>5</td>
<td>10</td>
<td>5</td>
<td>5</td>
</tr>
<tr>
<td>Percent of Treatment Volume</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Forebay</td>
<td>10</td>
<td>10</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Micropool</td>
<td>10</td>
<td>10</td>
<td>10</td>
<td>0</td>
</tr>
<tr>
<td>Deepwater</td>
<td>10</td>
<td>0</td>
<td>60</td>
<td>20</td>
</tr>
<tr>
<td>Low Marsh</td>
<td>45</td>
<td>20</td>
<td>20</td>
<td>55</td>
</tr>
<tr>
<td>High Marsh</td>
<td>25</td>
<td>10</td>
<td>10</td>
<td>25</td>
</tr>
<tr>
<td>Semi-Wet</td>
<td>0</td>
<td>60</td>
<td>0</td>
<td>0</td>
</tr>
</tbody>
</table>

Depth:
- Deepwater - 0.5 - 2 meters (1.5 to 6 feet) below normal pool level
- Low Marsh - 0.17 - 0.5 meters (0.5 to 1.5 feet) below normal pool level
- High Marsh - 0.5 feet below normal pool level
- Semi-Wet - 0 to 2 feet above normal pool level (includes Extended Detention)

Source: Modified from MWCOG, 1992b.

MWCOG has also established criteria for water balance, maximum flow path, allocation of treatment volume, minimum surface area, allocation of the surface area, and extended detention. As previously discussed, during dry weather, flow must be adequate to provide a baseflow and to maintain the vegetation. The flow path should be maximized to increase the runoff’s contact time with plants and sediments. The recommended minimum length to width ratio of the wetland is 2:1. If a ratio of less than 2:1 is necessary, the use of baffles, islands, and peninsulas can minimize short circuiting (allowing runoff to escape treatment) by ensuring a long distance from inlet to outlet.

A suggestion for allocating treatment volumes is shown in Table 1. The wetland surface area is allocated to four different depth zones: deepwater (0.5 to 2 meters, or 1.5 to 6 feet, below normal pool), low marsh (0.17 to 0.5 meters, or 0.5 to 1.5 feet, below normal pool), high marsh (up to 0.17 meters, or 0.5 feet, below normal pool), and semi-wet areas (above normal pool). The allocation to the various depth zones will create a complex internal topography that will maximize plant diversity and increase pollutant removal. The State of Maryland requires that 50 percent of the shallow marsh be less than 0.17 meters (0.5 feet) deep, that 25 percent range from 0.17 to 0.33 meters (0.5 feet to 1 foot) deep, and that the remaining 25 percent range from 0.67 to 1 meter (2 to 3 feet) deep.
Extending detention within the wetland increases the time for sedimentation and other pollutant-removal processes to occur and also provides for attenuation of flows. Up to 50 percent extra treatment volume can be added into the wetland system for extended detention. However, to prevent large fluctuations in the water level that could potentially harm the vegetation, Extended Detention elevation should be limited to 11 meters (33 feet) above the normal pool elevation. The Extended Detention volume should be detained between 12 and 24 hours.

Sediment forebays are recommended to decrease the velocity and sediment loading to the wetland. The forebays provide the additional benefits of creating sheet flow, extending the flow path, and preventing short circuiting. The forebay should contain at least 10 percent of the wetland’s treatment volume and should be 2 to 3 meters (4 to 6 feet) deep. The State of Maryland recommends a depth of at least 1 meter (3 feet). The forebay is typically separated from the wetland by gabions or by an earthen berm (MWCOG, 1992b).

Flow from the wetland should be conveyed through an outlet structure that is located within the deeper areas of the wetland. Discharging from the deeper areas using a reverse slope pipe prevents the outlet from becoming clogged. A micropool just prior to the outlet will also prevent outlet clogging. The micropool should contain approximately 10 percent of the treatment volume and be 2 to 3 meters (4 to 6 feet) deep. An adjustable gate-controlled drain capable of dewatering the wetland within 24 hours should be located within the micropool. A typical drain may be constructed with an upward-facing inverted elbow with its opening above the accumulated sediment. The dewatering feature eases planting and follow-up maintenance (MWCOG, 1992b).

Vegetation can be established by any of five methods: mulching; allowing volunteer vegetation to become established; planting nursery vegetation; planting underground dormant parts of a plant; and seeding. Donor soils from existing wetlands can be used to establish vegetation within a wetland. This technique, known as mulching, has the advantage of quickly establishing a diverse wetland community. However, with mulching, the types of species that grow within the wetland are unpredictable.

Allowing species transmitted by wind and waterfowl to voluntarily become established in the wetland is also unpredictable. Volunteer species are usually well established within 3 to 5 years. Wetlands established with volunteers are usually characterized by low plant diversity with monotypic stands of exotic or invasive species. A higher-diversity wetland can be established when nursery plants or dormant rhizomes are planted. Vegetation from a nursery should be planted during the growing season - not during late summer or fall - to allow vegetation time to store food reserves for their dormant period. Separate underground parts of vegetation are planted during the plants’ dormant period, usually October through April, but the months will vary with local climate. Another planting technique, the spreading of seeds, has not been very successful and therefore is not widely practiced as a principal planting technique.

Appropriate plant types vary with locations and climate. The wetland designer should select five to seven plants native to the area and design the depth zones in the wetland to be appropriate for the type of plant and its associated maximum water depth. Approximately half of the wetland should be planted. Of the five to seven species selected, three should be aggressive plants or those that become established quickly. Examples of aggressive species used in the Mid-Atlantic Region include softstem bulrush (Scirpus validus) and common three-square (Scirpus americanus). Aggressive plants as well as other native wetland plants are available from numerous nurseries. Most vendors require an advance order of 3 to 6 months.

After excavation and grading the wetland should be kept flooded until planting. Six to nine months after being flooded and two weeks before planting, the wetland is typically drained and surveyed to ensure that depth zones are appropriate for plant growth. Revisions may be necessary to account for any changes in depth. Next, the site is staked to ensure that the planting crew spaces the plants within the correct planting zone. Species are planted in separate zones to avoid competition. The State of Maryland recommends planting two
aggressive or primary species in four specific areas and planting an additional 40 clumps (one or more individuals of a single species) per acre of each primary species over the rest of the wetland. Three secondary species are planted close to the edge of the wetland at an application rate of 10 clumps of 5 individual plants per acre of wetland, for a total of 50 individuals of each secondary species per acre of wetland. At least 48 hours prior to planting, the wetland should be drained; within 24 hours after planting, it should be re-flooded.

The wetland design should include a buffer to separate the wetland from surrounding land. Buffers may alleviate some potential wetland nuisances, such as accumulated floatables or odors. MWCOG recommends a buffer of 8 meters (25 feet) from the maximum water surface elevation, plus an additional 8 meters (25 feet) when wildlife habitat is of concern. Leaving trees undisturbed in the buffer zone will minimize the disruption to wildlife and reduce the chance for invasion of nuisance vegetation such as cattails and primrose willow. If tree removal is necessary, the buffer area should be reforested. Reforestation also discourages the settlement of geese, which prefer open areas.

PERFORMANCE

Wetlands remove pollutants from storm water through physical, chemical, and biological processes. Chemical and physical assimilation mechanisms include sedimentation, adsorption, filtration, and volatilization.

Sedimentation is the primary removal mechanism for pollutants such as suspended solids, particulate nitrogen, and heavy metals. Particulate settling is influenced by the velocity of the runoff through the wetland, the particle size, and turbulence. Sedimentation can be maximized by creating sheet flow conditions, slowing the velocities through the wetland, and providing morphology and vegetation conducive to settling. The vegetation and its root system will also decrease the resuspension of settled particles.

Some pollutants, including metals, phosphorus, and some hydrocarbons, are removed by adsorption—the process whereby pollutants attach to surfaces of suspended or settled sediments and vegetation. For this removal process to occur, adequate contact time between the surface and pollutant must be provided in the design of the system.

Wetland plants filter trash, debris, and other floatables. Particulates (e.g., settleable solids and colloidal solids) are also filtered mechanically as water passes through root masses. Filtration can be enhanced by slow velocities, sheet flow, and sufficient quantities of vegetation. By increasing detention and contact time and providing a surface for microbial growth, wetland plants also increase the pollutant removal achieved through sedimentation, adsorption, and microbial activity.

Volatilization plays a minor role in pollutant removal from wetlands. Pollutants such as oils and hydrocarbons can be removed from the wetland via evaporation or by aerosol formation under windy conditions.

Biological processes that occur in wetlands result in pollutant uptake by wetland plants and algae. Emergent wetland plants absorb settled nutrients and metals through their roots, creating new sites in the sediment for pollutant adsorption. During the fall the plants' above-ground parts typically die back and the plants may potentially release the nutrients and metals back into the water column (MWCOG, 1992b). Recent studies, however, indicate that most pollutants are stored in the roots of aquatic plants, rather than the stems and leaves (CWF, 1995). Additional studies are required to determine the extent of pollutant release during the fall die-back.

Microbial activity helps to remove nitrogen and organic matter from wetlands. Nitrogen is removed by nitrifying and denitrifying bacteria; aerobic bacteria are responsible for the decomposition of the organic matter. Microbial processes require oxygen and can deplete oxygen levels in the top layer of wetland sediments. The low oxygen levels and the decomposed organic matter help immobilize metals.

Soluble forms of phosphorus, as well as ammonia, are partially removed by planktonic or benthic
algae. The algae consume the nutrients and convert them into biomass, which settles to the bottom of the wetland.

The removal effectiveness of shallow marsh and pond/wetland systems has been fairly well documented, while the amount of removal efficiency data for Extended Detention wetlands and pocket wetlands is limited. Average long-term pollutant removal rates for constructed wetlands, as a whole, are presented in Table 2 (CWP, 1997).

**TABLE 2 PERFORMANCE OF STORM WATER WETLANDS**

<table>
<thead>
<tr>
<th>Pollutant</th>
<th>Removal Rate</th>
</tr>
</thead>
<tbody>
<tr>
<td>Total Suspended Solids</td>
<td>67%</td>
</tr>
<tr>
<td>Total Phosphorus</td>
<td>49%</td>
</tr>
<tr>
<td>Total Nitrogen</td>
<td>28%</td>
</tr>
<tr>
<td>Organic Carbon</td>
<td>34%</td>
</tr>
<tr>
<td>Petroleum Hydrocarbons</td>
<td>87%</td>
</tr>
<tr>
<td>Cadmium</td>
<td>36%</td>
</tr>
<tr>
<td>Copper</td>
<td>41%</td>
</tr>
<tr>
<td>Lead</td>
<td>62%</td>
</tr>
<tr>
<td>Zinc</td>
<td>45%</td>
</tr>
<tr>
<td>Bacteria</td>
<td>77%</td>
</tr>
</tbody>
</table>


As shown, petroleum hydrocarbons (87%), total suspended solids (TSS) (67%), lead (62%), and bacteria (77%) have the highest removal rates. Lower removal rates have been documented for nutrients, organic carbon, and other heavy metals. The removal rates will vary with the loadings to the wetland, retention time in the BMP, and other factors such as BMP geometry, site characteristics, and monitoring methodology (CWP, 1997). Excessive pollutant loadings (e.g., suspended solids) may exceed the wetlands’ removal capabilities.

In general, wetlands remove pollutants about as effectively as do conventional pond systems. Constructed storm water wetlands are more effective than natural wetlands, probably because of their intricate design and continued monitoring and maintenance (MWCOC, 1992). The wetlands’ effectiveness seems to improve after the first few years of use as the vegetation becomes established and organic matter accumulates.

**OPERATION AND MAINTENANCE**

Well-designed and maintained wetlands can function as designed for 20 years or longer. However, wetland maintenance must actually begin during the construction phase. During construction and excavation, many constructed wetlands lose organic matter in the soils. The organic matter provides exchange sites for pollutants, and, therefore, plays an important role in pollutant removal. Replacing or adding organic matter after construction improves performance.

After the wetland has been constructed, its vegetation must be maintained on a regular basis. Maintenance requirements for constructed wetlands are particularly high while vegetation is being established (usually the first three years) (U.S. EPA, 1996). Monitoring during these first years is crucial to the future success of the wetland as a storm water BMP. Inspections should be conducted at least twice per year for the first three years and annually thereafter. Maintenance requirements may also include replacement planting, sediment removal, and possibly plant harvesting. Wetland design should include access to facilitate these maintenance activities.

Vegetative cover on embankments and spillways should be dense and healthy. Replacement planting may be required during the first several years if the original plants do not flourish. First year wetland vegetation growth at the water’s edge and on the side slopes of the wetland can be protected from birds by surrounding the open water area of the wetland with wire to limit access to the vegetation. The embankment and maintenance bench should be mowed twice each year. Other areas surrounding the wetland should not require mowing. Mowing and fertilizing help promote vigorous growth of plant roots that resist erosion. Mowing also prevents the growth of unwanted woody vegetation. Additional routine maintenance that can be conducted on the same schedule should include removal of accumulated trash from trash racks,
outlet structures, and valves, as well as debris on plants that could inhibit growth.

Constructed wetlands should be inspected after major storms during the first year of establishment. The inspector should assess bank stability, erosion damage, flow channelization, and sediment accumulation within the wetland. The inspector shall also take note of species distribution/survival, damage to embankments and spillways from burrowing animals, water elevations, and outlet condition. Water elevations can be raised or lowered by adjusting the outlet's gate valve if plants are not receiving an appropriate water supply.

Accumulated sediments will gradually decrease wetland storage and performance. There are two options to mitigate the effects of accumulated sediments: either the sediments should be removed as necessary or the water level in the wetland should be raised (i.e., the outlet should be adjusted to increase discharge elevation).

The construction of a sediment forebay will decrease the accumulation of sediments within the wetland and increase the wetland’s longevity. The forebay will likely require sediment to be cleaned out every three to five years. The forebay design should allow drainage so that a skid loader or backhoe can be used to remove the accumulated deposits (MWCOG, 1992). Accumulation of organic matter can be reduced by plant harvesting or seasonal drawdown to allow organic material to oxidize (U.S. EPA, 1996).

A number of studies have been performed to determine the toxicity of pond sediments and whether they can be landfilled or land applied without having to meet hazardous waste requirements. Many studies to date have found sediments are not hazardous. However, one study showed that toxic levels of zinc had accumulated in sediment from the pretreatment pond (SFWMD, 1995). If toxic levels of metals have not accumulated in the sediment, then on-site land application of the sediments away from the shoreline will probably be the most cost-effective disposal method (no transportation costs or disposal fees are incurred). Wetlands that receive flow from a drainage area containing commercial or industrial land use and/or activities associated with hazardous waste may contain toxic levels of heavy metals in the sediments. Testing may be required for these sediments prior to land application or disposal.

COSTS

Costs incurred for storm water wetlands include those for permitting, design, construction and maintenance. Permitting costs vary depending on state and local regulations, but permitting, design, and contingency costs are estimated at 25 percent of the construction cost. Construction costs for an emergent wetland with a sediment forebay range from $65,000 to $137,500 per hectare ($26,000 to $55,000 per acre) of wetland. This includes costs for clearing and grubbing, erosion and sediment control, excavating, grading, staking, and planting. The cost for constructing the wetland depends largely upon the amount of excavation required at a site and plant selection. The cost for forested wetlands could be double that of an emergent wetland. Maintenance costs for wetlands are estimated at 2 percent per year of the construction costs (CWP, 1998).

REFERENCES


ADDITIONAL INFORMATION

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Vegetated Swales
Storm Water Technology Fact Sheet
Vegetated Swales

DESCRIPTION

A vegetated swale is a broad, shallow channel with a dense stand of vegetation covering the side slopes and bottom. Swales can be natural or manmade, and are designed to trap particulate pollutants (suspended solids and trace metals), promote infiltration, and reduce the flow velocity of storm water runoff. A typical design is shown in Figure 1.

Vegetated swales can serve as part of a storm water drainage system and can replace curbs, gutters and storm sewer systems. Therefore, swales are best suited for residential, industrial, and commercial areas with low flow and smaller populations.

APPLICABILITY

Vegetated swales can be used wherever the local climate and soils permit the establishment and maintenance of a dense vegetative cover. The feasibility of installing a vegetated swale at a

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FIGURE 1 EXAMPLE OF A VEGETATED SWALE

Notation:
- $L$ = Length of swale impoundment area per check dam (ft)
- $D_s$ = Depth of check dam (ft)
- $D_b$ = Bottom slope of swale (ft/ft)
- $W$ = Top width of check dam (ft)
- $W_b$ = Bottom width of check dam (ft)
- $Z_{hor}$ = Ratio of horizontal to vertical change in swale side slope (ft/ft)

Source: NVPDC, 1996.
particular site depends on the area, slope, and perviousness of the contributing watershed, as well as the dimensions, slope, and vegetative covering employed in the swale system.

Vegetated swales are easy to design and can be incorporated into a site drainage plan. While swales are generally used as a stand-alone storm water Best Management Practice (BMP), they are most effective when used in conjunction with other BMPs, such as wet ponds, infiltration strips, wetlands, etc.

While vegetated swales have been widely used as storm water BMPs, there are also certain aspects of vegetated swales that have yet to be quantified. Some of the issues being investigated are whether their pollutant removal rates decline with age, what effect the slope has on the filtration capacity of vegetation, the benefits of check dams, and the degree to which design factors can enhance the effectiveness of pollutant removal.

ADVANTAGES AND DISADVANTAGES

Swales typically have several advantages over conventional storm water management practice, such as storm sewer systems, including the reduction of peak flows; the removal of pollutants, the promotion of runoff infiltration, and lower capital costs. However, vegetated swales are typically ineffective in, and vulnerable to, large storms, because high-velocity flows can erode the vegetated cover.

Limitations of vegetated swales include the following:

- They are impractical in areas with very flat grades, steep topography, or wet or poorly drained soils.
- They are not effective and may even erode when flow volumes and/or velocities are high.
- They can become drowning hazards, mosquito breeding areas, and may emit odors.
- Land may not be available for them.
- In some places, their use is restricted by law: many local municipalities prohibit vegetated swales if peak discharges exceed 140 liters per second (five cubic feet per second) or if flow velocities are greater than 1 meter per second (three feet per second).
- They are impractical in areas with erosive soils or where a dense vegetative cover is difficult to maintain.

Negative environmental impacts of vegetated swales may include:

- Leaching from swale vegetation may increase the presence of trace metals and nutrients in the runoff.
- Infiltration through the swale may carry pollutants into local groundwater.
- Standing water in vegetated swales can result in potential safety, odor, and mosquito problems.

DESIGN CRITERIA

Design criteria for implementation of the vegetated swales are as follows:

Location

Vegetated swales are typically located along property boundaries along a natural grade, although they can be used effectively wherever the site provides adequate space. Swales can be used in place of curbs and gutters along parking lots.

Soil Requirements

Vegetated swales should not be constructed in gravelly and coarse sandy soils that cannot easily support dense vegetation. If available, alkaline soils and subsoils should be used to promote the removal and retention of metals. Soil infiltration rates should be greater than 0.2 millimeters per second (one-half inch per hour); therefore, care
must be taken to avoid compacting the soil during construction.

Vegetation

A fine, close-growing, water-resistant grass should be selected for use in vegetated swales, because increasing the surface area of the vegetation exposed to the runoff improves the effectiveness of the swale system. Pollutant removal efficiencies vary greatly depending on the specific plants involved, so the vegetation should be selected with pollution control objectives in mind. In addition, care should be taken to choose plants that will be able to thrive at the site. Examples of vegetation appropriate for swales include reed canary grass, grass-legume mixtures, and red fescue.

General Channel Configuration

A parabolic or trapezoidal cross-section with side slopes no steeper than 1:3 is recommended to maximize the wetted channel perimeter of the swale. Recommendations for longitudinal channel slopes vary within the existing literature. For example, Schueler (1987) recommends a vegetated swale slope as close to zero as drainage permits. The Minnesota Pollution Control Agency (1991) recommends that the channel slope be less than 2 percent. The Storm Water Management Manual for the Puget Sound Basin (1992) specifies channel slopes between 2 and 4 percent. This manual indicates that slopes of less than 2 percent can be used if drain tile is incorporated into the design, while slopes greater than 4 percent can be used if check dams are placed in the channel to reduce flow velocity.

Flows

A typical design storm used for sizing swales is a six-month frequency, 24-hour storm event. The exact intensity of this storm must be determined for your location and is generally available from the U.S. Geological Survey. Swales are generally not used where the maximum flow rate exceeds 140 liters/second (5 cubic feet per second).

Sizing Procedures

The width of the swale can be calculated using various forms of the Manning equation. However, this methodology can be simplified to the following rule of thumb: the total surface area of the swale should be one percent of the area (500 square feet for each acre) that drains to the swale.

Unless a bypass is provided, the swale must be sized both to treat the design flows and to pass the peak hydraulic flows. However, for the swale to work most effectively, the depth of the storm water should not exceed the height of the grass.

Construction

The subsurface of the swale should be carefully constructed to avoid compaction of the soil. Compacted soil reduces infiltration and inhibits growth of the grass. Damaged areas should be restored immediately to ensure that the desired level of treatment is maintained and to prevent further damage from erosion of exposed soil.

Check Dams

Check dams can be installed in swales to promote additional infiltration, to increase storage, and to reduce flow velocities. Earthen check dams are not recommended because of their potential to erode. Check dams should be installed every 17 meters (50 feet) if the longitudinal slope exceeds 4 percent.

Performance

The literature suggests that vegetated swales represent a practical and potentially effective technique for controlling urban runoff quality. While limited quantitative performance data exists for vegetated swales, it is known that check dams, slight slopes, permeable soils, dense grass cover, increased contact time, and small storm events all contribute to successful pollutant removal by the swale system. Factors decreasing the effectiveness of swales include compacted soils, short runoff contact time, large storm events, frozen ground, short grass heights, steep slopes, and high runoff velocities and discharge rates.
Conventional vegetated swale designs have achieved mixed results in removing particulate pollutants. A study performed by the Nationwide Urban Runoff Program (NURP) monitored three grass swales in the Washington, D.C., area and found no significant improvement in urban runoff quality for the pollutants analyzed. However, the weak performance of these swales was attributed to the high flow velocities in the swales, soil compaction, steep slopes, and short grass height. Another project in Durham, NC, monitored the performance of a carefully designed artificial swale that received runoff from a commercial parking lot. The project tracked 11 storms and concluded that particulate concentrations of heavy metals (Cu, Pb, Zn, and Cd) were reduced by approximately 50 percent. However, the swale proved largely ineffective for removing soluble nutrients. A conservative estimate would say that a properly designed vegetated swale may achieve a 25 to 50 percent reduction in particulate pollutants, including sediment and sediment-attached phosphorus, metals, and bacteria. Lower removal rates (less than 10 percent) can be expected for dissolved pollutants, such as soluble phosphorus, nitrate, and chloride. Table 1 summarizes some pollutant removal efficiencies for vegetated swales.

The effectiveness of vegetated swales can be enhanced by adding check dams at approximately 17 meter (50 foot) increments along their length (See Figure 1). These dams maximize the retention time within the swale, decrease flow velocities, and promote particulate settling. Structures to skim off floating debris may also be added to the swales. Finally, the incorporation of vegetated filter strips parallel to the top of the channel banks can help to treat sheet flows entering the swale.

**OPERATION AND MAINTENANCE**

The useful life of a vegetated swale system is directly proportional to its maintenance frequency. If properly designed and regularly maintained, vegetated swales can last indefinitely.

The maintenance objectives for vegetated swale systems include keeping up the hydraulic and removal efficiency of the channel and maintaining a dense, healthy grass cover. Maintenance activities should include periodic mowing (with grass never cut shorter than the design flow depth), weed control, watering during drought conditions, reseeding of bare areas, and clearing of debris and blockages. Cuttings should be removed from the channel and disposed in a local composting facility. Accumulated sediment should also be removed manually to avoid the transport of resuspended sediments in periods of low flow and to prevent a damming effect from sand bars. The application of fertilizers and pesticides should be minimal.

Another aspect of a good maintenance plan is repairing damaged areas within a channel. For example, if the channel develops ruts or holes, it should be repaired utilizing a suitable soil that is properly tamped and seeded. The grass cover should be thick; if it is not, reseed as necessary.

Any standing water removed during the maintenance operation must be disposed to a sanitary sewer at an approved discharge location. Residuals (e.g., silt, grass cuttings) must be disposed in accordance with local or State requirements.

**COSTS**

Vegetated swales typically cost less to construct than curbs and gutters or underground storm
sewers. Schueler (1987) reported that costs may vary from $16-$30 per linear meter ($4.90 to $9.00 per linear foot) for a 4.5 meter (15-foot) wide channel (top width).

The Southeastern Wisconsin Regional Planning Commission (SEWRPC, 1991) reported that costs may vary from $28 to $164 per linear meter ($8.50 to $50.00 per linear foot) depending upon swale depth and bottom width. These cost estimates are higher than other published estimates because they include the cost of activities (such as clearing, grubbing, leveling, filling, and sodding) that may not be included in other published estimates. Construction costs depend on specific site considerations and local costs for labor and materials. Table 2 shows the estimated capital costs of a vegetated swale.

Annual costs for maintaining vegetated swales are approximately $1.90 per linear meter ($0.58 per linear foot) for a 0.5 meter (1.5-foot) deep channel, according to SEWRPC (1991). Average annual operating and maintenance costs of vegetated swales can be estimated using Table 3.

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ADDITIONAL INFORMATION

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TABLE 2  ESTIMATED CAPITAL COST OF A 1.5- FOOT DEEP, 10-FOOT-WIDE GRASSED SWALES*

<table>
<thead>
<tr>
<th>Component</th>
<th>Unit</th>
<th>Extent</th>
<th>Low</th>
<th>Moderate</th>
<th>High</th>
<th>Low</th>
<th>Moderate</th>
<th>High</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mobilization / Demobilization-Light</td>
<td>Swale</td>
<td>1</td>
<td>$107</td>
<td>$274</td>
<td>$441</td>
<td>$107</td>
<td>$274</td>
<td>$441</td>
</tr>
<tr>
<td>Site Preparation</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Clearing*</td>
<td>Acre</td>
<td>0.5</td>
<td>$2,200</td>
<td>$3,800</td>
<td>$5,400</td>
<td>$1,100</td>
<td>$1,900</td>
<td>$2,700</td>
</tr>
<tr>
<td>Grubbling*</td>
<td>Acre</td>
<td>0.25</td>
<td>$3,800</td>
<td>$5,200</td>
<td>$6,600</td>
<td>$950</td>
<td>$1,300</td>
<td>$1,650</td>
</tr>
<tr>
<td>General</td>
<td>Yd²</td>
<td>372</td>
<td>$2.10</td>
<td>$3.70</td>
<td>$5.30</td>
<td>$781</td>
<td>$1,376</td>
<td>$1,972</td>
</tr>
<tr>
<td>Level and Till*</td>
<td>Yd²</td>
<td>1,210</td>
<td>$0.20</td>
<td>$0.35</td>
<td>$0.50</td>
<td>$242</td>
<td>$424</td>
<td>$605</td>
</tr>
<tr>
<td>Sites Development</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Salvaged Topsoil</td>
<td>Yd²</td>
<td>1,210</td>
<td>$0.40</td>
<td>$1.00</td>
<td>$1.60</td>
<td>$484</td>
<td>$1,210</td>
<td>$1,936</td>
</tr>
<tr>
<td>Seed, and Mulch*</td>
<td>Yd²</td>
<td>1,210</td>
<td>$1.20</td>
<td>$2.40</td>
<td>$3.60</td>
<td>$1,452</td>
<td>$2,904</td>
<td>$4,356</td>
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<tr>
<td>Subtotal</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
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<td></td>
<td></td>
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<tr>
<td>Contingencies</td>
<td>Swale</td>
<td>1</td>
<td>25%</td>
<td>25%</td>
<td>25%</td>
<td>$1,279</td>
<td>$2,347</td>
<td>$3,415</td>
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<tr>
<td>Total</td>
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<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>$5,395</td>
<td>$11,735</td>
<td>$17,075</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Source: (SEWRPC, 1991)

Note: Mobilization/demobilization refers to the organization and planning involved in establishing a vegetative swale.
* Swale has a bottom width of 1.0 foot, a top width of 10 feet with 1:3 side slopes, and a 1,000-foot length.
* Area cleared = (top width + 10 feet) x swale length.
* Area grubbed = (top width x swale length).
* Volume excavated = (0.87 x top width x swale depth) x swale length (parabolic cross-section).
* Area sodded = area cleared x 0.5.
* Area planted = area cleared x 0.5.
TABLE 3 ESTIMATED OPERATION AND MAINTENANCE COSTS

<table>
<thead>
<tr>
<th>Component</th>
<th>Unit Cost</th>
<th>Swale Size (Depth and Top Width)</th>
<th>Comment</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>1.5 Foot Depth, One-Foot Bottom Width, 10-Foot Top Width</td>
<td>3-Foot Depth, 3-Foot Bottom Width, 21-Foot Top Width</td>
</tr>
<tr>
<td>Lawn Mowing</td>
<td>$0.85 / 1,000 ft³/mowing</td>
<td>$0.14 / linear foot</td>
<td>$0.21 / linear foot</td>
</tr>
<tr>
<td>General Lawn Care</td>
<td>$9.00 / 1,000 ft³/year</td>
<td>$0.18 / linear foot</td>
<td>$0.28 / linear foot</td>
</tr>
<tr>
<td>Swale Debris and Litter Removal</td>
<td>$0.10 / linear foot / year</td>
<td>$0.10 / linear foot</td>
<td>$0.10 / linear foot</td>
</tr>
<tr>
<td>Grass Seeding with Mulch and Fertilizer</td>
<td>$0.30 / yd²</td>
<td>$0.01 / linear foot</td>
<td>$0.01 / linear foot</td>
</tr>
<tr>
<td>Program Administration and Swale Inspection</td>
<td>$0.15 / linear foot / year, plus $25 / inspection</td>
<td>$0.15 / linear foot</td>
<td>$0.15 / linear foot</td>
</tr>
<tr>
<td>Total</td>
<td></td>
<td>$0.58 / linear foot</td>
<td>$0.75 / linear foot</td>
</tr>
</tbody>
</table>


The mention of trade names or commercial products does not constitute endorsement or recommendation for the use by the U.S. Environmental Protection Agency.

For more information contact:

Municipal Technology Branch
U.S. EPA
Mail Code 4204
401 M St., S.W.
Washington, DC, 20460

MTB
Endorse to complete through mailed individual applicants
MUNICIPAL TECHNOLOGY BRANCH
Grassed Filter Strip
Post-Construction Storm Water Management in New Development & Redevelopment

Grassed Filter Strip

Description
Grassed filter strips (vegetated filter strips, filter strips, and grassed filters) are vegetated surfaces that are designed to treat sheet flow from adjacent surfaces. Filter strips function by slowing runoff velocities and filtering out sediment and other pollutants, and by providing some infiltration into underlying soils. Filter strips were originally used as an agricultural treatment practice, and have more recently evolved into an urban practice. With proper design and maintenance, filter strips can provide relatively high pollutant removal. One challenge associated with filter strips, however, is that it is difficult to maintain sheet flow, so the practice may be “short circuited” by concentrated flows, receiving little or no treatment.

Applicability
Filter strips are applicable in most regions, but are restricted in some situations because they consume a large amount of space relative to other practices. Filter strips are best suited to treating runoff from roads and highways, roof downsputs, very small parking lots, and pervious surfaces. They are also ideal components of the “outer zone” of a stream buffer (see Buffer Zones fact sheet), or as pretreatment to a structural practice. This recommendation is consistent with recommendations in the agricultural setting that filter strips are most effective when combined with another practice (Magette et al., 1989). In fact, the most recent storm water manual for Maryland does not consider the filter strip as a treatment practice, but does offer storm water volume reductions in exchange for using filter strips to treat some of a site.

Regional Applicability
Filter strips can be applied in most regions of the country.
In arid areas, however, the cost of irrigating the grass on the practice will most likely outweigh its water quality benefits.

Ultra-Urban Areas

Ultra-urban areas are densely developed urban areas in which little pervious surface exists. Filter strips are impractical in ultra-urban areas because they consume a large amount of space.

Storm Water Hot Spots

Storm water hot spots are areas where land use or activities generate highly contaminated runoff, with concentrations of pollutants in excess of those typically found in storm water. A typical example is a gas station. Filter strips should not receive hot spot runoff, because the practice encourages infiltration. In addition, it is questionable whether this practice can reliably remove pollutants, so it should definitely not be used as the sole treatment of hot spot runoff.

Storm Water Retrofit

A storm water retrofit is a storm water management practice (usually structural), put into place after development has occurred, to improve water quality, protect downstream channels, reduce flooding, or meet other specific objectives. Filter strips are generally a poor retrofit option because they consume a relatively large amount of space and cannot treat large drainage areas.

Cold Water (Trout) Streams

Some cold water species, such as trout, are sensitive to changes in temperature. While some treatment practices, such as wet ponds (see Wet Ponds fact sheet), can warm storm water substantially, filter strips do not warm pond water on the surface for long periods of time and are not expected to increase storm water temperatures. Thus, these practices are good for protection of cold-water streams.

Siting and Design Considerations

Siting Considerations

In addition to the restrictions and modifications to adapting filter strips to different regions and land uses, designers need to ensure that this management practice is feasible at the site in question. The following section provides basic guidelines for siting filter strips.

Drainage Area

Typically, filter strips are used to treat very small drainage areas. The limiting design factor, however, is not the drainage area the practice treats but the length of flow leading to it. As storm water runoff flows over the ground’s surface, it changes from sheet flow to concentrated flow. Rather than moving uniformly over the surface, the concentrated flow forms rivulets which are slightly deeper and cover less area than the sheet flow. When flow concentrates, it moves too rapidly to be effectively treated by a grassed filter strip. As a rule, flow concentrates within a maximum of 75 feet for impervious surfaces, and 150 feet for pervious surfaces (CWP, 1996). Using this rule, a filter strip can treat one acre of impervious surface per 580-foot length.

Slope

Filter strips should be designed on slopes between 2 and 6 percent. Greater slopes than this would encourage the formation of concentrated flow. Except in the case of very sandy or gravelly soil, runoff would pond on the surface on slopes flatter than 2 percent, creating potential mosquito breeding habitat.

Soils / Topography
Filter strips should not be used on soils with a high clay content, because they require some infiltration for proper treatment. Very poor soils that cannot sustain a grass cover crop are also a limiting factor.

**Ground Water**

Filter strips should be separated from the ground water by between 2 and 4 ft to prevent contamination and to ensure that the filter strip does not remain wet between storms.

**Design Considerations**

Filter strips appear to be a minimal design practice because they are basically no more than a grassed slope. However, some design features are critical to ensure that the filter strip provides some minimum amount of water quality treatment.

- A pea gravel diaphragm should be used at the top of the slope. The pea gravel diaphragm (a small trench running along the top of the filter strip) serves two purposes. First, it acts as a pretreatment device, settling out sediment particles before they reach the practice. Second, it acts as a level spreader, maintaining sheet flow as runoff flows over the filter strip.
- The filter strip should be designed with a pervious berm of sand and gravel at the toe of the slope. This feature provides an area for shallow ponding at the bottom of the filter strip. Runoff ponds behind the berm and gradually flows through outlet pipes in the berm. The volume ponded behind the berm should be equal to the water quality volume. The water quality volume is the amount of runoff that will be treated for pollutant removal in the practice. Typical water quality volumes are the runoff from a 1-inch storm or 1/2-inch of runoff over the entire drainage area to the practice.
- The filter strip should be at least 25 feet long to provide water quality treatment.
- Designers should choose a grass that can withstand relatively high velocity flows and both wet and dry periods.
- Both the top and toe of the slope should be as flat as possible to encourage sheet flow and prevent erosion.

**Regional Variations**

In cold climates, filter strips provide a convenient area for snow storage and treatment. If used for this purpose, vegetation in the filter strip should be salt-tolerant, (e.g., creeping bentgrass), and a maintenance schedule should include the removal of sand built up at the bottom of the slope. In arid or semi-arid climates, designers should specify drought-tolerant grasses (e.g., buffalo grass) to minimize irrigation requirements.

**Limitations**

Filter strips have several limitations related to their performance and space consumption:

- The practice has not been shown to achieve high pollutant removal.
- Filter strips require a large amount of space, typically equal to the impervious area they treat, making them often infeasible in urban environments where land prices are high.
- If improperly designed, filter strips can become a mosquito breeding ground.
- Proper design requires a great deal of finesse, and slight problems in the design, such as improper grading, can render the practice ineffective in terms of pollutant removal.

**Maintenance Considerations**

Filter strips require similar maintenance to other vegetative practices (see **Grassed Swales** fact sheet). These maintenance needs are outlined below. Maintenance is very important for filter strips, particularly in terms of ensuring that flow does not short circuit the practice.
Table 1. Typical maintenance activities for grassed filter strips (Source: CWP, 1999)

<table>
<thead>
<tr>
<th>Activity</th>
<th>Schedule</th>
</tr>
</thead>
<tbody>
<tr>
<td>- Inspect pea gravel diaphragm for clogging and remove built-up sediment.</td>
<td>Annual inspection (semi-annual the first year)</td>
</tr>
<tr>
<td>- Inspect vegetation for rills and gullies and correct. Seed or sod bare areas.</td>
<td></td>
</tr>
<tr>
<td>- Inspect to ensure that grass has established. If not, replace with an alternative species.</td>
<td></td>
</tr>
<tr>
<td>- Mow grass to maintain a 3&amp;ndash;4 inch height</td>
<td>Regular (frequent)</td>
</tr>
<tr>
<td>- Remove sediment build-up within the bottom when it has accumulated to 25% of the original capacity.</td>
<td>Regular (infrequent)</td>
</tr>
</tbody>
</table>

Effectiveness

Structural storm water management practices can be used to achieve four broad resource protection goals. These include flood control, channel protection, ground water recharge, and pollutant removal. The first two goals, flood control and channel protection, require that a storm water practice be able to reduce the peak flows of relatively large storm events (at least 1- to 2-year storms for channel protection and at least 10- to 50-year storms for flood control). Filter strips do not have the capacity to detain these events, but can be designed with a bypass system that routes these flows around the practice entirely.

Filter strips can provide a small amount of ground water recharge as runoff flows over the vegetated surface and ponds at the toe of the slope. In addition, it is believed that filter strips can provide modest pollutant removal. Studies from agricultural settings suggest that a 15-foot-wide grass buffer can achieve a 50 percent removal rate of nitrogen, phosphorus, and sediment, and that a 100-foot buffer can reach closer to 70 percent removal of these constituents (Desbonette et al., 1994). It is unclear how these results can be translated to the urban environment, however. The characteristics of the incoming flows are radically different both in terms of pollutant concentration and the peak flows associated with similar storm events. To date, only one study (Yu et al., 1992) has investigated the effectiveness of a grassed filter strip to treat runoff from a large parking lot. The study found that the pollutant removal varied depending on the length of flow in the filter strip. The narrower (75-foot) filter strip had moderate removal for some pollutants and actually appeared to export lead, phosphorus, and nutrients (See Table 2).

Table 2. Pollutant removal of an urban vegetated filter strip (Source: Yu et al., 1993)

<table>
<thead>
<tr>
<th>Pollutant</th>
<th>Pollutant Removal (%)</th>
<th>75-FT Filter Strip</th>
<th>150-FT Filter Strip</th>
</tr>
</thead>
<tbody>
<tr>
<td>Total suspended solids</td>
<td>54</td>
<td>84</td>
<td></td>
</tr>
<tr>
<td>Nitrate+nitrite</td>
<td>-27</td>
<td>20</td>
<td></td>
</tr>
<tr>
<td>Total phosphorus</td>
<td>-25</td>
<td>40</td>
<td></td>
</tr>
<tr>
<td>Extractable lead</td>
<td>-16</td>
<td>50</td>
<td></td>
</tr>
<tr>
<td>Extractable zinc</td>
<td>47</td>
<td>55</td>
<td></td>
</tr>
</tbody>
</table>

Cost Considerations

Little data are available on the actual construction costs of filter strips. One rough estimate can be the cost of seed or sod, which is approximately 30¢ per ft² for seed or 70¢ per ft² for sod. This amounts to between $13,000 and $30,000 per acre for a filter strip, or the same amount per impervious acre treated. This cost is relatively high compared with other treatment practices. However, the grassed area used as a filter strip may have been seeded or sodded even if it were not used for treatment. In these cases, the only additional
costs are the design, which is minimal, and the installation of a berm and gravel diaphragm. Typical maintenance costs are about $350/acre/year (adapted from SWRPC, 1991). This cost is relatively inexpensive and, again, might overlap with regular landscape maintenance costs.

The true cost of filter strips is the land they consume, which is higher than for any other treatment practice. In some situations this land is available as wasted space beyond back yards or adjacent to roadsides, but the practice is cost-prohibitive when land prices are high and land could be used for other purposes.

References

Design Reference


Other References


Information Resources


SILT FENCE GENERAL NOTES:

1. STEEL POSTS WHICH SUPPORT THE SILT FENCE SHALL BE INSTALLED ON A SLIGHT ANGLE TOWARD THE ANTICIPATED RUNOFF SOURCE. POST MUST BE EMBEDDED A MINIMUM OF ONE FOOT.

2. THE TOE OF THE SILT FENCE SHALL BE TRENCHED IN WITH A SPADE OR MECHANICAL TRENCHER, SO THAT THE DOWNSLOPE FACE OF THE TRENCH IS FLAT AND PERPENDICULAR TO THE LINE OF FLOW. WHERE FENCE CANNOT BE TRENCHED IN (E.G. PAVEMENT), WEIGHT FABRIC FLAP WITH ROCK ON UPHILL SIDE TO PREVENT FLOW FROM SEEPING UNDER FENCE.

3. THE TRENCH MUST BE A MINIMUM OF 6 INCHES DEEP AND 6 INCHES WIDE TO ALLOW FOR THE SILT FENCE FABRIC TO BE LAID IN THE GROUND AND BACKFILLED WITH COMPACTED MATERIAL.

4. SILT FENCE SHOULD BE SECURELY FASTENED TO EACH STEEL SUPPORT POST OR TO WOVEN WIRE, WHICH IN TURN IS ATTACHED TO THE STEEL FENCE POST. THERE SHALL BE A 3 FOOT OVERLAP, SECURELY FASTENED WHERE ENDS OF FABRIC MEET.

5. INSPECTION SHALL BE MADE EVERY TWO WEEKS AND AFTER EACH 1/2" RAINFALL. REPAIR OR REPLACEMENT SHALL BE MADE PROMPTLY AS NEEDED.

6. SILT FENCE SHALL BE REMOVED WHEN THE SITE IS COMPLETELY STABILIZED SO AS NOT TO BLOCK OR IMPED STORM FLOW OR DRAINAGE.

7. ACCUMULATED SILT SHALL BE REMOVED WHEN IT REACHES A DEPTH OF HALF THE HEIGHT OF THE FENCE. THE SILT SHALL BE DISPOSED OF AT AN APPROVED SITE AND IN SUCH A MANNER AS TO NOT CONTRIBUTE TO ADDITIONAL SILTATION.
In-line Storage
Post-Construction Storm Water Management in New Development & Redevelopment

In-Line Storage

Description

In-line storage refers to a number of practices designed to use the storage within the storm drain system to detain flows. While these practices can reduce storm peak flows, they are unable to improve water quality or protect downstream channels. Storage is achieved by placing devices in the storm drain system to restrict the rate of flow. Devices can slow the rate of flow by backing up flow, as in the case of a dam or weir, or through the use of vortex valves, devices that reduce flow rates by creating a helical flow path in the structure. A description of various flow regulators is included in Urbanos and Stahre (1990).

Applicability

In-line storage practices serve the same purpose as traditional detention basins (see Dry Extended Detention Pond). These practices can act as a surrogate for aboveground storage when little space is available for aboveground storage facilities.

Limitations

In-line storage has several limitations, including:

- In-line storage practices only control flow, and thus are not able to improve the water quality of storm water runoff.
- If improperly designed, these practices may cause upstream flooding.

Siting and Design Considerations
Flow regulators cannot be applied to all storm drain systems. In older cities, the storm drainpipes may not be oversized, and detaining storm water within them would cause upstream flooding. Another important issue in siting these practices is the slope of the pipes in the system. In areas with very flat slopes, restricting flow within the system is likely to cause upstream flooding because introducing a regulator into the system will cause flows to back up a long distance before the regulator. In steep pipes, on the other hand, a storage flow regulator cannot utilize much of the storage available in the storm drain system.

Maintenance Considerations

Flow regulators require very little maintenance, because they are designed to be "self cleaning," much like the storm drain system. In some cases, flow regulators may be modified based on downstream flows, new connections to the storm drain, or the application of other flow regulators within the system. For some designs, such as check dams, regulations will require only moderate construction in order to modify the structure's design.

Effectiveness

The effectiveness of in-line storage practices is site-specific and depends on the storage available in the storm drain system. In one study, a single application was able to reduce peak flows by approximately 50 percent (VDCR, 1999).

Cost Considerations

Flow regulators are relatively low cost options, particularly since they require little maintenance and consume little surface area.

References


Green Parking
Post-Construction Storm Water Management in New Development & Redevelopment

Green Parking

Description

Green parking refers to several techniques applied together to reduce the contribution of parking lots to the total impervious cover in a lot. From a storm water perspective, application of green parking techniques in the right combination can dramatically reduce impervious cover and, consequently, the amount of storm water runoff. Green parking lot techniques include setting maximums for the number of parking lots created, minimizing the dimensions of parking lot spaces, utilizing alternative pavers in overflow parking areas, using bioretention areas to treat storm water, encouraging shared parking, and providing economic incentives for structured parking.

Applicability

All of the green parking techniques can be applied in new developments and some can be applied in redevelopment projects, depending on the extent and parameters of the project. In urban areas, application of some techniques, like encouraging shared parking and providing economic incentives for structured parking, can be very practical and necessary. Commercial areas can have excessively high parking ratios, and application of green parking techniques in various combinations can dramatically reduce the impervious cover of a site.

Implementation

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. By determining
average parking demand instead, a lower maximum number of parking spaces can be set to accommodate most of the demand.

Table 1 provides examples of conventional parking requirements and compares them to average parking demand.

Table 1: Conventional minimum parking ratios (Source: ITE, 1987; Smith, 1984; Wells, 1994)

<table>
<thead>
<tr>
<th>Land Use</th>
<th>Parking Requirement</th>
<th>Actual Average Parking Demand</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Parking Ratio</td>
<td>Typical Range</td>
</tr>
<tr>
<td>Single family homes</td>
<td>2 spaces per</td>
<td>1.5 – 2.5</td>
</tr>
<tr>
<td></td>
<td>dwelling unit</td>
<td></td>
</tr>
<tr>
<td>Shopping center</td>
<td>5 spaces per</td>
<td>4.08 – 8.5</td>
</tr>
<tr>
<td></td>
<td>1000 ft² GFA</td>
<td></td>
</tr>
<tr>
<td>Convenience store</td>
<td>3.3 spaces per</td>
<td>2.0 – 10.0</td>
</tr>
<tr>
<td></td>
<td>1000 ft² GFA</td>
<td></td>
</tr>
<tr>
<td>Industrial</td>
<td>1 space per</td>
<td>0.5 – 2.0</td>
</tr>
<tr>
<td></td>
<td>1000 ft² GFA</td>
<td></td>
</tr>
<tr>
<td>Medical/ dental</td>
<td>5.7 spaces per</td>
<td>4.58 – 10.0</td>
</tr>
<tr>
<td>office</td>
<td>1000 ft² GFA</td>
<td></td>
</tr>
</tbody>
</table>

GFA = Gross floor area of a building without storage or utility spaces.

Another green parking lot technique is to minimize the dimensions of the parking spaces. This can be accomplished by reducing both the length and width of the parking stall. Parking stall dimensions can be further reduced if compact spaces are provided. While the trend toward larger sport utility vehicles (SUVs) is often cited as a barrier to implementing stall minimization technique, stall width requirements in most local parking codes are much larger than the widest SUVs (CWP, 1998).

Utilizing alternative pavers is also an effective green parking technique. They can replace conventional asphalt or concrete in both new developments and redevelopment projects. Alternative pavers can range from medium to relatively high effectiveness in meeting storm water quality goals. The different types of alternative pavers include gravel, cobbles, wood mulch, brick, grass pavers, turf blocks, natural stone, pervious concrete, and porous asphalt. In general, alternate pavers require proper installation and more maintenance than conventional asphalt or concrete. For more specific information on alternate pavers, refer to the Alternative Pavers fact sheet.

Bioretention areas can effectively treat storm water leaving a parking lot. Storm water is directed into a shallow, landscaped area and temporarily detained. The runoff then filters down through the bed of the facility and is infiltrated into the subsurface or collected into an underdrain pipe for discharge into a stream or another storm water facility. Bioretention facilities can be attractively integrated into landscaped areas and can be maintained by commercial landscaping firms. For detailed design specifications of bioretention areas, refer to the Bioretention fact sheet.

Shared parking in mixed-use areas and structured parking are also green parking techniques that can further reduce the conversion of land to impervious cover. A shared parking arrangement could include usage of the same parking lot by an office space that experiences peak parking demand during the weekday with a church that experiences parking demands during the weekends and evenings. Costs may dictate the usage of structured parking, but building upward or downward can help minimize surface parking.

Limitations

Some limitations to applying green parking techniques include applicability, cost, and maintenance. For example, shared parking is only practical in mixed use areas, and structured parking may be limited by the cost of land versus construction. Alternative pavers are currently only recommended for overflow parking because of the considerable cost of maintenance. Bioretention areas
increase construction costs.

The pressure to provide excessive parking spaces can come from fear of complaints as well as requirements of bank loans. These factors can pressure developers to construct more parking than necessary and present possible barriers to providing the greenest parking lot possible.

Effectiveness

Applied together, green parking techniques can effectively reduce the amount of impervious cover, help to protect local streams, result in storm water management costs savings, and visually enhance a site. Proper design of bioretention areas can help meet storm water management and landscaping requirements while keeping maintenance costs at a minimum.

Utilizing green parking lots can dramatically reduce the amount of impervious cover created. The level of the effectiveness depends on how much impervious cover is reduced as well as the combination of techniques utilized to provide the greenest parking lot. While the pollutant removal rates of bioretention areas have not been directly measured, their capability is considered comparable to a dry swale, which removes 91 percent of total suspended solids, 67 percent of total phosphorous, 92 percent of total nitrogen, and 80-99 percent of metals (Claytor and Schueler, 1998).

An excellent example of the multiple benefits of rethinking parking lot design is the Fort Bragg vehicle maintenance facility parking lot in North Carolina (NRDC, 1999). This redesign reduced impervious cover by 40 percent, increased parking by 20 percent, and saved $1.6 million (20 percent) on construction costs over the original, conventional design. Stormwater management features, such as detention basins located within grassed islands and an onsite drainage system that took advantage of existing sandy soils, were incorporated into the parking lot design as well.

Cost Considerations

Setting maximums for parking spaces, minimizing stall dimensions, and encouraging shared parking can result in considerable construction cost savings. At the same time, implementing green parking techniques can also reduce storm water management costs.

References


Information Resources


Modular Treatment System
DESCRIPTION

This fact sheet describes modular systems for treating storm water. One of the primary modular storm water treatment systems currently on the market is the StormTreat™ System, or STS. The STS, which was developed in 1994, is a storm water treatment technology consisting of a series of sedimentation chambers and constructed wetlands. These wetlands are contained within a modular, 2.9-meter (9.5 feet) diameter recycled-polyethylene tank. The STS can be applied in many different scenarios, ranging from residential areas to most industrial parks, but should not be used in extremely polluted areas, such as directly in wastewater streams. Figure 1 is a diagram of the STS. The STS works as follows: influent is piped into the unit’s sedimentation chambers, where pollutants are removed through sedimentation and filtration. Storm water is then conveyed from the sedimentation chambers to a surrounding constructed wetland. Unlike most constructed wetlands systems, STS conveys the storm water directly into the subsurface of the wetland and through the root zone. Pollutants are then removed through filtration, adsorption, and biochemical reactions. These processes occur at higher rates within the root zone, making STS more efficient in pollutant removal. Storm water is retained in the wetlands for five to ten days prior to discharge.

![Diagram of StormTreat™ System](image)


FIGURE 1 STORMTREAT™ SYSTEM
The STS is suitable for use throughout the U.S.; however, the system may require modification to function in different environments. For example, as an option in dry climates such as in the southwestern U.S., STS has designed a solar-powered water pump to redirect water that is stored in the bottom of the system to the wetland plants. In addition, in arid regions such as these that do not have enough groundwater to support the wetland vegetation, the unit may be altered to release flow at a slower rate, thereby increasing the amount of water retained in the bottom of the unit; or it may be designed with soils that retain water more efficiently. Alternatively, the unit could have a backup water supply to provide for extended dry periods.

The STS design can be modified for areas with high groundwater levels or tidal influence. In areas with high groundwater, the discharge pipework can be modified so that runoff is discharged downgradient to an area with a lower water table. In tidally-influenced areas, a check valve can be installed to prevent flow from re-entering the unit at its discharge point. This will also allow discharge to be released only during mid- to low-tide conditions.

Over 100 STS units have been installed nationwide, including installations in California, Washington, Oregon, Oklahoma, North Carolina, South Carolina, Maryland, New York, Connecticut, New Hampshire, Maine, Rhode Island, and Massachusetts. An STS has been operating in Kingston, Massachusetts, since November 1994. This unit was installed to prevent bacterial contamination from storm runoff from harming shellfish beds in the Jones River. Additional systems have been recently installed in various parts of Massachusetts, as well as in Maine. In Hingham, MA, six STSs were installed in an industrial park bordering a wetland that is a tributary to a drinking water supply. These STSs have been successful in preventing contamination of the water supply. In Ipswich, MA, and Barnstable, MA, several STS tanks were installed to treat road and parking lot drainage to prevent discharges to sensitive receiving waters. Finally, in Manchester, ME, five STSs were installed to help reduce the levels of phosphorus in storm water effluent after new regulations tightened runoff standards for phosphorous.

APPLICABILITY

The STS has applications in a wide range of settings. The system’s size and modular configuration make it adaptable to a wide range of site constraints and watershed sizes. Designers of the system indicate that the system can be used to treat runoff from highways, parking lots, airports, marinas, and commercial, industrial, and residential areas. The STS is an appropriate storm water treatment technology for both coastal and inland areas but is not designed to be used directly in wastewater streams.

ADVANTAGES AND DISADVANTAGES

Regulators and environmental groups in Massachusetts are utilizing storm water management practices, including the STS, to improve water quality in the shellfish beds located downstream from potentially contaminated runoff. The STS also protects groundwater by removing pollutants prior to infiltration. The STS has shown high total petroleum hydrocarbons (TPH), Total Phosphorus (TP), metals, and suspended solids removal rates, which improves water quality. An additional benefit of the STS is the system's spill containment feature, which can capture an upstream release and therefore lessen the spill’s impact on the environment. However, as previously discussed, the STS is relatively new and remains to be thoroughly tested in different geographical locations. There may be possible limitations in different areas, although soil types and high water tables surrounding the modular unit will not limit the system's effectiveness.

DESIGN CRITERIA

The STS is a modular, 2.9-meter (9.5-foot) diameter recycled-polyethylene tank containing a series of sedimentation chambers and constructed wetlands. The sedimentation chambers are in the inner ring of the tank, which has a diameter of nearly 1.7-meters (5.5 feet). The 2.9-meter diameter outer ring, which surrounds the sedimentation chambers, contains the wetland. The tank walls and bulkheads, which separate the sedimentation chambers, are 1.2-meters (4 feet) high.
STS tanks are designed to withstand the weight of the saturated soils surrounding the tanks. Influent is conveyed from a catch basin (and other preliminary detention structures) through poly-vinyl chloride (PVC) piping to the first of six internal sedimentation chambers. A synthetic woven sack placed at the end of the 10 centimeter (4 inch) diameter inlet pipe traps large particles and debris. Skimmers floating on the water surface within each chamber convey flow to the following chamber through an opening 15 centimeters (6 inches) below the surface. This prevents sediment and floatables from being transported to the subsequent chamber. Sediments that collect in the bottom of the chamber remain there until the unit is cleaned. The bulkhead separating the last two sedimentation chambers is fitted with an inverted elbow, which traps oil and grease. The settling efficiency increases by transferring water from the top of each chamber to the subsequent chamber.

Flow is conveyed from the final sedimentation chamber through four, slotted PVC outlet pipes, each 10 cm (4 inches) in diameter, into the wetland portion of the STS. Partially treated storm water flows beneath the soil through the wetland. The wetland has an approximate storage capacity of 2,880 liters (760 gallons). The entire system has a static holding volume of 5,270 liters (1,390 gallons). However, the system is sized based upon this volume plus associated detention structures.

Vegetation within the wetland will vary depending on the local conditions (climatological). Bulrush and burreeds (which have maximum root depths of 0.8 and 0.6 meters (2.6 and 2 feet), respectively [U.S. EPA, 1993]) have been used in Massachusetts. Mature vegetation in the outer ring should have roots that extend into the permanent 15 cm (6 inches) of water in the bottom of the tank. Insufficient root depth may result in a lack of water supply to the plants during the periods between storm events.

Effluent from the wetland is discharged through a 5 centimeter (2 inch) diameter pipe that is controlled by a valve. Flow rates and holding times can be varied by manipulating the outlet control valve. At the Kingston facility, the control valve is adjusted to provide the recommended discharge rate of 0.1 liters per second (0.2 gallons per minute) and a five day holding time in the wetland. The valve has an added benefit that in the event of an upstream toxic spill, it can be closed, trapping the pollutants in the STS.

Tanks are available in one size, but several tanks can be installed at a site to capture the projected volume of runoff. The determination of the number of tanks needed for a site is based on three factors:

- Area of impervious drainage surfaces.
- Design storm to be treated.
- Detention storage prior to the STS tanks.

Generally 1-2 units are required for each acre of impervious surface. The system is sized based upon the design storm which is determined by state regulations (i.e., Maine requires treatment of first half inch of storm and Washington requires treatment of a six month storm). This first flush storage volume is stored in preliminary storage structures such as underground tanks and large diameter pipes (which can be placed underground parking areas).

**PERFORMANCE**

Runoff from the STS installed in Kingston, MA, was analyzed to assess pollutant removal efficiency. Thirty-three samples were collected over eight independent storm events during both winter and summer conditions. Sampling results are shown in Table 1. The results indicate removal rates of 97 percent for fecal coliform bacteria, 99 percent for total suspended solids, and 90 percent for total petroleum hydrocarbons. Nutrient removal rates were 82 percent chemical oxygen demand, 77 percent total dissolved nitrogen, and 90 percent phosphorus. Metal removal rates were 77 percent for lead, 98 percent for chromium, and 90 percent for zinc.

In addition to the study in Kingston, MA, several other studies are currently being conducted in Connecticut, California, and Massachusetts. This
TABLE 1 STORMTREAT™ SAMPLING RESULTS FOR KINGSTON, MA

<table>
<thead>
<tr>
<th>Pollutant</th>
<th>Percent Removed</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fecal Coliform Bacteria</td>
<td>97</td>
</tr>
<tr>
<td>Total Suspended Solids</td>
<td>99</td>
</tr>
<tr>
<td>Chemical Oxygen Demand</td>
<td>82</td>
</tr>
<tr>
<td>Total Dissolved Nitrogen</td>
<td>77</td>
</tr>
<tr>
<td>Phosphorous</td>
<td>90</td>
</tr>
<tr>
<td>Total Petroleum Hydrocarbons</td>
<td>90</td>
</tr>
<tr>
<td>Lead</td>
<td>77</td>
</tr>
<tr>
<td>Chromium</td>
<td>98</td>
</tr>
<tr>
<td>Zinc</td>
<td>90</td>
</tr>
</tbody>
</table>


Data has not been fully developed and is not yet available.

OPERATION AND MAINTENANCE

Anticipated maintenance of the STS is minimal. The system should be observed at least once a year to be sure that it is operating effectively. At that time, the burlap sack that covers the influent line should be replaced. If the installed system uses filters, these should be removed, cleaned, and rein stalled. Sediment should be removed from the system once every three to five years, more often if the system has higher than normal sediment loads. The sediment level may be measured with a probe or even a yard stick. It is recommended that the sediment be removed when 0.3 meters (1 foot) of sediment has accumulated. After six months of operation the unit installed in Kingston, MA was found to have 5 centimeters (2 inches) of accumulated sediment. The sediment can be pumped from the tank by septic haulers or by maintenance personnel responsible for sediment removal from catch basins. It is not anticipated that the sediment will be toxic, and it may be safely landfilled. However, sediment toxicity will depend on the activities in the contributing drainage area and testing of the sediment may be required to determine if it is considered hazardous. Because the STS system is relatively new, there is no definitive data on the lifetime of the plants and gravel in the system. However, it is estimated that these will need to be replaced every 10 to 20 years.

COSTS

The STS is a prefabricated unit that is easily installed in most locations. The cost for one unit is $4,900, and the installation cost is usually between $500 and $1,000 (which is provided by the manufacturer). Additional materials required include gravel, PVC piping, and wetland plants, at a total cost of about $350 to $400 per tank. Capital and installation costs per tank decrease as the number of units on a site increases. Installation will cost less if construction on that site is new (not retrofitted) because drainage lines will be more easily accessible. Installation will cost more if there are extra construction costs (for example, retrofit design) or if there are complications. StormTreat™ Systems recommends one STS unit per one acre of impervious surface.

The estimated maintenance cost for removal of sediment from one tank ranges from $80 to $120. This cost is incurred every three to five years, when sediment is removed. Costs have not been determined for an annual site inspection or for removing any debris from the wetland area. However, these costs should be minimal (i.e., one day of labor for one person per year).

REFERENCES


**ADDITIONAL INFORMATION**

Ecocycle
George Lord
P.O. Box 228
Manchester, ME 04351

Land Use Consultants, Inc.
Pat Clark
966 Riverside Street
Portland, ME 04103

New Hampshire Department of Environmental Services
Steve Landry
6 Hazen Drive
Concord, NH 03302

For more information contact:

Municipal Technology Branch
U.S. EPA
Mail Code 4204
401 M St., S.W.

The mention of trade names or commercial products does not constitute endorsement or recommendation for the use by the U.S. Environmental Protection Agency.
Water Quality Inlets
Storm Water
Technology Fact Sheet
Water Quality Inlets

DESCRIPTION

Water quality inlets (WQIs), also commonly called oil/grit separators or oil/water separators, consist of a series of chambers that promote sedimentation of coarse materials and separation of free oil (as opposed to emulsified or dissolved oil) from storm water. Most WQIs also contain screens to help retain larger or floating debris, and many of the newer designs also include a coalescing unit that helps to promote oil/water separation. WQIs typically capture only the first portion of runoff for treatment and are generally used for pretreatment before discharging to other best management practices (BMPs).

A typical WQI, as shown in Figure 1, consists of a sedimentation chamber, an oil separation chamber, and a discharge chamber. The basic WQI design is often modified to improve performance. Possible

FIGURE 1 PROFILE OF A TYPICAL WATER QUALITY INLET
modifications include: an additional orifice and chamber that replace the inverted pipe elbow; the extension of the second chamber wall up to the top of the structure; or the addition of a diffusion device at the inlet. The diffusion device is intended to dissipate the velocity head and turbulence and distribute the flow more evenly over the entire cross-sectional area of the sedimentation chamber (API, 1990).

The addition of a coalescing unit to the WQI can dramatically increase its effectiveness in oil/water separation while also greatly reducing the size of the required unit. Coalescing units are made from oil-attracting materials, such as polypropylene or other materials. These units attract small oil droplets, which begin to concentrate until they are large enough to float to the surface and separate from the storm water. Without these units, the oil and grease particles must concentrate and separate naturally. This requires a much larger surface area; and therefore, units that do not use the coalescing process must be larger than units utilizing a coalescing unit.

WQIs can be purchased as pre-manufactured units (primarily oil/water separator tanks) or constructed on site. Suppliers of pre-manufactured units (e.g., Highland Tank and Manufacturing, Jay R. Smith Manufacturing, etc.) can also provide modifications of the typical design for special conditions.

APPLICABILITY

WQIs are widely used in the U.S. and can be adapted to all regions of the country. They are often used where land requirements and cost prohibit the use of larger BMP devices, such as ponds or wetlands. WQIs are also used to treat runoff prior to discharge to other BMPs.

Because of their ability to remove hydrocarbons, WQIs are typically located at sites with automotive-related contamination or at other sites that generate high hydrocarbon concentrations (MWCDBG, 1993). For example, WQIs may be ideal for small, highly impervious areas, such as gas stations, loading areas, or parking areas (Schueler, 1992). Many WQIs, particularly those installed at industrial sites, serve the dual purpose of treating storm water runoff from contaminated areas, and serving as collection and treatment units for washdown processes or petroleum spills.

Higher residual hydrocarbon concentrations in trapped sediments cause maintenance and residual disposal costs associated with WQIs to be higher than those of other BMPs. Therefore, planners should carefully evaluate maintenance and residual disposal issues for the site before selecting a WQI. Possible alternatives to the WQI include sand filters, oil absorbent materials, and other innovative BMPs (e.g., Stormceptor System).

ADVANTAGES AND DISADVANTAGES

WQIs can effectively trap trash, debris, oil and grease, and other floatables that would otherwise be discharged to surface waters (Schueler, 1992). In addition, a properly designed and maintained WQI can serve as an effective BMP for reducing hydrocarbon contamination in receiving water sediments. While WQIs are effective in removing heavy sediments and floating oil and grease, they have demonstrated limited ability to separate dissolved or emulsified oil from runoff. WQIs are also not very effective at removing pollutants such as nutrients or metals, except where the metals removal is directly related to sediment removal.

Several major constraints can limit the effectiveness of WQIs. The first is the size of the drainage area. WQIs are generally recommended for drainage areas of 0.4 hectares (1 acre) or less (Berg, 1991; NVPDC, 1992). Construction costs often become prohibitive for larger drainage areas. However, because WQIs are primarily designed for specific industrial sites that have the potential for petroleum-contaminated process washdown, spills, and storm water runoff, sizing considerations are not usually a problem.

Sediment can also cause problems for WQIs. There are several reasons for this. First, high sediment loads can interfere with the ability of the WQI to effectively separate oil and grease from the runoff. Second, during periods of high flow, sediment residuals may be resuspended and released from the WQI to surface waters. A 1993 Metropolitan Washington Council of Governments (MWCDBG)
long-term study evaluating the performance and effectiveness of more than 100 WQIs found that pollutants in the WQI sediments were similar to those pollutants found in downstream receiving water sediments (the tidal Anacostia River). This information suggests that downstream sediment contamination is linked to contaminated runoff and pass-through from WQIs (MWCOG, 1993). Third, WQI residuals accumulate quickly and require frequent removal. There is also some concern that because the collected residuals contain hydrocarbon by-products, the residuals may be considered too toxic for conventional landfill disposal. The 1993 MWCOG study found that the residuals from WQIs typically contain many priority pollutants, including polycyclic aromatic hydrocarbons, trace metals, phthalates, phenol, toluene, and possibly methylene chloride (MWCOG, 1993). Based on these considerations, WQIs should not be implemented at sites that generate large amounts of sediment in the runoff unless the runoff has been pretreated to reduce the sediment loads to manageable levels.

WQIs are also limited by maintenance requirements. Maintenance of underground WQIs can be easily neglected because the WQI is often "out of sight and out of mind." Regular maintenance is essential to ensuring effective pollutant removal. As discussed above, lack of maintenance will often result in resuspension of settled pollutants.

Finally, WQIs generally provide limited hydraulic and residuals storage. Due to the limited storage, WQIs do not provide adequate storm water quantity control.

**DESIGN CRITERIA**

Prior to WQI design, the site should be evaluated to determine if another BMP would be more cost-effective in removing the pollutants of concern. WQIs should be used when no other BMP is feasible. The WQI should be constructed near a storm drain network so that flow can be easily diverted to the WQI for treatment (NVPDC, 1992). Any construction activities within the drainage area should be completed before installation of the WQI, and the drainage area should be revegetated so that the sediment loading to the WQI is minimized.

Upstream sediment control measures should be implemented to decrease sediment loading.

WQIs are most effective for small drainage areas. Drainage areas of 0.4 hectares (1 acre) or less are often recommended. WQIs are typically used in an off-line configuration (i.e., portions of runoff are diverted to the WQI), but they can be used as on-line units (i.e., receive all runoff). Generally, off-line units are designed to handle the first 1.3 centimeters (0.5 inches) of runoff from the drainage areas. Upstream isolation/diversion structures can be used to divert the water to the off-line structure (Schueler, 1992). On-line units receive higher flows that will likely cause increased turbulence and resuspension of settled material, thereby reducing WQI performance.

As discussed above, oil/water separation tank units are often utilized in specific industrial areas, such as airport aprons, equipment washdown areas, or vehicle storage areas. In these instances, runoff from the area of concern will usually be diverted directly into the unit, while all other runoff is sent to the storm drain downstream from the oil/water separator. Oil/water separation tanks are often fitted with diffusion baffles at the inlets to prevent turbulent flow from entering the unit and resuspending settled pollutants.

WQIs are available as pre-manufactured units or can be cast in place. Reinforced concrete should be used to construct below-grade WQIs. The WQIs should be water tight to prevent possible ground water contamination.

**Chamber Design**

Structural loadings should be considered in the WQI design (Berg, 1991), particularly with respect to the sizing of the chambers. When the combined length of the first two chambers exceeds 4 meters (12 feet), the chambers are typically designed with the length of the first and second chamber being two-thirds and one-third of the combined length of the unit, respectively. Each of the chambers should have a separate manhole to provide access for cleaning and inspection.
The State of Maryland design standards indicate that the combined volume of the first and second chambers should be determined based on 1.1 cubic meters (40 cubic feet) per 0.04 hectares (0.10 acres) draining to the WQI. In Maryland, this is equivalent to capturing the first 0.33 centimeters (0.133 inches) of runoff from the contributing drainage area.

Permanent pools within the chambers help prevent the possibility of sediment resuspension. The first and second chambers should have permanent pools with depths of 1.2 meters (4 feet). If possible, the third chamber should also contain a permanent pool (NVPDC, 1992).

The first and second chambers are generally connected by an opening covered by a trash rack, a PVC pipe, or other suitable material pipe (Berg, 1991). If a pipe is used, it should also be covered by a trash rack or screen. The opening or pipe between the first and second chambers should be designed to pass the design storm without surcharging the first chamber (Berg, 1991). The design storm will vary depending on geographical location and is generally defined by local regulations.

In the standard WQI, an inverted elbow is installed between the second and third chamber. The elbow should extend a minimum of 1 meter (3 feet) into the second chamber's permanent pool. Because oil will naturally separate from, and float on top of, the water, water will be forced through the submerged elbow and into the third chamber while oil will be retained in the second chamber (NVPDC, 1992). The depth of the elbow into the permanent pool should be. The size of the elbow or the number of elbows can be adjusted to accommodate the design flow and prevent discharge of accumulated oil (Berg, 1991).

Pre-manufactured oil/water separation tanks do not usually follow the separated-chamber design; instead, these units often rely on baffle units to separate the different removal process. Particulates are thus retained near the inlet to the tank, while oil/water separation takes place closer to the tank outlet.

PERFORMANCE

WQIs are primarily utilized to remove sediments from storm water runoff. Grit and sediments are partially removed by gravity settling within the first two chambers. A WQI with a detention time of 1 hour may expect to have 20 to 40 percent removal of sediments. Hydrocarbons associated with the accumulated sediments are also often removed from the runoff through this process. The WQI achieves slight, if any, removal of nutrients, metals and organic pollutants other than free petroleum products (Schueler, 1992).

The 1993 MWCOC study discussed above found that an average of less than 5 centimeters (2 inches) of sediments (mostly coarse-grained grit and organic matter) were trapped in the WQIs. Hydrocarbon and total organic carbon (TOC) concentrations of the sediments averaged 8,150 and 53,900 milligrams per kilogram, respectively. The mean hydrocarbon concentration in the WQI water column was 10 milligrams per liter. The study also indicated that sediment accumulation did not increase over time, suggesting that the sediments become re-suspended during storm events. The authors concluded that although the WQI effectively separates oil and grease from water, re-suspension of the settled matter appears to limit removal efficiencies. Actual removal only occurs when the residuals are removed from the WQI (Schueler 1992).

A 1990 report by API found that the efficiency of oil and water separation in a WQI is inversely proportional to the ratio of the discharge rate to the unit's surface area. Due to the small capacity of the WQI, the discharge rate is typically very high and the detention time is very short. For example, the MWCOC study found that the average detention time in a WQI is less than 0.5 hour. This can result in minimal pollutant settling (API, 1990). However, the addition of coalescing units in many current WQI units may increase oil/water separation efficiency. Most coalescing units are designed to achieve a specific outlet concentration of oil and grease (for example, 10-15 parts per million oil and grease).
VI. General Site Requirements

A. Grading Criteria

1. Testing

On privately funded projects, the Contractor shall pay for all required tests. On projects funded by the Town, the Town will pay for all passing tests unless stated otherwise in the contract documents.

a. Earthwork

i) Utility trenches. Testing of backfilled trenches should be at least one density and moisture content test per 200 linear feet of trench, every other lift, per eight-inch compacted fill thickness and at all laterals. Reports shall be submitted daily to the Town’s Inspector and shall be approved before paving is allowed to begin. The amount of trench excavation shall not exceed 200 (two hundred) feet from the end of the pipe laying operations, and no more than 300 (three hundred) feet of total open trench will be allowed. At the end of each work day, all trench excavation shall be backfilled to the end of the pipe laying operation. Barricades, safety fencing, and lights will be required around any open trench left overnight.

ii) In paving and building areas, at least one moisture-density (Proctor) test, Atterberg limits test and percent finer than #200 sieve test should be performed per soil type for subgrade, backfill, fill and materials.

iii) In paving and building areas, at least one density and one moisture content test per 2,500 square feet of surface area should be performed for each compacted six-inch thickness of fill.

iv) At least three density and moisture content tests should be performed in the paving and building areas on the subgrade soils, and at least three density and moisture content tests should be performed per six-inch compact thickness of fill in the building area.

v) If lime stabilization of site clays is planned, Atterberg limit lime series tests and/or pH lime series tests should be performed for each soil type after each area is cut to grade to determine the required lime percentage for stabilization.

vi) In lime/cement stabilization areas the quantity of lime/cement spread should be compared to the area stabilized to verify that the required amount was utilized.
vii) At least one gradation and depth check test per 5,000 square feet should be performed in stabilized areas per compacted lift of eight inches or less to verify that particle size has been reduced to the required level.

2. Designated Flood Plain Areas

a. As described in the Town’s Code of Ordinances, Section 90-401 and 90-402 Drainage and section 98-1081, Floodplain Prefix to District Regulation, as amended, development is prohibited, in designated flood plain areas.

b. Designated floodplain shall be determined as described in the Town’s Code of Ordinances Section 90-401 and 90-402 Drainage and section 98-1081, Floodplain Prefix to District Regulation, as amended.

c. Flood plain shall not be plotted or included within the boundaries of a residential lot.

3. Residential Construction

a. Lot Grading

(1) No lot-to-lot drainage is permitted in residential sub-divisions; lots shall be designed so that they drain to either the ROW or to a drainage easement contained within an X-Lot. The Town Council may grant an exception to this section provided that the developer agrees to install a private drainage system that is located within a drainage easement dedicated to the HOA and that the system meets the minimum private storm water system standards contained in Part B, Section IV of the Design Criteria.

(2) The minimum allowed slope across any portion of a residential building lot, in the same direction of storm water runoff, shall be 1.00%.

(3) Slopes across any portion of a residential building lot may not be steeper than 3:1, unless it is being kept in an existing and natural state.

(4) If the grade difference of adjacent building pads is in excess of two (2) feet, then a retaining wall shall be provided along the adjacent property lines. These walls shall be clearly identified in the construction plans, and shall be installed prior to the acceptance of the public improvements. Subsurface drainage shall be conveyed to a drainage easement or street.

(5) Lot grades shall be verified by a survey prior to commencement of stabilization for paving. At the end of construction, a lot grading certification letter shall accompany the required as-built grading plans. All final grades shall be within +/- 0.3 feet of those shown on the approved construction plans.
(6) No slopes steeper than 3:1 shall be located within five (5) feet of a park or equestrian trail, without the installation of a retaining wall a minimum of two (2) feet from the park trail.

**b. Cuts and Fills**

(1) The maximum allowed cut shall be five (5) feet.

(2) The maximum allowed fill shall be five (5) feet.

(3) Any vertical cut in excess of two (2) feet, from natural grade, shall require a retaining wall to be constructed prior to the acceptance of the public improvements.

(4) Any vertical fill in excess of two (2) feet, from natural grade, shall require a retaining wall to be constructed prior to the acceptance of the public improvements.

(5) For the purposes of obtaining a building permit the existing finished pad elevation established within the approved engineering drawings cannot be altered more than -.25′, +0.50′, unless prior approval is obtained from the Town Manager or his designee or their designee.

(6) All retaining walls shall be designed and sealed by a licensed structural engineer.

**4. Non-Residential Construction**

**a. Grading**

(1) Slopes across any portion of a non-residential lot shall not be steeper than 3:1, unless it is being kept in an existing and natural state.

(2) No slopes steeper than 3:1 shall be located within five (5) feet of a pedestrian or equestrian walkway or trail, without the installation of a retaining wall a minimum of two (2) feet from the park trail.

(3) If grading activities are required off of the lot being developed, a grading easement shall be acquired from the adjacent property owner, and filed in the real property records of Denton County, TX, prior to the release of grading activities. A copy of the filed easement shall be provided to the Town.

(4) Lot grades shall be verified by a survey prior to commencement of stabilization for paving. At the end of construction, a lot grading certification letter shall accompany the required as built grading plans. All final grades shall be within +/- 0.3 feet of those shown on the approved construction plans.
b. Cuts and Fills

(1) The maximum allowed cut shall be ten (10) feet.

(2) The maximum allowed fill shall be ten (10) feet.

(3) Any vertical cut in excess of two (2) feet, from natural grade, shall require a retaining wall to be constructed prior to the acceptance of the public improvements.

(4) Any vertical fill in excess of two (2) feet, from natural grade, shall require a retaining wall to be constructed prior to the acceptance of the public improvements.

(5) All retaining walls shall be designed and sealed by a licensed structural engineer.

5. Retaining / Screening Walls

a. Screening Wall

(1) Plans and cross sections of the subdivision-screening wall, if required, shall accompany the engineering plans for the project and shall be designed and sealed by a licensed structural engineer.

(2) All pier holes will need to be inspected by a Town Construction Inspector.

(3) At the end of construction, the structural engineer shall conduct a final inspection and provide a letter to the Town stating that the wall was constructed according to the approved plans.

b. Retaining Walls

(1) Retaining walls shall be provided as described in the Residential and Commercial lot grading sections.

(2) The location of the retaining walls shall be clearly shown in the engineering plans, with a top and bottom of wall elevation provided.

(3) Retaining walls shall be clad with a material that blends with the development.
(4) All retaining walls shall be designed and sealed by a licensed structural engineer,

(5) When a retaining wall is required the residential lot shall be graded so water will not cascade over the top of the wall onto the adjacent property. Subsurface drainage shall be conveyed to a drainage easement or street.

(6) All pier holes must be inspected by a Town Construction Inspector.

(7) At the end of construction, the structural engineer shall conduct a final inspection and provide a letter to the Town stating that the wall was constructed according to the approved plans.

6. Grading Construction

a. Prior to the start of construction

(1) All tree protection measures shall be in place according to the requirements in Chapter 94 “Trees”.

(2) The parking and/or storage facility shall be in place.

(3) All erosion control devices measures shall be in place according to the engineering plans.

(4) Early release of grading activities may be approved according to the Land Development Code Preliminary grading release; bonding, as amended.

(5) A pre construction conference shall be held prior to the release of any grading activities. Contact the engineering offices to schedule a pre construction conference.

b. During Construction

(1) Keep all erosion control devices in a clean and working order.

(2) Keep all tree protection devices up and in working order as according to the requirements in Chapter 94 “Trees”.
B. Subsurface Drainage Systems

1. Design Criteria

Where the presence of underground water is encountered at a flow rate determined by the Town Manager or his designee to be detrimental to the adjoining structures or property, a subsurface drainage system shall be installed, of sufficient capacity to protect adjacent structure and property. The minimum pipe diameter is to be six (6) inches with cleanouts located at a maximum distance of 300 feet.

2. French Drain Systems

A French drain system, composed of a minimum four (4) inch diameter non-perforated PVC pipe, shall be installed between the back of curb and the right-of-way line whenever adjoining lot elevations necessitate the use of retaining walls to maintain lot grades. French drain systems must be connected to the storm sewer system. When connection to the storm sewer is not feasible as determined by the Town Manager or his designee, the french drain system may discharge through the curb. The diameter of the pipe through the curb may not exceed four (4) inches. A permit for the curb core must be obtained, from the Engineering Department, prior to the start of construction.

3. Pipe Materials

The perforated pipe shall be type PS 46, or approved equal, PVC pipe conforming to ASTM 758 and ASTM D-1784 with a minimum of four (4) hole rows of ¼ inch diameter perforations on four (4) inch maximum centers. The perforated pipe and conducting pipe shall be white in color.

C. Street Lights

1. Layout

The Town of Flower Mound requires street light layouts to be submitted prior to the approval of the engineering plans. Lights will be required at intersections, cul-de-sacs, and significant curves in the street and mid block on blocks longer than 600 feet max. Additional lighting may be required due to street length.

2. Installation

Lighting to be installed in any existing urban subdivision will be determined based on lack of lighting at intersections, cul-de-sacs, and significant curves in the street, electric accessibility and budget availability. The person making the request will be
responsible for obtaining adjacent homeowner agreement for the installation of the light and obtaining any easements at their cost.

END OF SECTION
ACCESS MANAGEMENT

POLICY AND CRITERIA

OCTOBER 2005
Introduction and Background
Access management is the practice of controlling the character of the access allowed to a roadway by applying criteria for the location, spacing, design and operation of driveways, median openings and intersections. In general, access management has the goal of balancing the access intensity with the desired mobility function of a particular roadway. For example, access management criteria would typically allow the least intense access to arterial or high-speed facilities to minimize the interruption to traffic flow that results from frequent access points. Local streets, on the other hand, would typically have the highest allowable access intensity because the mobility function is less of a priority. Access management criteria are applied during the development review process or through an application procedure for modifications to existing access for developed property.

Benefits of Access Management
Access management is one key to preserving the capacity of a particular facility by minimizing turbulence and speed reduction caused by driveway access and egress and proper use of auxiliary lanes at intersections. On January 1, 2004, the Texas Department of Transportation (TxDOT) implemented an access management policy that documented many of the benefits of access management. For instance, the typical reduction in free-flow speed in one direction is reduced by 0.15 mph per access point and .005 mph per right-turning movement per hour per mile of road. In other terms, allowing 10 access points along a mile-long stretch of road (average spacing of 528’) would reduce the free-flow speed by 2.5 mph, and allowing 20 access points would result in a reduction in free-flow speed of 5 mph.

Access management also reduces the potential for accidents by minimizing speed differentials between through vehicles and turning vehicles. Research has shown that accident rates increase consistently with an increase in access density, while accident rates decrease with the construction of raised medians and controlled cross-access.

Finally, access management has been shown to have an overall positive economic effect on communities and transportation corridors. Proper access management preserves the flow of traffic within and through corridors, thus supporting the transportation needs of retail and commercial development while providing improved mobility for commuters. Numerous studies have shown that access management has little or no adverse impact on individual business activity, and that corridors with completed access management projects had better retail sales when compared to surrounding communities.

Purpose of the Access Management Policy
This access management policy is needed in Flower Mound for three primary reasons. First, an access management policy will establish a consistent means of reviewing access requests to achieve the benefits described above. Second, this access management policy will support the Town of Flower Mound Thoroughfare Plan by allowing different levels of access intensity for roadways based on the intended function. And finally, TxDOT will allow municipalities to review and approve access
permits if the municipality has an approved access management policy. It is important to note that the Town’s access management criteria may not supercede items included in TxDOT’s Roadway Design Manual (e.g. median opening spacing and auxiliary lane design criteria).

**Access Management Criteria**
The Town of Flower Mound access management criteria are categorized by the functional classification of a facility as described in the Town’s Thoroughfare Plan. For the purposes of the criteria, five functional classes are used and are grouped by assumed operating speed: major and greenway arterials, minor arterials and collectors; and local roadways. The basis for the criteria will be the assumed operating speed for a particular functional classification. Table 1 provides assumed operating speeds by functional classification and corresponding access spacing distances. The access management criteria are to be applied to all existing or new facilities in the Town during the development or reconstruction process.

**Engineering Basis**
The access management criteria are based upon accepted engineering principles and draw upon work included in the TxDOT Access Management Manual and Roadway Design Manual, the AASHTO Policy on the Geometric Design of Highways and Streets, and the Transportation Research Board’s Access Management Manual. The criteria are based largely on satisfying safe stopping requirements included in these documents. For instance, the minimum driveway spacing should be such that a driver is not required to encounter more than one driveway within the stopping sight distance along a given facility. Other considerations include traffic signal operation and the ability to construct deceleration (left-turn or right-turn) lanes for driveways and intersections.

**Access Spacing Criteria**
As described in previous paragraphs, arterials have the primary function of providing mobility with minimal allowable access. As such, the access criteria are most restrictive along arterials. More access is allowed along collectors than arterials, and the most intense access is allowable along local streets. Note that these criteria do not apply to local residential streets for purposes of residential driveway spacing. Table 1 lists the access spacing criteria for each functional classification. Distances listed in the table are to be measured from street to driveway or driveway to driveway, with the measurement taken from the near edge of both facilities as shown in Figure 1.

<table>
<thead>
<tr>
<th>Functional Classification</th>
<th>Assumed Operating Speed (mph)</th>
<th>Access Spacing Distance</th>
</tr>
</thead>
<tbody>
<tr>
<td>Major &amp; Greenway Arterials</td>
<td>45</td>
<td>360’</td>
</tr>
<tr>
<td>Minor Arterials &amp; Collectors</td>
<td>35</td>
<td>250’</td>
</tr>
<tr>
<td>Locals &amp; Urban Minor Arterial with On-Street Parking</td>
<td>30</td>
<td>200’</td>
</tr>
</tbody>
</table>
In addition to meeting the driveway spacing criteria, all driveways must be located outside the limits of deceleration lanes serving the next adjacent driveway or intersection and designed such that adequate stopping sight distance is provided. Exceptions to the above criteria will be reviewed where properties would be denied access under this criteria and where access through adjacent properties via developers agreements or access easements is not feasible. Requests for deviation to these criteria must be supported by a transportation study and approved by the Town Engineer.

**Shared Access**

Mutual use access across property boundaries will be required for all new developments where it is deemed feasible. This practice ensures that all properties are allowed full access to the roadway system, thus minimizing undesirable traffic patterns and turning movements at nearby intersections.

**Median Openings**

Median openings can be particularly problematic due to the impact on traffic flow from left-turning and crossing maneuvers. For this reason, it is critical that median opening spacing be established to minimize the impacts on nearby intersections or driveways. Median openings must be spaced to accommodate fully-developed left-turn bays in both directions (refer to section on Auxiliary Lane Design Criteria). Median opening spacing must also be sufficient to avoid interfering with the advance detectors of a nearby signalized intersection. Requests for deviation to the median opening spacing criteria must be supported by a transportation study and approved by the Town Engineer.

**Modified (Hooded) Median Openings**

In some situations it is beneficial to prevent exiting traffic from making a left-hand or crossing maneuver. Typical applications of hooded median openings would be where cross-access driveways of major retail centers align, adjacent to any intersection that includes an arterial or collector, or in a retrofit situation where accident history indicates a safety problem at a median opening. The proposed modified median opening must be located such that fully-developed left-turn bays can be established in both directions of traffic flow. Requests for deviation to the median opening spacing criteria must be supported by a transportation study and approved by the Town Engineer.
Landscaping Requirements
Adequate stopping sight distance must be maintained at all intersections. All plants located within the required site lines of all intersections will be maintained such that a clear view is maintained. This allows the planting of low-growing shrubs and trees with a clear canopy high enough to permit an unobstructed view of approaching automobiles. Plantings near intersections will only be allowed after staff review to ensure that adequate sight distance is being maintained.

Auxiliary Lanes Required
Urban Minor Arterials shall be treated differently from other Minor Arterials. Due to the Mixed Use nature and pedestrian friendly environment, turn lanes for Urban Minor Arterials will be determined on a case by case basis where they are deemed appropriate.

In order to make a right turn into a development, vehicles must decelerate to a safe speed before proceeding into the development. The reduction in speed required to make this movement creates turbulence in the flow of through traffic and potentially creates a safety hazard resulting from the speed differential. Right turn deceleration lanes will be required at all driveways served by a median opening and all driveways with a projected peak-hour right-turn entering volume of 50 vehicles or more. Finally, right turn deceleration lanes will be required at any intersection of collectors and arterials as shown on the Town of Flower Mound Thoroughfare Plan.

Vehicles turning left into a development at a median opening must decelerate, possibly wait in a stacked queue, wait for an acceptable gap in opposing through traffic, and then turn into the driveway. This type of access offers the greatest potential for traffic incidents due to the speed differential as vehicles decelerate, and the side-collision potential as vehicles turn across opposing lanes of traffic. To minimize these risks, left-turn lanes will be constructed to serve all median openings that provide access to a driveway. Left-turn bays will also be constructed at any intersection of arterials and collectors as shown in the Town of Flower Mound Thoroughfare Plan.

Auxiliary Lane Design Criteria
The design of auxiliary lanes will be based on the assumption that vehicles will have to decelerate to a stop prior to proceeding into the driveway and that some deceleration takes place within the through lane prior to entering the deceleration lane. Storage is provided for a minimum of two vehicles (50 feet). Finally, the taper is considered part of the deceleration length and is minimized to provide a better target for the driver and to provide a longer section of full-width pavement for the auxiliary lane. The design criteria for left-turn and right-turn lanes are categorized by the functional class of the thoroughfare and are provided in Table 2 below. The lengths shown in Table 2 will be increased if a transportation study indicates the need for additional vehicle storage. Each element of the turn-lane is depicted in Figure 2.
Table 2. Turn Lane Design Criteria

<table>
<thead>
<tr>
<th>Functional Classification</th>
<th>Minimum Storage</th>
<th>Deceleration Length</th>
<th>Taper Length</th>
<th>Turn-Lane Length</th>
</tr>
</thead>
<tbody>
<tr>
<td>Major &amp; Greenway Arterials</td>
<td>50’</td>
<td>215’</td>
<td>100’</td>
<td>265’</td>
</tr>
<tr>
<td>Minor Arterials &amp; Collectors</td>
<td>50’</td>
<td>110’</td>
<td>100’</td>
<td>160’</td>
</tr>
<tr>
<td>Local Roads &amp; Urban Minor Arterial with On-Street Parking</td>
<td>50’</td>
<td>75’</td>
<td>50’</td>
<td>125’</td>
</tr>
</tbody>
</table>

Assumed deceleration rate = 11.2 ft/s²

Figure 2. Turn Lane Design Criteria

Where a deceleration lane is constructed to serve a signalized intersection, the bay should be long enough to remove turning traffic from the through traffic stream during critical periods and provide sufficient storage for efficient operation of the signal at all times. The required storage length can be determined through the use of a traffic model such as Synchro or HCM, or by using the equation provided below. At signalized intersections:

Turn Lane Length (signalized intersection) = Deceleration Length + Storage Length, where:
- Deceleration Length : See Table 2,
- Taper Length = 100’ for single-turn lanes; 150’ for dual-turn lanes
- Storage Length = (Left-Turn Volume/# of cycles) (2)(25)
Access to State Facilities
The authority for granting access to state facilities rests with the Texas Department of Transportation (TxDOT). TxDOT may grant permitting authority to municipalities or use a local government’s access management plan for approving access locations. At a minimum, the Town of Flower Mound will apply to have its access management spacing criteria applied to state facilities within the Town’s jurisdiction. Contact the Town Engineering Department for the current access permitting procedure for state highways within the Town’s jurisdiction.

Deviation and Dispute Resolution Process
Deviations to these criteria may be approved by the Town Engineer as described in the previous sections. In addition, the Town Engineer is hereby authorized to grant an exception to the access spacing criteria when one property is being developed or redeveloped and the adjacent property is not then being developed or redeveloped and the property not being developed contains an access point that is too close to the access point being proposed on the property being developed or redeveloped provided that the ultimate access points for both properties when they develop or redevelop have already been determined by the Town. The Town Engineer is also hereby authorized to grant an exception to the access spacing criteria in situations where a large physical obstruction, which cannot be reasonably relocated, exists on property being developed or redeveloped and such physical obstruction prevents compliance with the access spacing requirements and the requested deviation therefrom is twenty percent (20%) or less of the standard spacing.

Disputes will be deferred to and resolved by the Town Council. All access management disputes must be accompanied by a Traffic Impact Analysis paid for by the developer per Town of Flower Mound development procedures.

Administration of Policy
This policy will be administered by the Town Engineer or his designee.

Engineering Study and TIAs
An engineering study or Traffic Impact Analysis (TIA) may be required by the Town Engineer or initiated per the Town’s Land Development Code. Should an engineering study be required, it may include the following elements: trip generation, trip distribution, and traffic assignment at the proposed access point(s). Additionally, the engineering study may require that existing traffic volume data be collected, including turning movement volumes at intersections. The trip generation will be conducted using the latest edition of the Institute of Transportation Engineers Trip Generation manual unless there is acceptable data that supports the use of another trip generation source. Trip distribution will be performed with input from Town staff and may require the use of the Town’s travel demand model. The traffic assignment will be conducted to determine the forecasted turning movements attributable to the proposed development. The existing traffic counts will be grown using an annual growth rate as agreed to by the Town to the build-out year of the proposed development. Pass-by trips will be addressed using accepted practices as recommended in by the Institute of
Transportation Engineers. The resulting traffic volumes will be used as background traffic volumes, and the assigned forecasted turning movements will be added to the background traffic volumes resulting in the total traffic volumes.

If a TIA is required by the Town, it may include the above mentioned elements as well as the same type of data for intersections adjacent to the proposed site (specific study limits to be defined by the Town). Additionally, the TIA may require operational analyses (including LOS and capacity analyses) for the study intersections as determined during the initial meeting between the applicant and the Town. Furthermore, the applicant’s TIA should include recommendations for mitigation measures should the impact of the proposed access point(s) on the state highway system result in unacceptable levels of service.

**Need for Traffic Signals**
If there is reasonable evidence that a development will generate sufficient traffic to warrant a traffic signal per the Texas Manual on Uniform Traffic Control Devices, the internal circulation of the development must be designed such that the signal will be located at the intersection of two streets (public or private). Regardless of funding source, traffic signals will not be approved at the intersection of a driveway with a public street as driveways do not allow for the required stacking, visibility, or maneuvering areas required at signalized intersections.

**Conclusion**
This policy and associated criteria are based upon recognized engineering practice. They will be applied during the commercial development process and during the capital improvement process. They are to be administered by the Town Engineer, with disputes resolved by the Town Council.