City of Raleigh
Stormwater Management Design Manual

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Introduction

Statement of Intent

The intent of this manual is to serve as a reference for the City staff and practicing professionals in designing storm drainage facilities within the City of Raleigh and its extraterritorial jurisdiction. It is primarily a compilation of accepted design procedures, practices, and values along with summaries of the policies of the City. Design criteria listed herein are the general policy of the City of Raleigh and may not be applicable in every situation. Where the designer determines that conformance with this manual would create an unreasonable hardship or where an alternative design may be more appropriate, alternative designs may be accepted upon written authorization from the City Engineer or his designee. In order to insure good engineering design, the City staff may occasionally require more stringent standards than those presented here. This manual may also be subject to periodic change by the City staff. When changes are required, revisions will be made available to registered copy holders.

Disclaimer

To the best of their ability, the authors have insured that material presented in this manual is accurate and reliable. The design of engineered facilities however, requires considerable judgment on the part of designer. It is the responsibility of the designer to insure that techniques utilized are appropriate for a given situation. The City of Raleigh therefore accepts no responsibility for any loss, damage, or injury as a result of the use of this manual.

Chapter 1 General Information

1.1 Stormwater Management

Stormwater management in the City of Raleigh addresses issues related to flood control and water quality management. The City has adopted a variety of policies, regulations, and design standards for the management of the quantity and quality of stormwater runoff. The policies are designed to protect the health and welfare of the citizens of Raleigh, protect the environment, and protect those that live downstream of the City that must deal with the quantity and quality of stormwater leaving the City. The following sections of this chapter of the design manual describe the regulations applicable to stormwater management and drainage design in the City of Raleigh and provide information relevant to complying with stormwater management requirements in the City.

1.2 Stormwater Management Regulations

New construction within the Raleigh jurisdictional area is subject to numerous ordinances and policies of the City. For the purposes of this manual a brief summary of city storm drainage policy is included with Code references where applicable. These policies and standards are those to be used in private development as well as projects undertaken by the City.

The majority of new development is subject to Chapter 3 of the Raleigh City Code, "Subdivisions and Site Plans". The Code requires that the City review and approve site plans and subdivisions in two steps. First a preliminary site plan is required for review and approval (Sections 10-2132.2 and 10-3012, 3013). This plan must show all pipe sizes and drainage locations, easements, and the location and description of all flood restriction lines and elevations. At this time the staff will make recommendations regarding special storm drainage concerns.

The second step is the review and approval of the engineering plan. Along with other design features, this plan should include construction drawings showing the complete storm drainage system, calculations and supporting information for compliance with the Neuse River Nutrient Sensitive Waters Management Strategy, and all supporting calculations for review.

1.2.1 General Stormwater Design Requirements

The following information shall be provided with all submittals to allow efficient and thorough review of drainage plan submittals:

- A vicinity sketch or key map at a scale of not more than one thousand (1000) feet to the inch showing the position of the development with its relation to surrounding streets and properties, and oriented in the same direction as the remainder of the development plan.

- Site boundaries and total area of site

- Zoning, lot layout (including drainage easements), owner's names for adjoining properties, water and sewer easements, and street layout

- Floodway, floodplain, flood hazard, flood storage area, known wetland, water course buffer area boundaries, and greenway locations

- Regulatory Flood Protection Elevations for each lot for multiple lot developments or at the upstream and downstream property lines for parcels with over 400 feet of adjacent stream bank

- Actual land use within 200' of property boundary including connecting streets

- For ease of review, copies of topographic maps, clearly showing the limits of the site. These maps shall extend a minimum of 1000 feet beyond the site boundaries or to the ridge-lines of contributing drainage areas if less than 1000 feet.

- Existing and proposed topography at a minimum of 2' contour intervals where city topographic maps are available or at a minimum of 5' intervals otherwise.

- Existing and proposed drainage easements and water and sewer easements.

- Estimated flows for required design storms entering and leaving the site.

- All administrative preliminary site plan and administrative preliminary subdivision plan approval submittals must be accompanied by a narrative describing how stormwater entering, traveling within, and leaving the site will be controlled and to what extent the development will impact existing conditions on-site and off-site.

- All Council approved preliminary site plan and Planning Commission preliminary subdivision plan submittals must be accompanied by a note, prepared by a registered professional engineer, registered landscape architect, or registered land surveyor who are qualified in hydrology and hydraulics, stating that the plans are of a preliminary nature and that final plans will conform with the standards in the City of Raleigh Drainage Design Manual.

- All Administrative final site plan and construction drawing submittals must be accompanied by a certification, prepared by a registered professional engineer, registered landscape architect, or registered land surveyor who are qualified in hydrology and hydraulics, stating that the plans conform with all standards in the City of Raleigh Drainage Design Manual, and all other applicable City Code sections.

1.2.2 Street and Local Drainage

The following design standards should be followed unless the designer requests and receives approval for alternative designs from the City Engineer or his designee:

- All street and local drainage systems should be designed to pass the 10-year design storm unless more stringent requirements apply.
- Drainage facilities, such as pipes and channels, must be provided for 10-year storm discharges of 5 cfs or more if not already existing.
- Estimated headwater for all roads crossing watercourses for the peak flows from 10 and 100 year storms, including weir calculations, shall be provided for those situations where overtopping is allowed.
- Catch basins in streets may be designed for gutter spread using the 2-year storm provided a 5 minute time of concentration is used and the remainder of the system is designed for the 10-year storm assuming each inlet captures 100 percent of the flow (to provide additional capacity for future additions to the system and off-site drainage). Inlet capacity at sags, where relief by curb overflow is not provided, shall allow

  for debris blockage by providing twice the required computed opening for the 2-year storm.
- Storm drainage pipes in the City right-of-way shall be reinforced concrete with a minimum diameter of 15 inches. High Density Polyethelyne (HDPE) may be used in minor residential and residential streets as defined by the City of Raleigh Streets, Sidewalks, and Driveway Access Handbook provided it is installed according to the requirements of section 1.2.2.1 of this manual.
- All reinforced concrete pipe used within City right-of-way shall be Class III or higher.
- Cover for pipes within the R.O.W. shall be provided based on the following table:

MINIMUM PIPE CLEARANCE FROM INVERT TO SUBGRADE	
Pipe Size (in.)	Clearance Distance (ft)
15	2.4
18	2.7
24	3.3
30	3.8
36	4.4
42	4.9
48	5.4
54	6.0

60	6.5
66	7.0
72	7.6

- Curb inlets in the roadway shall be placed in such a way that the spread of water in the 2-year storm does not exceed one half of a lane width on 2 or 3 lane streets and one lane width on wider streets. When the typical section includes a full shoulder or parking lane, no encroachment into the travel lane will be allowed.

- Inlets shall be provided at sags, up-grade of intersections, up-grade of super-elevation crossovers, and where driveways would discharge more than 3 cfs into a street for the 10-year storm. Inlets should be provided to capture runoff and carry flow into the drainage system before it reaches the right of way.

- A minimum gutter gradient of 0.5 percent shall be utilized. When lesser slopes are encountered, the gutter shall be warped to provide the minimum slope.

- City of Raleigh standard inlets shall be used for all streets to be maintained by the City unless and alternative is specifically requested and approved by the City Engineer. Where streets are to be maintained by the State, other inlets (acceptable to the State) may be used.

- Roads which cross or parallel creeks serving more than 25 acres of drainage area must have at least 2 feet of freeboard in the 10-year storm and 6 inches in the 100 year storm. Upon written approval from the City Engineer, the street may be designed to flood in the 100 year event provided the flooding depth does not exceed 2 feet and substantial erosion protection is provided on the downstream side of the roadway embankment. In "regulated discharge floodplain areas" (areas subject to extended duration flooding), streets shall be constructed at or above the 100-year flood elevation

- For cross drainage serving 10 acres or more, the maximum depth of the water impounded during the one hundred year flood shall not exceed fifteen (15) feet as measured from the upstream invert of the culvert beneath the roadway section to the water surface elevation unless appropriate engineering calculations are submitted verifying the stability of the embankment against slope failure and seepage effects. Any detention facility designed and constructed in compliance with the North Carolina Dam Safety Regulations shall be acceptable to the City.

- No public or private roads are to be constructed on dams without the approval of the City Engineer.

- New street crossings in reservoir watershed protection areas or in watercourse buffers in metro-park protection overlay districts shall be as close to a perpendicular angle as possible (see City Code Section 10-2056(c)(2)).

- When development of an area changes the flow regime from sheet flow to concentrated flow, the drainage system shall be designed to minimize impacts of the concentrated flow on adjoining properties by tying into existing systems using multiple outlets, through agreements with adjacent owners, or other appropriate means.
- All reinforced concrete pipe should be Class III or higher. Where depth of fill exceeds 20 feet or high loads may be expected, appropriate calculations shall be made and the appropriate combination of pipe material, class of pipe, and bedding shall be specified on the construction plans.
- Minimum slopes for pipes and open ditches is 0.5%
- Maximum slopes for concrete pipes is 12.0%***.*** Greater slopes may be approved by the City Engineer upon submittal of appropriate detailed structural designs and other supporting documentation.
- Where storm drainage lines cross or parallel other utility lines, appropriate clearances shall be provided according to the Public Utilities Handbook.
- Minimum cover for pipes outside of the R.O.W. is 0.5 feet.
- No concentrated flow shall be discharged across walkways. Provisions are to be made through piping or other means to carry the flow under the walkway.
- All structures shall allow for access to the storm drainage system with a grate, manhole ring and cover, or a lid capable of being removed. No "blind boxes" are permitted.
- Following are the maximum pipe lengths permitted without some type of structure providing access:

Diameter	Maximum Pipe Length
48” or greater	400’
Less than 48”	300’

- A structure of some type is required at all changes in grade or direction or at any pipe junction. When possible, these structures shall be located outside of the R.O.W. unless they incorporate a yard or curb inlet. Details shall be provided on the plans for all such structures.
- Minimum drops in inlets, junction boxes and other structures are as follows:

Change in Alignment	
0 - 45 degrees	0.1 ft
45 – 90 degrees	0.2 ft
> 90 degrees (reverse flow conditions)	Only with detailed study and drop equal to or greater than the diameter of the pipe out

Change in Pipe Size	
Increase in pipe size	0.2 ft
Decrease in pipe size	Only with a detailed study and special provisions for maintenance

- For open channels, gradual changes in alignment, not to exceed a minimum radius of 4 times the top width of the channel, is recommended. Where no other options are available, sharper changes in alignment may be allowed under the following conditions:

20 - 45 degrees	bank stabilization must be provided according to tractive force analysis
>45 degrees	Same as for above but in addition, freeboard equal to or greater than 1/2 of the Q10 depth of flow must be provided utilizing berms or other appropriate means to increase depth of the channel.

- Side slopes for vegetated open channels in residential areas should be no greater than 3 to 1 for stability, safety, and ease of maintenance. Where the channel width must be limited, side slopes may be increased if suitable vegetative or structural stabilization techniques (see following table) and safety measures are utilized. Aesthetics and ease of maintenance should also be considered in the design.

MAXIMUM SLOPES FOR COMMON STABILIZATION TECHNIQUES	
Vegetative*	2:1
Stone	1.5:1
Grid Pavers	1.5:1
Paving**	1:1
Gabions	Vertical
Retaining Walls	Vertical

**Note: Special consideration must be given to the use of vegetative linings in channels. In some cases, structural stabilization is required along the lower portions of the channel bank where continuous or frequent water contact weakens the soil structure and may impede the growth of vegetation (recommend protection to a point 2' above the bottom of the channel or the high water mark for the 2-year storm, which ever is greater). The reader is directed to City of Raleigh "Guidelines for Land Disturbing Activities" and State of North Carolina "Erosion and Sediment Control Planning and Design Manual" for the selection of appropriate vegetation based on soil types and flow velocities.*

***Note: Asphalt channel linings are not allowed in the City R.O.W.*

- No flow greater than 3 cfs, for the 10-year storm, may run down a driveway into the street without the placement of a catch basin to intercept the flow.
- No grate type inlets are allowed in city streets. Projects funded by North Carolina State Department of Transportation may use grate type catch basins.
- New and existing on-site storm drainage facilities, piped or open, serving 25 acres or less of drainage area, and adversely impacted by the proposed development shall be designed and constructed to pass the 10-year storm. Peak flows for the 100-year storm should be checked for possible structure flooding.
- New and existing on-site storm drainage facilities, piped or open, serving more than 25 acres of drainage area, which are impacted by the proposed development, shall be designed to pass the 100-year storm unless the following criteria are met:

- New open channels serving more than 25 acres may be constructed to a 10-year design standard provided the developer demonstrates that the 100-year discharge will have no adverse impacts on adjacent properties and the limits of the 100-year floodplain are determined and recorded.

- Existing natural channels serving greater than 25 acres do not have to be improved to carry the 10-year design flow but the limits of the 100-year floodplain must be established and recorded.

- In the interest of preserving existing vegetation (which helps to stabilize stream banks and provides shade thereby reducing temperature extremes) and in order to preserve the aesthetics of natural channels, not all streams have to be altered to protect them from erosion. However, existing channels which are an integral part of the development and which may endanger new or existing structures or other improvements (such as parking lots and tennis courts) as the result of future stream bank erosion, should be evaluated for the need for additional erosion protection. In addition, those existing channels which will be subject to peak flow increases of 100% or more as the result of complete build-out of the contributing watershed and those existing channels with sharp bends, should also be evaluated for the need for additional erosion protection.

1.2.2.1 Requirements for Acceptance of HDPE Pipe

- HDPE to be allowed only on minor residential/resident public streets as defined by the City of Raleigh's Streets, Sidewalks, and Driveway Access Handbook.
- Conform to AASHTO M294 – corrugated exterior/smooth interior pipe (Type S).
- Certification by PPI (Plastic Pipe Institute)
- Bell and spigot joints with O-ring gasket (on spigot end) required on all pip installed within right-of-way. Bells shall cover two full corrugations on each section of pipe. Gasket to conform to ASTM F477.
- Installation trench width shall be a minimum of the outside diameter of the pipe + 4 feet.
- HDPE to be backfilled with 4" of #57 stone bedding under the pipe and to the springline of pipe. Remaining backfill shall be installed in accordance to current city standards.
- Third party certification by a licensed professional engineer. Certification will be based upon periodic observations of installation procedures.
- Minimum diameter of 15" – Maximum diameter of 48".
- Cover for HDPE shall be a minimum of 18 inches from the outside wall of pipe to finished grade (note that a minimum of 24" of cover should be provided for HDPE exposed to heavy traffic during construction).
- Maximum slope is 12%. Greater slopes may be approved by the City Engineer upon submittal of appropriate detailed structural designs and other supporting documentation.
- No HDPE end treatments allowed. Reinforced concrete pipe/headwall to be used for all end treatments.

- Transition of HDPE to RCP to require Dissimilar Materials Adapter incorporating a geotextile coupler with mastic coating and stainless steel straps, and a full concrete encasement around connection.
- All HDPE to be mirrored by City of Raleigh Engineering Inspectors.
- Bury depths greater than 20 ft. to have prior approval by the City Engineer.
- 24-hour notice required prior to installation.

1.2.3 Drainage Easements

Drainage easements are required for any development that involves more than one lot. This includes commercial developments with out-parcels, phased development, and other developments with surrounding land under the same ownership as the tract being developed.

Drainage easements shall be provided:

- for all culverts and pipes;
- for all new or existing open channels or watercourses with peak flows of 15 cubic feet per second or more for the 10 year storm;
- below all new or existing pipes and other points of concentrated flow;
- for primary and emergency dam spillways;
- or at other locations deemed appropriate by the City Engineer or his staff.

Access easements, dedicated to the City of Raleigh, shall be provided for access and repair of velocity dissipaters, headwalls, and other structural portions of the drainage system located outside of the right of way which are immediately adjacent to and directly associated with the City owned portion of the drainage system. These easements are requested to allow City staff access to repair and maintain those drainage facilities located immediately adjacent to the right of way which would endanger the roadway should they fail. Adequate easements shall be provided to allow access of construction equipment, taking into consideration the limitations that may be imposed by embankment slopes or other obstacles.

Drainage easements, containing only storm drainage facilities, should be centered over the culvert or watercourse with minimum widths based on the following:

Easement widths for culverts:

Easement Width = the greater of 20 feet or 10' + the diameter or total outside width for multiple culverts + 2 x invert depth (rounded to the nearest 5 feet).

Drainage easements associated with culverts may be offset as long as a minimum of 10 feet is provided on both sides. Where other utilities such as water and sewer, are involved, additional width shall be provided according to guidelines in the Public Utilities Handbook, but in no case shall the easement widths be less than those listed above.

Easement widths for open channels:

Drainage Area, ac	**Easement Width*, ft**
< 10 ac	10' on each side**
10 – <25 ac	20' on each side
25 - <50 ac	30' on each side
50 - <100 ac	40' on each side
> 100 ac	The greater of the floodway width or 50'

*Note: Widths shall be determined from the top of the bank or centerline if no banks are discernible.

**Easements of lesser widths (i.e. 10’ total width) may be specified, at the designer/developer’s discretion, for those watercourses carrying 10-year peak flows of less than 15 cubic feet per second.

Where the designer desires to reduce the easement widths from those listed above, the calculated width of flow for the 100-year peak discharge may be substituted, but in no case shall the width be less than 20 feet. Manning’s equation and other acceptable methods may be utilized in place of HEC-II models to determine flow areas for those channels draining less than 100 acres. In order to insure the accuracy of these simplified methods, representative cross sections shall be provided at a minimum at upstream and downstream site boundaries, for each additional 300 feet of channel length, and at those locations where the channel geometry changes significantly. The worst-case easement width, rounded to the nearest 10 feet, shall be utilized, or the width adjusted at appropriate locations, to take into account fluctuations in the calculated flood width.

All drainage easements should be recorded based on field surveys, following construction, to insure that the drainage structure or watercourse is centered within the easement (unless specifically offset). Where this is not possible, a note shall be added to recorded plats establishing that easements are to be centered over the pipe or channel.

All drainage easements shall be designed to tie into existing easements, existing watercourses, or to other appropriate locations when possible.

1.2.4 Water Supply Watershed Protection

The Water Supply Watershed Protection program (WSWP) was developed and implemented by the State of North Carolina in 1989[1]. Under this program, the state requires cities and counties statewide to implement watershed protection programs for areas where drinking water is supplied by surface impoundments or by direct withdrawal from streams. This program defines geographic boundaries relative to water supply intakes and includes requirements for varying levels of protection based on the level of development that existed within the defined protection area at the time that the rule became effective.

Each city and county impacted by this regulation was required to adopt ordinances constraining development with buffer and BMP requirements in the protection areas. The indicator pollutant for this program is total suspended solids (TSS). Division of Water Quality developed simple wet pond sizing criteria that produces the required 85% reduction in TSS from runoff when ponds are designed and built to these criteria and when required periodic maintenance is performed.

Development in the Swift Creek and Falls Lake watersheds are subject to additional restrictions as presented in Section 10-3059 of the City of Raleigh Code. Those requirements are as follows:

a. Lots in the primary watershed protection area are allowed a maximum impervious area of 12% (6% for Falls Lake Basin) or 3500 square feet, which ever is larger.

b. Lots in the secondary watershed protection area not served by city water and sewer are allowed a maximum of 12% impervious area or 3500 square feet, which ever is larger.

c. Lots in the secondary watershed protection area served by city water and sewer service may have impervious areas up to 30% if the first one half inch of rainfall on the additional impervious area is retained (or detained for over 12 hours).

Lots in the Metro Focus Area may have up to 70% impervious area if the first one-inch of rainfall on the additional impervious area is collected in a wet detention facility. The size of the basin shall be determined based on the following:

Percent Impervious Area of Lot	**Percent of Permanent Wet Pond Surface to Total Drainage Area**				
	Basin Depth (ft)				
	3	3.5	4	5	6
6% - 30%	1.0	0.9	0.8	0.7	0.5
30%	2.4	2.1	1.8	1.5	1.3
50%	4.0	3.5	3.0	2.5	2.1
70%	5.7	4.8	4.3	3.5	2.9

[1] NC 1989 Session Law Chapter 426

1.2.5 Preliminary Stormwater Requirements for Rezoning Requests (CR-7107)

The City of Raleigh adopted an interim measure designed to control the impact of rezoning on stormwater runoff in 1988. This interim policy, CR-7107, requires that the drainage system for rezoned properties be designed such that post development peak discharge is released at a rate equal to or less than the rate expected if the site were zoned at Residential - 4 (1/4 acre lots) or the rate expected for the existing zoning, whichever is greater. This guideline is to be met for the two and ten year frequency storms. When the rezoning is located where reduced discharges would provide no benefit, this guideline is not implemented based on the discretion of City staff. The policy also recognizes that for small sites (1 or 2 acres) it may not be practical to implement such controls. In cases where the practicality is at issue the developers and staff are to look for practical alternatives.

1.2.6 Neuse River Nutrient Management Strategy

The Neuse River basin has shown signs of stress due to nutrient loads and has been designated as a Nutrient Sensitive Water (NSW) for more than a decade. In February 1996 the Environmental Management Commission approved a draft of a comprehensive strategy for the management of nutrients in the Neuse River basin. The goal of the strategy is to achieve a 30 percent nitrogen reduction from each controllable and quantifiable source of nitrogen in the basin. These sources are: wastewater treatment, urban stormwater, agriculture, and nutrient application. The NSW strategy includes a rule to protect riparian buffers in order to maintain their existing nitrogen removal capabilities.

The strategy also requires that there be no net increase in peak flow leaving the developed site from the predevelopment conditions for the 1-year, 24-hour storm. However, since Raleigh has historically utilized the 2-year design storm for peak runoff control and since rainfall statistics are not available for the 1-year storm, new development within Raleigh's jurisdiction is required to attenuate the 2- year design storm, not the 1-year, 24-hour storm.

1.2.6.1 Applicability – General

The Neuse River Nutrient Management Strategy is applicable to all new development in the City's jurisdiction. All existing development as of the effective date of the rule is grandfathered, subject to further subdivision, development, or redevelopment of those properties.

1.2.6.2 New Development Exemptions

The following development activities are exempt from the Neuse Rules;

(1) any single family detached dwelling, any single family attached dwelling not exceeding two dwelling units, and any duplex dwelling, including their accessory uses, placed on any vacant lot which was recorded either at the time of application of this regulation* or was a subdivision approved for recordation prior to the application of this regulation and which has not sunsetted;

(2) any plot plan and site plan, including their accessory uses, situated on any vacant lot of one-half acre or less in size which was either recorded at the time of application of this regulation* or was a subdivision approved for recordation prior to the application of this regulation and which has not sunsetted;

(3) any single family detached dwelling, any single family <u>attached</u> dwelling not exceeding two dwelling units, and any duplex dwelling, including their accessory uses, placed within any subdivision of one acre or less in aggregate size approved after application of this regulation;

(4) any plot plan and site plan, including their accessory uses, placed within any subdivision of one-half acre or less in aggregate size approved after the application of this regulation which cumulatively contains less than twelve thousand (12,000) square feet of impervious surface, including impervious surfaces of related on-site or off-site facilities;

(5) any land-disturbing activity that does not require a land-disturbing permit under §10-5008 provided that, upon application of any impervious surfaces this exemption *shall* not apply;

(6) substitution of impervious surfaces when all the standards of §10-2146.2(a)(5) are met. Substitution must take place within one year or prior to expiration of a valid building permit or sunsetting of an approved subdivision or site plan.

1.2.6.3 Vested Rights

Property owners that can demonstrate that they have vested rights as of the effective date of the Local Stormwater Program for Nitrogen Control will not be subject to the requirements for new development. Vested rights may be based on at least one of the following criteria:

1. approved subdivisions and site plans which have not sunsetted.

2. Projects which have an outstanding unexpired valid building permit in compliance with either G.S. 160A-422 or G.S. 153A-357 or have an outstanding unexpired valid soil erosion permit in compliance with G.S.160A-458; provided that, upon application of any impervious surfaces, the exemption based on a valid soil erosion permit shall not apply.

3. Projects which have obtained a pertinent state permit, such as landfills and land application of residuals.

4. Projects which have continuing vested rights in compliance with G.S. 160A-385.1 or G.S. 153A-344.1.

1.2.6.4 Exclusions

Unless otherwise regulated by the State of North Carolina, stormwater control regulations shall not apply to the following:

1. Agriculture, being activities undertaken on agricultural land for the production of plants, crops, fruits, vegetables, ornamental and flowering plants and animals useful to man, including but not limited to forages and sod crops, grains and feed crops, tobacco, cotton, and peanuts; dairy animals and diary products; poultry and poultry products; livestock, including beef cattle, sheep, swine, horses, ponies, mules, or goats, including the breeding and grazing of any or all such animals; bees and apiary products; and fur animals, including the breeding and grazing of such livestock.

2. Forestry, being activities undertaken on woodland areas where all of the following occur:

   a) The growing of trees; and
   b) The harvesting of timber, leaves or seeds; and
   c) The regeneration of either timely replanting of trees or natural generation; and
   d) The application of applicable "best management practices", including the N.C. Department of Environment, Health and Natural Resources. **"Forest Practice Guidelines Related to Water Quality"** – Title 15A North Carolina Administrative *Code* subchapter 11 sections 1.010 - .0209 and all successor documents; and
   e) A forest management plan is prepared or approved either by a professional forester registered in the State of North Carolina or by the Division of North Carolina Forest Resources. Copies of the forest management plan *shall* be provided to the *City* upon request.

3. Activities for which a permit is required under the Mining Act of 1972, G.S. Chapter 74, Article 7.

4. Activities undertaken for the duration of an emergency, activities essential to protection of human life.

5. Activities conducted by the State of North Carolina;

6. Activities conducted by the United States;

7. Activities conducted by *persons* having the power of eminent domain;

8. Activities directly funded, in whole or in part, by the State of North Carolina or the United States; and

9. Projects which commenced prior to the application of this chapter; such projects are:

   a) approved subdivisions and site plans which have not sunsetted.
   b) Projects which have an outstanding unexpired valid building permit in compliance with either G.S. 160A-422 or G.S. 153A-357 or have an outstanding unexpired valid soil erosion permit in compliance with G.S.160A-458; provided that, upon application of any impervious surfaces, the exemption based on a valid soil erosion permit shall not apply.
   c) Projects which have obtained a pertinent state permit, such as landfills and land application of residuals.
   d) Projects which have continuing vested rights in compliance with G.S. 160A-385.1 or G.S. 153A-344.1.

1.2.6.5 Exemptions

Stormwater runoff control requirements *shall* not apply to one or more of the following:

1. The increase in peak stormwater runoff between pre-development and post development conditions for the site for the two-year storm is ten per cent (10%) or less. This includes undeveloped lots, redevelopment, or additions. This exemption does not apply to individual outfalls. The entire site must be considered to determine if this exemption applies.

2. The maximum *impervious surface* coverage of the lot, including both existing and new impervious surfaces, is no more than fifteen per cent (15%) and the remaining pervious portions of the lot are utilized to convey and control the stormwater runoff of the lot to the maximum extent practical. This exemption does not apply to individual outfalls. The entire site must be considered to determine if this exemption applies. Any lot which is exempted from the runoff control requirements by subsection (b)(2), *shall* comply with all the requirements of subsection (a) whenever:

   (a) the exempted lot is subdivided; or
   (b) the exempted lot size is reduced by recombination; or
   (c) *impervious surfaces* on the exempted lot equal or exceed fifteen per cent (15%).

3. Compliance with the runoff limitations would result in greater adverse downstream impact, such as local flooding, as determined by *City* approved engineering studies.

Exempted projects *shall* protect all affected lands and receiving watercourses from accelerated erosion as defined in Chapter 5 Part 10.

1.2.7 Neuse Buffer Rule

The Neuse Buffer Rule requires local governments to ensure that riparian areas are protected on new developments in accordance with the Riparian Buffer Rule (15A NCAC 2B .0233). This Rule, which is part of the Nutrient Management Strategy, applies to existing 50-foot wide riparian buffers directly adjacent to surface waters in the Neuse River Basin (intermittent streams, perennial streams, lakes, ponds, and estuaries). The riparian buffers protected by this Rule are measured perpendicular to the water body and from top of bank for streams and from edge of normal pool for lakes and ponds. A surface water subject to the Rule is considered to be present if the feature is approximately shown on either the most recent version of the County soil survey map prepared by the Natural Resources Conservation Service or on the most recent version of the 1:24,000 scale (7.5 minute) quadrangle topographic maps prepared by the United States Geologic Survey (USGS). Riparian buffers adjacent to surface waters that do not appear on either of the maps are not subject to this Rule. Riparian buffers adjacent to surface waters that appear on the maps are subject to this Rule unless one of the following applies.

- An on-site determination shows that surface waters are not present, or
- Existing uses were present and ongoing on July 22, 1997.

Allowable uses in the riparian buffers have one of four (4) classifications. These uses are designated as exempt, allowable, allowable with mitigation and prohibited and are described below.

a. EXEMPT. Uses designated as exempt are allowed within the riparian buffer. Exempt uses shall be designed, constructed and maintained to minimize soil disturbance and to provide the maximum water quality protection practicable.
b. ALLOWABLE. Uses designated as allowable may proceed within the riparian buffer provided that there are no practical alternatives to the requested use. These uses require written authorization from the Division of Water Quality or the delegated local authority.
c. ALLOWABLE WITH MITIGATION. Uses designated as allowable with mitigation may proceed within the riparian buffer provided that there are no practical alternatives to the requested use and an appropriate mitigation strategy has been approved. These uses require written authorization from the Division of Water Quality or the delegated local authority.
d. PROHIBITED. Uses designated as prohibited may not proceed within the riparian buffer unless a variance is granted.

Persons who wish to undertake uses designated as allowable or allowable with mitigation shall submit a request for a “no practical alternatives” determination to the Division of Water Quality or to the delegated local authority. The applicant shall certify that the criteria identified below are met. The Division of Water Quality or the delegated local authority shall grant an Authorization Certificate upon a “no practical alternatives” determination. The procedure for making an Authorization Certificate shall be as follows:

For any request for an Authorization Certificate, the Division of Water Quality or the delegated local authority will review the entire project and make a finding of fact as to whether the following requirements have been met in support of a “no practical alternatives” determination:

(i) The basic project purpose cannot be practically accomplished in a manner that would better minimize disturbance, preserve aquatic life and habitat, and protect water quality.
(ii) The use cannot practically be reduced in size or density, reconfigured or redesigned to better minimize disturbance, preserve aquatic life and habitat, and protect water quality.
(iii) Best management practices will be used if necessary to minimize disturbance, preserve aquatic life and habitat, and protect water quality.

State law requires that diffuse flow of runoff shall be maintained in the riparian buffer by dispersing concentrated flow and reestablishing vegetation. Concentrated runoff from new ditches or manmade conveyances shall be converted to diffuse flow before the runoff enters the riparian buffer. Periodic corrective action to restore diffuse flow shall be taken if necessary to impede the formation of erosion gullies.

This requirement may be satisfied using level spreaders on gentle slopes. On steeper slopes, detention or other BMPs to slow the release of stored runoff may be necessary. The State Division of Water Quality should be contacted for current standards.

The State has determined that required riparian buffers may not be used as a BMP to treat runoff and reduce nitrogen. However, required riparian buffers may be counted as protected open space when calculating total nitrogen loading for a site.

No activity will be permitted in a required riparian buffer without State approval.

1.3 Stormwater Management Policies

The City of Raleigh has adopted several policies related to the management of stormwater runoff quantity and quality. These policies include: hydrology and hydraulics, water quality, BMP maintenance, and pond preservation.

1.3.1 Hydrology and Hydraulics

The City allows both the Rational and Natural Resources Conservation Service methods for determination of peak flows as well as other methods listed in section 10-5006(5) of the Raleigh City Code. The maximum drainage area for use of the Rational Method is 100 acres in order to maintain consistency with the basin planning process. The City reserves the right to require verification of hydrologic computations by use of a second computational method at its discretion.

The City requires drainage systems to be designed assuming future conditions or build-out of the contributing watershed.

The 2-year storm shall be used for calculating the pre- and post-development runoff to satisfy the Neuse Stormwater control regulations peak runoff control requirements (Code Section 10-9023). Analysis of other frequency and duration storms may need to be evaluated to satisfy other applicable regulations such as CR-7107.

Pond routing is required for computing flow rates through detention ponds. Multiple methods, such as the “short cut” and NRCS routing methods are accepted.

1.3.2 Water Quality

These policies implement citywide, measurable performance goals for control of total nitrogen in stormwater runoff as required by the Neuse Rules, NPDES regulations and others. The control of sediment is also required for construction site runoff citywide, and specific restrictions and performance-based criteria for controlling total suspended solids in stormwater runoff exist in the water supply watershed protection area.

A. When a new development project is located within a Water Supply Watershed Protection area, the more stringent rules apply.

B. Regional and/or minor regional facilities are preferable to on-site BMPs.

C. A timeline for design and construction of regional controls must be provided, beginning when the first project in such a drainage area is approved.

D. The preferred BMPs will be retention facilities, preferably wet ponds. Bioretention, buffers, vegetated swales, and artificial wetlands are acceptable BMPs.

E. Infiltration-based BMPs, such as trenches and pits, should be avoided.

F. BMPs that require frequent replacement of media are not recommended.

G. Installation of BMPs should be scheduled with site stabilization and removal of temporary sedimentation and erosion control devices to avoid contamination, clogging, and premature failure of the BMP.

H. Garbage dumpsters and apartment or condominium car washing areas must be located such that runoff from these areas sheet flows across a densely vegetated area. These facilities cannot be located in close proximity to streams or other watercourses.

I. The Neuse Riparian Buffer rules require 50 foot riparian buffers along all intermittent and perennial streams.

Phase I of the NPDES stormwater discharge permit program was implemented by USEPA in November of 1990. This program was developed to address what USEPA determined to be the most significant sources of nonpoint source runoff pollution in urbanized areas and from specific industrial sources of nonpoint source pollution. During the fist phase USEPA addressed urban runoff from large municipalities (population greater than or equal to 250,000[2]) and from medium municipalities (population greater than or equal to 100,000 and less than 250,000). The City of Raleigh was classified as a medium sized municipality under this regulation[3] and was required to prepare an application for a NPDES stormwater discharge permit for its municipal separate storm sewer system (MS4). The permit, NCS000245, was issued to the City in January 1995 by the Division of Environmental Management (DEM), now the Division of Water Quality (DWQ). This permit does not have numeric limits, but requires that the City develop and implement a city-wide stormwater quality management program based on best management practices (BMPs) and on a prohibition on the discharge of anything but stormwater to the MS4. Because the NPDES permit is a federal program required under the Clean Water Act (CWA), EPA has the authority to levy steep fines for non-compliance.

1.3.3 BMP Maintenance

Proper operation and maintenance of BMPs is critical to insure that the effectiveness and integrity of the BMPs as water quality controls is maximized. This insurance is critical in a performance-based program of stormwater runoff controls.

A. BMP maintenance is the responsibility of the facility owner.

B. Private easements are to be provided whenever a facility treats runoff from more than one property.

C. The City will provide BMP maintenance when the responsible party defaults on that responsibility. Compensation to the City for these services will be made from the maintenance escrow account or as a lien against properties or both.

D. BMPs and other installed measures must be located on lots containing improvements equal to or greater in value than the replacement value of the measures and devices.

[2] Based on the 1990 census.
[3] 40 CFR 122

1.3.4 Pond Preservation Policy

The Pond Preservation Policy is a local policy that was developed to encourage the preservation of existing ponds that either currently provide water quality and/or quantity control benefits to a drainage area, or that could provide those benefits with a moderate level of structural modification. The Raleigh area has hundreds of these ponds, located in perennial, intermittent, and ephemeral streams. Most of these ponds were developed for local livestock water supply and local flood control, but in many cases the pond sites and sizes are strategically suited for retrofitting to provide stormwater management functions related to both water quality and quantity control.

1.4 Stormwater Quantity and Quality Information Sources

Much information is available on stormwater runoff and runoff water quality. Typical information sources include the National Weather Service for rainfall information; the USGS for streamflow and water quality information: local universities and the Water Resources Research Institute for information on specific studies; and the City of Raleigh for information on studies and data evaluations that have been performed in the past. The following sections describe the sources of data for information pertinent to hydrologic and water quality design in the City of Raleigh.

1.4.1 Design Storms

In order to provide both a common baseline for the evaluation of hydrologic and hydraulic computations and in order to evaluate designs based on accepted risk levels for hydraulic structures, design storms have been identified and implemented in the City of Raleigh. These design storms have been in use for several years for hydrologic and hydraulic design and can be found in Chapter 2 of this manual.

The second commonly used design rainfall depth is referred to as the *first flush*, or 1 inch of rainfall. The first flush represents the higher levels of initial concentrations of constituents that are washed off from a surface at the very beginning of a rainfall event. The first flush depth is used for sizing smaller facilities, such as bioretention areas, perimeter sand filters, and other first flush devices. The first flush, or water quality volume WQ_v, can be calculated with the following equation:

(1.1) Water Quality Volume

$$WQ_v = \frac{(P)(R_v)(A)}{12}$$

where:

WQ_v = water quality volume in ac-ft
P = 1 inch of rainfall
R_v = 0.05 + 0.009(I), where I= the percent impervious cover
A = drainage area in acres

1.4.2 Typical Nonpoint Source Pollutant Concentrations

Much information is available on the typical event mean concentrations (EMCs) of nonpoint source pollutants in urban runoff. The information is available nationally through such resources as the USEPA NURP (Nationwide Urban Runoff Program), the USEPA STORET (STOrage and RETrieval environmental database), and the USGS WATSTORE (WATer data STORage and REtrieval system) databases, and regionally through the municipal NPDES storm water discharge permits' monitoring requirements. The EMCs obtained from the NURP program participants (1979 through 1982, 28 cities) are widely regarded as the most statistically significant data on urban stormwater runoff quality in existence. These data, modified based on the results of wet weather sampling performed by Raleigh and Durham, were used by the staff of DWQ in determination of total nitrogen export rates.

It should be noted that in the final report of the NURP project the USEPA reported that the statistical analysis of the physical and chemical data collected during the study identified the single most significant relationship between water quality and other parameters to be the relationship with imperviousness. This means that there is also a relationship between increased stormwater runoff rates and runoff water quality. This is consistent with the approach taken by the State and incorporated by the City into this document for the estimation of total nitrogen export to receiving streams.

1.4.3 Best Management Practices

There are two major categories of best management practices (BMPs), non-structural and structural. Non-structural BMPs are basically passive or programmatic BMPs. Non-structural BMPs include public education and outreach, used oil recycling, household hazardous waste turn-in, litter control programs, zoning and land use controls, chemical applicator certification and training, etc. Non-structural BMPs tend to be source control BMPs that reduce pollution in runoff by reducing the opportunity for the pollutants to be exposed to stormwater runoff. Structural BMPs are physical structures that can be seen on the ground, including wet and dry ponds, bioretention areas, artificial wetlands, grassed swales, filters strips, buffer strips, and manufactured BMPs, such as catch basin inserts. Some structural BMPs are passive and would be considered source controls while others would be considered end of pipe treatment.

The impact of non-structural controls, other than restrictive zoning or impervious area limitations, on total nitrogen or total suspended solids reduction programs is relatively insignificant when compared to the amounts of the pollutants that must be controlled. The BMPs specified in Chapter 3 of this manual are structural in nature and can provide significant reductions in the export of total nitrogen. These BMPs have been in use nationally and regionally for more than two decades. A great deal of follow-up work has been done in Northern Virginia, Maryland, and Florida in the investigation of BMP performance, costs, and maintenance criteria.

1.5 Submittal Requirements

When preparing a plan and specification submittal for new development the following materials will be required:

1.5.1 Preliminary Site Plan

A preliminary site plan is required for review and approval. This plan must show:

- all pipe sizes and drainage locations,
- all existing and proposed easements,
- the location and description of all flood restriction lines and elevations, and
- the location of riparian buffers.

1.5.2 Stormwater Control Plan

Upon completion of the preliminary site plan the stormwater control plan must be submitted and approved. This plan should include the complete storm drainage system and all supporting calculations for review. Specific supporting documentation for compliance with the Neuse River NSW Management Strategy includes;

- location map,
- site map,
- locations of dumpsters and/or apartment or condominium car washing areas,
- zoning information, including existing and proposed drainage easements and floodplains / floodways, wetland delineations, and watercourse buffer boundaries,
- the location of required structural BMPs,
- total nitrogen export computations,
- documentation of the determination of appropriate BMPs and offset fees,
- a schedule for implementation of all proposed water quality BMPs that specifies when the BMP(s) will be on-line with respect to the development schedule for the drainage area serviced by the BMP,
- certification by a North Carolina registered professional engineer, registered landscape architect, or registered land surveyor who is qualified in hydrology and hydraulics, stating that the plans comply with the standards in the City of Raleigh, as per Section 1.5.4.
- computation of maintenance escrow, if required,
- peak runoff calculations to each outfall leaving a site and any required BMP's to meet peak runoff control requirements
- a completed plan submittal checklist, included in Section 4 – Appendices.
- BMP maintenance manual, budget and any necessary

agreements.

1.5.3 As-Built Plans

Upon completion of the new construction, the developer is required to provide "as-built" plans, certified by a NC registered professional engineer, landscape architect, or land surveyor, prior to receiving an occupancy permit for the property. These plans are also to be certified using the language provided in Section 1.5.4.

1.5.4 Engineer's Certification

The stormwater quality management program of the City of Raleigh is a performance-based program. In order to achieve the performance that the BMPs are intended to provide, proper design, construction, operation, and maintenance of stormwater management facilities and BMPs are essential.

The certification must be provided by a North Carolina registered professional engineer, registered landscape architect, or registered land surveyor who is qualified in hydrology and hydraulics. The certification to be provided with engineering or stormwater control plans is provided below and should be stamped or sealed, signed and dated with the submittal.

> "I certify that this plan complies with the ordinances, rules, regulations, and stormwater drainage design standards of the City of Raleigh."

Certification must also be provided by a North Carolina registered professional engineer, registered landscape architect, or registered land surveyor for the "as-built" plans. The certification to be made is provided below and should be stamped or sealed, signed and dated with the submittal.

> "I certify that the stormwater management facilities are constructed and installed in conformance with the ordinances, rules, regulations, drainage design standards of the City of Raleigh, and the approved stormwater management plan."

1.5.5 Annual Inspections

Inspections of the BMPs and other stormwater controls are to be made and submitted to the Inspections Department annually to ensure that routine and remedial maintenance is being performed and that the BMPs are operating properly. The annual certification that appropriate maintenance is being performed is to be made by a North Carolina registered professional engineer, registered landscape architect, or registered land surveyor. The following certification is to be made with accompanying stamp or seal, signature and date.

> "I certify that the BMPs, stormwater management facilities, and open space areas referenced in this document have been maintained in conformance with the approved stormwater management plan and maintenance manual. This certification is made based on personal observation or on the observation by someone under my direct supervision of the site and review of maintenance records."

1.6 Regional Stormwater Management Plans (SWMPs)

The most efficient and cost effective means of managing both the quantity and quality of stormwater runoff is through the use of regional Stormwater Management Plans (SWMPs). Regional SWMPs may include a system of BMPs in series or parallel designed to treat the runoff from a large site or network of contiguous sites, or a single BMP designed to treat the runoff from a site or new development. When a site is part of an approved regional SWMP, there will be no requirement for on-site structural controls unless they are part of the regional plan. Non-structural controls required by the City as part of its stormwater management program are not exempted by regional SWMPs.

In some instances it will be necessary to provide BMPs prior to complete development of the drainage area when a multi-phased development is planned. In these instances, any regional BMPs may have to be implemented as part of the initial development of the site. This can be accomplished by the developer(s) and/or the City front-ending the cost of a regional BMP(s) and recovering the incremental cost as development continues through a latecomer, or impact fee, assessed as each additional property in the drainage area is developed.

In cases where the public's best interest is served by the public construction of regional BMPs or BMP systems, a predetermined schedule of fees will be established at the time of construction of the BMP. The BMP(s) will be sized to control the quantity and quality of runoff from the upstream drainage areas under future conditions. Future development in the drainage area will pay a fee in-lieu-of providing on-site stormwater management.

For many regional SWMPs, there is a high likelihood that both existing and new development will be present in the drainage area. In these instances the BMP will need to be sized appropriately to treat ALL runoff from the drainage area. In these instances property owners may be able to obtain partial offsetting credit against the need to provide water quality treatment for areas of a contiguous development that do not fall within the drainage area of the regional BMP when the runoff from existing development is being treated by the regional BMP. This credit will be discretionary on the part of the City and will require documentation of the respective total nitrogen removal provided by the BMP from new development and existing development, as well as the export calculations for the new development scheduled to occur outside the drainage area of the regional BMP.

Where it is desired to use an adequately sized existing pond or other impoundment and the impoundment is in the control of, and consequently could be removed by the owner, then the owner may consider the pre-development condition to be that which would exist if the impoundment were removed. In this way, the impoundment owner may receive credit for the stormwater management benefits provided by the existing impoundment without having to go through the mechanics of draining the impoundment to establish this condition as the pre-developed condition. In such a situation, the existing impoundment may receive credit for both peak flow attenuation and pollutant reduction.

1.7 Maintenance

As stated in Section 10-9027 of the City Code, the land *owner* or *person* in possession or control of the land *shall maintain* all stormwater control measures and devices and all *open space areas* required by the approved stormwater control plan unless those measures, devices, and *open space areas* are accepted for *maintenance* by a governmental agency.

For *off-site stormwater control facilities*, and for all other *stormwater control facilities* which serve more than one (1) lot, that are not accepted for *maintenance* by a governmental agency, a maintenance covenant complying with the standards set forth in subsections (1) through (4) of §10-5007(c) is required. The maintenance covenant *shall* be recorded with the local county register of deeds prior to the recording of any new lot served by the *stormwater control facility* or prior to the issuance of any development permit for any existing lot except for improvements made pursuant to Part 10 chapter 3 of the Code referenced previously. The maintenance covenant *shall* be binding on all subsequent owners of land served by the *stormwater control facilities.* A recorded copy of the maintenance covenant *shall* be given to the Department of Inspections of the *City* within fourteen (14) days following the recordation of the maintenance covenant.

Where a pipe or other man-made conveyance is used to transport runoff to an off-site facility and that conveyance passes through a public street ROW or other publicly owned property, an encroachment and maintenance agreement shall be approved by the City prior to the issuance of permits.

For all *stormwater control facilities* which are to be or are owned and maintained by a property owner's association or similar entity, in addition to the required maintenance covenant, the developer and the association *shall* enter into an agreement with *City*. The agreement *shall* contain all of the following provisions:

(1) The percent of developer contribution and lengths of time to fund the escrow account may be varied by the *City* depending on the design and materials of the *stormwater control facility*.

(2) Acknowledgment that the association *shall* continuously operate and *maintain* the *stormwater control facilities.*

(3) Establishment of an escrow account which can be spent solely for sediment removal, structural, biological or vegetative replacement, major repair, or reconstruction of the stormwater control measures and devices of the particular site plan or subdivision. If *stormwater control facilities* are not performing adequately or as intended or are not properly *maintained*, the *City*, in its sole discretion, may remedy the situation, and in such instances the *City shall* be fully reimbursed from the escrow account. Escrowed funds may be spent by the association for sediment removal, structural, biological or vegetative replacement, major repair, and reconstruction of the *stormwater control facilities*; provided that, the *City shall* first consent to the expenditure.

(4) Both developer contribution and annual sinking funds *shall* fund the escrow account. Prior to plat recordation or issuance of construction permits, whichever *shall* first occur, the developer *shall* pay into the escrow account an amount equal to fifteen (15) per cent of the initial construction cost of the *stormwater control facilities*. As determined from the sinking fund budget set forth in §10-5007(c)(4), two-thirds (2/3) of the total amount of sinking fund budget *shall* be deposited into the escrow account within the first five (5) *years* and the full amount *shall* be deposited within ten (10) *years* following initial construction of the stormwater control measure or device. Moneys *shall* be deposited each *year* into the escrow account. A portion of the annual assessments of the property owners association *shall* include an allocation into the escrow account. Any funds drawn down from the escrow account *shall* be replaced in accordance with the schedule of anticipated work used to create the sinking fund budget.

(5) Granting to the *City* a right of entry to inspect, monitor, *maintain*, repair, and reconstruct *stormwater control facilities*.

(6) Allowing the *City* to recover from the association and its members any and all costs the *City* expends to maintain or repair the *stormwater control facility* or to correct any operational deficiencies. Failure to pay to the *City* all of its expended costs, after forty-five (45) days *written* notice, *shall* constitute a breach of the agreement. The *City shall* thereafter be entitled to bring an action against the association and its members to pay, or foreclose upon the lien herein authorized by the agreement against the property, or both in the case of a deficiency. Interest, collection costs, and attorney fees *shall* be added to the recovery.

(7) A statement that this agreement *shall* not obligate the *City* to *maintain* or repair any stormwater control measure or device, and that the *City shall* not be liable to any *person* for the condition or operation of *stormwater control facilities*.

(8) A statement that this agreement *shall* not in any way diminish, limit, or restrict the right of the *City* to enforce any of its ordinances as authorized by law.

Chapter 2 Hydrology and Hydraulics

2.1 Hydrologic Methods

The design of properly sized storm drainage facilities requires some knowledge of the hydrologic behavior of the watershed in question. For most designs it is adequate to estimate the peak discharge of the drainage area for the required design frequency. Larger, more complicated watersheds may require use of a distributed element model in order to estimate the discharge hydrograph. Several simple methods as well as several distributed element models commonly used are presented in this manual. Other models may be used in lieu of these if the model is appropriate for the watershed.

Every model has certain limitations that will effect its behavior for different size drainage areas. The designer should be familiar with the limitations of the method he is using. In general, street drainage and small drainage areas (less than a hundred acres) can be modeled using the rational equation. Larger areas can be modeled using methods developed by the Natural Resources Conservation Service. Distributed element models generally use one of these hydrologic methods but also employ algorithms to account for reduction in the peak discharge due to storage in the watershed from reservoirs or road crossings. The hydrologic model used in the distributed element model should be appropriate for each of the sub-watershed areas.

Many hydrologic methods are available. The following methods are recommended and the circumstances for their use are listed in Table 2.1 below. If other methods are used they should first be calibrated to local conditions and tested for accuracy and reliability.

Table2.1 Recommended Hydrologic Methods

Method[1]	Size Limitations[2]	Comments
Rational	0 – 100 acres	Method can be used for estimating peak flows and the design of small sub-division type storm sewer systems. Method should not be used for storage design.
NRCS (SCS)	100 – 2000 acres	Method can be used for estimating peak flows and TP-149 hydrographs. Method can be used for the design of all drainage structures including storage facilities.

NOTES:
[1] There are many readily available programs (such as HEC-1, TR-20, XP-SWMM, and Pond-Pack) that utilize these methodologies.
[2]Size limitation refers to the drainage basin for the stormwater management facility (i.e., culvert, inlet).

The chosen methods were selected based on several considerations, including the following:

- Historical use in the City of Raleigh.
- Verification of their accuracy in duplicating local hydrologic estimates of a range of design storms throughout the State of North Carolina.
- Availability of equations, nomographs, and computer programs.
- Use and familiarity with the methods by local municipalities and consulting engineers.

2.2 Rational Method

It is usually acceptable to design storm drainage facilities for street drainage and relatively small areas (less than one hundred acres) using the rational method. Beyond this, the designer should employ another model to perform his design, or at least validate the use of the rational equation. In general, for larger areas the rational method will yield over-simplified results.

When using the rational method some precautions should be considered:

- In determining the C value (land use) for the drainage area, hydrologic analysis should take into account any changes in land use.
- The rational method uses a composite C value for the entire drainage area. If the distribution of land uses within the drainage basin will affect the results of hydrologic analysis, then the basin should be divided into two or more sub-drainage basins for analysis.
- The charts, graphs, and tables included in this section are given to assist the designer in applying the rational method. The designer should use good engineering judgment in applying these design aids and should make appropriate adjustments when specific site characteristics dictate that these adjustments are appropriate.

2.2.1 Equation

The rational formula estimates the peak rate of runoff at any location in a watershed as a function of the drainage area, runoff coefficient, and mean rainfall intensity for a duration equal to the time of concentration (the time required for water to flow from the most remote point of the basin to the location being analyzed). The rational formula is expressed as follows:

Equation 2.1 Rational Equation

$$Q = C\ I\ A$$

Where:

Q	=	Peak flow from the drainage area (cfs)
C	=	Coefficient of runoff (dimensionless)
I	=	Rainfall intensity for a given time to peak (in/hr)
A	=	Drainage area (acres)

The rational equation is based on the assumption that rainfall is uniformly distributed over the entire drainage area and at a steady rate, causing flow to reach a maximum at the outlet to the watershed at the time to peak (Tp). The rational method also assumes that all land uses within a drainage area are uniformly distributed throughout the area. If it is important to locate a specific land use within the drainage area then another hydrologic method should be used where hydrographs can be generated and routed through the drainage system.

2.2.2 Runoff Coefficient

The runoff coefficient (C) is the variable of the rational method least susceptible to precise determination and requires judgment and - understanding on the part of the design engineer. While engineering judgment will always be required in the selection of runoff coefficients, typical coefficients represent the integrated effects of many drainage basin parameters. Table 2.2 gives the recommended runoff coefficients for the rational method.

Table 2.2 Recommended Runoff Coefficient Values

(Sources: North Carolina Erosion and Sediment Control Planning and Design Manual and The City of Raleigh's Storm Drainage Design Manual, 1989)

Description of Area	Runoff Coefficient, C
Woodlands	0.20 - .025
Parks, cemeteries	0.25
Playgrounds	0.35
<u>Lawns:</u>	
Sandy soil, flat, 2%	0.10
Sandy soil, average, 2 - 7%	0.15
Sandy soil, steep, > 7%	0.20
Clay soil, flat, 2%	0.17
Clay soil, average, 2 - 7%	0.22
Clay soil, steep, > 7%	0.35
<u>Graded or no plant cover:</u>	
Sandy soil, flat, 0 - 5%	0.30
Sandy soil, flat, 5 - 10%	0.40
Clayey soil, flat, 0 - 5%	0.50
Clayey soil, average, 5 - 10%	0.60
<u>Residential:</u>	
Single-family (R - 4)	0.50
Single-family (R - 6)	0.55
Multi-family (R - 10)	0.60
Multi-family (R - 20)	0.70
Multi-family (R - 30)	0.75
<u>Business:</u>	
O & I (I, II, III)	0.85
I1 & I2	0.85 – 0.95
Shopping Centers	0.85 – 0.95
<u>Streets:</u>	
Gravel areas	0.50
Drives, walks, and roofs	0.95
Asphalt and Concrete	0.95 – 1.00

It is often desirable to develop a composite runoff coefficient based on the percentage of different types of surfaces in the drainage areas. Composites can be made with the values from Table 2.2 by using percentages of different land uses, as illustrated in Equation 2.2. In addition, more detailed composites can be made with coefficients for different surface types such as roofs, asphalt, and concrete streets, drives and walks. The composite procedure can be applied to an entire drainage area or to typical "sample" blocks as a guide to the selection of reasonable values of the coefficient for an entire area.

Equation 2.2 Composite C

$$\text{Composite C} = \frac{C1 \cdot A1 + C2 \cdot A2 + \ldots Cx \cdot Ax}{A1 + A2 + \ldots Ax}$$

2.2.3 Rainfall Intensity

The rainfall intensity (I) is the average rainfall rate in in./hr for a duration equal to the time of concentration for a selected return period. Once a particular return period has been selected for design and a time of concentration calculated for the drainage area, the rainfall intensity can be determined from the intensity-duration-frequency (IDF) data for the City of Raleigh given in Table 2.3.

Table 2.3 Intensity – Duration - Frequency Table
City of Raleigh, NC
(Developed by Dr. H.R. Malcom, North Carolina State University, Dept. of Civil Engineering, and the authors based on NOAA HYDRO-35 and USWB TP-40)

Duration	**Frequency (Yrs)**					
	2	5	10	25	50	100
5 mins	5.76	6.58	7.22	8.19	8.96	9.72
10	4.76	5.54	6.13	7.01	7.71	8.40
15	4.04	4.74	5.25	6.03	6.64	7.24
20	3.47	4.12	4.64	5.42	5.93	6.47
30	2.70	3.28	3.71	4.32	4.80	5.28
40	2.28	2.77	3.15	3.70	4.08	4.48
50	1.94	2.38	2.71	3.19	3.53	3.88
60	1.70	2.12	2.41	2.84	3.17	3.50
90	1.22	1.52	1.74	2.06	2.29	2.53
2 hr	0.95	1.20	1.37	1.62	1.81	2.00
3	0.71	0.89	1.02	1.21	1.35	1.50
6	0.44	0.56	0.65	0.77	0.86	0.96
12	0.26	0.33	0.39	0.46	0.52	0.57
24	0.15	0.19	0.22	0.27	0.30	0.33

2.2.4 Time Of Concentration

Use of the rational formula requires the time of concentration (t_C) for each design point within the drainage basin. The duration of rainfall is then set equal to the time of concentration and is used to estimate the design average rainfall intensity (I). The time of concentration consists of an overland flow time to the point where the runoff enters a defined drainage feature (i.e., open channel) plus the time of flow in a closed conduit or open channel to the design point.

There are several acceptable methods for calculating the time of concentration, including a simple nomograph for use with the rational formula or the use of equations such as the kinematic wave or Kirpich equations.

Simple Nomograph

Figure 2.1 is a simple nomograph that can be used to estimate overland flow time. For each drainage area, the distance is determined from the inlet to the most remote point in the tributary area. From a topographic map, the average slope is determined for the same distance. The runoff coefficient (C) is determined by the procedure described in a subsequent section of this chapter.

To obtain the total time of concentration, the pipe or open channel flow time must be calculated and added to the inlet time. After first - determining the average flow velocity in the pipe or channel, the travel time is obtained by dividing velocity into the pipe or channel length. Velocity can be estimated by using the nomograph shown on Figure 2.2. Note: time of concentration cannot be less than 5 minutes.

Kinematic Wave

Another method that can be used to determine the overland flow portion of the time of concentration is the "Kinematic Wave Nomograph – Figure 2.3." The kinematic wave method incorporates several variables including rainfall intensity and Manning's "n". In using the nomograph, the engineer has two unknowns starting the computations: the time of concentration and the rainfall intensity. The problem is attempting to determine a rainfall intensity, which in turn actually determines the time of concentration. Thus, the problem is one of iteration. A value of "I" must be assumed, compute a time of concentration and then check back to see if the rainfall intensity that was assumed is consistent with the rainfall intensity from the rainfall intensity in Table 2.3. If one has determined the length, slope, roughness coefficient, and selected a rainfall intensity table, the steps to use Figure 2.3 are as follows:

1. Assume a rainfall intensity.

2. Use Figure 2.3 (or the equation given in the figure) to obtain the first estimate of time of concentration.

3. Using the time of concentration obtained from Step 2,

enter the rainfall intensity table and find the rainfall intensity corresponding to the computed time of concentration. If this rainfall intensity corresponds with the assumed intensity, the problem is solved. If not, proceed to Step 4.

4. Assume a new rainfall intensity that is between that assumed in Step 1 and that determined in Step 3.

5. Repeat Steps 1 through 3 until there is good agreement between the assumed rainfall intensity and that obtained from the rainfall intensity tables.

Generally, the time of concentration for overland flow is only a part of the overall design problem. Often one encounters swale fow, confined channel flow, and closed conduit flow-times that must be added as part of the overall time of concentration.

Figure 2.1 Rational Formula - Overland Time of Flow Nomograph

(Source: Airport Drainage, Federal Aviation Administration, 1965)

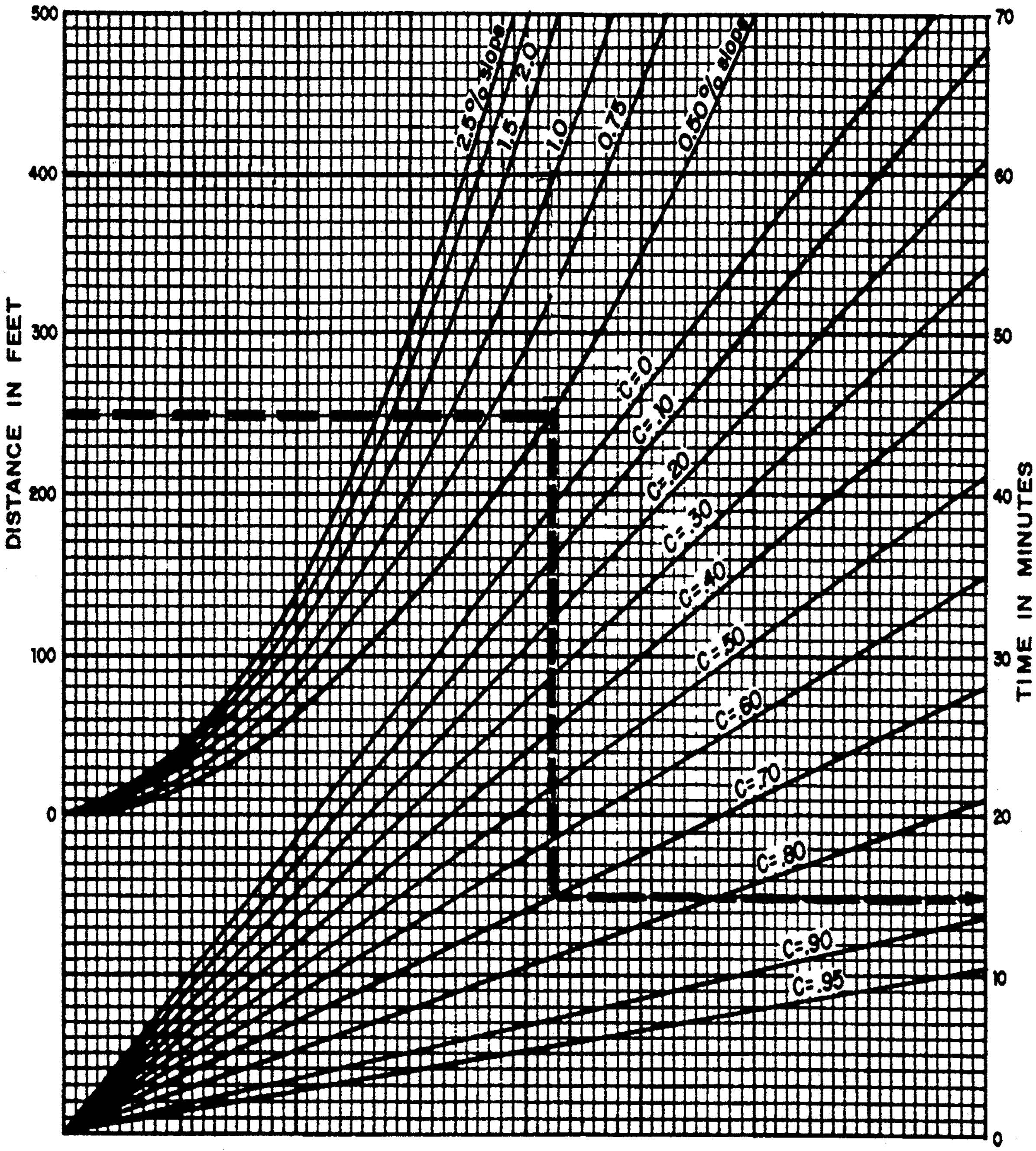
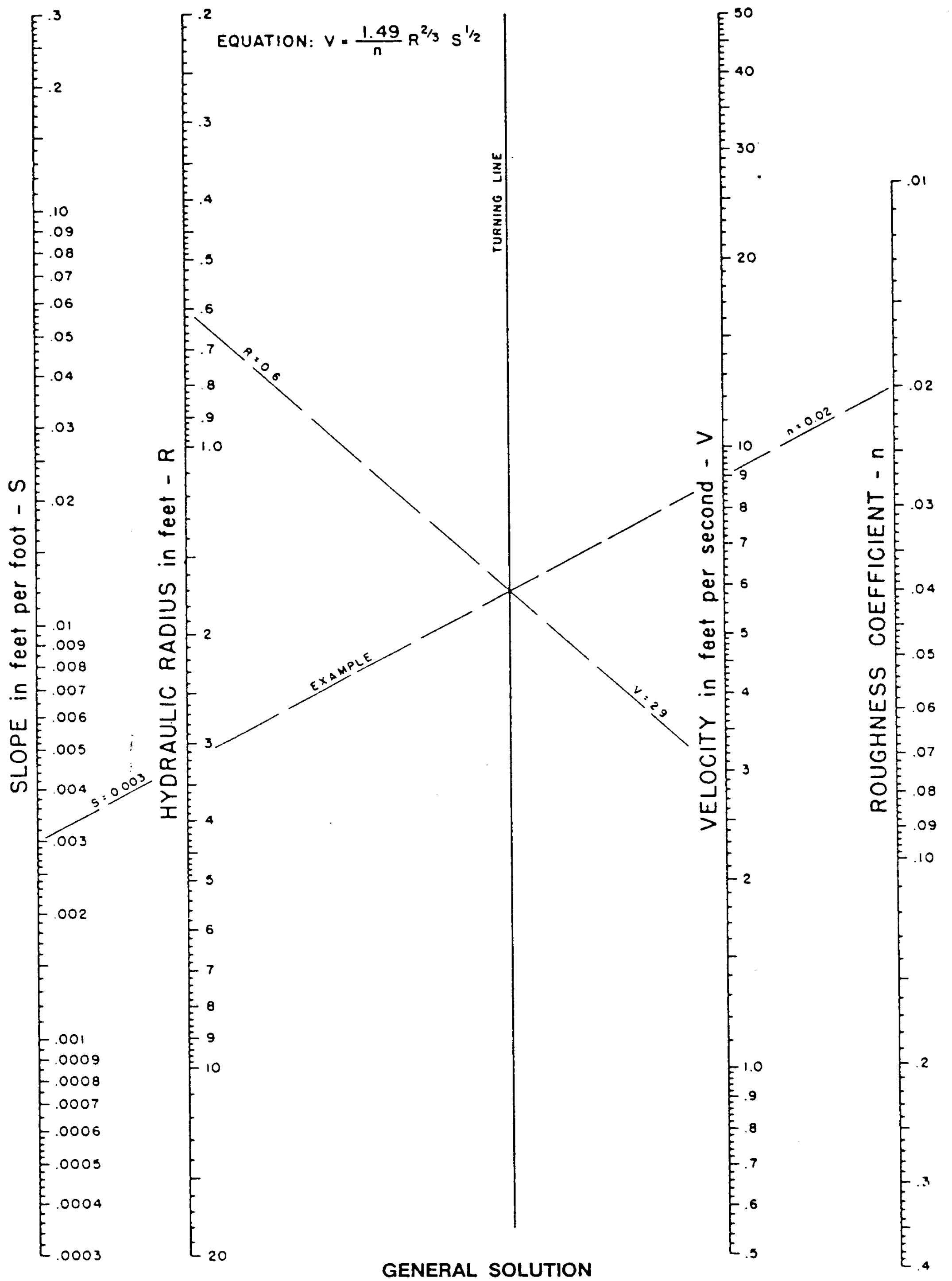


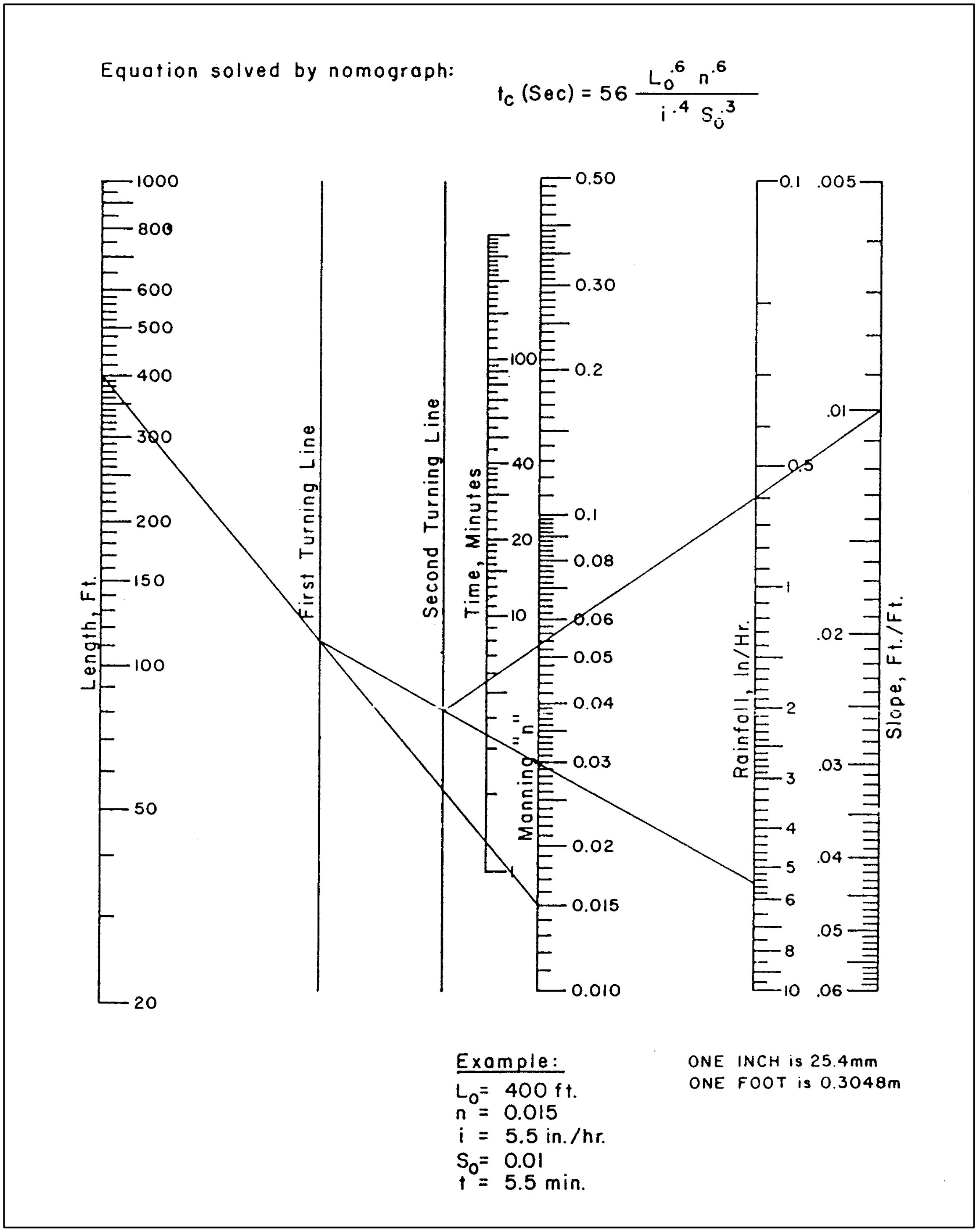
Figure 2.2 Manning's Equation Nomograph



Reference: USDOT, FHWA, HDS-3 (1961).

Figure 2.3 Kinematic Wave Nomograph

(Source: Regan, R. M., A Nomograph Based On Kinematic Wave Theory for Determingin Time of Concentration for Overland Flow, 1971)



When this situation is encountered, it is best to compute the confined flow-times as the first step in the overall determination of the time of concentration. This will give the designer a rough estimate of the time involved for the overland flow that will give a better first start on the rainfall intensity assumption. For example, if the flow time in a channel is 15 minutes and the overland flow time from the ridgeline to the channel is 10 minutes, then the total time of concentration is 25 minutes.

2.2.5 Kirpich Equation

The Kirpich equation is based on empirical data and observation. Although it has no analytical basis, it has proven an effective method in many years of use. It is therefore widely considered an acceptable method for estimating time of concentration for small drainage areas. The basic form of the equation is:

Equation 2.3 Kirpich Equation

$$Tc = \frac{(L^3 / H)^{0.385}}{128}$$

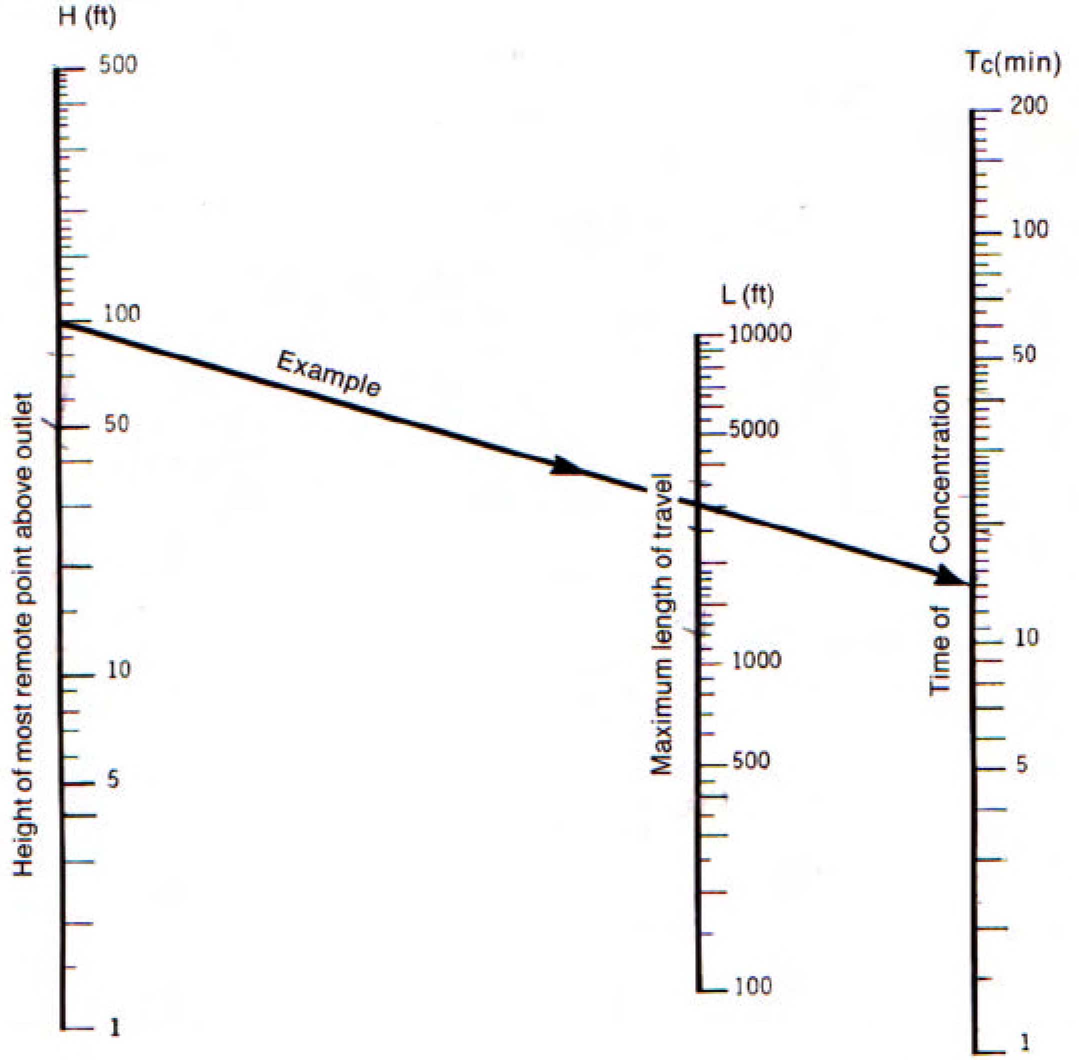
Where :

Tc	=Time of Concentration (min)
H	=Height of the most remote point on the watershed above the outlet (ft)
L	=Length of flow from the most remote point on the watershed to the outlet (ft)

(Civil Engineering, Vol. 10, No. 6,June 1940,p.362.)

A graph of the Kirpich Equation also appears in Figure 2.4.

Figure 2.4 Kirpich Equation
(Source: North Carolina Erosion and Sediment Control Planning and Design Manual)



Example:
Height above outlet, H = 100’
Length of travel, L = 3000’
Time of concentration, Tc = 14 mins

Time of Concentration Notes

Two common errors should be avoided when calculating time of concentration - T_c.

In some cases runoff from a portion of the drainage area which is highly impervious may result in a greater peak discharge than would occur if the entire area were considered. In these cases, adjustments can be made to the drainage area by disregarding those areas where flow time is too slow to add to the peak discharge.

When designing a drainage system, the overland flow path is not necessarily the same before and after development and grading operations have been completed. Selecting overland flow paths in excess of 100 feet in urban areas and 300 feet in rural areas should be done only after careful consideration. Except in very flat areas, overland flow time should not be greater than the pipe or channel flow time.

The following guidance should be considered:

a. For most applications using the rational method the designer may assume the time of concentration (T_c) is equal to the time to peak (Tp). Adjustments are commonly made to Kirpich Equation to compensate for channelization.

b. For well-defined natural channels, use T_c.

c. For overland flow on grassy surfaces, use T_c * 2.

d. For overland flow on paved surfaces, use T_c * 0.4.

e. For concrete channels, use T_c * 0.2.

2.2.6 Example Problem

A new road culvert is proposed as a single reinforced concrete pipe (RCP).

- Proposed Invert In = 300.0 ft above MSL
- High Point on Watershed = 350.0 ft
- Drainage Area = 50 Ac.
- Length of Flow = 1400 ft
- The watershed consists of 20 acres of R-4 zoning, 20 acres of R-10 zoning and a 10-acre park.

Calculate the 10-year peak discharge at the proposed pipe.

Solution:

<u>Step 1:</u> Calculation of Composite C Value

$$C = \frac{(20\ Ac)(0.4) + (20\ Ac)(0.6) + (10\ Ac)(0.3)}{50\ Ac} = \mathbf{0.46}$$

<u>Step 2:</u> Calculate the time of concentration from the Kirpich equation.

Since the time increment is short, assume Tp = Tc

$$Tc = \frac{(L^3 / H)^{0.385}}{128} = \frac{(1400^3 / 50)^{0.385}}{128} = \mathbf{7.46}$$

From Table 2.3, I = **6.8 in/hr**

<u>Step 3:</u> Calculate the peak discharge.

Q = C I A = (0.46 in/hr)*(6.8)*(50 Ac) = **156 cfs**

The initial pipe selection can then be made on the basis of a design discharge of 156 cfs.

2.3 NRCS (SCS) Unit Hydrograph

The NRCS (SCS) curve number method for estimating runoff has appeared in several forms over the past few years. The methods presented here are simplifications of the NRCS TR-55 Graphical Method (1986 version), and the ES 1027 charts for flat, moderate, and steep slopes.

The curve number method was originally based on studies of small watersheds, 1 square mile, and is considered by many engineers to be limited in its application to highly urbanized watersheds. For use within the city of Raleigh, the NRCS method as presented here should be limited to use on drainage areas larger than about one hundred acres. If the watershed has extensive pipe systems or improved channels, the engineer should take care in computing travel times and should consider using a distributed element model. The ES 1027 charts should be used with great caution in urban watersheds because of the variety of drainage surfaces. Highly urbanized areas may have a much shorter travel time than those assumed by the ES 1027charts.
In the Raleigh area, the NRCS Type II storm is typical of the larger storms experienced. Smaller watersheds are more sensitive to short, more intense rainfall than larger areas. Larger areas are in turn more sensitive to longer rainfall duration. In developing the NRCS Type II storm pattern small time increments were used in order to encompass a wide range of storms. The result is a generalized center-weighted storm used as the design storm.

The quantity of runoff in the NRCS method can be attributed to several factors. Watershed slope, soil type, ground cover, and antecedent moisture content all effect the quantity of runoff.

Soil types are divided into four major hydrologic soil groups denoted by the letters A through D. A soils are those which have high infiltration capacity and subsequently low runoff rates. D soils are those with very low infiltration capacity and very high runoff rates. A list of soils common in North Carolina can be found at the local NRCS office. Those soils given dual notation represent hydrologic classifications for drained and undrained conditions. The Wake County Soil Survey or maps from the Central Engineering or Inspections Departments are good sources of information on soil types for specific locations.
For the purposes of the NRCS method, antecedent moisture content (AMC) is divided into dry, normal, and wet conditions based on the rainfall in the prior five days. If the 5-day antecedent rainfall is greater than 2.1 inches in the growing season or 1.1 inches in the dormant season the moisture content is presumed wet (AMC III). If the 5 day antecedent rainfall is less than 1.4 inches or 0.5 inches respectively, it is presumed dry (AMC I). The runoff curve numbers presented here are based on normal conditions (AMC II). For design of proposed facilities, normal conditions are generally used.

In order to calculate the peak discharge using the Graphical Method one must calculate the volume of runoff from the watershed, the time of concentration, and apply this to the standard NRCS hydrograph shape.

2.3.1 Runoff Volume

The volume of flood runoff can be calculated by the following equation.

Equation 2.4 Runoff Volume

$$Q = (P - I_a)^2 \ / \ (P - I_a) + S$$

Where:
- Q = accumulated direct runoff (in.)
- P = accumulated rainfall (potential maximum runoff) (in.)
- I_a = initial abstraction including surface storage, interception, and infiltration prior to runoff (in.)
- S = potential maximum soil retention (in.)

The empirical relationship used in the NRCS method for estimating I_a is:

Equation 2.5 Infiltration for TR55

$$I_a = 0.2S$$

And CN can be expressed as:

Equation 2.6 CN for TR55

$$CN = 1000 / (10 + S)$$

Where: CN = NRCS curve number

General and Raleigh-specific CN numbers can be found in Table 2.4.

Table 2-4 Runoff Curve Numbers[1]

Cover Description	Curve Numbers for Hydrologic Soil Groups			
Cover type and hydrologic condition	**A**	**B**	**C**	**D**
Cultivated land:				
without conservation treatment	72	81	88	91
with conservation treatment	62	71	78	81
Pasture or range land				
poor condition	68	79	86	89
good condition	39	61	74	80
Meadow:				
good condition	30	58	71	78
Wood or forest land:				
thin stand, poor cover	45	66	77	83
good cover	25	55	70	77
Open Space (lawns, parks, golf courses, cemeteries, etc.) [2]				
Poor condition (grass cover <50%)	68	79	86	89
Fair condition (grass cover 50% - 75%)	49	69	79	84
Good condition (grass cover >75%)	39	61	74	80
Impervious areas:				
Paved parking lots, roofs, driveways, etc. (excluding right-of-way)	98	98	98	98
Streets and Roads:				
Paved; curbs and storm drains (excluding right-of-way)	98	98	98	98
Paved; open ditches (including right-of-way)	83	89	92	93
Gravel (including right-of-way)	76	85	89	91
Dirt (including right-of-way)	72	82	87	89
Developing urban areas and newly graded areas (pervious area only, no vegetation)	77	86	91	94
Raleigh Specific				
Urban districts by zoning:				
O&I (III)	96	97	98	98
O&I (I & II)	98	98	98	98
Buffer Community, Shopping Center, Neighborhood Business, Industrial I & II				
Residential districts by zoning:				
R-4	61	75	83	87
R-6	71	80	87	92
R-10	80	85	90	95
R-20	86	90	93	96
R-30	92	94	96	97

[1] Average runoff condition, and $I_a = 0.2S$

[2] CNs shown are equivalent to those of pasture. Composite CNs may be computed for other combinations of open space cover type. 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. If the impervious area is not connected, the NRCS method has an adjustment to reduce the effect.

For the Raleigh area the design rainfall depths and runoff depths for the 24 hour design storm are as follows:

Table 2-5 Runoff Depth for Raleigh (in)

24 hours storm with Ia = 0.2*S

Frequency, yr	**2**	**5**	**10**	**25**	**50**	**100**
Rainfall, in	**3.60**	**4.56**	**5.28**	**6.48**	**7.2**	**8.0**
CN 60	0.58	1.05	1.47	2.24	2.75	3.33
65	0.81	1.37	1.84	2.71	3.26	3.89
70	1.07	1.72	2.25	3.19	3.79	4.46
75	1.37	2.10	2.68	3.69	4.33	5.04
80	1.72	2.51	3.14	4.22	4.88	5.63
85	2.10	2.96	3.63	4.76	5.44	6.21
90	2.54	3.45	4.15	5.31	6.02	6.81
95	3.04	3.98	4.70	5.89	6.60	7.40
98	3.37	4.32	5.04	6.24	6.96	7.76

The CN is used to determine the initial abstraction, Ia, in Table 2-6. Ia/P is then computed using Figure 2.6.

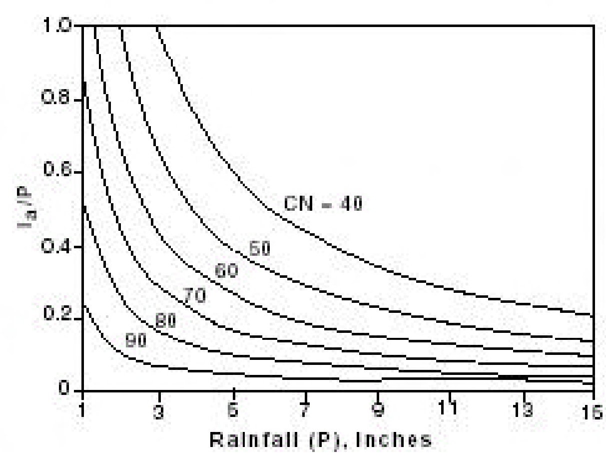
Table 2-6 Ia Values for Runoff Curve Numbers

(NRCS 210-VI-TR-55, Second Ed., June 1986, pg. 4-1)

Curve No.	Ia (in)	Curve No.	Ia (in)
40	3.000	70	0.857
41	2.878	71	0.817
42	2.762	72	0.778
43	2.651	73	0.740
44	2.545	74	0.703
45	2.444	75	0.667
46	2.348	76	0.632
47	2.255	77	0.597
48	2.167	78	0.564
49	2.082	79	0.532
50	2.000	80	0.500
51	1.922	81	0.469
52	1.846	82	0.439
53	1.774	83	0.410
54	1.704	84	0.381
55	1.636	85	0.353
56	1.571	86	0.326
57	1.509	87	0.299
58	1.448	88	0.273
59	1.390	89	0.247
60	1.333	90	0.222
61	1.279	91	0.198
62	1.226	92	0.174
63	1.175	93	0.151
64	1.125	94	0.128
65	1.077	95	0.105
66	1.030	96	0.083
67	0.985	97	0.062
68	0.941	98	0.041
69	0.899		

Figure 2.5 Variation of Ia/P for P and CN

(NRCS 210-VI-TR-55, Second Ed., June 1986, Pg. 4-1)



2.3.2 Travel Time

For rural applications it is common to describe the watershed as being either flat (0 to 3%), moderate (3 to 8%), or steep (>8%) in slope. In such situations the peak discharge curves found at the end of this chapter may be used with the interpolating factors given. For highly urbanized watersheds, however, these generalizations may not be appropriate. The NRCS suggests another method for computing the peak discharge based on a calculated travel time within the watershed.

The time of concentration can be broken into three types of flow, sheet flow, shallow concentrated flow, and channel flow (or pipe flow). Sheet flow is assumed to be no longer than a few hundred feet and can be described by Manning's kinematic solution:

Equation 2.7 Time of Concentration

$$Tc = \frac{0.007\,(n\,L)^{0.8}}{P2^{0.5}\,S^{0.4}}$$

Where: Tc = Travel time (hours)
N = Manning roughness coefficient
L = Flow length (ft)
P2 = 2-yr 24 hour rainfall (in)
S= Ground slope (ft/ft)

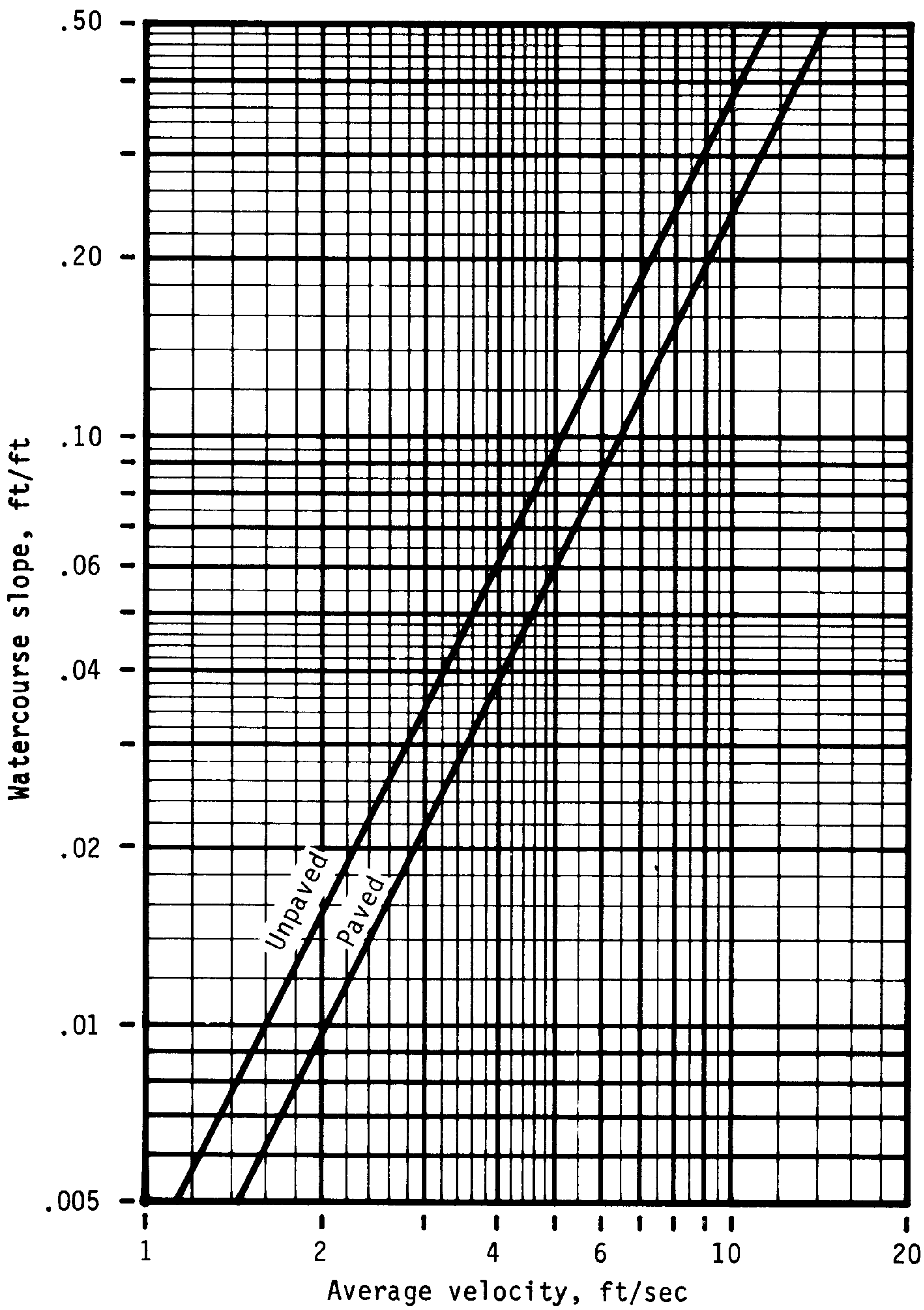
Table 2-7 Manning's "n" Value for Sheet Flow

(Source: North Carolina Erosion and Sediment Control Planning and Design Manual)

Description	"n"
Smooth surfaces:	
Concrete, asphalt	0.011
Bare soil, gravel	0.011
Sparse grasses	0.150
Dense grasses	0.240
Bermuda grass	0.410
Woods, light underbrush	0.40
Dense underbrush	0.80

Shallow concentrated flow travel time is best estimated by calculating the average flow velocity from the figure on the following page. The travel time is estimated as the average flow velocity multiplied by the flow length.

Figure 2.6 Average Velocity
(Source: NRCS TR-55 Urban Hydrology for Small Watersheds, Second Edition, June 1986)



At the point where a defined channel or pipe system begins, the flow velocity can be estimated by the Manning equation. For open channels the equation has the form:

Equation 2.8 Open Channel Velocity

$$V = \frac{1.49\ R^{0.667}\ S^{0.5}}{n}$$

where: V = Average flow velocity (fps)
R = Hydraulic radius (ft)
S = Channel slope (ft/ft)
N = Manning's roughness coefficient

Equation 2.9 Hydraulic Radius

$$R = A/P$$

Where: A = Cross-sectional area (sq. ft)
P = Wetted perimeter (ft)

For pipe systems the flow velocity can be estimated by the Manning equation as well. Assuming the pipe is circular and is flowing just full, the equation simplifies to the form:

Equation 2.10 Flow in a Pipe

$$V = \frac{0.59\ D^{0.667}\ S^{0.5}}{n}$$

Where: D = pipe diameter (ft)
Other variables are as defined previously

2.3.3 Peak Discharge

The peak discharge equation used by the Natural Resources Conservation Services has the form:

Equation 2.11 Peak Discharge

$$Qp = Qu\ A\ Q\ Fp$$

Where: Qp = Peak discharge (cfs)
Qu = Unit peak discharge found from Figure 2.7 (csm/in)
A = Drainage area (sq mi)
Q = Runoff depth (in)
Fp = Pond and swamp adjustment factor from Table 2.8

Table 2.8. Swamp Correction Factors

Percentage of pond or swamp areas	Fp
0	1.00
0.2	0.97
1.0	0.87
3.0	0.87
5.0	0.72

Note that swamp correction factors should only be used if the area will remain in place. If there is a possibility if the area being re-graded in the future, no correction should be made. In these rare situations, the correction factors will be reviewed by the City staff.

Example:

New residential development, R4 zoning
Total DA = site area = 38 ac
Impervious area = 13.8 ac
Soils: 60% C
40% B
Predevelopment condition: pasture
Calculate the peak discharge for the 10-year storm.

1) Calculate CN – use Table 2.4
CNpre = 60%(74) + 40%(61) = 68.8
CNpost = 60%(83) + 40%(75) = 79.8

2) Calculate time of travel, tc

$$\mathbf{Tc = \frac{0.007\ (n\ L)^{0.8}}{P2^{0.5}\ S^{0.4}}}$$

n = 0.240 for dense grasses, from Table 2.5
P2 = 3.6 in
S = 0.02 ft/ft for sheet flow area; 0.04 ft/ft for shallow concentrated flow
L = 300' sheet flow; 450' shallow concentrated flow

Sheet flow tc = 0.54 hr

Shallow concentrated flow, tc':
From Table 2.6, find the average velocity = 3.2 fps

$$tc' = \frac{L}{3600 \times V} = 0.04 \text{ hr}$$

tc_{total} = 0.04 hr + 0.54 hr = 0.58 hr

3) Calculate peak discharge for the 10-year storm

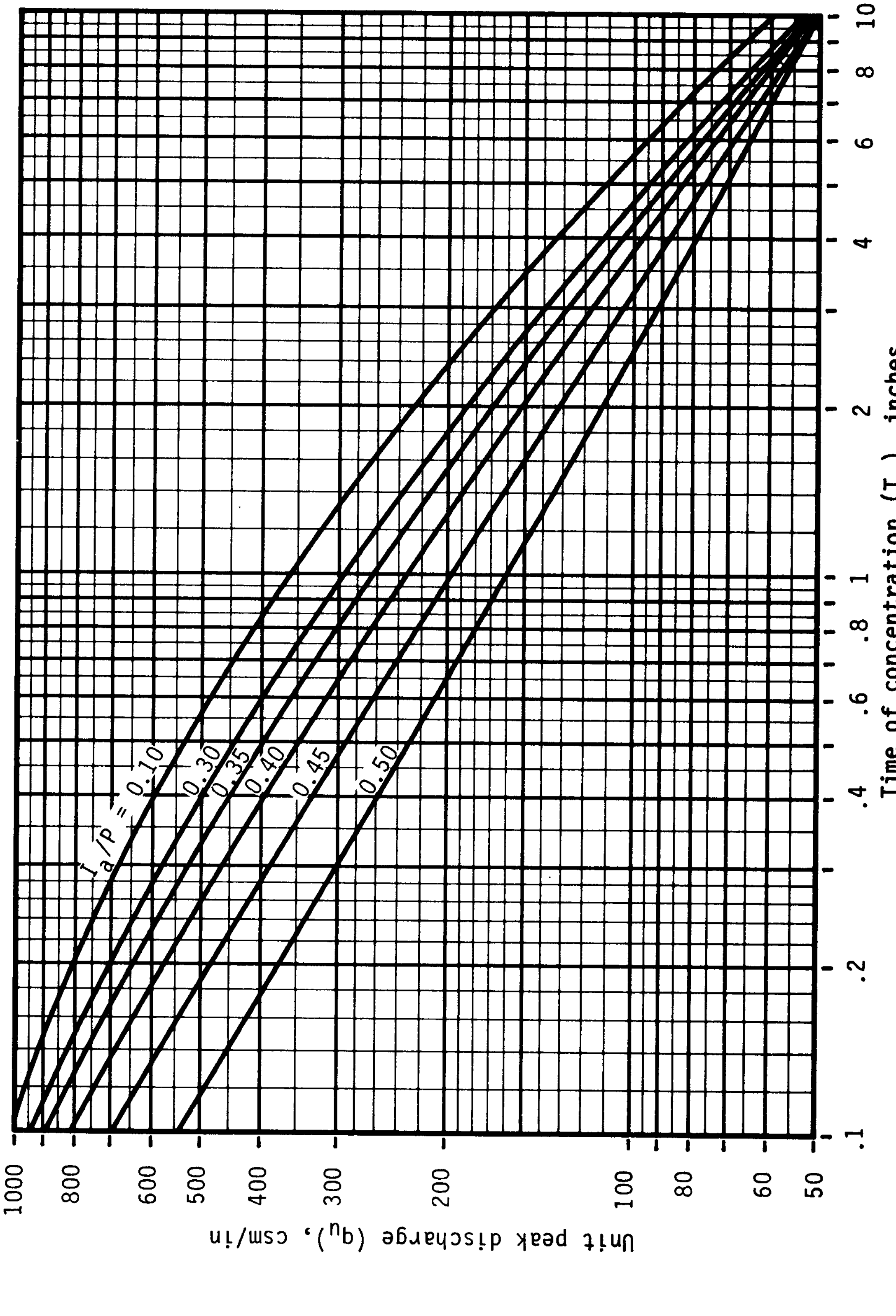
Qp = Qu A Q Fp

Find Ia/P to determine Qu

Ia = 0.5064 interpolating from Table 2.6
P = 5.28 in
Ia/P = 0.5064/5.28 = 0.10
Qu = Unit peak discharge from Figure 2.7 = 490 cfs
A = 38ac x 1 ac/640 mi^2 = 0.06 mi^2
Q10 = 3.12 interpolating from Table 2.5
Fp = 1, since there are no swamp areas within the drainage area

Qp = 490 cfs x 0.06 mi2 x 3.12 in x 1 = 91.7 cfs

Figure 2.7 NRCS Type II Unit Peak Discharge Graph
(Source: NRCS TR-55 Urban Hydrology for Small Watersheds, Second Edition, June 1986)



2.4 Other Methods

In general, any hydrologic model that is appropriate for the drainage area in question will be considered acceptable by the City staff. Numerous models have been developed for the analysis of larger watersheds. The U.S. Army Corps of Engineers HEC-1 model and SCS TR-20 model are but a few. Both models are Fortran programs that have been adapted to microcomputers from older mainframe systems. Consequently they have numerous unique anomalies that may prove difficult to the first time user.

As stated previously, a distributed element models hydrologic routine must be applicable to the smallest as well as the largest sub-watershed in the drainage area. On occasion it maybe helpful to formulate a new model specifically designed for a given watershed. This can easily be accomplished in the spreadsheet format with routing done on a time by time basis. This allows for the same type of variable scenario but with a model specific to that watershed. If a computer program other than the industry standards listed above is used, adequate documentation and source codes must be provided for the staff to review.

2.5 Impoundment Design

City standards may require that some form of impoundment, either detention or retention, be used for new developments. The type and size of facility required will usually depend on the size of the proposed development, its impact on the downstream watercourse and whether or not downstream water quality is of primary concern. If areas immediately downstream of the proposed development are not sensitive to increases in runoff but areas further downstream are sensitive, the City will prefer a regional impoundment facility. If areas immediately downstream of the proposed development are sensitive to any increases in runoff, an on-site impoundment will be required. It is the intention of the City to control stormwater problems resulting from urbanization and lessen some existing flooding problems through the placement of impoundments as close to the problem area as possible. Therefore, the City's impoundment requirement will not apply to every new development but will be a combination of large and small facilities. If on-site facilities are not required, a fee determined by the Inspections Department may be required in lieu of any impoundment facility.

The design of impoundment facilities may be quite simple, as is the case with some small on-site facilities. Larger facilities, however, can be quite complex. This manual is intended to serve as a reference to the designer by providing general guidelines and techniques for analysis. Complex designs should only be undertaken by professionals with a thorough knowledge of impoundments. All impoundment designs and their associated calculations should be sealed by a Professional Engineer registered in North Carolina.

2.5.1 Discharge Limitations

When an impoundment is required, the peak discharge rate after development must not exceed that specified by the particular requirement for every point of discharge from the site. Please note that the exemption for sites having less than 15% impervious area applies to the entire site, not each individual outfall, and must include existing as well as new impervious surfaces. For example, a wooded site has a 2-year discharge of 100 cfs. After development, the site has a peak discharge rate of 200 cfs. If the site is required to control to the pre-developed rate, then the impoundment facility must therefore be designed to limit the discharge to 100 cfs instead of the 200 cfs that would result after development. The 10-year design peak may also be similarly impounded

2.5.2 Required Storage Volume

The quantity of water that must be detained in order to adequately reduce the peak discharge is referred to as the Required Storage Volume. This is the volume that must be available in the facility without exceeding the maximum permissible release rate. Although the required volume can only be found by routing the design storm through the proposed facility, for smaller volumes (less than 20,000 cf), it is adequate to estimate the volume by subtracting the permissible outflow from the basin from the peak inflow for the critical storm duration.

Equation 2.12 Storage Volume

S = (Qp - MPRR)*Tp

Where: S = Estimated storage volume (cf)

Qp	= Peak inflow (cfs)
MPRR	= Maximum permissible release rate (cfs)
Tp	= Time to peak (seconds)

This may also be a good initial estimate for larger basins. Note that it is only an estimate and should be verified by routing the design storm through the proposed facility.

2.5.3 Types of Storage

In general, the type of storage device selected depends on the quantity of water to be stored and the associated cost of storage. Guidelines for each are included herein. The selection of the type of storage used is up the individual owner or engineer. Although all of the following types of facilities will work, some will present more of a maintenance problem.

2.5.3.1 Dry Basins

Detention basins should be designed such that the primary outlet devices restrict the flow and allow water to pond in a safe contained fashion. A properly designed emergency spillway should be provided capable of passing the 100-year storm if the drainage area is greater than 25 acres. Side slopes should be no steeper than 2:1 but if vegetative groundcover is to be used, flatter slopes are highly recommended. The basin should be constructed to insure positive drainage. This will reduce the risk of mosquito problems and reduce maintenance costs. In larger basins, a concrete low flow swale is recommended since vegetation may be difficult to maintain with frequent flow through the basin.

2.5.3.2 Wet Basins

The same basic standards apply to wet basins as to dry impoundment facilities. Outlet devices should be appropriately sized and an emergency spillway provided. Because of their added benefit to water quality, wet basins are highly encouraged by the City staff. Although they may be impractical for smaller areas, their use in larger drainage areas can provide improved water quality and an attractive, aesthetic component to the development. Hydrologic modeling of the wet basin is similar to that of other basins with only some minor changes to the stage-storage curve.

Careful consideration should be given to the frequency of inflow and nutrient levels in the influent when deciding whether or not to use a wet impoundment basin. Low flows and high nutrient levels may result in the eutrophication of the pond and subsequently high maintenance costs.

2.5.3.3 *Parking Lot Storage*

For on-site detention where topography or space is a problem, parking lot storage may be an option. Naturally, not much water can be stored in a parking lot. Therefore it should be considered for only small sites with little or no off site drainage entering the parking lot. The depth of storage should be limited to 8 inches and if possible should be restricted to a remote portion of the parking lot. Storage may not inundate handicap spaces or the primary access to the site. Some form of emergency overflow should be provided to pass the 100-year storm for drainage areas over 25 acres, usually by overtopping the curb. Since small outlet devices are required for parking lot storage, weirs are preferred to orifices. In general, parking lot storage creates more of a maintenance problem than any other type of on-site storage and should be used only when other facilities are impractical.

2.5.3.4 *Pipe Storage*

When space is severely limited on the site pipe storage may be an option. Oversized pipes with a restricted outlet can provide storage but usually only at a very high cost. Access to the pipe and outlet device must be provided for adequate maintenance. Debris control should be a prime consideration in designing pipe storage since the restricting outlet device is generally much smaller than the storage or inflow pipes.

2.5.4 Outlet Devices

The following sections are a general description of some common outlet devices used in impoundment facilities. Other devices are available. Because controlling multiple design storms may be required, some rather imaginative outlet devices may result. To the extent possible, outlet devices should be kept simple. This may require and optimal design for one storm frequency and an over design for the other storm event.

2.5.4.1 *Orifices*

The discharge through an orifice can be described by an energy balance analysis. Assuming the upstream velocity is negligible (i.e. a reservoir) and the water surfaces both upstream and downstream are free surfaces, the energy balance can be simplified to what is referred to as the orifice equation.

Equation 2.13 Orifice Equation

$$Q = Cd\ A\ (2gh)$$

Where: Q = Discharge (cfs)

A = Cross-sectional area of the orifice (sq ft.)

G = Gravitational acceleration

H = Driving head to the centroid of the orifice (where H > D/2)

Cd = Coefficient of discharge (usually 0.50-0.70)

The orifice equation is only appropriate when the headwater depth is above the top of the orifice (HW>D). When the flow through the orifice is lower than the top of the orifice other forms of analysis such as a modified Weir Equation are required. For manual computations of discharge the charts used for the inlet control may also be helpful. These charts are similar to the orifice equation but were developed using empirical data. In many cases they include discharges for depths as low as half the orifice diameter (HW/D = 0.5).

The most common problem encountered with this limitation of the orifice equation is in routing a detention facility. In the early stages of a storm, the depth of water stored may be below the top of the orifice, resulting in an error in the routing. When the discharge occurring under these circumstances is smaller relative to the entire routing, the following approach may be helpful.

1. Calculate the discharge at HW/D = 1.
2. Derive a power curve with exponent greater than 1 (like the weir equation) that matches the known data.
3. Formulate an approximated stage discharge curve.

Example:

Formulate a stage-discharge curve from a depth of 0 to 5 feet for a circular orifice with a diameter of 1 foot and Cd = 0.65.

For HW/D > 1 (i.e. h > 0.5 ft.)

$Q = Cd\ A\ (2gh)^{0.5}$
$A = 0.785$ sq ft.
$Cd = 0.65$

At HW /D = 1, h = 0.5 ft.

$Q = 0.65 * 0.785 * [(2)(32.2)(0.5)]^{0.5} = 2.9$ cfs

When the headwater depth is below the top of the orifice, h < 0.5 the discharge can be described by a power curve, the constant for which can be solved for by substituting in the value at h = 0.5.

The resulting power curve for 3/2 power would then be:

$Qp = K(HW)$

Where $Qp = Q$ (at HW/D = 1) = 2.9 cfs
K = some constant
$2.9 = K(1)^{1.5}$
$K = 2.9$

Therefore:

$Qp = 2.9\ (HW)^{1.5}$

If the depth is less than 1 foot

$Q = 2.9\ \text{depth}^{1.5}$

If the depth is greater than 1 foot

$Q = 0.65 * 0.785 * (2gh)^{0.5}$

Admittedly this method is an approximation for lower depths. When routed, if the depth is only below the top of the orifice for a short period it may be an appropriate estimate. If the depth is below the top of the orifice for a longer period of time, as is the case for some street culverts, this method would not be applicable. In these situations, the methods for analyzing street culverts in Chapter 4 should be considered.

2.5.4.2 Weir Equation

Most weirs used in impoundments will fall into one of two categories; sharp-crested weirs, such as flow over a standpipe, or broad-crested weirs such as emergency overflows in basins. Although considerable research has been conducted in the modeling of weirs, a simple expression can be applied to most weirs used in stormwater impoundments. The equation is usually expressed as:

Equation 2.14 Weir Equation

$$Q = Cw\ L\ H^{1.5}$$

Where:
Q = Discharge (cfs)
Cw = Weir coefficient
L = Length (ft)
H = Height of water above the crest of the weir (ft)

For sharp-crested weirs, Cw is usually taken to be about 3.33. For broad-crested weirs, 3.0 is generally used. Cw is not a true a constant, but rather a function of flow depth and geometry. For horizontal weirs used in storm drainage, these values will usually suffice.

The discharge calculation of compound weirs can usually be estimated by superposition. For example, the total discharge of the compound weir (Qtot) shown below is the sum of the two partial discharges (Qa & Qb).

Equation 2.15 Compound Weir Equation

$$Qtot = Qa + Qb$$

Or

$$Qtot = Cw\ La\ Ha^{1.5} + Cw\ Lb\ Hb^{1.5}$$

2.5.4.3 Riser Barrel Outlets

Riser-barrel outlets act as a combination of several types of outlet devices. At different stages the outlet may behave differently. At shallow depths the riser may act as a weir. As the depth increases the riser may begin to act as an orifice or the barrel may begin to control. The controlling factor will be that with the smallest discharge at a given depth.

Riser as a sharp-crested weir:

Equation 2.16 Sharp Crested Weir

$$Q = Cw\ L\ h^{1.5}$$

Where: L = The circumference of the riser (ft)
h = Head above the top of the riser (ft)
All other terms are as previously defined.

Riser as an orifice:

Equation 2.17 Orifice Equation

$$Q = Cd\ A\ (2gh)^{0.5}$$

Where: A = Cross-sectional area of the riser (sq ft)
All other terms are as previously defined.

Barrel as an orifice:

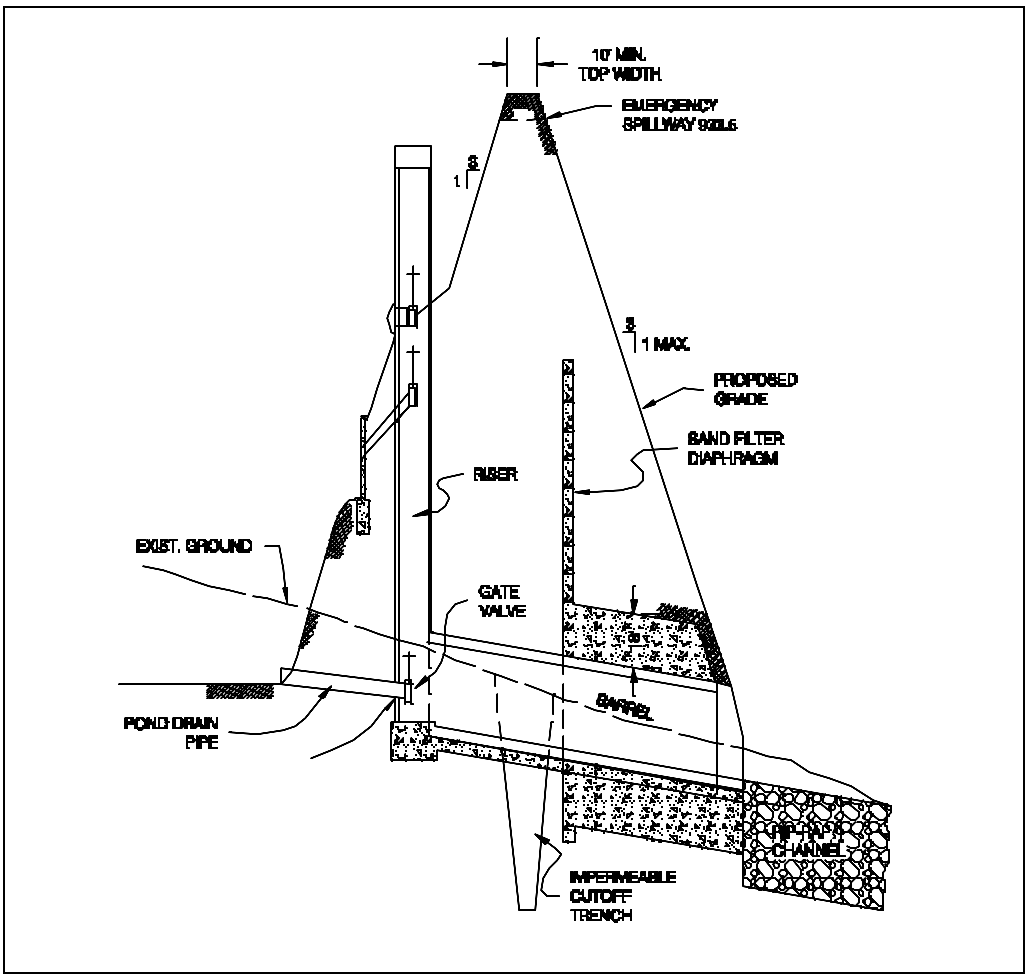
Equation 2.18 Barrel as Orifice

$$\mathbf{Q = Cd\ A\ (2gH)^{0.5}}$$

Where: A = Cross-sectional area of the barrel (sq ft)
H = Head above the centroid of the barrel (ft)

Note that vortex action can and should be eliminated by using an anti-vortex plate or designing a riser-barrel combination incapable of producing a vortex.

Figure 2.8 Cross Section of a Riser Barrel



2.5.5 Routing

Hydrologic routing is an attempt to model the change in storage in a detention facility by comparing inflow and outflow at every point in time. The differential equation of the change in storage has the simplified form:

Equation 2.19 Change in Storage

$$\frac{dS}{dT} = I - O$$

Where:

$\frac{dS}{dT}$ = Change in storage in the basin with respect to time

I = Inflow

O = Outflow

Although many methods have been developed to perform routings, only two are presented here, the Storage Indication Method and a short cut method developed by Dr. H. R. Malcom (North Carolina State University, Department of Civil Engineering). Both require the same basic input elements. A description of the inflow hydrograph, the relationship between stage and discharge, and the relationship between stage and storage are required for either.

2.5.5.1 Inflow Hydrograph Formulation

The nature of impoundment basin routing is such that the inflow to the basin must be described in small time increments. In order to accomplish this, an inflow hydrograph must be formulated for each of the post-development design storms. If one of the more complex hydrologic methods is used to determine discharge from the drainage area, the inflow hydrograph is already available and can be used for the routing. If only the peak discharge has been determined, a hydrograph must be formulated based on that peak.

It is widely accepted that storms in the Raleigh area can generally be described by the SCS type II storm distribution. For the type II storm the volume of runoff can be expressed as a percentage of the total runoff.

Table 2.9 SCS Type II Storm Distribution

T/Tp	Q/Qp	T/Tp	Q/Qp
0.0	0.0	1.3	0.84
0.1	0.015	1.4	0.75
0.2	0.075	1.5	0.66
0.3	0.16	1.6	0.56
0.4	0.28	1.7	0.49
0.5	0.43	1.8	0.42
0.6	0.60	1.9	0.37
0.7	0.77	2.0	0.32
0.8	0.89	2.1	0.28
0.9	0.97	2.2	0.24
1.0	1.00	2.3	0.21
1.1	0.98	2.4	0.18
1.2	0.92	2.5	0.15

(U.S. Bureau of Reclamation, DESIGN OF SMALL DAMS, Denver, Colorado, 1974.)

Therefore, if the volume of runoff is known, the shape of the hydrograph can be calculated. The following method is offered as a reasonable way to estimate the inflow hydrograph (Malcom):

1. Calculate the peak flow for the given design storm (rational method, SCS methods, etc.)
2. Estimate the volume of runoff (Type II 6 hr. storm runoff for example)
3. Given that

Equation 2.20 Runoff Volume

Vol = 1.39 Qp Tp

Where Vol = Volume of runoff from 2 above
Qp = Peak discharge from 1 above
Tp = Time to peak of the hydrograph

Note: Keep consistent units

4. Adopt the shape to be used. For graphical work, the type II coordinates discussed previously may be used. For a more precise calculation, a step function closely approximating the shape may be used (Malcom).

For $0 < t < 1.25tp$

Equation 2.21 Step Function

$$Q = \frac{Qp}{2}\left(1 - \cos\frac{\Pi \times t}{tp}\right) \text{ in radians}$$

For $t > 1.25\ tp$

Equation 2.22 Step Function

$$Q = 4.34Qpe^{-1.3\left(t/tp\right)} \text{ in radians}$$

2.5.5.2 Formulation of the Stage-Storage Function

For routing a storm by the short cut method in a computer program or by hand, it is necessary to formulate an expression for the stage-storage relationship. For routing by hand, a plot of the relationship is adequate. For computer application, the relationship can usually be expressed by a power curve. The simplest way to calculate the volume of storage is to planimeter (or digitize) a topographic map of the basin and calculate using the average end areas method as demonstrated below.

The resulting plot of stage vs. storage may be used for routing by hand or a "best fit" equation of the points may be used. The best fit is usually of the form

Equation 2.23 Stage-Storage Function

$$Storage = K \times Stage^{b}$$

Where
Storage is in ft^3
Stage is in feet
K = 200
b = 3

Therefore, to calculate the stage for the associated storage volume:

$$Stage = K \times Storage^{1/3}$$

Table 2.10 Formulation of Stage Storage Function

Contour (elev)	Stage (ft)	Planimeter Reading (sq in)	Area (sf)	Incremental Volume (cf)	Total Volume (cf)
200	0	0	0	0	0
202	2	0.64	1600	1600	1600
204	4	3.84	9600	11,200	12,800
206	6	8.32	20,800	30,400	43,200
208	8	15.36	38,400	59,200	102,400
210	10	23.68	59,200	97,600	200,000
212	12	38.16	95,400	154,600	345,600
214	14	43.12	107,800	203,200	548,800

2.5.5.3 *Storage Indication Method*

Simplifying the differential equation in the previous section, the change in storage from time "i" to time "j" can be described by the average inflow and outflow over the period.

Equation 2.24 Storage Indication

$$\frac{Sj - Si}{Tj - Ti} = \frac{Ij - Ii}{2} - \frac{Oj - Oi}{2}$$

Where: Si,Sj = Storage at time i and j respectively
Ii,Ij = Inflow at time i and j respectively
Oi,Oj = Outflow at time i and j respectively
Tj - Ti = Change in time

Rearranging, the equation has the more common form:

$$Ii + Ij + \left(\frac{2Si}{Tj - Ti} - Oi\right) = \frac{2Si}{Tj - Ti} + Oj$$

In order to route the hydrograph, it is necessary to plot the relationship between outflow and the right half of the equation above. Since the stage-storage and stage-discharge relationships are known, this can be easily accomplished. The routing is best explained by example.

Example:

For a proposed detention facility the following relationships have been developed.

- The target maximum release rate is 150 cfs.
- The outlet device should be a single 42 inch RCP.
- The stage can be described by a power curve with K = 200 and b = 3 and should not exceed 12 feet. The power curve was developed previously.
- The inflow hydrograph has a peak discharge of 400 cfs occurring at 20 minutes and has the shape of the step function discussed in Section 2.5.5.1.

Solution:

1. Select a time increment for the routing. Since the routing is actually a numerical integration of the differential equation, the time increment should be small enough to accurately reflect the area under the hydrograph. Therefore, for any routing technique, the time step should be no longer than 10 percent of the time to peak. The time step for this routing will be 2 minutes.
2. Develop the inflow hydrograph. Using the shape of the step function in Section 2.5.5.1, the inflow hydrograph is listed in column 2 of the routing worksheet.
3. Develop the stage-discharge function. The outlet device will be a single 42 inch RCP with no tailwater constraints. The discharge function can then be developed as discussed in Section 2.5.4.1. The Cd is taken to be 0.65.

$$Q = Cd\ A\ (2gh)^{1/2}$$

Where: Q = Discharge (cfs)
Cd = 0.65
A = Cross-sectional area = 9.62 sq ft
H = Driving head = stage - d/2 = stage - 1.75 ft

Therefore :

For HW < 42 inches

$$Q = 10.14 * stage^{3/2}$$

For HW > 42 inches

$$Q = 0.65 * 9.62 * (2 * 32.2 * (stage - 1.75))^{1/2}$$

4. The stage-storage curve used is the one developed in Section 2.5.5.2.

$$Stage = (Storage/K)^{1/b}$$
$$= (Storage/200)^{1/3}$$

5. Plot the indication curve.

Stage	Discharge	Storage	O + 2S /(Tj-Ti)
0	0	0	0
1	14	200	17
2	25	1600	52
3	56	5400	146
4	75	12800	289
5	90	25000	507
6	103	43200	823
7	115	68600	1258
8	125	102400	1832
9	135	145800	2565
10	144	200000	3477
11	153	266200	4589
12	161	345600	5291
13	168	439400	7492
14	176	548800	9322
15	183	675000	11433

6. Routing

Time (min)	Ii (cfs)	Ij (cfs)	2S/(Tj-Ti) - O (cfs)	2S/(Tj-Ti) + O (cfs)	O (cfs)
0	0	10	0		0
2	10	38	10	10	0
4	38	82	58	58	0
6	82	138	46	178	66
8	138	200	122	266	72
10	200	262	288	460	86
12	262	318	550	750	100
14	318	362	906	1130	112
16	362	390	1344	1586	121
18	390	400	1840	2096	128
20	400	390	2358	2630	136
22	390	362	2866	3148	141
24	362	320	3328	3618	145
26	320	281	3712	4010	149
28	281	247	4011	4313	151
30	247	217	4233	4539	153
32	217	190	4389	4697	154
34	190	167	4486	4796	155
36	167	147	4533	4843	155
38	147	129	4537	4847	155
40	129	113	4503	4813	155
42	113	99	4437	4745	154
44	99	87	4341	4649	154
46	87	77	4221	4527	153
48	77	67	4081	4385	152
50	67	59	3925	4225	150
52	59	52	3753	4051	149
54	52	46	3570	3864	147
56	46	40	3376	3668	146
58	40	35	3174	3462	144
60	35			3249	142

The peak discharge will be about 155 cfs occurring about 38 minutes into the storm. Note that there is some minor instability in the first few routing steps. As long as these errors do not account for much change in storage volume they can be ignored.

2.5.5.4 Short Cut Routing Method

An alternative to the storage indication method of routing is a method that may lend itself to computer spreadsheet application. (Developed by Dr. H.R. Malcom, NCSU, Dept. of Civil Engineering) The components required for the short cut method are similar to those of storage-indication method. The short cut is an incremental tabular application of the same differential equation but simplified to the form:

Si = (Ii-Oi)*(Ti-Tj)

Where: Si = Incremental change in storage at time i (sec)

	Ii	= Inflow at time i (cfs)
	Oi	= Outflow at time i (cfs)
	Tj-Ti	= Time step (sec)

The short-cut method may not be as intuitively satisfying as other methods since the outflow at any time is based on the storage volume prior to that time step. The method does however lend itself to spreadsheet application and with sufficiently short time steps provides reasonable results. Here again the method is best explained by example.

Example:

Repeat the same example used in Section 2.5.5.3 for the storage indication method using the short-cut method.

Solution:

1. The time step and inflow hydrograph are the same as those used in the previous example and appear in columns 1 and 2 respectively.
2. The stage-storage curve is the same as the one developed previously:

$$Stage = \left(\frac{Storage}{2}\right)^{\frac{1}{3}}$$

3. The stage-discharge function is the same as the one developed previously:

For Stage < 3.5 feet

$$Q = 10.14 * stage^{3/2}$$

For Stage > 3.5 feet

$$Q = 0.65 * 9.62 * (2 * 32.2 * (stage-1.75))^{1/2}$$

4. Routing

Time (min)	Inflow (cfs)	Storage (cu ft)	Stage (ft)	Outflow (cfs)
0	0	0	0	0
2	10	0	0.00	0
4	38	1175	1.80	12
6	82	4356	2.79	51
8	138	8103	3.43	65
10	200	16873	4.39	81
12	262	31100	5.38	96
14	318	51052	6.34	108
16	362	76257	7.25	118
18	390	105554	8.08	126
20	400	137233	8.82	133
22	390	169227	9.46	139
24	362	199339	9.99	144
26	320	225477	10.41	148
28	281	246204	10.72	150
30	247	261931	10.94	152
32	217	273320	11.10	153
34	190	280942	11.20	154
36	167	285290	11.26	155
38	147	286797	11.28	155
40	129	285837	11.26	155
42	113	282743	11.22	154
44	99	277801	11.16	154
46	87	271268	11.07	153
48	77	263368	10.96	152
50	67	254297	10.83	151
52	59	244232	10.69	150
54	52	233327	10.53	149
56	46	221722	10.35	147
58	40	209538	10.16	145
60	35	196887	9.95	144

Note that the peak discharge found by the short-cut method is the same as that found by the storage-indication method and that the peak occurs at about the same time.

2.5.6 Ten Percent Rule

The "ten percent rule" may be used to determine the downstream extent of design considerations for new detention. This rule recognizes that in addition to controlling the peak discharge from the outlet works, storage facilities change the timing of the entire outflow hydrograph. Where required, channel routing calculations must proceed downstream to a confluence point where the drainage area being analyzed represents ten percent or less of the total drainage area. At this point, the effect of the hydrograph routed through the proposed storage facility on the downstream hydrograph is assessed and shown not to have detrimental effects on downstream hydrographs. If detrimental impacts are suspected, then backwater calculations and determination of flood elevations for the areas impacted by increased flows, if any, must be prepared.

Chapter 3 Stormwater Quality Management

3 Stormwater Quality Management

Urban stormwater runoff contains many types and forms of constituents. When compared to stormwater runoff from pre-development conditions, higher concentrations and some contaminants that are not naturally present in surface runoff from undeveloped local lands are found. Runoff from undeveloped watersheds contains sediment particles, oxygen-demanding compounds, nutrients, metals, and other constituents. Once developed, constituent loads increase because surface runoff volumes increase and the sources of many of these pollutants also increase. Supplemental applications of compounds, such as fertilizers, also tend to increase the availability of some pollutants to stormwater runoff.

Runoff water quality in urban areas can be extremely detrimental to local habitat. Paved surfaces and standing water bodies for stormwater management control elevate the temperature of water entering streams. Chemicals in standing water and ponds are oxidized, resulting in depressed levels of dissolved oxygen. Increased runoff volumes and rates creates scour and deposition damage to in-stream habitat. Activities in urbanized areas, such as vehicular traffic, deposit pollutants such as heavy metals and oil & grease on paved surfaces where they easily wash off into the streams.

3.1 Stormwater Quality Control Requirements

Federal regulations require that pollutants in runoff must be controlled to the maximum extent practicable. The State of North Carolina has implemented water quality control requirements based on the control of total nitrogen in stormwater runoff that apply to new development citywide. Total maximum daily loads (TMDLs) may be assigned for specific pollutants such as fecal coliforms. These requirements are being adopted as the basis of the city's stormwater runoff quality management program. It should be noted that control of sediment is also required for construction site runoff citywide and that specific restrictions and performance based criteria for controlling total suspended solids in stormwater runoff exist in the water supply watershed protection area.

This section of the manual explains how the loads for total nitrogen for new development sites are to be calculated, which BMPs are acceptable for management of different levels of total nitrogen export, and what procedures must be followed when preparing a BMP plan for city staff to review.

3.1.1 Total Nitrogen Standards and Requirements for Stormwater Runoff

The State of North Carolina adopted a comprehensive strategy for the control of total nitrogen in stormwater runoff for new developments when it adopted the Neuse River Basin Nutrient Management Strategy. The goal of the strategy is to achieve a 30 percent nitrogen reduction from each controllable and quantifiable source of nitrogen in the basin. These sources are: wastewater treatment, urban stormwater, agriculture and nutrient application. The Neuse strategy also includes a rule to protect riparian buffers in order to maintain their existing nitrogen removal capabilities. The Neuse Strategy also includes the control of peak runoff to predevelopment rates for the purpose of protecting streams and the nutrient reduction functions of riparian buffers from accelerated erosion.

New development must meet the 30 percent reduction goal by implementing planning considerations and best management practices. The rule imposes a nitrogen load limit on new development of 3.6 pounds per acre per year (lb/ac/yr). Nitrogen load from new development that exceeds this performance standard must be reduced to 3.6 lb/ac/yr by the use of BMP's, by payment of an offset fee to the Wetlands Restoration Fund, or a combination of these two. The offset payment alternative may only be used when the nitrogen load from residential development is 6.0 lb/ac/yr or less and when the load for non-residential development is 10.0 lb/ac/yr or less. When the nitrogen load exceeds these levels, then BMP's must be used to bring the nitrogen load down to 6.0 lb/ac/yr or 10 lb/ac/yr for residential and commercial development respectively.

Classification of residential or commercial use shall be based on City of Raleigh zoning categories. However, permitted non-residential uses in residential districts such as schools, churches, day cares, etc. may use the higher limits established for non-residential development."

3.1.2 Computing Total Nitrogen Loads in Stormwater Runoff

In order to determine what, if any, level of water quality control must be provided, the nitrogen export from the development must be calculated. This export will be calculated in pounds per acre per year (lbs/ac/yr). Two methodologies that may be used to make this calculation are presented below.

Method 1 is intended for residential developments where lots are shown but the actual footprint of buildings, driveways, and other impervious features are not yet known or shown on site plans. This method does not require calculation of the area of building footprints. The impervious surface resulting from building footprints is estimated based on typical impervious areas associated with a given lot size. This method is shown in Exhibit 3-1.

Exhibit 3-1 Method 1 for Quantifying TN Export from Residential Developments when Building and Driveway Footprints are Not Shown

Step 1: Determine area for each type of land use and enter in Column (2).

Step 2: Total the areas for each type of land use and enter at the bottom of Column (2).

Step 3: Determine the TN export coefficient associated with right-of-way using Figure 3-1.

Step 4: Determine the TN export coefficient associated with lots using Figure 3-2.

Step 5: Multiply the areas in Column (2) by the TN export coefficients in Column (3) and enter in Column (4).

Step 6: Total the TN exports for each type of land use and enter at the bottom of Column (4).

Step 7: Determine the export coefficient for site by dividing the total TN export from uses at the bottom of Column (4) by the total area at the bottom of Column (2) and enter it in at the bottom of Column (5).

(1) Type of Land Cover	**(2) Site Area (Acres)**	**(3) TN Export Coeff. (lbs/ac/yr)**	**(4) TN Export by Land Use (lbs/yr)**	**(5) TN Export From Site (lbs/ac/yr)**
Permanently preserved undisturbed open space (forest, unmown meadow)		0.6		
Permanently preserved managed open space (grass, landscaping, etc.)		1.2		
Right-of-way (read TN export from Figure 1)				
Lots (read TN export from Figure 2)				
Total				
Average for Site				

Figure 3-1 Total Nitrogen Export from Right-of-Way

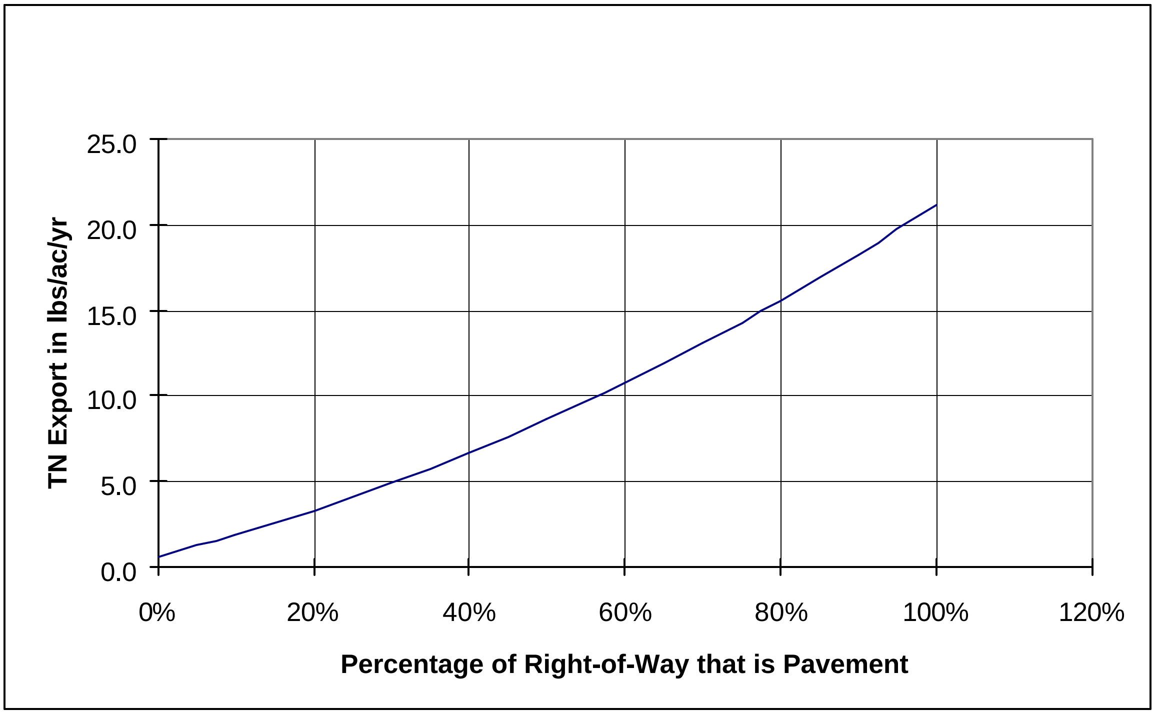
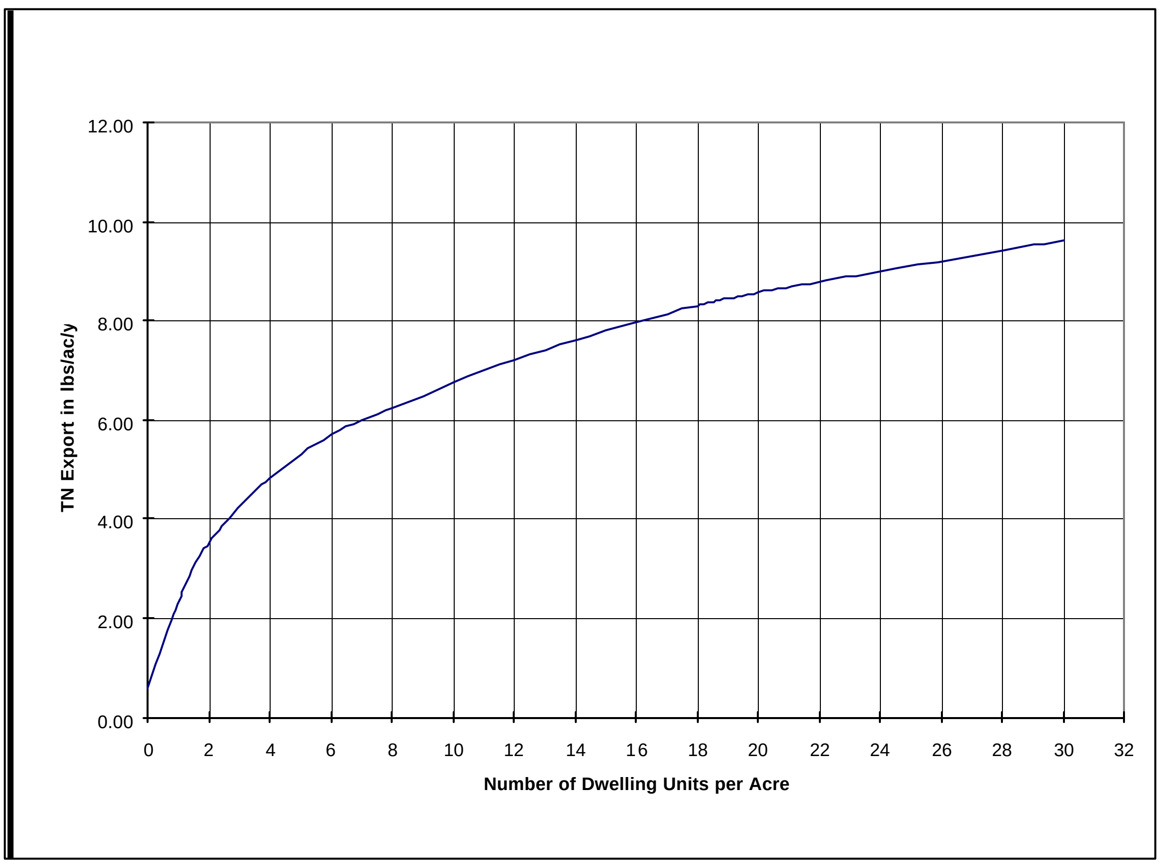


Figure 3-2 Total Nitrogen Export from Lots



Method 2 is for residential, commercial and industrial developments when the entire footprint of the roads, parking lots, buildings and any other built-upon area is shown on the site plans. This method is simpler and more accurate since it does not require estimating the impervious surface. Method 2 is shown in Exhibit 3-2.

Exhibit 3-2 Method 2 for Quantifying TN Export from Residential / Industrial / Commercial Developments when Footprints of all Impervious Surfaces are Shown

Step 1: Determine area for each type of land use and enter in Column (2).

Step 2: Total the areas for each type of land use and enter at the bottom of Column (2).

Step 3: Multiply the areas in Column (2) by the TN export coefficients in Column (3) and enter in Column (4).

Step 4: Total the TN exports for each type of land use and enter at the bottom of Column (4).

Step 5: Determine the export coefficient for site by dividing the total TN export from uses at the bottom of Column (4) by the total area at the bottom of Column (2) and enter the result at the bottom of Column (5).

(1) Type of Land Cover	(2) Site Area (Acres)	(3) TN Export Coeff. (lbs/ac/yr)	(4) TN Export by Land Use (lbs/yr)	(5) TN Export From Site (lbs/ac/yr)
Permanently preserved undisturbed open space (forest, unmown meadow)		0.6		
Permanently preserved managed open space (grass, landscaping, etc.)		1.2		
Impervious surfaces (roads, parking lots, driveways, roofs, paved storage areas, etc.)		21.2		
Total				
Average for Site				

3.1.2.1 Apportioning Method for Computing Nitrogen Loading from Expansions and Other New Development

When an expansion is proposed for an existing site or when it is desired to determine the appropriate credit for existing open space as part of a larger development, an apportioning technique may be utilized. This method assigns appropriate portions of open space to new development when the existing and future amounts of impervious and pervious surfaces are known. This technique is best described through example.

Example:

Given:
10 acre site with existing, approved 5 acres of impervious surface
Expansion to add an additional 3 acres of impervious surface for a total of 8 acres of impervious and 2 acres pervious.

Using the apportioning system, the existing development gets credit for 5/8 of the 2 acres of the remaining open space for 1.25 acres pervious. The new development gets 3/8 credit of the 2 acres for 0.75 acres. Assuming the open space is managed, the nitrogen load calculations would look like this:

(5 acre x 21.2 lb/acre) + (1.25 acre x 1.2 lb/acre)
= 106 lb + 1.5 lb = 107.5 lb (existing)

(3 acre x 21.2 lb/acre) + (0.75 acre x 1.2 lb/acre)
= 63.6 + 0.9 lb = 64.5 lb (new)

Total TN export = 107.5 + 64.5 = 172 lb

Since a portion of this may have been addressed as part of the original development, the expansion is responsible for mitigating any difference.

3.1.3 Best Management Practices

Based on national and regional studies, most BMPs are capable of removing only 20 to 40 percent of total nitrogen on a consistent basis. All BMPs require regular maintenance and some have varying performance depending on soil type and the season. Crucial design criteria for the BMPs identified below are presented in Section 3.2.

***3.1.3.1* Approved BMPs**

The following BMPs have been approved by both the State and the City of Raleigh and can be utilized for reducing nitrogen in stormwater runoff from new developments:

- Vegetated filter strips
- Wet detention ponds
- Bioretention facilities
- Water quality swales
- Riparian buffers
- Sand filters
- Constructed wetlands

The State's Neuse Stormwater Team estimated total nitrogen removal rates for various BMPs by conducting a literature search of studies performed on BMPs. The total nitrogen BMP removal rates based on current literature studies are provided below.

Table 3-1 Total Nitrogen and Total Solids Removal Rates by BMP

BMP Type	TN Removal Rate	TSS Removal Rates
Vegetated filters w/spreader	25%	35%
Wet detention ponds	25%	85%
Bioretention facilities	25%	85%
Water quality swales	30%	35%
Riparian buffers	30%	80%
Sand filters	35%	85%
Constructed wetlands	40%	85%

If more than one BMP is installed in series on a development, then the removal rate shall be determined through serial rather than additive calculations. *For example*, if a wet detention pond discharges through a riparian buffer, then the removal rate shall be estimated to be 47.5 percent.

1. The pond removes 25 percent of the nitrogen and discharges 75 percent to the buffer.
2. The buffer then removes 30 percent of the nitrogen that discharged from the pond, which is 22.5 percent.
3. The sum of 25 and 22.5 is 47.5. *(NOT 25 plus 30, or 55 percent)*

Alternative or innovative BMPs, such as manufactured BMPs, will require that complete documentation on the BMP, including specifications, performance, and maintenance requirements, be submitted to the city along with the pollutant reduction computations.

3.1.4 BMP Maintenance Requirements

When selecting and siting BMPs, the designer should carefully consider long term maintenance. The City of Raleigh requires the landowner to maintain BMPs unless the City has entered into a maintenance agreement with the landowner. Section 10-9028 of the Raleigh City Code requires the property owner to submit annual inspection reports, sealed by a qualified registered professional, on the anniversary of the project's approval. Specific maintenance requirements for each type of BMP are noted in each BMP section.

Section 10-9025(b) of the Raleigh City Code requires that a maintenance manual and budget accompany all BMPs. Types of maintenance required, frequency, and a budget for annual costs, major repairs and replacement must be included. In addition, all BMPs must be located on lots containing improvements equal to or greater in value than the replacement value of the BMP.

Where a BMP serves more than one lot or is located off-site, Section 10-9027(b) & (c) of the City Code requires a maintenance covenant and an escrow agreement.

3.1.5 Selecting the Appropriate BMPs

The rule requires that all new developments achieve a nitrogen export of less than or equal to 3.6 pounds per acre per year. If the development contributes greater than 3.6 lbs/ac/yr of nitrogen, then the options shown in the table below are available based on whether the development is zoned residential or non-residential.

Once the BMP has been chosen, the BMP must be designed based upon the requirements in Sections 1.4.1 and 2.5.1.

Exhibit 3-3 Nitrogen Export Reduction Options

Residential Use	Commercial / Industrial Use
If the computed export is less than 6.0 lbs/ac/yr, then the owner may either: 1. Install BMPs to remove enough nitrogen to bring the development down to 3.6 lbs/ac/yr. 2. Pay a one-time offset payment of $330/lb to bring the nitrogen down to the 3.6 lbs/ac/yr. 3. Do a combination of BMPs and offset payment to achieve a 3.6 lbs/ac/yr export.	If the computed export is less than 10.0 lbs/ac/yr, then the owner may either: 1. Install BMPs to remove enough nitrogen to bring the development down to 3.6 lbs/ac/yr. 2. Pay a one-time offset payment of $330/lb to bring the nitrogen down to the 3.6 lbs/ac/yr. 3. Do a combination of BMPs and offset payment to achieve a 3.6 lbs/ac/yr export.
If the computed export is greater than 6.0 lbs/ac/yr, then the owner must use on-site BMPs to bring the development's export down to 6.0 lbs/ac/yr. Then, the owner may use one of the three options above to achieve the reduction between 6.0 and 3.6 lbs/ac/yr.	If the computed export is greater than 10.0 lbs/ac/yr, then the owner must use on-site BMPs to bring the development's export down to 10.0 lbs/ac/yr. Then, the owner may use one of the three options above to achieve the reduction between 10.0 and 3.6 lbs/ac/yr.

The table above discusses the option of using offset fees to meet the nitrogen export levels set for new development activities. These offset fees go to the NC Wetlands Restoration Program (WRP). The WRP will utilize these fees in accordance with the WRP's Basinwide Wetlands and Riparian Restoration plans. It is the policy of the WRP to utilize the funds where they are generated to the maximum extent possible as long as they can obtain the cooperation of the local government and landowner(s).

3.1.6 Designing for the First Flush, WQ_v

As defined in Section 1.4.1, the first flush runoff or water quality volume, WQ_v, is the runoff from the first inch of precipitation, which is generally the portion of the runoff with the highest concentrations of most conventional nonpoint source runoff contaminants. There are several types of BMPs that are designed to treat the WQ_v and to allow the runoff from excess rainfall to bypass the treatment. The two most common of these BMPs are bioretention areas and sand filters. First flush BMPs typically treat the runoff from small drainage areas and are often times considered source controls.

Based on estimates in relevant literature, including Watershed Protection Techniques and the final report of the Nationwide Urban Runoff Program, BMPs designed to treat the WQ_v in the Raleigh area would treat the runoff of up to 75% of the storms annually because the total storm depth of those storms is less than one inch. Furthermore, the highest pollution concentrations of the remaining 25 percent of the storms occurs during the first inch of rainfall, so the bulk of the remaining nonpoint source load is treated also. Overall, treatment approaches 85 to 90 percent of the potential nonpoint source load in urban stormwater runoff.

The City of Raleigh encourages the use of these types of runoff controls for areas of high imperviousness and also for areas of moderate imperviousness. Bioretention (with or without underdrains) in residential areas can provide all of the pollutant removal necessary to comply with the Neuse River Nutrient Management Strategy. These bioretention areas, often referred to as rain gardens, can be located in an attempt to take maximum advantage of the permeability of the soils and the location of trees and shrubs.

Bioretention areas and perimeter sand filters can be used to provide pre-treatment for discharge to buffer areas or in some cases may be used as mitigation for riparian buffers. According to the literature and the statistics of the DWQ, bioretention areas provide at least as great a removal of total nitrogen as do buffers and sand filters and approximately thirty percent more effective than buffers.

When underdrains are use in conjunction with bioretention areas the outflow of the underdrain can discharge directly to a stream channel or to another BMP, such as a grassed swale or detention area.

3.2 BMP Design Criteria

In the following sections specific design criteria are provided for a selection of structural BMPs that have been shown to be effective elements of a total nitrogen control program. These BMPs include wet ponds, vegetative filter strips, bio-retention areas, vegetated swales, buffers, sand filters, and artificial wetlands.

The BMP sections are separated into required standards; recommended standards; maintenance recommendations; performance standards; and, if necessary, landscaping requirements. The required standards must be followed and pertain to the design of the measure. The recommended standards *should* be followed and typically pertain to siting and landscaping the measure to make it more acceptable to the surrounding environment. Maintenance requirements are also addressed in each BMP section. Inspection frequency and maintenance needs are specific for each type of BMP. Performance standards set the percent reduction that each BMP will achieve *if designed, installed and maintained* as outlined in this manual. And finally, several water quality BMPs are dependent on a detailed landscaping plan. Detailed landscaping information is provided for those BMPs in the respective sections.

3.2.1 Wet Ponds

Wet ponds have a permanent pool between storm events, and can be designed for the WQ_v as well as to meet detention requirements. Wet retention / detention ponds should be designed with a minimum detention time of 48 hours. If the retention pond is to be designed for only water quality purposes, then the pond should be designed to capture the water quality volumes as specified by the City.

Required Standards

- Design for the WQ_v
- Minimum drainage area of 10 acres.
- Minimum length to width ratio of 3:1 (preferably expanding outward toward the outlet). Irregular shorelines for larger ponds also provide visual variety.
- Inlet and outlet located to maximize flow length. Use baffles if short-circuiting cannot be prevented with inlet-outlet placement. Long flow paths and irregular shapes are recommended.
- Minimum depth of permanent pool 2 to 3 feet, maximum depth of 6 to 8 feet. Average depth should be 3 to 7 feet, based on the criteria of the State of North Carolina as shown in the table below.
- Design for full development upstream of control.
- Side slopes should be no greater than 3:1 if mowed.
- Riprap protection should be provided (or other suitable erosion control means) for the outlet and all inlet structures into the pond. Individual boulders or baffle plates can work for this.
- Anti-seep collars or filter diaphragms should be provided on barrel of principal spillway.
- If reinforced concrete pipe is used for the principal spillway, O-ring gaskets (ASTM C361) should be used to create watertight joints.
- One (1) foot minimum freeboard above peak stage for top of

embankment.

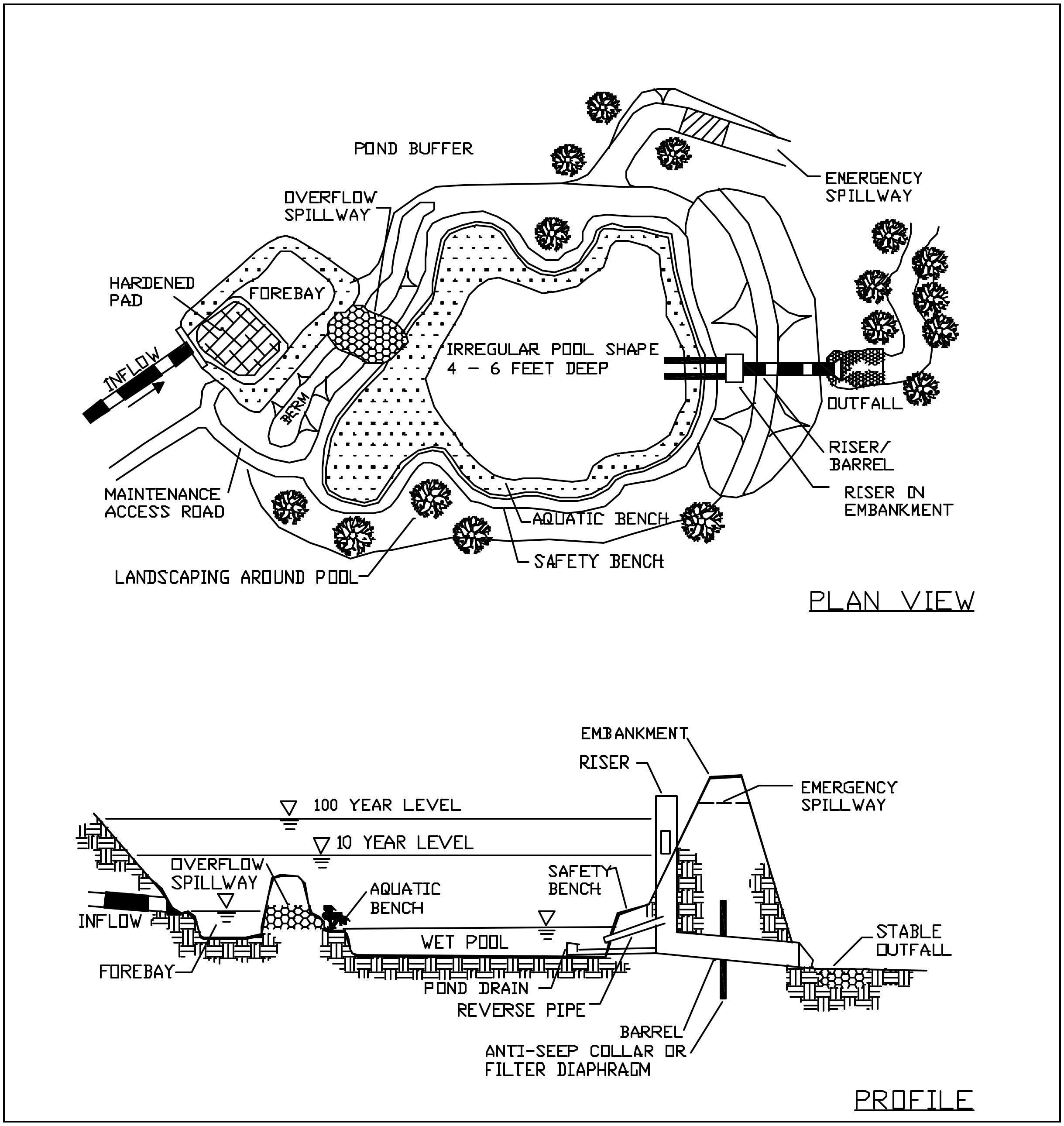
- An emergency drain (i.e. sluice gate, drawdown pipe) capable of draining the pond within 24 hours should be installed.
- Design the emergency spillway to pass the 100-year storm event.
- Provide bypass for storms greater than the design storms.
- Provide trash racks, filters, hoods or other debris control provided on riser.
- Principal spillway/riser should incorporate anti-floatation, anti-vortex, and trash-rack designs.
- Provide for maintenance access, a minimum of 25 feet wide. Provisions should be made for vehicle access at a 4:1 slope.
- Provide benchmark for sediment removal.
- Install emergency drain to allow draw-down within 24 hours.

Recommended Standards

- Design the pond for multi-objective use, such as amenities or flood control.
- Landscaping management of buffer as meadow.
- Design for multi-function as flood control and extended detention.
- Minimum length to width ratio of 3:1 to 4:1 (preferably wedge shaped).
- Use reinforced concrete instead of corrugated metal for pipes.
- Larger ponds must have a sediment forebay (often designed for 5 to 15 percent of total volume). The forebay should have separate drain for de-watering. Grass biofilters can be used to filter sediment for smaller ponds.
- Consider artificial mixing for small sheltered ponds.
- Impervious soil boundary to prevent drawdown may be needed.
- Shallow marsh area around fringe 25 to 50 percent of area (including aquatic vegetation) should be established.
- A safety bench with a minimum width of 10 feet should be provided around the permanent pool.
- The perimeter of all deep permanent pool areas (four feet or greater in depth) should be surrounded by two benches with a combined minimum width of 15 feet:
- Include a safety bench that extends outward from the normal water edge to the toe of the pond side slope. The maximum slope of the safety bench should be 6%.
- Include an aquatic bench that extends inward from the normal shoreline and has a maximum depth of eighteen inches below the normal pool water surface elevation. An aquatic bench is not required in forebays.
- Minimum 25 foot wide buffer around pool.
- On-site disposal areas, for two sediment removal cycles, should be provided and protected from runoff.
- An oil and grease skimmer may be needed for sites with high production of pollutants.

Figure 3-3 Wet (Retention) Pond

Source: Controlling Urban Runoff



Operation and Maintenance Recommendations

- Sediment should be removed when 20% of storage volume of the facility is filled (design storage volume must account for volume lost to sediment storage).
- No woody vegetation should be allowed on the embankment without special designs.
- Vegetation over 18 inches high should be cut unless it is part of planned landscaping.
- Debris should be removed from blocking inlet and outlet structures and from areas of potential clogging.
- The outlet control should be kept structurally sound, free from erosion, and functioning as designed.
- Periodic removal of dead vegetation should be accomplished.
- Inspection requirements should be outlined in the maintenance manual.
- The site should be inspected and debris removed after every major storm.
- All special maintenance responsibilities will be listed in the maintenance manual.
- Mow embankment and side slopes at least twice a year.
- Consider chemical treatment by alum if algal blooms are a problem.

Performance Standards

- Wet ponds are very effective in removal of both the soluble and particulate fractions of pollution.
- Average annual pollutant removal capability of wet ponds is as follows.

Pollutant	**0.5 Inch Per Impervious Acre**
BOD	20-40%
TSS	60-80%
Total Nitrogen	20-40%
Total Phosphorus	40-60%
Metals	20-40%

Table 3-2 SA/DA Ratios for Permanent Pool Sizing to Achieve 85% TSS Reduction

IMPERVIOUS PERCENT	PERMANENT POOL DEPTHS IN FEET 3.0	3.5	4.0	4.5	5.0	5.5	6.0
10%	0.59	0.54	0.49	0.47	0.43	0.39	0.35
20%	0.97	0.88	0.79	0.75	0.70	0.65	0.59
30%	1.34	1.20	1.08	1.03	0.97	0.91	0.85
40%	1.73	1.58	1.43	1.36	1.25	1.14	1.03
50%	2.00	1.82	1.73	1.64	1.50	1.40	1.33
60%	2.39	2.09	2.03	1.87	1.66	1.56	1.51
70%	2.75	2.44	2.27	2.12	1.96	1.87	1.79

Sediment Forebay

Sediment forebays, or equivalent upstream pretreatment, should be included. Following are the general criteria to be used for sediment forebay design.

- The forebay should consist of a separate cell, formed by an acceptable barrier.
- The forebay should be sized to contain 0.1 inches of runoff per impervious acre of contributing drainage. The forebay storage volume counts toward the total water quality storage requirements.
- Exit velocities from the forebay should be non-erosive.
- Direct maintenance access for appropriate equipment should be provided to the forebay.
- The bottom of the forebay may be hardened (e.g., using concrete, paver blocks, etc.) to make sediment removal easier.
- A fixed vertical sediment depth maker should be installed in the forebay to measure sediment deposition over time.
- Sediment removal in forebay should occur when 50% of the total capacity has been lost.

3.2.2 Level Spreaders

Required Specifications

- Drainage area into spreader should be restricted so that maximum flow will not exceed 30 cfs.
- Channel grade for the last 20 feet of the dike or diversion entering the level spreader should be less than or equal to 1% and designed to provide a smooth transition into spreader.
- Grade of a level spreader should be 0%.
- Depth of a level spreader as measured from the lip should be at least 6 inches.
- Appropriate length, width, and depth of flow spreader should be selected from the following table.

Design Flow (cfs)	Min. Entrance Width (ft)	Min. Depth (ft)	Min. End Width (ft)	Min. Length (ft)
0 - 10	10	0.5	3	10
10 - 20	16	0.6	3	20
20 - 30	24	0.7	3	30

- The level spreader lip should be constructed on undisturbed soil (not fill material) to uniform height and zero grade over length of the spreader.
- The released runoff to the outlet should be on undisturbed stabilized areas in sheet flow and not allowed to re-concentrate below the structure.
- Spreader lip to be protected with erosion resistant material, such as fiberglass matting or a rigid non-erodible material for higher flows, to prevent erosion and allow vegetation to be established.

Operation and Maintenance Recommendations

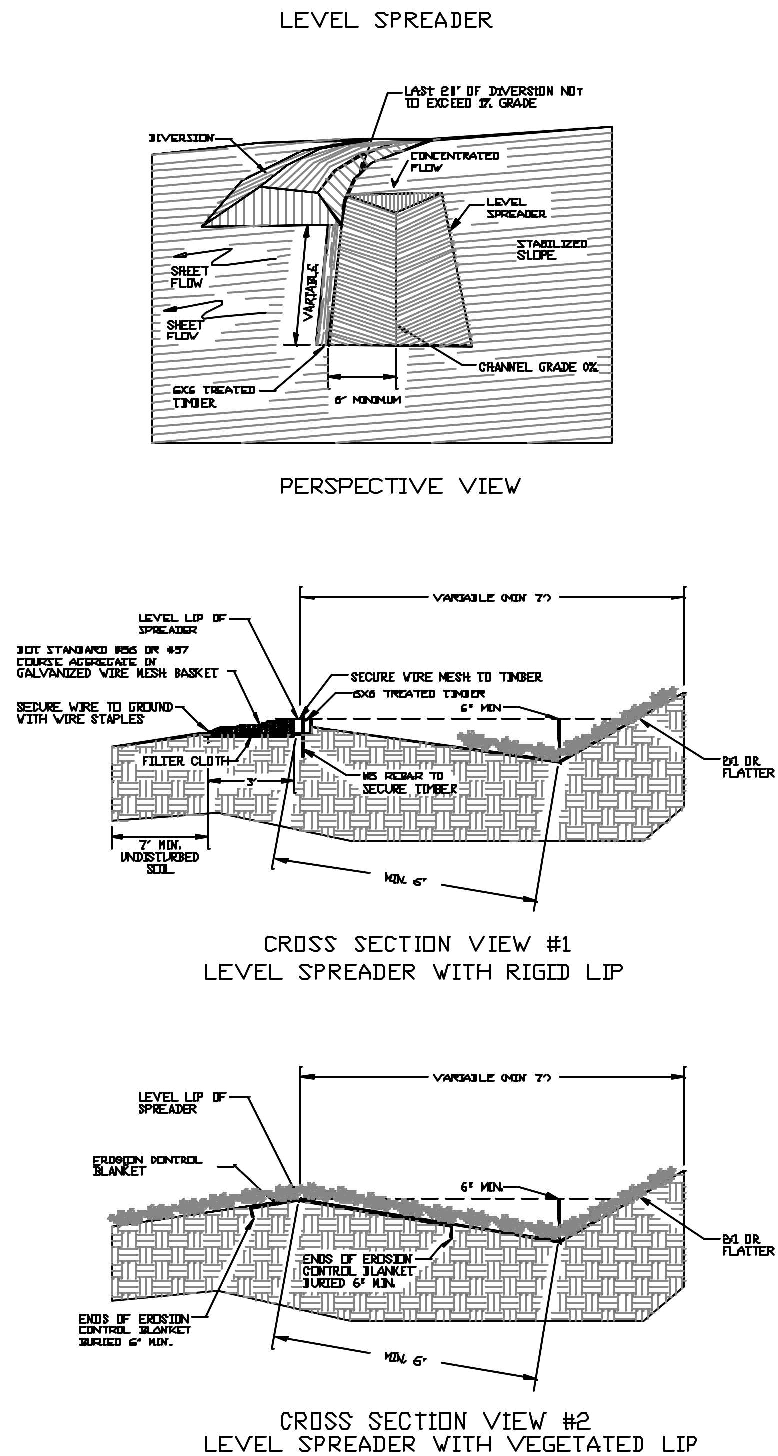
- A maintenance manual is required for each facility. The maintenance manual should require the owner of the level spreader to periodically clean the structure.
- Level spreader should be inspected after every rainfall until vegetation is established, and needed repairs made promptly.
- After area is stabilized, inspections should be made quarterly.
- Vegetation should be kept in a healthy, vigorous condition.
- Level spreaders must be maintained in a manner to achieve sheet flow.

Performance Standards

Pollutant removal rates are not available for level spreaders installed alone. Level spreaders are only considered water quality BMPs when installed with other BMPs such as filter strips or buffers.

Figure 3-4 Level Spreader

Source: North Carolina Erosion And Sediment Control Planning And Design Manual



3.2.3 Vegetated Filter Strips

Required Specifications

- Filter strips must accept stormwater runoff as overland sheet flow in order to effectively filter suspended materials out of the overland flow.
- In order to function properly, the strip should be at least as wide as the flow path entering the filter, and flow entering a filter strip must be spread relatively uniformly over the width of the strip.
- In order to insure sheet flow through the filter strips, the maximum allowed flow is limited to 0.1 cfs per linear foot of filter strip.
- All disturbed areas should be vegetated immediately after construction.
- Filter strip width to be a minimum of 20 feet.
- The use of filter strips and flow spreaders should be limited to drainage areas of 10 acres or less with the optimal size being less than 5 acres.
- The use of filter strips to treat parking lot runoff or street runoff should incorporate a level spreading device such as a shallow stone filled trench or slotted parking blocks.
- Capacity of the spreader and/or filter strip length (perpendicular to flow) should be determined by estimating the volume of flow that is diverted to the spreader for water quality control.
- Slope of the filter strip from a level spreader should not exceed 10 percent.

Recommended Specifications

- Top edge of filter strip should directly abut the contributing impervious area and follow the same elevation contour line.
- Runoff water containing high sediment loads to be treated in a sediment trapping device before release in a flow spreader.
- Wooded filter strips are preferred to gravel strips.

Operation and Maintenance Recommendations

- A maintenance manual is required for each facility. The maintenance manual should require the owner of the filter strip to periodically clean the structure.
- Inspect the filter strip after every rainfall until vegetation is established, and make repairs promptly.
- After area is stabilized, inspections should be made quarterly.
- Vegetation should be kept in a healthy, vigorous condition.
- Filter strips should be maintained in a manner to achieve sheet flow.

Performance Standards

- The removal of soluble pollutants is low because the degree of infiltration provided is generally very small.
- Removals of nutrients and oxygen demand decrease as the amount of clay in the soil increases.
- Minimal pollutant removal. This design primarily removes the coarser suspended particles in runoff by the lowering of runoff velocities.
- Pollutant removal enhanced by mild slopes, minimal mowing/ maintenance, sustaining natural cover if possible.
- Long term estimated removal of pollutants is as follows:

- *20-Foot Wide Grassed Filter Strip:*
  - Pollutant removal efficiencies for 20-foot wide filter strips, designed, installed and maintained as required in this manual, are:

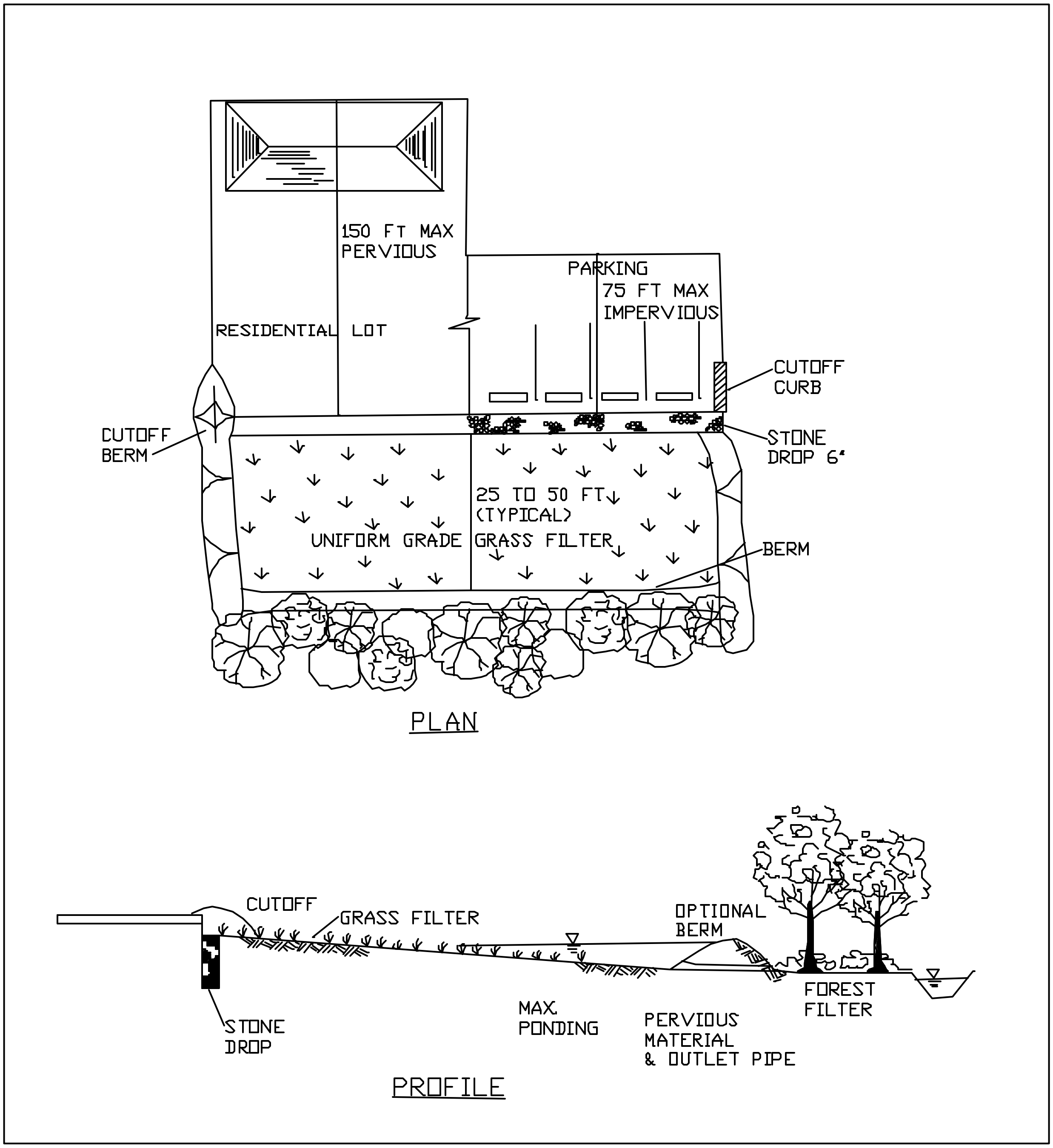
Pollutant	**Removal Rate (%)**
BOD	10
TSS	35
Total Nitrogen	10
Total Phosphorus	10
Metals	30

- *100-Foot Wide Grassed Filter Strip:*
  - Maximal natural pollutant removal. This design removes both fine and coarse suspended particles in runoff by lowering runoff velocities over a significant length of flow path.
  - Pollutant removal can be enhanced by mild slopes, minimal mowing / maintenance, sustaining natural cover if possible.
  - Long term estimated removal of pollutants when filter strips are 100-foot wide, and designed installed and maintained as required in this manual, is as follows:

Pollutant	**Removal Rate (%)**
BOD	70
TSS	90
Total Nitrogen	50
Total Phosphorus	50
Metals	90

Figure 3-5 Schematic of A Filter Strip

Source: Controlling Urban Runoff



3.2.4 Bioretention

Bioretention areas, or rain gardens, are structural stormwater controls that capture and temporarily store the WQ_v using soils and vegetation in landscaped areas to remove pollutants from stormwater runoff. Bioretention areas are engineered facilities in which runoff is conveyed as sheet flow to the "treatment area," consisting of a grass buffer strip, ponding area, organic or mulch layer, planting soil, and vegetation. An optional sand bed can be included in the design to provide aeration and drainage of the planting soil. The filtered runoff is typically collected and returned to the conveyance system, though it can be exfiltrated into the surrounding soil in areas with porous soils.

There are numerous design applications, both on- and off-line, for bioretention areas. These include use on single family residential lots (rain gardens), as off-line facilities adjacent to parking lots, along highways and road drainage swales, within larger landscaped pervious areas, and as landscaped islands in impervious or high-density environments.

Figure 3-6 illustrates a bioretention area. Bioretention areas consist of:

1. Grass filter strip between the contributing drainage area and the ponding area;

2. Ponding areas containing vegetation with a planting soil bed,

3. Organic/mulch layer, and

4. Gravel and perforated pipe underdrain system to collect runoff that has filtered through the soil layers (bioretention areas can optionally be designed to infiltrate into the soil).

Optional design components include:

1. Sand filter layer to spread flow, filter runoff and aid in aeration and drainage of the planting soil;

2. Stone diaphragm at the beginning of the grass filter strip to reduce velocities and spread flow into the grass filter;

3. Inflow diversion or an overflow structure.

<u>Required Specifications</u>

- Design for the WQ_v. Bioretention areas are primarily water quality control structures. However, they can provide a limited amount of storm water quantity control.
- They are suitable for many types of development, including single family residential. However, they are not suitable for regional stormwater control.
- The contributing drainage area must be 5 acres or less, though 0.5 to 2 acres is preferred.
- The minimum size for facility is 200 ft^2, with a length to width ratio of 2:1. Slope of the site can be no more than 6%.
- Planting soil filter bed is sized using a Darcy's Law equation

with a filter bed drain time of 48 hours and a coefficient of permeability (k) of 0.5 ft/day. The planting soil bed must be at least 4 feet deep. Planting soils must be sandy loam, loamy sand or loam texture with a clay content rating from 10 to 25 percent. The soil must have an infiltration rate of at least 0.5 inches per hour and a pH between 5.5 and 6.5. In addition, the planting soil should have a 1.5 to 3 percent organic content and a maximum 500-ppm concentration of soluble salts.

- The maximum ponding depth in bioretention areas is 6 inches.
- Filter strip design for pre-treatment should follow the requirements outlined in Section 3.2.2.
- The mulch layer should consist of 2-4 inches of commercially available fine shredded hardwood mulch or shredded hardwood chips.
- The sand bed should be 12-18 inches thick. Sand should be clean and have less than15% silt or clay content.
- Pea gravel for the diaphragm and curtain, where used, should be ASTM D 448 size No. 6 (1/8" to ¼").
- The underdrain collection system should be equipped with a 6 inch perforated PVC pipe in an 8-inch gravel layer. The pipe should have 3/8-inch perforations, spaced on 6-inch centers with a minimum of 4 holes per row. The pipe is spaced at a maximum of 10 feet on center, and a minimum grade of 0.5% must be maintained. A permeable filter fabric is placed between the gravel layer and the planting soil bed.
- The required elevation difference needed from the inflow to the outflow is 5 feet.
- The depth to the water table from the bottom of the bioretention facility to the high water table should be a minimum of 2 feet.
- Runoff captured by facility must be sheet flow to prevent erosion of the organic or mulch layer.
- Bioretention areas are designed for intermittent flow and to drain and aerate between rainfall events. Sites with continuous flow from groundwater, sump pumps or other areas should be avoided.
- An overflow structure and a non-erosive overflow channel must be provided to safely pass the flow from the bioretention area that exceeds the storage capacity to a stabilized downstream area. The high flow structure within the bioretention area can consist of a yard drain catchbasin, with the throat of the catchbasin inlet typically 6 inches above the elevation of the shallow ponding area.

Recommended Standards

- Bioretention areas can be incorporated into the landscaping plan as depressed parking lot islands.

Operation and Maintenance Requirements

- A maintenance manual is required for each facility. The maintenance manual should require the owner of the bioretention area to periodically clean the facility and replace dead, dying or diseased plant material.
- Inlets should be inspected for signs of erosion after every significant rainfall.
- Flow spreaders at inlets should be inspected and repaired to insure sheet flow.
- Vegetation should be kept healthy.
- Mulch should be replaced as needed.

Performance Standards

- Bioretention areas designed, constructed and maintained as noted in this manual provide the following pollutant reductions:

Pollutant	Removal Rate (%)
TSS	85%
Total P	29
Total N	49
Metals	61

Landscaping

Landscaping is critical to the performance and function of the bioretention area. A dense and vigorous groundcover should be established over the contributing pervious drainage area before runoff can be diverted into the facility.

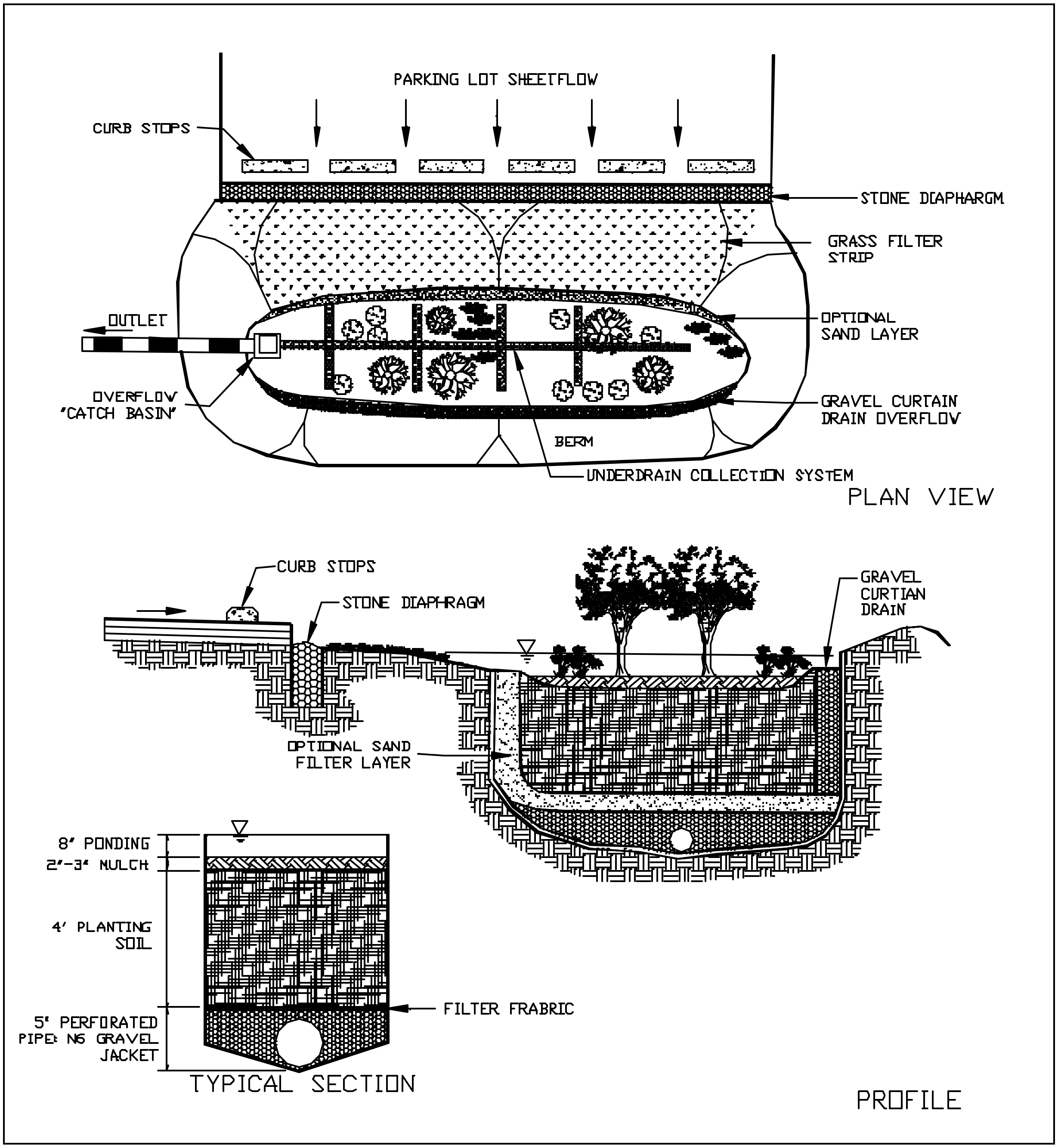
- The bioretention area should be vegetated like a terrestrial forest ecosystem, with a mature tree canopy, subcanopy of understory trees, scrub layer and herbaceous ground cover. Three species of each tree and shrub type should be planted.

- The tree-to-shrub ration should be 2:1 to 3:1. On average, trees should be spaced 8 feet apart. Plants should be placed at regular intervals to replicate a natural forest. Woody vegetation should not be planted at inflow locations.

- After the trees and shrubs are established, the ground cover and mulch should be established.

- Use native plants, selected based upon hardiness and hydric tolerance.

Figure 3-6 Bioretention Area



3.2.5 Water Quality Swales

Water quality swales are also described as biofiltration swales with the major difference being that water quality swales often have check dams where biofiltration swales do not.

Required Specifications

- Design for the WQ_v
- Water quality swales should only convey standing or flowing water following a storm.
- As a water quality BMP, swales should be designed for the water quality volumes specified by the City. If the entire channel design storm is to be accommodated in the swale (e.g., 25-year) then the swale should be designed for this event.
- Limited to peak discharges generally less than 5 to 10 cfs.
- Limited to runoff velocities less than 2.5 ft/s.
- Maximum design flow depth to be 1 foot.
- Swale slopes should be graded as close to zero as drainage will permit.
- Swale slope should not exceed 4 percent (2 percent is preferred).
- Swale cross-section should have side slopes of 3:1 (h:v) or flatter.
- Underlying soils should have a high permeability (fc > 0.5 inches per hour).
- Swale area should be tilled before grass cover is established.
- Dense cover of a water tolerant, erosion resistant grass should be established.
- To obtain credit as a water quality BMP, water quality swales must have a minimum length of 100 feet.

Recommended Specifications

- As a BMP, water quality swales are limited to residential or institutional areas where percentage of impervious area is relatively small.
- Seasonally high water table to be greater than 3 feet below the bottom of the swale.
- Check dams can be installed in swales to promote additional infiltration. Recommended method is to sink a railroad tie halfway into the swale. Riprap stone should be placed on the downstream side to prevent erosion.
- Maximum ponding time behind check dam to be less than 48 hours. Minimum ponding time of 30 minutes is recommended to meet water quality goals.

Operation and Maintenance Recommendations

- A stormwater maintenance manual is required for each facility. The manual should require the owner of the water quality swale to periodically clean the structure.
- Water quality swale should be maintained to keep grass cover dense and vigorous.
- Maintenance should include periodic mowing, occasional spot reseeding, and weed control.

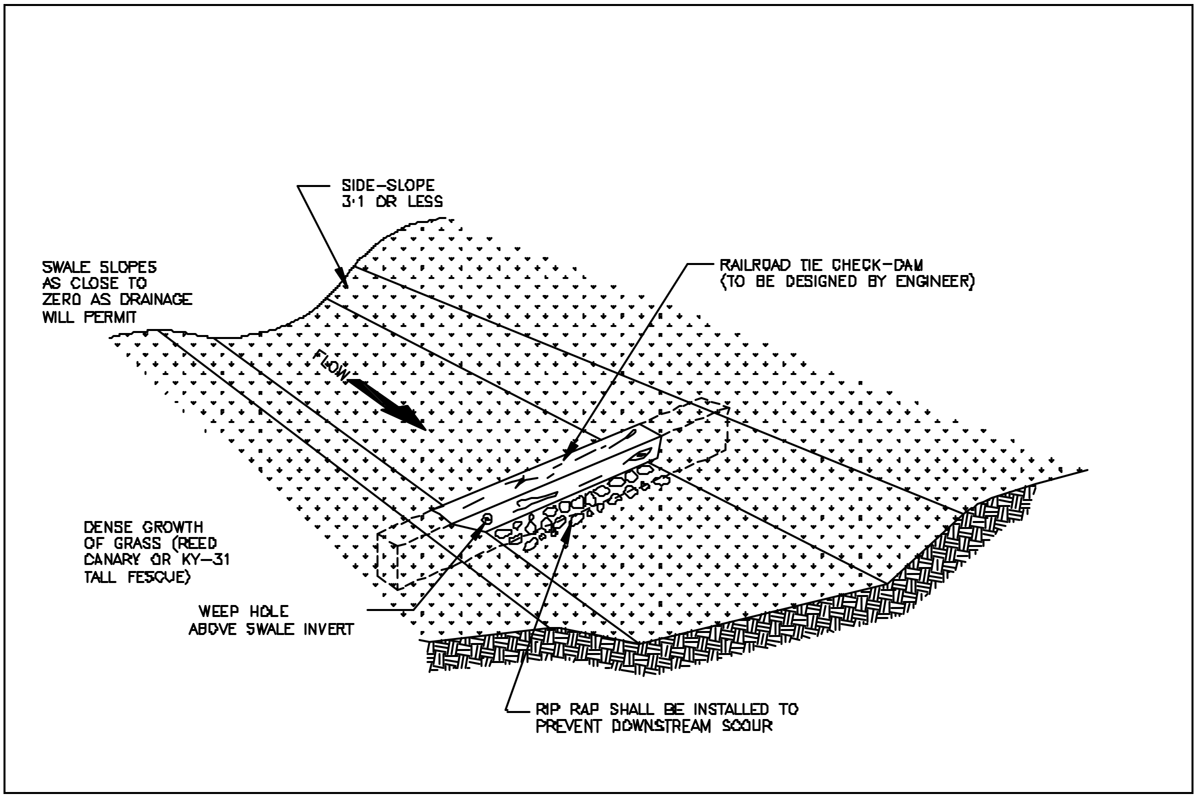
- Swale grasses should never be mowed close to the ground. Grass heights in the 4 to 6 inch range are recommended.
- Fertilization of water quality swale should be done when needed to maintain the health of the grass, with care not to over-apply the fertilizer.

Performance Standards

- Water quality swales provide a water quality benefit by filtering suspended material out of the overland flow. They have little or no value at removing soluble pollutants because the degree of infiltration provided is generally small.
- In order to function optimally, a water quality swale must be in an area where its longitudinal slope is 2% or less. The table below shows the low removal rates for water quality swales on a 5 percent slope. If discharges or velocities are greater than those recommended (greater than 10 cfs or 2.5 ft/s, respectively), the ability of the swale to perform as a water quality BMP is severely impaired.
- The use of check dams in the swale helps to lower the discharge velocity and can, in some cases, allow their beneficial use in situations where the swale slope is greater than recommended.
- Rainfall events of less than 0.25 inches may show increased removals due to the slower velocities in the swales.

Figure 3-7. Schematic of a Water Quality Swale

Source: Controlling Urban Runoff



- *Water Quality Swales on a 5% Slope*
- Long term estimated removal of pollutants is as follows:

Pollutant	**Removal Rate (%)**
BOD	10
TSS	10
Total P	10
Total N	10
Metals	10

- *Grassed Swales on a Slope Less Than 5% With Check Dams*
- Long term estimated removal of pollutants is as follows:

Pollutant	**Removal Rate (%)**
BOD	30
TSS	30
Total P	30
Total N	30
Metals	10

3.2.6 Buffers

The Neuse Rules require maintenance of existing riparian buffers. This section describes how to design a buffer strip for nutrient reduction. Once a buffer has been installed, it may become a regulated riparian buffer.

Required Specifications

- The use of buffers should be limited to drainage areas of 10 acres or less with the optimal size being less than 5 acres.
- Runoff entering the buffer must be sheet flow. See Section 3.2.2 for design specification for level spreaders.
- All disturbed areas should be vegetated immediately after construction.

Recommended Specifications

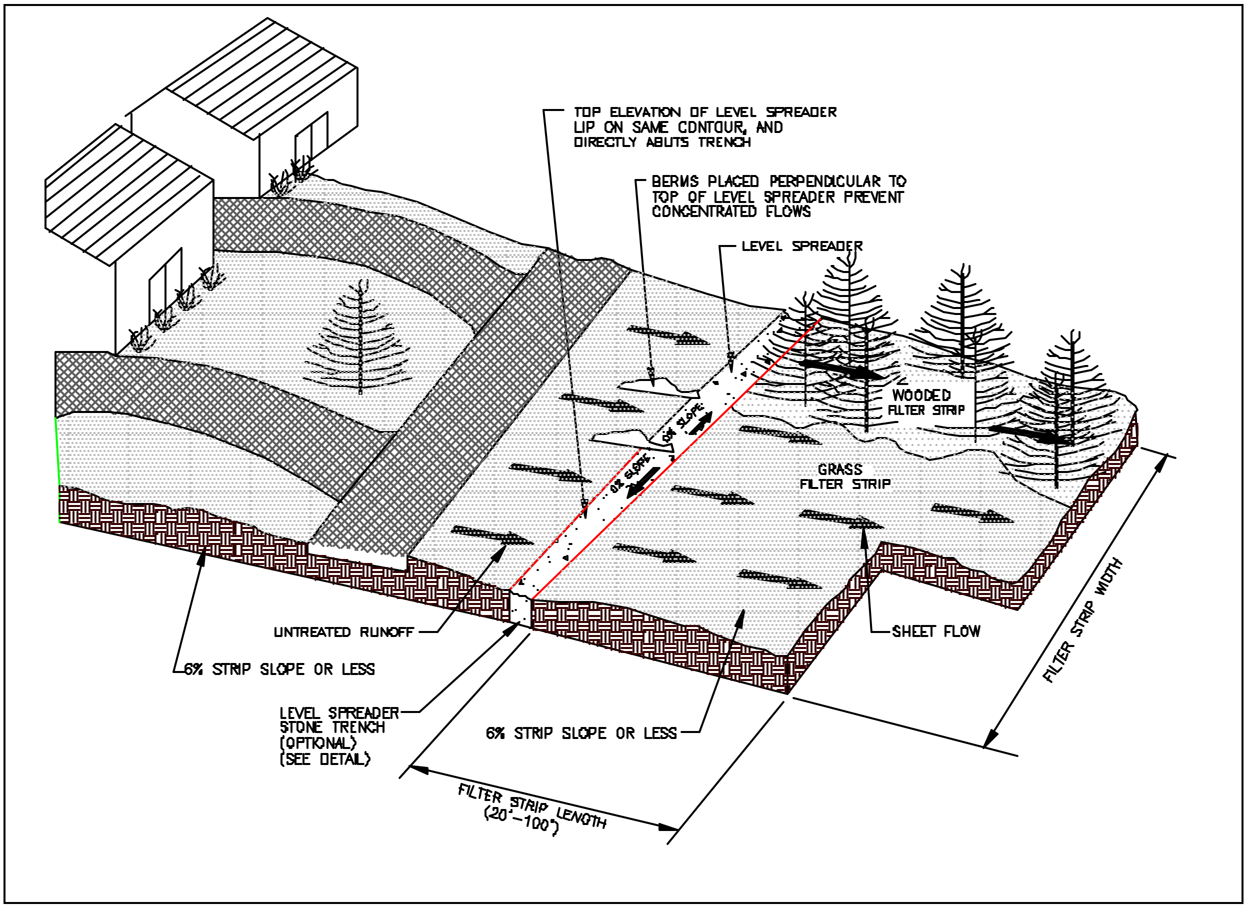
- Top edge of buffer should directly abut the contributing impervious area and follow the same elevation contour line.
- Runoff water containing high sediment loads to be treated in a sediment trapping device before release in a flow spreader.
- Buffers should not be cleaned of litter or "managed," particularly Zone 1 of a regulated riparian buffer.

Operation and Maintenance Recommendations

- A stormwater management easement and maintenance agreement should be required for each facility.
- The buffer should be inspected for signs of erosion or concentrated flow after every rainfall until vegetation is established, and needed repairs made promptly.
- After area is stabilized, inspections should be made quarterly.

Figure 3-8 Schematic of a Buffer

Source: Controlling Urban Runoff



Performance Standards

- Buffers must accept stormwater runoff as overland sheet flow in order to effectively filter suspended materials out of the overland flow.
- In order to function properly, the strip should be at least as wide as the flow path entering the filter, and flow entering a buffer must be spread relatively uniformly over the width of the strip.
- The removal of soluble pollutants is low because the degree of infiltration provided is generally very small.
- Removals of nutrients and oxygen demand decrease as the amount of clay in the soil increases.
- Filter strip applications should be limited to drainage areas of 10 acres or less, with the optimal size being less than 5 acres.
- The use of buffers to treat parking lot runoff or street runoff should incorporate a level spreading device such as a shallow stone filled trench or slotted parking blocks.

50-Foot Wide Buffer Strip

- This design significantly removes the coarser suspended particles in runoff by the lowering of runoff velocities.
- Pollutant removal enhanced by mild slopes, minimal mowing/ maintenance, sustaining natural cover.
- Buffer strips, designed, installed and maintained as required in this manual, provide the following pollutant removal rates:

Pollutant	**Removal Rate (%)**
BOD	40
TSS	80
Total P	35
Total N	25
Metals	70

3.2.7 Sand Filters

Most sand filters have two chambers. The first chamber is a sedimentation chamber that removes floatables and other heavy sediments. The second chamber is the filtration chamber. This chamber removes additional pollutants by filtering the runoff through a sand bed.

Sand filters are primarily designed as off-line structures, allowing larger storms to bypass the structure. Sand filters must be used in conjunction with other measures to achieve detention requirements.

Three types of sand filter designs are recommended:

Surface sand filter - The surface sand filter is a ground-level open-air structure that consists of a pretreatment sediment forebay and a filter bed chamber. This system can treat drainage areas up to 10 acres in size and is typically located off-line. Surface sand filters can be designed as an excavation with an earthen embankment or as a concrete structure. Refer to Figure 3 – X for a schematic of a surface sand filter.

Perimeter sand filter - The perimeter sand filter is an enclosed filter system typically constructed just below grade in a vault along the edge of an impervious area such as a parking lot. The system consists of a sedimentation chamber and a sand bed filter. Runoff flows into the structure through a series of inlet grates located along the top of the control. Refer to Figure 3-X for a schematic of a perimeter sand filter.

Underground Sand Filter - The underground sand filter is intended primarily for extremely space-limited and high-density areas. Refer to Figure 3-X for a schematic of an underground sand filter.

Required Design Standards

- Maximum contributing drainage area to an individual stormwater filtering system should be less than 10 acres.
- Design volume based on WQ_v.
- Adequate pretreatment (e.g., filter strips) is required to prevent sediment from overloading the filters.
- Most sand filters normally require one to six feet of head.
- Designed to completely empty in 36 hours.
- Inlet structure should be designed to spread the flow uniformly across the surface of the filter media.
- Stone riprap or other dissipation devices should be installed to prevent gouging of the sand media and to promote uniform flow.
- Final sand bed depth should be at least 18 inches.
- Underdrain pipes should consist of main collector pipes and perforated lateral branch pipes.

- The underdrain piping should be reinforced to withstand the weight of the overburden.
- Internal diameters of lateral branch pipes should be 4 inches or greater (6 inches preferred) and perforations should be 3/8 inch.
- Maximum spacing between rows of perforations should not

exceed 6 inches.

- All piping should be schedule 40 polyvinyl chloride or greater strength.
- Minimum grade of piping should be 1/8 inch per foot (1% slope).
- Access for cleaning all underdrain piping should be provided.
- Surface filters may have a grass cover to aid in pollution adsorption.
- Vegetation should be established over the contributing drainage areas before runoff can be accepted into the facility.

Recommended Standards

- Pretreatment (sedimentation or oil and grease removal) will enhance the performance of the filter and will decrease the maintenance frequency required to maintain effective performance.
- Two sand bed configurations are recommended for use:

1) Sand Bed with Gravel Layer;
   - Top layer of sand should be a minimum of 18 inches of 0.02 - 0.04 inch diameter sand (smaller sand size is acceptable).
   - A layer of one-half to 2-inch diameter gravel under the sand should be provided for a minimum of 2 inches of cover over the top of the under-drain lateral pipes.
   - No gravel is required under the lateral pipes.
   - A layer of geotextile fabric (permeable filter fabric) should separate the sand and gravel.

2) Sand Bed with Trench Design;
   - Top layer of sand is to be 12-18 inches of 0.02 - 0.04 inch diameter sand (smaller size is acceptable).
   - Laterals to be placed in trenches with a covering of one-half to 2-inch gravel and geotextile fabric.
   - The lateral pipes are to be underlain by a layer of drainage matting.
   - A presettling basin and/or biofiltration swale is recommended to pretreat runoff discharging to the sand filter.
   - A maximum spacing of 10 feet between lateral underdrain pipes is recommended.

Operation and Maintenance Recommendations

- A stormwater maintenance manual is required for each facility. The maintenance manual should require the owner of the sand filter to periodically clean the structure.
- Scrape off sediment layer buildup during dry periods with steel rakes or other devices.
- Replace some or all of the sand when permeability of the filter media is reduced to unacceptable levels, which should be specified in the design of the facility. A minimum infiltration rate of 0.5 inches per hour should be used for all infiltration designs.

Performance Standards

Sand Filtration Basins

- Estimated long-term pollutant removal rates as follow:

Pollutant	**Removal Rate (%)**
BOD	60
TSS	85
Total P	65
Total N	50
Lead	60
Metals	70

- Filtration System Performance Enhancement
  - Sand/peat beds have higher removal effectiveness due to adsorptive properties of peat.
  - Designs incorporating vegetative cover on the filter bed increase nutrient removal.

Figure 3-9 Surface Sand Filter

Source: Center for Watershed Protection

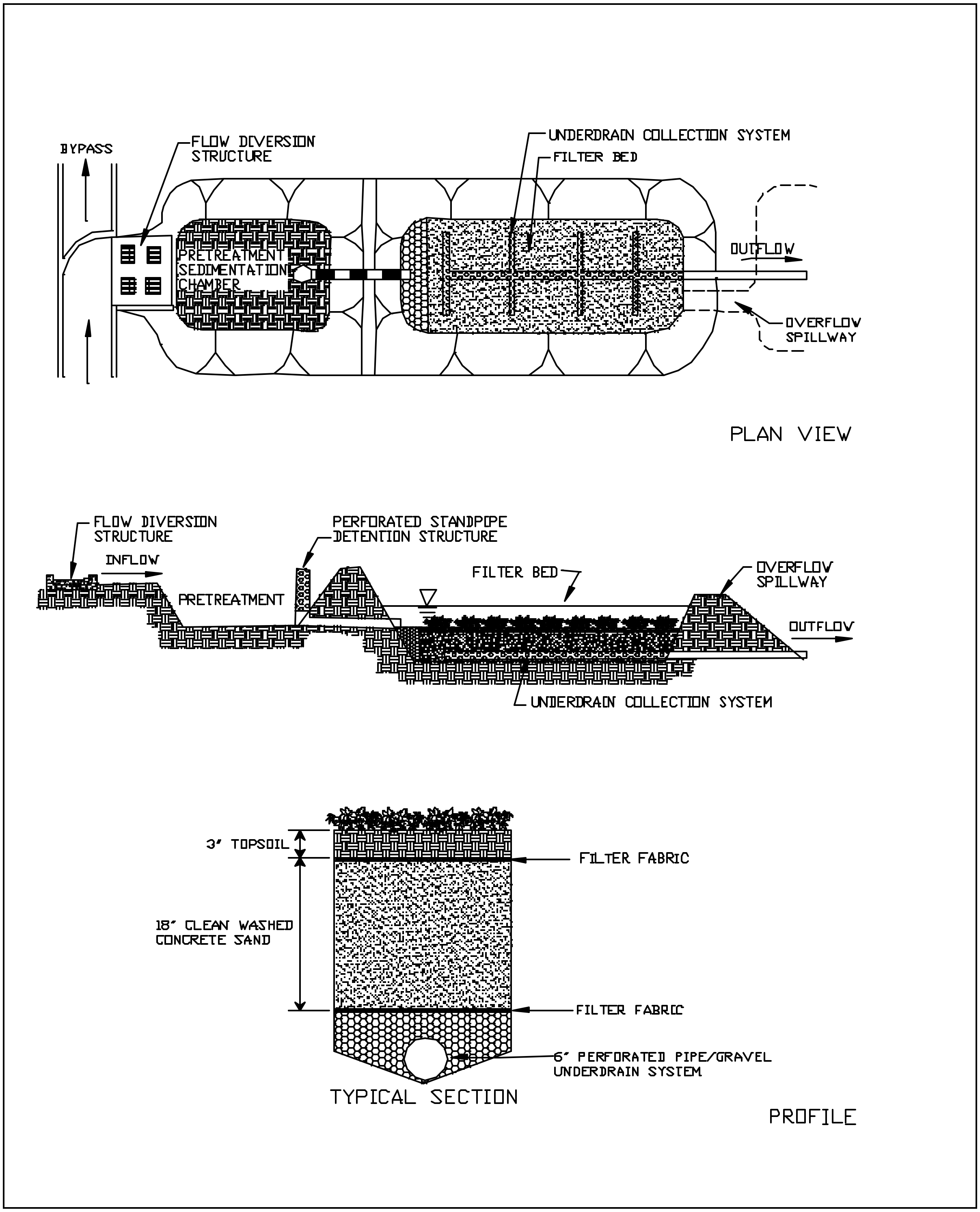


Figure 3-10 Perimeter Sand Filter

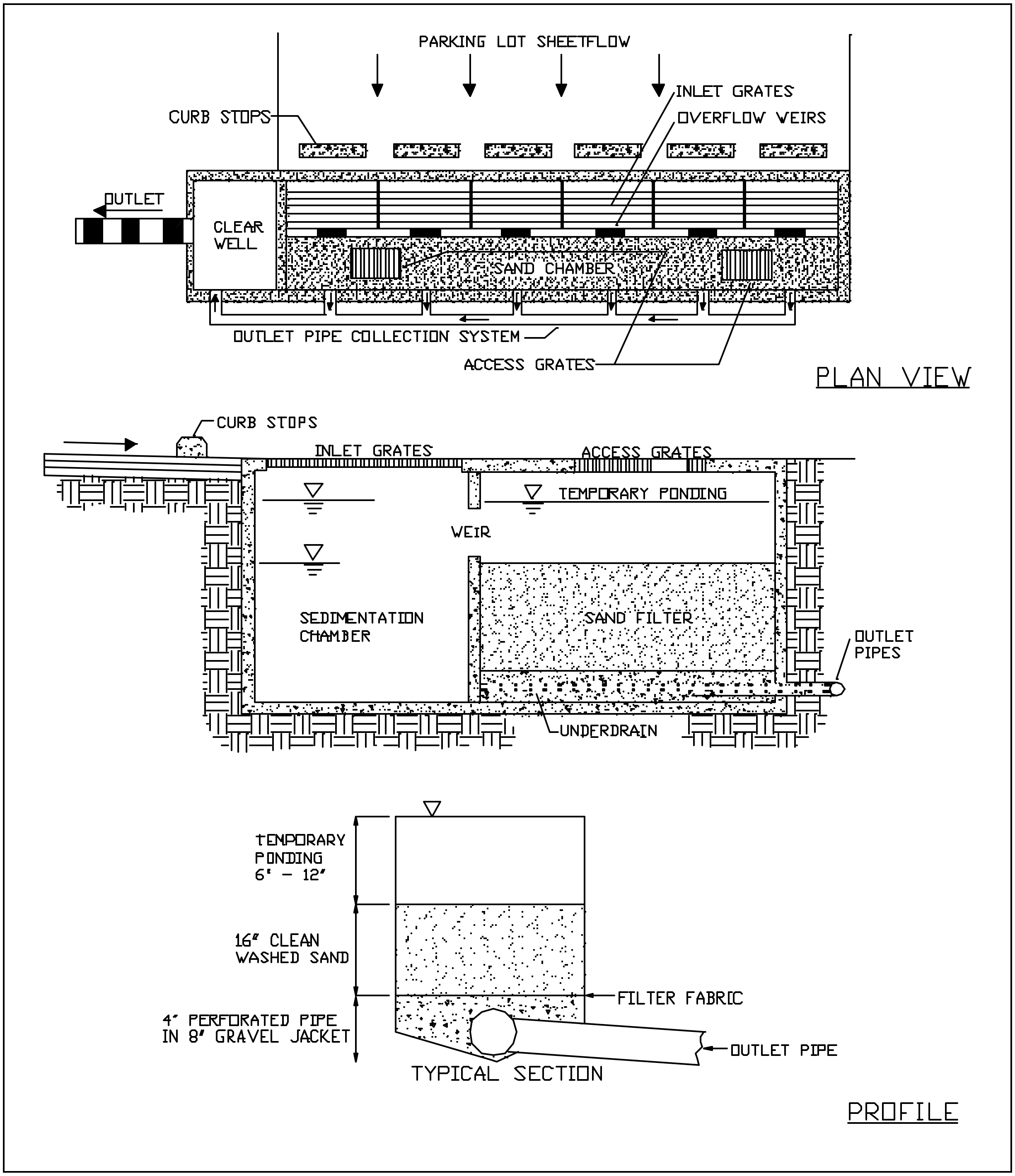
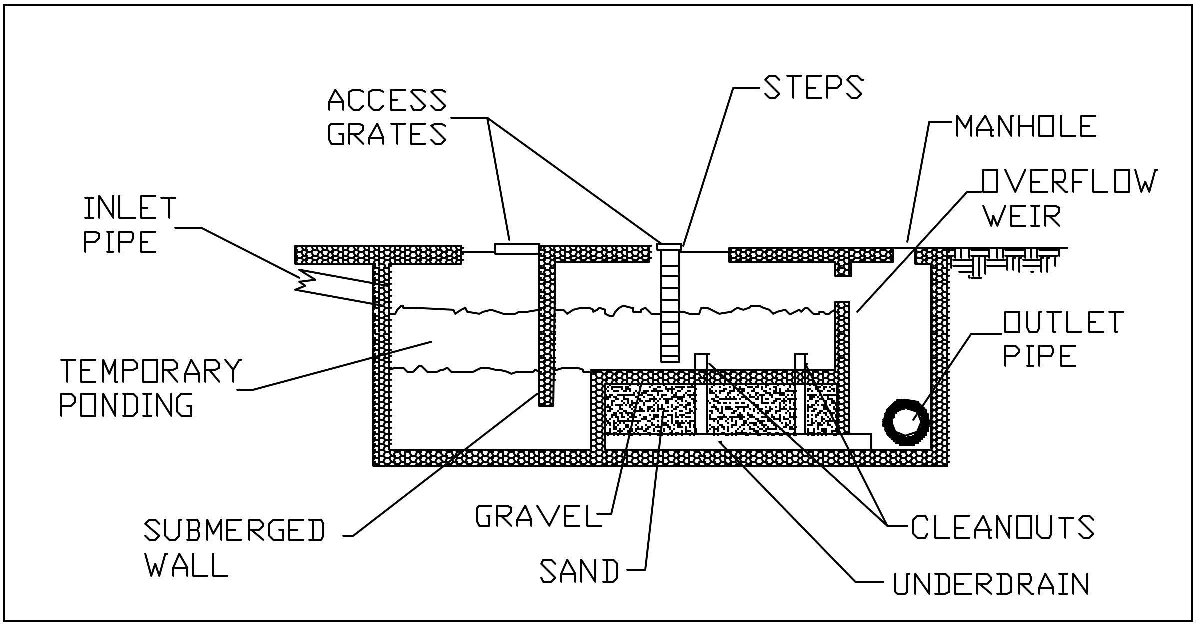


Figure 3-11 Underground Sand Filter



3.2.8 Artificial Wetlands

Artificial wetlands are designed for stormwater benefits. Wetlands can be designed to treat the WQv and to meet detention requirements. It is the intent of the City to encourage regional wetlands for water quantity and quality benefits.

Several design variations exist.
1) *Pocket wetland*: Intended for drainage areas of 5-10 acres. Wetland area should be excavated below the water table to maintain water elevation.
2) *Shallow wetland*: WQ_v treatment is in relatively shallow marsh areas, with the deepest sections of the wetland being the forebay at the inlet and the micropool at the outlet. The disadvantage of this configuration is the surface area required to treat the WQ_v.
3) *Pond/Wetland system*: This configuration has 2 separate cells: a wet pond and a shallow marsh. The pond cell is on the upstream end of the system, settles sediments and reduces velocities prior to discharging into the wetlands.

Required Design Specifications

- Design for the WQ_v. Can be designed to meet detention requirements.
- Drainage area of 25 acres or more; 5 acres for pocket wetlands
- Inflow of water must be greater than that leaving the basin by infiltration or exfiltration.
- A water balance should be performed to demonstrate that a stormwater wetland could withstand a thirty day drought at summer evaporation rates without completely drawing down.
- Designed for an extended detention time of 48 hours for the WQ_v (see Section 1.4).
- The orifices used for extended detention will be vulnerable to blockage from plant material or other debris that will enter the basin with stormwater runoff. Therefore, some form of protection against blockage should be installed (such as some type of non-corrodible wire mesh).
- Surface area of the wetland should account for a minimum of 1 percent of the area of the watershed draining into it (1.5 percent for a shallow marsh design).
- The length to width ratio should be at least 2 to 1.
- A soil depth of at least 4 inches should be used for shallow wetland basins.
- A minimum of 35 percent of the total surface area should have a depth of six inches or less and at least 65 percent of the total surface area should be shallower than 18 inches.
- The deeper area of the wetland should include the outlet structure so outflow from the basin is not interfered with by sediment buildup.

- The designer should maximize use of existing- and post-grading pondscaping design to create both horizontal and vertical diversity and habitat.
- Stabilize surrounding slopes with vegetation to trap sediments and other pollutants, preventing them from entering the wetland.
- A maintenance plan should be provided and adequate provision made for ongoing inspection and maintenance, with more intense activity for the first three years after construction.
- The wetland should be maintained to prevent loss of area of ponded water available for emergent vegetation due to sedimentation and/or accumulation of plant material.
- Sediment Forebay:
  - A 4 to 5 foot deep forebay should be established at the pond inflow points to capture larger sediments. Direct maintenance access to the forebay should be provided with access 25 feet wide minimum and a 5:1 slope maximum. Sediment depth markers should be provided.
  - The sediment forebay should be designed to treat 0.1"/impervious acre. This volume can be included with the overall WQ_v.
  - If high water velocity at the outlet of the forebay is a problem, energy dissipation should be installed.

Recommended Specifications

- It is recommended that the frequently flooded zone surrounding the wetland be located within approximately 10 to 20 feet from the edge of the permanent pool.
- Soil types conducive to wetland vegetation should be used during construction.
- The wetland should be designed to allow slow percolation of the runoff through the substrate (add a layer of clay for porous substrates).
- The depth of the forebay should contain approximately 10 percent of the total volume of the normal pool.
- As much vegetation as possible and as much distance as possible should separate the basin inlet from the outlet.
- Of the 75 percent of the wetland that should be 12 inches deep or less, it is recommended that approximately 25 percent range from 6 inches deep to 12 inches deep, and that the remaining 50 percent be 6 inches or less in depth.
- The water should gradually get shallower about 10 feet from the edge of the pond.
- The planted areas should be made as square as possible within the overall design of the wetland, rather than long and narrow.
- The only site preparation that is necessary for the actual planting (besides flooding the basin) is to ensure that the substrate is soft enough to permit relatively easy insertion of the plants.

Operation and Maintenance Recommendations

- A stormwater maintenance manual is required for each facility. The maintenance manual should require the owner of the wetland to periodically clean the structure. The manual should provide for ongoing inspection and maintenance, with more intense activity for the first three years after construction.
- The wetland should be maintained to prevent loss of area of ponded water available for emergent vegetation due to sedimentation and/or accumulation of plant material.
- Sediment forebays should be cleaned every 2 to 5 years except for pocket wetlands without forebays, which are cleaned after a six-inch accumulation of sediment.
- The ponded water area may be maintained by raising the elevation of the water level in the permanent pond, by raising the height of the orifice in the outlet structure, or by removing accumulated solids by excavation.
- Water levels may need to be supplemented or drained periodically until vegetation is fully established.
- It may be desirable to remove contaminated sediment deposits or to harvest above ground biomass and remove it from the site to permanently remove pollutants from the wetland.

Figure 3-12 Shallow Marsh Planting Strategies

Source: Controlling Urban Runoff

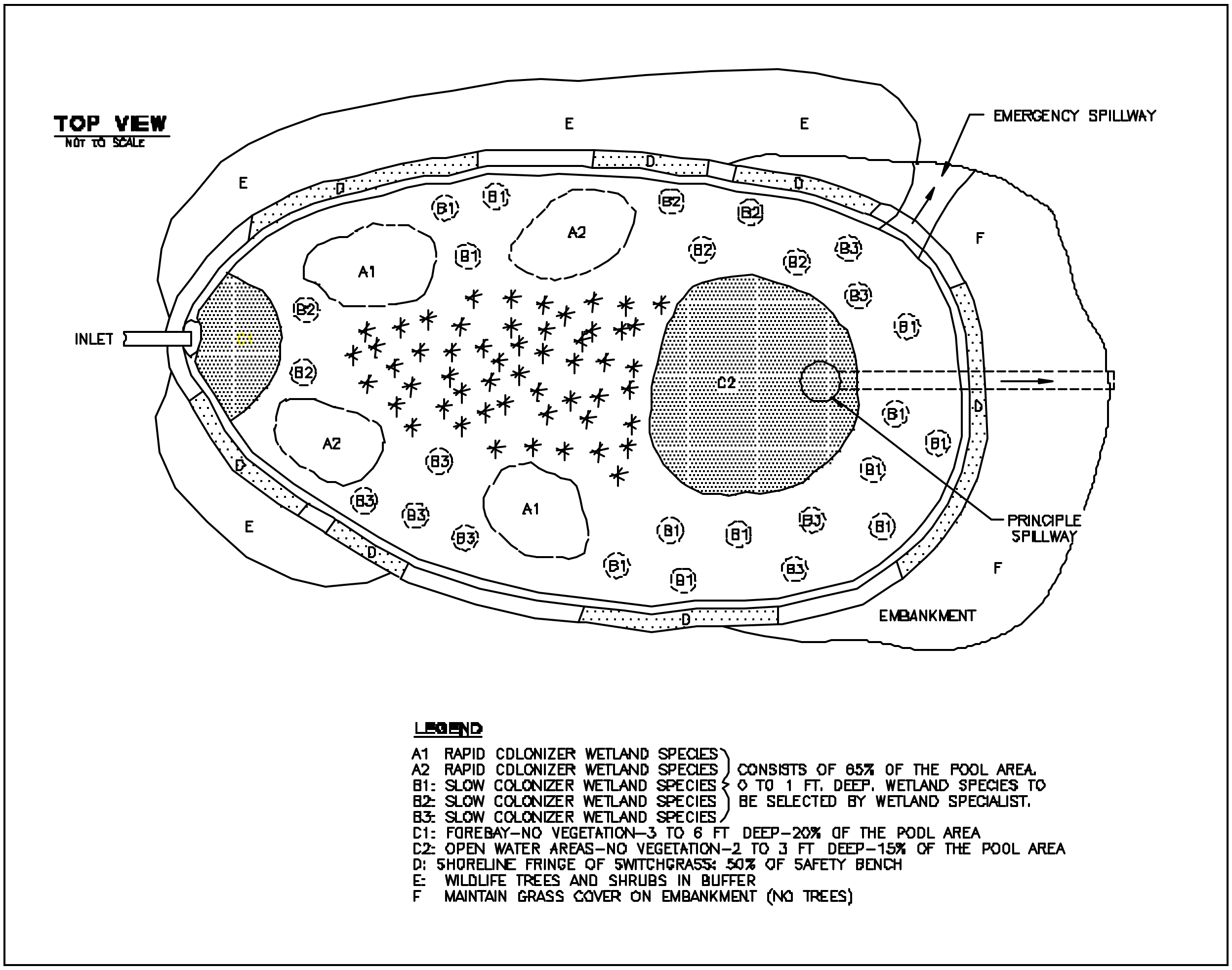
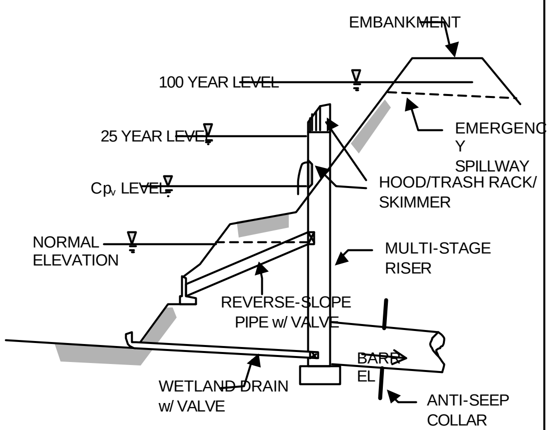


Figure 3-13 Spillway Configuration



POOL

Figure 3-14 Pond/Wetland System

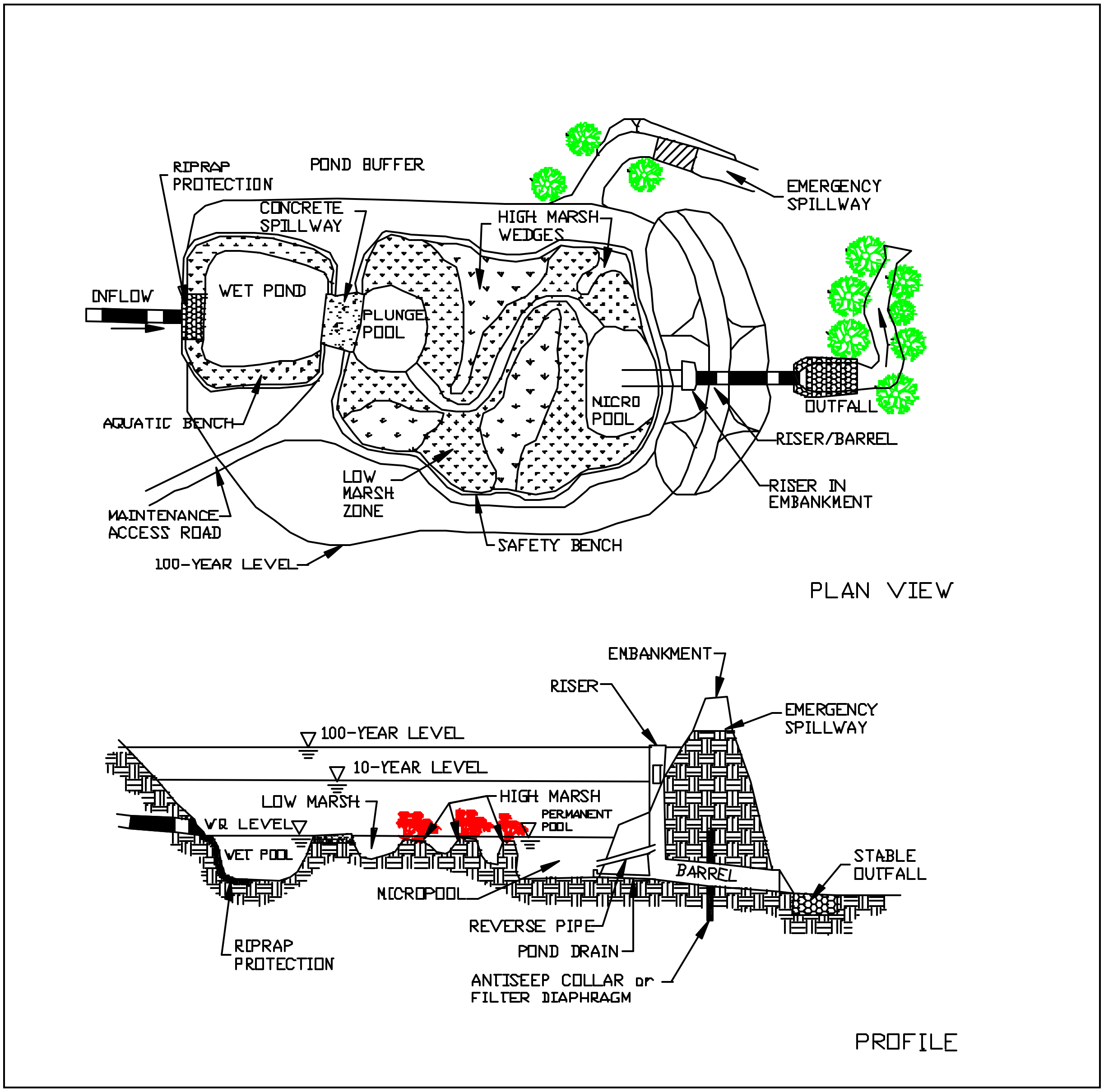


Figure 3-15 Shallow Wetland

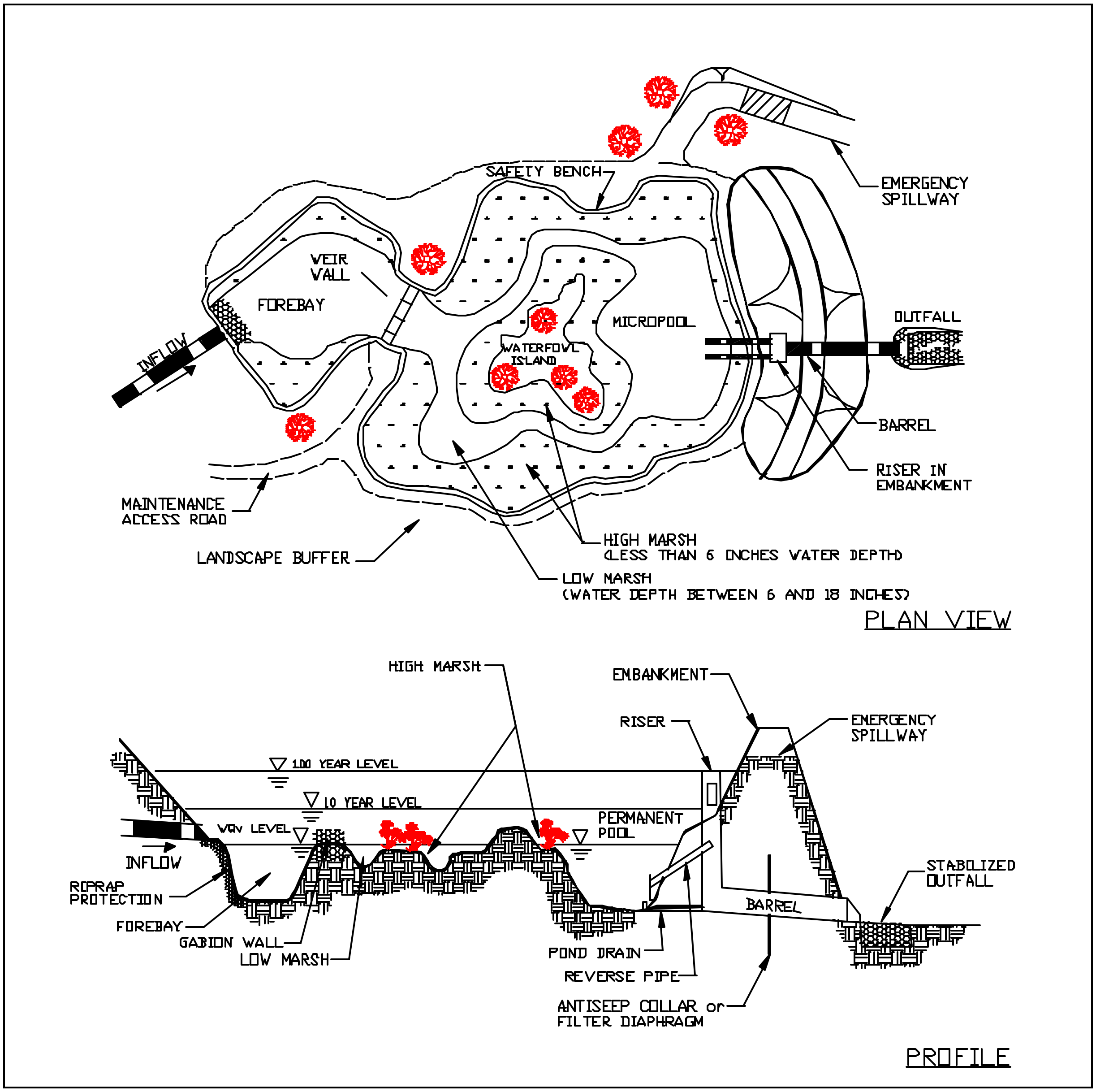
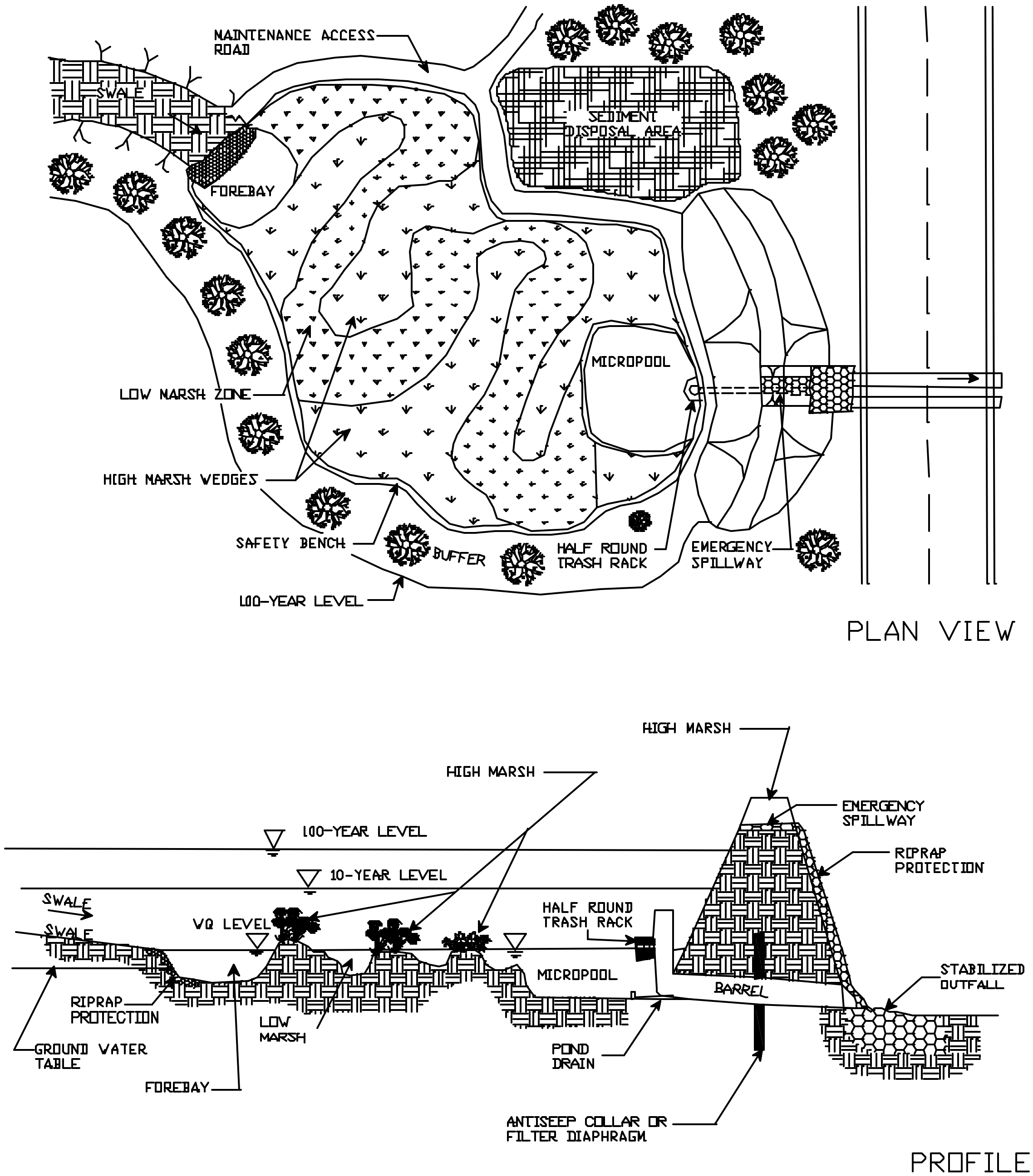


Figure 3-16 Pocket Wetland



Performance Standards

- Performance depends on appropriate plantings for the soils, climate, and types of pollutants or land use (oil and grease, high sediment loads, high nutrient loads) in the drainage area.
- Design performance depends on protecting marsh-type plantings.
- Performance enhancement can be obtained by increasing the size of the marsh area, by incorporating multiple pools into marsh area, or by incorporating a network of shallow channels in the marshy area.
- For wetlands designed, installed and constructed as required in this manual, the estimated long-term pollutant removal rates are as follows:

Pollutant	**Removal Rate (%)**
BOD	55
TSS	85
Total P	55
Total N	45
Metals	80

Landscaping Considerations

- A minimum of 2 aggressive wetland species (primary species - Figure 1-5) of vegetation should be established in quantity on the wetland.
- Three additional wetland species (secondary species- Figure 1-5) of vegetation should be planted on the wetland, although in far less numbers than the two primary species.
- 30 to 50 percent of the shallow (12 inches or less) area of the basin should be planted with wetland vegetation. The optimal depth requirements for several common species of emergent wetland plants are often six inches of water or less.
- Approximately 50 individuals of each secondary species should be planted per acre; set out in 10 clumps of approximately 5 individuals and planted within 6 feet of the edge of the pond in the shallow area leading up to the ponds edge; spaced as far apart as possible, but no need to segregate species to different areas of the wetland.
- Wetland mulch, if used, should be spread over the high marsh area and adjacent wet zones (-6 to +6 inches of depth) to depths of 3 to 6 inches.
- A minimum 25-foot buffer, for all but pocket wetlands, should be established and planted with riparian and upland vegetation (50 foot buffer if wildlife habitat value required in design). In addition, an additional 15 feet setback to structures should be included.
- Local assistance should be obtained for information concerning plants to be used, planting schedule, soil requirements, mulch requirements, etc.

3.3 Riparian Buffers

The Neuse Buffer Rule requires that riparian buffer areas be protected. This Rule applies a 50-foot wide riparian buffer on each side of and adjacent to surface waters in the Neuse River Basin. Surface waters subject to this Rule are defined as intermittent streams, perennial streams, lakes, and ponds if shown on either the most recent version of the Natural Resources Conservation Service soil survey map or on the most recent version of the United States Geologic Survey's 1:24,000 scale (7.5 minute) quadrangle topographic maps. Riparian buffers adjacent to surface waters that do not appear on either of the maps are not subject to this Rule. Riparian buffers adjacent to surface waters that appear on the maps are subject to this Rule unless one of the following applies:

- An on-site determination shows that surface waters are not present, or
- Existing uses were present and ongoing on July 22, 1997.

The riparian buffers protected by this Rule are measured perpendicular to the water body and from top of bank for streams and from edge of normal pool for lakes and ponds. The buffer has two zones. The first 30 feet measured from the water body is Zone 1 and is to be left undisturbed unless the NC Environmental Management Commission (EMC) has granted a variance. The next 20 feet is considered Zone 2 and should be left undisturbed were possible but may be modified provided that vegetation is re-established and sheet flow is maintained.

Runoff entering the riparian buffers should enter as sheet flow unless the local topographic conditions are such that predevelopment runoff does not enter the buffer as sheet flow.

Allowable uses in the riparian buffers have one of four (4) classifications. These uses are designated as exempt, allowable, allowable with mitigation and prohibited and are described below.

a. EXEMPT. Uses designated as exempt are allowed within the riparian buffer. Exempt uses shall be designed, constructed and maintained to minimize soil disturbance and to provide the maximum water quality protection practicable.
b. ALLOWABLE. Uses designated as allowable may proceed within the riparian buffer provided that there are no practical alternatives to the requested use. These uses require written authorization from the NC Division of Water Quality.
c. ALLOWABLE WITH MITIGATION. Uses designated as allowable with mitigation may proceed within the riparian buffer provided that there are no practical alternatives to the requested use and an appropriate mitigation strategy has been approved. These uses require written authorization from the NC Division of Water Quality.
d. PROHIBITED. Uses designated as prohibited may not proceed within the riparian buffer unless a variance is granted by the EMC.

Persons who wish to undertake uses designated as allowable or allowable with mitigation shall submit a request for a "no practical alternatives" determination to the NC Division of Water Quality. The applicant shall certify that the criteria identified below are met. The NC Division of Water Quality shall grant an Authorization Certificate upon a "no practical alternatives" determination. The procedure for making an Authorization Certificate shall be as follows:

For any request for an Authorization Certificate, the NC Division of Water Quality will review the entire project and make a finding of fact as to whether the following requirements have been met in support of a "no practical alternatives" determination:

(i) The basic project purpose cannot be practically accomplished in a manner that would better minimize disturbance, preserve aquatic life and habitat, and protect water quality.

(ii) The use cannot practically be reduced in size or density, reconfigured or redesigned to better minimize disturbance, preserve aquatic life and habitat, and protect water quality.

(iii) Best management practices will be used if necessary to minimize disturbance, preserve aquatic life and habitat, and protect water quality.

3.4 *Water Quality Design Examples*

Example 1:

Given:
New residential subdivision, zoned R-4; lot sizes approx. 11,000ft2 and 130 dwelling units anticipated = 32.5 acres in homes; 2.6 acres of ROW
DA = site acreage = 38 acres
1 acre wooded area to be preserved
60% of ROW paved

A. Calculate TN export. TN export goal is 3.6 lbs/ac/yr. Residential sites must reach 6 lbs/ac/yr with BMPs and then can pay the offset fee of $330/lb.

1) Since building footprints aren't shown, use **Method 1.**

(1) Type of Land Cover	**(2) Site Area (Acres)**	**(3) TN Export Coeff. (lbs/ac/yr)**	**(4) TN Export by Land Use (lbs/yr)**	**(5) TN Export From Site (lbs/ac/yr)**
Permanently preserved undisturbed open space (forest, unmown meadow)	1	0.6	0.6	
Permanently preserved managed open space (grass, landscaping, etc.)	1.9	1.2	2.28	
Right-of-way (read TN export from Figure 1)	2.6	10.5	27.3	
Lots (read TN export from Figure 2)	32.5	5.8	188.5	
Total	38		218.68	
Average for Site				5.75

TN export = 5.75 lbs/ac/yr > 3.6 lbs/ac/yr

B. Determine methods to reduce TN export

Option 1

1) Install wet detention pond for water quality and quantity
Wet ponds provide a 25% reduction.
5.75 lbs/ac/yr – 25% = 4.31 lbs/ac/yr

2) Calculate payment into the Wetland Restoration Fund.
(4.31 lbs/ac/yr – 3.6 lbs/ac/yr) x 38 ac x $330/lb = $8,903.40

Option 2

1) Since the TN export rate is less than 6 lbs/ac/yr, a one-time offset payment can be made in lieu of a structural BMP.
(5.75 lbs/ac/yr – 3.6 lbs/ac/yr) x 38 ac x $330/LB = $26,961.00

Example 2:

Given:
New commercial development
10 ac site, 8 acres in building and pavement
2 ac in landscaping and lawn

A. Calculate TN loading
   1) Since building footprint and parking lot are shown, use Method 2.

(1) Type of Land Cover	(2) Site Area (Acres)	(3) TN Export Coeff. (lbs/ac/yr)	(4) TN Export by Land Use (lbs/yr)	(5) TN Export From Site (lbs/ac/yr)
Permanently preserved undisturbed open space (forest, unmown meadow)	0	0.6	0	
Permanently preserved managed open space (grass, landscaping, etc.)	2	1.2	2.4	
Impervious surfaces (roads, parking lots, driveways, roofs, paved storage areas, etc.)	8	21.2	169.6	
Total	10		172	
Average for Site				17.2

B. Determine methods to reduce TN export. TN export goal is 3.6 lbs/ac/yr. Commercial sites must use BMPs to get TN export to 10

Option 1

1) Install Bioretention areas in the medians. Discharge from these areas will be piped directly into the wet pond. Bioretention areas provide a 25% reduction
   17.2 lbs/ac/yr – 25% = 12.9 lbs/ac/yr

2) Install a wet pond for water quality and quantity. Wet ponds provide 25% reduction credit
   12.9 lbs/ac/yr – 25% = 9.675 lbs/ac/yr

3) Calculate the offset payment.
(9.675lbs/ac/yr –3.6 lbs/ac/yr) x 10 ac x $330/lb = $20,047.50

Option 2

A. Reduce the impervious area by rearranging parking spaces. New impervious area is 6.8 acre, with 3.2 acres in managed landscape areas.

B. Calculate TN export.

(1) Type of Land Cover	(2) Site Area (Acres)	(3) TN Export Coeff. (lbs/ac/yr)	(4) TN Export by Land Use (lbs/yr)	(5) TN Export From Site (lbs/ac/yr)
Permanently preserved undisturbed open space (forest, unmown meadow)	0	0.6	0	
Permanently preserved managed open space (grass, landscaping, etc.)	4	1.2	4.8	
Impervious surfaces (roads, parking lots, driveways, roofs, paved storage areas, etc.)	6	21.2	127.2	
Total	10		132	
Average for Site				13.2

C. Determine methods to reduce TN loading.

1) Install a wet pond for 25% reduction

13.6 lbs/ac/yr – 25% = 9.9 lbs/ac/yr

2) Calculate the offset payment

(9.9 lbs/ac/yr – 3.6 lbs/ac/yr) x 10 ac x $330/lb = $20,790.00