STORMWATER MANAGEMENT TECHNICAL MANUAL
SOUTHPORT, NORTH CAROLINA

TABLE OF CONTENTS

<u>Page</u>

Attachment

SECTION A

GENERAL

This technical manual is intended to set forth performance, design, and construction standards for the stormwater management systems of the City of Southport, the plan submittal requirements, and design procedures for use by the designer. The methods and procedures presented are intended to simplify the design and review process for residential and commercial stormwater management systems in the City of Southport. It is not required that the designer use the methods presented, however, these methods, charts, and procedures will be used by the city to determine adequacy of the design of the stormwater management systems.

It is recommended that anyone planning to use this manual have, as a minimum, knowledge of stormwater hydrology and hydraulics as presented in the manual "Elements of Urban Stormwater Design," North Carolina State University, by Dr. H. Rooney Malcom, P.E., dated 1989, or one of equivalent content.

This is to acknowledge that the material in Section H.3.4, Flood Routing, was taken directly from "Elements of Urban Stormwater Design" by Dr. H. Rooney Malcom, P.E., North Carolina State University, dated 1989, with some changes in references, with the permission of North Carolina State University Industrial Extension Service. Copies of the complete text, "Elements of Urban Stormwater Design," ISBN 1-56049-016-0, may be purchased from the Industrial Extension Service at North Carolina State University by calling (919) 515-5326.

Brunswick County is on the North Carolina Department of Environment and Natural Resources list of coastal counties. This requires that the City of Southport comply with the discharge and design standards associated with this designation.

SECTION B

STORMWATER MANAGEMENT POLICY

It is the policy of the City of Southport that all developed land within the city have adequate stormwater facilities and administrative controls to ensure the protection and safety of life and property. The city may accept a stormwater management system for maintenance if the system provides drainage for streets accepted for maintenance by the Board of Aldermen and was designed and constructed in accordance with the provisions in the City Code and this manual.

SECTION C

PLAN SUBMITTALS

One set of site plans and the stormwater discharge control plans shall be submitted to the City Public Works Department for review and approval and shall include items C.1. and C.2.

C.1 Checklist.

All applicable items scheduled in the "Checklist for the City of Southport Stormwater Standards." A copy of this checklist is provided at the end of this manual.

C.2 Certification Requirements.

The following certifications shall appear on all plans:

- **<u>Designer's Certification</u>**

  "I hereby certify that this plan has been prepared in accordance with the latest city Standards and Specifications for Stormwater Management and the City Code of Ordinances. "

  Signature: ______________________________
  Printed Name and Title: ______________________________
  Date: ______________________________
  Registration Number: ______________________________

- **<u>Owner's/Developer's Certification</u>**

  "I/We hereby certify that any clearing, grading, construction or development, or all of these, will be done pursuant to this plan and that the applicable Stormwater Management conditions and requirements of the City of Southport, the State of North Carolina, and the Federal Government and its agencies are hereby made part of this plan."

  Signature: ______________________________
  Printed Name and Title: ______________________________
  Date: ______________________________

SECTION D

MINIMUM DESIGN STANDARDS

D.1 Piped Drainage Systems

D.1.1 Piped Collection Systems. Piped collection systems shall be designed for the 25-year frequency storm event.

D.1.2 Areas Requiring Piping. Open streams, channels, and ditches contained in or partly contained within the property being developed that are to be piped shall consider all offsite areas that will drain through the project site for ultimate development in accordance with current zoning designations. Channels that remain open shall be designed and constructed in accordance with Section G -Open Channel Design.

D.1.3 Minimum Size Pipe. The minimum size storm drainpipe shall be 12 inches in diameter.

D.1.4 Subdrains. When the design requires the use of subdrains, stone shall completely encircle the perforated pipe for a minimum distance of 6 inches from the pipe in all directions. The stone shall be #467M washed stone. The perforated pipe shall be a minimum of 6-inches in diameter and meet the specifications in Section J.1.4. The nonwoven fabric shall be installed in strict accordance with the manufacturer's recommendation.

D.1.5 Minimum and Maximum Velocity. The minimum allowable velocity for a pipe segment shall be 2.5 feet per second. The maximum velocity shall vary per specific situation, but shall be designed such that scour or other deteriorative conditions will not exist.

D.1.6 Minimum Cover. The minimum cover over a storm drainage pipe shall be 2.0 feet. Cover shall be measured from the top of the pipe to the bottom of the base course in roadways. For alternative materials a greater minimum cover may be required, as determined by the manufacturer. In the event lesser cover requirements are determined to be necessary, specific approval of the city will be required.

D.1.7 Easements. Piped systems to be dedicated as public shall be located within the public rights-of-way where possible. Where this is not possible, a dedicated drainage easement will be required with a width determined by the following equation, rounded to the nearest 5-foot increment, except that in no case shall the easement be less than 20 feet:

Width = (2 x depth) + Dia. + 12'

The easement shall be continuous along the pipe to a point of junction with an existing public right-of-way or easement. See Section D.3.6 for open channel right-of-way or easement.

D.1.8 Manhole Spacing. Manholes shall be required at changes in grade and/or alignment and at pipe junctions. The maximum spacing between two manholes in any instance shall be 400 feet. The city may approve spacing in excess of 400 feet on pipelines greater than 54 inches in diameter on a case-by-case basis.

D.1.9 Headwalls. Headwalls or flared end sections shall be required at the inlet and outlet of all pipe systems with a diameter of48" or greater.

D.1.10 Energy Dissipaters. Energy dissipaters shall be designed and constructed at the outlets of all pipe systems to prevent erosion in the receiving watercourse. These shall be designed for the 25-year storm.

D.1.11 Existing Systems. Where feasible, the piped drainage system shall connect to an existing piped drainage system.

D.2 Inlet Design

D.2.1 Design Storm Event. Curb inlets and catch basins shall be designed for the 25-year storm.

D.2.2 Inlet Locations.

a. Curb inlets shall be located such that the gutter flow spread does not exceed eight (8) feet or 1/3 of the street width, whichever is less, during a 25-year storm event with a maximum spacing of 400 feet.

b. Curb inlets shall be located at all low points to prevent ponding water.

c. Curb inlets shall be located on the upstream sides of intersecting streets to prevent flow across the intersecting street.

d. No curb inlets shall be constructed in the radius of curbing at intersections. The minimum distance from the end of radius (PC/PT) to the center of a catch basin is five (5) feet.

D.3 Open Channels

D.3.1 Design Storm Event. Open channels, where allowed, shall be designed for the peak runoff produced by a 25-year frequency storm. The designer shall include in his calculations the runoff from offsite areas, assuming ultimate development in accordance with the current zoning regulations as well as the runoff from the property being developed.

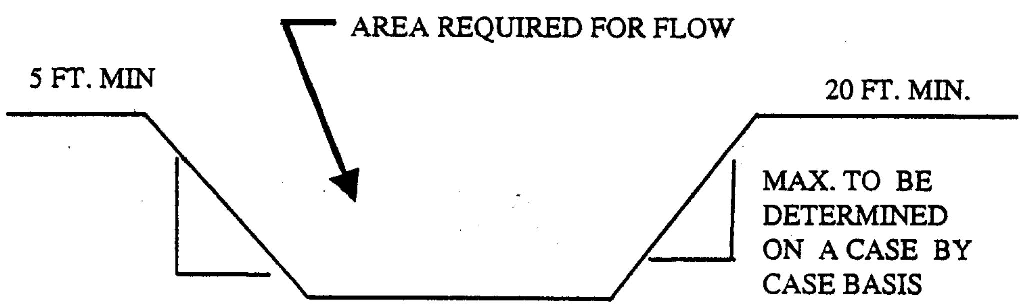
D.3.2 Side Slopes. The side slopes for vegetated channels shall be 3 (horizontal) to 1 (vertical) or flatter. Where the side slopes are protected with riprap, fabric form, or other approved armoring, side slopes of 2 to 1 will be permitted. The city may approve steeper slopes on a case-by-case basis.

D.3.3 Maximum Velocities. The maximum velocities for a channel are dependent upon the type of lining used. Refer to Section G for maximum allowable velocities for various linings.

D.3.4 Alternative Solutions. The linings referred to in Section G are not intended to limit the designer in his choice of linings. Innovative techniques used in design of channels are encouraged. The city, prior to use, shall approve proposed innovative linings. Complete supporting documentation shall be submitted for approval of alternative lining methods in these situations.

D.3.5 Rights-or-Way and Easements. In order to maintain municipally owned, unpiped stormwater management systems in public or private developments, rights-of-way and easements will be required by the city for all open ditches, swales, or other systems dedicated to the city and located outside of public rights-of-way. Such rights-of-way or easements shall be provided in accordance with the following sketch, and shall be continuous to a point of junction with a public right-of-way where possible. Widths in excess of the typical cross-section shown may be necessary and required by the city on a case-by-case basis.

TYPICAL CROSS SECTION



NOTES:

1. See paragraph D.3.2 above for allowable side slopes.
2. The designer should refer to the North Carolina Department of Environment and Natural Resources "Erosion and Sediment Control Planning and Design Manual" for supplemental information.

D.4 Detention and Wet Retention Facilities

The city encourages the use of innovative techniques and designs that will provide the necessary protection for the receiving watercourse. These facilities shall be designed for the runoff produced by the 25-year storm. The design shall be checked to make sure that the facility does not cause any increase in the damages from the 100-year storm. Detailed drawings, substantiating data, calculations, and specifications shall be submitted for designs of this nature. The use of open ponds has been utilized most frequently for stormwater control; therefore, design standards and procedures for this approach have been included in this design guide. However, the city suggests that the developer or owner incorporate stormwater controls into the overall site as an amenity and/or visual enhancement. The following are minimum requirements for detention/ retention facilities.

D.4.1 Minimum Slopes. Side slopes, where vegetation is used for stabilization, shall be 3 (horizontal) to 1 (vertical) or flatter. Where the side slopes are protected with riprap, fabric form, or other approved armoring, side slopes of 2 to 1 will be permitted. The city on a case-by-case basis may approve steeper slopes.

D.4.2 Vegetation. Vegetation for stabilization of side slopes shall be a hearty ground cover such as the following listed in order of best overall suitability:

Tall Fescue	Bermuda Grass	Pensacola Bahiagrass	Reed Canary Grass

All of these are well suited for flooding tolerance, waterways, and channels. The bahiagrass is excellent for sandy sites. The others spread by rootstocks making a well-anchored and stable ground covering. Consideration shall be given to the effects to the surrounding area when vegetative cover is selected.

The designer shall consult with the City Code Enforcement Office regarding landscaping standards, such as selection, spacing, location, and planting requirements of all grasses and plants, which are to be incorporated in the system. Approval of a landscaping plan by the Code Enforcement Office will be required prior to issuance of a construction permit.

D.4.3 Risers. Risers should be a minimum of 12 inches in diameter and pipes shall be a minimum of 12 inches in diameter to reduce the potential for clogging the outlet system. A trash rack with 4-inch maximum openings shall be provided to avoid pipe clogging. The design shall include consideration of anti-vortex measures where deemed necessary for stability of the outlet structure.

D.4.4 Drain. The design of the pond should incorporate a method of draining all water by use of a valve assembly. Where this is not possible, a well-defined low point shall be constructed to allow for pumping out the facility.

D.4.5 Overflow. An emergency outlet or overflow designed for the 100-year storm shall be provided for all detention/retention facilities. This overflow should be designed assuming the principal spillway is obstructed and cannot convey any water.

D.4.6 Other Utilities. No other utilities shall be constructed within 5 feet of the stormwater detention/retention pond unless specifically approved by the city.

D.4.7 Landscaping. The recommendations in the NCDENR "Erosion and Sediment Control, Planning and Design Manual" should be considered when designing the landscaping. Sketch D-2 provides a concept.

D.4.8 Access. A stable access and maintenance shoulder with a minimum width of 10 feet measured from the top of bank shall be provided to allow for the periodic removal of sediment from the system. This access shall be coordinated with the landscaping zone around the basin. The landscaping zone shall not be incorporated in to the access/maintenance way.

D.4.9 Fencing. Fencing for private facilities shall be at the option of the developer.

D.4.10 Flooding of Parking Areas. The city discourages the flooding of parking areas for providing stormwater storage volume. However, when this method is used, no more than 60% of the required parking area may be flooded by 6 inches or more of water during a 25-year storm. The stormwater system shall be designed to completely drain from the parking lot within 2 hours after the storm.

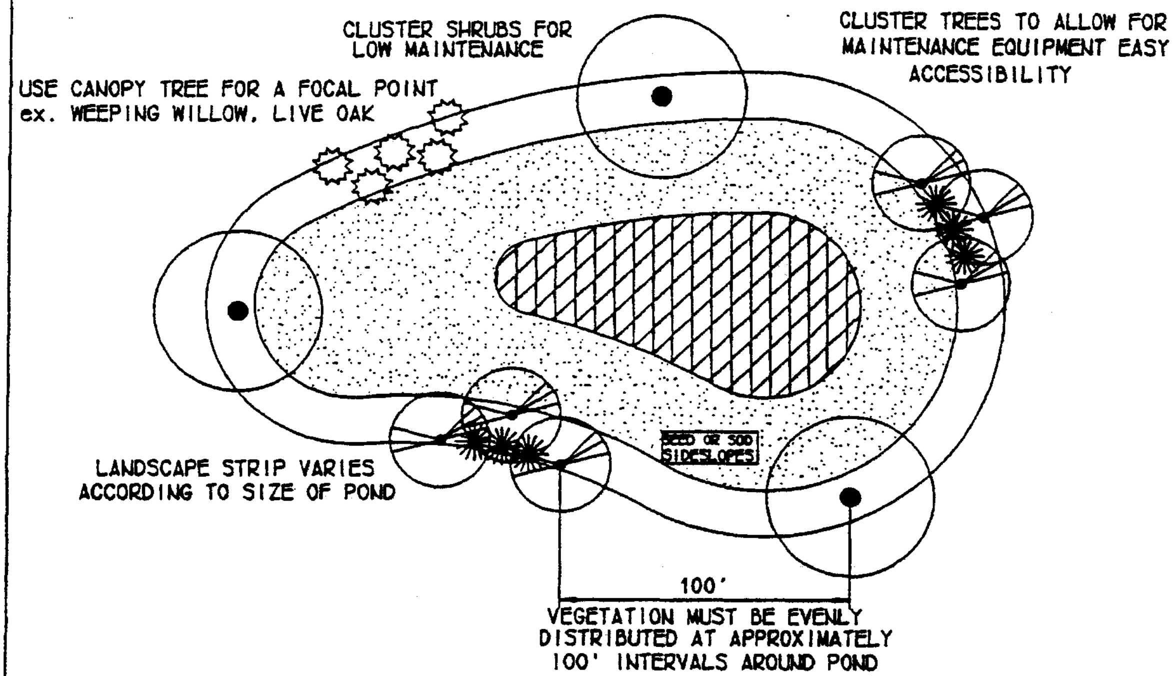
D.5 Coordination

D.5.1 The requirements of the North Carolina Division of Environmental Management's Coastal Stormwater Regulations will impact design of stormwater facilities. In case of conflicting requirements between the regulations set forth by the City of Southport and the Division of Environmental Management, the more stringent shall apply. Otherwise, stormwater facilities will be required to meet both criteria.

D.5.2 The designer shall coordinate with the Brunswick County Health Department for vector control considerations.

D.5.3 Dams constructed as part of stormwater facilities, which meet the criteria for regulation by the North Carolina Department of Environment and Natural Resources, require a permit. If needed the designer shall acquire a dam safety permit prior to the city's approval of the planned stormwater facilities.

TYPICAL
STORM WATER FACILITY
LANDSCAPING
PLAN



Notes:

1. If possible locate pond where vegetation exists.
2. Suggest minimal clearing to conserve visual quality of site and minimize the additional cost of tree planting. An irregular shape provides a more natural appearance.
3. Landscape strip shall be a maximum slope of 7:1 in order to plant vegetation.
4. Provide a minimum of 3 inches of mulch around all vegetation.

SKETCH D-2
Typical Stormwater Facility Landscaping Plan

SECTION E

DETERMINATION OF RUNOFF

One of the most frequently used methods for determining the peak rate of runoff for small watersheds is the Rational Method. Other methods such as the SCS Curve Number Method should be utilized where the Rational Method is less appropriate. The Rational formula is presented below as a guide to the designer.

$$Q = CIA$$

Where **Q** is the maximum rate of runoff from the drainage area expressed in cubic feet per second (cfs), **C** is a runoff coefficient, which is unitless; **I** is the rainfall intensity expressed in inches per hour, and **A** is the drainage area expressed in acres. These components to the Rational formula are explained in detail in the following sections.

The designer should be cautious in the use of the Rational Method and use it only with the understanding of the inherent limitations. Some of the more critical limitations are described below:

a. This method uses one coefficient to describe the surface conditions in a watershed that may have greatly varying surfaces and soils. The use of a composite runoff coefficient is normally used to represent the variations in surface conditions.
b. The return period for the resulting runoff is assumed to be directly related to the rainfall event regardless of the watershed conditions. An attempt to compensate for the antecedent moisture conditions is usually made in the determination of the runoff coefficient.
c. It assumes the rain event to have a constant intensity throughout the storm.

E.1 Runoff Coefficient.

There are several factors to be considered in the determination of the runoff coefficient "**C**". The drainage area slope, soil type, land use, imperviousness, and antecedent moisture condition are the major factors to be considered. The designer shall use a composite or weighted "**C**" to reflect the variations encountered within a drainage area. Table E-1 gives typical values of "**C**" for the area. This table should be used as a guide to develop a composite "**C**" for the drainage area in questions.

E.2 Drainage Area.

The area that contributes runoff to the point of interest for storm drainage design can be determined by utilizing a suitable topographic map. The ridgelines of the drainage basin are first delineated on the map. A planimeter or other suitable method can then measure the area defined.

TABLE E-1
Runoff Coefficient "C" for Various Land Uses**

Slope	Land Use	Sandy Soils Avg.	Clay Soils Avg.
Flat (0-2%)	Woodlands (minor ground litter)	0.10	0.15
	Woodlands (heavy ground litter)	0.15	0.20
	*Pastures, Grass, and Farmland	0.17	0.22
	Lawns	0.10	0.17
	Rooftops and Pavement	0.95	0.95
	Single-Family Residential:		
	< 1 unit per acre (3% imperv)	0.13	0.19
	1-2 units per acre (15% imperv)	0.25	0.30
	3-4 units per acre (23% imperv)	0.32	0.38
	Multi-Family Residential:		
	Duplexes (40% imperv)	0.45	0.50
	Apartments, Townhouses, and Condominiums (60% imperv)	0.64	0.67
	Commercial and Industrial:		
	Industrial Park (25% imperv)	0.35	0.40
	Shopping or Office (50% imperv)	0.55	0.59
Rolling (2-7%)	Woodlands (minor ground litter)	0.15	0.20
	Woodlands (heavy ground litter)	0.20	0.25
	*Pastures, Grass, & Farmland	0.25	0.30
	Rooftops and Pavement	0.95	0.95
	Single-Family Residential:		
	< 1 unit per acre (3% imperv)	0.17	0.24
	1-2 units per acre (15% imperv)	0.28	0.34
	3-4 units per acre (23% imperv)	0.31	0.35
	Multi-Family Residential:		
	Duplexes (40% imperv)	0.49	0.53
	Apartments, Townhouses, and Condominiums (60% imperv)	0.66	0.69
	Commercial and Industrial:		
	Industrial Park (25% imperv)	0.36	0.42
	Shopping or Office (50% imperv)	0.58	0.61

*Coefficients assume good ground cover and conservation treatment.
**Developed using Handbook of Applied Hydrology, Chow, 1964 and Flood Mapping in Charlotte and Mecklenburg County, NC, US Geological Survey, 1975.

E.3 Rainfall Intensity.

The rainfall intensity (**I**) is a function of storm duration and return frequency. The storm duration used for design is normally equal to the time of concentration, or the time for water to travel from the most remote point in the watershed to the point of design. The first step to determine the applicable rainfall intensity is to estimate the time of concentration (**Tc**). Two methods for use in determining the **Tc** are described below. (See Chart E-6)

E.3.1 Kirpich Method. The Kirpich method should be limited in use for watersheds 20 acres or less. The **Tc** can be determined using the nomograph labeled Chart E-2. To use the nomograph, the length of travel and the difference in elevation from the most remote point in the watershed to the point in question shall be determined. A straight line is drawn to connect the length and elevation difference determined above and extended to the intersection with the time of concentration line and **Tc** is read in minutes. The following equation can also be used:

$$T_C = T_t = (L^3/H)^{0.385}/128$$

where:

L is hydraulic length from the most remote point in the watershed measured in feet;
H is the height of the most remote point above the point of interest in feet.

E.3.2 TR-SS Method. This method should be used for watersheds larger than 20 acres up to 1,300 acres. The TR-55 approach to determining the time of concentration is a method of estimating the velocity of flow through the different parts of the watershed including sheet flow, shallow concentrated flow, and flow in open channels or conduits. Using the velocity and length of travel through the different segments, the travel time is determined and summed to obtain the time of concentration (**Tc**). The following are steps taken to estimate **Tc** for the three flow types:

a. Sheet Flow. Sheet flow normally occurs at the headwater of streams in the drainage basin. The flow is usually over plane surfaces for a maximum length of approximately 300 feet. Manning's "**n**" values are used to compensate for the various types of surfaces. Chart E-3 gives "**n**" values for normally encountered surfaces. The time of travel for sheet flow is computed using the following formula:

$$T_t = .007(n\ L)^{0.8}/P_2^{0.5}S^{0.4}$$

where:

T_t is travel time in hours
n is Mannings roughness coefficient
L is length of flow in feet
P_2 is 2-year 24-hour rainfall
S is land slope in ft/ft

b. Shallow Concentrated Flow. After 300 feet, sheet flow usually becomes shallow concentrated flow. The average velocity for this type of flow can be found by using Chart E-4 or by using the following formulas:

$$\textbf{Unpaved } V = 16.1345\,(s)^{0.05}$$
$$\textbf{Paved } V = 20.3282\,(s)^{0.05}$$

where s = slope:

Once the average velocity is determined, the travel time can be computed as:

$$T_t = L/3600V$$

where **L** is the length in feet.

c. Open Channel Flow. For open channel flow, the travel time is computed using an estimated velocity of flow in the channel section. The velocity is estimated using Manning's equation expressed as follows:

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

where:

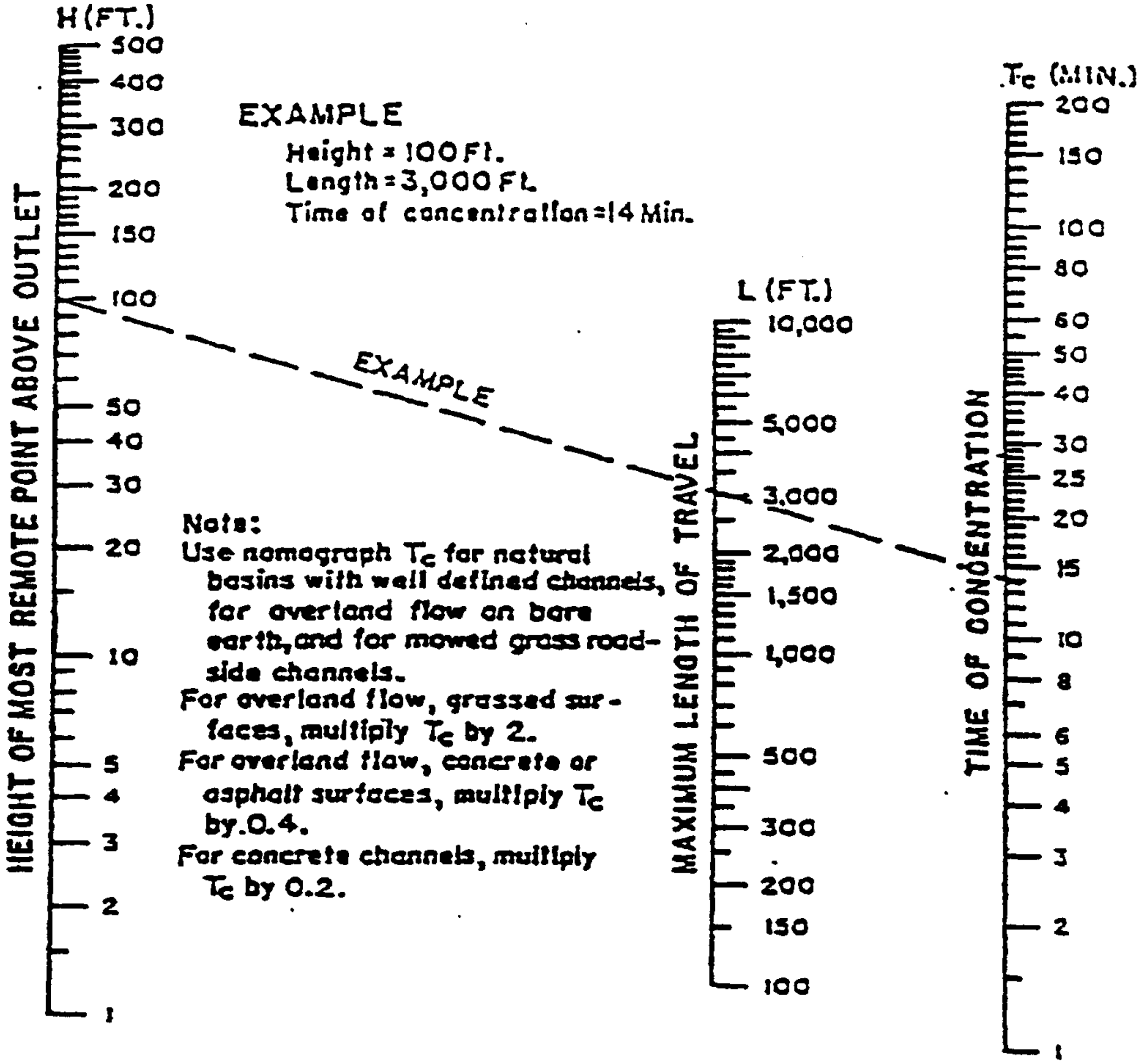
V is velocity in feet per second (fps)
r is the hydraulic radius which is equal to the channel area divided by the wetted perimeter or the perimeter length of sides and bottom below the line of flow
s is the slope of the hydraulic grade line or channel slope in ft/ft and
n is the Manning's roughness coefficient

Once the average velocity is determined from the above equation, the travel time can be computed as shown in paragraph E.3.2.b. above.

The $\mathbf{T_c}$ is a summation of the three travel times computed in the preceding steps.

The rainfall intensity for design can be obtained from the Intensity-Duration Frequency Table. The return period for design in the City of Southport for stormwater management systems is the 25-year return frequency event. (See Chart E-6)

CHART E-2



Based on study by P.Z. Kirpich,
Civil Engineering, Vol.10, No.6, June 1940, p.362

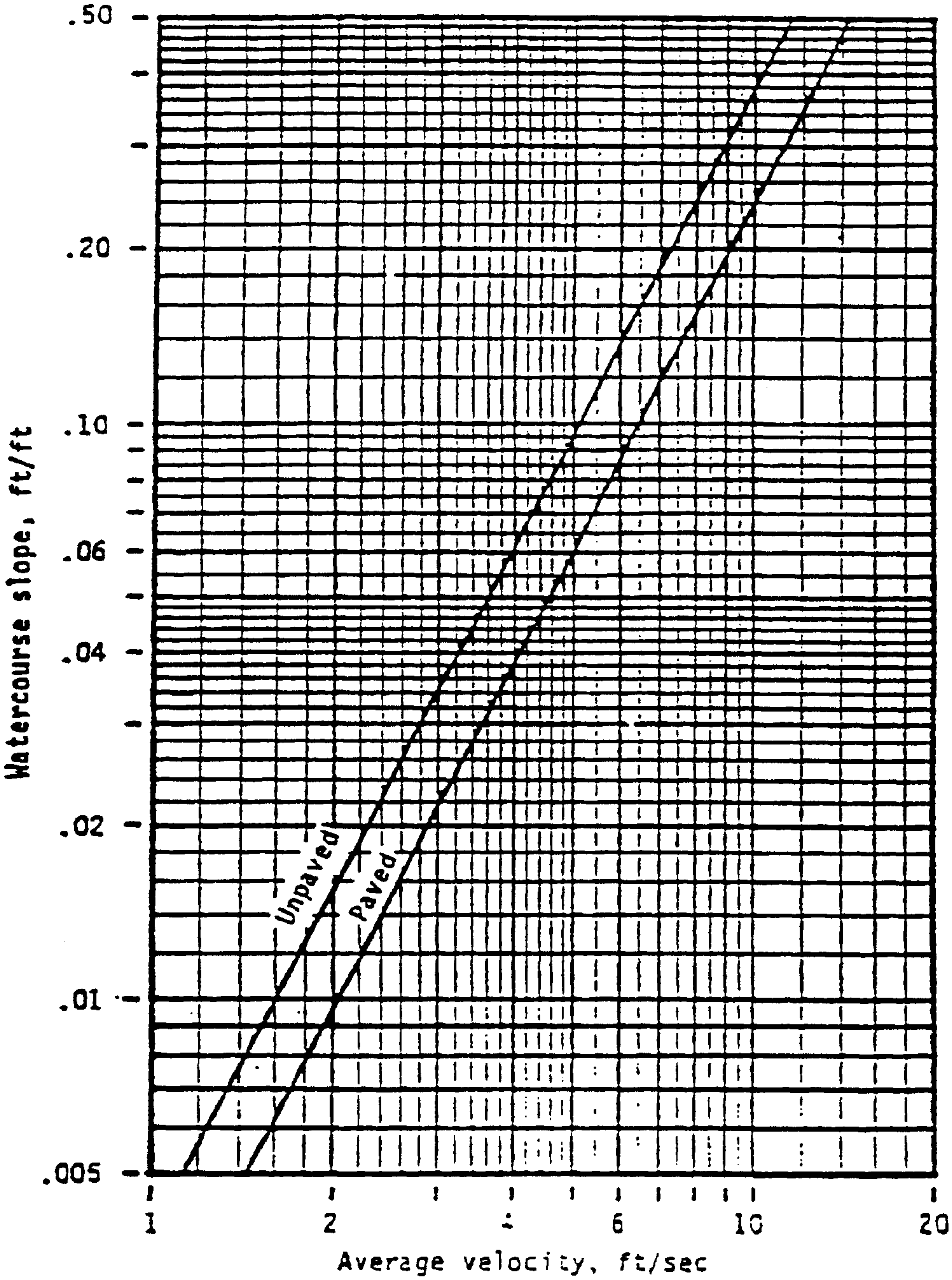
CHART E-3

ROUGHNESS COEFFICIENTS (MANNING'S N) FOR SHEET FLOW

Surface Description	n
Smooth surfaces (concrete, asphalt, gravel, or bare soil)	0.011
Uncultivated soil (no residue)	0.05
Cultivated soils:	
Residue cover < 20%	0.06
Residue cover > 20%	0.17
Grass:	
Short grass prairie	0.15
Dense grasses	0.24
Bermuda grass	0.41
Range (natural)	0.13
Woods:	
Light underbrush	0.40
Dense underbrush	0.80

*Source: Urban Hydrology for Small Watersheds, USDA-SCS 210-VI-Tr-55, Second Ed., June, 1986)

CHART E-4



(210-VI-TR-55), Second Ed., June 1986.

CHART E-6*

Intensity - Duration - Frequency
Southport, North Carolina

	RETURN PERIOD					
Duration	2-Year (in/hr)	5-Year (in/hr)	10-Year (in/hr)	25-Year (in/hr)	50-Year (in/hr)	100-Year (in/hr)
5 Min	7.10	8.66	10.21	11.77	12.92	13.20
10 Min	5.47	6.67	7.87	9.06	9.95	10.26
15 Min	4.61	5.62	6.62	7.63	8.38	8.64
30 Min	3.20	3.90	4.60	5.30	5.82	6.00
1 Hr	2.00	2.50	2.90	3.33	3.71	4.00
2 Hr	1.22	1.60	1.78	2.07	2.28	2.59
3 Hr	0.91	1.13	1.31	1.50	1.67	1.85
6 Hr	0.52	0.67	0.77	0.90	1.00	1.13
12 Hr	0.31	0.40	0.47	0.54	0.60	0.67
24 Hr	0.17	0.23	0.28	0.32	0.35	0.40

*Source: Rainfall Frequency Atlas of the United States, Technical Paper 40, U.S. Weather Bureau, May, 1961.

SECTION F

PIPE SYSTEM DESIGN

F.1 Curb Inlet Design.

The curb inlet design procedure is based on a standard curb inlet with a 4-foot opening. The following procedure is used to locate inlets. Factors as outlined in Section D.2 shall be considered including location of low points, intersections, and layout of pipe system.

F.1.1 Determine maximum gutter flow from the nomograph (Chart F-l) "Flow in Triangular Gutter Sections." The known values of street longitudinal slope, **S**, street cross slope, **Sx**, allowable spread, **T=8 feet**, and roughness coefficient, **n**, are used to solve for the gutter capacity, **Q**.

F.1.2 Determine location of inlet using the rational formula rearranged so that:

$$\mathbf{A = Q/CI}$$

Locate first inlet by trial and error with planimeter and topographic map so that gutter flow does not exceed capacity determined above. Check selection of "**C**" and "**I**" for the actual area determined.

F.1.3 The next inlets are located utilizing Charts F-l and F-2. Determine the required curb opening length, **L**, and enter in Chart F-2, with the ratio **L/L** where **L** is the actual curb opening of 4 feet for standard curb inlets. This will yield the percentage of flow intercepted by each basin. The remainder of this flow shall be considered in determining the location of the next downstream inlet. This procedure is repeated as necessary. The designer is encouraged to use the Catch Basin Design and Data Sheet, Chart F-4.

F.2 Pipe Design.

After curb inlets have been preliminarily located, the pipe system may be designed in accordance with Section D.1. Pipes shall be designed based on Manning's equation for gravity pipe flow.

Determine area, intensity, and runoff coefficients for the rational formula to obtain the design flow for each section of pipe. (Note that the design is based on the sum of the individual area runoff and not the sum of the individual catch basin capacity). The designer is encouraged to use the Storm Drainage Design Data Sheet (Chart F-5) included in this manual.

Required pipe size may be determined by solving the following equation:

$$\mathbf{D = 16\,(Qn/s^{1/2})^{3/8}}$$

where:

D is minimum pipe diameter in inches
Q is design flow in cfs
n is Manning roughness coefficient
s is pipe slope in feet per foot

Check the preliminary design to ensure that velocity is within acceptable range (see D.1.5).

The following example demonstrates use of the modified Manning's equation solution.

Given Q = 20 cfs
Available slope is .4%
Pipe is concrete

Using n = .012 for concrete

$D = 16 (20 \times .012/ (.004)^{1/2})^{3/8}$

D = 26.38, therefore use 30" pipe

F.3 Stormwater Drainage Design.

The columns of the table are treated as follows:

1. **FROM** - The designation of the structure at the 'upper end of the pipe.

2. **TO** - The designation of the structure at the lower end of the pipe.

3. **SUBTOTAL AREA** - The drainage area contributing to the upstream end of the pipe, listed by designation. (ac)

4. **TOTAL AREA** - The total area (ac) draining to the upstream end of the pipe.

5. **INLET TIME** - Flow time (min) on the ground to the most remote inlet on the longest flow path (in terms of time) to the pipe being sized.

6. **PIPE TIME** - Sum of flow times (min) in pipe along the longest flow path (in terms of time) to the pipe being sized.

7. **TIME OF CONC. (Col 5 + Col 6)** - This is the longest flow time of all possible paths from the most remote point in the system to the upstream end of the pipe being sized. It is the time of concentration (min).

8. **INTENSITY** - The rainfall intensity (in/hr) for the design storm of interest and the time of concentration (Col 7), taken from the Intensity-Duration-Frequency Chart E-6 or computed from an IDF equation.

9. **RUNOFF COEF.** - The composite runoff coefficient for the areas in Col 3. This usually changes for the individual areas (see Table E-1).

10. **DISCHARGE** - The design discharge (cfs). Q = CIA. (CoI 9)*(CoI 8)*(Col 4)

11. **SLOPE** - Invert of pipe (ft/ft), as decided within profile constraints.

12. **DIA. theo.** - Theoretical minimum pipe diameter (in), from the following equation: $D = 16(Qn/s^{1/2})^{3/8}$.

13. **SIZE** - Standard pipe size (in), as selected - equal to or greater than Col 12.

14. **V full** - Full-flow average cross-sectional velocity (ft/sec), computed from the following equation: $V = (s^{1/2}D^{2/3})/8.9n$

15. **LENGTH** - Length (ft) of pipe segment of interest from either a map or given data.

16. **SEGMENT TIME** - Flow time (min) through pipe segment of interest, (CoI 15)/((CoI 14)*60).

17. **UPPER INVERT** - Invert elevation of the upper end of the pipe of interest, set by reference to upstream pipes and cover requirements.

18. **LOWER INVERT** - Invert elevation of the lower end of the pipe of interest, set equal to (Col 17)-((Col 11)*(Col 15)). Check for adequate cover; revise slope (Col 11) if necessary.

19. **TOP ELEVATION** - Ground elevation at upstream end, for reference.

Note: Make sure that the minimum design standards as outlined in Section D.1 are met.

F.4 Culvert Design.

Whenever open channels are used to convey stormwater, it may be necessary to cross under a roadway using a culvert. The culvert shall be designed to meet several hydraulic conditions based on headwater depth, full or partial flow, roughness, slope, entrance and exit types, and tail water depth.

Laboratory tests and field observations have determined that all these conditions can be grouped into two control conditions, inlet and outlet. Both types shall be considered separately in the design of culverts.

F.4.1 Inlet Control. Inlet control exists when the culvert barrel is not flowing full the entire length. The discharge capacity is controlled at the entrance by the depth of headwater (depth from culvert invert) and entrance geometry. Nomographs for inlet control are provided as Charts F-6 through F-10.

F.4.2 Outlet Control. Outlet control exists when the culvert barrel is flowing full the entire length, or only part of the length. This is why it is necessary to design for both types of control. The controlling factors in outlet control are tail water elevation in the outlet channel, slope, roughness, and length of culvert barrel.

Nomographs for outlet control are provided as Charts F-11 through F-15. In order to use these charts, it is necessary to determine the coefficient of entrance loss, **Ke**. Table F-1, Entrance Loss Coefficients, is provided in order to determine the value of **Ke**.

To use the nomographs, enter the chart with a known pipe length and entrance coefficient on the curved portion and a trial pipe size. Extend a line between these two points on the respective graphs and mark the intersection with the Turning Line. Using the known flow, draw a line from the flow graph through the mark on the Turning Line and extend to the Head graph. Read head value, **H**, from the chart and use the following equation to compute headwater, **HW**, for the given situation:

$$\mathbf{HW = H + ho - sL}$$

where:

H is Head as determined from the nomograph
ho is the tail water depth at the culvert outlet
s is the pipe slope
L is the length of the culvert

Compare the headwater values determined from the inlet control and outlet control procedures. The larger of the two is the controlling factor in culvert capacity.

TABLE F-1

ENTRANCE LOSS COEFFICIENTS*

Type of Structure and Design of Entrance	Coefficient, Ke
Pipe, concrete	
Projecting from fill, socket end (grooved end)	0.2
Projecting from fill, square cut end	0.5
Headwall or headwall and wingwalls	
Socket end of pipe	0.2
Square-end	0.5
Rounded (radius = 1/12D)	0.2
Mitered to conform to fill slope	0.7
End section conforming to fill slope	0.5
Pipe or pipe-arch, corrugated metal	
Projecting from fill (no headwall)	0.9
Headwall or headwall and wingwalls	
Square edge	0.5
Mitered to conform to fill slope	0.7
End section conforming to fill slope	0.5

*Source: Design of Culverts, NRCD-Land Quality, August 1986.

The following example demonstrates the use of the Nomographs provided:

Given:

Q = 89 cfs
Length of culvert = 100 ft
Pipe Slope = 2.0%
Inlet Type - Square Edge with Headwall
Pipe Type - Use RCP

Sizing for inlet control from the Chart F-6, a concrete pipe 48 inches in diameter has a headwater depth (HW/D) of 1.15 diameters or 4.6 feet. Check the embankment height to ensure at least one foot of free board remains. If inlet control design is satisfactory, check the 48-inch pipe for outlet control. Determine entrance loss coefficient, **Ke**, from the preceding table. For concrete pipe with square edge and headwall, **Ke** is shown to be 0.5. Find the length of pipe on Chart F-11 for **Ke** = 0.5. Draw a line from this point to the size of pipe, which for this example would be 48 inch. Mark the intersection of this line with the Turning Line. Next, draw a line from the known discharge of 89 cfs through the Turning Line at the mark previously made and extend to the line which gives a value of head in feet. For this example, the value of head, **H** is 1.5 feet. Assume since no information is available that the tail water depth, **ho**, is equal to the diameter of the culvert. Compute the actual headwater depth as follows:

HW = H+ ho -sL
HW = 1.5 + 4.0 - (.02) (100)
HW = 3.5

Therefore, inlet control is limiting actual flow since headwater depth is 4.6 feet. Use of 48-inch culvert is acceptable in this example.

When culverts are flowing full and the outlet is under tail water conditions, outlet control charts should be used. The friction coefficient and other values for the pipe section should be estimated and the elevation (**Za**) of the upstream point on the HGL should be determined. This elevation should become the reference elevation (**Zb**) of the next section upstream.

F.5 Energy Dissipaters.

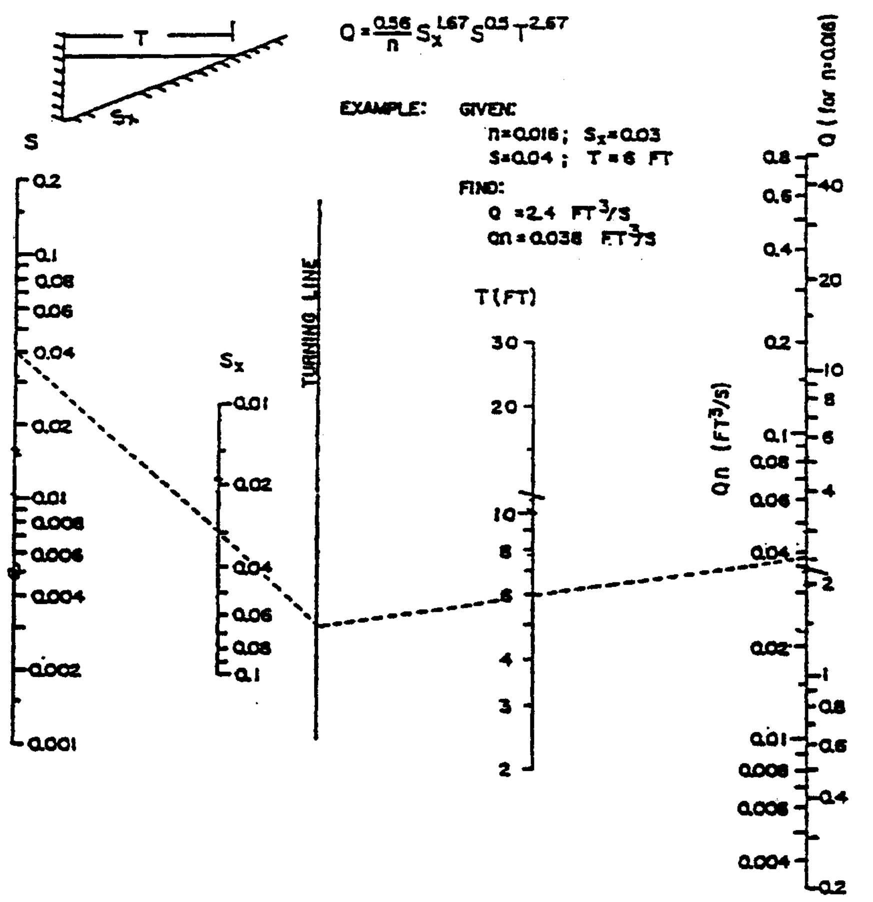
Energy dissipaters are de-energizing devices and/or erosion resistant channel sections provided between storm drain or culvert outlets and existing downstream channels, to provide for stable flow transitions and reduce the velocity of stormwater discharges sufficient to prevent erosion of the receiving channel.

These devices are needed for any storm drain outlet, culvert outlet, or channel outlet where the receiving channel or discharge area is subject to erosion.

Size of riprap as well as the velocity of the outgoing water from the culvert controls length and width of apron. A simple and widely used method for estimating the stone size and dimensions for culvert aprons is provided in Charts F-16, F-17, and F-18.

The velocity of flow can be determined by dividing the known flow (**Q**) by the culvert area: **V = Q/A**. Follow the steps on the charts to determine apron length to prevent scour at the outlet in questions. The width of stone apron at the downstream edge should be approximately equal to the length except where conformance to the receiving channel is necessary. Chart F-17 is included as a guide to determine stone size, however, readily available stone sizes may be used with judgment.

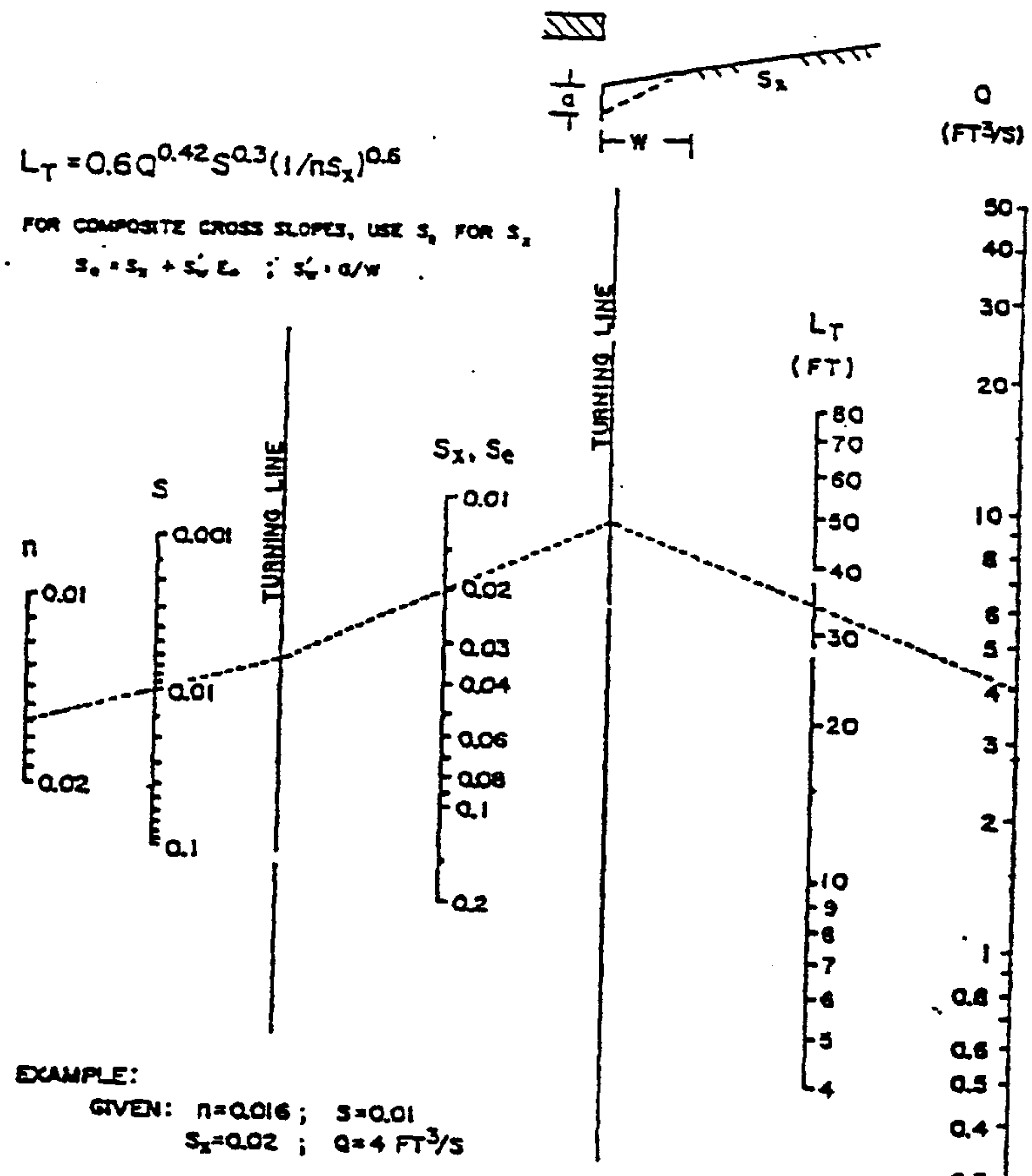
CHART F-1



Source: FHWA-TS-84-202

Flow in triangular gutter sections.

CHART F-2



EXAMPLE:

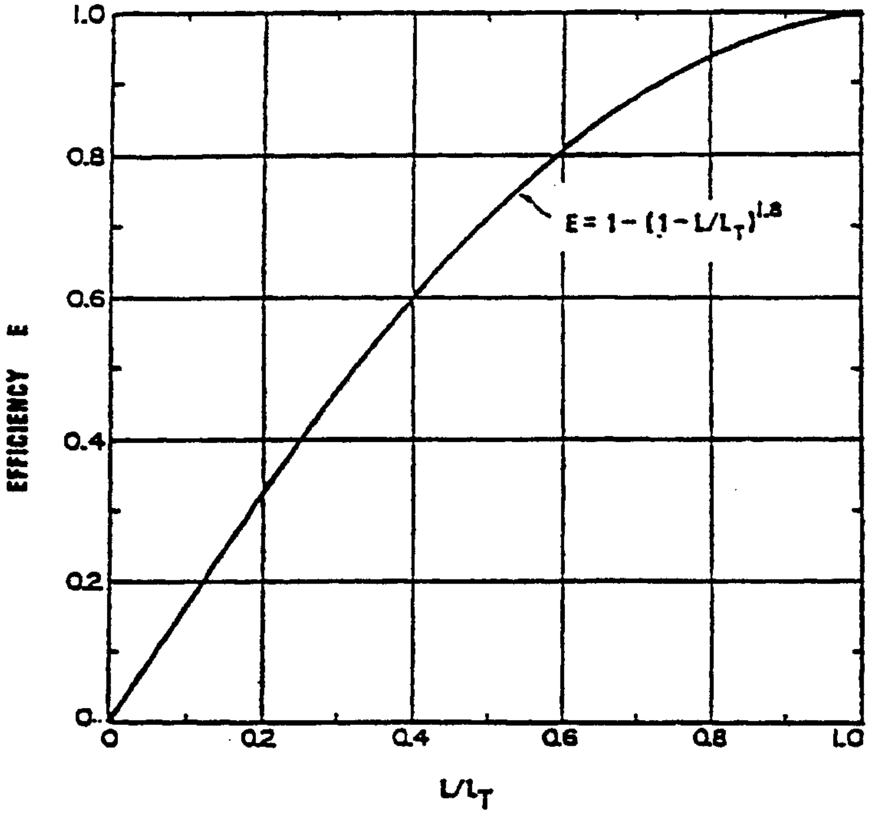
GIVEN: n=0.016 ; S=0.01

S_x=0.02 ; Q=4 FT³/S

FIND: L_T= 34 FT

Source: FHWA-TS-84-202

CHART F-3



Source: FHWA-TS-84-202

CATCH BASIN DESIGN DATA SHEET

PROJECT____________ COMPUTED BY____________ DATE______, ____.

LOCATION____________ CHECKED BY____________ SHEET____ OF____.

STORM FREQUENCY____ YRS.

	1	2	3	4	5	6	7	8	9	10	11	12	13	
CATCH BASIN NO.	GUTTER SPREAD	LONGITUDINAL STREET GRADE (S_T)	MAX. GUTTER FLOW (C.F.S.)	INTERCEPTED FLOW (C.F.S.)	TRIAL COEF. OF RUNOFF	TRIAL INTENSITY (in.)	DERIVED AREA $= \frac{4}{5 \times 6}$	LENGTH OF DRAINAGE AREA	HEIGHT ABOVE MOST REMOTE POINT	COEF. OF RUNOFF	TIME OF CONCENTRATION (T_C)	ACTUAL INTENSITY (in.)	MAX. ALLOWABLE DRAINAGE AREA $= \frac{4}{10 \times 12}$	REMARKS (COLUMN 13 MUST BE W/N 10% OF COLUMN 7)

CHART F-4

CHART F-5

STORM DRAINAGE DESIGN DATA SHEET

PROJECT: ______________ COMPUTED BY: ______________ DATE ______________

LOCATION: ______________ CHECKED BY: ______________ SHEET ________ OF ______

STORM FREQUENCY ______ YRS S = ________ h = ________ PIPE TYPE: ______________ N: ______

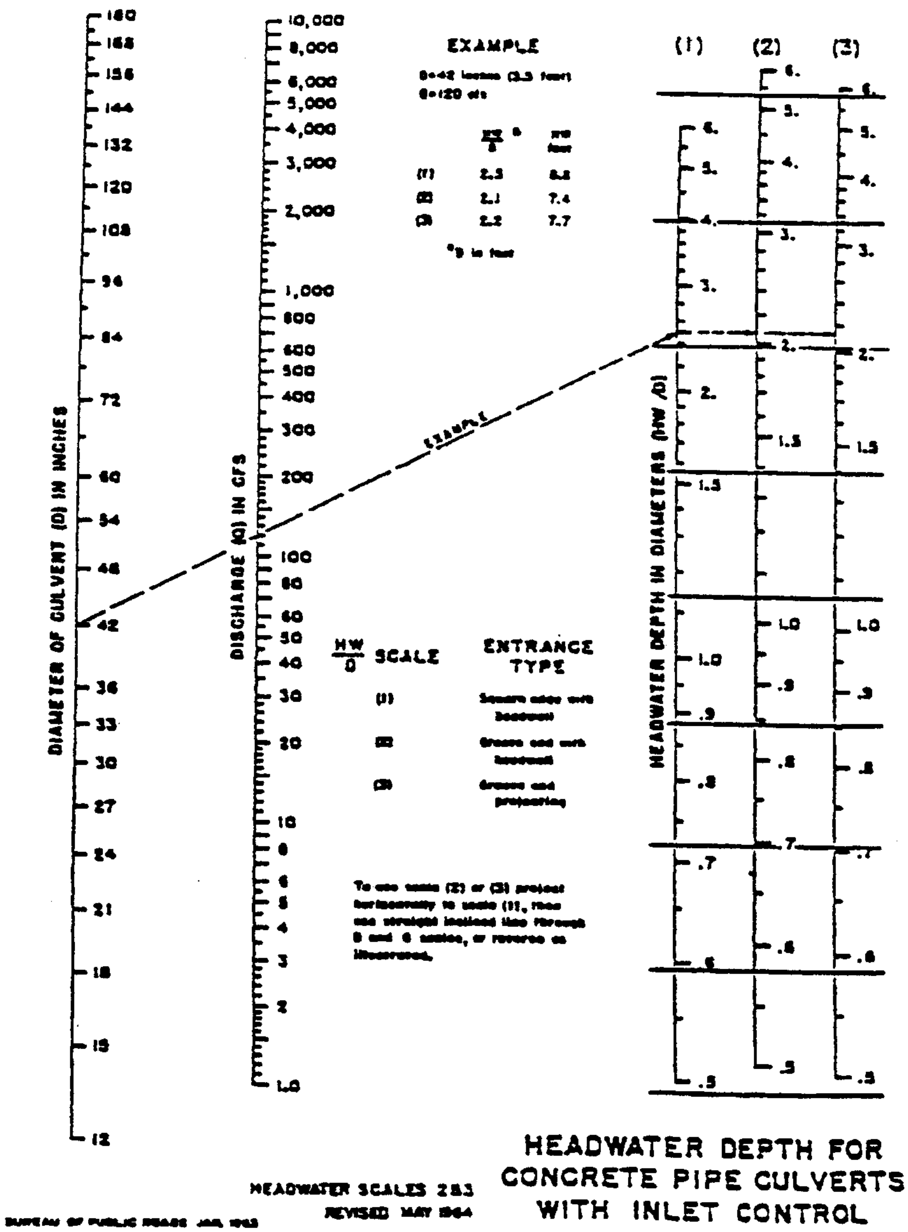
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19
LOCATION		AREA				tc		C		PIPE DATA								
FROM	TO	SUB TOTAL (acres)	TOTAL (acres)	INLET TIME (min)	PIPE TIME (ft)	TIME OF CONC. (min)	I INTENSITY (in/hr)	RUNOFF COEF.	Q=CIA (cfs)	SLOPE %	DIA. [illegible] (in)	SIZE (in)	V full (fps)	LENGTH (ft)	SEGMENT TIME (min)	UPPER INVERT (ft)	LOWER INVERT (ft)	UPSTRM TOP EL. (ft)

C = RUNOFF COEFFICIENT
H = HEIGHT ABOVE INLET OF MOST REMOTE POINT
L = LENGTH OF DRAINAGE AREA
I = INTENSITY

N = COEFFICIENT OF FRICTION
S = SLOPE (%)
Q = FLOW (CFS)

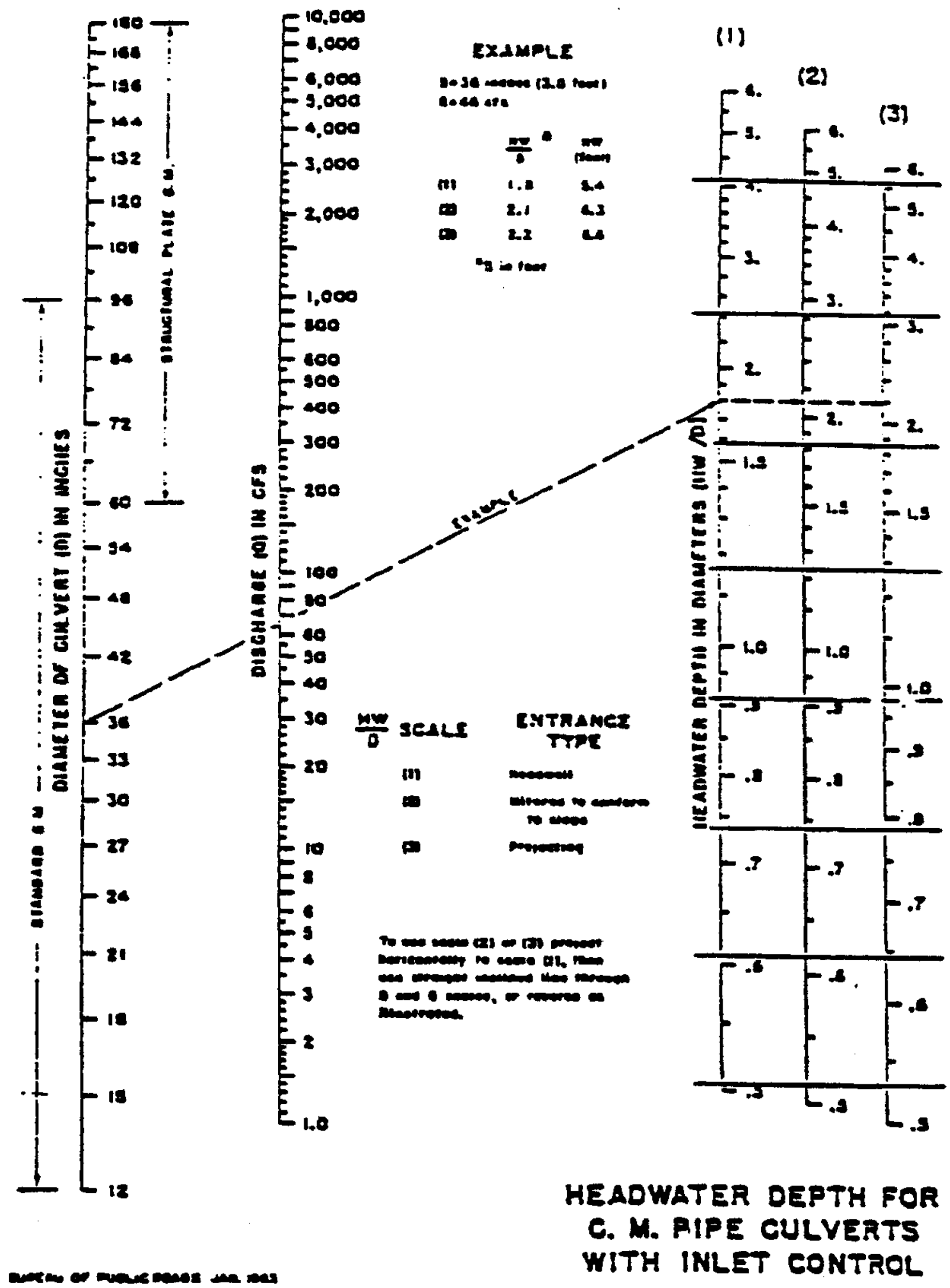
NOTE: DESIGN IS BASED ON THE SUM OF THE AREAS AND NOT THE SUM OF THE DISCHARGES

CHART F-6



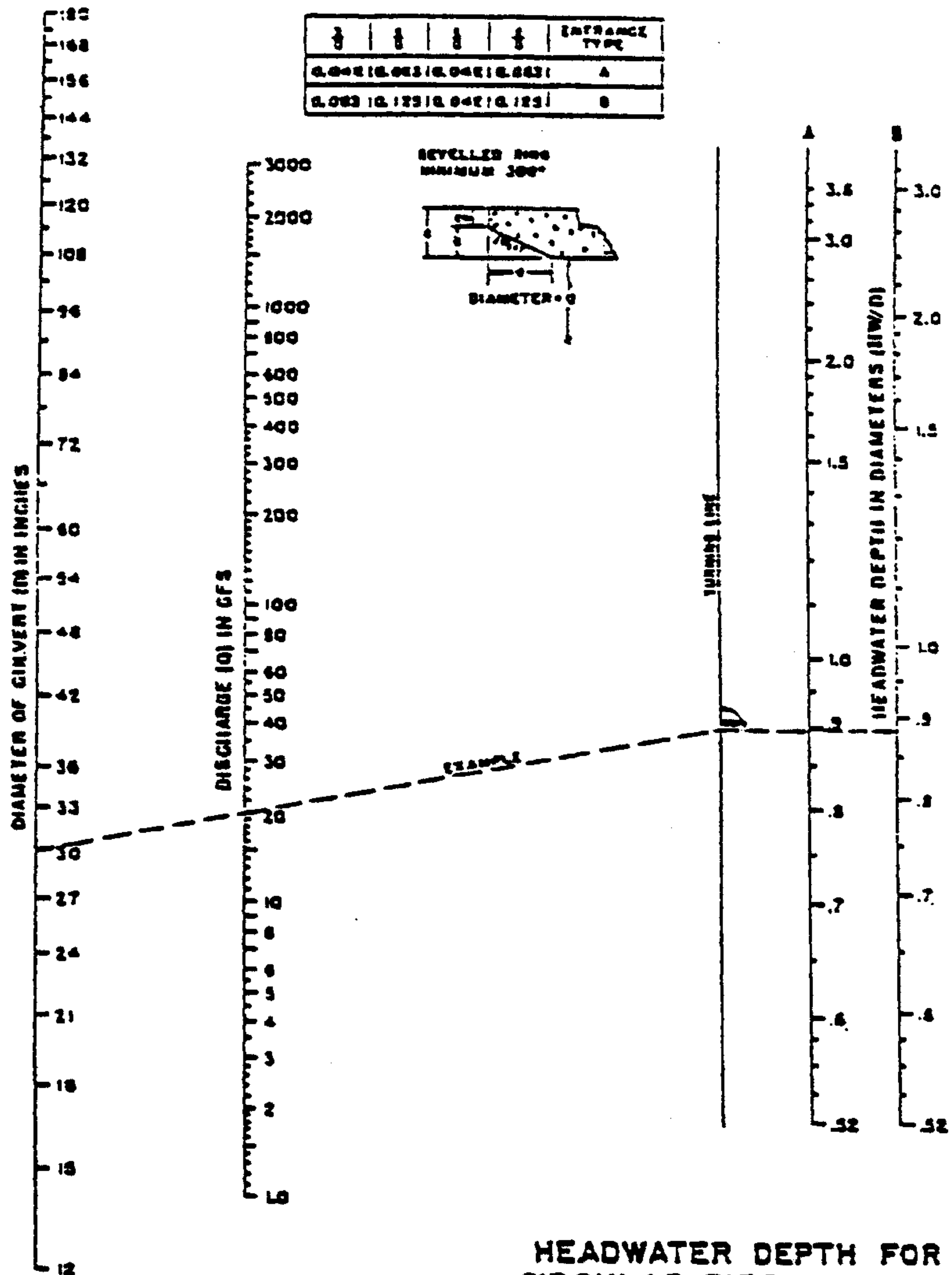
HEADWATER DEPTH FOR CONCRETE PIPE CULVERTS WITH INLET CONTROL

CHART F-7



EXAMPLE
HW/D SCALE
ENTRANCE TYPE
(1)
(2)
(3)
HEADWATER DEPTH FOR
C. M. PIPE CULVERTS
WITH INLET CONTROL

CHART F-8



ERAL HIGHWAY ADMINISTRATION
MAY 1973

HEADWATER DEPTH FOR CIRCULAR PIPE CULVERTS WITH BEVELED RING INLET CONTROL

CHART F-9

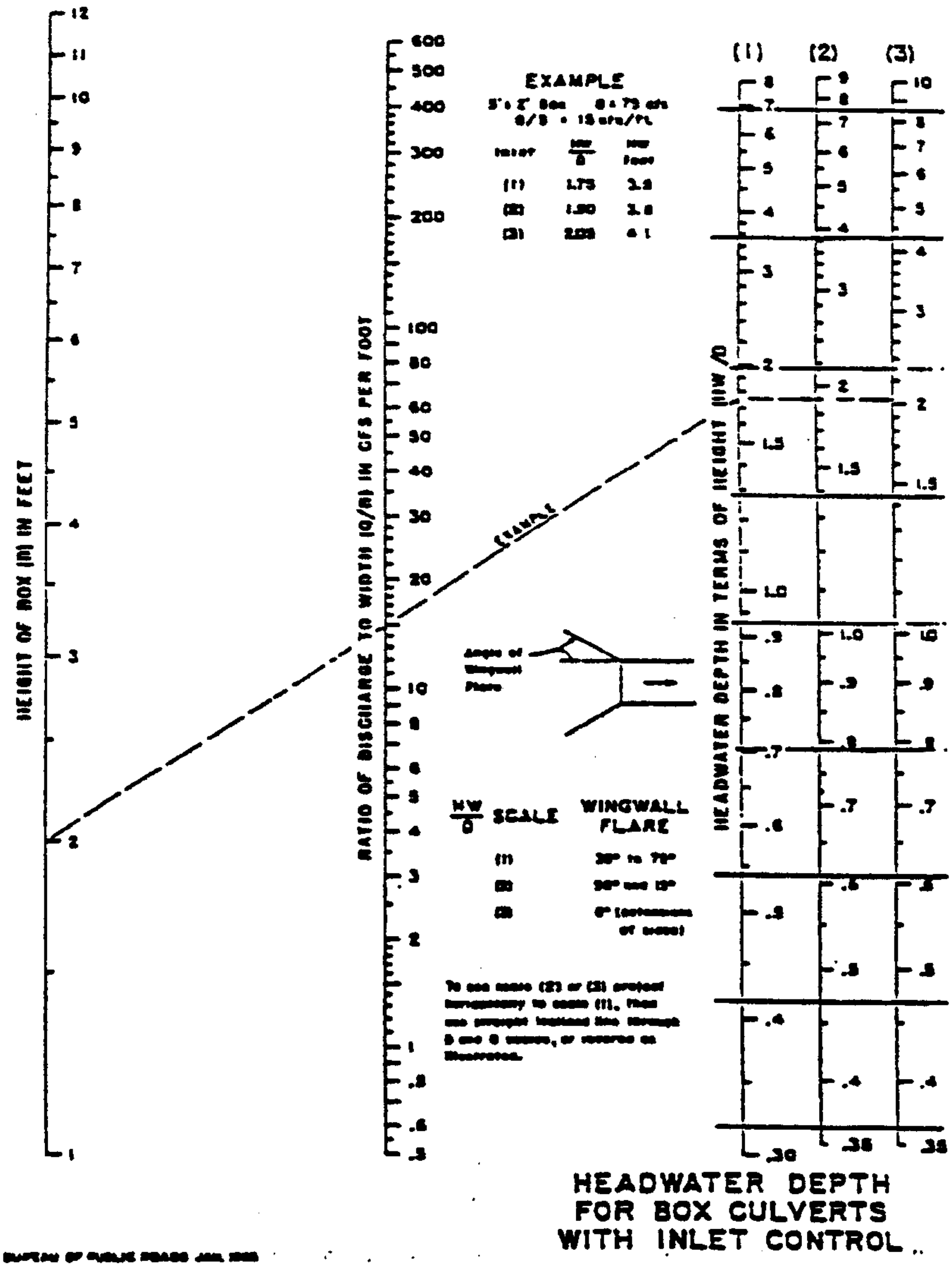
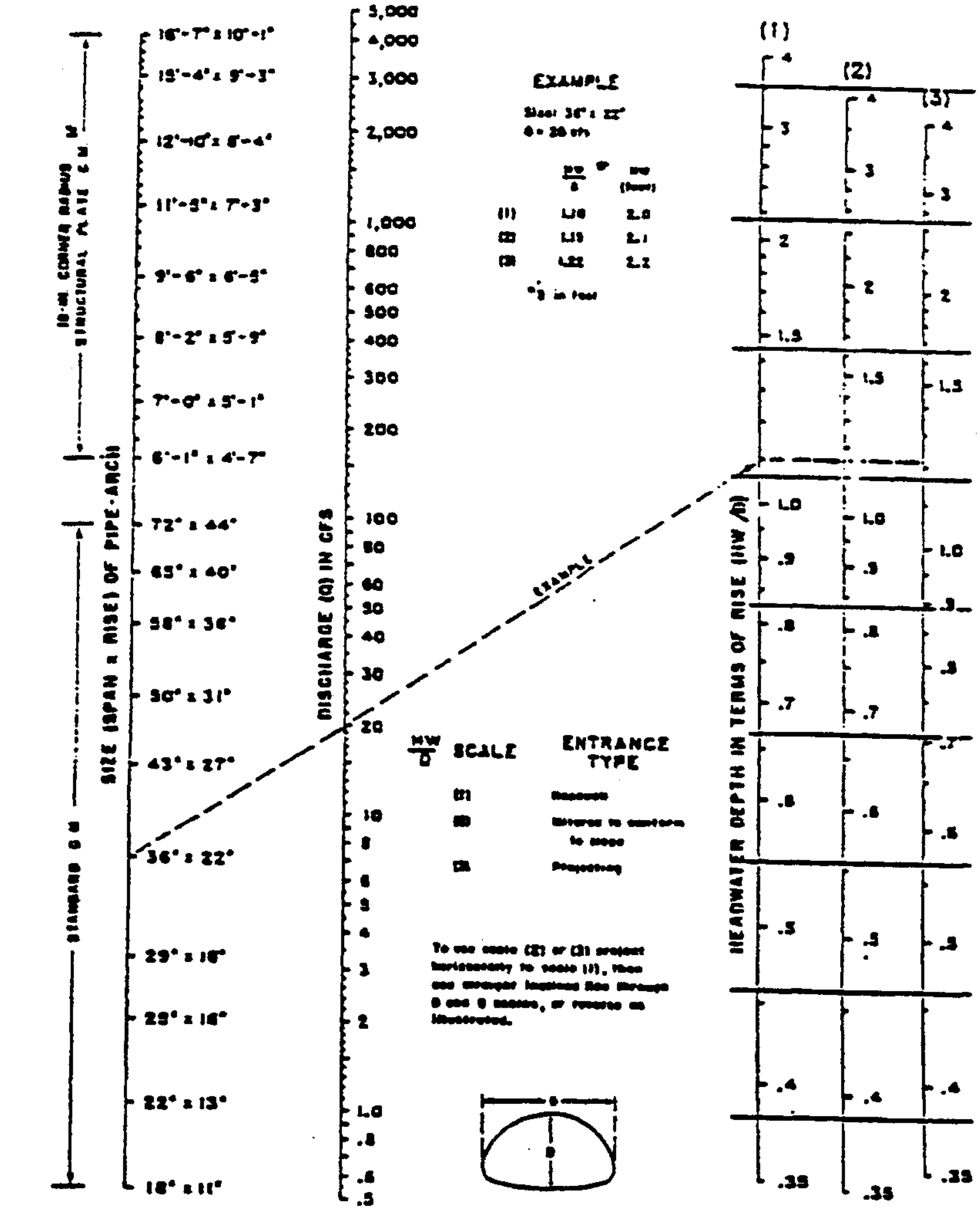

HEIGHT OF BOX (D) IN FEET
RATIO OF DISCHARGE TO WIDTH (Q/B) IN CFS PER FOOT
HEADWATER DEPTH IN TERMS OF HEIGHT (HW/D)
(1) (2) (3)
EXAMPLE
HW/D SCALE
WINGWALL FLARE
HEADWATER DEPTH
FOR BOX CULVERTS
WITH INLET CONTROL

CHART F-10



*ADDITIONAL SIZES NOT DIMENSIONED ARE LISTED IN FABRICATOR'S CATALOG

HEADWATER DEPTH FOR C. M. PIPE-ARCH CULVERTS WITH INLET CONTROL

CHART F-11

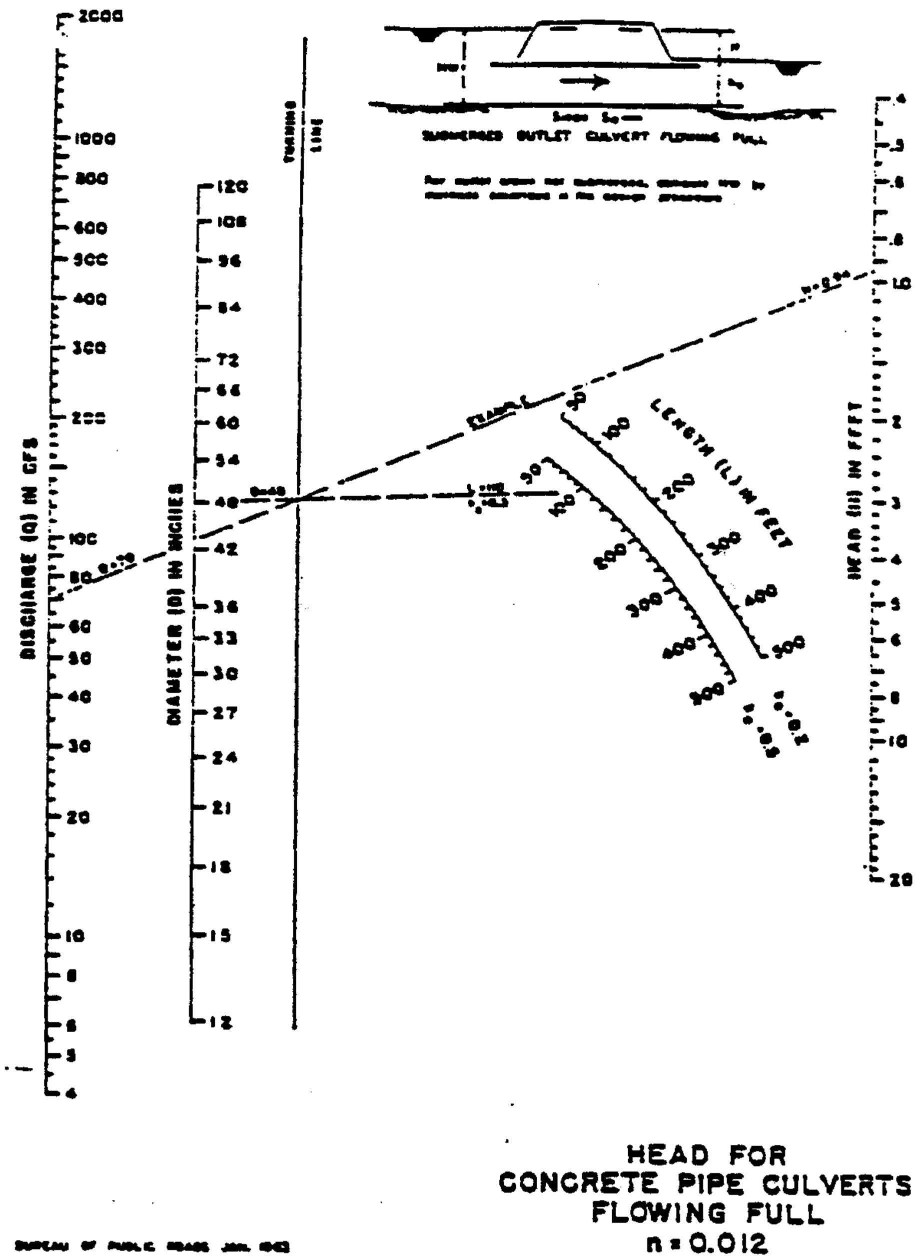


CHART F-12

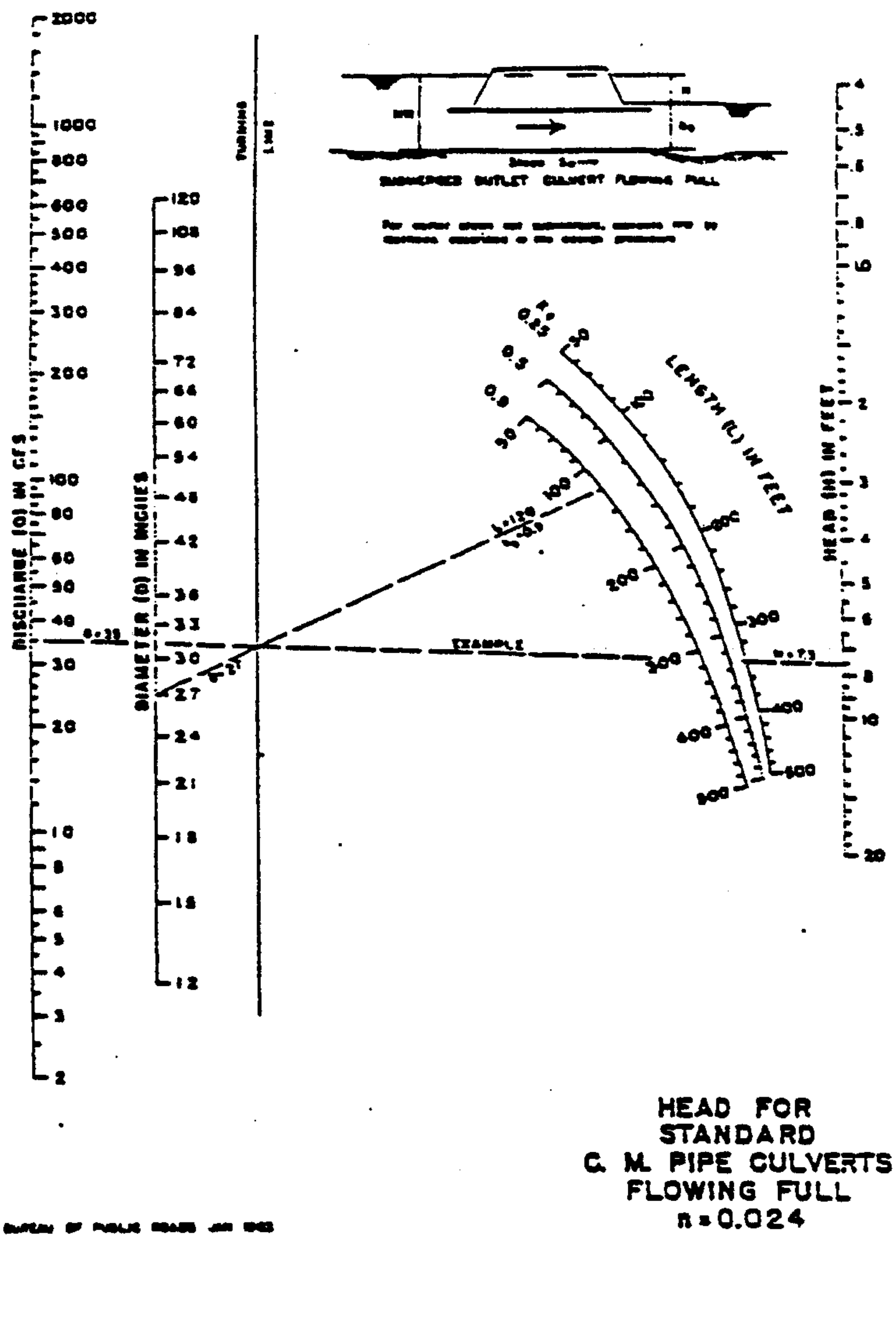
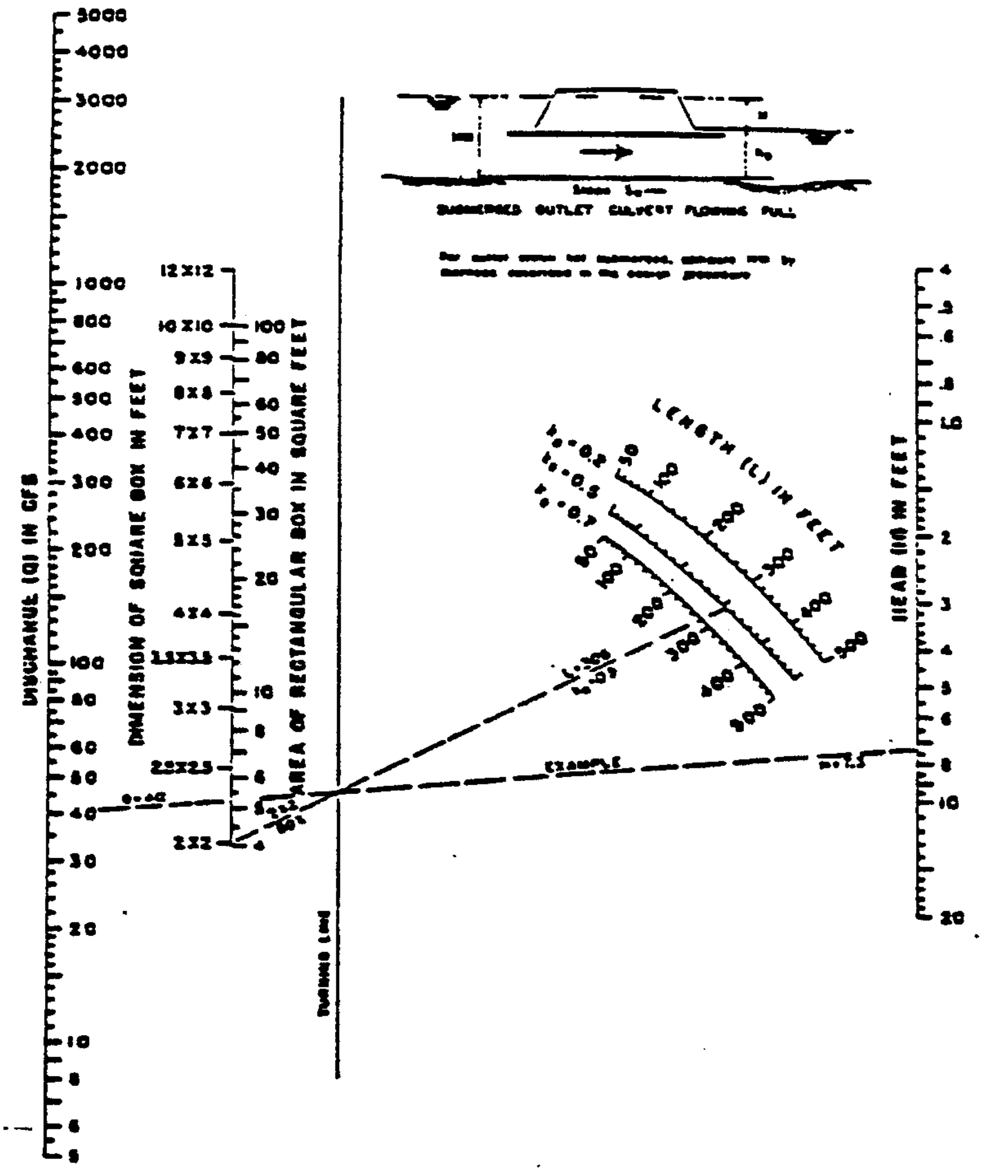
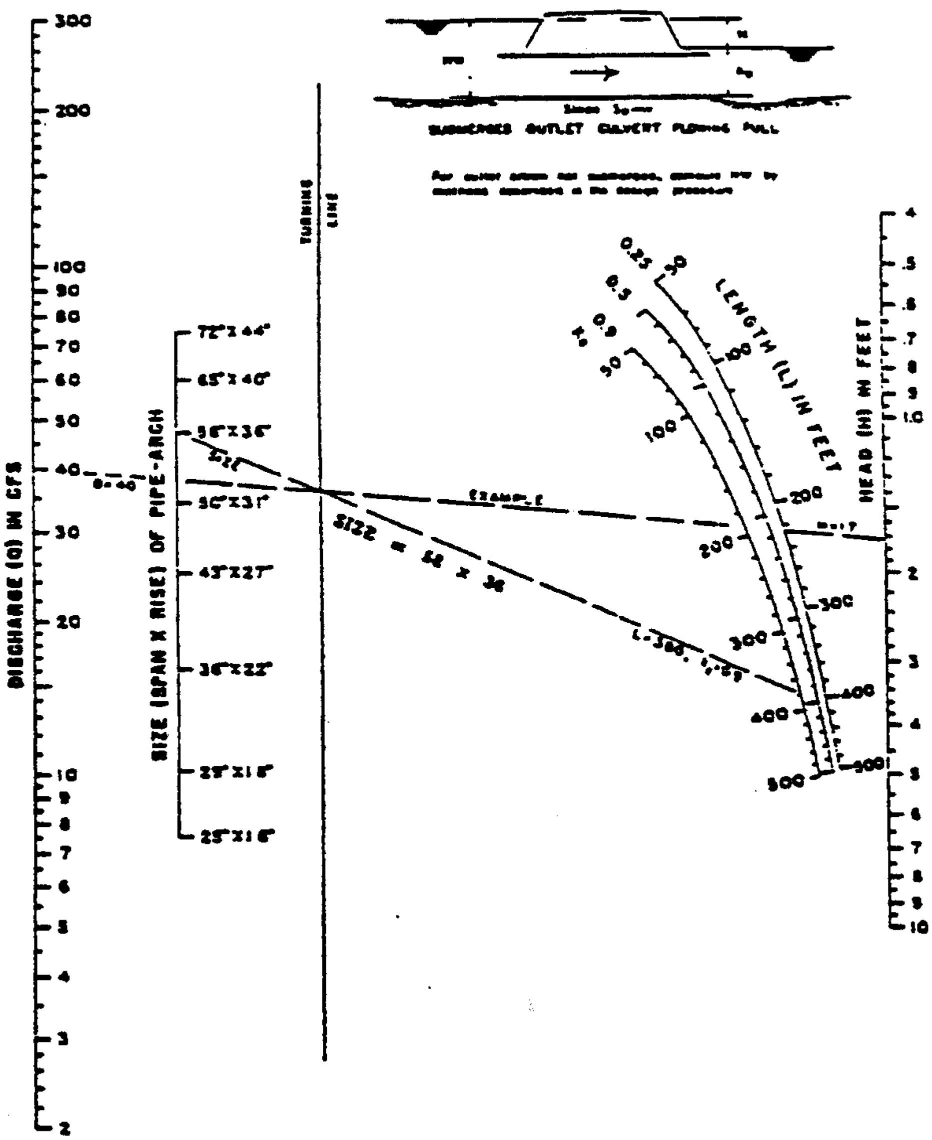

DISCHARGE (Q) IN CFS
DIAMETER (D) IN INCHES
TURNING LINE
SUBMERGED OUTLET CULVERT FLOWING FULL
LENGTH (L) IN FEET
HEAD (H) IN FEET
EXAMPLE
HEAD FOR STANDARD C. M. PIPE CULVERTS FLOWING FULL n = 0.024

CHART F-13



HEAD FOR
CONCRETE BOX CULVERTS
FLOWING FULL
n = 0.012

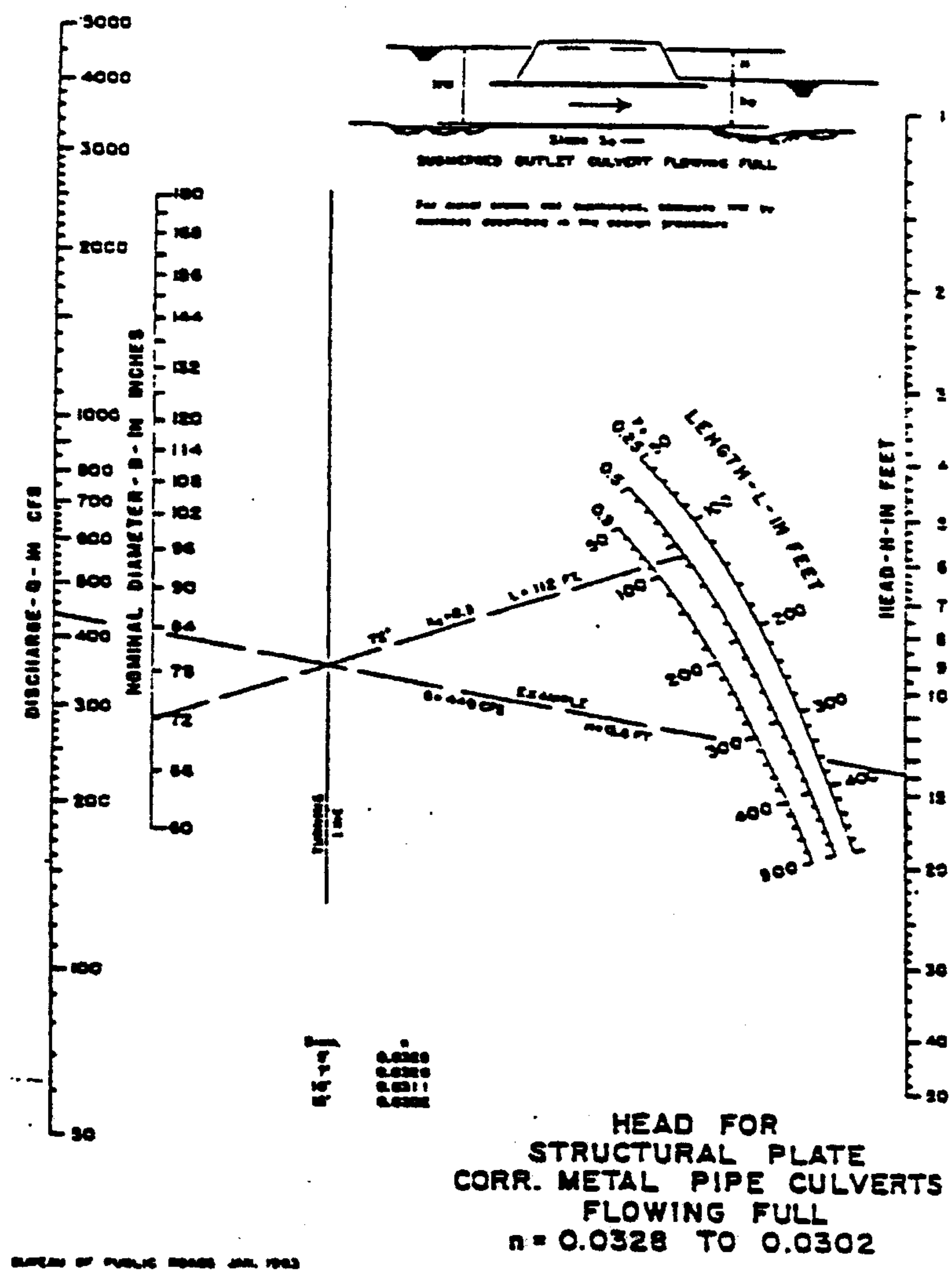
CHART F-14



HEAD FOR
STANDARD C. M. PIPE-ARCH CULVERTS
FLOWING FULL
n=0.024

BUREAU OF PUBLIC ROADS JAN. 1963

CHART F-15



DISCHARGE-Q-IN CFS
NOMINAL DIAMETER-D-IN INCHES
LENGTH-L-IN FEET
HEAD-H-IN FEET
SUBMERGED OUTLET CULVERT FLOWING FULL
EXAMPLE
HEAD FOR
STRUCTURAL PLATE
CORR. METAL PIPE CULVERTS
FLOWING FULL
n = 0.0328 TO 0.0302

CHART F-16

NEW YORK DOT DISSIPATER METHOD FOR USE IN DEFINED CHANNELS

(Source: "Bank and Channel Lining Procedures," New York Department of Transportation, Division of Design and Construction. 1971)

Note: To use the following graph you must know:

1. Q full capacity
2. Q_{10}
3. V full
4. V_{10}

Where Q = discharge in cfs and V = Velocity in fps.

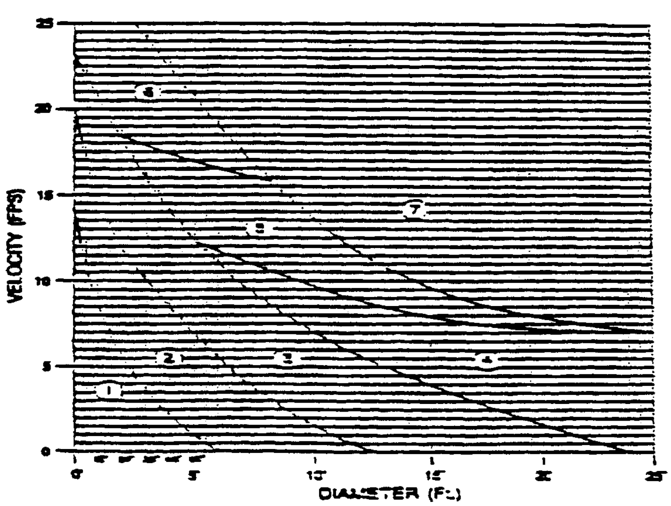
ESTIMATION OF STONE SIZE AND DIMENSIONS FOR CULVERT APRONS

Step 1) Compute flow velocity V_0 at culvert or paved channel outlet.

Step 2) For pipe culverts D_0 is diameter.
For pipe arch, arch and box culverts, and paved channel outlets, $D_0 = A_0$, where A = cross-sectional area of flow at outlet.

For multiple culverts, use $D_0 = 1.25 \times D_0$ of single culvert.

Step 3) For apron grades of 10% or steeper, use recommendations for next higher zone. (Zones 1 through 6).



(Source: Erosion and Sediment Control Planning and Design Manual, NC Department of NRCD, Land Quality Section, September 1988, Rev. 12/93).

CHART F-17

ZONE	APRON MATERIAL		LENGTH OF APRON: TO PROTECT CULVERT L1	LENGTH OF APRON: TO PREVENT SCOUR HOLE USE L2 ALWAYS L2
1	Stone Filling (Fine)	Class A	3 x Do	4 x Do
2	Stone Filling (Light)	Class B	3 x Do	6 x Do
3	Stone Filling (Medium)	Class 1	4 x Do	8 x Do
4	Stone Filling (Heavy)	Class 1	4 x Do	8 x Do
5	Stone Filling (Heavy)	Class 2	5 x Do	10 x Do
6	Stone Filling (Heavy)	Class 2	6 x Do	10 x Do
7	Special Study Required (Energy Dissipaters, Stilling Basing or Larger Size Stone)			

Width = 3 times pipe diameter (minimum)

DETERMINATION OF STONE SIZE FOR DUMPED STONE CHANNEL LININGS AND REVETMENTS

Step 1) Use Chart F-18a to determine maximum stone size
(e.g., for 12 fps = 20" or 550 lbs.)

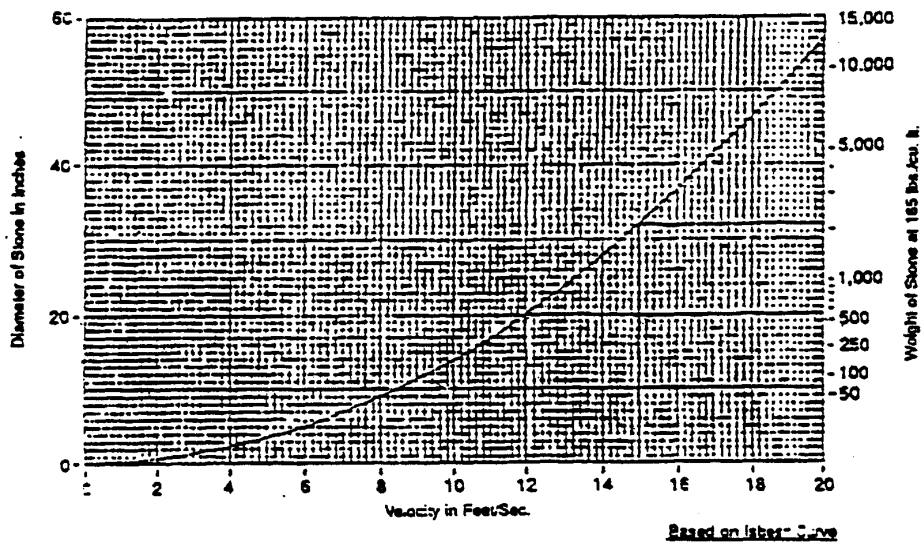
Step 2) Use Chart F-18b to determine acceptable size range for stone
(for 12 fps it is 125 - 500 lbs. for 75% of stone, and the minimum and maximum range in weight should be 25 - 500 lbs.)

NOTE: In determining channel velocities for stone linings and revetment, use the following coefficients of roughness:

	Diameter (inches)	Manning's "n"	Minimum of Lining	Thickness (inches)
Fine	3	0.031	9	12
Light	6	0.035	12	18
Medium	13	0.040	18	24
Heavy	23	0.044	30	36
			(Channels)	(Dissipaters)

Source: Erosion and Sediment Control Planning and Design Manual, NC Department of NRCD, Land Quality Section, September 1988, Rev. 12/93.

CHART F-18



F-18a Max. Stone Size for Rip Rap

MAXIMUM WEIGHT OF STONE REQUIRED (lbs.)	MINIMUM AND MAXIMUM RANGE IN WEIGHT OF STONES (lbs.)	WEIGHT RANGE OF 75% OF STONES (lbs.)
150	25 - 150	50 - 150
200	25 - 200	50 - 200
250	25 - 250	50 - 250
400	25 - 400	100 - 400
600	25 - 600	150 - 600
800	25 - 800	200 - 800
1,000	50 - 1,000	250 - 1,000
1,300	50 - 1,300	325 - 1,300
1,600	50 - 1,600	400 - 1,600
2,000	75 - 2,000	600 - 2,000
2,700	100 - 2,700	800 - 2,700

F-18b Gradation of Rip Rap

Source: "Bank and Channel Lining Procedures," New York Department of Transportation, Division of Design and Construction, 1971.

Source: Erosion and Sediment Control Planning and Design Manual, NC Department of NRCD, Land Quality Section, September, 1988. Rev. 12/93.

SECTION G

OPEN CHANNEL DESIGN

In this area, proper consideration and design of the channel is of utmost importance, due to the high degree of potential erosion caused by the loose, non-cohesive soils native to the area.

Typical linings used to protect channel slopes include grasses, grass reinforced with a stabilization mat, stone riprap, paved slopes, and concrete fabric-form. The city encourages any introduction of new proven methods for lining channels. The city or its authorized representative shall approve alternative linings.

G.1 Vegetative Channels

Typical channel cross-sections include the trapezoidal cross-section, triangular cross-section, and the parabolic cross-section. Chart G-2 may be used to determine geometric and hydraulic properties. Note that open channels to be dedicated to the city shall meet the geometric requirements stated in Section D. The triangular and parabolic section design guide is included for sites where these will be used as on-site water quality features for filtering and conveying stormwater.

Channel capacity shall be computed using the Manning's equation and trial and error solution. The basic equation is:

$$\mathbf{Q = 1.485\ a\ r^{2/3}\ s^{1/2}\ /\ n}$$

where:

Q is flow in cubic feet per second (cfs)
a is flow area in square feet (ft)
r is hydraulic radius which is equal to the channel area divided by the wetted perimeter,
s is the slope of the hydraulic grade fine or channel bottom in ft/ft
n is the roughness coefficient of the channel

Steps for determining channel design when using vegetative lining follows:

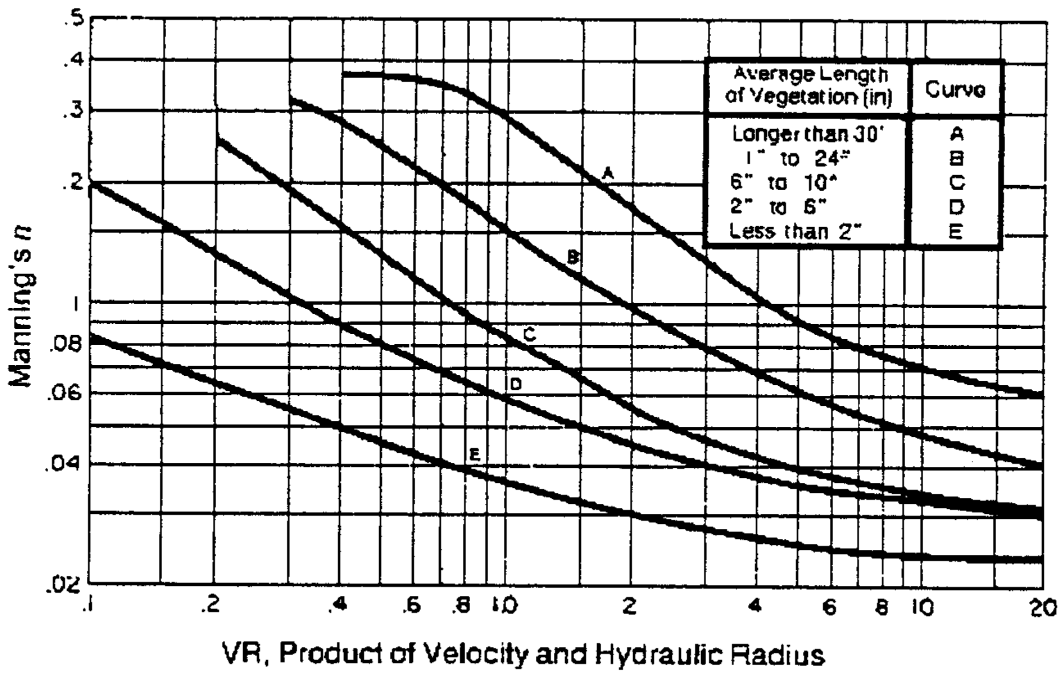
a. With the known flow rate, **Q**, and channel slope, **s**, select channel geometry type and lining type.

b. Determine permissible velocity from Chart G-3.

c. Estimate channel size by dividing **Q** by the permissible velocity. Then estimate channel width, depth and side slope to fit site conditions.

d. Compute the hydraulic radius of the trial channel configuration.

e. Determine Manning's **n** using vegetative retardance class from Chart G-4 and the Figure 8.05c, Manning's **n** related to velocity, hydraulic radius, and vegetal retardance, using the permissible velocity and hydraulic radius. It is recommended to use at least one retardance class higher than the one determined from the chart.



F i
gure 8.05c Manning's **n** related to velocity, hydraulic radius, and vegetal retardance.

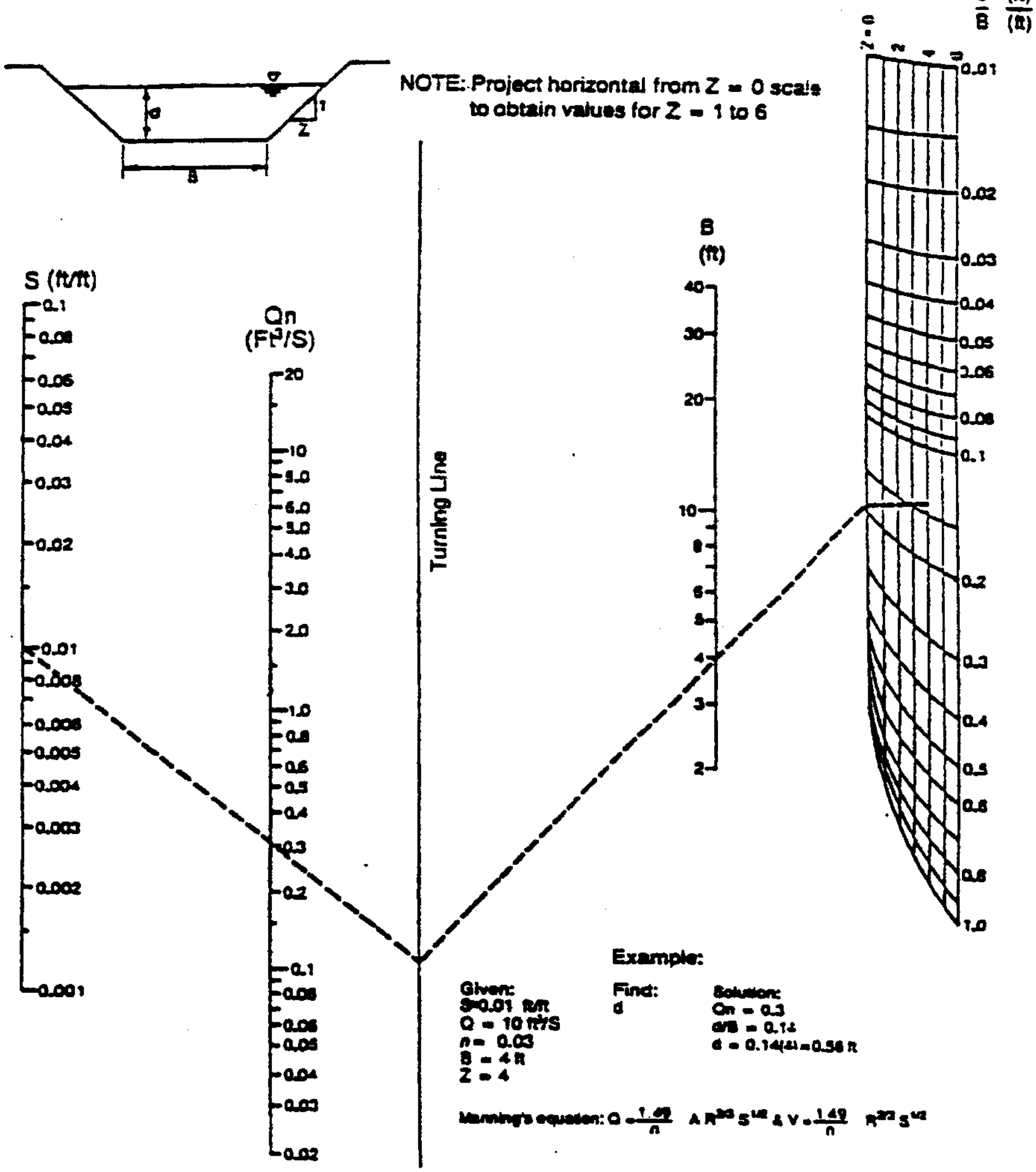
f. Calculate velocity and flow using Manning's equation. Compare results with permissible velocity and required capacity.

g. Repeat steps a. through f. as necessary adjusting channel geometry to obtain required capacity within velocity limitations.

Chart G-1 on the following page provides for the solution of Manning's equation for a trapezoidal channel with side slopes from 1 to 6 horizontal to 1 vertical. An example is shown in the lower right corner of the nomograph.

If the channel velocity exceeds 2 fps, a temporary lining may be required to stabilize the channel prior to the vegetation establishing. The procedure for designing temporary linings is the same as shown for rip rap lined channels.

CHART G-1

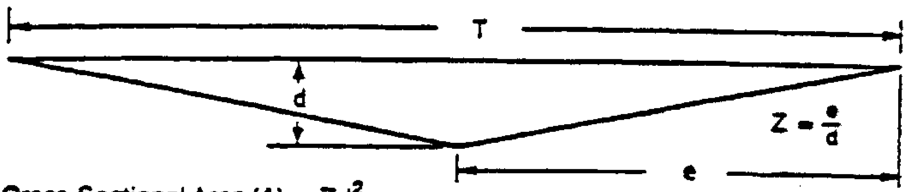


Solution of Manning's equation for trapezoidal channels of various side slopes.
Adapted from: FHWA-HEC. 15, Pg 40 - April 1988

*Source: Erosion and Sediment Control Planning and Design Manual, NC Department of NRCD, Land Quality Section, September 1988.

CHART G-2

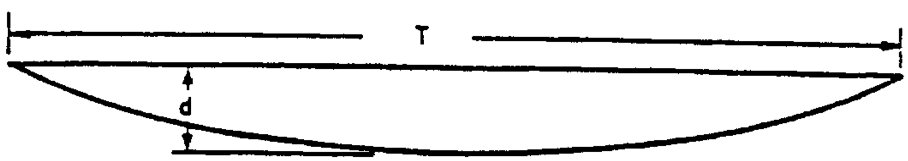
V-Shape



Cross-Sectional Area (A) = Zd^2
Top Width (T) = $2dZ$

Hydraulic Radius (R) = $\frac{Zd}{2\sqrt{Z^2 + 1}}$

Parabolic Shape

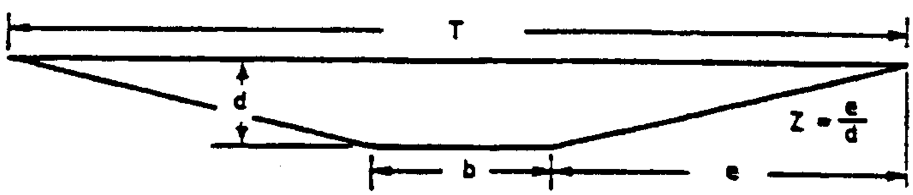


Cross-Sectional Area (A) = $\frac{2}{3}Td$

Top Width (T) = $\frac{1.5A}{d}$

Hydraulic Radius = $\frac{T^2d}{1.5T^2 + 4d^2}$

Trapezoidal Shape



Cross-Sectional Area (A) = $bd + Zd^2$
Top Width (T) = $b + 2dZ$

Hydraulic Radius = $\frac{bd + Zd^2}{b + 2d\sqrt{Z^2 + 1}}$

Channel geometries for v-shaped, parabolic and trapezoidal channels.

***Source: Erosion and Sediment Control Planning and Design Manual, NC Department of NRCD, Land Quality Section, September 1988.**

CHART G-3

Maximum Allowable Design Velocities for Vegetated Channels[1]

Typical Channel Slope	Soil Characteristics	Grass Lining	Permissible Velocity[3] for Established Grass Lining (ft/sec)
0-5%	Easily Erodible	Bermudagrass	5.0
	Non-plastic	Tall Fescue	4.5
	(sand & silts)	Bahiagrass	4.5
		Kentucky bluegrass	4.5
		Grass-legume mixture	3.5
	Erosion Resistant	Bermudagrass	6.0
	Plastic	Tall Fescue	5.5
	(clay mixes)	Bahiagrass	5.5
		Kentucky bluegrass	5.5
		Grass-legume mixture	4.5
5-10%	Easily Erodible	Bermudagrass	4.5
	Non-plastic	Tall Fescue	4.0
	(sand & silts)	Bahiagrass	4.0
		Kentucky bluegrass	4.0
		Grass-legume mixture	3.0
	Erosion Resistant	Bermudagrass	5.5
	Plastic	Tall Fescue	5.0
	(clay mixes)	Bahiagrass	5.0
		Kentucky bluegrass	5.0
		Grass-legume mixture	3.5
>10%	Easily Erodible	Bermudagrass	3.5
	Non-Plastic	Tall Fescue	2.5
	(sand & silts)	Bahiagrass	2.5
		Kentucky bluegrass	2.5
	Erosion Resistant	Bermudagrass	4.5
	Plastic	Tall Fescue	3.5
	(clay mixes)	Bahiagrass	3.5
		Kentucky bluegrass	3.5

*Source: USDA-SCS Modified, Erosion and Sediment Control Planning and Design Manual, NC Department of Natural Resources and Community Development, Land Quality Section, May 1994.

NOTE: [1]Permissible velocity based on 25-year storm peak runoff.
[2]Soil erodibility based on resistance to soil movement from concentrated flowing water.
[3]Before grass is established, the type of temporary litter used determines permissible velocity.

CHART G-4

RETARDANCE CLASSIFICATION FOR VEGETAL COVERS

RETARDANCE	COVER	CONDITION
A	Reed canarygrass	Excellent stand, tall (average 36")
	Weeping lovegrass	Excellent stand, tall (average 30")
B	Tall Fescue	Good stand, uncut (average 18")
	Bermudagrass	Good stand, tall (average 12")
	Grass-legume numre (tall fescue, red fescue, sericea lespedeza)	Good stand, uncut
	Grass mixture (timothy, smooth bromegrass, or orchardgrass)	Good stand, uncut (average 20")
	Sericea lespedeza	Good stand, not woody, tall (average 19")
	Reed canarygrass	Good stand, cut (average 12 - 15")
	Alfalfa	Good stand, uncut (average 11")
C	Tall Fescue	Good stand (8 - 12")
	Bermudagrass	Good stand, cut (average 6")
	Bahiagrass	Good stand, uncut (6 - 8")
	Grass-legume mixture summer (orchardgrass, redtop, and annual lespedeza)	Good stand, uncut (6 - 8")
	Centipedegrass	Very dense cover (average 6")
	Kentucky bluegrass	Good stand, headed (6 - 12")
	Redtop	Good stand, uncut (15 - 20")
D	Tall Fescue	Good stand, cut (3 - 4")
	Bermudagrass	Good stand, cut (2.5")
	Bahiagrass	Good stand, cut (3 - 4")
	Grass-legume mixture fall-spring (orchardgrass, redtop, and annual lespedeza)	Good stand, uncut (4 -5")
	Red Fescue	Good stand, uncut (12 - 18")
	Centipedegrass	Good stand, cut (3 - 4")
	Kentucky bluegrass	Good stand, cut (3 - 4")
E	Bermudagrass	Good stand, cut (1.5")
	Bermudagrass	Bumed stubble

Modified from: USDA-SCS, 1969, Engineering Field Manual.

G.2 Riprap Lined Channels

The important consideration in design of riprap linings is that the velocity is not so great that the stones become dislodged. Chart G-5 is provided to determine Manning's "**n**" coefficient at different depths of flow.

The method in choosing the size riprap and depth is trial and error. The basic equation for use in design is:

$$\mathbf{T = (y)\,(d)\,(s)}$$

where:

T is shear stress in lb/ft
y is 62.4 lb/ft, density of water
d is flow depth in feet
s is channel slope

The value of "**d**" is determined by the procedure described above or by using the nomograph for trapezoidal channels (Chart G-1). Once the depth is determined, the assumed roughness coefficient should be checked with Chart G-5. With the known channel slope, **s**, compute **T** using the above equation. This computed value should be less than **Td** as shown in Chart G-6.

CHART G-5

Manning's Roughness Coefficient "n"

		"n" value for Depth Ranges		
Lining Category	**Lining Type**	**0-0.5 ft (0-15 cm)**	**0.5-2.0 ft (15-60 cm)**	**>2.0 ft (>60 cm)**
Rigid	Concrete	0.015	0.013	0.013
	Grouted Riprap	0.040	0.030	0.028
	Stone Masonry	0.042	0.032	0.030
	Soil Cement	0.025	0.022	0.020
	Asphalt	0.018	0.016	0.016
Unlined	Bare Soil	0.023	0.020	0.020
	Rock Cut	0.045	0.035	0.025
Gravel Riprap	1-inch (2.5-cm) D_{50}	0.044	0.033	0.003
	2-inch (5-cm) D_{50}	0.066	0.041	0.034
Rock Riprap	6-inch (15-cm) D_{50}	0.104	0.069	0.035
	12-inch (30-cm) D_{50}		0.078	0.040

CHART G-6

PERMISSIBLE SHEAR STRESSES FOR RIPRAP AND TEMPORARY LINERS

Lining Category	Lining Type	Permissible Unit Shear Stress, Td (lb/ft)
Temporary	Woven Paper Net	0.15
	Jute Net	0.45
	Fiberglass Roving:	
	Single	0.60
	Double	0.85
	Straw with Net	1.45
	Curled Wood Mat	1.55
	Synthetic Mat	2.00
Gravel	D_{50} Stone Size (inches)	
	1	0.33
	2	0.67
	3	2.00
Rock Riprap	6	3.00
	9	4.00
	12	5.00
	15	6.00
	18	7.00
	21	7.80
	24	8.00

*Source: Erosion and Sediment Control Planning and Design Manual, NC Department of NRCD, Land Quality Section, December 1993.

G.3 Alternative Liners

When the velocity in an open channel exceeds permissible velocities for vegetation shown in Chart G-3, it becomes necessary to use a permanent protective lining on the slopes and bottom. Riprap linings discussed in the previous section provide very economical and dependable protection for most flows if sized properly. There are other linings available, such as fabric-form concrete paving, gabions, brick, or a combination of any of these (paved channels with grassed side slopes), that may be preferred due to cost, appearance, maintenance, or dependability.

Chart G-7 provides the Manning's roughness coefficient "**n**" to assist in the analysis of open channels.

CHART G-7

Values of "n" in Manning's Formula

Lining	"n"
Brick	.013 - .015
Concrete	
Trowel Finish	.013
Float Finish	.015
Unfinished	.017
Concrete, Bottom Float Finished, with Sides of	
Dressed Stone	.017
Random Stone	.020
Cement Rubble Masonry	.025
Dry Rubble or Riprap	.030
Fabric-Form (Uniform Cross-Section)	.015
Fabric-Form (Filter Points)	.025 - .030
Gabions	.030 - .035
Gravel Bottom, with Sides of	
Random Stone	.023
Riprap	.033

*Sources: F.E. McJunkin and P.A. Vesilind, "Practical Hydraulics for the Public Works Engineer," Reprinted from Public Works Magazine
Fabric Forms for Concrete, Tri-State Consultants, 1988
The Reno Mattress, @caferri Gabions, 1983

SECTION H

STORMWATER DETENTION PONDS

H.1 Design Objective

Stormwater detention ponds shall be designed so that the peak discharge from the site for the 25-year storm after development shall not exceed the peak discharge from the same site for the 25-year storm prior to development. An emergency spillway shall be provided so that it can handle the 100-year storm assuming the principal spillway is obstructed or not operating properly. The elevation of the top of the dam shall be a minimum of 0.5 feet above the peak water surface elevation for the 100-year storm.

H.2 Submission Requirements

The information outlined below shall accompany stormwater detention facility designs submitted for approval based on the size of watershed.

H.2.1 Information required for facilities where the watershed is one acre or greater.

a. A plan showing the drainage area of the watershed in sufficient detail to confirm the area and composition.

b. A site plan showing the detention basin with complete construction details.

c. Detailed computations of design discharges, including the peak 25-year discharge prior to development, the peak 100-year discharge after development, and the peak 25-year discharge after development.

d. Detailed computations of the stage-storage relationship, including a graph or table of water-surface elevation versus volume of storage.

e. Detailed computations of the stage-discharge relationship, including a graph or table of water surface elevation versus total discharge through the outlet system.

f. Detailed computations of flood routing for the 25-year storm in suitable time increments, including a table showing at least the four columns of elapsed time, accumulated storage, stage, and discharge.

g. Supporting calculations for the arrangement of the emergency spillway, particularly showing the determination of the elevations of the crest and top of dam and the length of the spillway.

h. Detailed calculations for energy dissipaters design and outlet velocities for the 25-year storm.

i. Evaluation of composite hydrographs in the downstream system when required by the city.

H.2.2 Information required for facilities where the watershed is less than 1 acre.

a. A plan showing the drainage area of the watershed in sufficient detail to confirm the area and composition.

b. A site plan showing the detention basin with complete construction details.

c. Computations of the peak 25-year discharge prior to development and the 25-year discharge after development. The designer should also evaluate the system for the peak 100-year discharge after development.

d. Calculations of available storage and outlet structure hydraulics necessary to show that the post development discharge will be no greater than the predevelopment discharge based on a 25-year storm.

e. Calculations for energy dissipation of the outlet of the system for the 25-year storm.

f. The volume shall be calculated using the equation for estimated volume of required storage as written in **Section H.3.1.d**.

H.3 System Analysis

The four procedures described below are the ones that will be used by the city to review the detention pond design. Other procedures may be used by the designer to design or analyze the pond and system. In cases where the designer proposes to use computer programs or desktop procedures not recognized by the city, the designer shall show that the pond adequately performs according to the following analysis.

H.3.1 Hydrograph Formulation. The three important aspects of the design hydrograph are the magnitude of the peak discharge, the volume of runoff (area under the hydrograph), and the hydrograph shape. A simplified method of hydrograph formulation, based on these aspects, are acceptable for analyzing detention pond designs from watersheds of less than one square mile. (For background see Malcom, 1987). For larger watersheds, detailed hydrologic modeling shall be done by accepted methods. The simplified method for hydrograph formulation is:

a. Estimate the Peak. The peak discharge of the hydrograph of the 25-year storm may be estimated by applying the Rational method as described in Section E, with the following provisions:

- Determine **C**, the composite runoff coefficient, in the conventional way.

- Compute **I** the applicable intensity (in/hr), for the 25-year storm at the appropriate time of concentration. Determine time of concentration by the Kirpich equation. Determine the intensity by the equation or graph given in Section E.

- Measure the watershed area contributory to the pond as delineated on a suitable topographic map.

b. Estimate the Volume of Runoff. The volume of runoff may be estimated by applying the runoff estimation methods of the Soil Conservation Service to an appropriate design storm. In this application, the design storm is taken to be the 25-year storm of 6 hours duration, which is a depth of 4.63 inches in Southport. See Chart E-5. (The 6-hour duration is selected in this method to yield a hydrograph comparable in size to that produced in SCS TR-55, U.S. Army Corps of Engineers HEC-1, and similar methods). The total volume of the runoff is the runoff depth multiplied by the watershed area. The volume estimate may be made as follows:

- Estimate the effective SCS Curve Number (CN) for the watershed under future development conditions. Refer to the SCS Soil Survey of Brunswick County to determine the soil types distributed in the watershed. Use Tables H-1 and H-2 to estimate the effective Curve Number for the watershed from observed soil types and cover conditions, or use SCS publications. Calculate a composite CN if the site should require one.

- From the six-hour, 25-year rainfall and the applicable SCS Curve Number, compute the runoff depth by the SCS procedure listed below.

  Determine the ultimate soil storage capacity, **S**:

  $$\mathbf{S} = (1000 / CN) - 10$$

  Determine runoff:

  $$\mathbf{RUNOFF} = (P - 0.2\ S)^2 / (P + 0.8\ S)$$

  Where **P** is the 6 hour, 25-year storm; use 4.63 inches for Southport

c. Set the Shape of the Hydrograph. The shape is determined by a pattern function that will preserve the estimated peak and estimated volume of runoff. The time to peak, **Tp**, is computed by the following expression:

$$\mathbf{Tp} = (43.5 * Area * Runoff) / Qp$$

In which **runoff** is the SCS runoff depth in inches, **area** is the watershed area in acres, **Qp** is the estimated peak discharge in cfs, and **Tp** is the time to peak in minutes.

When the values of the peak, **Qp**, and the time to peak, **Tp**, have been set, the discharge at any time **T** may be determined by the pattern function (a step function).

For time T from zero to 1.25 * Tp:

$$\mathbf{Q} = (Qp / 2)(1 - \cos((\pi * t) / Tp)$$

For times greater than 1.25 * Tp:

$$\mathbf{Q} = 4.34\ Qp\ \exp(-1.3(t / Tp))$$

In the step function, **Q** is the hydrograph discharge at the time of interest, **t**, in minutes. **Qp** is the estimated peak discharge in cfs, and **Tp** is the calculated time to peak in minutes. It is important to note that the argument of the cosine is in radians. If the calculations are carried out on a manual calculator, it shall be set to calculate in radians mode.

CHART H-1*

Depth - Duration - Frequency
Southport, North Carolina

	RETURN PERIOD					
Duration	2-Year (in)	5-Year (in)	10-Year (in)	25-Year (in)	50-Year (in)	100-Year (in)
5 Min	0.59	0.72	0.85	1.98	1.08	1.11
10 Min	0.91	1.11	1.31	1.51	1.66	1.71
15 Min	1.15	1.40	1.66	1.91	2.10	2.16
30 Min	1.60	1.95	2.30	2.65	2.91	3.00
1 Hr	2.00	2.50	2.90	3.33	3.71	4.00
2 Hr	2.43	3.20	3.55	4.13	4.55	5.17
3 Hr	2.73	3.39	3.94	4.50	5.00	5.56
6 Hr	3.09	4.00	4.63	5.38	6.00	6.76
12 Hr	3.67	4.80	5.63	6.45	7.14	8.07
24 Hr	4.10	5.50	6.61	7.67	8.33	9.50

TABLE H-1

Hydrologic Soil Groups for Local Soil Types

Map Symbol	Hydrologic Soil Group	Soil Type
Ba	D	Bayboro
Be	A	Baymeade
Bh	*	Baymeade-Urban Land Complex
Bp	*	Borrow Pit
Cr	C	Craven
DO	D	Dorovan
Jo	D	Johnston
Ke	A	Kenansville
Kr	A	Kureb
Ku	*	Kureb-Urban Land Complex
La	A	Lakeland
Le	B/D	Leon
LO	*	Leon-Urban Land Complex
Ls	C	Lynchburg
LY	B/D	Lynn Haven
MP	*	Nfine Pits
mu	A/D	Murvflle
Nh	A	Newhan
No	B	Norfolk
On	B	Onslow
PM	D	Pamlico
Pn	B/D	Pantego
Ra	B/D	Rains
Rm	A	Rimini
Se	A/D	Seagate
Sh	*	Seagate-Urban Land Complex
St	C	Stallings
TM	*	Tidal Marsh
To	C	Torhunta
Ur	*	Urban Land
Wa	A	Wakulla
Wo	B/D	Woodington
Wr	*	Wrightsboro

*Requires field judgment.
A/D refers to drained/undrained.
Sources: Urban Hydrology for Small Watersheds, USDA-SCS, 210-VI-TR-55, Second Ed., June 1986.

TABLE H-2

SCS Curve Numbers for Various Cover Conditions

	Hydraulic Soil Group				Percent
Cover Description	**A**	**B**	**C**	**D**	**Impervious**
Fully Developed Urban Areas					
Open Space					
Poor Condition (<50% grass)	68	79	86	89	
Fair Condition (50-75% grass)	49	69	79	84	
Good Condition (>75% grass)	39	61	74	80	
Impervious Areas					
Pavement, roofs	98	98	98	98	
Gravel	76	85	89	91	
Dirt	72	82	87	89	
Urban Districts					
Commercial and Business	89	92	94	95	85
Industrial	81	88	91	93	72
Residential Areas (by lot size)					
1/8 acre (townhouses)	77	85	90	92	65
1/4 acre	61	75	83	87	38
1/3 acre	57	72	81	86	30
1/2 acre	54	70	80	85	25
1 acre	51	68	79	84	20
2 acres	46	65	77	82	12
Agricultural Areas					
Pasture, grassland					
Poor	68	79	86	89	
Fair	49	69	79	84	
Good	39	61	74	80	
Meadow (mowed)	30	58	71	78	
Brush					
Poor	48	67	77	83	
Fair	35	56	70	77	
Good	30	48	65	73	
Woods and grass (orchard)					
Poor	57	73	82	86	
Fair	43	65	76	82	
Good	32	58	72	79	
Woods					
Poor	45	66	77	83	
Fair	36	60	73	79	
Good	30	55	70	77	
Row crops, straight, good	67	78	85	89	
Row crops, contoured, good	65	75	82	86	
Small grain, good	63	75	83	87	
Farmsteads	59	74	82	86	

Source: SCS TR-55 (SCS, 1986).

d. Estimating Required Storage. The required storage can be estimated using the following equation:

$$S = (Q_p - Q_a) \times T_p \times 1.39 \times 60$$

in which:

S = Estimated stormwater storage required (cu ft)
Q = Estimated peak flow (cfs) (postdevelopment flow rate)
$\mathbf{Q_a}$ = Allowable maximum outflow (cfs) (predevelopment flow rate)
$\mathbf{T_p}$ = Estimated time to peak (min)

On sites of one acre or more, the system shall be routed to verify that sufficient storage is provided to meet the design criteria. Sites less than one acre are encouraged to be routed but will not be required.

H.3.2 **Stage-Storage Formulation**. The stage-storage function represents the most important aspects of the size and shape of the storage container. In the submission documents, it is presented as a graph of water surface elevation versus storage volume. Plotted values are normally computed from the topographic map of the detention pond. Areas of contours within the pond are measured. From these, the incremental volumes of water storage between the contours are computed, then accumulated to yield points of volume stored below each contour. Orderly supporting calculations shall be submitted with the stage-storage plot.

The stage-storage relation can be formulated as a graph or as a mathematical expression. The latter is more useful in this application because it includes both water volume information and surface area information as they relate to depth in a pond of complex shape. Stage is the depth of water relative to the bottom of the pond. Storage is the volume of water at a given stage.

A stage-storage function may be formulated for a given basin as follows:

Compute a set of representative storage volumes at various stages by applying the average-end method of volume computation vertically to the set of known contours that express the basin topography. Arrange them as a list of stages, **Z**, and associated storage, **S**. If one plots the logarithms of storage versus the logarithms of stage, the resulting graph is usually remarkably a straight line, even the apparently complex topography of a natural draw or swale. This observation leads to the power-curve representation described below.

The expression for the stage-storage function is:

$$\mathbf{S = K_s Z^b}$$

in which:

$\mathbf{Z}$ = Stage (ft above the pond bottom)
$\mathbf{S}$ = Storage (cu ft)
$\mathbf{K_s}$ and $\mathbf{b}$ are constants to be determined for the basin of interest

There are two reasonable ways to determine $\mathbf{K_s}$ and $\mathbf{b}$ from the stage-storage list. One is to use a linear regression routine applied to the logarithms of the data and back calculate the constants, $\mathbf{K_s}$, and $\mathbf{b}$, from the regression results. The regression procedure is preferred because the shape information contained in a number of contours can be used to set the constants.

The other method is to obtain an approximation of the constants algebraically by using stage and storage values from two of the contours. It is usually best to select one point near the maximum expected water-surface elevation and the other at about mid-depth. The precision of the result can be tested and improved by trial and error.

Select two points on the stage-storage function as described above. Let the lower be point number one and the upper be point number two.

Estimate the exponent:

$$\mathbf{b = [\ ln(S_2\ /\ S_1)\ ]\ [\ ln(Z_2\ /\ Z_{1)\ ]}}$$

Estimate the coefficient:

$$\mathbf{K_s = S_2\ Z^b{}_2}$$

in which:

$\mathbf{Z}$ = Stage of the specified point (ft above the pond bottom)
$\mathbf{S}$ = Storage of the specified point (cu ft)
$\mathbf{K_s}$ and $\mathbf{b}$ are constants determined for the basin of interest

Test the validity of the function by substitution of known values of storage to estimate the associated stages. If the stages agree acceptably with the actual stages (say within 0.1 ft or so), the expression is valid. For that check, the expression can be reformulated as:

$$\mathbf{Z = [S\ /\ K_s]^{1/b}} \qquad \text{(equation H-1)}$$

In which the variables are the same as above.

Reference: North Carolina Erosion and Sediment Control Planning and Design Manual, pp. 8.07.29 (rev 12/93), 8.07.30 (rev 12/93)

H.3.3 Stage Discharge Function. The stage-discharge function represents the most important aspects of the hydraulic behavior of the outlet system. Because there are many combinations of acceptable outlet devices, there can be no simple specification of permissible devices. The designer, having verified through analysis that the proposed outlet system is satisfactory, is expected to present the stage-discharge function as a graph of water-surface elevation versus outflow from the system. Detailed drawings of the outlet configuration and supporting hydraulic calculations shall be submitted with the stage-discharge plot.

Commonly, outlet devices are constructed as a combination of pipes and weirs. These combinations may require that the stage-discharge function be developed for one or more combinations of the following types of flow.

a. Inlet control. Pipes acting under inlet control can be represented in calculations by the orifice equation, if the inlet is fully submerged, or by the charts published by the Federal Highway Administration (FHW A, 1985). See Charts F-6 thru F-10.

The equation for orifice flow is:

$$\mathbf{Q = C_d\, A\, (2gh)^{0.5}}$$

where:

Q is the discharge in cfs
C_d is the coefficient of discharge (Table H-3)
A is the area of the orifice in sq ft.
g is the acceleration of gravity = 32.2 ft/sec^2
h is the height of the headwater above the center of the orifice in feet

TABLE B-3

Values of Coefficient of Discharge (Cd) for Pipes under Inlet Control

Reinforced Concrete	
Socket end flush with headwall	0.65
Socket end projecting from fill	0.64
Square edge in headwall	0.59
Corrugated Metal Pipe	
End flush with headwall	0.59
End mitered to conform to slope	0.52
End projecting from fill	0.51

b. Outlet Control. If the outlet end of the pipe system can be submerged by the 25-year storm, the system shall be analyzed under outlet control, including routing, to confirm that the operation will be satisfactory. The FHWA outlet control charts can be used to make the analysis. See Charts F-11 through F-15.

c. Weir Calculations. The spillway component of the stage-discharge computations can be computed using the equation for weir flow. When computing flow into a drop structure as weir flow or if the spillway has a concrete weir section the coefficient should be 3.3. When the flow is over the top of a road or grassed berm used as a spillway the coefficient should be 3.0.

The weir equation is as follows:

$$\mathbf{Q = C_w \, L \, H^{3/2}}$$

where:

Q is the discharge in cfs
$\mathbf{C_w}$ is the weir coefficient, 3.0 or 3.3
L is the length of the weir measured along the crest in feet (perpendicular to flow)
H is the depth from the top of the weir to the water surface at a point where the velocity is minimized

The discharge determined using the FHWA outlet-control charts should be compared to the discharge computed using the orifice equation. The smaller of two discharges determines the control for the pipe. This flow is then used in developing the stage-discharge information. If both pipe flow and weir flow exist at the same time the two values for the same stage should be added together to determine the total discharge for a given stage.

To draw the stage-discharge curve, you plot the stage verses total discharge on semi-log paper with the stage being plotted on the arithmetic side, usually the vertical, and the discharge on the log side, usually the horizontal.

Designers are encouraged to exercise innovative planning in detention pond design to produce facilities that are effective, attractive, and easily maintained. In the interest of efficiency in design and review, designers of unusual ponds are encouraged to confer with the city early in the design process for a preliminary reaction.

H.3.4 Flood Routing.* The hydrograph of the 25-year storm should be routed through the detention pond to verify that the detention objective is met. The Chainsaw Routing method, devised by Dr. H. Rooney Malcom, P .E., is included for its inherent simplicity and efficiency. The name is a reminder of the coarseness of the information on which analysis and design of these systems are based.

To execute the flood routing, one first formulates the three sets of input data described in sections H.3.1, H.3.2, and H.3.3.

Routing of the flood proceeds by time steps. At each step in time during the passage of the inflow hydrograph through the reservoir, the outflow is computed. The result is a list of values of outflow at stated times --the outflow hydrograph.

The continuity principle states that the rate of change of storage with respect to time is the difference between inflow and outflow:

$$ds/dt = I - O$$

Over a time increment:

$$\Delta s/\Delta t = I - O$$

where:

$\Delta s/\Delta t$ is the change in storage with respect to time
I is inflow
O is outflow

The incremental change in storage can be estimated as:

$$\Delta S_{ij} = (I_i - O_i)\,\Delta T_{ij} \qquad \text{(equation H-2)}$$

in which:

ΔS_{ij} is the change in storage in the time increment **i** to **j**
I_i is the inflow at time i
O_i is the outflow at time i
ΔT_{ij} is the time increment

The simplification of this method is to consider that the change in storage may be adequately estimated by viewing the time increment as a parallelogram, whereas it is more precisely viewed as a trapezoid (the view taken in the Storage-Indication method). Equation H-2 becomes the basis for taking a step through time in the routing. Note the equation H-2 is in consistent units. If inflow and outflow are in cfs and if the time increment is in seconds, then storage will be computed in cubic feet. These are the most convenient units.

*Malcom, H. Rooney. 1989. Elements Of Urban Stormwater Design, pp. III-17 - III-21. North Carolina State University.

The routing is conventionally carried out in a table, such as Table H-4, which was executed in a spreadsheet.

The reservoir in this case is a normally dry detention basin designed as a culvert. The inflow hydrograph peaks at 368 cfs and 3 minutes and follows the step function given above. The stage-storage function for the area upstream of the culvert was formulated such that Ks = 284 and b = 3.30. The culvert consists of one 48-inch diameter reinforced concrete pipe with grooved end flush with a headwall for which the coefficient of discharge is estimated at 0.65. There is a roadway that serves as an overflow spillway. The crest elevation is at stage of 10.0 feet, the weir length is 120 feet and the weir coefficient is 3.0 (broadcrested case). The stage reference (Z = 0) is to the invert of the entrance of the culvert which is also the bottom of the dry pond.

A Word about Spreadsheets: In the tabular computations that follow (Table H-4), the values shown in the cells of the tables were rounded back to the precision displayed. Internally in the spreadsheet program, the computations were carried out to several significant figures. As the reader computes values in a given cell based on displayed values in other cells, some differences may be noticed.

Selection of the Time Increment: The time increment should be about one tenth of the time to peak (**Tp**), where time to peak is considered to be measured from the time of significant rise of the rising limb to the time at which the peak occurs. In this case the time to peak is 36 minutes, so the time increment was conveniently selected at 4 minutes. Note that in calculations the time increment was expressed in seconds.

TABLE H-4
CHAINSAW ROUTING APPLIED
TO A SITE WITH CULVERT AND OVERFLOW WEIR

Input Data:

Qp	368		
Tp	36		
RESULTS			
dT	4		
		OUTFLOW PEAK	173
Ks	284	**MAX STAGE**	8.94
b	3.3		
N	1		
Cd	0.65		
D	48		
Zi	0		
Cw	3		
L	120		
Zcr	10		

*Malcom, H. Rooney. 1989. Elements Of Urban Stormwater Design,, pp. Ill-17 - Ill-21. North Carolina State University.

Initialization of the Routing Table: In every routing method, the routing table must be initialized to represent the state of the system at time zero. In this case, along the row at time zero, the following were set:

> **Col 2**: The initial inflow is zero; the system starts with no inflow. In some cases there may be some trivially low flow to be entered here.
>
> **Col 5**: The initial outflow is set equal to initial inflow (CoI 2) at time zero.
>
> **Col 4**: Initial stage is set to reflect the water level in the reservoir at the beginning of the storm. In this case the pond is dry, and the stage is zero. In some cases, there is a normally wet pond. Then the stage is set to the initial stage of the water surface.
>
> **Col 3**: Initial storage is the volume of water (cubic feet) in the reservoir at time zero. In this case, the system is dry and the volume is zero. In the case of a normally wet pond, the initial volume can be computed from the stage-storage function using the stage of the initial water surface.

ROUTING

1	2	3	4	5	6	7
TIME [min]	INFLOW [cfs]	STORAGE [cu ft]	STAGE [ft]	OUTFLOW [cfs]	CULVERT [cfs]	WEIR [cfs]
0	0	0	0	0	0	0
4	11	0	0.0	0	0.0	0.0
8	44	2718	1.98	32	32.4	0.0
12	94	5484	2.45	44	44.1	0.0
16	155	17396	3.48	80	79.7	0.0
20	219	35378	4.31	100	99.7	0.0
24	279	64046	5.17	117	116.6	0.0
28	328	103103	5.97	130	130.5	0.0
32	358	150432	6.69	142	141.9	0.0
36	368	202386	7.32	151	151.1	0.0
40	354	254328	7.84	158	158.4	0.0
44	320	301293	8.26	164	163.9	0.0
48	277	338670	8.56	168	167.8	0.0
52	239	364822	8.75	170	170.2	0.0
56	207	381358	8.87	172	171.7	0.0
60	179	389740	8.93	172	172.5	0.0
64	154	391210	8.94	173	172.6	0.0
68	133	386828	8.91	172	172.2	0.0
72	115	377506	8.84	171	171.4	0.0
76	100	364032	8.75	170	170.2	0.0
80	86	347094	8.62	169	168.6	0.0
84	74	327290	8.47	167	166.6	0.0

[*]Malcom, H. Rooney. 1989. Elements Of Urban Stormwater Design, pp. III-17 - III-21. North Carolina State University.

Taking a Time Step: In every time step, the objective of the computation is to determine the outflow at the end of the time interval. So in this case, one would use the information at time zero and compute the values at time 4 min. Then use the 4 min values to find those at 8 min, and so on. Let time **i** be the time at the beginning of the interval and time **j** be the time at the end of the interval. In the chainsaw routine, one begins by using values at time **i** to estimate the change in storage at time **j** and to update the storage volume. Then with the known storage at time **j**, the stage can be computed from the stage-storage function, and from stage the outflow can be computed from the stage-discharge function, all at time **j**.

As an example, in the interval from time 28 to time 32 min, here is the order of computation:

Col 3: The change in storage, from Equation H-2, is inflow at time **i** (time 28 minutes) minus outflow at time **i** multiplied by the time increment (240 sec). The change is 47,520 cu ft, which is added to the storage at time **i** (103,103 cu ft) to yield the storage at time **j** (150,432 cu ft). (The numbers do not agree as printed because the spreadsheet program calculates all values to maximum precision, and the values are rounded for display.)

Col 4: At time **j** (32 min), calculate stage from updated storage using the rearranged stage-storage function, equation H-1. At 32 min water has risen to stage 6.69 ft.

Col 6: At time **j** (32 min), calculate the flow through the pipe by the orifice equation, or by the culvert capacity charts. For stage of 6.69 ft, the equation yields 141.9 cfs.

Col 7: At time **j** (32 minutes), calculate the flow over the weir by weir equation. Note that for this whole routing, the water level never rises above the crest of the weir, so there is no weir flow.

Col 5: At time **j** (32 min), the outflow is the sum of the contributions of the pipe (Col 6) and the weir (Col 7).

Col 2: At time **j** (32 min), the inflow is updated by reading values from a plotted hydrograph developed in Section H.3.1 (rounded values are displayed) to compute the discharge at time 32 minutes.

One can run the routing table as far as needed to determine the system responses of interest. Usually, these are the peak outflow, the largest value in Col 5, and the maximum stage, the largest value in Col 4.

Sometimes this method is subject to numerical instability. It will occur if the outlet system is of high discharge capacity and the storage container is of low storage capacity. In the real situation the outflow hydrograph is tracking closely the inflow hydrograph.

[*]Malcom, H. Rooney. 1989. Elements Of Urban Stormwater Design,, pp. III-17 - III-21. North Carolina State University.

The effect of storage upon outflow is negligible. In the routing table, it will present itself when outflow exceeds inflow on the rising limb of the inflow hydrograph. In the extreme, change in storage becomes negative and large, perhaps large enough to make total storage go negative, and the computation of stage becomes impossible. Should this happen, it may be corrected by reinitializing the system on the line where the fault occurs as follows:

a. Set outflow (Col 5) equal to inflow (Col 2).

b. Set Stage (CoI4) equivalent to outflow (Col 5), by reference to the stage-discharge function

c. Set Storage (Col 3) equivalent to stage (CoI 4), by using the stage-storage function

d. Restart the routing repeating the reinitialization, if necessary, until the system behaves.

If instability occurs while stage is low in a multiple pipe outlet, it is reasonable to reinitialize where stage is near the top of the pipe. If instability persists to the inflow peak, it indicates that storage is ineffective in the system – there is no detention effect.

This routing procedure is performed in order to confirm that the detention pond is sized to accomplish the intended purpose of reducing the postdevelopment peak discharge to one that is equal to or less than the predevelopment peak discharge.

Other routing methods, including commercially available software, may be acceptable provided the city recognizes them. The test of acceptability is by verification.

*Malcom, H. Rooney. 1989. Elements Of Urban Stormwater Design, pp. III-17 - III-21. North Carolina State University.

SECTION I

INFILTRATION SYSTEM

The city encourages the use of innovative techniques and designs that will help reduce the amount of stormwater runoff getting into drainage ways and streams. These facilities shall be designed for the runoff produced from the 25-year storm. Detailed drawings, substantiating data, calculations, and specifications shall be submitted for designs of this nature. The use of infiltration systems has not been a frequent technique for control of storm water in the city; however, minimum standards have been established and are included in this design guide.

I.1 Definition

An infiltration system is defined as a stormwater management facility that is designed to let stormwater move or infiltrate into the soil. Types of systems shall include but not be limited to infiltration basins, swales, subsurface galleries, and vegetative filters.

I.2 Vegetation

Refer to Section D.4.2 for minimum vegetative requirements that may apply.

I.3 Subsurface Information

The minimum distance between the bottom of the infiltration system and the surface of the seasonal high ground water table shall be two feet. Soil types and infiltration rates shall be determined in order to size the infiltration area and assess the feasibility of this type of infiltration system. This information shall be submitted as part of the stormwater permits application package.

I.4 Storage Capacity

The infiltration system shall be designed to provide storage equivalent to the runoff volume from the 25-year storm. Infiltration systems may be used in combination with other systems to allow the predevelopment runoff rate to the leave the site.

I.5 Overflow

An emergency outlet or overflow device shall be designed such that in the event of a system failure (i.e., stormwater will not infiltrate) during the 25-year storm, stormwater will be conveyed to an existing drainage way or structure and not damage property. An emergency outlet or overflow device for the 100-year storm shall be provided (i.e., piped system, driveway, overland flow, etc.).

I.6 Access

Adequate access shall be provided for inspection and maintenance of the system in the form of cleanouts, grit chambers, and inspection ports, etc.

I.7 Materials

The following are the minimum requirements for materials used in infiltration systems:

Bedding Stone - No. 57 washed stone or manufacturer recommended bedding material.

Geotextile Fabric - This fabric should be used to wrap the sides, bottom, and top of the stone that surrounds the infiltration structures in order to prevent intrusion by fines. The top should be overlapped a minimum of 12 inches. Material should be manufacturer recommended.

Infiltration Chambers - The design should incorporate necessary loads that are to be expected from the area of infiltration

SECTION J

CONSTRUCTION STANDARDS

J.1 Materials for Pipe Collection System

In general, when a pipe system is intended to be deeded to the city for maintenance and ownership, it shall be constructed of reinforced concrete pipe. Collection systems that will remain privately owned may be constructed of corrugated metal, concrete, or other materials. The city realizes there may be situations where corrugated metal pipe is the better alternative due to height restrictions, etc., such as when arch or elliptical shapes are needed. When the developer prefers to pipe a ditch that requires larger than a 54-inch pipe, corrugated metal pipe may be considered. The city shall review and approve these alternatives on a case-by-case basis.

Pipe for storm water collection systems shall meet the requirements set forth in the following minimum specifications:

J.1.1 Reinforced Concrete Pipe. Reinforced concrete pipe shall conform to ASTM C- 76, Class Ill, latest revision. Joint material shall be Butyl Rubber.

J.1.2 Corrugated Metal Storm Drain Pipe. Corrugated steel pipe/pipe arch shall be of an approved gauge and shall be fully bituminous coated with a paved invert. For pipe sizes in excess of 60-inch corrugated steel pipe/pipe arch shall be fully bituminous coated and one hundred percent (100%) paved and shall meet the applicable requirements of AASHTO M36.

J.1.3 Pipe in Detention/Retention Ponds. Pipe used for detention/retention pond outlet structures shall meet the minimum specifications above or shall be aluminum pipe conforming to AASHTO M196 and shall have a gauge thickness determined in accordance with appropriate design standards.

J.1.4 Perforated Pipe. Perforated metal pipe for use as subdrains shall be a minimum of 6-inch galvanized Helcore pipe or equal. PVC pipe shall be a minimum of 6-inch ASTM D-1785, Schedule 40. Perforated corrugated polyethylene tubing for use as subdrains shall be in conformance with ASTM F-40S and a minimum of 6 inches in diameter.

J.1.5 Alternative Pipe Materials. Other pipe materials may be considered on a case-by-case basis and shall be approved by the city before use.

J.1.6 Precast Concrete Manholes. Precast concrete manholes shall be minimum 4'0" inside diameter, and shall have a monolithic extended concrete base. Manhole shall have minimum 5-inch wall thickness and be constructed of four thousand (4,000) psi concrete. Manholes with diameters greater than 4'0" are not required to have extended bases. All precast manholes must have 12" of stone bedding. Inverts shall be formed to provide a definite channel of flow through the structure. All manholes shall have steps cast in the walls in accordance with J.1.8.

J.1.7 Brick for Manholes and Catch Basins. Brick for manholes and catch basins shall be whole, solid, uniform concrete or clay brick, with straight, even faces free of injurious defects.

J.1.8 Storm Drain Manhole Steps. Steps shall meet the requirements of AASHTO M199 for design, materials, and dimensions. Steps shall be incorporated in all drainage structures over 3'-6 inches in height. The lowest step shall be no more than 16 inches from the bottom.

J.1.9 Gray Iron Castings. Gray iron castings used for manhole frames and covers and inlet frames, grates, and covers shall conform to the requirements for Gray Iron Castings of the American Society of Testing Materials (ASTM). The castings shall be true to pattern and free from cracks, gas holes, flaws, and other defects. All surfaces shall be thoroughly coated with spray asphalt coating containing no asbestos material. If, during handling of castings, the coating is damaged, it shall be recoated to provide a complete covering. Material shall be an asphaltic solution with no asbestos. Surface shall be smooth and free from runners, fins, and other cast-on pieces.

J.2 Installation

J.2.1 Bedding. Excavation for storm drainage pipe shall be to the lines and grades as shown on the plans. The bedding shall provide a firm foundation of uniform density along the entire length of pipe. Recesses shall be made to accommodate bells and joints. Where unstable soils are encountered, a minimum 6-inch thick bedding of stone shall be used. The stone shall be uniformly graded from 3/4-inch to No. 4 in accordance with ASTM C-33. Care shall be taken to prevent undercutting in suitable soil. Areas undercut shall be filled with suitable soil and compacted to 95% of maximum density at optimum moisture content as determined by ASTM D 1557 Standard Test Method.

J.2.2 Laying and Joints. Storm drainpipe shall be laid to the line and grade as shown on the approved plans. Joints shall be as recommended by the pipe manufacturer. Joints shall be sealed tightly to assure prevention of infiltration of groundwater, soil, and other undesirable material.

J.2.3 Backfill. Backfill for storm drain pipe and appurtenances shall be free from all perishable and objectionable material including all rubbish, forms, blocks, etc. Backfill shall be placed around and above the pipe and solidly tamped to prevent movement of the pipe. Backfill shall then be placed and compacted to 95 percent as determined by ASTM D 1557 Standard Test Method in layers not to exceed 12 inches.

J.2.4 Manholes.

a. Brick manholes shall be constructed of good hard burned brick laid in with cement mortar. All brick, when set, shall be pushed to firm seating in mortar and all joints well filled and spaded. All inlet and outlet pipes shall be placed prior to building the manhole walls and care shall be taken to ensure a tight joint around such pipe where it passes through the walls. Manholes shall be plastered with cement mortar on the outside to a thickness of 3/4 of an inch. The base of the catch basins shall have a minimum thickness of 6 inches of Class A concrete and have 6-inch extended sides

b. Precast manholes shall be installed as per the manufacturer's recommendations. Manholes shall be set with 6-inch extended bases and a minimum 12-inch stone bedding. The tops shall be set to grade using a maximum of two 6-inch adjustment rings and casting. When pipe is laid into manholes, a watertight seal shall be provided at the opening using concrete grout.

J.2.5 Catch Basins.

a. Brick catch basins shall be constructed of good hard burned brick laid in with cement mortar. All brick, when set, shall be pushed to a firm seating in mortar and all joints well filled and spaded. All catch basins shall be of the open throat type unless otherwise permitted. The base of the catch basins shall have a minimum thickness of 6 inches of Class A concrete and have 6 inch extended sides. All pipes shall be placed on the concrete base prior to beginning brickwork. The brickwork shall be brought up snugly around the pipes such that a tight connection is obtained. Catch basins shall be plastered with cement mortar on the outside to a thickness of 3/4 of an inch.

b. Precast catch basins shall be installed per the manufacturer's recommendations. Precast catch basins shall be sized properly to receive the approved inlet castings. Field adjustments to accommodate the approved inlet casting will not be allowed. A 6-inch thick concrete base with 6-inch extended sides shall be constructed with a smooth level surface. The precast basin shall be carefully placed on the clean surface at specified grades. Openings between wall and pipe shall be sealed with hydraulic cement to ensure a leak-free basin.

J.2.6 Detention/Retention Outlet Systems. Outlet structures for detention/retention facilities shall be provided with suitable foundation and support. Pipe systems shall be bedded as required or other suitable support provided. Outlet structures shall be properly anchored to prevent flotation.

SECTION K

MAINTENANCE

K.1 Responsibility

In order for the stormwater management system to work properly at all times, it will be necessary to maintain all elements of the system. A system includes open channels, catch basins, pipes, ponds, outlet controls, etc. It is especially important to maintain vegetative lined systems and silt or debris retaining devices.

The city will not maintain privately owned detention/retention systems. Approval and designation of a system as "private" requires that the owner maintain the system so that the intended function of the system is unimpaired.

In order for the City of Southport to ensure an acceptable level of maintenance of the private facilities, the following will be required to obtain approval.

K.1.1 Acceptable Entities. An acceptable entity shall be responsible for maintenance of the stormwater management system. The City of Southport considers the following entities acceptable:

a. Governmental Utilities and Private Corporations. If the entity is a governmental unit or private corporation, written proof shall be supplied in an appropriate form stating that the entity will operate and maintain the facilities.

b. Non-profit corporations including homeowners associations, property owners associations, condominium associations or associations of unit owners. The property owner or developer, as applicant for site plan or subdivision plat approval, is normally not acceptable as a responsible entity, especially when the property is to be sold to various third parties. However, the property owner may be acceptable if the property will be retained by the owner and will be rented, leased, or operated by the owner. The property owner shall supply evidence acceptable to the City Attorney that he will operate and maintain the facilities.

K.1.2 Powers. If a homeowner's association, property owners association, or association of unit owners is proposed for maintenance of the facilities, the applicant shall submit draft Articles of Incorporation, Declarations of Protective Covenants, Deed Restrictions, Declarations of Unit Ownership or By-laws.

The Association shall have the general power to:

- Own and convey property;
- Operate and maintain common property;
- Establish rules and regulations;
- Assess members and enforce said assessments;
- Sue and be sued;

- Contract for services to provide operation and maintenance;
- All lot owners, all home owners or unit owners shall be members of the Association; and
- The Association shall exist in perpetuity.

K.1.3 Claims of Maintenance. The Articles of Incorporation, Declaration of Protective Covenants, Deed Restrictions, Declaration of Unit Ownership or By-laws shall set forth the following:

a. That it is the responsibility of the Association to operate and maintain that portion of the stormwater management system not maintained by the city. A description specifying the areas of responsibility shall be included. These areas also shall be indicated on the subdivision plat or on the site plan on non-subdivision projects.

b. A maintenance plan with schedules and work generally following the minimum guidelines provided in this section.

c. A statement that those areas to be maintained by the Association are owned by the Association or that they are common areas or common property.

d. The method of assessment and collection for operation and maintenance costs of the stormwater management system.

e. The Declaration of Covenants to be in effect for a minimum of 25 years with provision for renewal in accordance with law.

K.1.4 Phasing Development. If a property owner's association or association of unit owners is proposed for a development that will be constructed in phases or that will be added to in the future, the organization shall be created with the ability to accept future phases into the organization in order to ensure the continued operation and maintenance of the stormwater management system for the development.

K.2 Operation and Maintenance for Stormwater Detention/Retention Facilities

K.2.1 General. Stormwater runoff is normally collected by a system of open channels and/or a piped collection system in developed areas and by sheet flow and swales in landscaped areas. For the system to operate in the correct manner, periodic maintenance will be required.

K.2.2 Maintenance. As a minimum the following maintenance items shall be performed:

a. Detention Pond

- Grassing around any detention/retention facility shall be maintained to prevent the erosion of these areas. The areas shall be periodically mowed to maintain the aesthetic quality of the site and to prevent a reduction in capacity of the stormwater system. Grass on slopes should not exceed a height of 15 inches.

- Open ditches shall be kept free of undesirable growth and mowed or maintained to the design cross-section and area as shown on the Stormwater Management Plan approved by the city and on file in the office of the Public Works Director. Growth on the slopes and bottom should not exceed a height of 8 inches.

- Landscaping of the area around the detention/retention facilities shall not reduce the capacity or hinder operation and maintenance of the stormwater system.

- The facility shall be routinely checked and cleared of all accumulation of debris, and the detention/retention facility outlet structure cleared of any blockage that is present.

- Storm drainage pipes and culverts shall be periodically inspected for debris and sand build-up. They shall be cleaned as necessary to provide for the free conveyance of stormwater as designed.

- The detention/retention facility shall be maintained at the design depth as shown on the Stormwater Management Plan approved by the city and on file in the Public Works office. The pond shall be inspected on a regular basis but not less than every six months. Debris and sedimentation shall be removed if: (1) the primary outlet capacity is impaired; and/or (2) the depth of the facility is more than one foot above the original facility depth or facility volume is reduced by 25% of the design impoundment volume.

- Landscaping shall be maintained to ensure that landscape materials live and prosper.

b. Oversized Pipe

- The pipe shall be routinely checked for and cleared of all accumulation of debris and the detention facility outlet structure cleared of any blockage.

- Storm drainage pipes and structures shall be periodically inspected for debris and sediment build-up. They shall be cleaned as necessary to provide for the conveyance of stormwater as designed.

- The pipes installed to provide detention shall be kept free of sediment build-up. The detention facility shall be maintained in accordance with the Stormwater Management Plan approved by the city and on file in the office of Public Works. The pipe shall be inspected on a regular basis but not less than every six months. Debris and sedimentation shall be removed if: (1) The storage volume is reduced by 25% or more, and/or; (2) The sediment and/or debris restricts the free flow of stormwater.

c. Infiltration System

- The system shall be routinely checked for and cleared of all accumulation of debris and the detention facility outlet structure cleared of any blockage.

- Storm drainage pipes and structures shall be periodically inspected for debris and sediment build-up. They shall be cleaned as necessary to provide for the conveyance of stormwater as designed.

- The pipes and stone installed to provide infiltration shall be kept free of sediment build-up. The infiltration system shall be maintained in accordance with the Stormwater Management Plan approved by the city and on file in the office of Public Works. The pipe shall be inspected on a regular basis but not less than every six months. Debris and sedimentation shall be removed if: (1) The infiltration capacity is impaired, and/or (2) The sediment and/or debris restricts the free flow of stormwater into the infiltration system and surrounding soils.

- The infiltration system shall be removed and replaced with new material when the system no longer permits the stormwater to freely infiltrate into the surrounding soils

ATTACHMENT

City of Southport
Requirements Checklist

______ Application Form
______ Application Fee
______ Stormwater Management Plan
    ______ Plan
        ______ Title Block
            ______ Development Name
            ______ Owner
            ______ Design Firm
            ______ Authorized Registered, Professional's Seal, Signature, & Date
            ______ Legend
            ______ North Arrow
            ______ Vicinity Map
            ______ Scale
            ______ Sheet Number
            ______ Date
            ______ Revision Numbers and Dates
            ______ Street Address of Building(s) Onsite
        ______ Topographical Features
            ______ Original Contours at Not More Than 2-foot Intervals
            ______ Existing Drainage Components (stream, ponds, watersheds, etc.)
            ______ Property Boundary Lines
            ______ Existing Streets, Buildings, Utilities, etc.
            ______ 100-Year Flood Line, Floodway, and Building Setbacks
            ______ Offsite Drainage Entering the Site (If so, make note on plans)
            ______ Where City/County Topographic Maps are Used, Sufficient Checks Provided
            ______ Soil Type
            ______ Wetlands
            ______ Wooded Area and Tree Groups
        ______ Site Plan
            ______ Existing & Proposed Structures, Roads, Buildings, Paved Areas, etc.
            ______ Ex. & Prop. Stormwater Management System & Components include Pipe Sizes, Lengths, Inverts and Slopes (Provide a table for proposed pipes)
            ______ Connection to Existing System
            ______ Swale Information includes: Location, Size, Grade, Cross-Section
            ______ Proposed Erosion Control Measures
            ______ Existing and Proposed Contours
            ______ Typical Street Cross-Section
            ______ Typical Driveway Detail
            ______ Typical Construction Entrance Detail
            ______ Total Impervious Area in Square Feet (Existing and Planned)
            ______ Soil Types
            ______ Work Limits and Areas to Remain Undisturbed Including Square Footage to be Disturbed
            ______ Wetlands

        ______ Wooded Areas and Tree Groups
        ______ Finish Floor Elevations
        ______ Provide Copy of Drainage Plan With Different Drainage Areas Distinguishable From Each Other (for each Catch Basin or Inlet, Delineate its watershed) (can be a red line)
    ______ Certifications
        ______ Designers
        ______ Owners
    ______ Design Calculations
        ______ Piped Systems
            ______ Design for 25-year Storm
            ______ Pipe Required for Streams. Ditches, Channels. etc.
            ______ Minimum Velocity for Pipe Segments is 2.5 FPS
            ______ Minimum Cover for Pipe is 2 ft Measured from TOP of Pipe to Bottom of Base Course
            ______ Maximum Manhole Spacing is 400 ft for All Pipes Less Than 60 Inches
            ______ Headwalls or Flared End Sections Required at all Inlets and Exits of Piped Systems
            ______ Energy Dissipaters Designed for 25-year Storm
            ______ Provide Detail of Standard Catch Basin, Drop Inlet, and Manhole (Check cover requirements depending on Pipe Size)
            ______ For Piped Systems where Tailwater Conditions Exist, Calculations Should be Provided
        ______ Open Channels
            ______ Design for 25-Year Storm
            ______ Side Slopes Shall be 3 to 1 or Flatter
            ______ Alternate Linings MAY be Permitted, Submittals Required
            ______ Easements Required for Public Dedication
            ______ Check Velocity for Proposed Lining
                ______ Temporarily Required if Velocity exceeds 2 fps for Vegetative Channels
        ______ Detention & Wet Retention Facilities
            ______ Strong effort to make the facility an amenity to the project
            ______ Design for 25-year Storm
                ______ Watershed is 1 Acre or more
                    ______ Plan showing entire Watershed with Sufficient Detail to Confirm Limits
                    ______ Detailed Calculations for Predevelopment & Post-development discharges
                    ______ Detail Calculations of Stage-storage to include Graph or Table
                    ______ Detail Calculations of Stage-discharge, include Graph or Table
                    ______ Detail Computations of Routing Showing at least Time, Storage, Stage, Discharge
                    ______ Emergency Spillway Calculations & Design Date for the 100-year storm

______ Energy Dissipaters Design Data and Calculations, for 25-year Storms

______ Watershed is Less Than 1 Acre

______ Plan Showing Entire Watershed With Sufficient Detail to Confirm Limits

______ Detailed Calculations for Pre-development & Post-Development Discharges

______ Calculations of Available Storage & Outlet Structure Hydraulics necessary to show that Post-development Runoff will not exceed Pre-development Discharge

______ Emergency Spillway Calculations & Design Date for the 100-Year Storm

______ Energy Dissipaters Design Data and Calculations for 25-Year Storms

______ Slopes for Vegetative Banks shall be 3 to 1 or Flatter

______ Vegetative Cover Type shall be Noted and Approved by city

______ Riser (if used)

______ Riser shall be Minimum of 12 inches

______ Pipe shall be Minimum of 12 inches

______ Trash Rack Required

______ Other Utilities shall be a Minimum of 5 feet from Basin

______ Landscaping Zone

______ Less than or Equal to 0.5 acres = 5 ft Minimum Zone

______ Greater than 0.5 acres = 10 ft Minimum Zone

______ Must be Approved by city

______ Access Zone, Minimum Zone of 10 feet, Cannot be Landscaping Zone

______ Fencing: not Required, Optional for Private Facilities

______ Must Meet NCDENR Requirements if Applicable

______ Infiltration Systems

______ Plan Showing Entire Watershed & How Offsite Drainage Will Be Handled

______ Detailed Calculations of Predevelopment & Postdevelopment Runoff for 25-year Storm

______ Certified Engineer's Report on Soils & the Permeability Rate of the Soils

______ Calculations for Sizing the Infiltration System and Required Storage

______ Provide an Emergency Spillway (Outlet) for the 100-Year Storm

______ Soil Types & Ground Water Level

______ ALL Required Easements with Recording Fees

______ Stormwater Management Maintenance Agreement with Recording Fee

______ Current Title Opinion

______ Subordination Agreement

______ Copies of Plans & Calculations as Needed