City of Denton

City Hall
215 E. McKinney St.
Denton, Texas 76201
www.cityofdenton.com

Meeting Agenda

Development Code Review Committee

Monday, January 12, 2026 | **3:00 PM** | **Development Services Center**

After determining that a quorum is present, the Development Code Review Committee of the City of Denton, Texas will convene in a Regular Meeting on Monday, January 12, 2026, at 3:00 p.m. in Training Rooms 3, 4, and 5, at the Development Service Center, 401 N. Elm Street, Denton, Texas, at which the following items will be considered:

1. ITEMS FOR CONSIDERATION

A. DCRC25-168 Consider approval of minutes for December 8, 2025.

Attachments: December 8, 2025

B. DCRC25-170 Receive a report, hold a discussion, and give staff direction regarding prioritizing future potential Denton Development Code amendments.

Attachments: Exhibit 1 - Agenda Information Sheet.pdf
Exhibit 2 - DDC Subsection 7.5.3F
Exhibit 3- DCM Section 4.9
Exhibit 4 - DCM Section 4.14
Exhibit 5 - iSWM Site Development Controls
Exhibit 6 - iSWM Hydraulics

C. DCRC25-162a Receive a report, hold a discussion, and give staff direction regarding proposed Code amendments related to the building design of single-family homes and other residential use types.

Attachments: Exhibit 1 - Agenda Information Sheet draft
Exhibit 2 - DDC 7.10.3D-F Residential Building Design Standards
Exhibit 3 - 2021 Residential Design Determination
Exhibit 4 - Memo on Building Design Interpretation (Landmark)
Exhibit 5 - Memo on Building Design Interpretation (Stella Hills)
Exhibit 6 - Additional Design Features Guide
Exhibit 7 - Presentation

D. DCRC25-095h Receive a report, hold a discussion, and give staff direction regarding proposed Code amendments related to Tree Preservation.

Attachments: Exhibit 1 - Agenda Information Sheet
Exhibit 2- Proposed Redline Markups (DDC 7.7.4.G and M- Minimum Preservat
Exhibit 3- Draft Presentation

2. DEVELOPMENT CODE REVIEW COMMITTEE PROJECT MATRIX

A. DCRC25-169 Hold a discussion regarding the Development Code Review Committee Project Matrix.

3. CONCLUDING ITEMS

A. Under Section 551.042 of the Texas Open Meetings Act, respond to inquiries from the Development Code Review Committee or the public with specific factual information or recitation of policy, or accept a proposal to place the matter on the agenda for an upcoming meeting AND Under Section 551.0415 of the Texas Open Meetings Act, provide reports about items of community interest regarding which no action will be taken, to include: expressions of thanks, congratulations, or condolence; information regarding holiday schedules; an honorary or salutary recognition of a public official, public employee, or other citizen; a reminder about an upcoming event organized or sponsored by the governing body; information regarding a social, ceremonial, or community event organized or sponsored by an entity other than the governing body that was attended or is scheduled to be attended by a member of the governing body or an official or employee of the municipality; or an announcement involving an imminent threat to the public health and safety of people in the municipality that has arisen after the posting of the agenda.

CERTIFICATE

I certify that the above notice of meeting was posted on the official website (https://tx-denton.civicplus.com/242/Public-Meetings-Agendas) and bulletin board at City Hall, 215 E. McKinney Street, Denton, Texas, on January 6, 2026, in advance of the three (3) business day posting deadline, as applicable, and in accordance with Chapter 551 of the Texas Government Code.

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OFFICE OF THE CITY SECRETARY

NOTE: THE CITY OF DENTON'S DESIGNATED PUBLIC MEETING FACILITIES ARE ACCESSIBLE IN ACCORDANCE WITH THE AMERICANS WITH DISABILITIES ACT. THE CITY WILL PROVIDE ACCOMMODATION, SUCH AS SIGN LANGUAGE INTERPRETERS FOR THE HEARING IMPAIRED, IF REQUESTED AT LEAST TWO (2) BUSINESS DAYS IN ADVANCE OF THE SCHEDULED MEETING. PLEASE CALL THE CITY SECRETARY'S OFFICE AT 940-349-8309 OR USE TELECOMMUNICATIONS DEVICES FOR THE DEAF (TDD) BY CALLING 1-800-RELAY-TX SO THAT REASONABLE ACCOMMODATION CAN BE ARRANGED.

MINUTES
DEVELOPMENT CODE REVIEW COMMITEE
December 8, 2025

After determining that a quorum is present, the Development Code Review Committee of the City of Denton, Texas will convene in a Regular Called Meeting on Monday, December 8, 2025, at 3:03 p.m. in Training Rooms 3, 4, and 5 at the Development Services Center, 401 N. Elm Street, Denton, Texas, at which the following items will be considered:

PRESENT: Chair Brian Beck, and Members: Suzi Rumohr, Eric Pruett, Lisa Dyer, and Clay Riggs

ABSENT: Member Vicki Byrd

REGULAR MEETING

1. ITEMS FOR CONSIDERATION

A. DCRC25-164: Consider approval of minutes for November 17, 2025.

AYES (4): Chair Brian Beck, and Members: Suzi Rumohr, Eric Pruett, and Lisa Dyer
NAYS (0): None
ABSENT FOR VOTE (1): Member Clay Riggs

Member Rumohr moved to approve the minutes as presented. Motion seconded by Member Pruett. Motion carried.

B. DCRC25-095g: Receive a report, hold a discussion, and give staff direction regarding proposed Code amendments related to Tree Preservation.

Staff presented the item.

Member Riggs joined the meeting at 3:09p.m.

Tree Replacement Rates – Preservation: Committee consensus was to move forward with Scenario 3 which increases the cost of removal, has the strongest incentivization for preservation, and includes a 65% increase in the removal owed.

Preservation Credits: Committee consensus was to move forward with Scenario 3 which covers 55% of standard mitigation, has a strong emphasis on heritage trees, and has the entire mitigation covered at 44% preservation.

C. DCRC25-162: Receive a report, hold a discussion, and give staff direction regarding proposed Code amendments related to the building design of single-family homes and other residential use types.

Matt Bodine, Assistant Planner, presented the item.

Building Mass and Form – DDC 7.10.3E: Committee consensus was to move forward with staff recommendation of requiring 4 design features and adding the proposed 10 features to the list.

The rest of this presentation was tabled for the next meeting due to time constraints.

2. DEVELOPMENT CODE REVIEW COMMITTEE PROJECT MATRIX

A. DCRC25-165: Hold a discussion regarding the Development Code Review Committee Project Matrix.

Angie Manglaris, Assistant Planning Director, presented the item.

Discussion followed.

3. CONCLUDING ITEMS

With no further business, the Regular Meeting was adjourned at 4:56 p.m.

X _____
Brian Beck
Chair

X _____
Carly Blondin
Administrative Assistant

Minutes approved on: _____

City of Denton

City Hall
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AGENDA INFORMATION SHEET

DEPARTMENT: Department of Development Services

ACM: Cassey Ogden

DATE: January 12, 2026

SUBJECT

Receive a report, hold a discussion, and give staff direction regarding prioritizing future potential Denton Development Code amendments.

BACKGROUND

On December 9, 2024, staff presented a proposed list of priority topics for the Development Code Review Committee (DCRC) to work through as part of their 2025 work program. The proposed work program included topics remaining on the DCRC's list of priorities from 2023 but also incorporated new priorities that had risen in 2024 and needed to be addressed. Topics which the DCRC agreed were a priority were placed on the DCRC's Matrix as items the Committee identified as future discussion items, which included the following:

- **Access and Circulation**: This topic was introduced to DCRC in 2023. Proposed areas for discussion as part of this topic are related to compliance with specifications, developer responsibility for access and circulation improvements, access to Collector and Arterial roadways, and public transit. The following is a summary of the Committee's feedback from the October 23, 2023, meeting:
    - At the October 23, 2023, meeting, staff gave a brief overview of the access topics and proposed amendments to Denton Development Code (DDC) Subchapter 7.8 Access and Circulation. The purpose was to introduce the access topics and reserve a detailed discussion for future DCRC meetings.
    - The Committee provided the following feedback and direction for a more detailed discussion of the topic:
        - Request to include the definition of a "perimeter street" to the DDC;
        - Request for examples of direct residential street connections to arterial streets and how this impacts traffic;
        - Request for clarification regarding ADA compliance, regulation of barrier-free ramps in development review, and coordination with Capital Projects to improve pedestrian connectivity;
        - General understanding that the DDC should include minimum threshold to exempt smaller projects from Pedestrian TIA requirement;
        - Request for the City to create a method to collect and organize approved TIAs into a database;
        - Clarification of the proposed amendments to Public Transit and how implementation of DCTA standards will affect development review;

    - Request to discuss language related to street connectivity between subdivisions, including a discussion related to stub-outs to vacant land, block lengths, perimeter block lengths and cul-de-sac requirements;
    - Request to discussion the minimum required bicycle and pedestrian infrastructure within residential streets; and
    - Request to expand amendments to include subdivision regulations regarding block lengths in Subchapter 8.
- **Accessory Dwelling Units**: This topic was prioritized, reviewed by the DCRC, and is scheduled to be considered by City Council on January 13, 2026.
- **Affordable and Missing Middle Housing**: As part of the FY26 Planning division work program, staff will be evaluating and identifying infill development incentives which could include supporting infill housing, as well as regulatory provisions that may be barriers to development.
- **Definitions**: related to adding and clarifying definitions not associated with any other Code amendments. This topic has been identified as one to complete concurrently with the Tree Code Discussions given it is not a time-intensive discussion and could easily be worked into future meetings.
- **Land Disturbance and ESAs**: related to applicability, general regulations, compliance requirements, ESAs, Alternative ESA Plan.
- **Land-use and Land-preservation**: related to coming into compliance with the latest Comprehensive Plan objectives, such as the Preferred Pattern of Preservation and Wildlife Corridors, and including updates to 'transfer of development right, TDR', 'conservation easement', and 'green rights-of-way' amendments".
- **Landscaping, Screening, Buffering and Fences**: This item has been discussed with the DCRC and will be brought forward to Planning and Zoning Commission and City Council in tandem with Tree Code amendments.
- **Low-intensity Neighborhood Commercial**: related to ACUs (Accessory Commercial Units), expand allowable uses along collector streets/intersections, or other ways to allow low-intensity (daytime, low vehicle usage) commercial uses within neighborhoods. Encouraging economic diversity and opportunity through ACUs while minimizing any adverse impacts on surrounding properties and the community.
- **Parking Minimum Ratios**: related to removal of parking minimum ratios following recent Code amendments. Parking requirements were introduced to the DCRC in 2023. Additional discussion surrounding parking is pending the completion of the City-wide Parking Study, and City Council direction regarding next steps.
- **Public Notification**: (COMPLETED)
- **Site and Building Design**: related to garages and building mass and form. (CURRENTLY BEING DISCUSSED)
- **Subdivision Design Improvements**: related to shorter block lengths, link-to-node ratio, bike/pedestrian cut-throughs, broaden the stub-out requirement along abutting property beyond just ‘vacant land’, or other staff ideas to improve connectivity. Amend open space lot requirements to maximize resident use, preserve adaptability, and minimize externalities like increasing speeding due to wide visual road design.
- **Table of Allowed Uses and Use-Specific Standards**: related to adding new uses, amending where existing uses are allowed either by right or by SUP, and adding and amending Use-Specific Standards. This topic has been identified as one to complete concurrently with the Tree Code Discussions given it is not a time-intensive discussion and could easily be worked into future meetings.
- **Tree and Landscaping Requirements**: (CURRENTLY BEING DISCUSSED). Staff has identified areas within the DDC that need to be updated or clarified related to enforcement of the tree preservation regulations. Additionally, staff has identified several areas for improvement and updates in the landscape regulations needed to align with the City’s Water Efficiency Plan Update.

In addition to the above list of topics, the DCRC has requested a discussion related to Detention Ponds/Basins and vegetation plans to determine if this topic is one the Committee is interested in reviewing as part of their upcoming work plan.

Detention Facilities/Systems are defined by the DDC as: Physical provisions to accommodate and regulate stormwater runoff to preclude excessive erosion and sedimentation and to control and regulate the rate of flow. Facilities/systems can include natural features and conduits, channels, ditches, swales, pipes, detention devices or other devices designed or intended to carry, direct, detain or otherwise control stormwater.

References to detention facilities are primarily found in Subsection 7.5.3F.- Detention Facilities of the DDC (Exhibit 2). This Subsection outlines general requirements for detention facilities and refers to the Design Criteria Manual (DCM) for the technical design standards and requirements for construction of the facilities. Section 4.9 of DCM is provided as Exhibit 3. Additionally, Section 4.14 of the DCM discusses Stormwater Facility Maintenance Agreements (SFMAs). These agreements delineate responsibility for detention basin maintenance and provide owners with a site-specific operation and maintenance plan developed by the engineer of record for the project.

The DCM provides additional guidance and support for stormwater facilities by citing sections in the iSWM technical manuals specific to the design of detention structures. The regulations and technical specifications within the various design and technical manuals allow for a variety in design of detention facilities.

During the meeting on January 12, 2026, staff will seek guidance and feedback the Committee regarding prioritization of topics, if any additional topics need to be added, and seek consensus on how to move forward.

EXHIBITS

Exhibit 1 - Agenda Information Sheet
Exhibit 2 – DDC Subsection 7.5.3F
Exhibit 3 – DCM Section 4.9
Exhibit 4 – DCM Section 4.14
Exhibit 5 – iSWM Site Development Controls
Exhibit 6 – iSWM Hydraulics

Respectfully submitted:
Hayley Zagurski, AICP
Planning Director

Prepared by:
Angie Manglaris, AICP
Assistant Planning Director

7.5.3 General Drainage Requirements.

F. **Detention Facilities.** All detention facilities, whether maintained by the city, private property owner, home owners association, or private entity, shall comply with any applicable design requirements of the city and any state or federal laws or regulations, as amended, including the regulations of the Texas Commission on Environmental Quality (TCEQ) or its successor agency. The following standards shall apply to all detention facilities, to the extent they do not conflict with any applicable federal or state laws or regulations, as amended:

1. The 100-year flood shall be used to determine the volume of detention storage required. Water quality volume shall be designed per the Stormwater Design Criteria Manual.
2. Detention facilities shall be designed so that any additional runoff generated by the proposed development will not increase the amount of original discharge for storm frequencies from the 1-year, 25-year, and 100-year flood;
3. Publicly dedicated or privately maintained detention facilities may be used to reduce peak discharges where conditions prevent conveying stormwater to an adequate discharge point or studies show that off-site structural facilities will not mitigate hydraulic effects more efficiently;
4. All detention facilities shall comply with the standards of this section and the Stormwater Design Criteria Manual, as amended;
5. A development may provide for drainage by participating in the design and construction of a regional detention facility. Detailed engineering studies of the entire basin shall be required to ensure that the timing of peak flows has not been altered to create higher peak flows elsewhere in the basin; and
6. Detention facilities may be constructed in phases, if phased to provide for the timely needs of the development.

B. Design guidance is provided in *Section 3.4 of the iSWM™ Hydraulics Technical Manual Document*.

4.8.3 Other Criteria

A. Where applicable, bridges must comply with DDC Section 7.4 regarding Environmentally Sensitive Areas.

4.9 Detention Facilities

4.9.1 Design Requirements

A. Detention facilities shall be designed for the four storms ("*Water Quality*", "*Streambank Protection*", "*Conveyance*", and *"Flood Mitigation"*) for the critical storm duration that results in the maximum (or near maximum) peak flow.

B. Dry detention basins must be sized to temporarily store the volume of runoff required to provide flood protection up to the "*Flood Mitigation*" storm.

C. Routing calculations must be used to demonstrate that the storage volume and outlet structure configuration comply with Section 4.5.5 of this Manual.

D. A calculation summary shall be provided on construction plans as shown in the computation sheet in Figure 4.9 or its equivalent. Stage-storage-discharge values shall be tabulated and flow calculations for discharge structures shall be shown on the construction plans. Detention design shall follow *i*SWM™ guidelines. It is the responsibility of the Engineer of Record to use appropriate methodologies presented in *i*SWM™ based on specific basin characteristics. Detailed calculations and a design narrative shall be provided for review in a supplemental report that is referenced on the construction plans. In general, the narrative shall provide basic design information, such as the hydrologic method applied, design assumptions, pre- and post-development site conditions, downstream constraints, environmental considerations, and design software version used, if applicable.

E. Storage and dam safety design may be subject to the requirements of the Texas Dam Safety Program based on the volume, dam height, and level of hazard. Earthen embankments six (6) feet in height or greater shall be designed per the TCEQ guidelines for dam safety (*See* Texas Administrative Code, Title 30, Part 1, Chapter 299 Dams and Reservoirs for current dam safety criteria).

F. An Operation and Maintenance Manual must be submitted with the civil engineering plans for all private detention basins. Private detention basins shall be inspected by Public Works Inspection to ensure compliance with the design standards required by Section 7.5.3.F. of the DDC. Inspection fees will apply.

4.9.2 Design Criteria for Above Grade Detention Facilities

A. Grading Standards for Above Grade Detention Facilities:

1. Vegetated channel slopes shall not exceed 4H:1V slope. Concrete-lined embankment slopes shall not exceed 2H:1V slope. Vertical walls may be allowed but must be structurally designed to account for inundation of the base and drawdown upon pond draining and must have a 6-ft. high security fence at the top.

2. The side slope for any excavated detention basin, which is not in rock, shall not exceed a 4H:1V slope.
3. Bottom slopes of detention facilities should not be less than 1%.
4. Armored slopes shall be no steeper than 2H:1V.
5. The embankment crown width shall be determined based on a geotechnical investigation of the detention facility site. The minimum width of the embankment crown shall be 12 feet.
6. Earthen embankments used to impound detention water must have a non-permeable core and shall be based on a geotechnical investigation of the site. The geotechnical investigation shall be performed by a Professional Engineer (PE), licensed by the State of Texas, and shall include at a minimum the type of material on-site (or other material to be used in the embankment), moisture content, liquid limit, plasticity index, and required compaction.
7. A Concrete pilot channel with a minimum width of 10 feet, and a minimum slope of 0.5% shall be constructed at the bottom of the detention pond. Privately maintained ponds shall have a concrete pilot channel with a minimum width of six (6) feet.
8. Private Detention Basins shall be designed with 10-ft. wide unobstructed maintenance access around the entire perimeter of the pond.
9. Public Detention Basins shall be designed with 20-ft. wide unobstructed maintenance access around the entire perimeter of the pond.
10. Where deemed necessary by the City's Engineer, security fencing with a minimum height of six (6) feet shall encompass the detention storage area if the velocity, depth, or slopes create a potentially dangerous condition. The fence shall be designed to allow access for maintenance and so as not to restrict stormwater flow into or out of the detention basin. A maintenance equipment access ramp shall be provided for all detention facilities. The slope of the ramp shall not exceed 6H:1V and the minimum width shall be 12 feet.

B. Emergency Spillway, Overflow Path, and Freeboard:

1. A freeboard of one (1) foot will be required between the *"Flood Mitigation"* stormwater surface elevation and top of bank.
2. An emergency spillway shall be provided at the flood mitigation maximum storage elevation with sufficient capacity to convey *"Flood Mitigation"* storm inflow rates with six (6) inches of freeboard. This is the peak of the inflow hydrograph coming into the pond and must not account for attenuation effects of the pond. Spillway requirements must also meet all appropriate State and Federal criteria.
3. An emergency overflow path, free of structures or obstructions, must be provided to convey the spillway design discharge to a downstream ROW or drainageway with adequate capacity for the discharge. No impediments to flow are allowed within the emergency overflow path (E.g., fences, trees, parking areas, or buildings). If a fence around a detention facility is needed to restrict access, the bottom of the fence must be elevated to provide a minimum of one foot of freeboard above the emergency overflow WSEL where the fence crosses the spillway. Nevertheless, fences across spillways should be avoided whenever possible. Even if the fence is elevated to the freeboard elevation, there is still the potential for the fence to catch debris floating on the water surface.
4. An emergency spillway must be constructed of concrete.

5. Design calculations shall be provided for all spillways and outlet structures.

C. Landscaping Requirements:

1. All detention basins shall be designed with plantings that minimize erosion based on expected inundation frequencies. Design guidance is provided in the *iSWM™ Landscape Technical Manual Document*.

D. Multiple Use Guidance

Limited recreational equipment (such as picnic tables or playground equipment) and trees may be permitted in private detention facilities with the following restrictions:

1. User access must be provided at a maximum 10% slope in at least two (2) locations, or one (1) location that comprises not less than 20% of the perimeter of the facility.

2. No recreational equipment is permitted in any portion of the facility that lies more than 18 inches below the flood mitigation water surface elevation level of the facility.

3. Recreational structures including, but not limited to, picnic tables and playground equipment, must be rust-resistant and anchored to the ground.

4. Mulch, wood chips, gravel, or rubberized pellets are not permitted within a detention facility due to the likelihood of their floating into the outlet structure.

5. Trees, shrubs, and other woody vegetation will not be permitted in the embankment of any detention facility or in the maintenance access area around such a facility.

6. A maximum of one (1) isolated tree per 5,600 sq. ft. may be permitted in the recreational area of the pond. A trash rack must be used to prevent clogging of the outlet structure. No bark mulch may be used around trees.

E. Retaining Walls

1. Any freestanding retaining wall used to detain water must be designed by a structural engineer to withstand the expected hydraulic forces when the detention area is filled to capacity. These walls must be constructed using reinforced concrete.

2. Any inlet or pipe connections to retaining walls must be designed with a reinforced concrete headwall. The height of this headwall shall match the height of the adjacent retaining wall. The most recent TxDOT details for concrete headwalls shall be used to determine other headwall dimensions and reinforced schedule.

4.9.3 Design Criteria for Underground Detention Facilities

Underground detention is highly discouraged because of the potential for deferred maintenance, the difficult and potentially hazardous nature of access for maintenance, and issues related to anaerobic conditions and pollutant mobility from devices that retain water between events. In any instance where underground detention is contemplated, thorough consideration must be given to the concerns described above to ensure ongoing inspection, maintenance, and functionality. Care should be taken to address material selection for underground detention due to the potential for adverse soil conditions to inhibit the system from functioning properly. A geotechnical engineer shall be consulted to ensure soil and other conditions are appropriate for the selected detention material.

A. Underground detention facilities shall be designed with reinforced concrete and have a minimum pipe diameter of 30 inches to allow for safe access and maintenance of the facility.

B. If an underground vault has multiple chambers, access openings with a maximum spacing of 500 feet must be provided for each chamber.

C. Underground facilities must be located to enable safe access for maintenance and minimize disruption of aboveground uses during maintenance. Access openings shall not be placed in areas that are routinely used for parking.

D. Easements must be provided for all underground detention facilities. If underground detention facilities are bound by private property, the easements must include an additional four (4) feet from the perimeter of the facilities.

E. Underground detention and water quality facilities are prohibited underneath buildings, walls, or other structures.

F. Every SWFMA involving underground detention and water quality facilities must include inspection and maintenance requirements to be performed quarterly and following any rainfall event of 0.5 inches or more. The inspection and maintenance frequency may be reduced after five (5) years of operation, if the owner demonstrates that a lesser frequency is appropriate.

G. A surface emergency overflow path, free of obstructions, must be provided for all underground detention facilities. The emergency overflow path must have sufficient capacity to convey the *"Flood Mitigation"* storm inflow rates to the underground detention facility. This is the peak of the inflow hydrograph coming into the underground detention facility, and must not account for any attenuation effects from the facility. The criteria for emergency overflow paths for surface detention facilities denoted in Section 4.9.2 above also apply to underground detention facilities.

H. Outlets from underground detention must consist of a pipe that can convey 120% of the 100-year outflow, with a minimum diameter of 12 inches. The invert of the outlet pipe must be at the lowest point in the detention facility to ensure that it fully drains.

I. Underground detention facilities shall be sloped to drain at a minimum floor slope of one (1) %.

4.9.4 Design Criteria for Parking Lot Detention

Parking lot detention may be allowed if the following minimum criteria are met:

A. Use of parking lot surface area as detention is permitted, but only up to the lowest curb elevation of the parking lot.

B. The maximum ponding depth for the 100-year storm must be no more than 12 inches at the deepest point.

C. The outlet must be designed to minimize modifications that affect detention functions. The applicant must evaluate potential future resurfacing activities for impacts to detention volumes and release rates.

D. Ponding water in frequently used portions of parking lots must be avoided. At least two signs are required for all parking lot detention areas. The signs must have a minimum area of 1.5 sq. ft. and contain the following message:

WARNING

This area is a detention basin and is subject to periodic flooding to a depth of (provide design depth).

Sign materials, geometry, and location must be submitted to Development Facilitation and approved by the City's Engineer.

4.9.5 Design Criteria for Pumped Detention

A. A detention facility may not rely on a pump or other mechanical equipment to drain water during a design storm.

4.9.6 Outlet Structures for Detention Facilities

A. Design Frequency

1. *"Water Quality"* storm
2. *"Streambank Protection"* storm
3. *"Conveyance"* storm
4. *"Flood Mitigation"* storm

B. Design Criteria

1. Outlet structures shall be designed in accordance with Section 2.2 of the *iSWM™ Hydraulics Technical Manual*. For water quality, refer to Section 2.2.3 for design of extended detention outlets.
2. The required storage volumes for "*Water Quality*", "*Streambank Protection*", "*Conveyance*", and *"Flood Mitigation"* storm events must be estimated.
3. If a detention facility includes extended detention, refer to Section 2.2.3 of the *iSWM™ Hydraulics Technical Manual* for design requirements.
4. All outlet orifices must be adequately protected from clogging and designed to ensure that people and large animals are kept out of confined outlet areas. Refer to Sections 2.2.5-*Extended Detention Outlet Protection* and 2.2.6-*Trash Racks and Safety Grates* of the *iSWM™ Hydraulics Technical Manual* for design guidance.
5. Any top orifice on an outlet riser must be designed with a grate to prevent fall injuries.
6. Outlet velocities shall be within the maximum allowable range based on channel material, as shown in Table 4.6-D and Table 4.6-E.
7. Outlet protection and energy dissipation facilities must be designed to avoid erosion problems downstream from outlet devices and emergency spillway(s).
8. Buoyancy calculations must be performed for the outlet structure and footing to ensure the outlet structure will not float. Flotation will occur when the weight of the structure is less than or equal to the buoyant force exerted by the water.
9. Any outflow structure that conveys water through an embankment in a conduit shall be reinforced concrete designed to support the external loads. The conduit shall be able to withstand the internal hydraulic pressure without leakage under full external load or settlement and must convey water at the design velocity without damage to the interior surface of the conduit.
10. The minimum opening of an inlet shall be six (6) inches in diameter or a 6-in. by 6-in. square. Smaller inlet openings may be used with a junction box and properly sized outlet structure.

11. A concrete headwall and wingwalls shall be constructed at the outlet pipe opening. Orientation of the wingwalls will be governed by site specific conditions. Headwalls and wingwalls shall be designed to TxDOT standards.

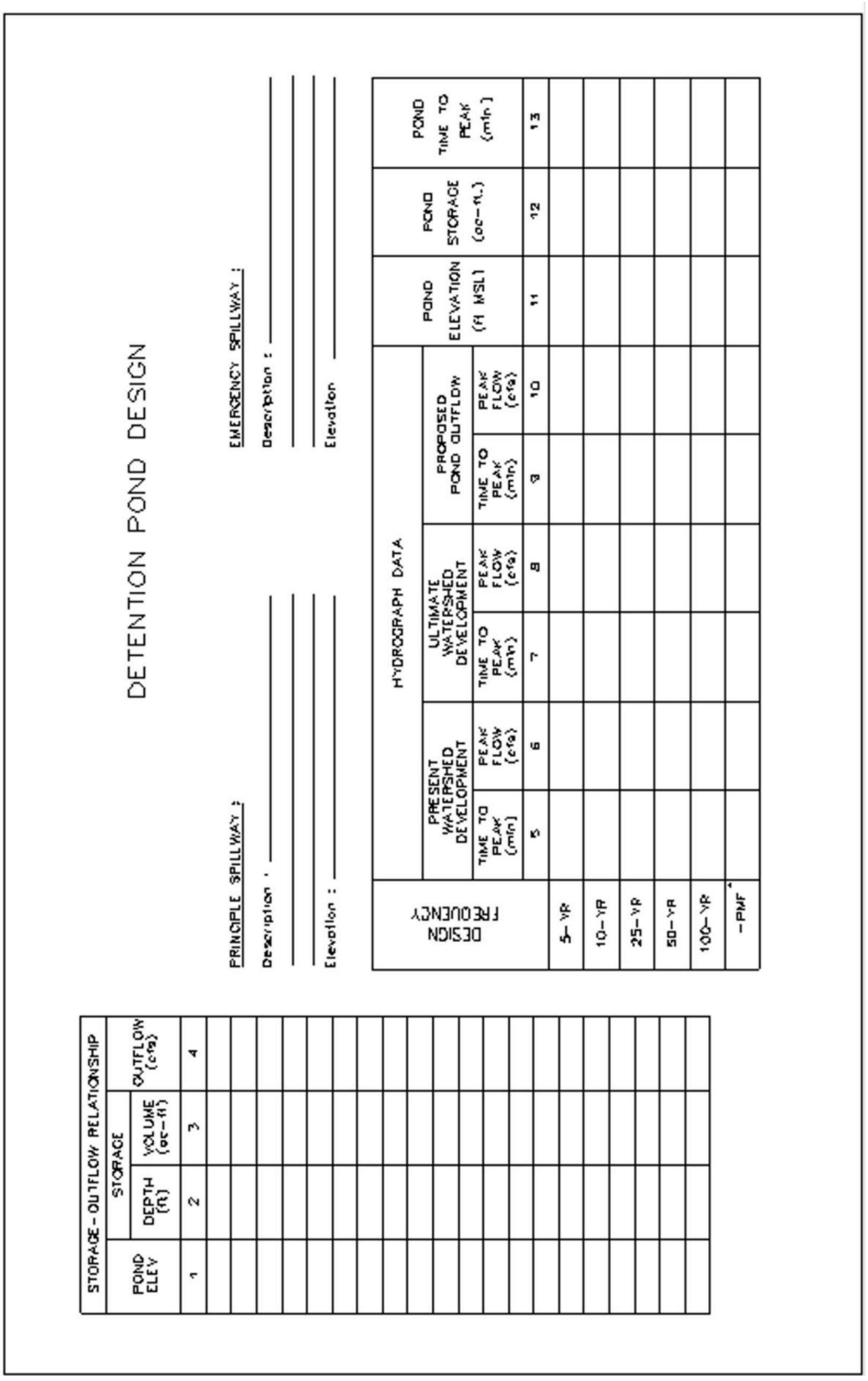


Figure 4.9 Computation Sheet for Detention Pond Design

iv. Permanent stabilization must be initiated within 48 hours of completion of construction activities on areas of the site and completed for project prior to issuance of final Certificate of Occupancy.

v. Vegetative stabilization must demonstrate consistent progress toward 70% density of perennial vegetative cover.

3. Reinspection

a. Reinspection is conducted for sites that are out of compliance.

i. Standard timeframe for corrections is seven (7) business days.

ii. Enforcement may be escalated if there are impacts to offsite property or waterways.

iii. A reinspection fee may be assessed.

b. Reinspection with Notice of Violation (NOV) will be issued for continued stormwater non-compliance affecting stormwater, with reinspection fee assessed.

c. Reinspection with Stop Work Order (SWO) – Escalation issued for continued non-compliance affecting stormwater through failed erosion and sediment controls. Reinspection fee assessed.

i. Effective until deficiencies are corrected.

ii. Erosion and sediment control failures which threaten life or property may result in SWO without prior reinspection or NOV.

d. Reinspection Fees

i. Applicability – applies only to reinspection due to failure to comply with site stormwater quality requirements.

ii. Amount – As established in the Water and Wastewater Rate Book.

iii. Time of Payment – Payment is due per standard City of Denton utility billing dates.

4.14 Stormwater Facility Maintenance Agreements

4.14.1 Maintenance Agreements

All drainage improvements constructed within a development and any existing or natural drainage systems shall require a maintenance agreement that identifies responsible parties for maintenance. The maintenance agreement shall be written such that it remains in force upon sale of transfer of the property.

As part of the Operations and Maintenance Plan submittal, a SWFMA must be prepared by the engineer of record for each stormwater control that will not be wholly maintained by the City. This agreement must outline preventive maintenance tasks and major repairs, identify the schedule for each task, assign clear roles to affected parties, and provide a maintenance checklist to guide future owners, including an annual self-inspection to be provided to the City. Multiple stormwater controls may be contained within a single Stormwater Facility Maintenance Agreement. When areas are identified for detention that also serve other purposes for the development (e.g. parking lots, loading docks) the requirement for a SWFMA may be waived.

4.14.2 Private Maintenance (SWFMA Required)

A. Private drainage facilities include those drainage improvements which are located on private property and which serve the needs of private development.

B. Private drainage facilities may also include detention or retention ponds, dams, and retaining walls intended to direct or contain runoff. Such facilities must be designed in accordance with sound engineering practices and reviewed and inspected by the City.

C. All SWFMA exhibits shall be reviewed and approved by the City with the civil engineering plans. The agreement for perpetual maintenance of private drainage facilities shall be executed with the City during per-construction. This agreement shall run with the land and can be tied to commercial property or to an owner's association, but not to individual residential lots.

D. The SWFMA shall provide the City with access to all private drainage facilities.

4.14.3 Maintenance Agreement Requirements

Details of the SWFMA must be set forth in a series of exhibits:

A. Exhibit A Legal Description - This includes the Metes and Bounds, a Surveyor's Drawing of the area occupied by the facility, and a copy of the preliminary or recorded plat containing the facility.

B. Exhibit B Design Plan and Specifications - These are summary documents intended for the use of future owners in conducting routine maintenance, inspections, and repairs. The documents include:

1. Design Data and Calculations - This can be in the form of a letter or statement from the engineer of record which summarizes critical design calculations related to the functionality of the facility, such as storage volume or TSS removal, and attests to the facility conforming to applicable *i*SWM standards;

2. Schematic Plan - This should be prepared by the engineer of record from construction drawings to show the general layout of the facility. Major features requiring regular or special maintenance should be shown and labeled in general terms understandable to a layman. A profile should be given showing critical elevations that control the function and capacity of the facility, and one or more cross-sections should be provided to indicate the general grading of the facility. A typical example of a schematic plan for a simple detention basin is shown in Figure 4.13 below; and

3. Landscaping - Vegetation should be shown consistent with the accepted Landscape Plan, either on the Schematic Plan or as a separate drawing.

C. Exhibit C Operations and Maintenance Plan - Specific maintenance tasks should be defined for each element of the facility. Maintenance tasks specific to the facility should be described in simple terms consistent with terminology contained in the Schematic and Landscape plans. An inspection and maintenance frequency should be established for each task.

D. Exhibit D Maintenance Checklist - A checklist consistent with the Operations and Maintenance Plan shall be provided for the use of future owners in performing routine and special maintenance tasks. This list should describe work required and frequency in language that is easy to understand and specific for the facility to be maintained. This form will be completed by the Owner and submitted to the City annually as part of a regular self-inspection program. *See* Appendix A for an example checklist for preparing a SWFMA for a simple detention basin.

Additional facility maintenance guidance for several types of stormwater controls is provided in the *iSWM Technical Manual*. The engineer of record must certify that the construction has been completed in accordance with the general plans and Schematic Plan. After approval of construction by the City, the engineer of record is expected to provide guidance to the owner's representative in implementing the accepted maintenance program and to co-sign the first annual inspection after the construction.

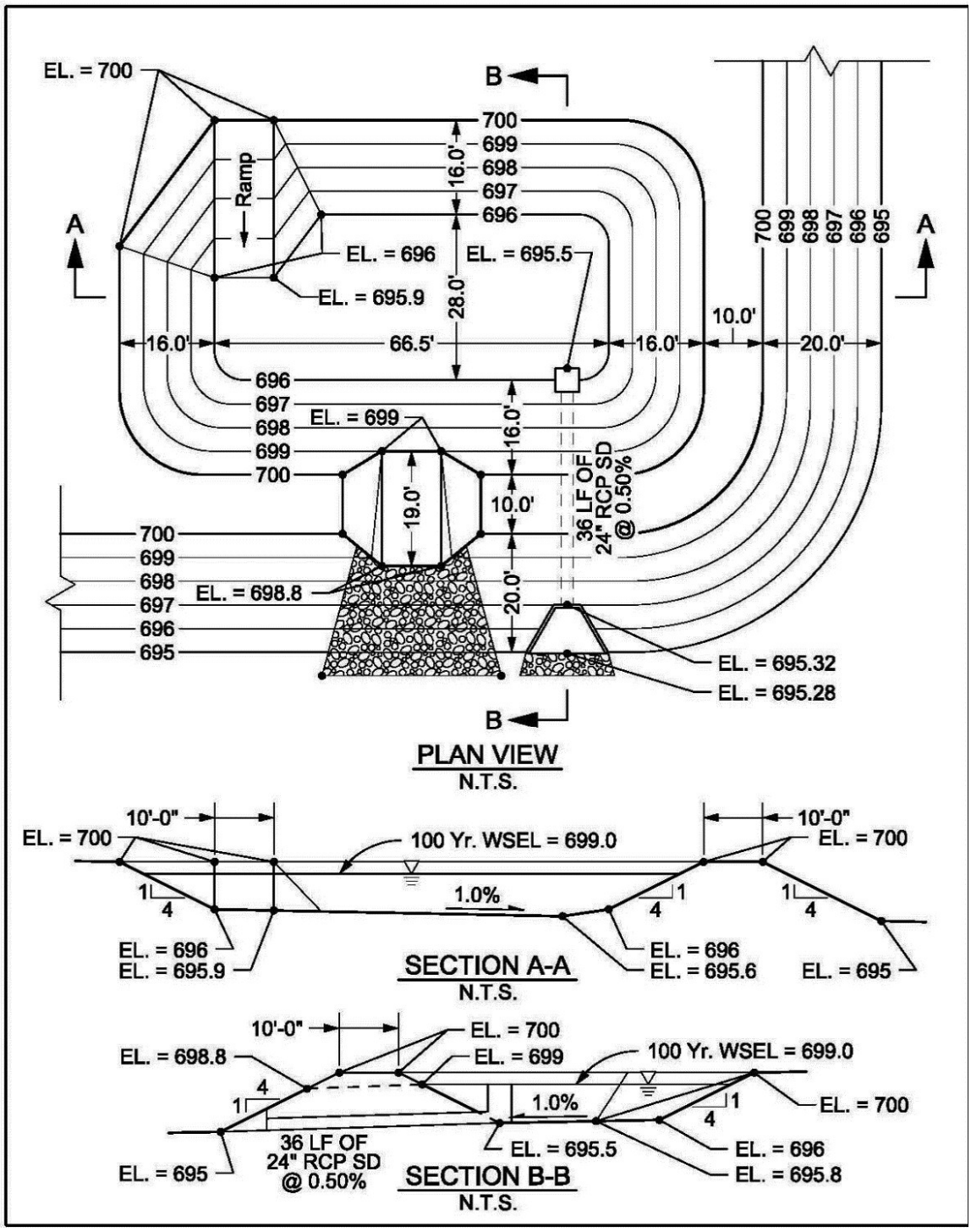


Figure 4.12 Simple Detention Basin Plan Schematic

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1.0 Overview of Stormwater Controls for Site Development

1.1 Stormwater Controls - Categories and Applicability

1.1.1 Introduction

Structural stormwater controls are engineered facilities intended to treat stormwater runoff and/or mitigate the effects of increased stormwater runoff peak rate, volume, and velocity due to urbanization. This section provides an overview of structural stormwater controls that can be used to address the minimum stormwater management standards outlined in *Section 1.1.2*.

In terms of the *Integrated* Design Focus Areas, a structural stormwater control, or set of structural controls, must:

- **Water Quality:** Remove pollutants in stormwater runoff to protect water quality;
- **Streambank Protection:** Regulate discharge from the site to minimize downstream bank and channel erosion; and
- **Flood Control:** Control conveyance of runoff within and from the site to minimize flood risk to people and properties.

1.1.2 Control Categories

The stormwater control practices recommended in this Manual vary in their applicability and ability to meet stormwater management goals:

Primary Controls

Primary Structural Stormwater Controls have the ability to fully address one or more of the Steps in the *integrated* Design Focus Areas if designed appropriately. Structural controls are recommended for use with a wide variety of land uses and development types. These structural controls have a demonstrated ability to effectively treat the Water Quality Volume (WQ_v) and have been shown to be able to remove 70% to 80% of the annual average total suspended solids (TSS) load in typical post-development urban runoff when designed, constructed, and maintained in accordance with recommended specifications. Several of these structural controls can also be designed to provide primary control for downstream streambank protection (SP_v) and flood control (Q_f). These structural controls are recommended stormwater management facilities for a site wherever feasible and practical.

Secondary Controls

However, a number of structural controls are recommended only for limited use or for special site or design conditions. Generally, these practices either: (1) do not have the ability on their own to fully address one or more of the Steps in the *integrated* Design Focus Areas, (2) are intended to address hotspot or specific land use constraints or conditions, and/or (3) may have high or special maintenance requirements that may preclude their use. These types of structural controls are typically used for *water quality treatment only*. Some of these controls can be used as a pretreatment measure or in series with other structural controls to meet pollutant removal goals. Such structural controls should be considered mostly for commercial, industrial, or institutional developments.

Table 1.1 lists the structural stormwater control practices. These structural controls are recommended for use in a wide variety of applications. A detailed discussion of each of the controls, as well as design criteria and procedures can be found in Sections 2 through 28 of this manual.

Table 1.1 Structural Controls

Structural Control	Description
Bioretention Areas	*Bioretention areas* are shallow stormwater basins or landscaped areas which utilize engineered soils and vegetation to capture and treat stormwater runoff. Runoff may be returned to the conveyance system, or allowed to partially exfiltrate into the soil.
Channels • Enhanced Swale (Dry, Wet, or Wetland) • Grass Channel (biofilter)	• *Enhanced swales* are vegetated open channels that are explicitly designed and constructed to capture and treat stormwater runoff within dry or wet cells formed by check dams or other means • *Grass channels* provide "biofiltering" of stormwater runoff as it flows across the grass surface. However, a grass channel alone cannot meet the 70% TSS removal performance goal. Consequently, grass channels should only be used as pretreatment measure or as part of a treatment train approach.
Chemical Treatment • Alum Treatment	*Alum treatment* provides for the removal of suspended solids from stormwater runoff entering a wet pond by injecting liquid alum into storm sewer lines on a flow-weighted basis during rain events. Alum treatment should only be considered for large-scale projects where high water quality is desired.
Conveyance Components • Culvert • Inlet • Pipe Systems • Energy Dissipators • Open Conveyance Channel	• A *culvert* is a short, closed (covered) conduit that conveys stormwater runoff under an embankment, usually a roadway. • *Inlets* are drainage structures used to collect surface water through grate or curb openings and convey it to storm drains or direct outlet to culverts. • *Pipe systems* are used for transporting runoff from roadway and other inlets to outfalls at structural stormwater controls and receiving waters. Culverts, inlets, and pipe systems alone do not provide water quality treatment.
Detention • Dry Detention / Dry Extended Detention Basins • Multi-Purpose Detention Areas • Underground Detention	• *Dry detention basins and dry extended detention (ED) basins* are surface facilities intended to provide for the temporary storage of stormwater runoff to reduce downstream water quantity impacts. • *Multi-purpose detention areas* are site areas used for one or more specific activities, such as parking lots and rooftops, which are also designed for the temporary storage of runoff. • *Underground detention tanks and vaults* are an alternative to surface dry detention for space-limited areas where there is not adequate land for a dry detention basin or multi-purpose detention area.

Table 1.1 Structural Controls

Structural Control	Description
Filtration • Filter Strip • Organic Filter • Planter Boxes • Surface Sand Filter/ Perimeter Sand Filter • Underground Sand Filter	• *Filter strips* provide "biofiltering" of stormwater runoff as it flows across the grass surface. However, filter strips alone cannot meet the 70% TSS removal performance goal. Consequently, filter strips should only be used as pretreatment measure or as part of a treatment train approach. • *Organic filters* are surface sand filters where organic materials such as a leaf compost or peat/sand mixture are used as the filter media. These media may be able to provide enhanced removal of some contaminants, such as heavy metals. Given their potentially high maintenance requirements, they should only be used in environments that warrant their use. • *Planter boxes* are used on impervious surfaces in highly urbanized areas to collect and detain / infiltrate rainfall and runoff. The boxes may be prefabricated or constructed in place and contain growing medium, plants, and a reservoir. • *Sand filters* are multi-chamber structures designed to treat stormwater runoff through filtration, using a sand bed as its primary filter media. Filtered runoff may be returned to the conveyance system, or allowed to partially exfiltrate into the soil. • *Underground sand filters* are sand filter systems located in an underground vault. These systems should only be considered for extremely high density or space-limited sites.
Hydrodynamic Devices • Gravity (Oil-Grit) Separator	*Hydrodynamic controls* use the movement of stormwater runoff through a specially designed structure to remove target pollutants. They are typically used on smaller impervious commercial sites and urban hotspots. These controls typically do not meet the Primary TSS removal performance goal and therefore should only be used as a pretreatment measure and as part of a treatment train approach.
Infiltration • Downspout Dry Wells • Infiltration Trench • Soakage Trenches	• *Downspout dry wells* are essentially perforated manholes, but they can be manufactured in various sizes. Located underground, they allow stormwater infiltration even in highly urbanized areas. They should be used in conjunction with some type of pretreatment devices where there are minimal risks of groundwater contamination. • An *infiltration trench* is an excavated trench filled with stone aggregate used to capture and allow infiltration of stormwater runoff into the surrounding soils from the bottom and sides of the trench. • *Soakage trenches* are a variation of infiltration trenches. Soakage trenches drain through a perforated pipe buried in gravel. They are used in highly impervious areas where conditions do not allow surface infiltration and where pollutant concentrations in runoff are minimal (i.e. non-industrial rooftops). They may be used in conjunction with other stormwater devices, such as draining downspouts or planter boxes.

Table 1.1 Structural Controls

Structural Control	Description
Stormwater Ponds • Micropool Extended Detention Pond • Multiple Pond Systems • Wet Extended Detention Pond • Wet Pond	*Stormwater ponds* are constructed stormwater retention basins that have a permanent pool (or micropool) of water. Runoff from each rain event is detained and treated in the pool.
Porous Surfaces • Green Roofs • Modular Porous Paver Systems • Porous Concrete	• A *green roof* uses a small amount of substrate over an impermeable membrane to support a covering of plants. The green roof slows down runoff from the otherwise impervious roof surface as well as moderating rooftop temperatures. With the right plants, a green roof will also provide aesthetic or habitat benefits. • *Modular porous paver systems* consist of open void paver units laid on a gravel subgrade. Both porous concrete and porous paver systems provide water quality and quantity benefits, but have high workmanship and maintenance requirements, as well as high failure rates. • *Porous surfaces* are permeable pavement surfaces with an underlying stone reservoir to temporarily store surface runoff before it infiltrates into the subsoil. *Porous concrete* is the term for a mixture of course aggregate, Portland cement, and water that allows for rapid infiltration of water.
Proprietary Systems • Commercial Stormwater Controls	*Proprietary controls* are manufactured structural control systems available from commercial vendors designed to treat stormwater runoff and/or provide water quantity control. Proprietary systems often can be used on small sites and in space-limited areas, as well as in pretreatment applications. However, proprietary systems are often more costly than other alternatives, may have high maintenance requirements, and often lack adequate independent performance data.
Re-Use • Rain Harvesting (tanks/barrels)	*Rain harvesting* is a container or system designed to capture and store rainwater discharged from a roof. The rain harvesting system consists of a storage container, a downspout diversion, a sealed lid, and an overflow system. Typical rain harvesting systems hold between 50 and 500 gallons of water and may work in series to provide larger volumes of storage.
Stormwater Wetlands • Extended Detention Shallow Wetland • Pocket Wetland • Pond/Wetland Systems • Shallow Wetland • Submerged Gravel Wetlands	• *Stormwater wetlands* are constructed wetland systems used for stormwater management. Stormwater wetlands consist of a combination of shallow marsh areas, open water, and semi-wet areas above the permanent water surface. • *Submerged gravel wetland systems* use wetland plants in submerged gravel or crushed rock media to remove stormwater pollutants. These systems should only be used in mid- to high-density environments where the use of other structural controls may be precluded. The long-term maintenance burden of these systems is uncertain.

1.1.3 Using Other or New Structural Stormwater Controls

Innovative technologies should be allowed and encouraged providing there is sufficient documentation as to their effectiveness and reliability. Communities can allow controls not included in this Manual at their discretion, but should not do so without independently derived information concerning performance, maintenance, application requirements, and limitations.

More specifically, new structural stormwater control designs will not be accepted for inclusion in the manual until independent performance data shows that the structural control conforms to local and/or State criteria for treatment, conveyance, maintenance, and environmental impact.

1.2 Suitability of Stormwater Controls

Some structural stormwater controls are intended to provide water quality treatment for stormwater runoff. Though most of these structural controls provides pollutant removal capabilities, the relative capabilities vary between structural control practices and for different pollutant types.

1.2.1 Water Quality

Pollutant removal capabilities for a given structural stormwater control practice are based on a number of factors including the physical, chemical, and/or biological processes that take place in the structural control and the design and sizing of the facility. In addition, pollutant removal efficiencies for the same structural control type and facility design can vary widely depending on the tributary land use and area, incoming pollutant concentration, flow rate, volume, pollutant loads, rainfall pattern, time of year, maintenance frequency, and numerous other factors.

To assist the designer in evaluating the relative pollutant removal performance of the various structural control options, Table 1.2 provides design removal efficiencies for each of the control practices. It should be noted that these values are *conservative* average pollutant reduction percentages for design purposes derived from sampling data, modeling, and professional judgment. A structural control design may be capable of exceeding these performances, however the values in the table are minimum reasonable values that can be assumed to be achieved when the structural control is sized, designed, constructed, and maintained in accordance with recommended specifications in this Manual.

Where the pollutant removal capabilities of an individual structural stormwater control are not deemed sufficient for a given site application, additional controls may be used in series in a “treatment train” approach. More detail on using structural stormwater controls in series is provided in *Section 1.6*.

For additional information and data on the range of pollutant removal capabilities for various structural stormwater controls, the reader is referred to the National Pollutant Removal Performance Database (2nd Edition) available at www.cwp.org and the International Stormwater Best Management Practices (BMP) Database at www.bmpdatabase.org

Table 1.2 Design Pollutant Removal Efficiencies for Stormwater Controls (Percentage)

Structural Control	Total Suspended Solids	Total Phosphorus	Total Nitrogen	Fecal Coliform	Metals
Bioretention Areas	80	60	50	---	80
Grass Channel	50	25	20	---	30
Enhanced Dry Swale	80	50	50	---	40
Enhanced Wet Swale	80	25	40	---	20
Alum Treatment	80	80	60	90	75
Filter Strip	50	20	20	---	40
Dry Detention	65	50	30	70	---
Organic Filter	80	60	40	50	75

Table 1.2 Design Pollutant Removal Efficiencies for Stormwater Controls (Percentage)

Structural Control	Total Suspended Solids	Total Phosphorus	Total Nitrogen	Fecal Coliform	Metals
Planter Boxes	80	60	40	50	60
Sand Filters	80	50	25	40	50
Underground Sand Filter	80	50	25	40	50
Gravity (Oil-Grit) Separator	40	5	5	---	---
Downspout Drywell	80	60	60	90	90
Infiltration Trench	80	60	60	90	90
Soakage Trench	80	60	60	90	90
Stormwater Ponds	80	50	30	70*	50
Green Roof	85	---	25	---	95
Modular Porous Paver Systems with infiltration	**	80	80	---	90
Porous Concrete with infiltration	**	50	65	---	60
Proprietary Systems	***	***	***	***	***
Rain Harvesting	---	---	---	---	---
Stormwater Wetlands	80	40	30	70*	50
Submerged Gravel Wetland	80	50	20	70	50

* If no resident waterfowl population present

** Due to the potential for clogging, porous concrete and modular block paver systems should not be used for the removal of sediment or other coarse particulate pollutants

*** The performance of specific proprietary commercial devices and systems must be provided by the manufacturer and should be verified by independent third-party sources and data

--- Insufficient data to provide design removal efficiency

1.2.2 Streambank Protection

These controls have the ability to detain the volume and regulate the discharge of the 1-year, 24-hour storm event to protect natural waterways downstream of the development. Controls that provide streambank protection include detention, energy dissipation, stormwater ponds, stormwater wetlands, and pipe systems.

1.2.3 Flood Control

- **On-Site:** These controls have the ability to safely convey stormwater through a development to minimize the flood risk to persons and property on-site. On-site flood control structures include channels, culverts, detentions, enhanced swales, open conveyance channels, stormwater ponds, conveyance components (inlets and pipe systems), and stormwater wetlands.
- **Downstream:** These controls have the ability to detain the volume and regulate the discharge from the controlling storm event, as determined by downstream assessment, and to minimize flood risk to persons and property downstream of the development. Downstream flood controls include open channels, pipe systems, detention, stormwater ponds, and stormwater wetlands.

1.3 Stormwater Control Selection

1.3.1 Control Screening Process

Outlined below is a screening process for structural stormwater controls which can effectively treat the water quality volume as well as provide water quantity control. This process is intended to assist the site designer and design engineer in the selection of the most appropriate structural controls for a development site, and provides guidance on factors to consider in their location.

In general the following four criteria should be evaluated in order to select the appropriate structural control(s) or group of controls for a development:

- Stormwater Treatment Suitability
- Water Quality Performance
- Site Applicability
- Implementation Considerations

In addition, for a given site, the following factors should be considered and any specific design criteria or restrictions need to be evaluated:

- Physiographic Factors
- Soils
- Special Watershed or Stream Considerations

Finally, environmental regulations should be considered as they may influence the location of a structural control on site, or may require a permit.

The following pages provide a selection process for comparing and evaluating various structural stormwater controls using a screening matrix and a list of location and permitting factors. These tools are provided to assist the design engineer in selecting the subset of structural controls that will meet the stormwater management and design objectives for a development site or project.

Step 1 Overall Applicability

Through the use of the first four screening categories in Table 1.3, the site designer evaluates and screens the overall applicability of the full set of structural controls as well as the constraints of the site in question. The following are the details of the various screening categories and individual characteristics used to evaluate the structural controls.

Stormwater Management Suitability

The first category in the Matrix examines the capability of each structural control option to provide water quality treatment, downstream streambank protection, and flood control. A blank entry means that the structural control cannot or is not typically used to meet an *integrated* Design Focus Areas. This does not necessarily mean that it should be eliminated from consideration, but rather is a reminder that more than one structural control may be needed at a site (e.g., a bioretention area used in conjunction with dry detention storage).

Ability to treat the Water Quality Volume (WQ_v). This indicates whether a structural control provides treatment of the water quality volume (WQ_v). The presence of a "P" or an "S" indicates whether the control is a Primary or Secondary control for meeting the TSS reduction goal.

Ability to provide Streambank Protection (SP_v). This indicates whether the structural control can be used to provide the extended detention of the streambank protection volume (SP_v). The presence of a "P" indicates that the structural control can be used to meet SP_v requirements. An "S" indicates that the structural control may be sized to provide streambank protection in certain situations, for instance on small sites.

Ability to provide Flood Control (Q_f). This indicates whether a structural control can be used to meet the flood control criteria. The presence of a "P" indicates that the structural control can be used to provide peak reduction of the flood mitigation storm event.

Relative Water Quality Performance

The second category of the Matrix provides an overview of the pollutant removal performance of each structural control option, when designed, constructed, and maintained according to the criteria and specifications in this Manual.

Ability to provide TSS and Sediment Removal. This column indicates the capability of a structural control to remove sediment in runoff. All of the Primary structural controls are presumed to remove 70% to 80% of the average annual total suspended solids (TSS) load in typical urban post-development runoff (and a proportional removal of other pollutants).

Ability to provide Nutrient Treatment. This column indicates the capability of a structural control to remove the nutrients nitrogen and phosphorus in runoff, which may be of particular concern with certain downstream receiving waters.

Ability to provide Bacteria Removal. This column indicates the capability of a structural control to remove bacteria in runoff. This capability may be of particular focus in areas with public beaches, shellfish beds, or to meet water regulatory quality criteria under the Total Maximum Daily Load (TMDL) program.

Ability to accept Hotspot Runoff. This last column indicates the capability of a structural control to treat runoff from designated hotspots. Hotspots are land uses or activities which produce higher concentrations of trace metals, hydrocarbons, or other priority pollutants. Examples of hotspots might include: gas stations, convenience stores, marinas, public works storage areas, garbage transfer facilities, material storage sites, vehicle service and maintenance areas, commercial nurseries, vehicle washing/steam cleaning, landfills, construction sites, industrial sites, industrial rooftops, and auto salvage or recycling facilities. A check mark indicates that the structural control may be used on hotspot site; however, it may have specific design restrictions. Please see the specific design criteria of the structural control for more details. Local jurisdictions may have other site uses which they designate as Hotspots, so their criteria should be checked as well.

Site Applicability

The third category of the Matrix provides an overview of the specific site conditions or criteria that must be met for a particular structural control to be suitable. In some cases, these values are recommended values or limits and can be exceeded or reduced with proper design or depending on specific circumstances. Please see the specific criteria section of the structural control for more details.

Drainage Area. This column indicates the approximate minimum or maximum drainage area considered suitable for the structural control practice. If the drainage area present at a site is slightly greater than the maximum allowable drainage area for a practice, some leeway can be permitted if more than one practice can be installed. The minimum drainage areas indicated for ponds and wetlands should not be considered inflexible limits, and may be increased or decreased depending on water availability (baseflow or groundwater), the mechanisms employed to prevent outlet clogging, or design variations used to maintain a permanent pool (e.g., liners).

Space Required (Space Consumed). This comparative index expresses how much space a structural control typically consumes at a site in terms of the approximate area required as a percentage of the impervious area draining to the control.

Slope. This column evaluates the effect of slope on the structural control practice. Specifically, the slope restrictions refer to how flat the area where the facility is installed must be and/or how steep the contributing drainage area or flow length can be.

Minimum Head. This column provides an estimate of the minimum elevation difference needed at a site (from the inflow to the outflow) to allow for gravity operation within the structural control.

Water Table. This column indicates the minimum depth to the seasonally high water table from the bottom or floor of a structural control.

Implementation Considerations

The fourth category in the Matrix provides additional considerations for the applicability of each structural control option.

Residential Subdivision Use. This column identifies whether or not a structural control is suitable for typical residential subdivision development (not including high-density or ultra-urban areas).

Ultra-Urban. This column identifies those structural controls appropriate for use in very high-density (ultra-urban) areas, or areas where space is a premium.

Construction Cost. The structural controls are ranked according to their relative construction cost per impervious acre treated, as determined from cost surveys.

Maintenance. This column assesses the relative maintenance effort needed for a structural stormwater control, in terms of three criteria: frequency of scheduled maintenance, chronic maintenance problems (such as clogging), and reported failure rates. It should be noted that **all structural controls** require routine inspection and maintenance.

Step 2 Specific Criteria

The last three categories in the Structural Control Screening matrix (Table 1.3) provides an overview of various specific design criteria and specifications, or exclusions for a structural control that may be present due to a site's general physiographic character, soils, or location in a watershed with special water resources considerations.

Physiographic Factors

Three key factors to consider are low-relief, high-relief, and karst terrain. In the North Central Texas, low relief (very flat) areas are primarily located east of the Dallas metropolitan area. High relief (steep and hilly) areas are primarily located west of the Fort Worth metropolitan area. Karst and major carbonaceous rock areas are limited to portions of Palo Pinto, Erath, Hood, Johnson, and Somerveil counties. Special geotechnical testing requirements may be needed in karst areas. The local reviewing authority should be consulted to determine if a project is subject to terrain constraints.

- Low relief areas need special consideration because many structural controls require a hydraulic head to move stormwater runoff through the facility.
- High relief may limit the use of some structural controls that need flat or gently sloping areas to settle out sediment or to reduce velocities. In other cases, high relief may impact dam heights to the point that a structural control becomes infeasible.
- Karst terrain can limit the use of some structural controls as the infiltration of polluted waters directly into underground streams found in karst areas may be prohibited. In addition, ponding areas may not reliably hold water in karst areas.

Soils

- The key evaluation factors are based on an initial investigation of the NRCS hydrologic soils groups at the site. Note that more detailed geotechnical tests are usually required for infiltration feasibility and during design to confirm permeability and other factors.

Special Watershed or Stream Considerations

- The design of structural stormwater controls is fundamentally influenced by the nature of the downstream water body that will be receiving the stormwater discharge. In addition, the designer should consult with the appropriate review authority to determine if their development project is subject to additional structural control criteria as a result of an adopted local watershed plan or special provision.

Table 1.3 Structural Control Screening Matrix

Category	On-Site Storm Water Controls	STORM WATER TREATMENT SUITABILITY				WATER QUALITY PERFORMANCE				SITE APPLICABILITY					IMPLEMENTATION CONSIDERATIONS			
		Water Quality Protection	Streambank Protection	On-Site Flood Control	Downstream Flood Control	TSS/ Sediment Removal Rate	Nutrient Removal Rate (TP/TN)	Bacteria Removal Rate	Hotspot Application	Drainage Area (acres)	Space Req'd (% of tributary imp. Area)	Site Slope	Minimum Head Required	Depth to Water Table	Residential Subdivision Use	High Density/Ultra Urban	Capital Cost	Maintenance Burden
Bioretention Areas	Bioretention Areas	**P**	**S**	**S**	-	**80%**	**60%/50%**	-	✓	5 max***	5-7%	6% max	5 ft	2 feet	✓	✓	Moderate	Low
Channels	Enhanced Swales	**P**	**S**	**S**	**S**	**80%**	**25%/40%**	-	✓	5 max	10-20%	4% max	1 ft	below WT	✓		High	Low
	Channels, Grass	**S**	**S**	**P**	**S**	**50%**	**25%/20%**	-							✓		Low	Moderate
	Channels, Open	-	-	**P**	**S**	-	-	-							✓		Low	Low
Chemical Treatment	Alum Treatment System	**P**	-	-	-	**90%**	**80%/60%**	**90%**	✓	25 min	None				✓	✓	High	High
Conveyance Components	Culverts	-	-	**P**	**P**	-	-	-							✓	✓	Low	Low
	Energy Dissipation	-	**P**	**S**	**S**	-	-	-							✓	✓	Low	Low
	Inlets/Street Gutters	-	-	**P**	-	-	-	-							✓	✓	Low	Low
	Pipe Systems	-	**P**	**P**	**P**	-	-	-							✓	✓	Low	Low
Detention	Detention, Dry	**S**	**P**	**P**	**P**	**65%**	**50%/30%**	**70%**	✓		2 - 3%	15% across pond	6 to 8 ft	2 feet	✓		Low	Moderate to High
	Detention, Extended Dry	**S**	**P**	**P**	**P**	**65%**	**50%/30%**	**70%**	✓		2 - 3%	15% across pond	6 to 8 ft	2 feet	✓		Low	Moderate to High
	Detention, Multi-purpose Areas	-	**P**	**P**	**P**	-	-	-		200 max		1% for Parking Lot; 0.25 in/ft for Rooftop			✓	✓	Low	Low
	Detention, Underground	-	**P**	**P**	**P**	-	-	-		200 max						✓	High	Moderate
Filtration	Filter Strips	**S**	-	-	-	**50%**	**20%/20%**	-		2 max***	20-25%	2-6%			✓		Low	Moderate
	Organic Filters	**P**	-	-	-	**80%**	**60%/40%**	**50%**	✓	10 max***	2-3%		5 to 8 ft			✓	High	High
	Planter Boxes	**P**	-	-	-	**80%**	**60%/40%**	-			6%					✓	Low	Moderate
	Sand Filters, Surface/ Perimeter	**P**	**S**	-	-	**80%**	**50%/25%**	**40%**	✓	10 max***/ 2 max***	2-3%	6% max	5 ft/ 2 to 3 ft	2 feet		✓	High	High
	Sand Filters, Underground	**P**	-	-	-	**80%**	**50%/25%**	**40%**	✓	5 max	None					✓	High	High
Hydrodynamic Devices	Gravity (Oil-Grit) Separator	**S**	-	-	-	**40%**	**5%/5%**	-		1 max***	None					✓	High	High
Infiltration	Downspout Drywell	**P**	-	-	-	**80%**	**60%/60%**	**90%**							✓	✓	Low	Moderate
	Infiltration Trenches	**P**	**S**	-	-	**80%**	**60%/60%**	**90%**		5 max	2-3%	6% max	1 ft	4 feet	✓	✓	High	High
	Soakage Trenches	**P**	**S**	-	-	**80%**	**60%/60%**	**90%**		5 max	27' per 1000 ft^2 impervious area	6% max	1 ft	4 feet	✓	✓	High	High
Ponds	Wet Pond	**P**	**P**	**P**	**P**	**80%**	**50%/30%**	**70%***	✓		2-3%	15% max	6 to 8 ft	2 feet, if hotspot or aquifer	✓		Low	Low
	Wet ED Pond	**P**	**P**	**P**	**P**	**80%**	**50%/30%**	**70%***	✓	25 min**					✓		Low	Low
	Micropool ED Pond	**P**	**P**	**P**	**P**	**80%**	**50%/30%**	**70%***	✓	10 min**					✓		Low	Moderate
	Multiple ponds	**P**	**P**	**P**	**P**	**80%**	**50%/30%**	**70%***	✓	25 min**					✓		Low	Low
Porous Surfaces	Green Roof	**P**	**S**	-	-	**85%**	**95%/16%**	-	✓							✓	High	High
	Modular Porous Paver Systems	**S**	**S**	-	-	**	**80%/80%**	-		5 max	Varies					✓	Moderate	High
	Porous Concrete	**S**	**S**	-	-	**	**50%/65%**	-		5 max	Varies					✓	High	High
Proprietary Systems	Proprietary Systems ****	**S**	**S**	**S**	**S**	****	****	****		****	****				****	✓	High	High
Re-Use	Rain Harvesting	**P**	-	-	-	-	-	-							✓	✓	Low	High
Wetlands	Wetlands, Storm Water	**P**	**P**	**P**	**P**	**80%**	**40%/30%**	**70%***	✓	25 min	3-5%	8% max	3 to 5 ft (shallow) 6 to 8 ft (pond)	2 feet, if hotspot or aquifer	✓		Moderate	Moderate
	Wetlands, Submerged Gravel	**P**	**P**	**S**	-	**80%**	**50%/20%**	**70%**	✓	5 min			2 to 3 ft	below WT	✓	✓	Moderate	High

Table 1.3 Structural Control Screening Matrix

✓ **-** Meets suitability criteria

P - Primary Control, meets suitability criteria

S - Secondary Control, can be incorporated into the structural control in certain situations

***** Provides less than 80% TSS removal efficiency. May be used in pretreatment and as part of a "treatment train"

****** Smaller area acceptable with adequate water balance and anti-clogging device

******* Drainage area can be larger in some instances

******** The application and performance of specific commercial devices and systems must be provided by the manufacturer and should be verified by independent third-party sources and data

1 Porous surfaces provide water quantity benefits by reducing the effective impervious area

2 Due to the potential for clogging, porous surfaces should not be used for the removal of sediment or other coarse particulate pollutants

Category	On-Site Storm Water Controls	PHYSIOGRAPHIC FACTORS			Soils	SPECIAL WATERSHED CONSIDERATIONS		
		Low Relief	High Relief	Karst		High Quality Stream	Aquifer Protection	Reservior Protection
Bioretention Areas	Bioretention Areas	Several design variations will likely be limited by low head		Use poly-liner or impermeable membrane to seal bottom	Clay or silty soils may require pretreatment	Evaluate for stream warming	Needs to be designed with no exfiltration (i.e. outflow to groundwater)	
Channels	Enhanced Swales	Generally feasible however slope <1% may lead to standing water in dry swales	Often infeasible if slopes are 4% or greater				Hotspot runoff must be adequately treated	Hotspot runoff must be adequately treated
	Channels, Grass							
	Channels, Open							
Chemical Treatment	Alum Treatment System							
Conveyance Components	Culverts							
	Energy Dissipation							
	Inlets/Street Gutters							
	Pipe Systems							
Detention	Detention, Dry		Embankment heights restricted	Require poly or clay liner, Max ponding depth, Geotechnical tests	Underlying soils of hydrologic group “C” or “D” should be adequate to maintain a permanent pool. Most group “A” soils and some group “B” soils will require a pond liner.			
	Detention, Extended Dry							
	Detention, Multi-purpose Areas							
	Detention, Underground			GENERALLY NOT ALLOWED				
Filtration	Filter Strips							
	Organic Filters							
	Planter Boxes				Type A or B			
	Sand Filters, Surface/ Perimeter	Several design variations will likely be limited by low head		Use poly-liner or impermeable membrane to seal bottom	Clay or silty soils may require pretreatment	Evaluate for stream warming	Needs to be designed with no exfiltration (i.e. outflow to groundwater)	
	Sand Filters, Underground							
Hydrodynamic Devices	Gravity (Oil-Grit) Separator							
Infiltration	Downspout Drywell	Minimum distance to water table of 4 feet		GENERALLY NOT ALLOWED	Infiltration rate > 0.5 inch/hr			
	Infiltration Trenches	Minimum distance to water table of 2 feet	Maximum slope of 6% Trenches must have flat bottom	GENERALLY NOT ALLOWED	Infiltration rate > 0.5 inch/hr		Maintain safe distance from wells and water table. No hotspot runoff	Maintain safe distance from bedrock and water table. Pretreat runoff
	Soakage Trenches	Minimum distance to water table of 4 feet	Maximum slope of 6% Trenches must have flat bottom	GENERALLY NOT ALLOWED	Infiltration rate > 0.5 inch/hr			
Ponds	Wet Pond	Limit maximum normal pool depth to about 4 feet (dugout) Providing pond drain can be problematic	Embankment heights restricted	Require poly or clay liner Max ponding depth Geotechnical tests	“A” soils may require pond liner “B” soils may require infiltration testing	Evaluate for stream warming	May require liner if “A” soils are present Pretreat hotspots 2 to 4 ft separation distance from water table	
	Wet ED Pond							
	Micropool ED Pond							
	Multiple ponds							
Porous Surfaces	Green Roof							
	Modular Porous Paver Systems							
	Porous Concrete							
Proprietary Systems	Proprietary Systems *							
Re-Use	Rain Harvesting							
Wetlands	Wetlands, Storm Water		Embankment heights restricted	Require poly-liner Geotechnical tests	“A” soils may require pond liner	Evaluate for stream warming	May require liner if “A” soils are present Pretreat hotspots 2 to 4 ft separation distance from water table	
	Wetlands, Submerged Gravel							

✓ **-** Meets suitability criteria

P - Primary Control, meets suitability criteria

S - Secondary Control, can be incorporated into the structural control in certain situations

***** Provides less than 80% TSS removal efficiency. May be used in pretreatment and as part of a "treatment train"

****** Smaller area acceptable with adequate water balance and anti-clogging device

******* Drainage area can be larger in some instances

******** The application and performance of specific commercial devices and systems must be provided by the manufacturer and should be verified by independent third-party sources and data

1 Porous surfaces provide water quantity benefits by reducing the effective impervious area

2 Due to the potential for clogging, porous surfaces should not be used for the removal of sediment or other coarse particulate pollutants

In some cases, higher pollutant removal or environmental performance is needed to fully protect aquatic resources and/or human health and safety within a particular watershed or receiving water. Therefore, special design criteria for a particular structural control or the exclusion of one or more controls may need to be considered within these watersheds or areas. Examples of important watershed factors to consider include:

High Quality Streams (Streams with a watershed impervious cover less than approximately 15%). These streams may also possess high quality cool water or warm water aquatic resources or endangered species. The design objectives are to maintain habitat quality through the same techniques used for cold-water streams, with the exception that stream warming is not as severe of a design constraint. These streams may also be specially designated by local authorities.

Wellhead Protection. Areas that recharge existing public water supply wells present a unique management challenge. The key design constraint is to prevent possible groundwater contamination by preventing infiltration of hotspot runoff. At the same time, recharge of unpolluted stormwater is encouraged to maintain flow in streams and wells during dry weather.

Reservoir or Drinking Water Protection. Watersheds that deliver surface runoff to a public water supply reservoir or impoundment are a special concern. Depending on the treatment available, it may be necessary to achieve a greater level of pollutant removal for the pollutants of concern, such as bacteria pathogens, nutrients, sediment, or metals. One particular management concern for reservoirs is ensuring stormwater hotspots are adequately treated so they do not contaminate drinking water.

Step 3 Location and Permitting Considerations

In the last step, a site designer assesses the physical and environmental features at the site to determine the optimal location for the selected structural control or group of controls. The checklist below (Table 1.4) provides a condensed summary of current restrictions as they relate to common site features that may be regulated under local, state, or federal law. These restrictions fall into one of three general categories:

- Locating a structural control within an area when expressly prohibited by law.
- Locating a structural control within an area that is strongly discouraged, and is only allowed on a case by case basis. Local, state, and/or federal permits shall be obtained, and the applicant will need to supply additional documentation to justify locating the stormwater control within the regulated area.
- Structural stormwater controls must be setback a fixed distance from a site feature.

This checklist is only intended as a general guide to location and permitting requirements as they relate to siting of stormwater structural controls. Consultation with the appropriate regulatory agency is the best strategy.

Table 1.4 Location and Permitting Checklist

Site Feature	Location and Permitting Guidance
Jurisdictional Wetland (Waters of the U.S) U.S. Army Corps of Engineers Regulattory Permit	• Jurisdictional wetlands should be delineated prior to siting structural control. • Use of natural wetlands for stormwater quality treatment is contrary to the goals of the Clean Water Act and should be avoided. • Stormwater should be treated prior to discharge into a natural wetland. • Structural controls may also be *restricted* in local buffer zones. Buffer zones may be utilized as a non-structural filter strip (i.e., accept sheet flow). • Should justify that no practical upland treatment alternatives exist. • Where practical, excess stormwater flows should be conveyed away from jurisdictional wetlands.
Stream Channel (Waters of the U.S) U.S. Army Corps of Engineers Section 404 Permit	• All Waters of the U.S. (streams, ponds, lakes, etc.) should be delineated prior to design. • Use of any Waters of the U.S. for stormwater quality treatment is contrary to the goals of the Clean Water Act and should be avoided. • Stormwater should be treated prior to discharge into Waters of the U.S. • In-stream ponds for stormwater quality treatment are highly discouraged. • Must justify that no practical upland treatment alternatives exist. • Temporary runoff storage preferred over permanent pools. • Implement measures that reduce downstream warming.
Texas Commission on Environmental Quality Groundwater Management Areas	• Conserve, preserve, protect, recharge, and prevent waste of groundwater resources through Groundwater Conservation Districts • Groundwater Conservation District pending for Middle Trinity. • Detailed mapping available from Texas Alliance of Groundwater Districts.
Texas Commission on Environmental Quality Surface Water Quality Standards	• Specific stream and reservoir buffer requirements. • May be imperviousness limitations • May be specific structural control requirements. • TCEQ provides water quality certification – in conjunction with 404 permit • Mitigation will be required for imparts to existing aquatic and terrestrial habitat.
100 Year Floodplain Local Stormwater review Authority	• Grading and fill for structural control construction is generally discouraged within the 100 year floodplain, as delineated by FEMA flood insurance rate maps, FEMA flood boundary and floodway maps, or more stringent local floodplain maps. • Floodplain fill cannot raise the floodplain water surface elevation by more than limits set by the appropriate jurisdiction.

Table 1.4 Location and Permitting Checklist	
Site Feature	**Location and Permitting Guidance**
Stream Buffer Check with appropriate review authority whether stream buffers are required	• Consult local authority for stormwater policy. • Structural controls are discouraged in the streamside zone (within 25 feet or more of streambank, depending on the specific regulations).
Utilities Local Review Authority	• Call appropriate agency to locate existing utilities prior to design. • Note the location of proposed utilities to serve development. • Structural controls are discouraged within utility easements or rights of way for public or private utilities.
Roads TxDOT or DPW	• Consult TxDOT for any setback requirement from local roads. • Consult DOT for setbacks from State maintained roads. • Approval must also be obtained for any stormwater discharges to a local or state-owned conveyance channel.
Structures Local Review Authority	• Consult local review authority for structural control setbacks from structures. • Recommended setbacks for each structural control group are provided in the performance criteria in this manual.
Septic Drain fields Local Health Authority	• Consult local health authority. • Recommended setback is a minimum of 50 feet from drain field edge or spray area.
Water Wells Local Health Authority	• 100-foot setback for stormwater infiltration. • 50-foot setback for all other structural controls.

1.3.2 Example Application

A 20-acre institutional area (e.g., church and associated buildings) is being constructed in a dense urban area within Dallas/Fort Worth metropolitan area. The impervious coverage of the site is 40%. The site drains to an urban stream that is highly impacted from hydrologic alterations (accelerated channel erosion). The stream channel is deeply incised; consequently, flooding is not a problem. The channel drains to an urban river that is tributary to a phosphorus limited drinking water reservoir. Low permeability soils limit infiltration practices.

Objective: Avoid additional disruptions to receiving channel and reduce pollutant loads for sediment and phosphorus to receiving waters.

Target Removals: Provide stormwater management to mitigate for accelerated channel incision and reduce loadings of key pollutants by the following:

- Sediment: 70% to 80%
- Phosphorus: 40%

Activity/Runoff Characteristics: The proposed site is to have large areas of impervious surface in the form of parking and structures. However, there will be a large contiguous portion of turf grass proposed for the front of the parcel that will have a relatively steep slope (approximately 10%) and will drain to the storm drain system associated with the entrance drive. Stormwater runoff from the site is expected to exhibit fairly high sediment levels and seasonally high phosphorus levels (due to turf grass management).

Table 1.5 lists the results of the selection analysis using the screening matrix described previously.

The highlighted rows indicate the controls selected for this example. The **X**'s indicate inadequacies in the control for this site. The ✓'s indicate adequate control capabilities for this site.

While there is a downstream reservoir to consider, there are no special watershed factors or physiographic factors to preclude the use of any of the practices from the structural control list. However, due to the size of the drainage area, most stormwater ponds and wetlands are removed from consideration. In addition, the site's impermeable soils remove an infiltration trench from being considered. Due to the need to provide flood control as well as streambank protection storage, an extended detention micropool pond will likely be needed, unless some downstream regional storage is available to control flood waters.

To provide additional pollutant removal capabilities in an attempt to better meet the target removals, bioretention, surface sand filters, and/or perimeter sand filters can be used to treat the parking lot and driveway runoff. The bioretention will provide some removal of phosphorus while improving the aesthetics of the site. Surface sand filters provide higher phosphorus removal at a comparable unit cost to bioretention, but are not as aesthetically pleasing. The perimeter sand filter, is a flexible, easy to access practice (but at higher cost) that provides good phosphorus removal and additionally high oil and grease trapping ability.

The site drainage system can be designed so the bioretention and/or sand filters drain to the extended detention micropool pond for redundant treatment. Vegetated dry swales could also be used to convey runoff to the pond, which would provide pretreatment. Pocket wetlands and wet swales were eliminated from consideration due to potential for nuisance conditions. Underground sand filters could also be used at the site; however, cost and aesthetic considerations were significant enough to eliminate from consideration.

Table 1.5 Sample Structural Control Selection Matrix

Structural Control Alternative	Stormwater Treatment Suitability	Site Applicability	Implementation Considerations	Physiographic Factors/Soils	Special Watershed Considerations	Other Issues
Bioretention	✓[1]	✓[2]	✓	✓	none	
Dry Swale	✓[1]	✓[2]	✓	✓	none	
Wet Swale	✓[1]	✓[2]	✓	✓	none	Odor / mosquitoes
Perimeter Sand Filter	✓[1]	✓[2]	✓	✓	none	Higher cost
Surface Sand Filter	✓[1]	✓[2]	✓	✓	none	Aesthetics
Infiltration Trench	✓[1]	✓	✓	**X**		
Extended Detention Micropool Pond	✓	✓	✓	✓	none	
Multiple Ponds	✓	**X**				
Wet Extended Detention Pond	✓	**X**				
Wet Pond	✓	**X**				
Extended Detention Shallow Wetland	✓	**X**				
Pocket Wetland	✓	✓	✓	✓	none	Odor / mosquitoes

Table 1.5 Sample Structural Control Selection Matrix

Structural Control Alternative	Stormwater Treatment Suitability	Site Applicability	Implementation Considerations	Physiographic Factors/Soils	Special Watershed Considerations	Other Issues
Shallow Wetland	✓	X				

Notes:
1. Only when used with another structural control that provides water quantity control
2. Can treat a portion of the site

1.4 On-Line Versus Off-Line Structural Controls

1.4.1 Introduction

Structural stormwater controls are designed to be either "on-line" or "off-line." On-line facilities are designed to receive, but not necessarily control or treat, the entire runoff volume above the Q_f up to the peak flood mitigation storm discharge (Q_{p100}). On-line structural controls must be able to handle the entire range of storm flows.

Off-line facilities on the other hand are designed to receive only a specified flow rate or volume through the use of a flow regulator (i.e. diversion structure, flow splitter, etc). Flow regulators are typically used to divert the water quality volume (WQ_v) to an off-line structural control sized and designed to treat and control the WQ_v. After the design runoff flow has been treated and/or controlled, it is returned to the conveyance system. Figure 1.1 shows an example of an off-line sand filter and an on-line enhanced dry swale.

1.4.2 Flow Regulators

Flow regulation to off-line structural stormwater controls can be achieved by either:

- Diverting the water quality volume or other specific maximum flow rate to an off-line structural stormwater control, or
- Bypassing flows in excess of the design flow rate

The peak water quality flow rate (Q_{wq}) can be calculated using the procedure found in *Section 1.4 of the Water Quality Technical Manual.*

Flow regulators can be flow splitter devices, diversion structures, or overflow structures. A number of examples are shown in Figures 1.2 through 1.4.

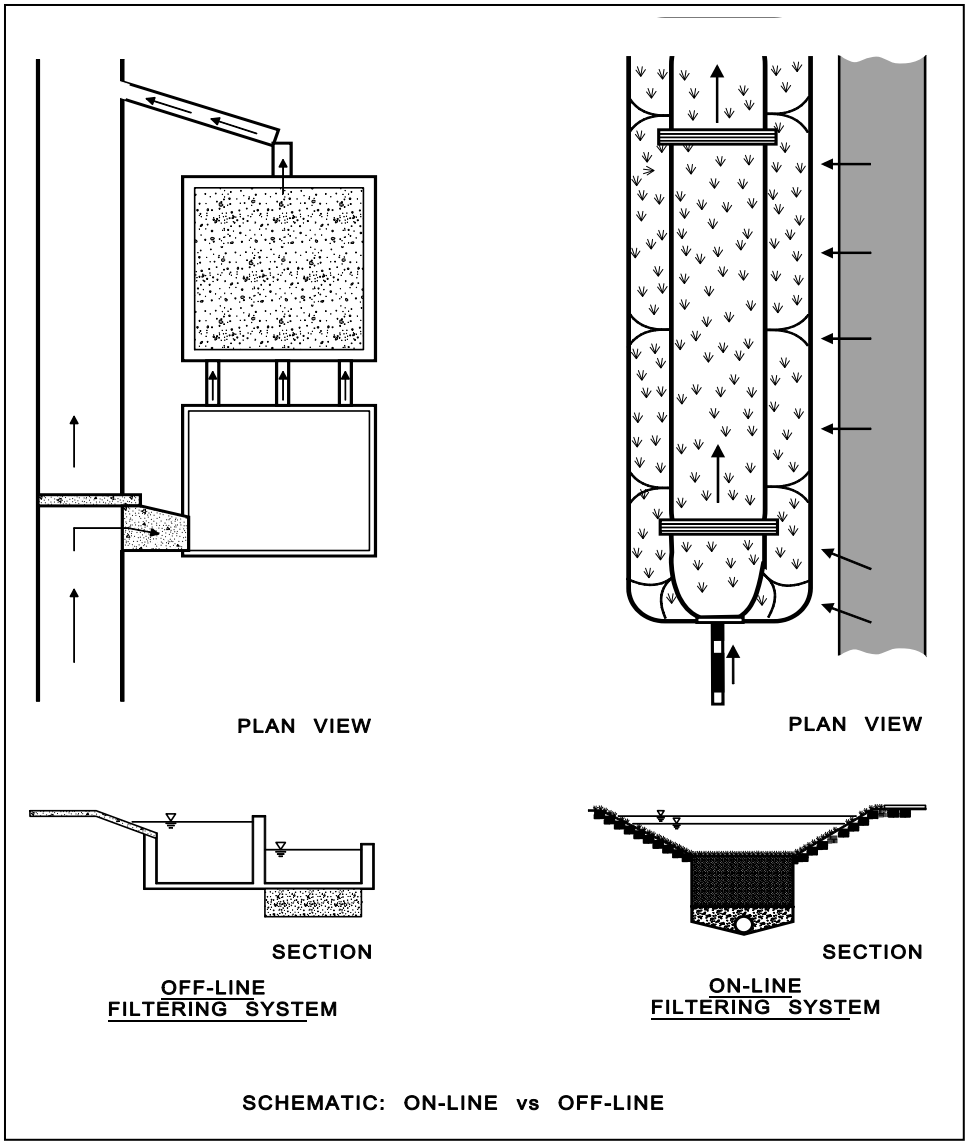


Figure 1.1 Example of On-Line versus Off-Line Structural Controls
(Source: CWP, 1996)

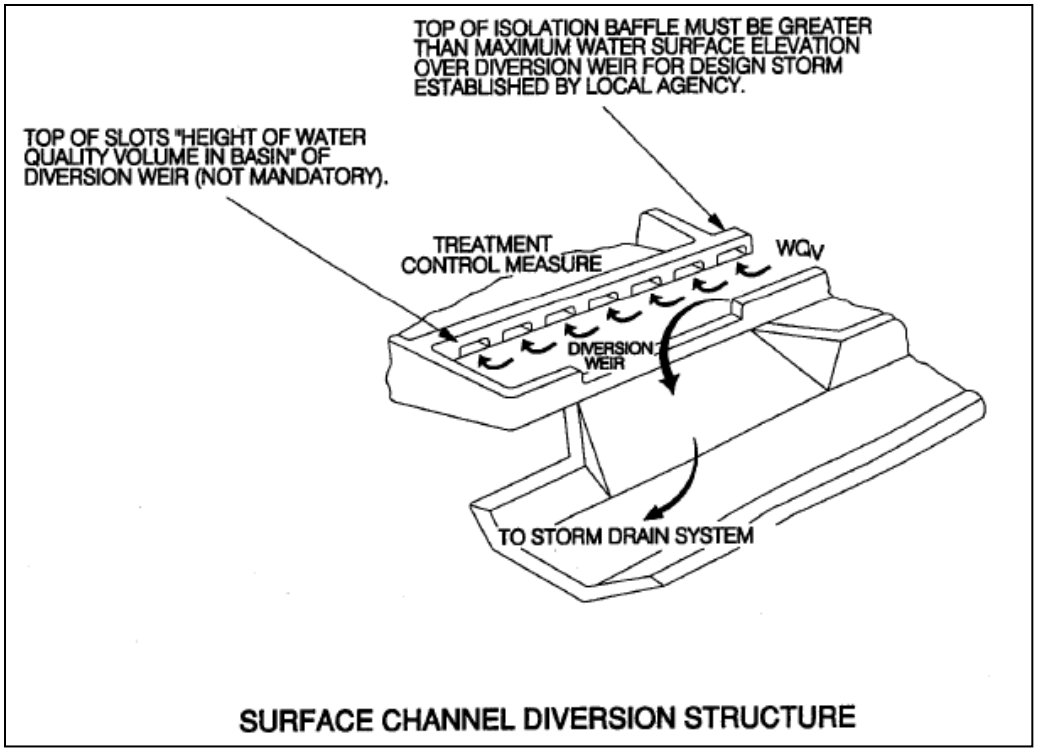


Figure 1.2 Pipe Interceptor Diversion Structure
(Source: City of Sacramento, 2000)

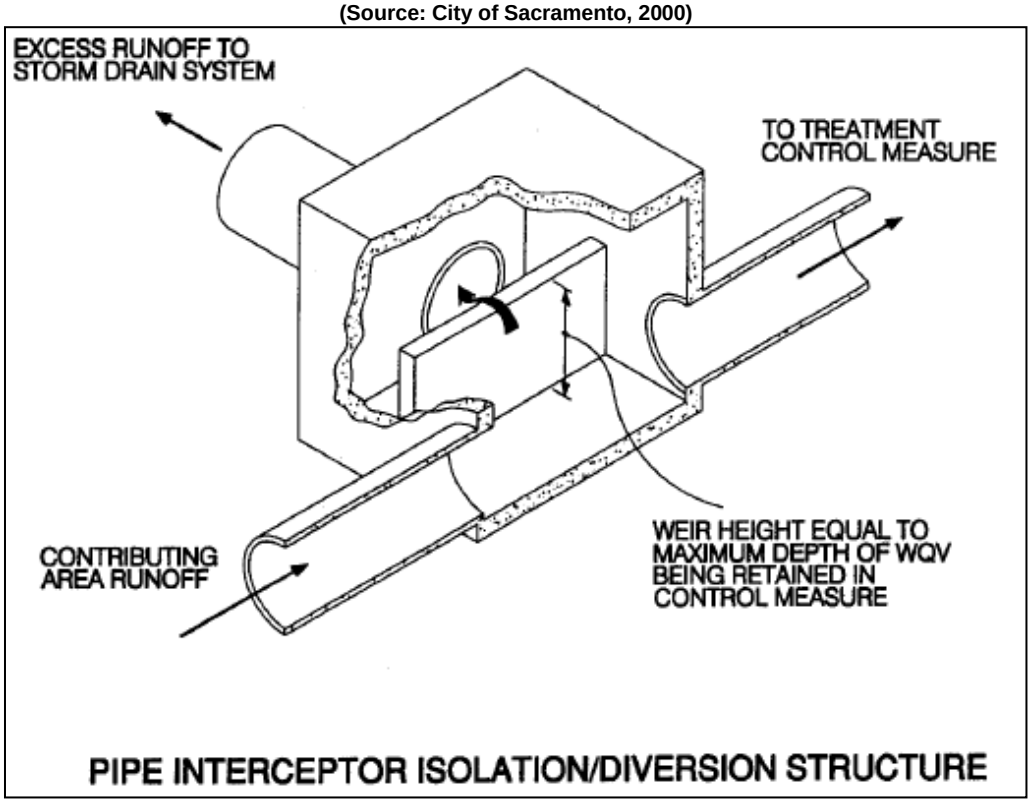


Figure 1.3 Surface Channel Diversion Structure
(Source: City of Sacramento, 2000)

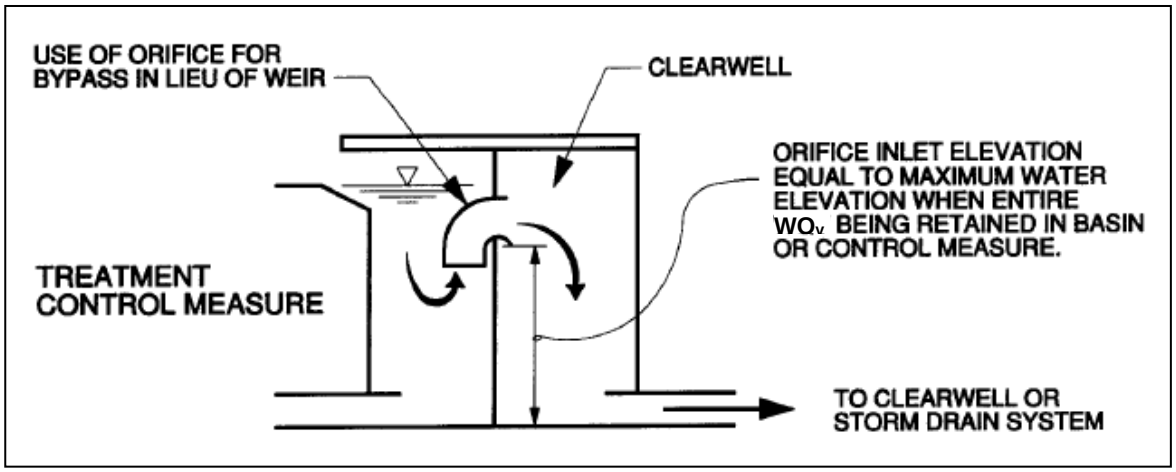


Figure 1.4 Outlet Flow Regulator
(Source: City of Sacramento, 2000)

1.5 Regional Versus On-Site Stormwater Management

1.5.1 Introduction

Using individual, on-site structural stormwater controls for each development is the typical approach for controlling stormwater quantity and quality. The developer finances the design and construction of these controls and, initially, is responsible for all operation and maintenance.

A potential alternative approach is for a community to install a few strategically located regional stormwater controls in a subwatershed rather than require on-site controls (see Figure 1.5). For this Manual, regional stormwater controls are defined as facilities designed to manage stormwater runoff from multiple projects and/or properties through a local jurisdiction-sponsored program, where the individual properties may assist in the financing of the facility, and the requirement for on-site controls is either eliminated or reduced.

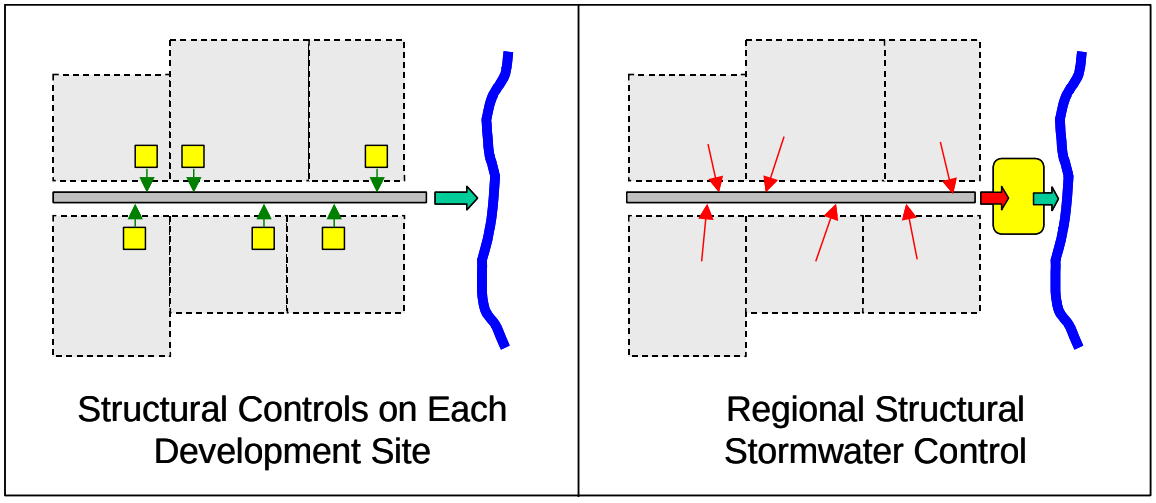


Figure 1.5 On-Site versus Regional Stormwater Management

1.5.2 Advantages and Disadvantages of Regional Stormwater Controls

Regional stormwater facilities are significantly more cost-effective because it is easier and less expensive to build, operate, and maintain one large facility than several small ones. Regional stormwater controls are generally better maintained than individual site controls because they are large, highly visible, and typically the responsibility of the local government. In addition, a larger facility poses less of a safety hazard than numerous small ones because it is more visible and is easier to secure.

There are also several disadvantages to regional stormwater controls. In many cases, a community must provide capital construction funds for a regional facility, including the costs of land acquisition. However, if a downstream developer is the first to build, that person could be required to construct the facility and later be compensated by upstream developers for the capital construction costs and annual maintenance expenditures. Conversely, an upstream developer may have to establish temporary control structures if the regional facility is not in place before construction. Maintenance responsibilities generally shift from the homeowner or developer to the local government when a regional approach is selected. The local government would need to establish a stormwater utility or some other program to fund and implement stormwater control. Finally, a large in-stream facility can pose a greater disruption to the natural flow network and is more likely to affect wetlands within the watershed.

Below are summarized some of the "pros" and "cons" of regional stormwater controls.

Advantages of Regional Stormwater Controls

- **Reduced Construction Costs** – Design and construction of a single regional stormwater control facility can be far more cost-effective than numerous individual on-site structural controls.
- **Reduced Operation and Maintenance Costs** – Rather than multiple owners and associations being responsible for the maintenance of several stormwater facilities on their developments, it is simpler and more cost effective to establish scheduled maintenance of a single regional facility.
- **Higher Assurance of Maintenance** – Regional stormwater facilities are far more likely to be adequately maintained as they are large and have a higher visibility, and are typically the responsibility of the local government.
- **Maximum Utilization of Developable Land** – Developers would be able to maximize the utilization of the proposed development for the purpose intended by minimizing the land normally set aside for the construction of stormwater structural controls.
- **Retrofit Potential** – Regional facilities can be used by a community to mitigate existing developed areas that have insufficient or no structural controls for water quality and/or quantity, as well as provide for future development.
- **Other Benefits** – Well-sited regional stormwater facilities can serve as a recreational and aesthetic amenity for a community.

Disadvantages of Regional Stormwater Controls

- **Location and Siting** – Regional stormwater facilities may be difficult to site, particularly for large facilities or in areas with existing development.
- **Capital Costs** – The community must typically provide capital construction funds for a regional facility, including the costs of land acquisition.
- **Maintenance** – The local government is typically responsible for the operation and maintenance of a regional stormwater facility.
- **Need for Planning** – The implementation of regional stormwater controls requires substantial planning, financing, and permitting. Land acquisition must be in place ahead of future projected growth.

For in-stream regional facilities:

- **Water Quality and Streambank Protection** – Without on-site water quality and streambank protection, regional controls do not protect smaller streams upstream from the facility from degradation and streambank erosion.
- **Ponding Impacts** – Upstream inundation from a regional facility impoundment can eliminate floodplains, wetlands, and other habitat.

1.5.3 Important Considerations for the Use of Regional Stormwater Controls

If a community decides to implement a regional stormwater control, then it must ensure that the conveyances between the individual upstream developments and the regional facility can handle the design peak flows and volumes without causing adverse impact or property damage. Fully developed conditions in the regional facility drainage area should be used in the analysis.

Furthermore, unless the system consists of completely man-made conveyances (i.e. storm drains, pipes, concrete channels, etc) the on-site structural controls for water quality and downstream streambank protection will likely be required for all developments within the regional facility's drainage area. Federal water quality provisions do not allow the degradation of water bodies from untreated stormwater discharges, and it is U.S. EPA policy to not allow regional stormwater controls that would degrade stream quality between the upstream development and the regional facility. Further, without adequate streambank protection, aquatic habitats and water quality in the channel network upstream of a regional facility may be degraded by streambank erosion if they are not protected from bankfull flows and high velocities.

Based on these concerns, both the EPA and the U.S. Army Corps of Engineers have expressed opposition to *in-stream* regional stormwater control facilities. In-stream facilities should be avoided if possible and will likely be permitted on a case-by-case basis only.

It is important to note that siting and designing regional facilities should ideally be done within a context of stormwater master planning or watershed planning to be effective.

1.6 Using Structural Stormwater Controls in Series

1.6.1 Stormwater Treatment Trains

The minimum stormwater management standards are an integrated planning and design focus areas whose components work together to limit the adverse impacts of urban development on downstream waters and riparian areas. This approach is sometimes called a stormwater "treatment train." When considered comprehensively, a treatment train consists of all the design concepts and nonstructural and structural controls that work to attain water quality and quantity goals. This is illustrated in Figure 1.6.



Figure 1.6 Generalized Stormwater Treatment Train

Runoff and Load Generation – The initial part of the "train" is located at the source of runoff and pollutant load generation, and consists of *integrated* site design and pollution prevention practices that reduce runoff and stormwater pollutants from the source.

Pretreatment – The next step in the treatment train consists of pretreatment measures. These measures typically do not provide sufficient pollutant removal to meet the Primary TSS reduction goal, but do provide

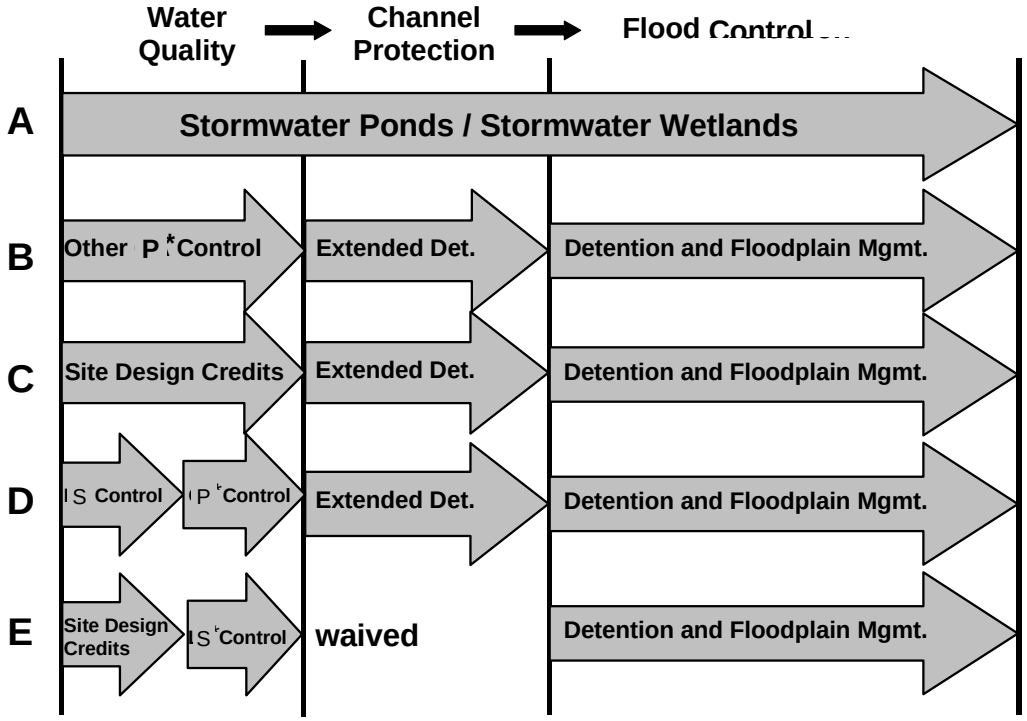
calculable water quality benefits that may be applied towards meeting the WQ_v treatment requirement. These measures include:

- The use of stormwater *integrated* site design practices and site design credits to reduce the water quality volume (WQ_v)
- Structural controls that achieve less than the Primary TSS removal rate, but provide pretreatment
- Pretreatment facilities such as sediment forebays

Primary Treatment and/or Quantity Control – The last step is primary water quality treatment and/or quantity (streambank protection and/or flood control) control. This is achieved through the use of either a structural control to achieve both water quality and quantity benefits or a structural control to achieve water quality benefits only.

1.6.2 Use of Multiple Structural Controls in Series

Many combinations of structural controls in series may exist for a site. Figure 1.7 provides a number of hypothetical examples of how the *integrated* Design Focus Areas may be addressed by using structural stormwater controls.



***P - Primary Control and S - Secondary Control Limited Application.**

Figure 1.7 Examples of Structural Controls Used in Series

Referring to Figure 1.7 by line letter:

A. Two structural controls achieving Primary TSS removal each, *stormwater ponds* and *stormwater wetlands*, can be used to meet all of the requirements of the *integrated* Design Focus Areas in a single facility.

B. The other structural controls achieving Primary TSS removal each (*bioretention, sand filters, infiltration trench and enhanced swale*) are typically used in combination with detention controls to meet the *integrated* Design Focus Areas. The detention facilities are located downstream from the water quality controls either on-site or combined into a regional or neighborhood facility.

C. Line C indicates the condition where an environmentally sensitive large lot subdivision has been developed that can be designed so as to waive the water quality treatment requirement altogether. However, detention controls may still be required for downstream streambank protection and flood control.

D. Where a structural control does not meet the Primary TSS removal criteria, another downstream structural control must be added. For example, urban hotspot land may be fit or retrofit with devices adjacent to parking or service areas designed to remove petroleum hydrocarbons. These devices may also serve as pretreatment devices removing the coarser fraction of sediment. One or more downstream structural controls are then used to meet the full Primary TSS removal goal, and well as water quantity control.

E. In line E, site design credits have been employed to reduce partially the water quality volume requirement. In this case, for a smaller site, a well designed and tested structural control provides limited TSS removal while a dry detention pond handles the flooding criteria. For this location, direct discharge to a large stream and local downstream floodplain management practices have eliminated the need for streambank protection and flood control storage on site.

The combinations of structural stormwater controls are limited only by the need to employ measures of proven effectiveness and meet local regulatory and physical site requirements. Figures 1.8 through 1.10 illustrate the application of the treatment train concept for: a moderate density residential neighborhood, a small commercial site, and a large shopping mall site.

In Figure 1.8 rooftop runoff drains over grassed yards to backyard grass channels. Runoff from front yards and driveways reaches roadside grass channels. Finally, all stormwater flows to a extended detention micropool stormwater pond.

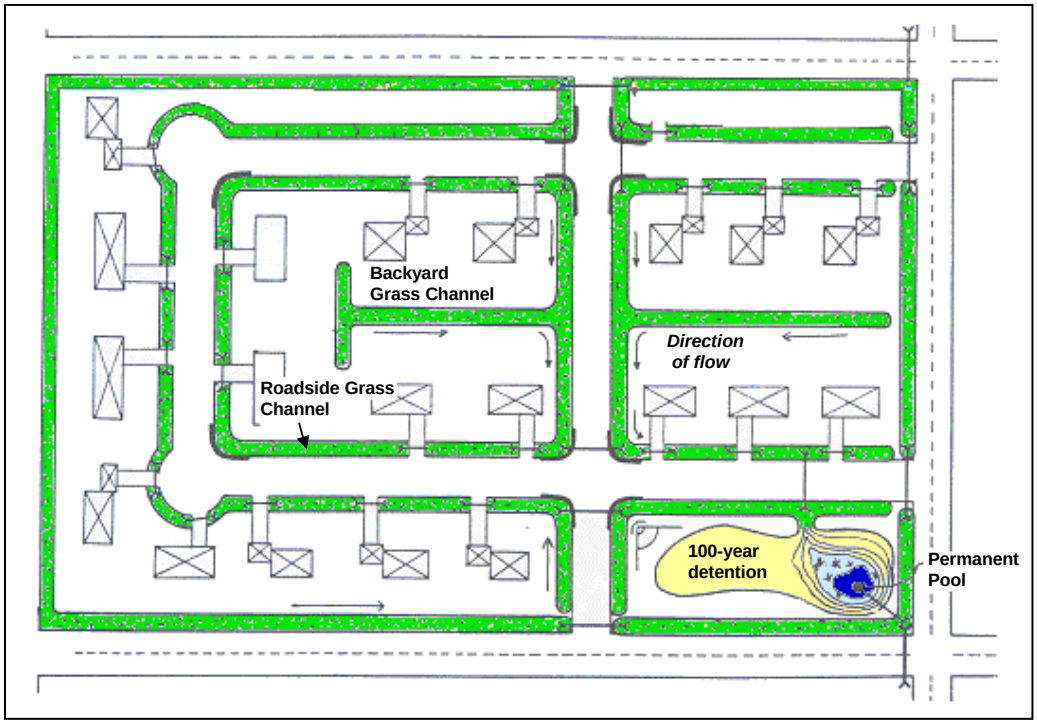


Figure 1.8 Example Treatment Train – Residential Subdivision
(Adapted from: NIPC, 2000)

A gas station and convenience store is depicted in Figure 1.9. In this case, the decision was made to intercept hydrocarbons and oils using a commercial gravity (oil-grit) separator located on the site prior to draining to perimeter sand filter for removal of finer particles and TSS. No stormwater control for streambank protection is required as the system drains to the municipal storm drain pipe system. Flood control is provided by a regional stormwater control downstream.

Figure 1.10 shows an example treatment train for a commercial shopping center. In this case, runoff from rooftops and parking lots drains to depressed parking lot islands, perimeter grass channels, and bioretention areas. Slotted curbs are used at the entrances to these swales to better distribute the flow and to settle out the very coarse particles at the parking lot edge for sweepers to remove. Runoff is then conveyed to an extended detention wet pond for additional pollutant removal and streambank protection. Flood control is provided through parking lot detention.

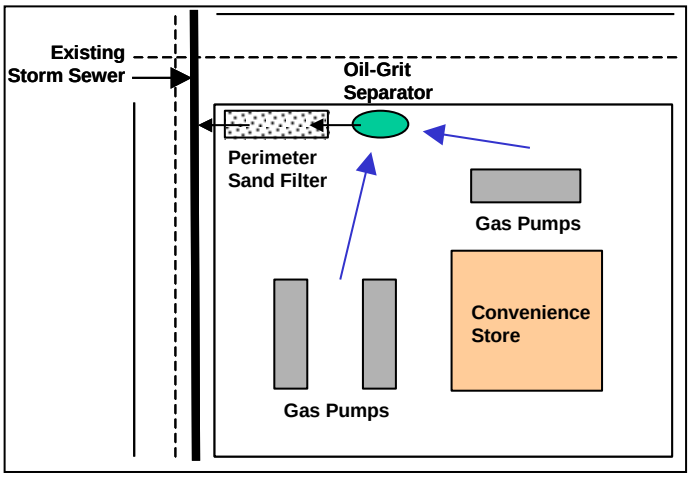


Figure 1.9 Example Treatment Train – Small Commercial Site

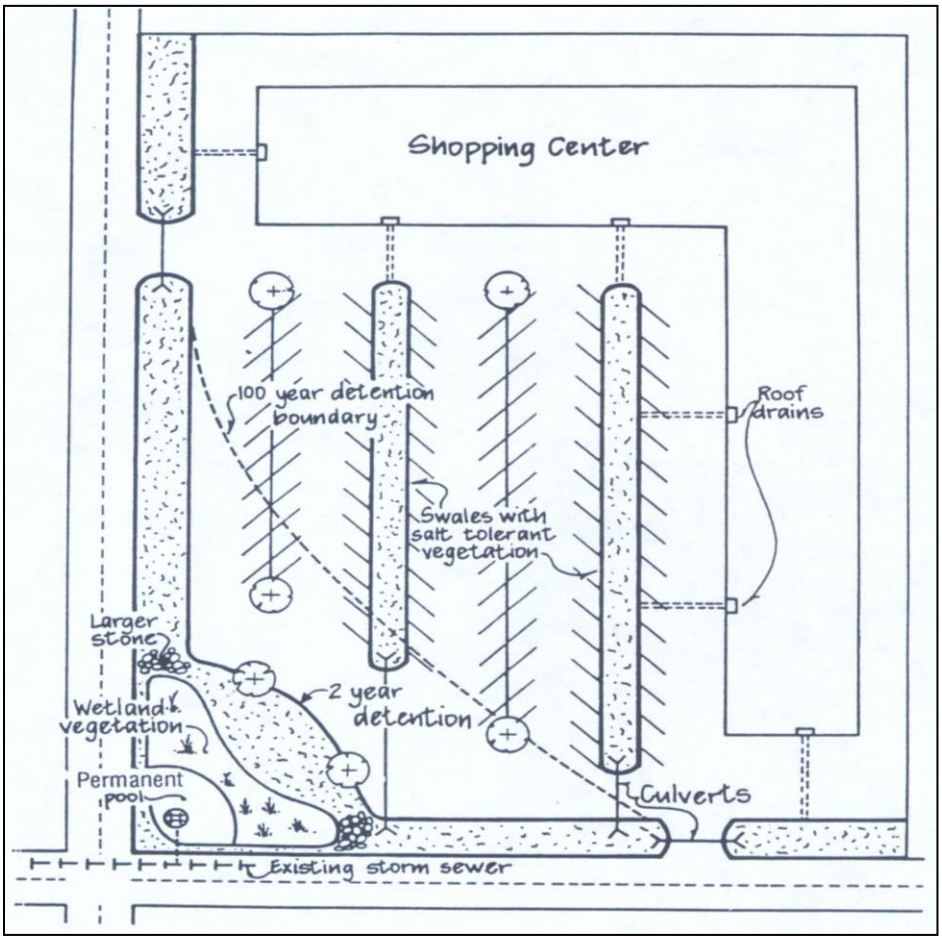


Figure 1.10 Example Treatment Train – Large Commercial Development
(Source: NIPC, 2000)

1.6.3 Calculation of Pollutant Removal for Structural Controls in Series

For two or more structural stormwater controls used in combination, it is often important to have an estimate of the pollutant removal efficiency of the treatment train. Pollutant removal rates for structural controls in series are not additive. For pollutants in particulate form, the actual removal rate (expressed in terms of percentage of pollution removed) varies directly with the pollution concentration and sediment size distribution of runoff entering a facility.

For example, a stormwater pond facility will have a much higher pollutant removal percentage for very turbid runoff than for clearer water. When two stormwater ponds are placed in series, the second pond will treat an incoming particulate pollutant load very different from the first pond. The upstream pond captures the easily removed larger sediment sizes, passing on an outflow with a lower concentration of TSS but with a higher proportion of finer particle sizes. Hence, the removal capability of the second pond for TSS is considerably less than the first pond. Recent findings suggest that the second pond in series can provide as little as half the removal efficiency of the upstream pond.

To estimate the pollutant removal rate of structural controls in series, a method is used in which the removal efficiency of a downstream structural control is reduced to account for the pollutant removal of the upstream control(s). The following equations are used to determine the pollutant removal:

For treatment trains with two BMPs the following equation is used.

$$\boldsymbol{E = A + B - \{(A * B)/100\}}$$

where:

E = total efficiency
A = efficiency of first or upstream BMP
B = efficiency of second BMP

For treatment trains with three BMPs the following equation is used.

$$\boldsymbol{E = 0.95 * [A_B + C - \{(A_B * C)/100\}]}$$

where:

E = total efficiency
A_B = A+B-{(A*B)/100}
A = efficiency of first or upstream BMP
B = efficiency of second BMP
C = efficiency of third or downstream BMP

Example

TSS is the pollutant of concern and a commercial device is inserted that has a 20% sediment removal rate. A stormwater pond is designed at the site outlet. A second stormwater pond is located downstream from the first one in series. What is the total TSS removal rate? The following information is given:

Control 1 (Commercial Device) = 20% TSS removal

Control 2 (Stormwater Pond 1) = 80% TSS removal

Control 3 (Stormwater Pond 2) = 80% TSS removal

Apply the equations to calculate the total efficienty:

$E = 0.95*[A_B+C - \{(A_B*C)/100\}]$

where $A_B = A+B - \{(A*B)/100\} = 20+80 – \{(20*80)/100\} = 84$

$E = 0.95*[84+80 - \{(84*80)/100\}] = 91.96$

For the treatment train in total = **91.96% removal**

1.6.4 Routing with WQv Removed

When off-line structural controls such as bioretention areas, sand filters and infiltration trenches capture and remove the water quality volume (WQ_v), downstream structural controls do not have to account for this volume during design. That is, the WQ_v may be subtracted from the total volume that would otherwise need to be routed through the downstream structural controls.

From a calculation standpoint this would amount to removing the initial WQ_v from the beginning of the runoff hydrograph – thus creating a “notch” in the runoff hydrograph. Since most commercially available hydrologic modeling packages cannot handle this type of action, the following method to adjust “CN” values has been created to facilitate removal from the runoff hydrograph of approximately the WQ_v:

- Enter the horizontal axis on Figure 1.11 with the impervious percentage of the watershed and read upward to the predominant soil type (interpolation between curves is permitted)
- Read left to the factor
- Multiply the curve number for the sub-watershed that includes the water quality basin by this factor – this provides a smaller curve number

The difference in curve number will generate a runoff hydrograph that has a volume less than the original volume by an amount approximately equal to the WQ_v. This method should be used only for bioretention areas, filter facilities, and infiltration trenches where the drawdown time is ≥ 24 hours.

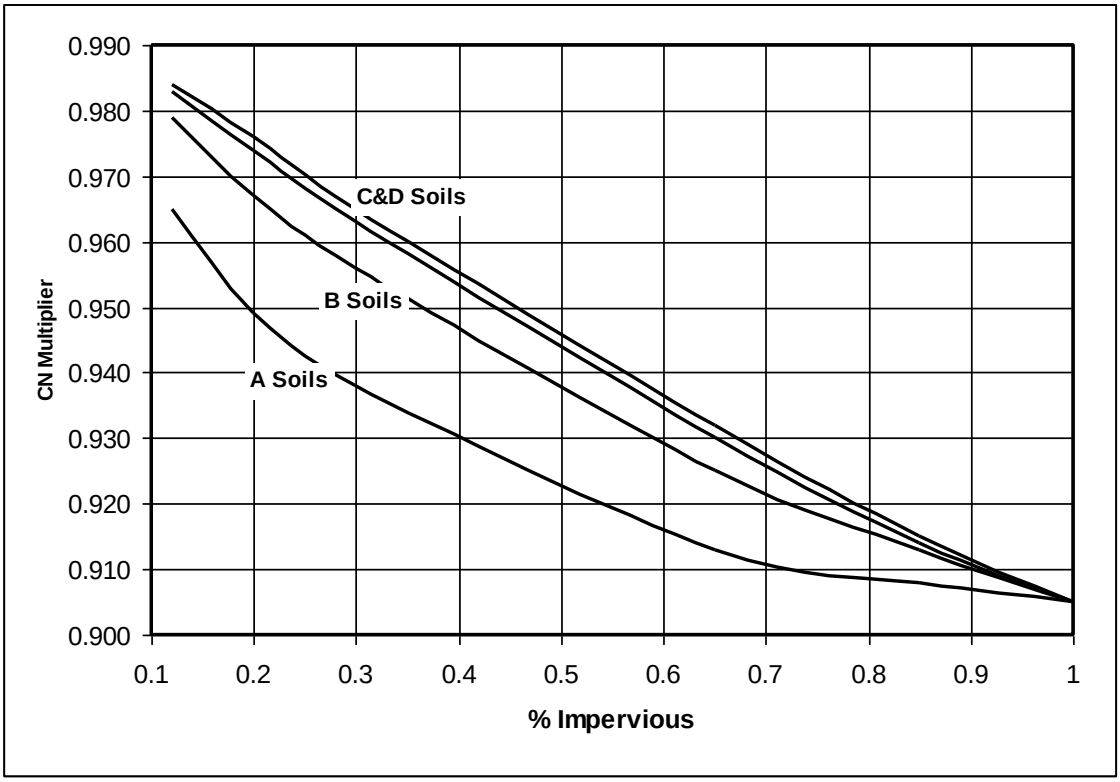


Figure 1.11 Curve Number Adjustment Factor

Example

A site design employs an infiltration trench for the WQ_v and has a curve number of 72, is B type soil, and has an impervious percentage of 60%, the factor from Figure 1.11 is 0.93. The curve number to be used in calculation of a runoff hydrograph for the quantity controls would be: (72*0.93) = 67.

2.0 Bioretention

Structural Stormwater Control

North Central Texas Council of Governments

Integrated Stormwater Management iSWM



Bioretention Facility in San Antonio, TX. *Source: Tetra Tech*

Description

Bioretention facilities, sometimes called rain gardens or bioretention filters, are vegetated basins or landscaped areas that capture stormwater runoff and provide filtration and treatment using engineered filter media. Bioretention areas are flexible per the needs of most site locations.

Design Considerations

- Consists of a grass filter, a sand bed, stormwater ponding area, an organic/mulch layer, planting soil, and selected landscaping for vegetation
- The facility works on any soil group
- Can be designed with an underdrain to send treated water into an outlet
- Use native plants as recommended
- Can be designed in-line or off-line
- Requires a footprint of 5-7% of the tributary impervious area

Key Advantages

- They are highly effective at removing pollutants and reducing peak flow storm events for small storms
- Bioretention areas work well in areas with a small drainage area (recommended for between 2 and 5 acres)
- Bioretention facilities can handle large amounts of impervious areas
- Bioretention areas have relatively low maintenance requirements
- Due to their incorporation of landscaping, bioretention facilities can be used as an aesthetic feature

Limitations

- Landscaping of bioretention facilities in public areas must be maintained to prevent overgrowth
- Bioretention areas cannot be used in areas with steep slopes
- Bioretention areas are not designed to manage peak flows from large storm events

Target Constituent	Removal Rate 0% → 100%
Total Suspended Solids	
Total Phosphorus	
Total Nitrogen	
Fecal Coliform	*insufficient data*
Heavy Metals	

Implementation Considerations



Suitability

The iSWM manual has designated that bioretention facilities are suitable for providing:



**in certain situations*

Maintenance

- Trash, leaf, debris and sediment removal
- Weeding/removing unwanted vegetation
- Replacing dead and dying vegetation
- Raking and replacing the top mulch layer
- Irrigating plants after planting and during the dry season
- Replace soil media on an as-needed basis
- Clean inlet and outlet pipes when required
- Repair eroded locations

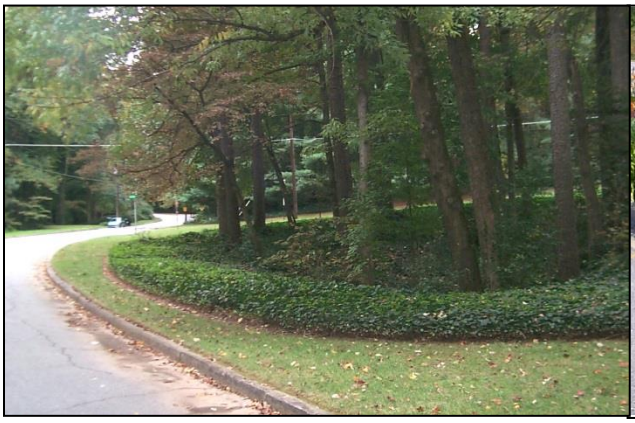
Published April 2020

2.1 General Description

Bioretention areas (also referred to as *bioretention filters* or *rain gardens*) are structural stormwater controls that capture and temporarily store the water quality protection volume (WQ_v) using soils and vegetation in shallow basins or landscaped areas to remove pollutants from stormwater runoff.

Bioretention areas are engineered facilities in which runoff is conveyed as sheet flow to the "treatment area" which consists of a grass buffer strip, ponding area, organic or mulch layer, planting soil, and vegetation. An optional sand bed can also be included in the design to provide aeration and drainage of the planting soil. The filtered runoff is typically collected and returned to the conveyance system, though it can also infiltrate into the surrounding soil in areas with porous soils.

There are numerous design applications, both on- and off-line, for bioretention areas. These include use on single-family residential lots (*rain gardens*), as off-line facilities adjacent to parking lots, along highway and road drainage swales, within larger landscaped pervious areas, and as landscaped islands in impervious or high-density environments. Figures 2.1 and 2.2 illustrate a number of examples of bioretention facilities in both photographs and drawings.



Single-Family Residential "Rain Garden"



Landscaped Island

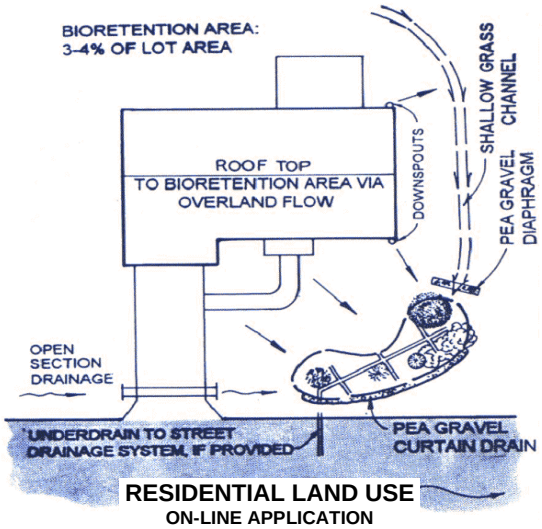


Newly Constructed Bioretention Area

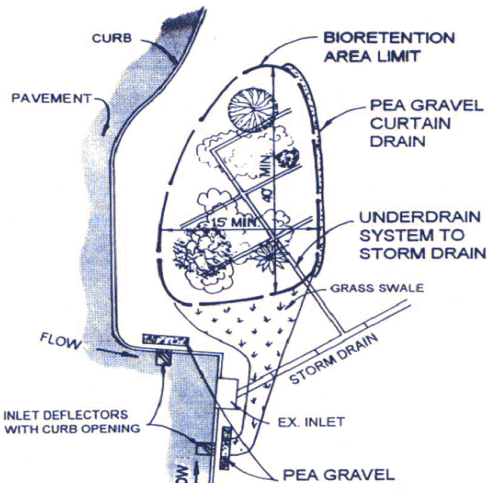


Newly Planted Bioretention Area After Storm

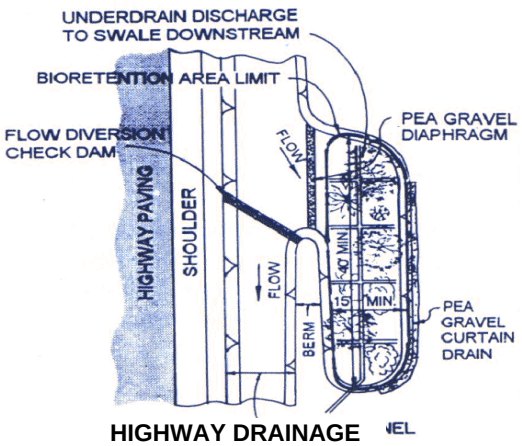
Figure 2.1 Bioretention Area Examples



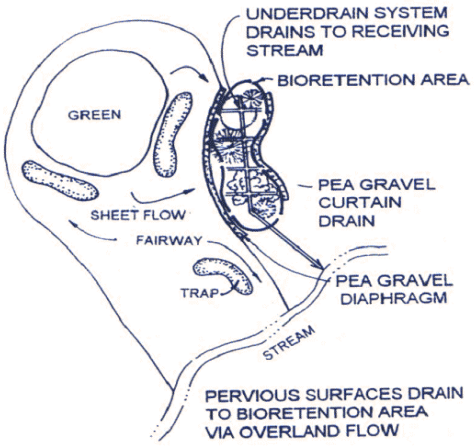
RESIDENTIAL LAND USE
ON-LINE APPLICATION

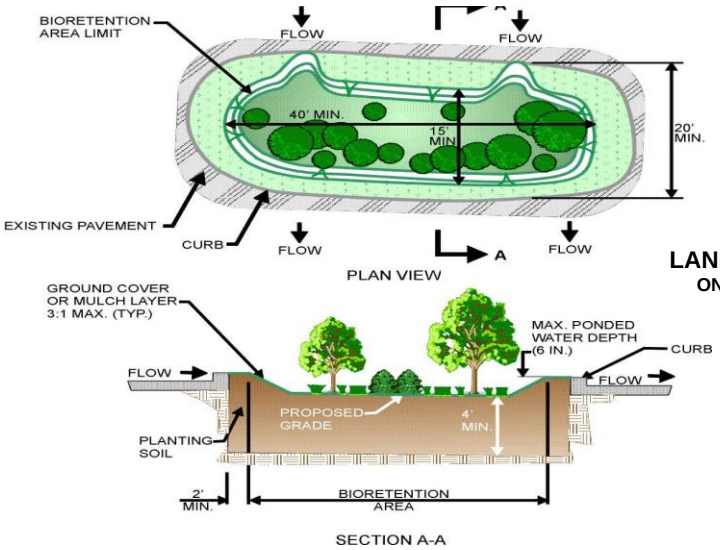


PARKING LOT RUNOFF
OFF-LINE APPLICATION



HIGHWAY DRAINAGE
OFF-LINE APPLICATION





LANDSCAPED ISLAND
ON-LINE APPLICATION

Figure 2.2 Bioretention Area Applications
(Source: Claytor and Schueler, 1996)

2.2 Stormwater Management Suitability

Bioretention areas are designed primarily for stormwater quality, i.e. the removal of stormwater pollutants. Bioretention can provide limited runoff quantity control, particularly for smaller storm events. These facilities may sometimes be used to partially or completely meet streambank protection requirements on smaller sites. However, bioretention areas will typically need to be used in conjunction with another structural control to provide streambank protection as well as flood control. It is important to ensure that a bioretention area safely bypasses higher flows.

Water Quality Protection

Bioretention is an excellent stormwater treatment practice due to the variety of pollutant removal mechanisms. Each of the components of the bioretention area is designed to perform a specific function (see Figure 2.3 of this section). The *grass filter strip (or grass channel)* reduces incoming runoff velocity and filters particulates from the runoff. The *ponding area* provides for temporary storage of stormwater runoff prior to its evaporation, infiltration, or uptake and provides additional settling capacity. The *organic or mulch layer* provides filtration as well as an environment conducive to the growth of microorganisms that degrade hydrocarbons and organic material. The *planting soil* in the bioretention facility acts as a filtration system, and clay in the soil provides adsorption sites for hydrocarbons, heavy metals, nutrients, and other pollutants. Both *woody and herbaceous plants* in the ponding area provide vegetative uptake of runoff and pollutants and also serve to stabilize the surrounding soils. Finally, an optional *sand bed* provides for positive drainage and aerobic conditions in the planting soil and provides a final polishing treatment media.

Section 2.3 gives data on pollutant removal efficiencies that can be used for planning and design purposes.

Streambank Protection

For smaller sites, a bioretention area may be designed to capture the entire streambank protection volume SP_v in either an off- or on-line configuration. Given that a bioretention facility is typically designed to completely drain over 48 hours, the requirement of extended detention of the 1-year, 24-hour storm runoff volume will be met. For larger sites where only the WQ_v is diverted to the bioretention facility, another structural control must be used to provide SP_v extended detention.

Flood Control

Bioretention areas must provide flow diversion and/or be designed to safely pass extreme storm flows and protect the ponding area, mulch layer, and vegetation.

Credit for the volume of runoff removed and treated in the bioretention area may be taken in flood control calculations (see Section 1.0).

2.3 Pollutant Removal Capabilities

Bioretention areas are presumed to be able to remove 80% of the total suspended solids load in typical urban post-development runoff when sized, designed, constructed, and maintained in accordance with the recommended specifications. Undersized or poorly designed bioretention areas can reduce TSS removal performance.

The following design pollutant removal rates are conservative average pollutant reduction percentages for design purposes derived from sampling data, modeling, and professional judgment. In a situation where a removal rate is not deemed sufficient, additional controls may be put in place at the given site in a series or “treatment train” approach.

- Total Suspended Solids – 80%
- Total Phosphorus – 60%
- Total Nitrogen – 50%

- Fecal Coliform – insufficient data
- Heavy Metals – 80%

For additional information and data on pollutant removal capabilities for bioretention areas, see the National Pollutant Removal Performance Database (2nd Edition) available at www.cwp.org and the National Stormwater Best Management Practices (BMP) Database at www.bmpdatabase.org

2.4 Application and Site Feasibility Criteria

Bioretention areas are suitable for many types of development, from single-family residential to high-density commercial projects. Bioretention is also well suited for small lots, including those of one acre or less. Because of its ability to be incorporated in landscaped areas, the use of bioretention is extremely flexible. Bioretention areas are an ideal structural stormwater control for use as roadway median strips and parking lot islands and are also good candidates for the treatment of runoff from pervious areas, such as a golf course. Bioretention can also be used to retrofit existing development with stormwater quality treatment capacity.

The following criteria should be evaluated to ensure the suitability of a bioretention area for meeting stormwater management objectives on a site or development.

General Feasibility

- Suitable for Residential Subdivision Usage – YES
- Suitable for High Density/Ultra Urban Areas – YES
- Regional Stormwater Control – NO
- Hot Spot - YES

Physical Feasibility - Physical Constraints at Project Site

- Drainage Area – 5 acres maximum; 0.5 to 2 acres are preferred.
- Space Required – Approximately 5-7% of the tributary impervious area is normally required.
- Site Slope – No more than 6% slope
- Minimum Head – Elevation difference needed at a site from the inflow to the outflow: 3-5 feet
- Minimum Depth to Water Table – A separation distance of 2 feet recommended between the bottom of the bioretention facility and the elevation of the seasonally high water table.
- Soils – No restrictions; engineered media required

Other Constraints / Considerations

- Aquifer Protection – Do not allow infiltration of filtered hotspot runoff into groundwater

2.5 Planning and Design Criteria

*The following criteria are to be considered **minimum** standards for the design of a bioretention facility. Consult with the local review authority to determine if there are any variations to these criteria or additional standards that must be followed.*

A Location and Siting

- Bioretention areas should have a maximum contributing drainage area of 5 acres or less; 0.5 to 2 acres are preferred. Multiple bioretention areas can be used for larger areas.
- Bioretention areas can either be used to capture sheet flow from a drainage area or function as an off-line device. On-line designs should be limited to a maximum drainage area of 0.5 acres unless special precautions are taken to protect from erosion during high flows.

- When used in an off-line configuration, the water quality protection volume (WQ_v) is diverted to the bioretention area through the use of a flow splitter. Stormwater flows greater than the WQ_v are diverted to other controls or downstream (see *Section 1.4.2* for more discussion of off-line systems and design guidance for diversion structures and flow splitters).
- Bioretention systems are designed for intermittent flow and must be allowed to drain and re-aerate between rainfall events. They should not be used on sites with a continuous flow from groundwater, sump pumps, or other sources.
- Bioretention area locations should be integrated into the site planning process, and aesthetic considerations should be taken into account in their siting and design. Elevations must be carefully worked out to ensure that the desired runoff flow enters the facility with no more than the maximum design depth.

B General Design

A well-designed bioretention area consists of:

- Grass filter strip (or grass channel) between the contributing drainage area and the ponding area, except where site conditions preclude its use,
- Ponding area containing vegetation with a planting soil bed,
- Organic/mulch layer,
- Pea gravel layer between the planting soil and the gravel underneath to provide filtering of the particles prior to entering gravel layer,
- Gravel and perforated pipe underdrain system to collect runoff that has filtered through the soil layers (bioretention areas can optionally be designed to infiltrate into the soil – see description of infiltration trenches for infiltration criteria).

A bioretention area design will also include some of the following:

- Optional **sand filter layer** to spread flow, filter runoff, and aid in aeration and drainage of the planting soil.
- **Stone diaphragm** at the beginning of the grass filter strip to reduce runoff velocities and spread flow into the grass filter.
- **Inflow diversion** or an **overflow structure** consisting of one of five main methods:
  1) Use a flow diversion structure
  2) For curbed pavements use an inlet deflector (see Figure 2.6).
  3) Use a slotted curb and design the parking lot grades to divert the WQ_v into the facility. Bypass additional runoff to a downstream catch basin inlet. Requires temporary ponding in the parking lot (see Figure 2.5).
  4) Figure 2.2 illustrates the use of a short deflector weir (maximum height 6 inches) designed to divert the maximum water quality peak flow into the bioretention area.
  5) An in-system overflow consisting of an overflow catch basin inlet and/or a pea gravel curtain drain overflow.

See Figure 2.3 for an overview of the various components of a bioretention area. Figure 2.4 provides a plan view and profile schematic of an on-line bioretention area. An example of an off-line facility is shown in Figure 2.5.

C Physical Specifications / Geometry

- Recommended minimum dimensions of a bioretention area are 10 feet wide by 40 feet long. All designs except small residential applications should maintain a length to width ratio of at least 2:1.

- The planting soil filter bed is sized using a Darcy's Law equation with a filter bed drain time of less than 48 hours (less than 6 hours residential neighborhoods and 24 hours non-residential preferred) and a coefficient of permeability (k) of greater than 0.5 ft/day.
- The maximum recommended ponding depth of the bioretention areas is 6 inches with a drain time normally of 3 to 4 hours in residential settings.
- The planting soil bed must be at least 2.5 feet in depth and up to 4 feet if large trees are to be planted. Planting soils should be sandy loam, loamy sand, or loam texture (should contain a minimum of 35 to 60% sand, by volume). The clay content for these soils should by less than 25% by volume. Soils should fall within the SM, ML, SC classifications of the Unified Soil Classification System (USCS). A permeability of at least 1.0 feet per day (0.5"/hr) is required (a conservative value of 0.5 feet per day should be used for design). The soil should be free of stones, stumps, roots, or other woody material over 1" in diameter. Brush or seeds from noxious weeds, such as Johnson Grass, Mugwort, Nutsedge, and Canadian Thistle should not be present in the soils. Placement of the planting soil should be in lifts of 12 to 18", loosely compacted (tamped lightly with a dozer or backhoe bucket). The specific characteristics are presented in the table below.

Planting Soil Characteristics	
Parameter	**Value**
pH range	5.2 to 7.00
Organic matter	1.5 to 4.0%
Magnesium	35 lbs. per acre, minimum (0.0072 lbs/Sq yd)
Phosphorus (P_2O_5)	75 lbs. per acre, minimum (0.0154 lbs/Sq yd)
Potassium (K_2O)	85lbs. per acre, minimum (0.0175 lbs/Sq yd)
Soluble salts	500 ppm
Clay	10 to 25%
Silt	30 to 55%
Sand	35 to 60%

(Adapted from EQR, 1996; ETAB, 1993)

- For on-line configurations, a grass filter strip with a pea gravel diaphragm is typically utilized (see Figure 2.3) as the pretreatment measure. The required length of the filter strip depends on the drainage area, imperviousness, and the filter strip slope. Design guidance on filter strips for pretreatment can be found in *Section 13.0* (*Filter Strip)*.
- For off-line applications, a grass channel with a pea gravel diaphragm flow spreader is used for pretreatment. The length of the grass channel depends on the drainage area, land use, and channel slope. The minimum grassed channel length should be 20 feet. Design guidance on grass channels for pretreatment can be found in *Section 4*.0 (*Grass Channel)*.
- The mulch layer should consist of 2 to 4 inches of commercially available standard landscape style, single or double, shredded hardwood mulch or chips. The mulch layer should be well aged (stockpiled or stored for at least 12 months), uniform in color, and free of other materials, such as weed seeds, soil, roots, etc.
- The sand bed (optional) should be 12 to 18 inches thick. Sand should be clean and have less than 15% silt or clay content.
- Pea gravel for the 4" to 9" thick layer above the gravel bedding (and diaphragm and curtain, where used), should be ASTM D 448 size No. 6 ($^1/_8$" to ¼").

- The underdrain collection system is equipped with a 6-inch perforated PVC pipe (AASHTO M 252) in an 8-inch gravel layer. The pipe should have 3/8-inch perforations, spaced at 6-inch centers, with a minimum of 4 holes per row. The pipe is spaced at a maximum of 10 feet on center and a minimum grade of 0.5% must be maintained.
- A narrow 24" wide permeable filter fabric is placed between the gravel layer and the pea gravel layer directly above the perforated pipes to limit piping of soil directly into the pipe. Filter fabric is also placed along the vertical or sloping outer walls of the bioretention system to limit vertical infiltration prior to filtration through the soil. The filter fabric should be non-woven with a minimum permittivity rate of 75 to 125 gal/min/ft^2. Additional geotextile properties are provided in the table below.

Filter Fabric Properties		
Geotextile Property	**Value**	**Test Method**
Trapezoidal Tear (lbs)	40 (min)	ASTM D4533
Permeability (cm/sec)	0.2 (min)	ASTM D4491
AOS (sieve size)	#60 - #70 (min)	ASTM D4751
Ultraviolet resistance	70% or greater	ASTM D4355

D Pretreatment / Inlets

- Adequate pretreatment and inlet protection for bioretention systems is provided when all of the following are provided: (a) grass filter strip below a level spreader, or grass channel, (b) pea gravel diaphragm and (c) an organic or mulch layer.

E Outlet Structures

- Outlet pipe is to be provided from the underdrain system to the facility discharge. Due to the slow rate of filtration, outlet protection is generally unnecessary.

F Emergency Spillway

- An overflow structure and nonerosive overflow channel must be provided to safely pass flows from the bioretention area that exceeds the storage capacity to a stabilized downstream area or watercourse. If the system is located off-line, the overflow should be set above the shallow ponding limit.
- The high flow overflow system within the structure consists of a yard drain catch basin (Figure 2.3), though any number of conventional systems could be used. The throat of the catch basin inlet is normally placed 6 inches above the mulch layer. It should be designed as a domed grate or a covered weir structure to avoid clogging with floatation mulch and debris, and should be located at a distance from inlets to avoid short circuiting of flow. It may also be placed into the side slope of the structure maintaining a neat contoured appearance.

G Maintenance Access

- Adequate access must be provided for all bioretention facilities for inspection, maintenance, and landscaping upkeep. Appropriate equipment and vehicles are essential.

H Safety Features

- Bioretention areas generally do not require any special safety features. Fencing of bioretention facilities is not generally desirable.

I Landscaping

- Landscaping is critical to the performance and function of bioretention areas.

- A dense and vigorous vegetative cover should be established over the contributing pervious drainage areas before runoff can be accepted into the facility. Side slopes should be sodded to limit erosion of fine particles onto the bioretention surface.
- The bioretention area should be vegetated to resemble a terrestrial forest ecosystem, with a mature tree canopy, subcanopy of understory trees, scrub layer, and herbaceous ground cover. Three species each of both trees and scrubs are recommended to be planted.
- The tree-to-shrub ratio should be 2:1 to 3:1. On average, the trees should be spaced 8 feet apart. Plants should be placed at regular intervals to replicate a natural forest. Woody vegetation should not be specified at inflow locations.
- After the trees and shrubs are established, the ground cover and mulch should be established.
- Choose plants based on factors such as whether native or not, resistance to drought and inundation, cost aesthetics, maintenance, etc. Planting recommendations for bioretention facilities are as follows:

- Native plant species should be specified over non-native species.
- Vegetation should be selected based on a specified zone of hydric tolerance.
- A selection of trees with an understory of shrubs and herbaceous materials should be provided.

Additional information and guidance on the appropriate woody and herbaceous species appropriate for bioretention in North Central Texas, and their planting and establishment, can be found in *Section 1.5.2 of the Landscape Technical Manual*.

J Additional Site-Specific Design Criteria and Issues

Physiographic Factors - Local terrain design constraints

- Low Relief – Use of bioretention areas may be limited by low head
- High Relief – Ponding area surface must be relatively level
- Karst – Use poly-liner or impermeable membrane to seal bottom

Soils

- No restrictions

Special Downstream Watershed Considerations

- Aquifer Protection – No restrictions, if designed with no infiltration (i.e. outflow to groundwater)

2.6 Design Procedures

Step 1 Compute runoff control volumes from the *integrated* Design Focus Areas

Calculate the Water Quality Protection Volume (WQ_v), Streambank Protection Volume (SP_v), and the flood mitigation storm Flood Discharge (Q_f).

Details on the *integrated* Design Focus Areas are found in *Section 1.0 of the Planning Technical Manual*.

Step 2 Determine if the development site and conditions are appropriate for the use of a bioretention area

Consider the Application and Site Feasibility Criteria in *Sections 2.4 and 2.5* (Location and Siting).

Step 3 Confirm local design criteria and applicability

Consider any special site-specific design conditions/criteria from *Section 2.5.10* (Additional Site-Specific Design Criteria and Issues).

Check with local officials and other agencies to determine if there are any additional restrictions and/or surface water or watershed requirements that may apply.

Step 4 Compute WQ_v peak discharge (Q_{wq})

The peak rate of discharge for water quality design storm is needed for sizing of off-line diversion structures (see *Section 1.4 of the Water Quality Technical Manual*).

(a) Using WQ_v (or total volume to be captured), compute CN

(b) Compute time of concentration using TR-55 method

(c) Determine appropriate unit peak discharge from time of concentration

(d) Compute Q_{wq} from unit peak discharge, drainage area, and WQ_v.

Step 5 Size flow diversion structure, if needed

A flow regulator (or flow splitter diversion structure) should be supplied to divert the WQ_v to the bioretention area.

Size low flow orifice, weir, or other device to pass Q_{wq}.

Step 6 Determine size of bioretention ponding/filter area

The required planting soil filter bed area is computed using the following equation (based on Darcy's Law):

$$\mathbf{A_f = (WQ_v)\,(d_f) / [\,(k)\,(h_f + d_f)\,(t_f)\,]} \qquad \mathbf{(2.1)}$$

where:

A_f = surface area of ponding area (ft^2)
WQ_v = water quality protection volume (or total volume to be captured)
d_f = filter bed depth (2.5 feet minimum)
k = coefficient of permeability of filter media (ft/day) (use 0.5 ft/day for silt-loam)
h_f = average height of water above filter bed (ft) (typically 3 inches, which is half of the 6-inch ponding depth)
t_f = design filter bed drain time (days) (2.0 days or 48 hours is recommended maximum)

Step 7 Set design elevations and dimensions of facility

See *Section 2.5.3* (Physical Specifications/Geometry).

Step 8 Design conveyances to facility (off-line systems)

See the example figures to determine the type of conveyances needed for the site.

Step 9 Design pretreatment

Pretreat with a grass filter strip (on-line configuration) or grass channel (off-line), and stone diaphragm.

Step 10 Size underdrain system

See *Section 2.5.3* (Physical Specifications/Geometry)

Step 11 Design emergency overflow

An overflow must be provided to bypass and/or convey larger flows to the downstream drainage system or stabilized watercourse. Nonerosive velocities need to be ensured at the outlet point.

Step 12 Prepare Vegetation and Landscaping Plan

A landscaping plan for the bioretention area should be prepared to indicate how it will be established with vegetation.

See *Section 3.6* (Landscaping) and *Section 1.5.2 of the Landscape Technical Manual* for more details.

See *Section 29.3* for a Bioretention Area Design Example

2.7 Inspection and Maintenance Requirements

Table 2.1 Typical Maintenance Activities for Bioretention Areas

Activity	Schedule
• Pruning and weeding to maintain appearance. • Mulch replacement when erosion is evident. • Remove trash and debris.	As needed
• Inspect inflow points for clogging (off-line systems). Remove any sediment. • Inspect filter strip/grass channel for erosion or gullying. Re-seed or sod as necessary. • Trees and shrubs should be inspected to evaluate their health and remove any dead or severely diseased vegetation.	Semi-annually
• The planting soils should be tested for pH to establish acidic levels. If the pH is below 5.2, limestone should be applied. If the pH is above 7.0 to 8.0, then iron sulfate plus sulfur can be added to reduce the pH.	Annually
• Replace mulch over the entire area. • Replace pea gravel diaphragm if warranted (or when the voids are obviously filled with sediment and water is no longer infiltrating).	2 to 3 years

(Source: EPA, 1999)

Additional Maintenance Considerations and Requirements

The surface of the ponding area may become clogged with fine sediment over time. Core aeration or cultivating of unvegetated areas may be required to ensure adequate filtration.



Regular inspection and maintenance is critical to the effective operation of bioretention facilities as designed. Maintenance responsibility for a bioretention area should be vested with a responsible authority by means of a legally binding and enforceable maintenance agreement that is executed as a condition of plan approval.

2.8 Example Schematics

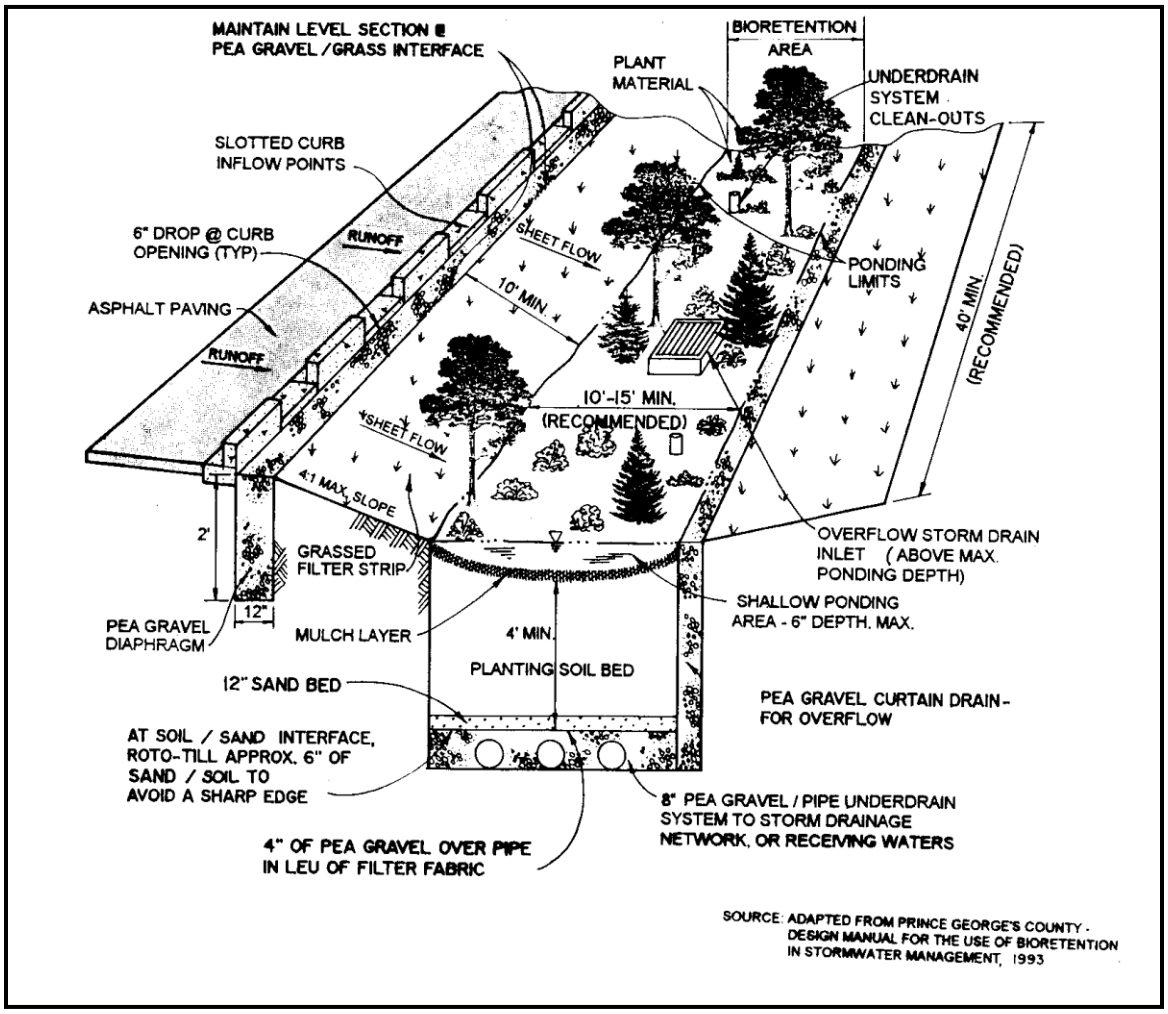
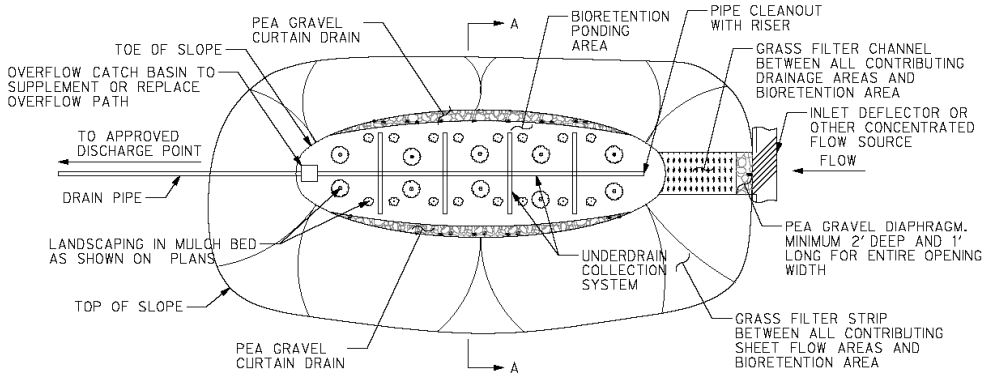
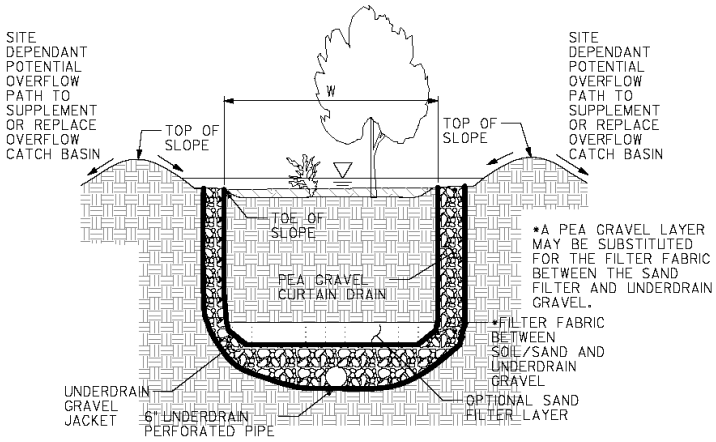


Figure 2.3 Schematic of a Typical Bioretention Area
(Source: Claytor and Schueler, 1996)



BIORETENTION AREA PLAN VIEW
NOT TO SCALE



SECTION A-A
NOT TO SCALE

NOTES:

1. RECOMMENDED MINIMUM WIDTH AND LENGTHS ARE 10' AND 40', RESPECTIVELY BUT MAY VARY SO LONG AS A 2L:1W RATIO IS MAINTAINED. ACTUAL DIMENSIONS SHALL BE PROVIDED BY DESIGNER IN PLANS AND SHALL BE BASED ON SITE SPECIFIC CONDITIONS. DESIGN PROFESSIONAL SHALL AT A MINIMUM PROVIDE THE FOLLOWING INFORMATION:
- HORIZONTAL CONTROL
- VERTICAL CONTROL
- LANDSCAPING
- WATER QUALITY PROTECTION VOLUME
- STREAMBANK PROTECTION VOLUME
- FLOOD DISCHARGE
- OVERFLOW (CATCH BASIN AND/OR PATH)
- MEDIA SECTION AND RESPECTIVE THICKNESSES
- DRAINAGE AGGREGATE AND UNDERDRAIN INFORMATION

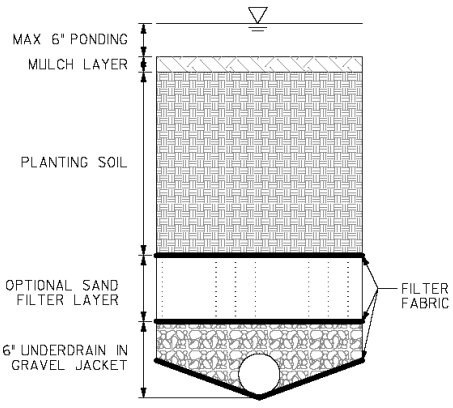
2. DESIGN ENGINEER IS RESPONSIBLE FOR PROVIDING CALCULATIONS THAT DEMONSTRATE THAT COMPOSITE MEDIA AND SYSTEM AS A WHOLE CAN EFFECTIVELY TREAT THE WATER QUALITY VOLUME (WQv) AS SPECIFIED IN SECTION 1.4 OF THE NCTCOG WATER QUALITY TECHNICAL MANUAL. FLOW IN EXCESS OF THE WQV SHALL BE DESIGNED TO BYPASS THE SYSTEM VIA THE OVERFLOW SYSTEM(S).

3. DESIGN OF THIS SYSTEM SHALL FOLLOW THAT OF SECTION 2 "BIORETENTION AREA" CONTAINED IN THE NCTCOG SITE DEVELOPMENT CONTROLS ISWM TECHNICAL MANUAL.

4. LANDSCAPING SPECIFIED BY DESIGNER SHOULD BE CARFULLY CHOSEN TO ENSURE SURVIVABILITY DURING BOTH SUBMERGED AND DROUGHT EXTREMES.

5. DESIGNER SHALL BE RESPONSIBLE FOR PREPARING AN OPERATIONS AND MAINTENANCE MANUAL OR MATRIX FOR SYSTEMS TO AID OWNER IN DETERMINING MAINTENANCE PROCEEDURES AND INTERVALS. DESIGNER SHALL ALSO CONSIDER MAINTENANCE ACCESS WHEN DESIGNING SYSTEM.

6. DESIGNER SHALL ENSURE THAT SYSTEM IS CAPABLE OF BEING FULLY DRAINED WITHIN 48-HOURS.



MEDIA CUT SECTION
NOT TO SCALE

Figure 2.4 Schematic of a Typical On-Line Bioretention Area
(Source: Claytor and Schueler, 1996)

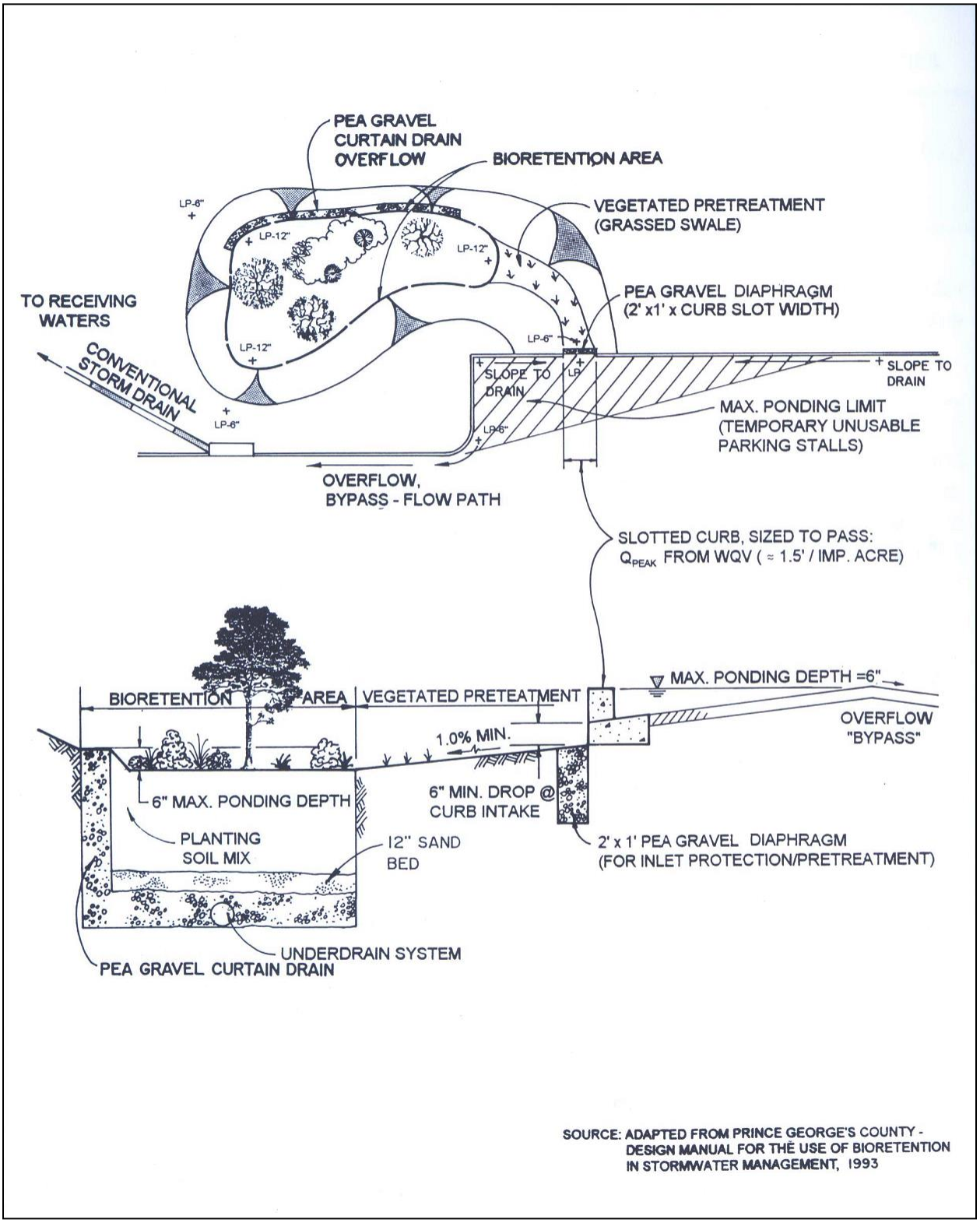


Figure 2.5 Schematic of a Typical Off-Line Bioretention Area
(Source: Claytor and Schueler, 1996)

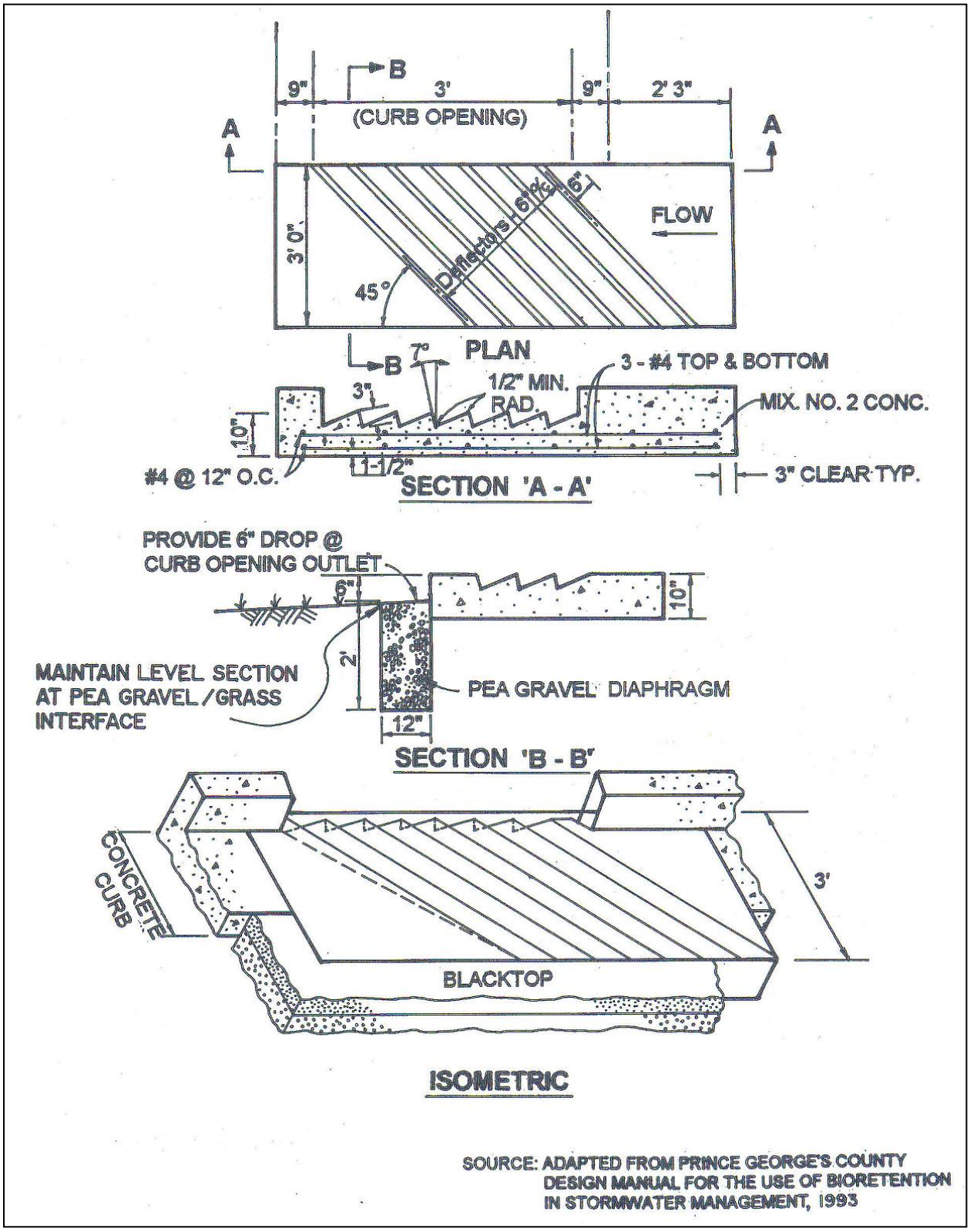
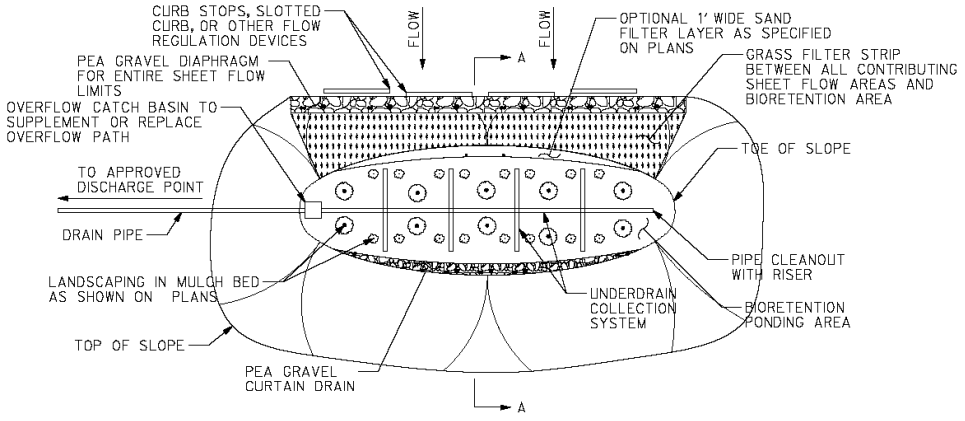
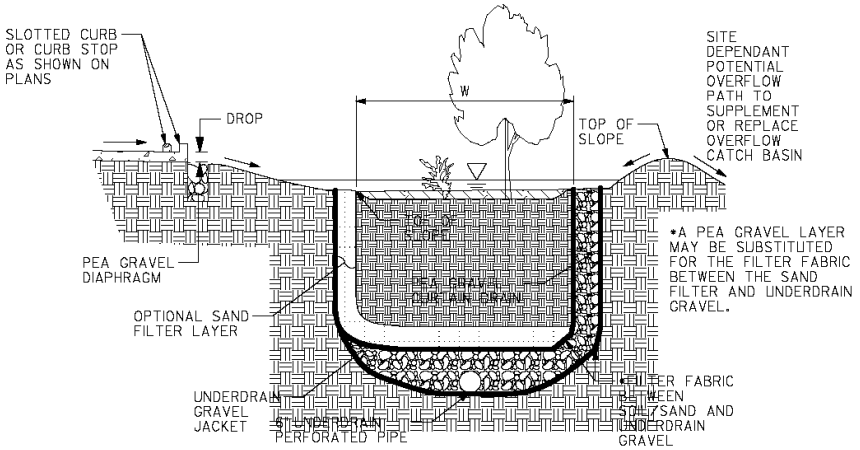


Figure 2.6 Schematic of a Typical Inlet Deflector
(Source: Claytor and Schueler, 1996)



BIORETENTION AREA PLAN VIEW

NOT TO SCALE



SECTION A-A

NOT TO SCALE

NOTES:

1. RECOMMENDED MINIMUM WIDTH AND LENGTHS ARE 10' AND 40', RESPECTIVELY BUT MAY VARY SO LONG AS A 2L:1W RATIO IS MAINTAINED. ACTUAL DIMENSIONS SHALL BE PROVIDED BY DESIGNER IN PLANS AND SHALL BE BASED ON SITE SPECIFIC CONDITIONS. DESIGN PROFESSIONAL SHALL AT A MINIMUM PROVIDE THE FOLLOWING INFORMATION:
- HORIZONTAL CONTROL
- VERTICAL CONTROL
- LANDSCAPING
- WATER QUALITY PROTECTION VOLUME
- STREAMBANK PROTECTION VOLUME
- FLOOD DISCHARGE
- OVERFLOW (CATCH BASIN AND/OR PATH)
- MEDIA SECTION AND RESPECTIVE THICKNESSES
- DRAINAGE AGGREGATE AND UNDERDRAIN INFORMATION

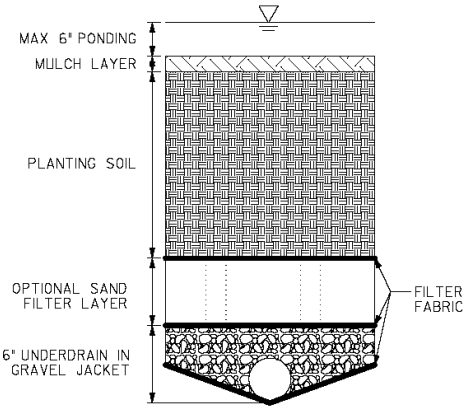
2. DESIGN ENGINEER IS RESPONSIBLE FOR PROVIDING CALCULATIONS THAT DEMONSTRATE THAT COMPOSITE MEDIA AND SYSTEM AS A WHOLE CAN EFFECTIVELY TREAT THE WATER QUALITY VOLUME (WQv) AS SPECIFIED IN SECTION 1.4 OF THE NCTCOG WATER QUALITY TECHNICAL MANUAL. FLOW IN EXCESS OF THE WQV SHALL BE DESIGNED TO BYPASS THE SYSTEM VIA THE OVERFLOW SYSTEM(S).

3. DESIGN OF THIS SYSTEM SHALL FOLLOW THAT OF SECTION 2 "BIORETENTION AREA" CONTIANED IN THE NCTCOG SITE DEVELOPMENT CONTROLS ISWM TECHNICAL MANUAL.

4. LANDSCAPING SPECIFIED BY DESIGNER SHOULD BE CARFULLY CHOSEN TO ENSURE SURVIVABILITY DURING BOTH SUBMERGED AND DROUGHT EXTREMES.

5. DESIGNER SHALL BE RESPONSIBLE FOR PREPARING AN OPERATIONS AND MAINTENANCE MANUAL OR MATRIX FOR SYSTEMS TO AID OWNER IN DETERMINING MAINTENANCE PROCEEDURES AND INTERVALS. DESIGNER SHALL ALSO CONSIDER MAINTENANCE ACCESS WHEN DESIGNING SYSTEM.

6. DESIGNER SHALL ENSURE THAT SYSTEM IS CAPABLE OF BEING FULLY DRAINED WITHIN 48-HOURS.



MEDIA CUT SECTION

NOT TO SCALE

Figure 2.7 Sheet Flow Bioretention Area

Table 2.2 Design Procedure Form: Bioretention Areas

Design Procedure Form: Bioretention Areas

PRELIMINARY HYDROLOGIC CALCULATIONS	
1a. Compute WQ_v volume requirements Compute Runoff Coefficient, R_v Compute WQ_v	R_v = _____ WQ_v = _____ acre-ft
1b. Compute SP_v Compute average release rate Compute (as necessary) Q_f	SP_v = _____ acre-ft release rate = _____ cfs Q_f = _____ cfs
BIORETENTION AREA DESIGN	
2. Is the use of a bioretention area appropriate?	***See subsections 5.2.1.4 and 5.2.1.5 - A***
3. Confirm local design criteria and applicability	
4. Determine size of bioretention filter area	A_f = _____ ft^2
5. Set design elevations and dimensions	Length = _____ ft Width = _____ ft _____ elevation top of facility _____ other elev: _____ _____ other elev: _____ _____ other elev: _____
6. Conveyance to bioretention facililty	_____Online or _____ Offline?
7. Pretreatment	Type: _____
8. Size underdrain area Based on guidance: Approx. 10% A_f	Length = _____ ft
9. Overdrain design	Type: _____ Size: _____
10. Emergency storm weir design Overflow weir - Weir equation	Length = _____ ft
11. Choose plants for planting area	*Select native plants based on resistance to drought and inundation, cost, aesthetics, maintenance, etc.* ***See Appendix F***

Notes: _____

3.0 Enhanced Swales

General Application
Structural Stormwater Control

North Central Texas Council of Governments
integrated Stormwater Management iSWM

Description

Enhanced swales are vegetated, open channels. They are designed to capture and treat stormwater runoff within dry or wet cells formed by check dams or other flow control devices. Enhanced swales can be wet or dry and are sometimes also called vegetated open channels or water quality swales.

Design Considerations

- Dry swales allow the entire water quality volume to be filtered or infiltrated and are often preferred in community settings.
- Wet swales are designed to retain water and act as a linear shallow wetland system.
- Enhanced swales require relatively low slopes, typically below 4%, to prevent scour.
- Side slopes need to be relatively shallow, typically 2:1 or flatter (recommended at 4:1).
- The bottom width of the facilities can vary but is typically between 2 and 8 feet.
- The conveyance storm event must be met, including the required minimum freeboard.
- Pretreatment is recommended and typically required.

Key Advantages

- Enhanced swales combine stormwater treatment with runoff conveyance.
- Enhanced swales are less expensive and more aesthetically pleasing than curb and gutter systems.
- Runoff velocity is reduced by enhanced swales.

Limitations

- Enhanced swales have higher maintenance requirements than curb and gutter systems.
- Enhanced swales are not appropriate for steep slopes (longitudinal slope must be less than 4%).
- Enhanced swales may cause a resuspension of sediment.
- Due to the wet nature of the swale, there is a potential for mosquitos or other vectors, as well as for odor, if not properly maintained.
- Residential areas may not like the 4"–6" grass height.



Enhanced Swale in San Diego, CA. *(Source: Tetra Tech)*

Target Constituent	Removal Rate (0% → 100%)
Total Suspended Solids	
Total Phosphorus	
Total Nitrogen	
Fecal Coliform	*insufficient data*
Heavy Metals	

Implementation Considerations



Suitability

The iSWM manual has designated enhanced swales as suitable for providing:



Maintenance

- Dry swales require a grass height of 4"–6", which needs to be maintained.
- Sediment accumulation should be removed from the forebay and the channel.
- Inspect for and repair erosion.
- Replant plant species if establishment does not occur.
- Trash, debris, sediment and grass clippings need to be removed.

Published October 2020

3.1 General Description

Enhanced swales (also referred to as *vegetated open channels* or *water quality swales*) are conveyance channels engineered to capture and treat the water quality volume (WQ_v) for a drainage area. They differ from a normal drainage channel or swale through the incorporation of specific features that enhance stormwater pollutant removal effectiveness.

Enhanced swales are designed with limited longitudinal slopes to force the flow to be slow and shallow, thus allowing for particulates to settle and limiting the effects of erosion. Berms and/or check dams installed perpendicular to the flow path promote settling and infiltration.

There are two primary enhanced swale designs, the *dry swale* and the *wet swale* (or *wetland channel*). Below are descriptions of these two designs:

- **Dry Swale** – The dry swale is a vegetated conveyance channel designed to include a filter bed of prepared soil that overlays an underdrain system. Dry swales are sized to allow the entire WQ_v to be filtered or infiltrated through the bottom of the swale. Because they are dry most of the time, they are often the preferred option in residential settings.
- **Wet Swale (Wetland Channel)** – The wet swale is a vegetated channel designed to retain water or marshy conditions that support wetland vegetation. A high water table or poorly drained soils are necessary to retain water. The wet swale essentially acts as a linear shallow wetland treatment system, where the WQ_v is retained.



Enhanced Dry Swale



Enhanced Dry Swale

Figure 3.1 Enhanced Swale Examples

Dry and wet swales are not to be confused with a *filter strip* or *grass channel*, which are Limited Application structural controls and not considered acceptable for meeting the TSS removal performance goal by themselves. Ordinary *grass channels* are not engineered to provide the same treatment capability as a well-designed dry swale with filter media. *Filter strips* are designed to accommodate overland flow rather than channelized flow and can be used as stormwater credits to help reduce the total water quality treatment volume for a site. Both of these practices may be used for pretreatment or included in a "treatment train" approach where redundant treatment is provided. Please see a further discussion of these limited application structural controls in *Sections 4.0 and 13.0 of the Site Development Controls Technical Manual*, respectively.

3.2 Stormwater Management Suitability

Enhanced swale systems are designed primarily for stormwater quality and have only a limited ability to provide streambank protection or to convey higher flows to other controls.

Water Quality

Dry swale systems rely primarily on filtration through an engineered media to provide removal of stormwater contaminants. Wet swales achieve pollutant removal both from sediment accumulation and biological removal.

Section 3.3 provides pollutant removal efficiencies that can be used for planning and design purposes.

Streambank Protection

Generally, only the WQ_v is treated by a dry or wet swale, and another structural control must be used to provide SP_v extended detention. However, for some smaller sites, a swale may be designed to capture and detain the full SP_v.

On-Site Flood Control

Enhanced swales must provide flow diversion and/or be designed to safely pass overbank flood flows. Another structural control must be used in conjunction with an enhanced swale system to reduce the post-development peak flow.

Downstream Flood Control

Enhanced swales must provide flow diversion and/or be designed to safely pass extreme storm flows. Another structural control must be used in conjunction with an enhanced swale system to reduce the post-development peak flow.

3.3 Pollutant Removal Capabilities

Both the dry and wet enhanced swale are presumed to be able to remove 80% of the total suspended solids load in typical urban post-development runoff when sized, designed, constructed, and maintained in accordance with the recommended specifications. Undersized or poorly designed swales can reduce TSS removal performance.

The following design pollutant removal rates are conservative average pollutant reduction percentages for design purposes derived from sampling data, modeling, and professional judgment. In a situation where a removal rate is not deemed sufficient, additional controls may be put in place at the given site in a series or "treatment train" approach.

- Total Suspended Solids – 80%
- Total Phosphorus – Dry Swale 50% / Wet Swale 25%
- Total Nitrogen – Dry Swale 50% / Wet Swale 40%
- Fecal Coliform – insufficient data
- Heavy Metals – Dry Swale 40% / Wet Swale 20%

For additional information and data on pollutant removal capabilities for enhanced dry and wet swales, see the National Pollutant Removal Performance Database (2nd Edition) available at www.cwp.org and the National Stormwater Best Management Practices (BMP) Database at www.bmpdatabase.org

3.4 Application and Feasibility Criteria

Enhanced swales can be used in a variety of development types; however, they are primarily applicable to residential and institutional areas of low to moderate density where the impervious cover in the contributing drainage area is relatively small and along roads and highways. Dry swales are mainly used in moderate to large lot residential developments, small impervious areas (parking lots and rooftops), and along rural

highways. Wet swales tend to be used for highway runoff applications, small parking areas, and in commercial developments as part of a landscaped area.

Because of their relatively large land requirement, enhanced swales are generally not used in higher density areas. In addition, wet swales may not be desirable for some residential applications, due to the presence of standing and stagnant water, which may create nuisance odor or mosquito problems.

The topography and soils of a site will determine the applicability of the use of one of the two enhanced swale designs. Overall, the topography should allow for the design of a swale with sufficient slope and cross-sectional area to maintain nonerosive velocities. The following criteria should be evaluated to ensure the suitability of a stormwater pond for meeting stormwater management objectives on a site or development.

General Feasibility

- Suitable for Residential Subdivision Usage – YES
- Suitable for High Density/Ultra Urban Areas – NO
- Regional Stormwater Control – NO

Physical Feasibility - Physical Constraints at Project Site

- Drainage Area – 5 acres maximum
- Space Required – Approximately 10 to 20% of the tributary impervious area
- Site Slope – Typically no more than 4% channel slope
- Minimum Head – Elevation difference needed at a site from the inflow to the outflow: 3 to 5 feet for dry swale; 1 foot for wet swale
- Minimum Depth to Water Table – 2 feet required between the bottom of a dry swale and the elevation of the seasonally high water table if treating a hotspot or an aquifer recharge zone. Wet swale is below water table or placed in poorly drained soils
- Soils – Engineered media for dry swale

Other Constraints / Considerations

- Aquifer Protection – Infiltration should not be allowed for hotspots

3.5 Planning and Design Criteria

*The following criteria are to be considered **minimum** standards for the design of an enhanced swale system. Consult with the local review authority to determine if there are any variations to these criteria or additional standards that must be followed.*

A Location and Siting

A dry or wet swale should be sited such that the topography allows for the design of a channel with sufficiently mild slope (unless small drop structures are used) and cross-sectional area to maintain nonerosive velocities.

Enhanced swale systems should have a contributing drainage area of 5 acres or less.

Swale siting should also take into account the location and use of other site features, such as buffers and undisturbed natural areas, and should attempt to aesthetically “fit” the facility into the landscape.

A wet swale can be used where the water table is at or near the soil surface, or where there is a sufficient water balance in poorly drained soils to support a wetland plant community.

B General Design

Both types of enhanced swales are designed to treat the WQ_v through a volume-based design, and to safely pass larger storm flows. Flow enters the channel through a pretreatment forebay. Runoff can also enter along the sides of the channel as sheet flow through the use of a pea gravel flow spreader trench along the top of the bank.

Dry Swale

A dry swale system consists of an open conveyance channel with a filter bed of permeable soils that overlays an underdrain system. Flow passes into and is detained in the main portion of the channel where it is filtered through the soil bed. Runoff is collected and conveyed by a perforated pipe and gravel underdrain system to the outlet. Figure 3.2 provides a plan view and cross-section schematic for the design of a dry swale system.

Wet Swale

A wet swale or wetland channel consists of an open conveyance channel which has been excavated to the water table or to poorly drained soils. Check dams are used to create multiple wetland "cells," which act as miniature shallow marshes. Figure 3.3 provides a plan view and cross-section schematic for the design of a wet swale system.

C Physical Specifications / Geometry

Channel slopes between 1% and 2% are recommended unless topography necessitates a steeper slope, in which case 6- to 12-inch drop structures can be placed to limit the energy slope to within the recommended 1 to 2% range. Energy dissipation will be required below the drops. Spacing between the drops should not be closer than 50 feet. Depth of the WQ_v at the downstream end should not exceed 18 inches.

Dry and wet swales should have a bottom width of 2 to 8 feet to ensure adequate filtration. Wider channels can be designed, but should contain berms, walls, or a multi-level cross section to prevent channel braiding or uncontrolled sub-channel formation.

Dry and wet swales are parabolic or trapezoidal in cross section and are typically designed with moderate side slopes no greater than 2:1 for ease of maintenance and side inflow by sheet flow (4:1 or flatter recommended).

Dry and wet swales should maintain a maximum WQ_v ponding depth of 18 inches at the end point of the channel. A 12-inch average depth should be maintained.

The peak velocity for the 3 storm events ("Streambank Protection", "Conveyance", and flood mitigation storm) must be nonerosive for the soil and vegetative cover provided.

If the system is on-line, channels should be sized to convey runoff from a flood event safely with a minimum freeboard and without damage to adjacent property.

Dry Swale

- Dry swale channels are sized to store and infiltrate the entire water quality volume (WQ_v) with less than 18 inches of ponding and allow for full filtering through the permeable soil layer. The maximum ponding time is 48 hours, though a 24-hour ponding time is more desirable.
- The bed of the dry swale consists of a permeable soil layer of at least 30 inches in depth, above a 4-inch diameter perforated PVC pipe (AASHTO M 252) longitudinal underdrain in a 6-inch gravel layer. The soil media should have an infiltration rate of at least 1 foot per day (1.5 feet per day maximum) and contain a high level of organic material to facilitate pollutant removal. A permeable filter fabric is placed between the gravel layer and the overlying soil.

- The channel and underdrain excavation should be limited to the width and depth specified in the design. The bottom of the excavated trench shall not be loaded in a way that causes soil compaction and scarified prior to placement of gravel and permeable soil. The sides of the channel shall be trimmed of all large roots. The sidewalls shall be uniform with no voids and scarified prior to backfilling.

Wet Swale

- Wet swale channels are sized to retain the entire water quality volume (WQ_v) with less than 18 inches of ponding at the maximum depth point.
- Check dams can be used to achieve multiple wetland cells. V-notch weirs in the check dams can be utilized to direct low flow volumes.

D Pretreatment/Inlets

Inlets to enhanced swales must be provided with energy dissipaters such as riprap.

Pretreatment of runoff in both a dry and wet swale system is typically provided by a sediment forebay located at the inlet. The pretreatment volume should be equal to 0.1 inches per impervious acre. This storage is usually obtained by providing check dams at pipe inlets and/or driveway crossings.

Enhanced swale systems that receive direct concentrated runoff may have a 6-inch drop to a pea gravel diaphragm flow spreader at the upstream end of the control.

A pea gravel diaphragm and gentle side slopes should be provided along the top of channels to provide pretreatment for lateral sheet flows.

E Outlet Structures

Dry Swale

- The underdrain system should discharge to the storm drainage infrastructure or a stable outfall.

Wet Swale

- Outlet protection must be used at any discharge point from a wet swale to prevent scour and downstream erosion.

F Emergency Spillway

Enhanced swales must be adequately designed to safely pass flows that exceed the design storm flows.

G Maintenance Access

Adequate access should be provided for all dry and wet swale systems for inspection and maintenance.

H Safety Features

Ponding depths should be limited to a maximum of 18 inches.

I Landscaping

Landscape design should specify proper grass species and wetland plants based on specific site, soils, and hydric conditions present along the channel. Below is some specific guidance for dry and wet swales:

Dry Swale

- Information on appropriate turf grass species for North Central Texas can be found in *Section 1.0 of the Landscape Technical Manual.*

Wet Swale

- Emergent vegetation should be planted, or wetland soils may be spread on the swale bottom for seed stock.
- Information on establishing wetland vegetation and appropriate wetland species for North Central Texas can be found in the *Landscape Technical Manual*
- Where wet swales do not intercept the groundwater table, a water balance calculation should be performed to ensure an adequate water budget to support the specified wetland species. See *Section 4.0 of the Hydrology Technical Manual* for guidance on water balance calculations.

J Additional Site-Specific Design Criteria and Issues

Physiographic Factors - Local terrain design constraints

- Low Relief – Reduced need for use of check dams
- High Relief – Often infeasible if slopes are greater than 4%
- Karst – No infiltration of hotspot runoff from dry swales; use impermeable liner

Soils

No additional criteria

Special Downstream Watershed Considerations

- Aquifer Protection – No infiltration of hotspot runoff from dry swales; use impermeable liner

3.6 Design Procedures

Step 1 Compute runoff control volumes from the *integrated* Design Focus Areas

Calculate the Water Quality Volume (WQ_v), Streambank Protection Volume (SP_v), On-Site Flood Control Volume (V_s), and the Downstream Flood Control Volume (V_f).

Details on the *integrated* Design Focus Areas are found in *Section 1.0 of the Planning Technical Manual.*

Step 2 Determine if the development site and conditions are appropriate for the use of an enhanced swale system (dry or wet swale).

Consider the Application and Site Feasibility Criteria in *Sections 3.4 and 3.5* (Location and Siting).

Step 3 Confirm local design criteria and applicability

Consider any special site-specific design conditions/criteria from *Section 3.5* (J) (Additional Site-Specific Design Criteria and Issues).

Check with local officials and other agencies to determine if there are any additional restrictions and/or surface water or watershed requirements that may apply.

Step 4 Determine pretreatment volume

The forebay should be sized to contain 0.1 inches per impervious acre of contributing drainage. The forebay storage volume counts toward the total WQ_v requirement, and should be subtracted from the WQ_v for subsequent calculations.

Step 5 Determine swale dimensions

Size bottom width, depth, length, and slope necessary to store WQ_v with less than 18 inches of ponding at the downstream end.

- Slope cannot exceed 4% (1 to 2% recommended)
- Bottom width should range from 2 to 8 feet
- Ensure that side slopes are no greater than 2:1 (4:1 recommended)

See *Section 3.5* (C) (Physical Specifications / Geometry) for more details

Step 6 Compute number of check dams (or similar structures) required to detain WQ_v

Step 7 Calculate draw-down time

Dry swale: Planting soil should pass a maximum rate of 1.5 feet in 24 hours and must completely filter WQ_v within 48 hours.

Wet swale: Must hold the WQ_v.

Step 8 Check low flow and design event velocity erosion potential and freeboard

Check for erosive velocities and modify design as appropriate. Provide 6 inches of freeboard.

Step 9 Design low flow orifice at downstream headwalls and checkdams

Design orifice to pass WQ_v in 6 hours. Use Orifice equation.

Step 10 Design inlets, sediment forebay(s), and underdrain system (dry swale)

See *Section 3.5* (D) through (H) for more details.

Step 11 Prepare Vegetation and Landscaping Plan

A landscaping plan for a dry or wet swale should be prepared to indicate how the enhanced swale system will be stabilized and established with vegetation.

See *Section 3.5* (Landscaping) and the *Landscape Technical Manual* for more details.

See *Section 29.6* for an Enhanced Swale Design Example

3.7 Inspection and Maintenance Requirements

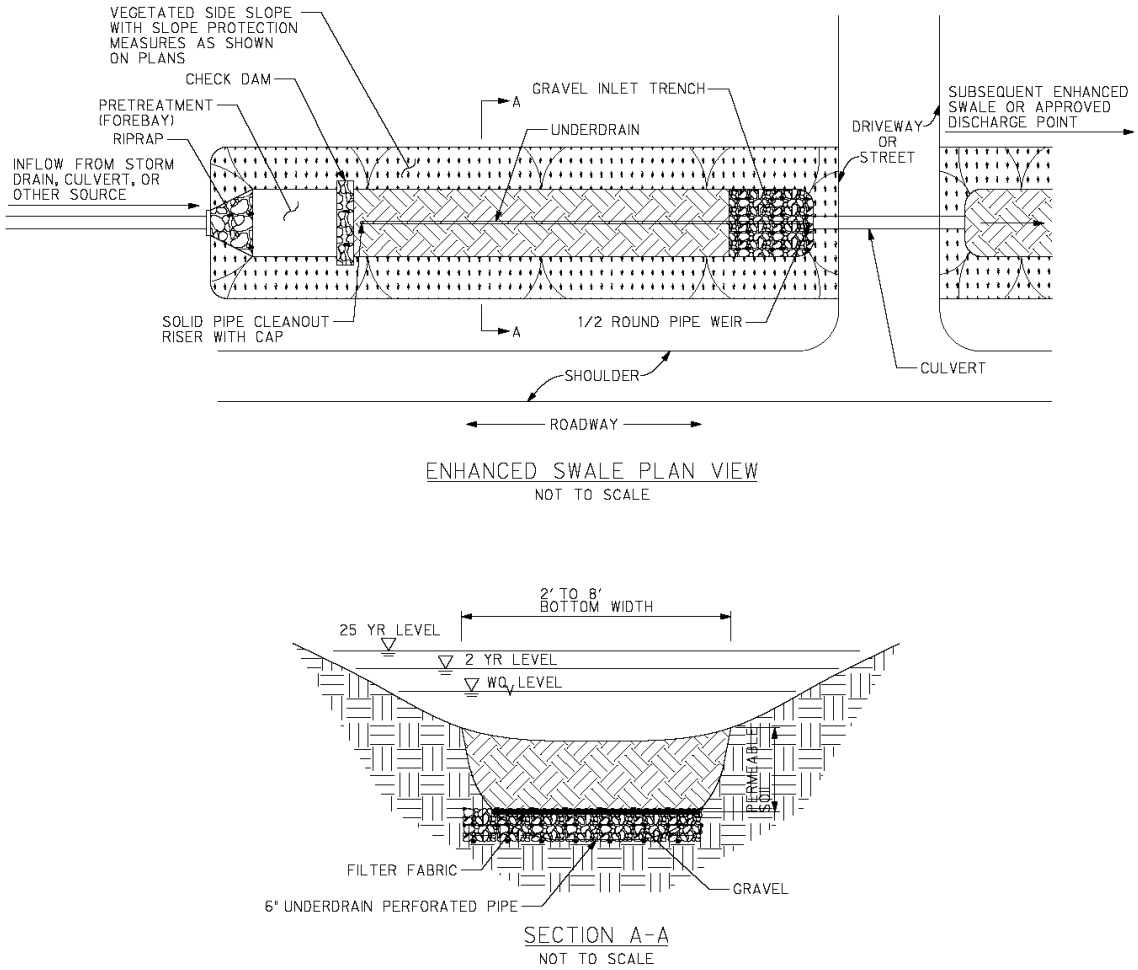
Table 3.1 Typical Maintenance Activities for Enhanced Swales
(Source: WMI, 1997; Pitt, 1997)

Activity	Schedule
• For dry swales, mow grass to maintain a height of 4 to 6 inches. Remove grass clippings.	As needed (frequent/seasonally)
• Inspect grass along side slopes for erosion and formation of rills or gullies and correct. • Remove trash and debris accumulated in the inflow forebay. • Inspect and correct erosion problems in the sand/soil bed of dry swales. • Based on inspection, plant an alternative grass species if the original grass cover has not been successfully established. • Replant wetland species (for wet swale) if not sufficiently established. • Inspect pea gravel diaphragm for clogging and correct the problem.	Annually (Semi-annually the first year)
• Roto-till or cultivate the surface of the sand/soil bed of dry swales if the swale does not draw down within 48 hours. • Remove sediment build-up within the bottom of the swale once it has accumulated to 25% of the original design volume.	As needed



Regular inspection and maintenance is critical to the effective operation of an enhanced swale system as designed. Maintenance responsibility for a dry or wet swale should be vested with a responsible authority by means of a legally binding and enforceable maintenance agreement that is executed as a condition of plan approval.

3.8 Example Schematics



NOTES:

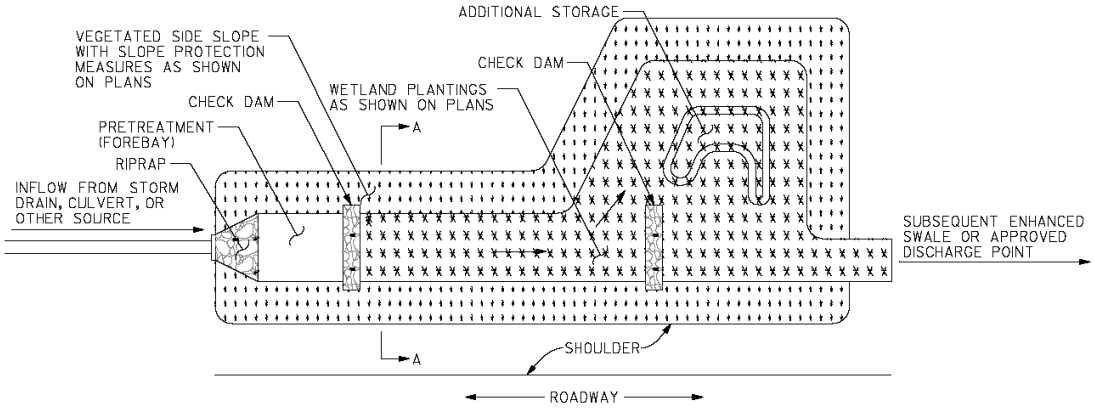
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- HORIZONTAL CONTROL
- VERTICAL CONTROL
- WATER QUALITY PROTECTION VOLUME
- STREAMBANK PROTECTION VOLUME
- FLOOD DISCHARGE
- PRETREATMENT INFORMATION
- PERMEABLE SOIL CHARACTERISTICS
- OVERFLOW PATH FOR 25-YR + EVENTS
- ROCK RIP RAP GRADATION
- CULVERT OR WEIR CALCULATIONS
- DRAINAGE AGGREGATE AND UNDERDRAIN INFORMATION

2. DESIGN ENGINEER IS RESPONSIBLE FOR PROVIDING CALCULATIONS THAT DEMONSTRATE THAT PERMEABLE MEDIA AND SYSTEM AS A WHOLE CAN EFFECTIVELY TREAT THE WATER QUALITY VOLUME (WQv) AS SPECIFIED IN SECTION 1.4 OF THE NCTCOG WATER QUALITY TECHNICAL MANUAL. FLOW IN EXCESS OF THE WQV SHALL BE DESIGNED TO BYPASS THE SYSTEM VIA THE OVERFLOW SYSTEM(S).

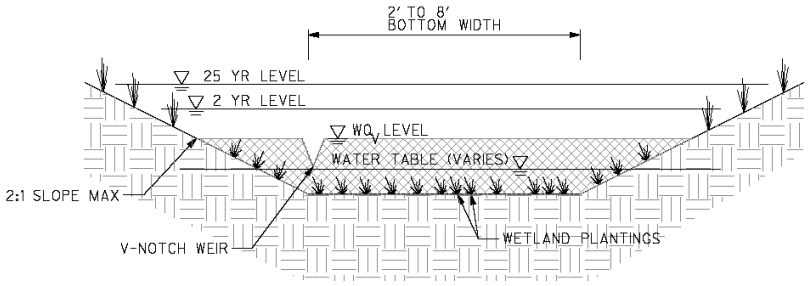
3. DESIGN OF THIS SYSTEM SHALL FOLLOW THAT OF SECTION 3 'ENHANCED SWALE' CONTAINED IN THE NCTCOG SITE DEVELOPMENT CONTROLS ISWM TECHNICAL MANUAL.

4. DESIGNER SHALL BE RESPONSIBLE FOR PREPARING AN OPERATIONS AND MAINTENANCE MANUAL OR MATRIX FOR SYSTEMS TO AID OWNER IN DETERMINING MAINTENANCE PROCEEDURES AND INTERVALS. DESIGNER SHALL ALSO CONSIDER MAINTENANCE ACCESS WHEN DESIGNING SYSTEM.

Figure 3.2 Schematic of Dry Swale
(Source: Center for Watershed Protection)



ENHANCED SWALE PLAN VIEW
NOT TO SCALE



SECTION A-A
NOT TO SCALE

NOTES:

1. RECOMMENDED WIDTHS OF ENHANCED SWALES IS BETWEEN 2 AND 8 FEET. DIMENSIONS SHALL BE PROVIDED BY DESIGNER IN PLANS AND SHALL BE BASED ON SITE SPECIFIC CONDITIONS. DESIGN PROFESSIONAL SHALL AT A MINIMUM PROVIDE THE FOLLOWING INFORMATION:
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- PRETREATMENT INFORMATION
- LANDSCAPING INFORMATION
- OVERFLOW PATH FOR 25-YR + EVENTS
- ROCK RIP RAP GRADATION
- WEIR CALCULATIONS
- DRAINAGE AGGREGATE INFORMATION

2. DESIGN ENGINEER IS RESPONSIBLE FOR PROVIDING CALCULATIONS THAT DEMONSTRATE THAT SYSTEM AS A WHOLE CAN EFFECTIVELY TREAT THE WATER QUALITY VOLUME (WQv) AS SPECIFIED IN SECTION 1.4 OF THE NCTCOG WATER QUALITY TECHNICAL MANUAL. FLOW IN EXCESS OF THE WQV SHALL BE DESIGNED TO BYPASS THE SYSTEM VIA THE OVERFLOW SYSTEM(S).

3. DESIGN OF THIS SYSTEM SHALL FOLLOW THAT OF SECTION 3 "ENHANCED SWALE" CONTAINED IN THE NCTCOG SITE DEVELOPMENT CONTROLS ISWM TECHNICAL MANUAL.

4. PLEASE NOTE THAT WET SWALE PLACEMENT SHOULD BE CAREFULLY CONSIDERED DUE TO THEIR PROPENSITY TO RETAIN STANDING WATER WHICH PROVIDES A BREEDING GROUND FOR MOSQUITOS AND OTHER DISEASE CARRYING SPECIES.

5. LANDSCAPING SPECIFIED BY DESIGNER SHOULD BE CARFULLY CHOSEN TO ENSURE SURVIVABILITY DURING BOTH SUBMERGED AND DROUGHT EXTREMES.

6. DESIGNER SHALL BE RESPONSIBLE FOR PREPARING AN OPERATIONS AND MAINTENANCE MANUAL OR MATRIX FOR SYSTEMS TO AID OWNER IN DETERMINING MAINTENANCE PROCEEDURES AND INTERVALS. DESIGNER SHALL ALSO CONSIDER MAINTENANCE ACCESS WHEN DESIGNING SYSTEM.

Figure 3.3 Schematic of Wet Swale
(Source: Center for Watershed Protection)

3.9 Design Forms

Design Procedure Form: Enhanced Swales

PRELIMINARY HYDROLOGIC CALCULATIONS

1a. Compute WQ_v volume requirements Compute Runoff Coefficient, R_v Compute WQ_v	R_v = _____ WQ_v = _____ acre-ft
1b. Compute SP_v Compute average release rate Compute Q_p (100-year detention volume required) Compute (as necessary) Q_f	SP_v = _____ acre-ft release rate = _____ cfs Q_p = _____ acre-ft Q_f = _____ cfs

ENHANCED SWALE DESIGN

2. Is the use of an enhanced swale appropriate?	***See subsections 5.2.2.4 and 5.2.2.5 - A***
3. Confirm local design criteria and applicability.	***See subsection 5.2.2.5 - J***
4. Pretreatment Volume Vol_{pre} = I (0.1")(1'/12")	Vol_{pre} = _____ acre-ft
5. Determine swale dimensions Assume trapezoidal channel with max depth of 18 inches	Length = _____ ft Width = _____ ft Side Slopes = _____ Area = _____ ft^2
6. Compute number of check dams (or similar structures) required to detain WQ_v	Slope = _____ ft/ft Depth = _____ ft Distance = _____ ft Number = _____ each
7. Calculate draw-down time Require k = 1.5 ft per day for dry swales	t = _____ hr
8. Check low flow and design storm velocity erosion potential and freeboard Requires separate computer analysis for velocity	V_{min} = _____ fps
Overflow wier (use weir equation) Use weir equation for slot length ($Q = CLH^{3/2}$)	Weir Length = _____ ft
9. Design low flow orifice at headwall Area of orifice from orifice equation $Q = CA(2gh)^{0.5}$	Area = _____ ft^2 diam = _____ inch
10. Design inlets, sediment forebays, outlet structures, maintenance access, and safety features.	***See subsection 5.2.2.5 - D through H***
11. Attach landscaping plan (including wetland vegetation)	***See Appendix F***

Notes: _____

4.0 Grass Channel

Structural Stormwater Control

North Central Texas Council of Governments

Integrated Stormwater Management iSWM



Grass Channel in Durham, NC. *(Source: Tetra Tech)*

Description

Grass channels are vegetated, open channels. They are designed to filter stormwater runoff and meet velocity targets for the water quality design storm and the streambank protection storm events. Grass channels are sometimes called biofilters. Grass channels are not the same treatment device as enhanced swales.

Design Considerations

- Grass channels require relatively low slopes, typically below 4%. Slopes are recommended at 1–2%.
- Grass channels are most effective at low flow rates (less than 1 foot per second).
- The total suspended solids (TSS) removal is not met by grass channels alone.
- A 5-minute residence time is recommended for the water quality peak flow.
- Check dams can be utilized to increase the flow residence time and increase the treatment provided.
- Most soil types can support a grass channel, although modifications may be required if impermeable soils are found at the site.
- The bottom width of the facilities can very but is typically between 2 and 6 feet.

Key Advantages

- Grass channels can provide pretreatment and be used as part of the runoff conveyance system.
- If underlying soils are pervious, grass channels can partially infiltrate runoff from small storm events.
- Grass channels are less expensive than curb and gutter systems.

Limitations

- Grass channels have higher maintenance requirements than curb and gutter systems.
- The TSS removal target cannot be achieved by grass channels alone.
- There is a potential that grass channels experience bottom erosion and a resuspension of sediment.
- Standing water may not be acceptable in some areas due to residents' concerns.

Target Constituent	Removal Rate (0% → 100%)
Total Suspended Solids	
Total Phosphorus	
Total Nitrogen	
Fecal Coliform	*insufficient data*
Heavy Metals	

Implementation Considerations



Suitability

The iSWM manual has designated grass channels as suitable for providing:



Maintenance

- Maintain grass height at 3 to 4 inches.
- Sediment accumulation should be removed from the forebay and the channel.
- Inspect for and repair erosion.
- Replant grass species if establishment does not occur.
- Trash, debris, sediment and grass clippings need to be removed.

Published October 2020

4.1 General Description

Grass channels, also termed “biofilters,” are typically designed to provide nominal treatment of runoff as well as meet runoff velocity targets for the water quality design storm. Grass channels are well suited to a number of applications and land uses, including treating runoff from roads and highways and pervious surfaces.

Grass channels differ from the enhanced dry swale design in that they do not have an engineered filter media to enhance pollutant removal capabilities and, therefore, have a lower pollutant removal rate than for a dry or wet (enhanced) swale. Grass channels can partially infiltrate runoff from small storm events in areas with pervious soils. When properly incorporated into an overall site design, grass channels can reduce impervious cover, accent the natural landscape, and provide aesthetic benefits.

When designing a grass channel, the two primary considerations are channel capacity and minimization of erosion. Runoff velocity should not exceed 1.0 foot per second during the peak discharge associated with the water quality design rainfall event, water depth should generally be less than 4 inches (height of the grass), and the total length of a grass channel should provide at least 5 minutes of residence time. To enhance water quality treatment, grass channels must have broader bottoms, lower slopes, and denser vegetation than most drainage channels. Additional treatment can be provided by placing check-dams across the channel below pipe inflows, and at various other points along the channel.

4.2 Pollutant Removal Capabilities

The following design pollutant removal rates are conservative average pollutant reduction percentages for design purposes derived from sampling data, modeling and professional judgment.

- Total Suspended Solids – 50%
- Total Phosphorus – 25%
- Total Nitrogen – 20%
- Fecal Coliform – insufficient data
- Heavy Metals – 30%

Fecal coliform removal is uncertain. In fact, grass channels are often a source of fecal coliforms from local residents walking their dogs.

4.3 Design Criteria and Specifications

Grass channels should generally be used to treat small drainage areas of less than 5 acres. If the practices are used on larger drainage areas, the flows and volumes through the channel become too large to allow for filtering and infiltration of runoff.

Grass channels should be designed on relatively flat slopes of less than 4%; channel slopes between 1% and 2% are recommended.

Grass channels can be used on most soils with some restrictions on the most impermeable soils. Grass channels should not be used on soils with infiltration rates less than 0.27 inches per hour if infiltration of small runoff flows is intended.

A grass channel should accommodate the peak flow for the water quality design storm Q_{wq} (see *Section 1.0 of the Water Quality Technical Manual*).

Grass channels should have a trapezoidal or parabolic cross section with relatively flat side slopes (generally 3:1 or flatter).

The bottom of the channel should be between 2 and 6 feet wide. The minimum width ensures an adequate filtering surface for water quality treatment, and the maximum width prevents braiding, which is the formation of small channels within the swale bottom. The bottom width is a dependent variable in the calculation of velocity based on Manning’s Equation. If a larger channel is needed, the use of a compound cross section is recommended.

Runoff velocities must be nonerosive. The full-channel design velocity will typically govern.

A 5-minute residence time is recommended for the water quality peak flow. Residence time may be increased by reducing the slope of the channel, increasing the wetted perimeter, or planting a denser grass (raising the Manning's n).

The depth from the bottom of the channel to the groundwater should be at least 2 feet to prevent a moist swale bottom, or contamination of the groundwater.

Incorporation of check dams within the channel will maximize retention time.

Designers should choose a grass that can withstand relatively high velocity flows at the entrances for both wet and dry periods. See the *Landscape Technical Manual* for a list of appropriate grasses for use in North Central Texas.

See *Section 3.2 of the Hydraulics Technical Manual* for more information and specifications on the design of grass channels.

Grass Channels for Pretreatment

A number of other structural controls, including bioretention areas and infiltration trenches, may utilize a grass channel as a pretreatment measure. The length of the grass channel depends on the drainage area, land use, and channel slope. Table 4.1 provides sizing guidance for grass channels for a 1-acre drainage area. The minimum grassed channel length should be 20 feet.

Table 4.1 Bioretention Grass Channel Sizing Guidance

Parameter	<= 33% Impervious		Between 34% and 66% Impervious		>= 67% Impervious	
Slope (max = 4%)	< 2%	> 2%	< 2%	> 2%	< 2%	> 2%
Grass channel minimum length* (feet) *assumes 2-foot wide bottom width	25	40	30	45	35	50

(Source: Claytor and Schueler, 1996)

4.4 Inspection and Maintenance Requirements

Table 4.2 Typical Maintenance Activities for Grass Channels

Activity	Schedule
• Mow grass to maintain a height of 3 to 4 inches.	As needed (frequently/seasonally)
• Remove sediment build-up within the bottom of the grass channel once it has accumulated to 25% of the original design volume.	As needed (Infrequently)
• Inspect grass along side slopes for erosion and formation of rills or gullies and correct. • Remove trash and debris accumulated in the channel. • Based on inspection, plant an alternative grass species if the original grass cover has not been successfully established.	Annually (Semi-annually the first year)

(Source: Adapted from CWP, 1996)

4.5 Example Schematics

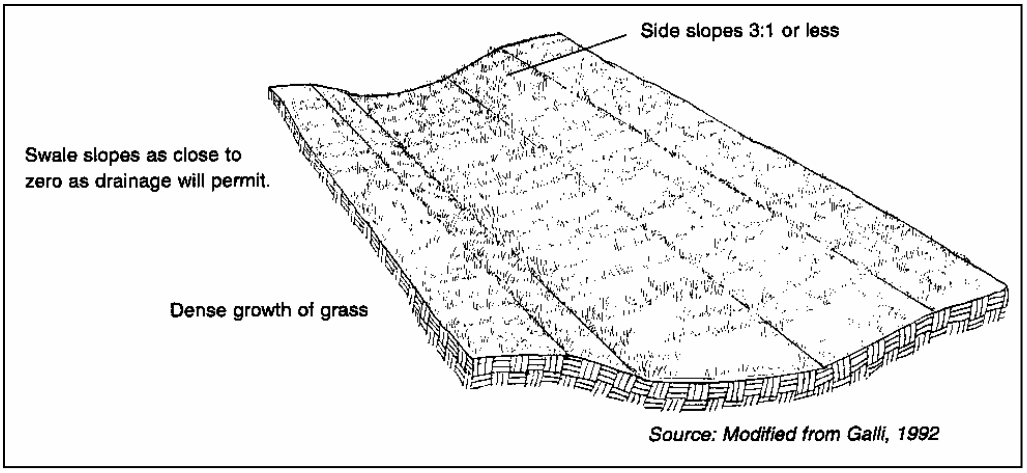


Figure 4.1 Typical Grass Channel

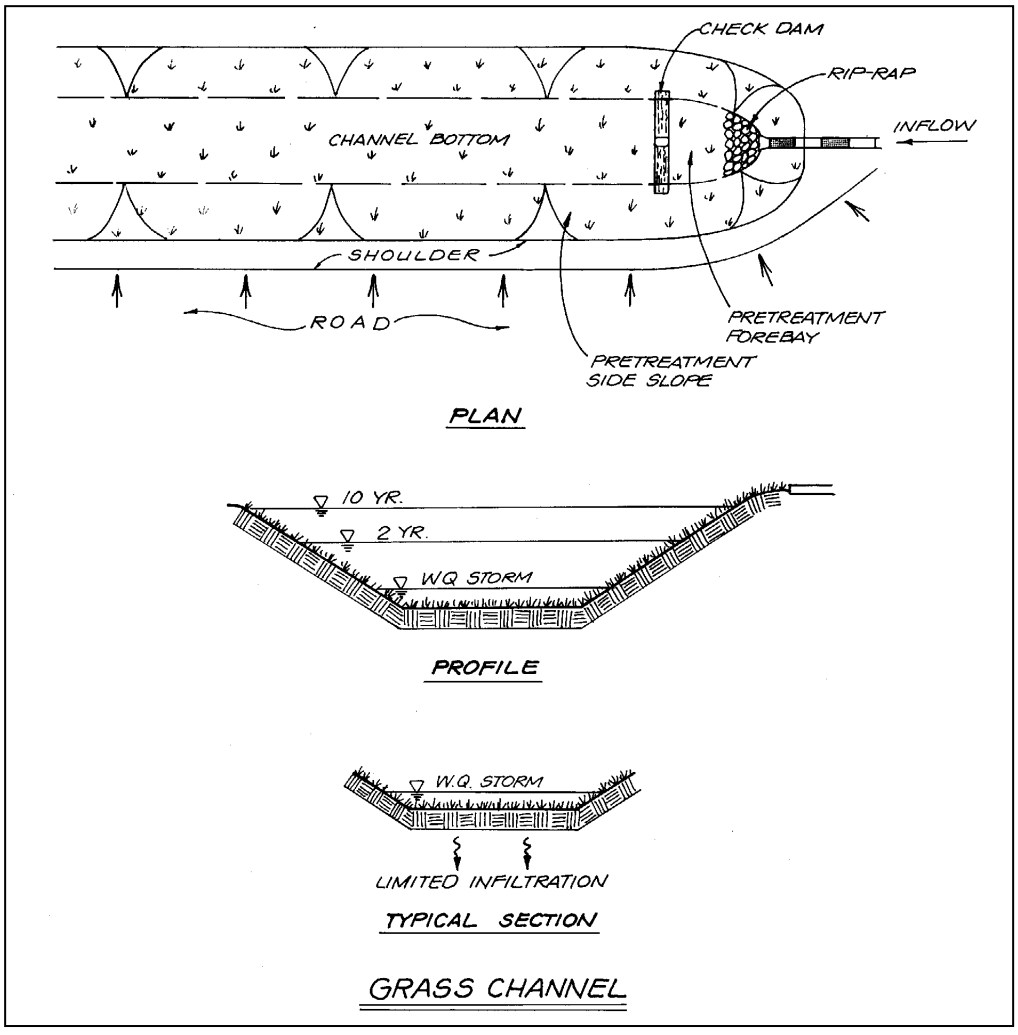


Figure 4.2 Schematic of Grass Channel

4.6 Design Example

Basic Data

Small commercial lot 300 feet deep x 145 feet wide

- Drainage area (A) = 1.0 acres
- Impervious percentage (I) = 70%

Water Quality Peak Flow

See Section 1.0 of the Water Quality Technical Manual for details

Compute the Water Quality Protection Volume in inches:

$WQ_v = 1.5 (0.05 + 0.009 * 70) = 1.02$ inches

Compute modified CN for 1.5-inch rainfall (P=1.5):

$CN = 1000/[10+5P+10Q-10(Q^2+1.25*Q*P)^{½}]$
$= 1000/[10+5*1.5+10*0.82-10(0.82^2+1.25*0.82*1.5)^{½}]$
$= 92.4$ (Use CN = 92)

For CN = 92 and an estimated time of concentration (T_c) of 8 minutes (0.13 hours), compute the Q_{wq} for a 1.5-inch storm.

From *Table 1.11 of the Hydrology Technical Manual*, $I_a = 0.174$, therefore $I_a/P = 0.174/1.5 = 0.116$.

From *Figure 1.10 of the Hydrology Technical Manual* for a Type II storm (using the limiting values) $q_u = 950$ csm/in, and therefore:

$Q_{wq} = (950 \text{ csm/in}) (1.0\text{ac}/640\text{ac/mi}^2) (1.02") = 1.51$ cfs

Utilize Q_{wq} to Size the Channel

The maximum flow depth for water quality treatment should be approximately the same height of the grass. A maximum flow depth of 4 inches is allowed for water quality design. A maximum flow velocity of 1.0 foot per second for water quality treatment is required. For Manning's n use 0.15 for medium grass, 0.25 for dense grass, and 0.35 for very dense Bermuda-type grass. Site slope is 2%.

Input variables:

n = 0.15
S = 0.02 ft/ft
D = 4/12 = 0.33 ft

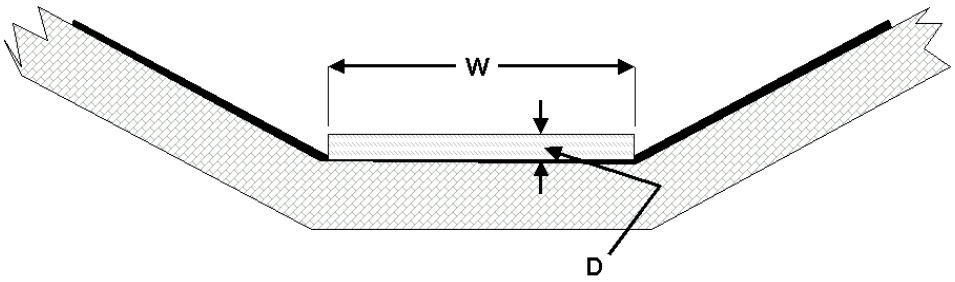
Then:

$Q_{wq} = Q = VA = 1.49/n\ D^{2/3}\ S^{1/2}\ DW$

where:

Q = peak flow (cfs)
V = velocity (ft/sec)
A = flow area (ft^2) = WD
W = channel bottom width (ft)
D = flow depth (ft)
S = slope (ft/ft)

(Note: D approximates hydraulic radius for shallow flows)



Then for a known n, Q, D and S minimum width can be calculated.

$(nQ)/(1.49\ D^{5/3}\ S^{1/2}) = W = (0.15*1.51)/(1.49*0.33^{5/3}*0.02^{1/2}) = 6.84$ feet minimum

$V = Q/(WD) = 1.51/(6.84 * 4/12) = 0.66$ fps (okay)

(Note: WD approximates flow area for shallow flows.)

Minimum length for 5-minute residence time, $L = V * (5*60) = 198$ feet

Depending on the site geometry, the width or slope or density of grass (Manning's n value) might be adjusted to slow the velocity and shorten the channel in the next design iteration. For example, using a 10-foot bottom width* of flow and a Manning's n of 0.20, solve for new depth and length.

$Q = VA = 1.49/n\ D^{5/3}\ S^{1/2}\ W$

$D = [(Q * n)/(1.49 * S^{1/2} * W)]^{3/5}$

$= [(1.51 * 0.20)/(1.49 * 0.02^{1/2} * 10.0)]^{3/5} = 0.31$ ft = 4" (okay)

$V = Q/WD = 1.51/(10.0 * 0.31) = 0.49$ feet per second

$L = V * 5 * 60 = 146$ feet

** In this case a dividing berm should be used to control potential braiding.*

Refer to *Section 3.2 of the Hydraulics Technical Manual* to complete the grass channel design for a specified design storm event.

5.0 Open Conveyance Channel

Stormwater Control



Description: An open channel is a conduit in which water flows with a free surface. Open channels include conveyance channels or drainage ditches; grass channels; and enhanced swales. An open conveyance channel is designed for conveyance purposes only.

KEY CONSIDERATIONS

DESIGN CRITERIA:

- A maximum of 2:1 should be used for channel side slopes. Roadside ditches should have maximum side slope of 3:1.
- Slope stability should be confirmed with a geotechnical study or investigation
- Channel banks should be stabilized at site

ADVANTAGES / BENEFITS:

- Can be aesthetically pleasing
- Vegetated channels provide natural habitats
- Once established, little maintenance is required

DISADVANTAGES / LIMITATIONS:

- Velocity will limit the type of channel lining, for example, vegetated channels require slower velocities and lower longitudinal slopes

STORMWATER MANAGEMENT SUITABILITY

	Water Quality Protection
	Streambank Protection
P	**On-Site Flood Control**
S	**Downstream Flood Control**

IMPLEMENTATION CONSIDERATIONS

H	**Land Requirement**
L	**Capital Cost**
L	**Maintenance Burden**

Residential Subdivision Use: Yes
High Density/Ultra-Urban: No
Drainage Area: 5 acres max.
Soils: No restrictions

L=Low **M**=Moderate **H**=High

5.1 General Description

An open channel is a conduit in which water flows with a free surface. Open channel systems and their design are an integral part of stormwater drainage design, particularly for development sites utilizing better sited design practices and open channel structural controls. The broad category of open channels includes conveyance channels or drainage ditches, grass channels, and dry and wet enhanced swales. Grass channels and enhanced swales are designed to provide water quality benefits and are further described in detail in *Sections 4.0* and *Section 3.0*, respectively.

Channel Classifications

Open channels may be classified into three main categories according to the type of channel linings: vegetated, flexible, and rigid. Vegetated linings include grass with mulch, sod and lapped sod, and wetland channels. Flexible linings include stone riprap and some forms of flexible man-made linings or gabions. Rigid linings are generally concrete or rigid block.



Figure 5.1 Open Channel Examples

5.2 Pollutant Removal Capabilities

Open conveyance channels or drainage ditches are designed for conveyance purposes only. For open channels with pollutant capabilities, refer to *Section 4.0* and *Section 3.0*, respectively.

5.3 Design Criteria and Specifications

Detailed design criteria and specifications, as prepared by the Federal Highway Administration, are presented in *Section 3.2 of the Hydraulics Technical Manual*. Uniform flow, critical flow, and design details for the three main categories of channel classification (vegetative, riprap, and rigid lining) are also included in the noted section.

In general, the following criteria should be followed for open channel design:

- Channels with bottom widths greater than 10 feet shall be designed with a minimum bottom cross slope of 12 to 1, or with compound cross sections.
- Channel side slopes shall be stable throughout the entire length and side slope shall depend on the channel material. A maximum of 2:1 should be used for channel side slopes, unless otherwise justified by calculations. Roadside ditches should have a maximum side slope of 3:1. All side slopes should be verified with a geotechnical evaluation to ensure slope stability.
- Trapezoidal or parabolic cross sections are preferred over triangular shapes.
- If relocation of a stream channel is unavoidable, the cross-sectional shape, meander, pattern, roughness, sediment transport, and slope should conform to the existing conditions insofar as practicable. Some means of energy dissipation may be necessary when existing conditions cannot be duplicated.

- Streambank stabilization should be provided, when appropriate, as a result of any stream disturbance such as encroachment and should include both upstream and downstream banks as well as the local site.
- Open channel drainage systems are sized to adequately convey the “Conveyance” design storm, and are normally checked with the flood mitigation storm event.

5.4 Inspection and Maintenance Requirements

Open channels should be inspected after large storm events for debris causing blockages or re-routing. Channels with vegetated linings should be inspected and maintained periodically to insure vegetation is still in place and prevent growth of taller or woody vegetation. Flexible linings, such as rock riprap, have self-healing qualities that reduce maintenance. However, they should be inspected and maintained periodically to prevent growth of trees, grass, and weeds. Concrete channels should be checked periodically for scour at the channel lining transitions and channel headcutting.

6.0 Alum Treatment System

Structural Stormwater Control



Description: Chemical treatment of stormwater runoff entering a wet pond by injecting liquid alum into storm sewer lines on a flow-weighted basis during rain events.

KEY CONSIDERATIONS

ADVANTAGES / BENEFITS:

- Requires no additional land purchase
- Reduces concentrations of total phosphorus, total aluminum and heavy metals
- Dependent on pH level ranging from 6.0 to 7.5 during treatment process

DISADVANTAGES / LIMITATIONS:

- Intended for areas requiring regional stormwater treatment from a piped stormwater drainage system
- High maintenance requirements
- Alum application will lower pH of receiving waters
- High capital and operations and maintenance costs

POLLUTANT REMOVAL

90%	**Total Suspended Solids**
80/60%	**Nutrients** - Total Phosphorus / Total Nitrogen removal
75%	**Metals** - Cadmium, Copper, Lead, and Zinc removal
90%	**Pathogens** - Coliform, Streptococci, E.Coli removal

STORMWATER MANAGEMENT SUITABILITY

P	**Water Quality Protection**
	Streambank Protection
	On-Site Flood Control
	Downstream Flood Control

IMPLEMENTATION CONSIDERATIONS

L	**Land Requirement**
H	**Capital Cost**
H	**Maintenance Burden**

Residential Subdivision Use: Yes
High Density/Ultra-Urban: Yes
Drainage Area: 25 acres min.
Soils: No restrictions
Other Considerations:

- Regional Treatment

L=Low **M**=Moderate **H**=High

6.1 General Description

The process of alum (aluminum sulfate) treatment provides treatment of stormwater runoff from a piped stormwater drainage system entering a wet pond by injecting liquid alum into storm sewer lines on a flow-weighted basis during rain events. When added to runoff, liquid alum forms nontoxic precipitates of aluminum hydroxide [$Al(OH)_3$] and aluminum phosphate [$AlPO_4$]. However, Alum will lower the pH of receiving waters and must be closely monitored to avoid adverse impacts on aquatic life.

The alum precipitate or "floc" formed during coagulation of stormwater combines with phosphorus, suspended solids, and heavy metals and removes them from the water column. The floc can be allowed to settle in receiving water or collected in small settling basins. Once settled, the floc is stable in sediments and will not re-dissolve due to changes in redox potential or pH under conditions normally found in surface water bodies. Laboratory or field testing may be necessary to verify feasibility and to establish design, maintenance, and operational parameters, such as the optimum coagulant dose required to achieve the desired water quality goals, chemical pumping rates and pump sizes.

Construction costs for existing alum stormwater treatment facilities in Florida have ranged from $135,000 to $400,000. The capital construction costs of alum stormwater treatment systems is independent of watershed size and depends primarily on the number of outfall locations treated.

Estimated annual operations and maintenance (O&M) costs for chemicals and routine inspections range from approximately $6,500 to $25,000 per year. O&M costs include chemical, power, manpower for routine inspections, equipment renewal, and replacement costs.

Ferric chloride has also been used for flow-proportional injection for removing phosphorus and other pollutants. Although ferric chloride is less toxic to aquatic life than alum, it has a number of significant disadvantages. Ferric chloride dosage rates are dependent on the pollutant concentrations in the stormwater runoff, unlike alum. Ferric chloride does not form a floc that settles out suspended pollutants. And, once settled, ferric chloride may be released from sediments under anoxic conditions.

6.2 Pollutant Removal Capabilities

Alum treatment has consistently achieved a 85 to 95% reduction in total phosphorus, 90 to 95% reduction in orthophosphorus, 60 to 70% reduction in total nitrogen, 50 to 90% reduction in heavy metals, 95 to 99% reduction in turbidity and TSS, 60% reduction in BOD, and >99% reduction in fecal coliform bacteria compared with raw stormwater characteristics.

The following design pollutant removal rates are conservative average pollutant reduction percentages for design purposes derived from sampling data, modeling, and professional judgment.

- Total Suspended Solids – 90%
- Total Phosphorus – 80%
- Total Nitrogen – 60%
- Fecal Coliform – 90%
- Heavy Metals – 75%

6.3 Design Criteria and Specifications

Alum treatment systems are fairly complex, and design details are beyond the scope of this Manual. However, further information can be obtained from the Internet and by contacting local municipalities and engineers who have designed and implemented successful systems. The following are general guidelines for alum treatment systems:

- Injection points should be 100 feet upstream of discharge points.
- Alum concentration is typically 10 μg/l.
- Alum treatment systems may need to control pH.

- For new pond design, the required size is approximately 1% of the drainage basin size, as opposed to 10 to 15% of the drainage basin area for a standard detention pond.
- No additional volume is required when discharging to existing lakes.

6.4 Inspection and Maintenance Requirements

Table 6.1 Typical Maintenance Activities for Alum Treatment

Activity	Schedule
• Perform routine inspection. • Monitor water quality and pH of receiving water.	Monthly
• Perform maintenance of pump equipment, chemical supplies, and delivery system.	As Needed

(Source: Harper, Herr, and Livingston)





Figure 6.1 Alum Treatment System and Injection Equipment

7.0 Culverts

Stormwater Control



Description: A short, closed (covered) conduit that conveys stormwater runoff under an embankment, usually a roadway.

KEY CONSIDERATIONS

DESIGN CRITERIA:

- Designed for conveyance purposes, not pollutant removal capability
- Normally designed for the "Conveyance" storm event

VELOCITY:

- Maximum velocity of 15 fps for corrugated metal pipe
- Minimum velocity of 2.5 fps, for the "Streambank Protection" storm event

SLOPE:

- Maximum slope of 14% for corrugated metal pipe
- Maximum slope of 10% for concrete pipe
- Maximum drop in a drainage structure is 10 feet.

OTHER:

- Skew not to exceed 45 degrees
- Minimum diameter of 18 inches

MAINTENANCE REQUIREMENTS:

- Reinforced concrete pipe for use (1) under roadway, (2) when pipe slopes are less than 1%, or (3) for all flowing streams.
- RCP and fully coated corrugated metal pipe
- High-density polyethylene (HDPE) may be used as specified in municipal regulations

STORMWATER MANAGEMENT SUITABILITY

	Water Quality Protection
	Streambank Protection
P	**On-Site Flood Control**
P	**Downstream Flood Control**

IMPLEMENTATION CONSIDERATIONS

L	**Land Requirement**
L	**Capital Cost**
L	**Maintenance Burden**

Residential Subdivision Use: Yes
High Density/Ultra-Urban: Yes
Drainage Area: No restrictions.
Soils: No restrictions

L=Low **M**=Moderate **H**=High

7.1 General Description

A culvert is a short, closed (covered) conduit that conveys stormwater runoff under an embankment, usually a roadway. The primary purpose of a culvert is to convey surface water, but properly designed it may also be used to restrict flow and reduce downstream peak flows. In addition to the hydraulic function, a culvert must also support the embankment and/or roadway, and protect traffic and adjacent property owners from flood hazards to the extent practicable.

7.2 Pollutant Removal Capabilities

Culverts are designed for stormwater conveyance purposes and do not provide pollutant removal capabilities.

7.3 Design Criteria and Specifications

The design of a culvert should take into account many different engineering and technical aspects at the culvert site and adjacent areas. The following design criteria should be considered for all culvert designs as applicable:

- Frequency Flood;
- Velocity Limitations;
- Buoyancy Protection;
- Length and Slope;
- Debris Control;
- Headwater Limitations;
- Tailwater Considerations;
- Storage;
- Inlets;
- Inlets with Headwalls;
- Wingwalls and Aprons;
- Improved Inlets;
- Material Selection;
- Culvert Skews;
- Culvert Sizes;
- Weep Holes;
- Outlet Protection;
- Erosion and Sediment Control; and
- Environmental Considerations.

There are two types of flow conditions for culverts (see Figure 7.1) that are based upon the location of the control section and the critical flow depth:

- Inlet Control – Inlet control occurs when the culvert barrel is capable of conveying more flow than the inlet will accept. This typically happens when a culvert is operating on a steep slope. The control section of a culvert is located just inside the entrance. Critical depth occurs at or near this location, and the flow regime immediately downstream is supercritical.
- Outlet Control – Outlet control flow occurs when the culvert barrel is not capable of conveying as much flow as the inlet opening will accept. The control section for outlet control flow in a culvert is located at the barrel exit or further downstream. Either subcritical or pressure flow exists in the culvert barrel under these conditions.

Proper culvert design and analysis requires checking for both inlet and outlet control to determine which will govern particular culvert designs.

There are three procedures for designing culverts: manual use of inlet and outlet control equations, nomographs, and the use of computer programs such as HY8. It is recommended that the HY8 computer model or equivalent be used for culvert design. The computer software package HYDRAIN, which includes HY8, uses the theoretical basis from the nomographs to size culverts. In addition, this software can evaluate improved inlets, route hydrographs, consider road overtopping, and evaluate outlet streambed scour. By using water surface profiles, this procedure is more accurate in predicting backwater effects and outlet scour.

Examples of small culverts are shown in Figure 7.2. See *Section 3.3 of the Hydraulics Technical Manual* for detailed culvert design procedures and instruction.

7.4 Inspection and Maintenance Requirements

Culverts located at the end of urban drainage channels are often clogged by refuse dumped into the channel or by trash washed off the city streets. Under such conditions, a debris rack can usually be installed at a low cost to prevent clogging. In designing debris control structures it is recommended that the Federal Highway Administration, Hydraulic Engineering Circular No. 9 entitled *Debris Control Structures* be consulted. This Circular discusses the variety of methods for controlling debris by: (a) intercepting the debris at or above the inlet; (b) deflecting the debris for detention near the inlet; or (c) passing the debris through the structure.

Additionally, to ensure self-cleaning during partial depth flow, a minimum velocity of 2.5 feet per second, for the 2-year flow, when the culvert is flowing partially full is required.

7.5 Example Schematic

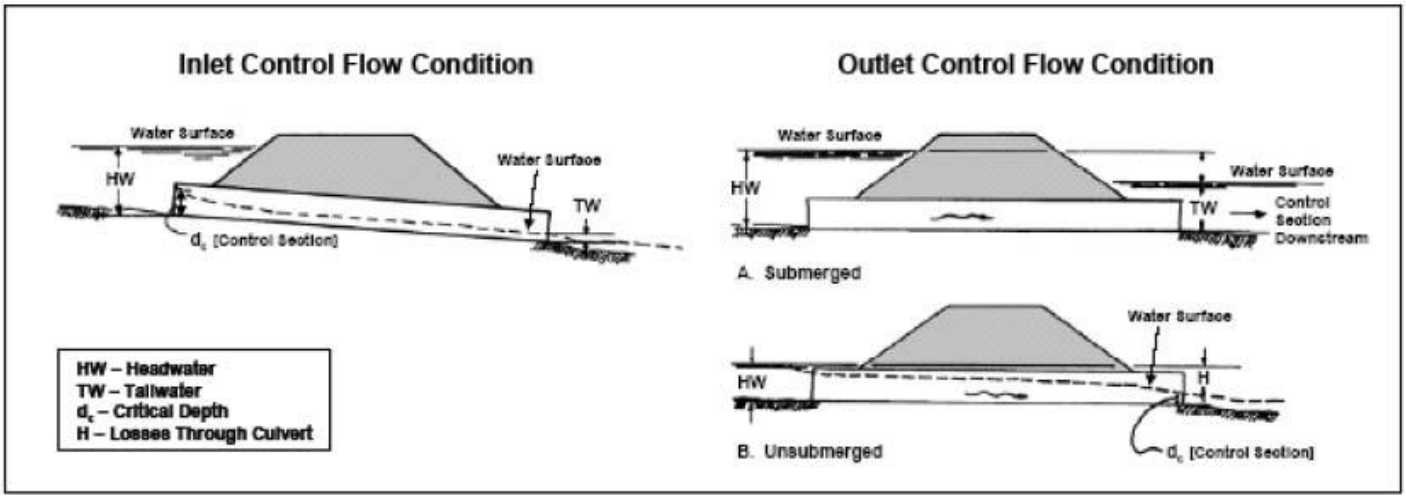


Figure 7.1 Culvert Flow Conditions



Figure 7.2 Culvert Examples

8.0 Inlets

Stormwater Controls



Description: Drainage structure used to collect surface water through grate or curb openings and convey it to pipe systems or direct outlet to culverts.

KEY CONSIDERATIONS

DESIGN CRITERIA:

- Designed for conveyance purposes, not pollutant removal capability
- Interception capacity depends on depth of water next to curb.

ADVANTAGES / BENEFITS:

- Aesthetically pleasing, less obvious than most stormwater structures.

DISADVANTAGES / LIMITATIONS:

- Efficiency is typically reduced in order to meet safety standards.

MAINTENANCE REQUIREMENTS:

- Must be inspected and cleaned regularly due to trash and debris build-up.

STORMWATER MANAGEMENT SUITABILITY

☐	**Water Quality Protection**
☐	**Streambank Protection**
P	**On-Site Flood Control**
☐	**Downstream Flood Control**

IMPLEMENTATION CONSIDERATIONS

L	**Land Requirement**
L	**Capital Cost**
L	**Maintenance Burden**

Residential Subdivision Use: Yes

High Density/Ultra-Urban: Yes

Drainage Area: No restrictions.

Soils: No restrictions

Other Considerations: Overflow parking, driveways and related issues

L=Low **M**=Moderate **H**=High

8.1 General Description

Inlets are drainage structures used to collect surface water through grate or curb openings and convey it to pipe systems or direct outlet to culverts. Inlets are typically located in close proximity to impervious areas such as streets and parking lots. Inlets can also be located at a low point in a channel or area that concentrates overland flow (ponding areas).

8.2 Pollutant Removal Capabilities

Although inlets prevent large debris from passing to the storm sewer system, they are designed for stormwater conveyance purposes and do not provide pollutant removal capabilities.

8.3 Design Criteria and Specifications

Inlets used for drainage of surfaces can be divided into three major classes:

- Grate Inlets – These inlets include grate inlets, consisting of an opening in the gutter covered by one or more grates, and slotted inlets, consisting of a pipe cut along the longitudinal axis with a grate or spacer bars to form slot openings.
- Curb-Opening Inlets – These inlets are vertical openings in the curb covered by a top slab.
- Combination Inlets – These inlets usually consist of both a curb-opening inlet and a grate inlet placed in a side-by-side configuration, but the curb opening may be located in part upstream of the grate.

Inlets may be classified as on a *continuous grade* or in a *sump*. The term “continuous grade” refers to an inlet located on the street with a continuous slope past the inlet with water entering from one direction. The “sump” condition exists when the inlet is located at a low point and water enters from both directions. There are specific design criteria for the following types of inlets:

- Grate Inlets on Grade
- Grate Inlets in Sag
- Curb Inlets on Grade
- Curb Inlets in Sump
- Combination Inlets on Grade
- Combination Inlets in Sump

Where significant ponding can occur, in locations such as underpasses and in sag vertical curves in depressed sections, it is good engineering practice to place flanking inlets on each side of the inlet at the low point in the sag. The flanking inlets should be placed so that they will limit spread on low gradient approaches to the level point and act in relief of the inlet at the low point if it should become clogged or if the design spread is exceeded.

When designing an inlet, the grate length, bar configuration, and gutter velocity must be taken into account. Inlet location should not compromise safety or aesthetics. It should not allow for standing water in areas of vehicular or pedestrian traffic, and should take advantage of natural depression storage where possible. Grate inlets subject to traffic should be bicycle safe (horizontal and vertical cross-bars) and provide adequate load-bearing capabilities.

See *Section 3.3 of the Hydraulics Technical Manual* for detailed inlet design procedures and instruction.

8.4 Inspection and Maintenance Requirements

Inlets are often blocked by trash washed off the city streets and parking lots. Regular inspection and removal of this debris will result in cleaner, more efficient stormwater conveyance systems. The public might also misunderstand inlets as places to dispose of household wastes. To prevent this and to promote water quality education, some municipalities and non-profit organizations have begun "stamping" inlets or attaching decals with "No Dumping" instructions.



Grate Inlet in Parking Lot



Slotted Inlet



Curb Inlet



Stamp on Inlet



Combination Inlet

Figure 8.1 Example Inlets

9.0 Pipe Systems

Stormwater Control



Description: Pipe conveyances used for transporting runoff from roadway and other inlets to outfalls at structural stormwater controls and receiving waters.

KEY CONSIDERATIONS

DESIGN CRITERIA:

- Designed for conveyance purposes, not pollutant removal capability
- All systems should be designed with velocities greater than 2.5 fps with a minimum slope of 0.5%
- Size system under the assumption of full flow, but not pressure flow
- Hydraulic gradient should not produce velocity in excess of 15 fps
- Manning's Equation recommended for capacity computations

STORMWATER MANAGEMENT SUITABILITY

	Water Quality Protection
P	**Streambank Protection**
P	**On-Site Flood Control**
P	**Downstream Flood Control**

IMPLEMENTATION CONSIDERATIONS

L	**Land Requirement**
L	**Capital Cost**
L	**Maintenance Burden**

Residential Subdivision Use: Yes
High Density/Ultra-Urban: Yes
Drainage Area: No restrictions.
Soils: No restrictions

L=Low **M**=Moderate **H**=High

9.1 General Description

Storm drain pipe systems, also known as *storm sewers*, are pipe conveyances used in the minor stormwater drainage system for transporting runoff from roadway and other inlets to outfalls at structural stormwater controls and receiving waters. Pipe drain systems are suitable mainly for medium to high-density residential and commercial/industrial development where the use of natural drainageways and/or vegetated open channels is not feasible.

9.2 Pollutant Removal Capabilities

The stormwater pipe system is designed for conveyance purposes and does not provide pollutant removal capabilities.

9.3 Design Criteria and Specifications

The design of storm drain systems generally follows these steps:

Step 1 Determine inlet location and spacing.

Step 2 Prepare a tentative plan layout of the storm sewer drainage system including:

a. Location of storm drains
b. Direction of flow
c. Location of manholes
d. Location of existing facilities such as water, gas, or underground cables

Step 3 Determine drainage areas and compute runoff using the Rational Method

Step 4 After the tentative locations of inlets, drain pipes, and outfalls (including tailwaters) have been determined and the inlets sized, compute the rate of discharge to be carried by each storm drain pipe and determine the size and gradient of pipe required to care for this discharge. This is done by proceeding in steps from upstream of a line to downstream to the point at which the line connects with other lines or the outfall, whichever is applicable. The discharge for a run is calculated, the pipe serving that discharge is sized, and the process is repeated for the next run downstream. The storm drain system design computation form (*Figure 1.27 of the Hydraulics Technical Manual*) can be used to summarize hydrologic, hydraulic, and design computations.

Step 5 Examine assumptions to determine if any adjustments are needed to the final design.

It should be recognized that the rate of discharge to be carried by any particular section of storm drain pipe is not necessarily the sum of the inlet design discharge rates of all inlets above that section of pipe, but as a general rule is somewhat less than this total. It is useful to understand that the time of concentration is most influential and as the time of concentration grows larger, the proper rainfall intensity to be used in the design grows smaller.

See *Section 1.2 of the Hydraulics Technical Manual* for detailed pipe system design procedures and instruction.

9.4 Inspection and Maintenance Requirements

Maintaining stormwater conveyance structures on a regular basis will prevent clogging of the downstream conveyance system and ensure the system functions properly hydraulically to avoid flooding.

Storm Drain Conveyance System

- Locate reaches of storm drain with deposit problems and develop a flushing schedule that keeps the pipe clear of excessive buildup.
- Collect flushed effluent and pump to the sanitary sewer for treatment after sediment removal, if necessary.

Illicit Connections and Discharges

- During routine maintenance of conveyance system and drainage structures field staff should look for evidence of illegal discharges or illicit connections:
  - Is there evidence of spills such as paints, discoloring, etc.
  - Are there any odors associated with the drainage system
  - Record locations of apparent illegal discharges/illicit connections
  - Track flows back to potential dischargers and conduct aboveground inspections. This can be done through visual inspection of up gradient manholes or alternate techniques including zinc chloride smoke testing, fluorometric dye testing, physical inspection testing, or television camera inspection.
  - Once the origin of flow is established, require illicit discharger to eliminate the discharge.
- Stencil storm drains, where applicable, to prevent illegal disposal of pollutants. Storm drain inlets should have messages such as “Dump No Waste Drains to Stream” stenciled next to them to warn against uninformed or intentional dumping of pollutants into the storm drainage system.

Illegal Dumping

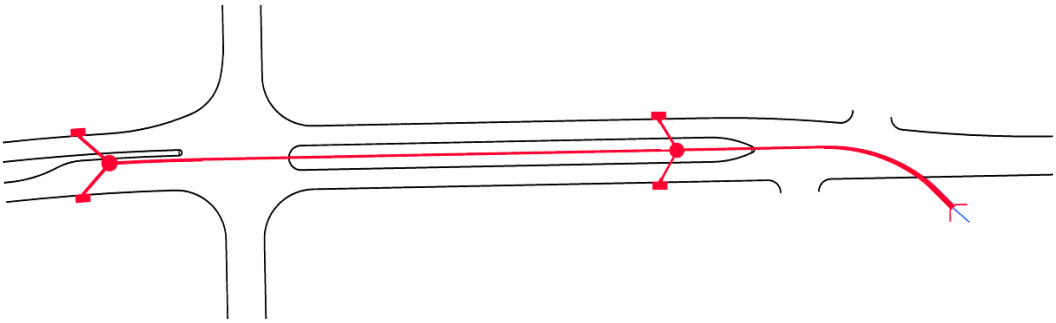
- Regularly inspect and clean up hot spots and other storm drainage areas where illegal dumping and disposal occurs.
- Establish a system for tracking incidents. The system should be designed to identify the following:
  - Illegal dumping hot spots
  - Types and quantities (in some cases) of wastes
  - Patterns in time of occurrence (time of day/night, month, or year)
  - Mode of dumping (abandoned containers, “midnight dumping” from moving vehicles, direct dumping of materials, accidents/spills)
  - Responsible parties
- Post “No Dumping” signs in problem areas with a phone number for reporting dumping and disposal. Signs should also indicate fines and penalties for illegal dumping.

Storm drain flushing is most effective in small diameter pipes (36-inch diameter pipe or less, depending on water supply and sediment collection capacity). Other considerations associated with storm drain flushing may include the availability of a water source, finding a downstream area to collect sediments, liquid/sediment disposal, and disposal of flushed effluent to sanitary sewer may be prohibited in some areas.

Maintenance

- Identifying illicit discharges requires teams of at least two people (volunteers can be used), plus administrative personnel, depending on the complexity of the storm sewer system.
- Arrangements must be made for proper disposal of collected wastes.
- Requires technical staff to detect and investigate illegal dumping violations, and to coordinate public education.

9.5 Example Schematic



10.0 Dry Detention / Extended Detention Dry Basins

Detention
Structural Stormwater Control

North Central Texas Council of Governments
integrated Stormwater Management iSWM



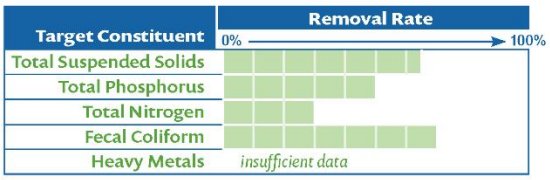
Dry Detention Facility in San Antonio, TX. *Source: Halff*

Description

Dry detention ponds are surface storage facilities that provide detention of stormwater runoff to reduce downstream water quality impacts. They temporarily detain stormwater and gradually release it following storm events. In between storm events, the facilities are typically dry.

Design Considerations

- Dry detention ponds are designed for the maximum reduction of peak flows and runoff reduction for larger storm events
- There are no restrictions for drainage area size
- Soil groups 'A' and 'B' may require a pond liner
- Often used as part of a treatment train to meet water quality requirements



Implementation Considerations



Suitability

The iSWM manual has designated that dry detention ponds are suitable for providing:



Key Advantages

- Since less excavation is required, dry detention ponds are typically less costly than wet ponds for equivalent flood storage
- Dry detention ponds are often used in conjunction with water quality structural control
- In between storm events, there are opportunities for the facility to be used for recreational activities

Limitations

- Extended detention may provide limited water quality treatment and streambank protection
- The area required for dry detention ponds is greater than the area required for other best management practices

Maintenance

- Trash, leaf, debris and sediment removal
- Provide removal of vegetation and weeds when overgrowth occurs
- Plant seed or sod in bare or dead spots
- Mow planted vegetation
- Clean inlets

Published April 2020

10.1 General Description

Dry detention and dry extended detention (ED) basins are surface facilities intended to provide for the temporary storage of stormwater runoff to reduce downstream water quantity impacts. These facilities temporarily detain stormwater runoff, releasing the flow over a period of time. They are designed to completely drain following a storm event and are normally dry between rain events.

Dry detention basins are intended to provide on-site flood control (peak flow reduction) and can be designed to control the extreme flood (flood mitigation storm) event. Extended detention dry basins provide downstream streambank protection through extended detention of the streambank protection volume (SP_v), flood control.

Both dry detention and extended detention dry basins provide limited pollutant removal benefits and are not intended for water quality treatment. Detention-only facilities must be used in a treatment train approach with other structural controls that provide full treatment of the WQ_v (see *Section 1.0*).

Compatible multi-objective use of dry detention facilities in strongly encouraged.

10.2 Design Criteria and Specifications

Location

Dry detention and extended detention dry basins are to be located downstream of other structural stormwater controls providing treatment of the water quality volume (WQ_v). Extended detention dry basins may be part of a treatment train which treats the WQ_v. See *Section 1.0* for more information on the use of multiple structural controls in a treatment train.

General Design

- Dry detention basins are sized to temporarily store the volume of runoff required to provide flood protection above the Q_f storm event up to the flood mitigation storm, if required.

  Extended detention dry basins are sized to provide extended detention of the streambank protection volume over 24 hours and can also provide additional storage volume for normal detention (peak flow reduction) of the flood mitigation storm event.

  Routing calculations must be used to demonstrate that the storage volume and outlet structure configuration are adequate. See *Section 2.0 of the Hydraulics Technical Manual* for procedures on the design of detention storage.

- Storage may be subject to the requirements of the Texas Dam Safety Program (see *iSWM Program Guidance – Dams and Reservoirs in Texas*) based on the volume, dam height, and level of hazard.

- Earthen embankments less than 6 feet in height that are exposed to flood waters shall have side slopes no greater than the natural angle of repose of the fill material as determined by a geotechnical study. In lieu of a geotechnical study side slopes shall be 4:1 (horizontal to vertical) maximum.

- Earthen embankments 6 feet in height or greater shall be designed per Texas Commission on Environmental Quality guidelines for dam safety (see *iSWM Program Guidance – Dams and Reservoirs in Texas*).

- Vegetated slopes shall be less than 20 feet in height and shall have side slopes no steeper than 2:1 (horizontal to vertical) although 3:1 is preferred. Riprap-protected slopes shall be no steeper than 2:1. Geotechnical slope stability analysis is recommended for slopes greater than 10 feet in height.

- Areas above the normal high water elevations of the detention facility should be sloped toward the basin to allow drainage and to prevent standing water. Careful finish grading is required to avoid creation of upland surface depressions that may retain runoff. The bottom area of storage facilities should be graded toward the outlet to prevent standing water conditions. A low flow or pilot channel across the facility bottom from the inlet to the outlet (often constructed with riprap) is recommended to convey low flows and prevent standing water conditions.

- Adequate maintenance access must be provided for all dry detention and extended detention dry basins.

Inlet and Outlet Structures

- Inflow channels are to be stabilized with flared riprap aprons, or the equivalent. A sediment forebay sized to 0.1 inches per impervious acre of contributing drainage should be provided for dry detention and extended detention dry basins that are in a treatment train with off-line water quality treatment structural controls.
- For a dry detention basin, the outlet structure is sized to its SP_v and Q_f functions (based upon hydrologic routing calculations) and can consist of a weir, orifice, outlet pipe, combination outlet, or other acceptable control structure. Small outlets that will be subject to clogging or are difficult to maintain are not acceptable.

  For an extended detention dry basin, a low flow orifice capable of releasing WQ_v and SP_v over 24 hours must be provided. The streambank protection orifice should have a minimum diameter of 3 inches and should be adequately protected from clogging by an acceptable external trash rack. The orifice diameter may be reduced to 1 inch if internal orifice protection is used (e.g., an overperforated vertical stand pipe with 0.5-inch orifices or slots that are protected by wirecloth and a stone filtering jacket). Adjustable gate valves can also be used to achieve this equivalent diameter.

  See *Section 2.2 of the Hydraulics Technical Manual* for more information on the design of outlet works.
- Seepage control or anti-seep collars should be provided for all outlet pipes.
- Riprap, plunge pools or pads, or other energy dissipators are to be placed at the end of the outlet to prevent scouring and erosion. If the basin discharges to a channel with dry weather flow, care should be taken to minimize tree clearing along the downstream channel, and to reestablish a forested riparian zone in the shortest possible distance. See *Section 4.0 of the Hydraulics Technical Manual*, for more guidance.
- An emergency spillway is to be included in the stormwater pond design to safely pass the extreme flood flow. The spillway prevents pond water levels from overtopping the embankment and causing structural damage. The emergency spillway must be designed to State of Texas guidelines for dam safety (see *iSWM Program Guidance - Dams and Reservoirs in Texas*) and must be located so that downstream structures will not be impacted by spillway discharges.
- A minimum of 1 foot of freeboard must be provided, measured from the top of the water surface elevation for the extreme flood, to the lowest point of the dam embankment not counting the emergency spillway.

10.3 Inspection and Maintenance Requirements

Table 10.1 Typical Maintenance Activities for Dry Detention / Extended Detention Dry Basins
(Source: Denver Urban Storm Drainage Manual, 1999)

Activity	Schedule
• Remove debris from basin surface to minimize outlet clogging and improve aesthetics.	Annually and following significant storm events
• Remove sediment buildup. • Repair and revegetate eroded areas. • Perform structural repairs to inlet and outlets.	As needed based on inspection
• Mow to limit unwanted vegetation.	Routine

10.4 Example Schematics

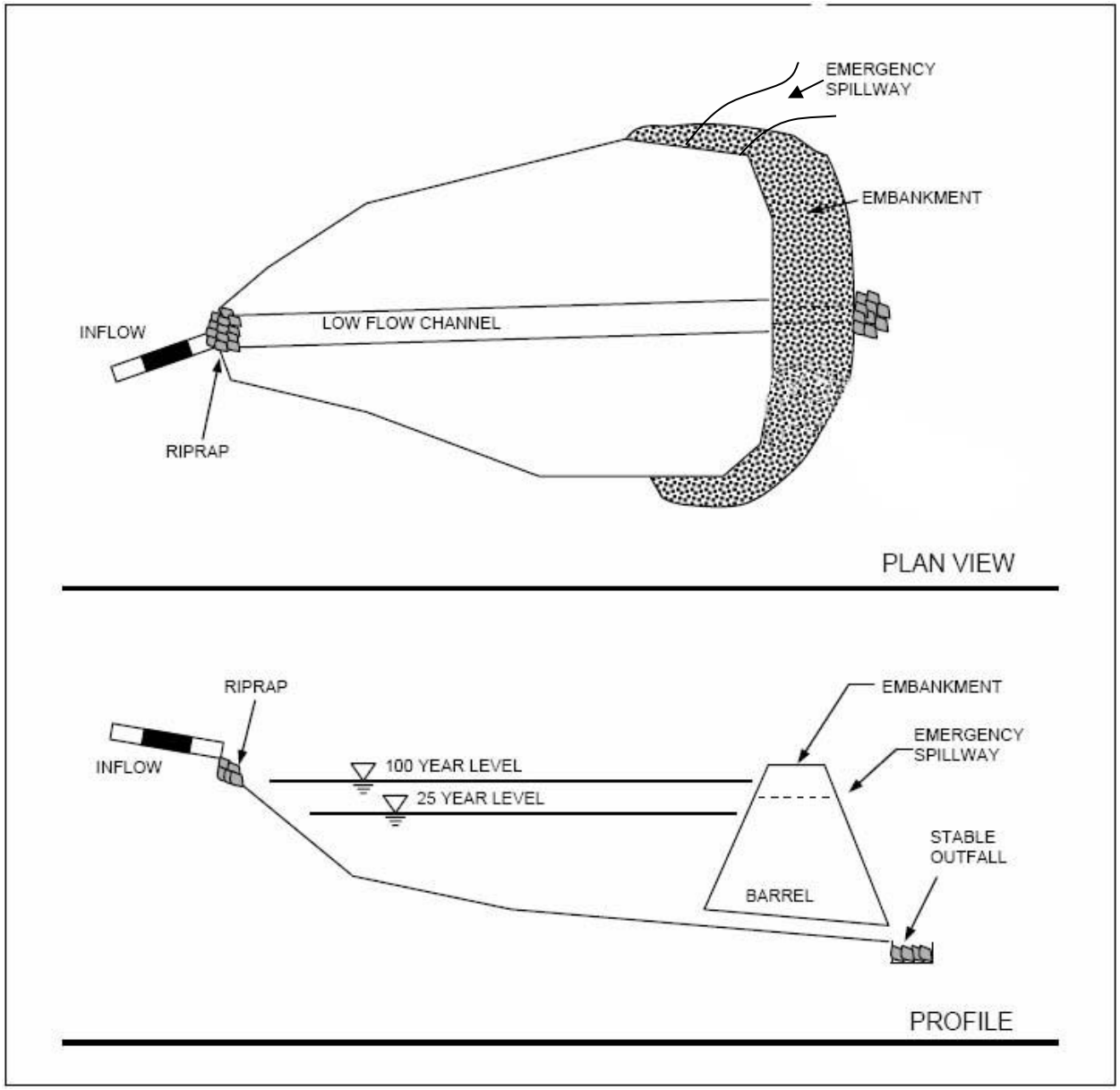


Figure 10.1 Schematic of Dry Detention Basin

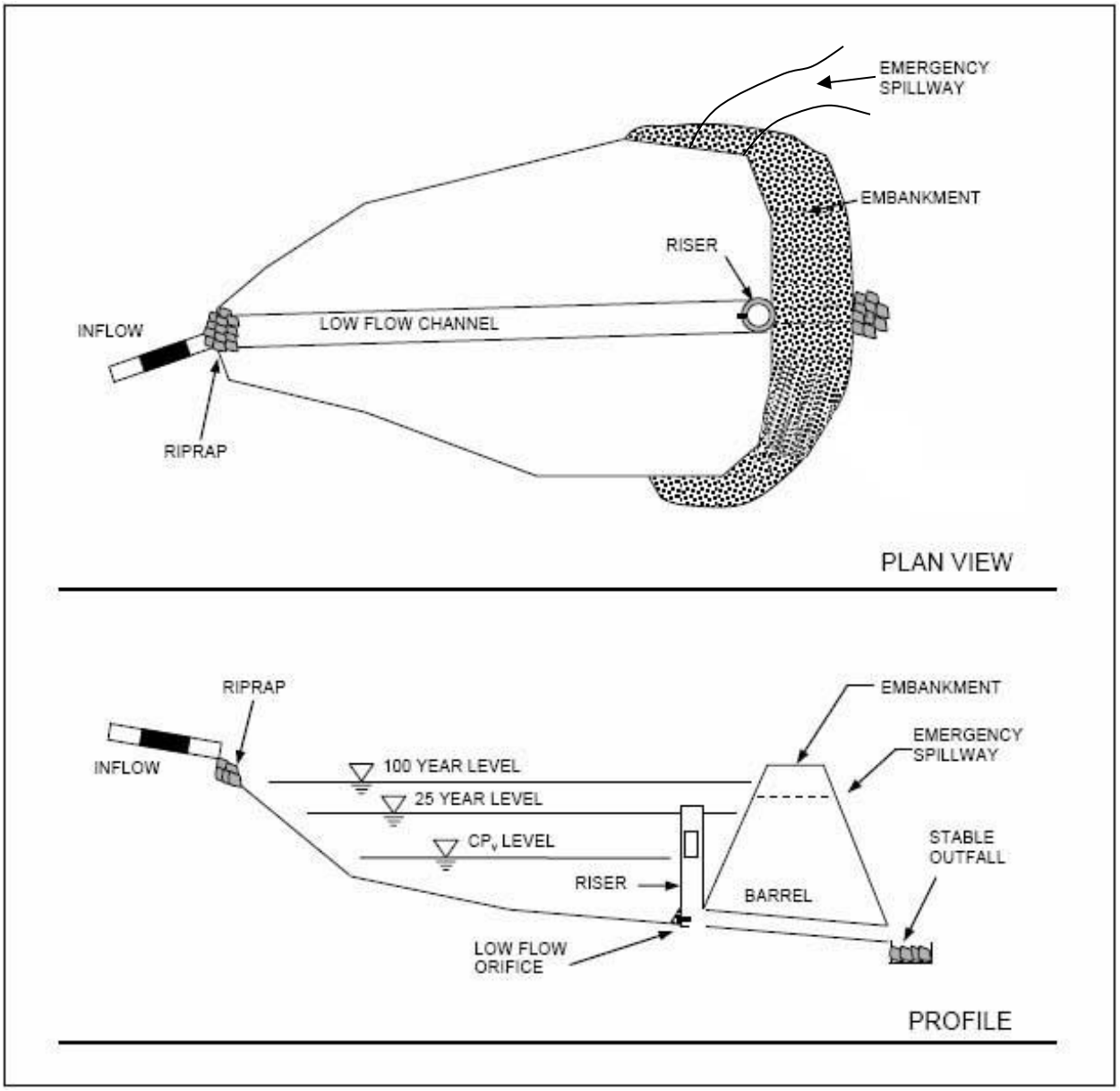


Figure 10.2 Schematic of Dry Extended Detention Basin

SP_v

11.0 Multi-Purpose Detention Areas

Structural Stormwater Control



Description: A facility designed primarily for another purpose, such as parking lots and rooftops that can provide water quantity control through detention of stormwater runoff.

KEY CONSIDERATIONS

DESIGN CRITERIA:

- Adequate grading and drainage must be provided to allow full use of facility's primary purposes following a storm event

ADVANTAGES / BENEFITS:

- Allows for multiple uses of site areas and reduces the need for downstream detention facilities
- Can be used in conjunction with water quality structural control

DISADVANTAGES / LIMITATIONS:

- Controls for stormwater quantity only – not intended to provide water quality protection
- Localized flooding of area as intended may lead to property damage and additional liability

STORMWATER MANAGEMENT SUITABILITY

	Water Quality Protection
P	**Streambank Protection**
P	**On-Site Flood Control**
P	**Downstream Flood Control**

IMPLEMENTATION CONSIDERATIONS

L	**Land Requirement**
L	**Capital Cost**
L	**Maintenance Burden**

Residential Subdivision Use: Yes

High Density/Ultra-Urban: Yes

Drainage Area: No restrictions.

Soils: No restrictions

L=Low **M**=Moderate **H**=High

11.1 General Description

Multi-purpose detention areas are site areas primarily used for one or more specific activities that are also designed to provide for the temporary storage of stormwater runoff to reduce downstream water quantity impacts. Example of multi-purpose detention areas include:

- Parking Lots
- Rooftops
- Sports Fields
- Recessed Plazas

Multi-purpose detention areas are normally dry between rain events, and by their very nature must be useable for their primary function the majority of the time. As such, multi-purpose detention areas should not be used for extended detention (SP_v control).

Multi-purpose detention areas are not intended for water quality protection and must be used in a treatment train approach with other structural controls that provide treatment of the WQ_v (see *Section 1.6*).

11.2 Design Criteria and Specifications

Location

Multi-purpose detention areas can be located upstream or downstream of other structural stormwater controls providing treatment of the water quality protection volume (WQ_v). See the *Section 1.6* for more information on the use of multiple structural controls in a treatment train.

General Design

Multi-purpose detention areas are sized to temporarily store a portion or all of the volume of runoff required to control the flood mitigation storm, if required.

Routing calculations must be used to demonstrate that the storage volume is adequate. See *Section 2.0 of the Hydraulics Technical Manual* for procedures on the design of detention storage.

All multi-purpose detention facilities must be designed to minimize potential safety risks, potential property damage, and inconvenience to the facility's primary purposes. Emergency overflows are to be provided for storm events larger than the design storm. The overflow must not create a significant adverse impact to downstream properties or the conveyance system.

Parking Lot Storage

Parking lot detention can be implemented in areas where portions of large, paved lots can be temporarily used for runoff storage without significantly interfering with normal vehicle and pedestrian traffic. Parking lot detention can be created in two ways: by using ponding areas along sections of raised curbing, or through depressed areas of pavement at drop inlet locations.

The maximum depth of detention ponding in a parking lot, except at a flow control structure, should be 6 inches for a 10-year storm, and 9 inches for a flood mitigation storm. The maximum depth of ponding at a flow control structure is 12 inches for a flood mitigation storm.

The storage area (portion of the parking lot subject to ponding) must have a minimum slope of 0.5% towards the outlet to ensure complete drainage following a storm. A slope of 1% or greater is recommended.

Fire lanes used for emergency equipment must be free of ponding water for runoff events up to the extreme storm (flood mitigation storm) event.

Flows are typically backed up in the parking lot using a wye inlet.

Rooftop Storage

Rooftops can be used for detention storage as long as the roof support structure is designed to address the weight of ponded water and is sufficiently waterproofed to achieve a minimum service life of 30 years. All rooftop detention designs must meet Texas State Building Code and local building code requirements.

The minimum pitch of the roof area subject to ponding is 0.25 inches per foot.

The rooftop storage system must include another mechanism for draining the ponding area in the event that the primary outlet is clogged.

Sports Fields

Athletic facilities such as football and soccer fields and tracks can be used to provide stormwater detention. This is accomplished by constructing berms around the facilities, which in essence creates very large detention basins. Outflow can be controlled through the use of an overflow weir or other appropriate control structure. Proper grading must be performed to ensure complete drainage of the facility.

Public Plazas

In high-density areas, recessed public common areas such as plazas and pavilions can be utilized for stormwater detention. These areas can be designed to flood no more than once or twice annually, and provide important open recreation space during the rest of the year.

11.3 Inspection and Maintenance Requirements

Table 11.1 Typical Maintenance Activities for Multi-Purpose Detention Areas

Activity	Schedule
• Remove debris from ponding area to minimize outlet clogging and improve aesthetics.	Annually and following significant storm events
• Remove sediment buildup. • Repair and revegetate eroded areas. • Perform structural repairs to inlet and outlets.	As needed based on inspection
• Perform additional maintenance activities specific to the type of facility.	As required

(Based on: Denver Urban Storm Drainage Manual, 1999)

12.0 Underground Detention

Detention
Structural Stormwater Control

North Central Texas Council of Governments
Integrated Stormwater Management iSWM

Description

Underground detention facilities provide water quality control through detention and temporary storage of storm water. The runoff is stored in underground vaults, pipe or tank systems. Water is gradually released following storm events. Underground detention facilities are alternatives to surface treatment.

Design Considerations

- Underground detention facilities are often used in conjunction with a water quality structural control device
- There are no restrictions for soil types
- The maximum drainage area for underground detention facilities is 160 acres
- Often used as part of a treatment train to meet water quality requirements
- Prefabricated concrete vaults are available from commercial vendors

Key Advantages

- Underground facilities do not take up any surface space, which is difficult to obtain on some sites
- Designs can be flexible between a concrete vault or a pipe/tank system

Limitations

- Underground detention facilities are not intended to provide water quality treatment
- These facilities are intended for applications where space is limited
- Both construction and replacement costs are high for these types of facilities



Underground Detention Facility in Los Angeles, CA. *Source: Tetra Tech*

Target Constituent	Removal Rate (0% → 100%)
Total Suspended Solids	
Total Phosphorus	
Total Nitrogen	
Fecal Coliform	
Heavy Metals	

Implementation Considerations



Suitability

The iSWM manual has designated that underground detention facilities are suitable for providing:



Maintenance

- Trash, leaf, debris and sediment removal
- Utilize a subsurface vacuum to remove pollutants and debris
- Clean inlets and outlets
- Clear pipes and underdrains as needed
- Provide structural repairs when required
- Address animal damage, including providing mosquito control

Published April 2020

12.1 General Description

Detention vaults are box-shaped underground stormwater storage facilities typically constructed with reinforced concrete. Detention pipe/tank systems are underground storage facilities typically constructed with large diameter metal or plastic pipe. Both serve as an alternative to surface dry detention for stormwater quantity control, particularly for space-limited areas where there is not adequate land for a dry detention basin or multi-purpose detention area.

Both underground vaults and pipe/tank systems can provide streambank protection through extended detention of the streambank protection volume (SP_v), and flood (in some cases extreme flood Q_f) control through normal detention. Basic storage design and routing methods are the same as for detention basins except that the bypass for high flows is typically included.

Underground detention vaults and pipe/tank systems are not intended for water quality treatment and must be used in a treatment train approach with other structural controls that provide treatment of the WQ_v (see *Section 1.6*). This will prevent the underground vault or tank from becoming clogged with trash or sediment and significantly reduces the maintenance requirements for an underground detention system.

Prefabricated concrete vaults are available from commercial vendors. In addition, several pipe manufacturers have developed packaged detention systems.

12.2 Design Criteria and Specifications

Location

Underground detention systems are to be located downstream of other structural stormwater controls providing treatment of the water quality volume (WQ_v). See *Section 1.6* for more information on the use of multiple structural controls in a treatment train.

The maximum contributing drainage area to be served by a single underground detention vault or tank is 200 acres.

General Design

Underground detention systems are sized to provide extended detention of the streambank protection volume over 24 hours and temporarily store the volume of runoff required to provide the desired flood protection.

Routing calculations must be used to demonstrate that the storage volume is adequate. See *Section 2.0 of the Hydraulics Technical Manual* for procedures on the design of detention storage.

Detention Vaults: Minimum 3,000 psi structural reinforced concrete may be used for underground detention vaults. All construction joints must be provided with water stops. Cast-in-place wall sections must be designed as retaining walls. The maximum depth from finished grade to the vault invert should be 20 feet.

Detention Pipe/Tank Systems: The minimum pipe diameter for underground detention tanks is 36 inches.

Underground detention vaults and pipe/tank systems must meet structural requirements for overburden support and traffic loading if appropriate.

Adequate maintenance access must be provided for all underground detention systems. Access must be provided over the inlet pipe and outflow structure. Access openings can consist of a standard frame, grate and solid cover, or a removable panel. Vaults with widths of 10 feet or less should have removable lids.

Inlet and Outlet Structures

- A separate sediment sump or vault chamber sized to 0.1 inches per impervious acre of contributing drainage should be provided at the inlet for underground detention systems that are in a treatment train with off-line water quality treatment structural controls.
- For SP_v control, a low flow orifice capable of releasing the streambank protection volume over 24 hours must be provided. The streambank protection orifice should have a minimum diameter of 3 inches and should be adequately protected from clogging by an acceptable external trash rack. The orifice

diameter may be reduced to 1 inch if internal orifice protection is used (i.e., an overperforated vertical stand pipe with 0.5-inch orifices or slots that are protected by wirecloth and a stone filtering jacket). Adjustable gate valves can also be used to achieve this equivalent diameter.

For on-site flood control, an additional outlet is sized for control of the chosen return period (based upon hydrologic routing calculations) and can consist of a weir, orifice, outlet pipe, combination outlet, or other acceptable control structure.

See *Section 2.2 of the Hydraulics Technical Manual* for more information on the design of outlet works.

- Riprap, plunge pools or pads, or other energy dissipators are to be placed at the end of the outlet to prevent scouring and erosion. See *Section 4.0 of the Hydraulics Technical Manual*, for more guidance.
- A high flow bypass is to be included in the underground detention system design to safely pass the extreme flood flow.

12.3 Inspection and Maintenance Requirements

Table 12.1 Typical Maintenance Activities for Underground Detention Systems

Activity	Schedule
• Remove any trash/debris and sediment buildup in the underground vaults or pipe/tank systems.	Annually
• Perform structural repairs to inlet and outlets.	As needed, based on inspection

12.4 Example Schematics

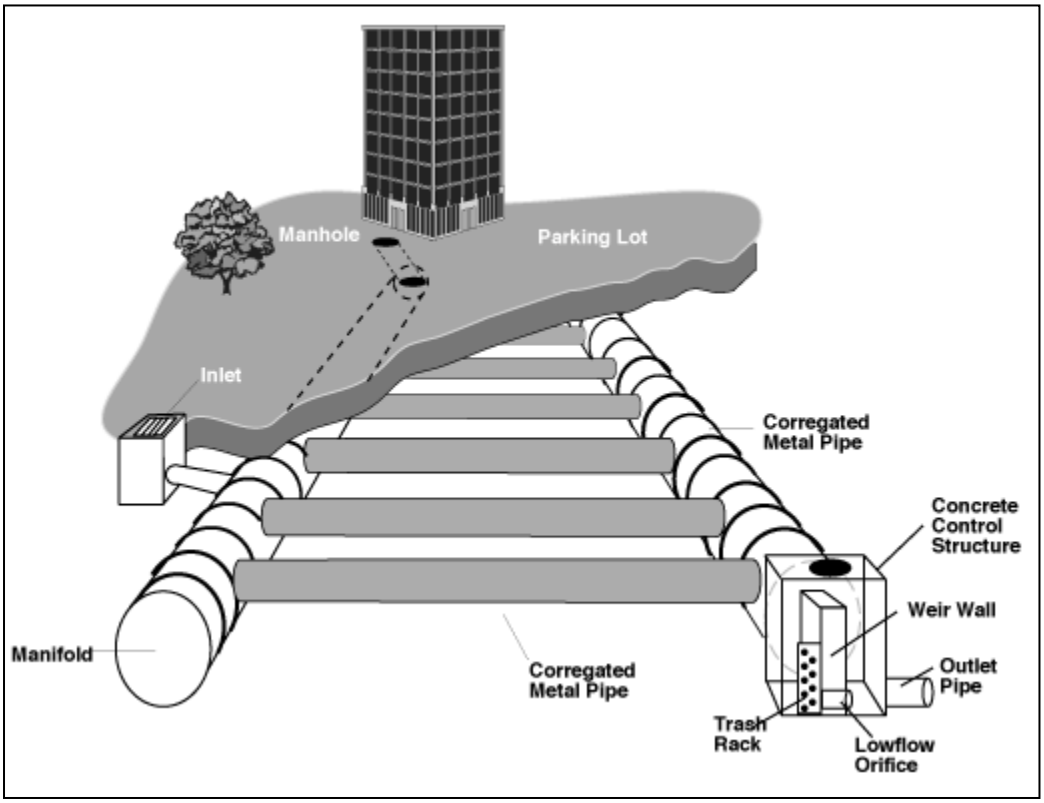


Figure 12.1 Example Underground Detention Tank System

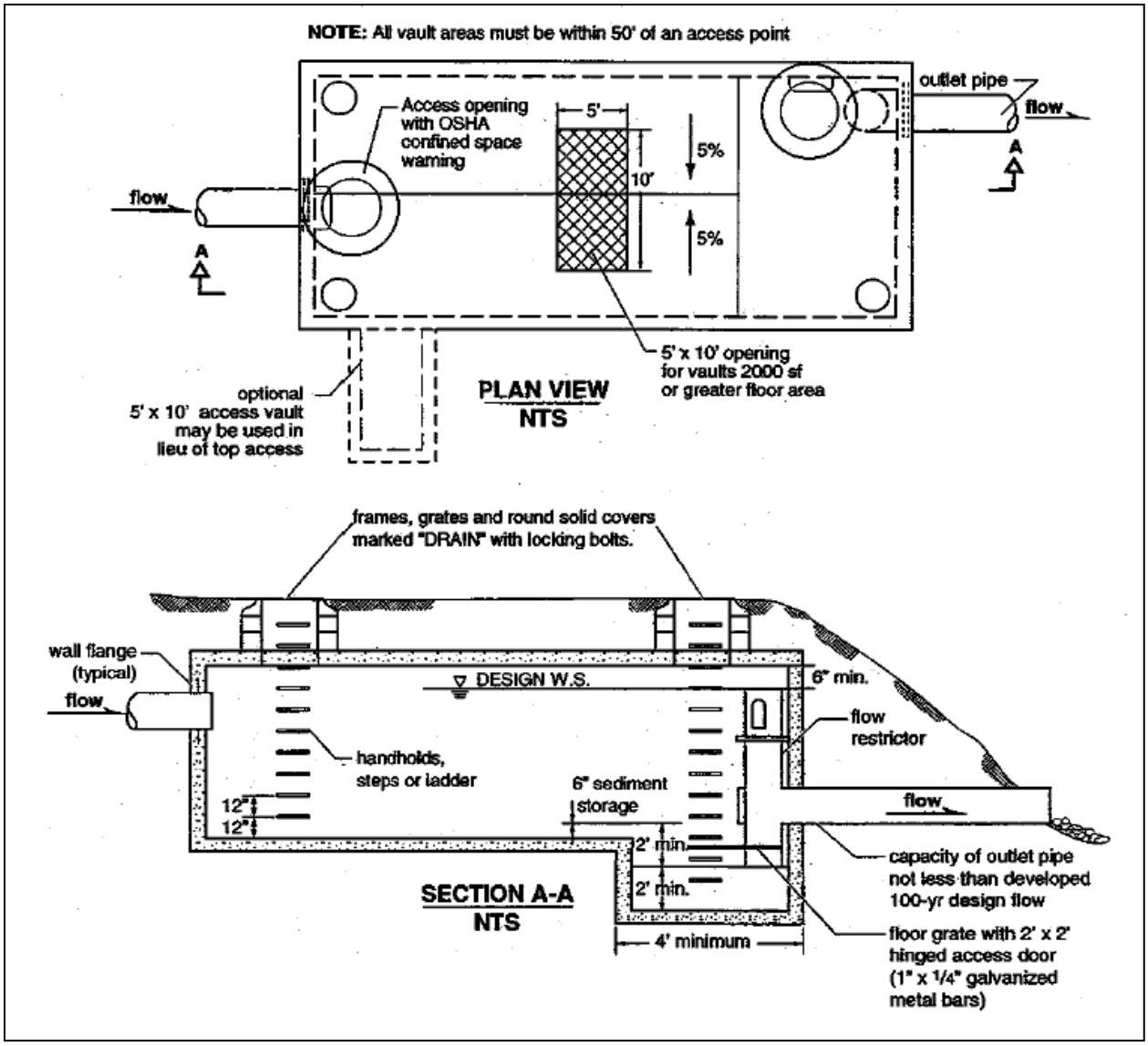


Figure 12.2 Schematic of Typical Underground Detention Vault
(Source: WDE, 2000)

13.0 Filter Strip

Structural Stormwater Control

North Central Texas Council of Governments

integrated Stormwater Management iSWM

Description

Filter strips are land that is engineered and designed to treat runoff from and remove pollutants through vegetative filtering and infiltration. The strips are uniformly graded and densely vegetated.



Filter Strip in Asheboro, NC. *(Source: Tetra Tech)*

Design Considerations

- Stormwater runoff that enters a filter strip must enter via sheet flow, which requires even distribution from the adjacent impervious surface. Flow spreaders may be required.
- Best suited for treating runoff from roads, highways, roof downspouts, small parking lots, and pervious surfaces.
- A permeable berm may be included to increase contact time between runoff and the filter strip.
- Small drainage areas are required.
- Filter strips are often used as a pretreatment option for other stormwater BMPs including bioretention areas.

Target Constituent	Removal Rate (0% → 100%)
Total Suspended Solids	
Total Phosphorus	
Total Nitrogen	
Fecal Coliform	*insufficient data*
Heavy Metals	

Implementation Considerations



Key Advantages

- Filter strips can provide pretreatment and be used as part of the runoff conveyance system.
- Groundwater recharge can be provided via filter strips.
- Filter strips have a relatively low construction cost.
- Landscaping can be performed to make filter strips aesthetically pleasing.

Suitability

The iSWM manual has designated filter strips as suitable for providing:

Water Quality Protection

Limitations

- The TSS removal target cannot be achieved by filter strips alone.
- Filter strips have a large land requirement.
- Filter strips require minimization of impacts that would lead to erosion or compaction.

Maintenance

- Grass should be maintained at 2"–4" in height.
- Periodic repair, regrading and sediment removal is required to prevent channelization.
- Rills, gullies, and other erosive spots should be remedied and repaired.
- Vegetation is required and should be replaced if not established.

Published October 2020

13.1 General Description

Filter strips are uniformly graded and densely vegetated sections of land engineered and designed to treat runoff and remove pollutants through vegetative filtering and infiltration. Filter strips are best suited to treating runoff from roads and highways, roof downspouts, very small parking lots, and pervious surfaces. They are also ideal components of the "outer zone" of a stream buffer, or as pretreatment for another structural stormwater control. Filter strips can serve as a buffer between incompatible land uses, be landscaped to be aesthetically pleasing, and provide groundwater recharge in areas with pervious soils. Filter strips are often used as an *integrated* site design reduction credit (see *Section 13.0* for more information).

Filter strips rely on the use of vegetation to slow runoff velocities and filter out sediment and other pollutants from urban stormwater. There can also be a significant reduction in runoff volume for smaller flows that infiltrate pervious soils while contained within the filter strip. To be effective, however, sheet flow must be maintained across the entire filter strip. Once runoff flow concentrates, it effectively short-circuits the filter strip and reduces any water quality benefits. Therefore, a flow spreader must normally be included in the filter strip design.

There are two different filter strip designs: a simple filter strip and a design that includes a permeable berm at the bottom. The presence of the berm increases the contact time with the runoff, thus reducing the overall width of the filter strip required to treat stormwater runoff. Filter strips are typically an on-line practice, so they must be designed to withstand the full range of storm events without eroding.

13.2 Pollutant Removal Capabilities

Pollutant removal from filter strips is highly variable and depends primarily on density of vegetation and contact time for filtration and infiltration. These, in turn, depend on soil and vegetation type, slope, and presence of sheet flow.

The following design pollutant removal rates are conservative average pollutant reduction percentages for design purposes derived from sampling data, modeling and professional judgment.

- Total Suspended Solids – 50%
- Total Phosphorus – 20%
- Total Nitrogen – 20%
- Fecal Coliform – insufficient data
- Heavy Metals – 40%

13.3 Design Criteria and Specifications

General Criteria

Filter strips should be used to treat small drainage areas. Flow must enter the filter strip as sheet flow spread out over the width (long dimension normal to flow) of the strip, generally no deeper than 1 to 2 inches. As a rule, flow concentrates within a maximum of 75 feet for impervious surfaces, and 150 feet for pervious surfaces (CWP, 1996). For longer flow paths, special provision must be made to ensure design flows spread evenly across the filter strip, instead of becoming concentrated.

Filter strips should be integrated within site designs.

Filter strips should be constructed outside the natural stream buffer area whenever possible to maintain a more natural buffer along the streambank.

Filter strips should be designed for slopes between 2% and 6%. Greater slopes than this would encourage the formation of concentrated flow. Flatter slopes would encourage standing water.

Filter strips should not be used on soils that cannot sustain a dense grass cover with high retardance. Designers should choose a grass that can withstand relatively high velocity flows at the entrances, and

both wet and dry periods. See *Section 1.0 of the Landscape Technical Manual* for a list of appropriate grasses for use in North Central Texas.

The flow path should be at least 15 feet across the strip to provide filtration and contact time for water quality treatment. Twenty-five (25) feet is preferred (where available), though the length of the flow path will normally be dictated by design method.

Both the top and toe of the slope should be as flat as possible to encourage sheet flow and prevent erosion.

An effective flow spreader is a pea gravel diaphragm at the top of the slope (ASTM D 448 size no. 6, 1/8" to 3/8"). The pea gravel diaphragm (a small trench running along the top of the filter strip) serves two purposes. First, it acts as a pretreatment device, settling out sediment particles before they reach the practice. Second it acts as a level spreader, maintaining sheet flow as runoff flows over the filter strip. Other types of flow spreaders include a concrete sill, curb stops, or curb and gutter with "sawteeth" cut into it.

Ensure that flows in excess of design flow move across or around the strip without damaging it. Often a bypass channel or overflow spillway with protected channel section is designed to handle higher flows.

Pedestrian traffic across the filter strip should be limited through channeling onto sidewalks.

Maximum discharge loading per foot of filter strip width (perpendicular to flow path) is found using the Manning's Equation:

$$q = \frac{0.023}{n} Y^{\frac{5}{3}} S^{\frac{1}{2}} \qquad \textbf{(13.1)}$$

where:

q = discharge per foot of width of filter strip (cfs/ft)
Y = allowable depth of flow (inches)
S = slope of filter strip (percent)
N = Manning's "n" roughness coefficient

(use 0.15 for medium grass, 0.25 for dense grass, and 0.35 for very dense Bermuda-type grass)

The minimum width of a filter strip is:

$$W_{fMIN} = \frac{Q}{q} \qquad \textbf{(13.2)}$$

where:

W_{fMIN} = minimum filter strip width perpendicular to flow (feet)

Filter without Berm

Size filter strip (parallel to flow path) for a contact time of 5 minutes minimum

Equation for filter length is based on the SCS TR55 travel time equation (SCS, 1986):

$$L_f = \frac{(T_t)^{1.25} (P_{2\text{-}24})^{0.625} (S)^{1/2}}{3.34n} \qquad \textbf{(13.3)}$$

L_f = length of filter strip parallel to flow path (ft)
T_t = travel time through filter strip (minutes)
$P_{2\text{-}24}$ = 2-year, 24-hour rainfall depth (inches)
S = slope of filter strip (percent)
n = Manning's "n" roughness coefficient

(use 0.15 for medium grass, 0.25 for dense grass, and 0.35 for very dense Bermuda-type grass)

Filter Strips with Berm

Size outlet pipes to ensure that the bermed area drains within 24 hours.

Specify grasses resistant to frequent inundation within the shallow ponding limit.

Berm material should be of sand, gravel and sandy loam to encourage grass cover (Sand: ASTM C-33 fine aggregate concrete sand 0.02”-0.04”, Gravel: AASHTO M-43 ½” to 1”).

Size filter strip to contain the WQ_v within the wedge of water backed up behind the berm.

Maximum berm height is 12 inches.

Filter Strips for Pretreatment

A number of other structural controls, including bioretention areas and infiltration trenches, may utilize a filter strip as a pretreatment measure. The required length of the filter strip flow path depends on the drainage area, imperviousness, and the filter strip slope. Table 13.1 provides sizing guidance for bioretention filter strips for pretreatment.

Table 13.1 Bioretention Filter Strip Sizing Guidance

Parameter	Impervious Areas				Pervious Areas (Lawns, etc)			
Maximum inflow approach length (feet)	35		75		75		100	
Filter strip slope (max = 6%)	< 2%	> 2%	< 2%	> 2%	< 2%	> 2%	< 2%	> 2%
Filter strip minimum length (feet)	10	15	20	25	10	12	15	18

(Source: Claytor and Schueler, 1996)

13.4 Inspection and Maintenance Requirements

Table 13.2 Typical Maintenance Activities for Filter Strips

Activity	Schedule
• Mow grass to maintain a 2 to 4 inch height.	Regularly (frequently)
• Inspect pea gravel diaphragm for clogging and remove built-up sediment. • Inspect vegetation for rills and gullies and correct. Seed or sod bare areas. • Inspect to ensure that grass has established. If not, replace with an alternative species.	Annual Inspection (Semi-annual first year)

(Source: CWP, 1996)
If berm is used per example, inspect outlet pipes for clogging

Additional Maintenance Considerations and Requirements

Filter strips require similar maintenance to other vegetative practices. Maintenance is very important for filter strips, particularly in terms of ensuring that flow does not short circuit the practice.

13.5 Example Schematic

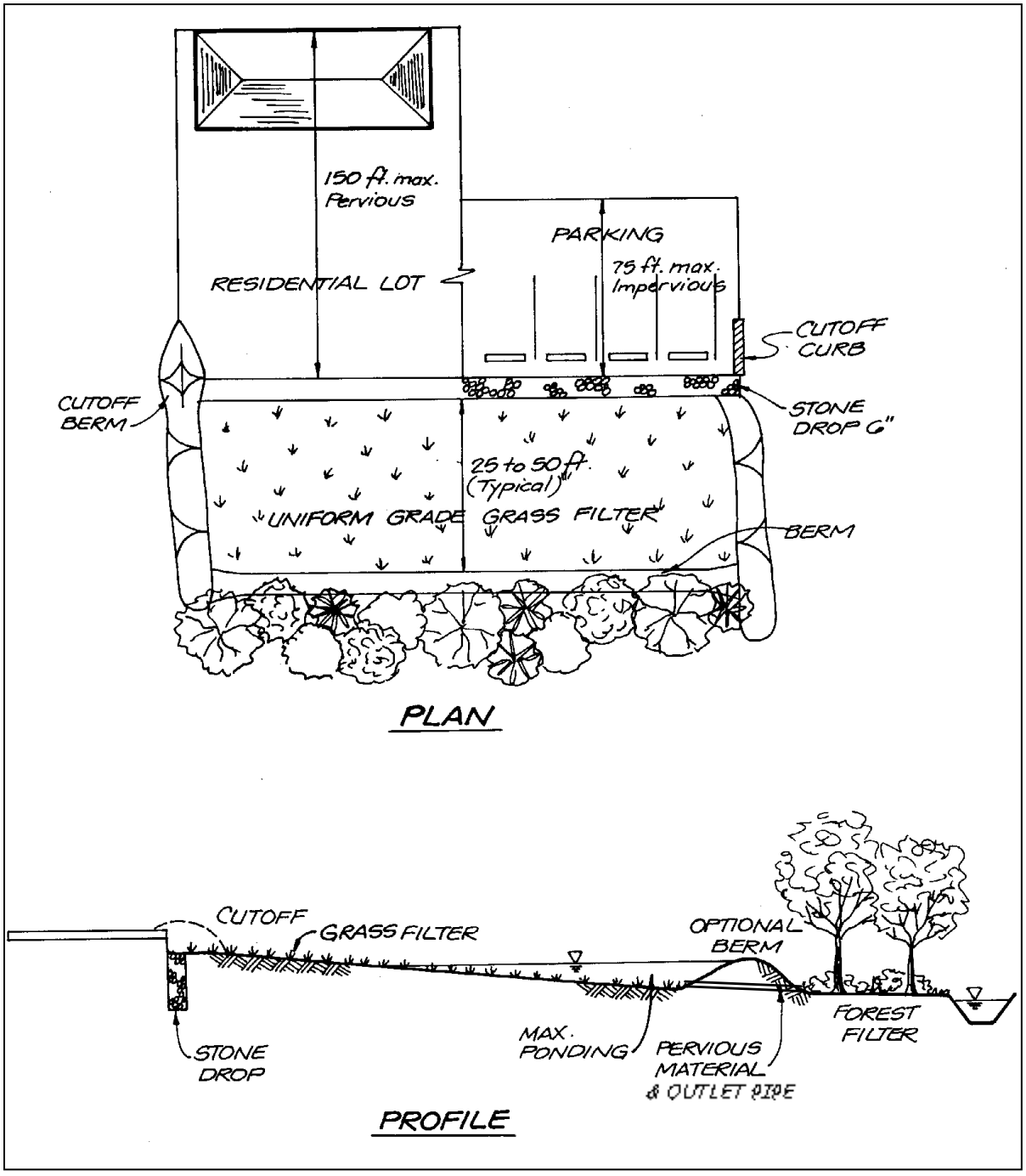


Figure 13.1 Schematic of Filter Strip (with Berm)

13.6 Design Example

Basic Data

Small commercial lot 150 feet deep x 100 feet wide located in Denison

- Drainage area (A) = 0.34 acres
- Impervious percentage (I) = 70%
- Slope equals 4%, Manning's n = 0.25

Calculate Maximum Discharge Loading Per Foot of Filter Strip Width

Using Equation 13.1:

$q = 0.0237/0.25 * (1.0)^{5/3} * (4)^{1/2} = 0.19$ cfs/ft

Water Quality Peak Flow

See *Section 1.4 of the Water Quality Technical Manual* for details

Compute the Water Quality Volume in inches:

WQv = 1.5 (0.05 + 0.009 * 70) = 1.02 inches

Compute modified CN for 1.5-inch rainfall (P=1.5):

CN = 1000/[10+5P+10Q-10(Q2+1.25*Q*P)½]

= 1000/[10+5*1.5+10*0.82-10(0.822+1.25*0.82*1.5)½]

= 92.4 (Use CN = 92)

For CN = 92 and an estimated time of concentration (T_c) of 8 minutes (0.13 hours), compute the Q_{wq} for a 1.5-inch storm.

From *Table 1.11 in the Hydrology Technical Manual*, I_a = 0.174, therefore I_a/P = 0.174/1.5 = 0.116.

From *Figure 1.10 in the Hydrology Technical Manual* for a Type II storm (using the limiting values) q_u = 950 csm/in, and therefore:

Q_{wq} = (950 csm/in) (0.34ac/640ac/mi^2) (1.02") = 0.51 cfs

Minimum Filter Width

Using Equation 13.2:

$W_{fMIN} = Q/q = 0.51/0.19 = 2.7$ feet

Since the width of the lot is 100 feet, the actual width of the filter strip will depend on site grading and the ability to deliver the drainage to the filter strip in sheet flow through a pea gravel filled trench.

Filter without Berm

- 2-year, 24-hour storm (see *Section 5.0 of the Hydrology Technical Manual*) = 0.17 in/hr or 0.17*24= 4.08 inches
- Use 5 minute travel (contact) time

Using Equation 13.3:

L_f = (5)1.25 * (4.08)0.625 * (4)0.5 / (3.34 * 0.25) = 43 feet

Note: Reducing the filter strip slope to 2% and planting a denser grass (raising the Manning n to 0.35) would reduce the filter strip length to 22 feet. Sensitivity to slope and Manning's n changes are illustrated for this example in Figure 13.2.

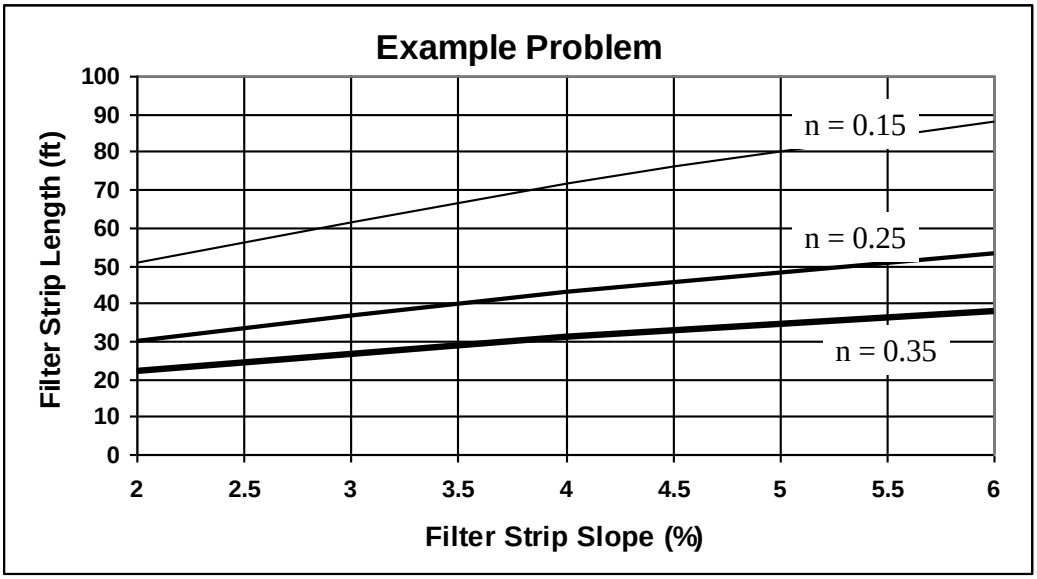


Figure 13.2 Example Problem Sensitivity of Filter Strip Length to Slope and Manning's n Values

Filter With Berm

- Pervious berm height is 6 inches

Compute the Water Quality Volume in cubic feet:

WQv = Rv * 1.5/12 * A = (0.05 + 0.009 * 70) * 1.5/12 * 0.34 = 0.029 Ac-ft or 1,259 ft3

For a berm height of 6 inches the "wedge" of volume captured by the filter strip is:

Volume = Wf * 0.5 * Lf * 0.5 = 0.25WfLf = 1,259 ft3

For a maximum width of the filter of 100 feet, the length of the filter would then be 50 feet.

For a 1-foot berm height, the length of the filter would be 25 feet.

14.0 Organic Filter

Structural Stormwater Control



Description: Design variant of the surface sand filter using organic materials in the filter media.

KEY CONSIDERATIONS

DESIGN CRITERIA:

- Minimum head requirement of 5 to 8 feet

ADVANTAGES / BENEFITS:

- High pollutant removal capability
- Removal of dissolved pollutants is greater than sand filters due to cation exchange capacity

DISADVANTAGES / LIMITATIONS:

- Severe clogging potential if exposed soil surfaces exist upstream
- Intended for hotspot or space-limited applications, or for areas requiring enhanced pollutant removal capability
- High maintenance requirements
- Filter may require more frequent maintenance than most of the other stormwater controls

POLLUTANT REMOVAL

80%	**Total Suspended Solids**
60/40%	**Nutrients** - Total Phosphorus / Total Nitrogen removal
75%	**Metals** - Cadmium, Copper, Lead, and Zinc removal
50%	**Pathogens** - Coliform, Streptococci, E.Coli removal

STORMWATER MANAGEMENT SUITABILITY

P	**Water Quality Protection**
	Streambank Protection
	On-Site Flood Control
	Downstream Flood Control

IMPLEMENTATION CONSIDERATIONS

L	**Land Requirement**
H	**Capital Cost**
H	**Maintenance Burden**

Residential Subdivision Use: No
High Density/Ultra-Urban: Yes
Drainage Area: 10 acres max.
Soils: No restrictions
Other Considerations:

- Hotspot areas

L=Low **M**=Moderate **H**=High

14.1 General Description

The organic filter is a design variant of the surface sand filter, which uses organic materials such as leaf compost or a peat/sand mixture as the filter media. The organic material enhances pollutant removal by providing adsorption of contaminants such as soluble metals, hydrocarbons, and other organic chemicals.

As with the surface sand filter, an organic filter consists of a pretreatment chamber, and one or more filter cells. Each filter bed contains a layer of leaf compost or the peat/sand mixture, followed by filter fabric and a gravel/perforated pipe underdrain system. The filter bed and subsoils can be separated by an impermeable polyliner or concrete structure to prevent movement into groundwater.

Organic filters are typically used in high-density applications, or for areas requiring enhanced pollutant removal ability. Maintenance is typically higher than the surface sand filter facility due to the potential for clogging. In addition, organic filter systems have a higher head requirement than sand filters.

14.2 Pollutant Removal Capabilities

Peat/sand filter systems provide good removal of bacteria and organic waste metals. The following design pollutant removal rates are conservative average pollutant reduction percentages for design purposes derived from sampling data, modeling and professional judgment. (Note: In some cases, organic materials may be a source of soluble phosphorus and nitrates.)

- Total Suspended Solids – 80%
- Total Phosphorus – 60%
- Total Nitrogen – 40%
- Fecal Coliform – 50%
- Heavy Metals – 75%

14.3 Design Criteria and Specifications

Organic filters are typically used on relatively small sites (up to 10 acres), to minimize potential clogging.

The minimum head requirement (elevation difference needed at a site from the inflow to the outflow) for an organic filter is 5 to 8 feet.

Organic filters can utilize a variety of organic materials as the filtering media. Two typical media bed configurations are the peat/sand filter and compost filter (see Figure 14.1). The peat filter includes an 18-inch 50/50 peat/sand mix over a 6-inch sand layer and can be optionally covered by 3 inches of topsoil and vegetation. The compost filter has an 18-inch compost layer. Both variants utilize a gravel underdrain system.

The type of peat used in a peat/sand filter is critically important. Fibric peat in which undecomposed fibrous organic material is readily identifiable is the preferred type. Hemic peat containing more decomposed material may also be used. Sapric peat made up of largely decomposed matter should *not* be used in an organic filter.

Typically, organic filters are designed as "off-line" systems, meaning that the water volume (WQ_v) is diverted to the filter facility through the use of a flow diversion structure and flow splitter. Stormwater flows greater than the WQ_v are diverted to other controls or downstream using a diversion structure or flow splitter.

Consult the design criteria for the surface sand filter (see *Section 16.0* on *Sand Filters*) for the organic filter sizing and design steps.

14.4 Inspection and Maintenance Requirements

The inspection and maintenance requirements for organic filters are similar to those for surface sand filter facilities (see *Section 16.0* on Sand Filters)

14.5 Example Schematic

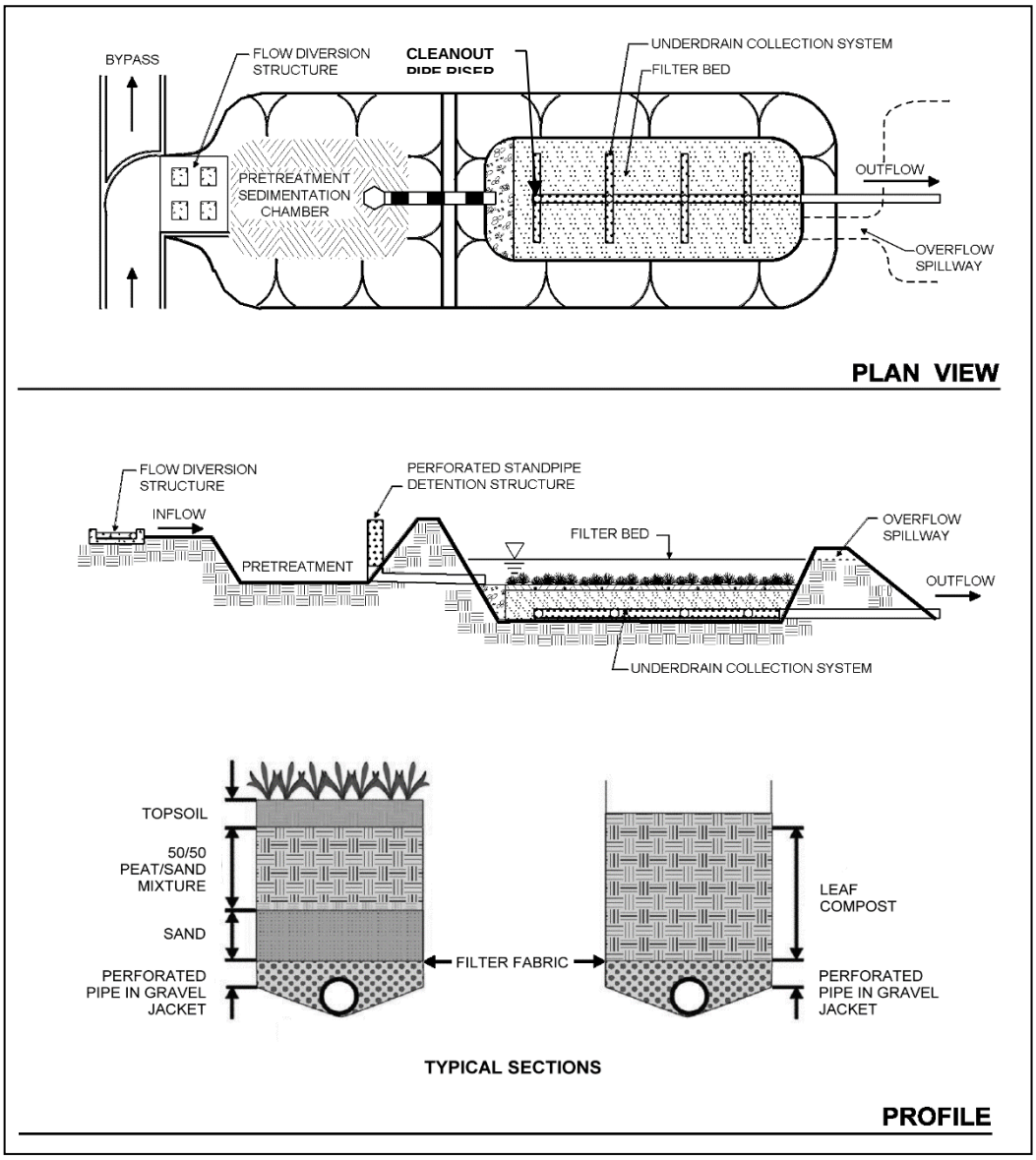


Figure 14.1 Schematic of Organic Filter
(Source: Center for Watershed Protection)

15.0 Planter Boxes

Structural Stormwater Control

North Central Texas Council of Governments

Integrated Stormwater Management iSWM



Planter Box in Nashville, TN. *(Source: Tetra Tech)*

Description

Planter boxes can be utilized in highly urbanized areas, especially areas with large volumes of impervious surfaces. Similar to bioretention areas, planter boxes contain vegetation and filter media to treat stormwater. Boxes can be constructed in place or pre-fabricated.

Design Considerations

- Contained planter boxes receive only rainfall. The rainfall filters through the soil and will either be taken up by vegetation or allowed to seep out of the bottom through an underdrain. These boxes can be relocated but do not have storage for flow control.
- Infiltration planter boxes can receive both rainfall and runoff. The planters are bottomless and runoff will eventually enter the underlying soil. They should not be used next to foundations.
- Flow-through planters collect stormwater in a perforated pipe along the bottom of the box. The treated runoff is discharged out of the side of the planter or into a storm sewer. These planters must be located by a suitable discharge point.
- Planter boxes can clog and should not be used in areas with a high sediment load.
- Infiltration planter boxes must have soil types with good underlying drainage. Other planters do not have soil type restrictions.
- Water should drain through a planter within 4 hours of a storm event.
- All planters require a minimum of 18 inches of growing media.

Key Advantages

- Filtration allows for pollutant removal.
- Planter boxes are flexible and can be used at a variety of sites.
- Planter boxes are often aesthetically pleasing.
- Peak flow rates can be limited by a reservoir.
- Planter boxes are great for urban retrofits.

Limitations

- These facilities are intended for applications where space is limited.
- There is limited data on pollutant removal effectiveness.

Target Constituent	Removal Rate 0% → 100%
Total Suspended Solids	
Total Phosphorus	
Total Nitrogen	
Fecal Coliform	*insufficient data*
Heavy Metals	

Implementation Considerations



Suitability

The iSWM manual has designated planter boxes as suitable for providing:

Water Quality Protection

Maintenance

(See Bioretention for more detail)

- Frequent maintenance for vegetation and aesthetics.
- Filters may require more frequent maintenance.
- Replenish the mulch annually.
- During dry spells, the vegetation should be watered to ensure it does not die.

For a similar design, see the Summary Page for Bioretention

Published October 2020

15.1 General Description

Planter boxes are essentially large pots filled with soil or other growing media. There are several variations of this basic design. The contained planter box receives only rainfall, which filters through the soil and is then either taken up by its vegetation or allowed to seep out the bottom of the planter to the pavement or sidewalk. The infiltration planter box can receive both rainfall and runoff, which eventually filters through the bottomless planter and enters the underlying soil. The flow-through planter box collects flow in a perforated pipe along the bottom of the box and discharges out the side of the planter or into a storm sewer.

Each of the three planter box types has certain advantages and drawbacks:

- The contained planter is not tied into underlying soil or pipes and can therefore be placed almost anywhere and moved when needed. However, it does not have a reservoir to provide additional storage for flow control. Care should also be used in placing it next to building foundations and heavy pedestrian traffic areas.
- The infiltration planter should not be used next to foundations and underlying soils must drain rapidly enough to avoid ponding.
- The flow-through planter can be used next to building foundations since it directs flow off to the side and away from the building. It must be located next to a suitable discharge point into the stormwater conveyance system.

15.2 Pollutant Removal Capabilities

Field tests of planters are lacking, however, tests of a bioretention cell by the EPA showed results that were generally similar to those of the Organic Filter, with somewhat less metals removal (43-78%).

15.3 Design Criteria and Specifications

The infiltration and flow-through planter boxes can capture runoff from surrounding areas and provide limited storage in reservoirs. The ratio of planter area to impervious area should be 7%, assuming a storm volume of 1.5 inches and a reservoir depth in the planter of 12 inches.

The planter should be constructed of stone, concrete, or brick. Pressure-treated wood may be used if it does not leach out toxic chemicals that might contaminate stormwater.

Filter media should consist of sand, gravel and topsoil as shown in Figures 15.1-15.3. As an alternative, compost/mulch can be used in place of the sand, gravel, and topsoil, but will have different infiltration characteristics. Compost with organics will aid in pollutant removal through absorption, but it will remove nitrogen from the plant material as it breaks down/decomposes. A nitrogen fertilizer may need to be added should this occur.

Planter vegetation should be relatively self-sustaining, with minimal fertilizer or pesticide requirements. Grasses, herbs, succulents, shrubs, and trees may be used in planter boxes. Examples include rushes, reeds, sedges, iris, dogwood, currants, and other approved species. Trees are encouraged as their foliage traps additional precipitation.

All of the planters require 18 inches of growing media. The contained planter does not require a minimum width. A minimum width of 30 inches is recommended for the infiltration planter. The flow-through planter should be at least 18 inches wide. The minimum widths help reduce water wicking down the insides of the planter wall.

Water should drain through a planter within 3-4 hours after the storm event.

Soils underneath an infiltration planter should be SCS Hydrologic Type A or B.

15.4 Inspection and Maintenance Requirements

The inspection and maintenance requirements for planter boxes focus on maintaining an adequate drainage rate through the planting media and attractive and healthy vegetation.

Table 15.1 Typical Maintenance Activities for Planter Boxes	
Activity	**Schedule**
• Ensure that downspout or sheet flow from paving is unimpeded. • Ensure planter reservoir drains within 3-4 hours. • Replace or amend topsoil if drainage unsatisfactory.	Quarterly and within 48 hours of major storms
• Ensure that contributing area and planter boxes are clear of debris. • Remove accumulated sediment if greater than 4 inches in depth. • Ensure that planter vegetation is healthy and planter is weeded and shrubs and trees pruned. • Planter vegetation may require watering during long dry spells.	As needed, based on inspection
• Fallen leaves and debris from deciduous plants should be removed.	Three to four times a year
• Replenish mulch. • Training/written materials provided to property owners and tenants.	Annually
• Replace planter if cracked or rotted.	Upon failure

15.5 Example Schematics

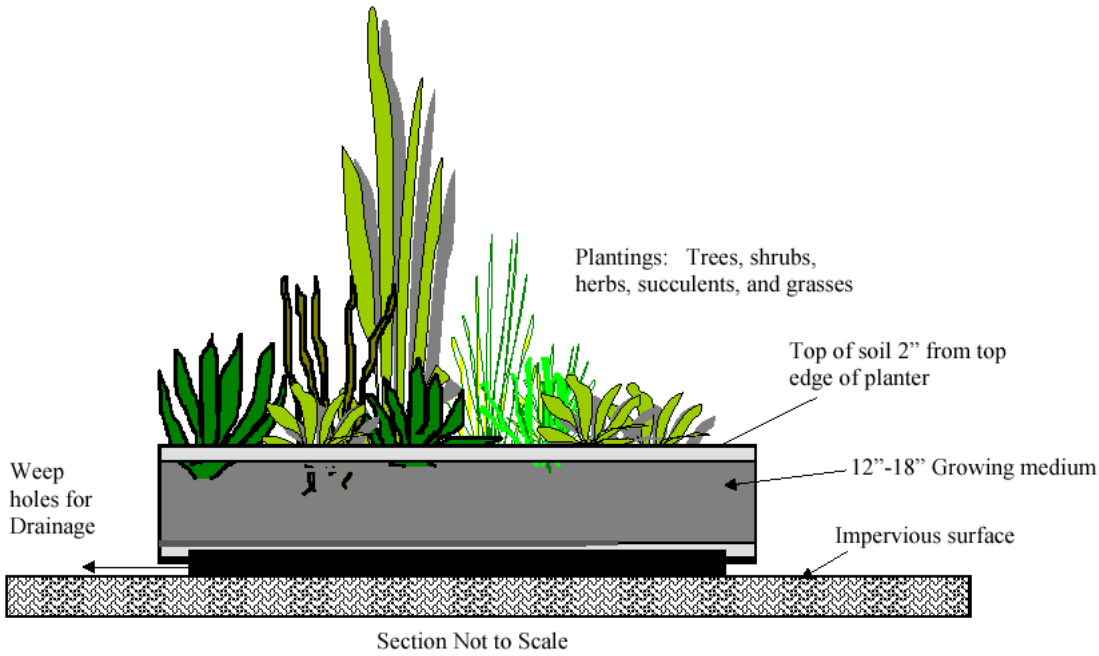
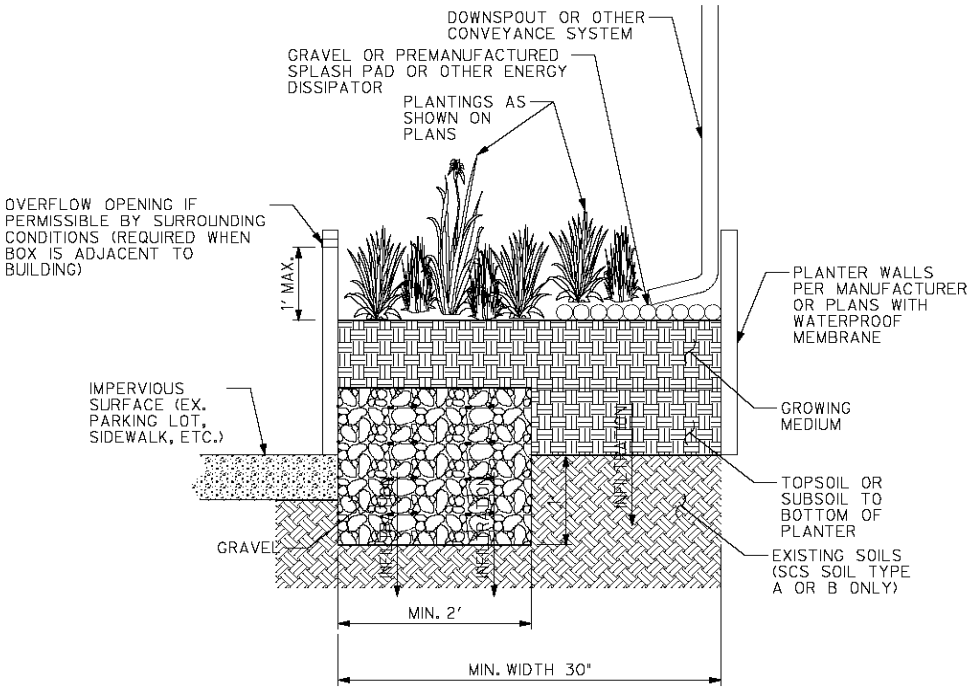


Figure 15.1 Schematic of Contained Planter Box
(Source: City of Portland, Oregon)



INFILTRATION PLANTER

NOT TO SCALE

NOTES:

1. DIMENSIONS SHALL BE PROVIDED BY DESIGNER IN PLANS AND SHALL BE BASED ON SITE SPECIFIC CONDITIONS. DESIGN PROFESSIONAL SHALL AT A MINIMUM PROVIDE THE FOLLOWING INFORMATION:
   - HORIZONTAL CONTROL
   - VERTICAL CONTROL
   - WATER QUALITY PROTECTION VOLUME
   - LANDSCAPING INFORMATION
   - PLANTER COMPOSITION AND STRUCTURAL INFORMATION (IF REQUIRED)
   - WATERPROOFING OR OTHER STRUCTURAL PROTECTIONS
   - GROWING MEDIUM INFORMATION
   - DRAINAGE AGGREGATE INFORMATION

2. DESIGN ENGINEER IS RESPONSIBLE FOR PROVIDING CALCULATIONS THAT DEMONSTRATE THAT GROWING MEDIUM AND SYSTEM AS A WHOLE CAN EFFECTIVELY TREAT THE WATER QUALITY VOLUME (WQv) AS SPECIFIED IN SECTION 1.4 OF THE NCTCOG WATER QUALITY TECHNICAL MANUAL. FLOW IN EXCESS OF THE WQV SHALL BE DESIGNED TO BYPASS THE SYSTEM VIA THE OVERFLOW SYSTEM(S).

3. DESIGN OF THIS SYSTEM SHALL FOLLOW THAT OF SECTION 15 "PLANTER BOXES" CONTAINED IN THE NCTCOG SITE DEVELOPMENT CONTROLS ISWM TECHNICAL MANUAL.

4. LANDSCAPING SPECIFIED BY DESIGNER SHOULD BE CARFULLY CHOSEN TO ENSURE SURVIVABILITY DURING BOTH SUBMERGED AND DROUGHT EXTREMES.

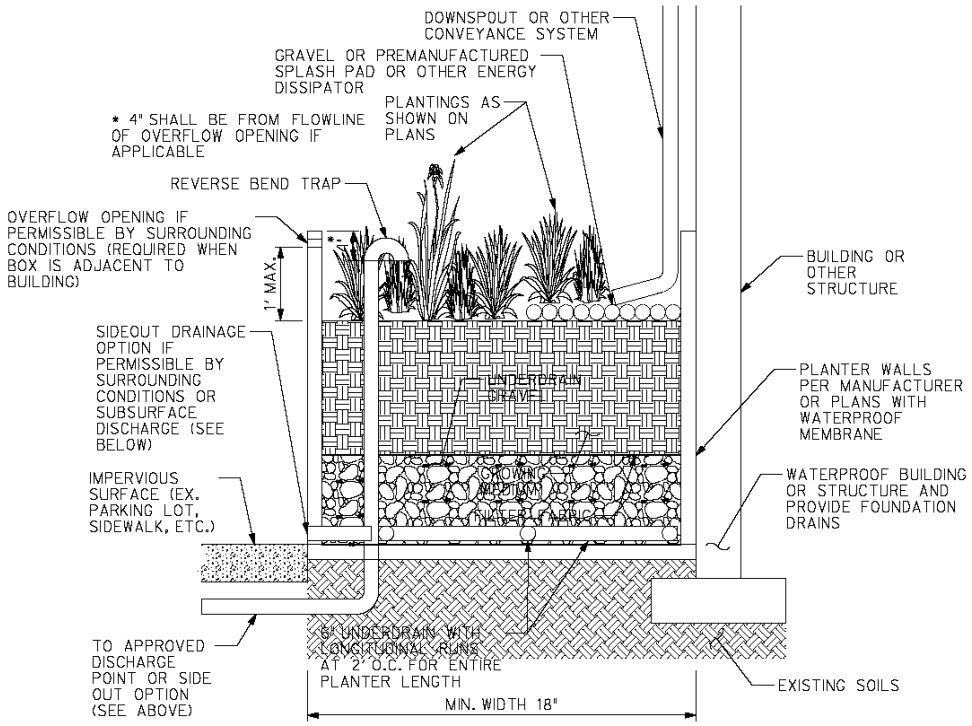
5. DESIGNER SHALL ENSURE THAT SYSTEM IS CAPABLE OF BEING FULLY DRAINED WITHIN 48-HOURS.

6. CARE SHOULD BE TAKEN TO ENSURE THAT OVERFLOW PATHS DO NOT CREATE AN UNSAFE CONDITION ON ADJACENT PEDESTRIAN SURFACES.

7. CARE SHOULD BE TAKEN TO ENSURE THAT EXISTING SOILS INTENDED TO FACILITATE INFILTRATION ARE NOT COMPACTED DURING ADJACENT CONSTRUCTION OPERATIONS.

8. DESIGNER SHALL BE RESPONSIBLE FOR PREPARING AN OPERATIONS AND MAINTENANCE MANUAL OR MATRIX FOR SYSTEMS TO AID OWNER IN DETERMINING MAINTENANCE PROCEEDURES AND INTERVALS. DESIGNER SHALL ALSO CONSIDER MAINTENANCE ACCESS WHEN DESIGNING SYSTEM.

Figure 15.2 Schematic of Infiltration Planter Box
(Source: City of Portland, Oregon)



FLOW-THROUGH PLANTER
NOT TO SCALE

NOTES:

1. DIMENSIONS SHALL BE PROVIDED BY DESIGNER IN PLANS AND SHALL BE BASED ON SITE SPECIFIC CONDITIONS. DESIGN PROFESSIONAL SHALL AT A MINIMUM PROVIDE THE FOLLOWING INFORMATION:
- HORIZONTAL CONTROL
- VERTICAL CONTROL
- WATER QUALITY PROTECTION VOLUME
- LANDSCAPING INFORMATION
- PLANTER COMPOSITION AND STRUCTURAL INFORMATION (IF REQUIRED)
- WATERPROOFING OR OTHER STRUCTURAL PROTECTIONS
- GROWING MEDIUM INFORMATION
- DRAINAGE AGGREGATE AND UNDERDRAIN INFORMATION

2. DESIGN ENGINEER IS RESPONSIBLE FOR PROVIDING CALCULATIONS THAT DEMONSTRATE THAT GROWING MEDIUM AND SYSTEM AS A WHOLE CAN EFFECTIVELY TREAT THE WATER QUALITY VOLUME (WQv) AS SPECIFIED IN SECTION 1.4 OF THE NCTCOG WATER QUALITY TECHNICAL MANUAL. FLOW IN EXCESS OF THE WQV SHALL BE DESIGNED TO BYPASS THE SYSTEM VIA THE OVERFLOW SYSTEM(S).

3. DESIGN OF THIS SYSTEM SHALL FOLLOW THAT OF SECTION 15 'PLANTER BOXES' CONTAINED IN THE NCTCOG SITE DEVELOPMENT CONTROLS ISWM TECHNICAL MANUAL.

4. LANDSCAPING SPECIFIED BY DESIGNER SHOULD BE CARFULLY CHOSEN TO ENSURE SURVIVABILITY DURING BOTH SUBMERGED AND DROUGHT EXTREMES.

5. CARE SHOULD BE TAKEN TO ENSURE THAT OVERFLOW PATHS DO NOT CREATE AN UNSAFE CONDITION ON ADJACENT PEDESTRIAN SURFACES.

6. DESIGNER SHALL BE RESPONSIBLE FOR PREPARING AN OPERATIONS AND MAINTENANCE MANUAL OR MATRIX FOR SYSTEMS TO AID OWNER IN DETERMINING MAINTENANCE PROCEEDURES AND INTERVALS. DESIGNER SHALL ALSO CONSIDER MAINTENANCE ACCESS WHEN DESIGNING SYSTEM.

Figure 15.3 Schematic of Flow-Through Planter Box
(Source: City of Portland, Oregon)

16.0 Sand Filters

General Application
Structural Stormwater Control

North Central Texas Council of Governments
integrated Stormwater Management iSWM



Sand Filter Drain in Raleigh, NC. *Source: Tetra Tech*

Description

Sand filters, also called filtration basins, are structural stormwater controls that capture and store runoff and pass it through a bed of filter sand. The facilities are multi-chamber structures that utilize a sediment forebay or sedimentation chamber, a sand bed for filter media, and often require an underdrain collection system. Sand filter designs are typically either a surface sand filter or a perimeter sand filter.

Design Considerations

- The facility consists of a sand filter media with an underdrain system
- Sand filters typically require 2 to 6 feet of head
- The maximum drainage area for a surface sand filter is 10 acres
- The maximum drainage area for a perimeter sand filter is 2 acres
- Clay or sandy soils may require a pretreatment device; otherwise any soil type can be utilized
- In order to provide water quantity control, other best management practices are required
- The selected site should not have a grade above 6%

Key Advantages

- Sand filters are applicable to small drainage areas
- Highly impervious areas can be drained to sand filters for pollutant removal
- Sand filters have good retrofit capacity
- Sand filters can be used in hotspot areas
- Typically, less space is required for a sand filter than for other facilities

Limitations

- There are high maintenance requirements associated with sand filters
- Sand filters are not recommended in areas with high sediment content loads or in clay/silt runoff areas
- Relative to other best management practices, sand filters are relatively costly
- There is a potential for odor problems to arise with sand filters

Target Constituent	Removal Rate 0% → 100%
Total Suspended Solids	
Total Phosphorus	
Total Nitrogen	
Fecal Coliform	
Heavy Metals	

Implementation Considerations



Suitability

The iSWM manual has designated that sand filter facilities are suitable for providing:



Maintenance

- Trash, leaf, debris and sediment removal
- Provide removal of vegetation (weeds) when a surface sand filter is utilized
- Scarify the media to promote pollutant removal
- Clean inlets and outlets
- Clear pipes and underdrains when required
- Provide erosion and structural repairs when required
- Address animal damage as needed
- Replace media upon failure of the device

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16.1 General Description

Sand filters (also referred to as *filtration basins*) are structural stormwater controls that capture and temporarily store stormwater runoff and pass it through a filter bed of sand. Most sand filter systems consist of two-chamber structures. The first chamber is a sediment forebay or sedimentation chamber, which removes floatables and heavy sediments. The second is the filtration chamber, which removes additional pollutants by filtering the runoff through a sand bed. The filtered runoff is typically collected and returned to the conveyance system, though it can also be partially or fully infiltrated into the surrounding soil in areas with porous soils.

Because they have few site constraints beside head requirements, sand filters can be used on development sites where the use of other structural controls may be precluded. However, sand filter systems can be relatively expensive to construct, install, and maintain.

There are two primary sand filter system designs, the *surface sand filter* and the *perimeter sand filter*. Below are descriptions of these filter systems:

- **Surface Sand Filter** – The surface sand filter is a ground-level open air structure that consists of a pretreatment sediment forebay and a filter bed chamber. This system can treat drainage areas up to 10 acres in size and is typically located off-line. Surface sand filters can be designed as an excavation with earthen embankments or as a concrete or block structure.
- **Perimeter Sand Filter** – The perimeter sand filter is an enclosed filter system typically constructed just below grade in a vault along the edge of an impervious area such as a parking lot. The system consists of a sedimentation chamber and a sand bed filter. Runoff flows into the structure through a series of inlet grates located along the top of the control.

A third design variant, the *underground sand filter*, is intended primarily for extremely space limited and high density areas and is thus considered a limited application structural control. See *Section 16.0* on Underground Sand Filters for more details.



Surface Sand Filter



Perimeter Sand Filter

Figure 16.1 Sand Filter Examples

16.2 Stormwater Management Suitability

Sand filter systems are designed primarily as off-line systems for stormwater quality (i.e., the removal of stormwater pollutants) and will typically need to be used in conjunction with another structural control to provide downstream streambank protection, on-site flood control, and downstream flood control, if required. However, under certain circumstances, filters can provide limited runoff quantity control, particularly for smaller storm events.

Water Quality

In sand filter systems, stormwater pollutants are removed through a combination of gravitational settling, filtration, and adsorption. The filtration process effectively removes suspended solids and particulates, biochemical oxygen demand (BOD), fecal coliform bacteria, and other pollutants. Surface sand filters with a grass cover have additional opportunities for bacterial decomposition as well as vegetation uptake of pollutants, particularly nutrients. *Section 16.3* provides pollutant removal efficiencies that can be used for planning and design purposes.

Streambank Protection

For smaller sites, a sand filter may be designed to capture the entire streambank protection volume SP_v in either an off- or on-line configuration. Given that a sand filter system is typically designed to completely drain over 40 hours, the requirement of extended detention of the 1-year, 24-hour storm runoff volume will be met. For larger sites or where only the WQ_v is diverted to the sand filter facility, another structural control must be used to provide SP_v extended detention.

On-Site Flood Control

Another structural control must be used in conjunction with a sand filter system to reduce the post-development peak flow to pre-development levels (detention) if needed.

Downstream Flood Control

Sand filter facilities must provide flow diversion and/or be designed to safely pass extreme storm flows and protect the filter bed and facility.

The volume of runoff removed and treated by the sand filter may be taken in the on-site flood control and downstream flood control calculations (see Section 1.0).

16.3 Pollutant Removal Capabilities

Both the surface and perimeter sand filters are presumed to be able to remove 80% of the total suspended solids load in typical urban post-development runoff when sized, designed, constructed and maintained in accordance with the recommended specifications. Undersized or poorly designed sand filters can reduce TSS removal performance.

The following design pollutant removal rates are conservative average pollutant reduction percentages for design purposes derived from sampling data, modeling, and professional judgment. In a situation where a removal rate is not deemed sufficient, additional controls may be put in place at the given site in a series or “treatment train” approach.

- Total Suspended Solids – 80%
- Total Phosphorus – 50%
- Total Nitrogen – 25%
- Fecal Coliform – 40%
- Heavy Metals – 50%

For additional information and data on pollutant removal capabilities for sand filters, see the National Pollutant Removal Performance Database (2nd Edition) available at www.cwp.org and the National Stormwater Best Management Practices (BMP) Database at www.bmpdatabase.org

16.4 Application and Site Feasibility Criteria

Sand filter systems are well suited for highly impervious areas where land available for structural controls is limited. Sand filters should primarily be considered for new construction or retrofit opportunities for commercial, industrial, and institutional areas where the sediment load is relatively low, such as: parking lots, driveways, loading docks, gas stations, garages, airport runways/taxiways, and storage yards. Sand filters may also be feasible and appropriate in some multi-family or higher density residential developments.

To avoid rapid clogging and failure of the filter media, the use of sand filters should be avoided in areas with less than 50% impervious cover, or high sediment yield sites with clay/silt soils.

The following basic criteria should be evaluated to ensure the suitability of a sand filter facility for meeting stormwater management objectives on a site or development.

General Feasibility

- Suitable for Residential Subdivision Usage – NO
- Suitable for High Density/Ultra Urban Areas – YES
- Regional Stormwater Control – NO

Physical Feasibility - Physical Constraints at Project Site

- Drainage Area – 10 acres maximum for surface sand filter; 2 acres maximum for perimeter sand filter
- Space Required – Function of available head at site
- Site Slope – No more than 6% slope across filter location
- Minimum Head – Elevation difference needed at a site from the inflow to the outflow: 5 feet for surface sand filters; 2 to 3 feet for perimeter sand filters
- Minimum Depth to Water Table – For a surface sand filter with infiltration (earthen structure), 2 feet are required between the bottom of the sand filter and the elevation of the seasonally high water table
- Soils – No restrictions; Group “A” soils generally required to allow infiltration (for surface sand filter earthen structure)
- Downstream Water Surface - Downstream flood conditions need to be verified to avoid surcharging and back washing of the filter material.

Other Constraints / Considerations

- Aquifer Protection – Do not allow infiltration of filtered hotspot runoff into groundwater

16.5 Planning and Design Criteria

*The following criteria are to be considered **minimum** standards for the design of a sand filter facility. Consult with the local review authority to determine if there are any variations to these criteria or additional standards that must be followed.*

A. Location and Siting

Surface sand filters should have a contributing drainage area of 10 acres or less. The maximum drainage area for a perimeter sand filter is 2 acres.

Sand filter systems are generally applied to land uses with a high percentage of impervious surfaces. Sites with less than 50% imperviousness or high clay/silt sediment loads must not use a sand filter without adequate pretreatment due to potential clogging and failure of the filter bed. Any disturbed areas within the

sand filter facility drainage area should be identified and stabilized. Filtration controls should only be constructed after the construction site is stabilized.

Surface sand filters are generally used in an off-line configuration where the water quality volume (WQ_v) is diverted to the filter facility through the use of a flow diversion structure and flow splitter. Stormwater flows greater than the WQ_v are diverted to other controls or downstream using a diversion structure or flow splitter.

Perimeter sand filters are typically sited along the edge, or perimeter, of an impervious area such as a parking lot.

Sand filter systems are designed for intermittent flow and must be allowed to drain and reaerate between rainfall events. They should not be used on sites with a continuous flow from groundwater, sump pumps, or other sources.

B. General Design

Surface Sand Filter

A surface sand filter facility consists of a two-chamber open-air structure, which is located at ground-level. The first chamber is the sediment forebay (a.k.a sedimentation chamber) while the second chamber houses the sand filter bed. Flow enters the sedimentation chamber where settling of larger sediment particles occurs. Runoff is then discharged from the sedimentation chamber through a perforated standpipe into the filtration chamber. After passing though the filter bed, runoff is collected by a perforated pipe and gravel underdrain system. Figure 16.6 provides plan view and profile schematics of a surface sand filter.

Perimeter Sand Filter

A perimeter sand filter facility is a vault structure located just below grade level. Runoff enters the device through inlet grates along the top of the structure into the sedimentation chamber. Runoff is discharged from the sedimentation chamber through a weir into the filtration chamber. After passing through the filter bed, runoff is collected by a perforated pipe and gravel underdrain system. Figure 16.7 provides plan view and profile schematics of a perimeter sand filter.

C. Physical Specifications / Geometry

Surface Sand Filter

The entire treatment system (including the sedimentation chamber) must temporarily hold at least 75% of the WQ_v prior to filtration. Figure 16.2 illustrates the distribution of the treatment volume (0.75 WQ_v) among the various components of the surface sand filter, including:

- V_s – volume within the sedimentation basin
- V_f – volume within the voids in the filter bed
- $V_{f\text{-}temp}$ – temporary volume stored above the filter bed
- A_s – the surface area of the sedimentation basin
- A_f – surface area of the filter media
- h_s – height of water in the sedimentation basin
- h_{temp} - depth of temporary volume
- h_f – average height of water above the filter media (1/2 h_{temp})
- d_f – depth of filter media

The sedimentation chamber must be sized to at least 25% of the computed WQ_v and have a length-to-width ratio of at least 2:1. Inlet and outlet structures should be located at opposite ends of the chamber.

The filter area is sized based on the principles of Darcy's Law. A coefficient of permeability (k) of 3.5 ft/day for sand should be used. The filter bed is typically designed to completely drain in 40 hours or less.

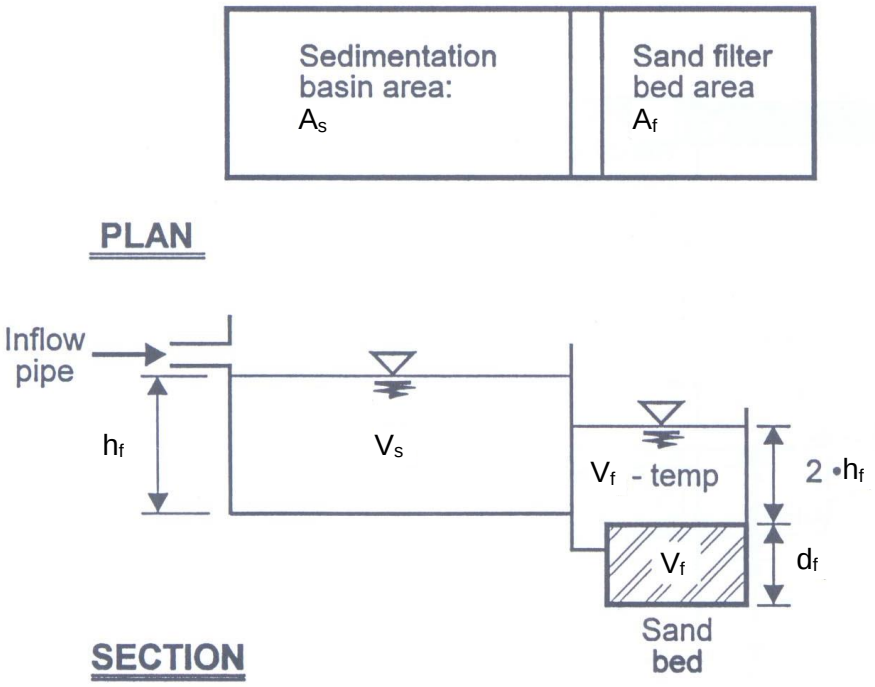


Figure 16.2 Surface Sand Filter Volumes
Source: Claytor and Schueler, 1996

The filter media consists of an 18-inch layer of clean washed medium sand (meeting ASTM C-33 concrete sand or TxDOT Fine Aggregate Grade No. 1) on top of the underdrain system. Three inches of topsoil are placed over the sand bed. Permeable filter fabric is placed both above and below the sand bed to prevent clogging of the sand filter and the underdrain system. A proper fabric selection is critical. Choose a filter fabric with equivalent pore openings as to prevent clogging by sandy filler material. Figure 16.4 illustrates a typical media cross section.

The filter bed is equipped with a 6-inch perforated PVC pipe (AASHTO M 252) underdrain in a gravel layer. The underdrain must have a minimum grade of 1/8-inch per foot (1% slope). Holes should be 3/8-inch diameter and spaced approximately 6 inches on center. Gravel should be clean washed aggregate with a maximum diameter of 3.5 inches and a minimum diameter of 1.5 inches with a void space of about 40% meeting the gradation listed below. Aggregate contaminated with soil shall not be used.

Gradation

Sieve Size	% Passing
2 ½"	100
2"	90 – 100
1 ½"	35 – 70
1"	0 – 15
½"	0 - 5

The structure of the surface sand filter may be constructed of impermeable media such as concrete, or through the use of excavations and earthen embankments. When constructed with earthen walls/embankments, filter fabric should be used to line the bottom and side slopes of the structures before installation of the underdrain system and filter media.

Perimeter Sand Filter

The entire treatment system (including the sedimentation chamber) must temporarily hold at least 75% of the WQ_v prior to filtration. Figure 16.3 illustrates the distribution of the treatment volume (0.75 WQ_v) among the various components of the perimeter sand filter, including:

- V_w – wet pool volume within the sedimentation basin
- V_f – volume within the voids in the filter bed
- V_{temp} – temporary volume stored above the filter bed
- A_s – the surface area of the sedimentation basin
- A_f – surface area of the filter media
- h_f – average height of water above the filter media (1/2 h_{temp})
- h_{temp} - depth of temporary volume
- d_f – depth of filter media

The sedimentation chamber must be sized to at least 50% of the computed WQ_v.

The filter area is sized based on the principles of Darcy's Law. A coefficient of permeability (k) of 3.5 ft/day for sand should be used. The filter bed is typically designed to completely drain in 40 hours or less.

The filter media should consist of a 12- to 18-inch layer of clean washed medium sand (meeting ASTM C-33 concrete sand or TxDOT Fine Aggregate Grade No. 1) on top of the underdrain system. Figure 16.4 illustrates a typical media cross section.

The perimeter sand filter is equipped with a 4 inch perforated PVC pipe (AASHTO M 252) underdrain in a gravel layer. The underdrain must have a minimum grade of 1/8 inch per foot (1% slope). Holes should be 3/8-inch diameter and spaced approximately 6 inches on center. A permeable filter fabric should be placed between the gravel layer and the filter media. Gravel should be clean washed aggregate with a maximum diameter of 3.5 inches and a minimum diameter of 1.5 inches with a void space of about 40% meeting the following gradation. Aggregate contaminated with soil shall not be used.

Gradation

Sieve Size	% Passing
2 ½"	100
2"	90 – 100
1 ½"	35 – 70
1"	0 – 15
½"	0 - 5

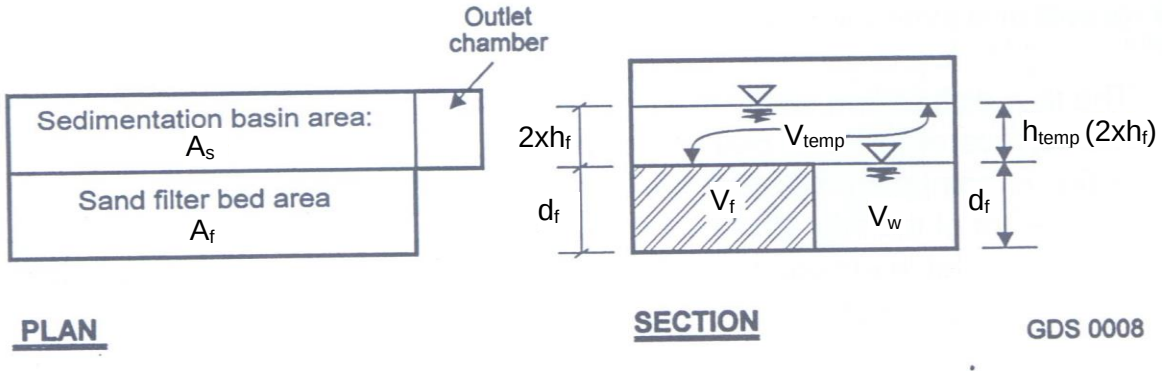


Figure 16.3 Perimeter Sand Filter Volumes
(Source: Claytor and Schueler, 1996)

D. Pretreatment / Inlets

Pretreatment of runoff in a sand filter system is provided by the sedimentation chamber.

Inlets to surface sand filters are to be provided with energy dissipaters. Exit velocities from the sedimentation chamber must be nonerosive.

Figure 16.5 shows a typical inlet pipe from the sedimentation basin to the filter media basin for the surface sand filter.

E. Outlet Structures

Outlet pipe is to be provided from the underdrain system to the facility discharge. Due to the slow rate of filtration, outlet protection is generally unnecessary (except for emergency overflows and spillways).

F. Emergency Spillway

An emergency or bypass spillway must be included in the surface sand filter to safely pass flows that exceed the design storm flows. The spillway prevents filter water levels from overtopping the embankment and causing structural damage. The emergency spillway should be located so that downstream buildings and structures will not be impacted by spillway discharges.

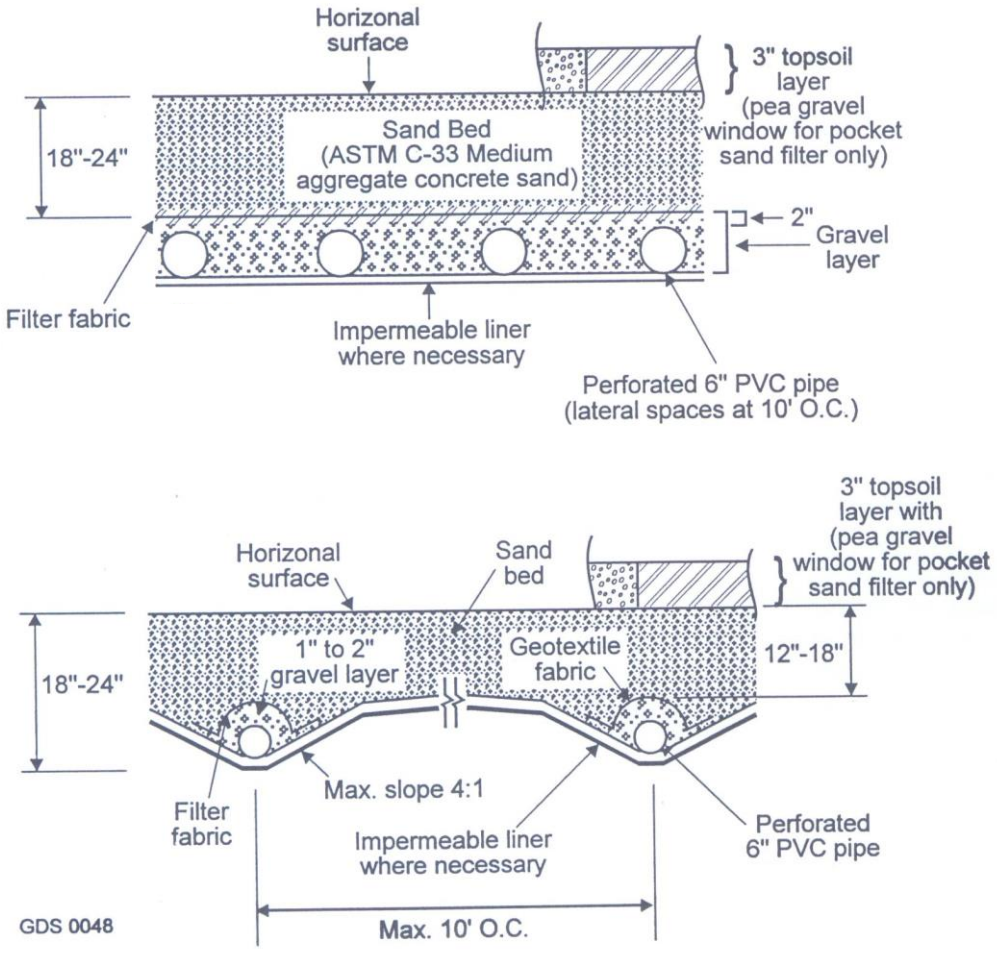


Figure 16.4 Typical Sand Filter Media Cross Sections
(Source: Claytor and Schueler, 1996)

G. Maintenance Access

Adequate access must be provided for all sand filter systems for inspection and maintenance, including the appropriate equipment and vehicles. Access grates to the filter bed need to be included in a perimeter sand filter design. Facility designs must enable maintenance personnel to easily replace upper layers of the filter media.

H. Safety Features

Surface sand filter facilities can be fenced to prevent access. Inlet and access grates to perimeter sand filters may be locked.

I. Landscaping

Surface filters can be designed with a grass cover to aid in pollutant removal and prevent clogging. The grass should be capable of withstanding frequent periods of inundation and drought.

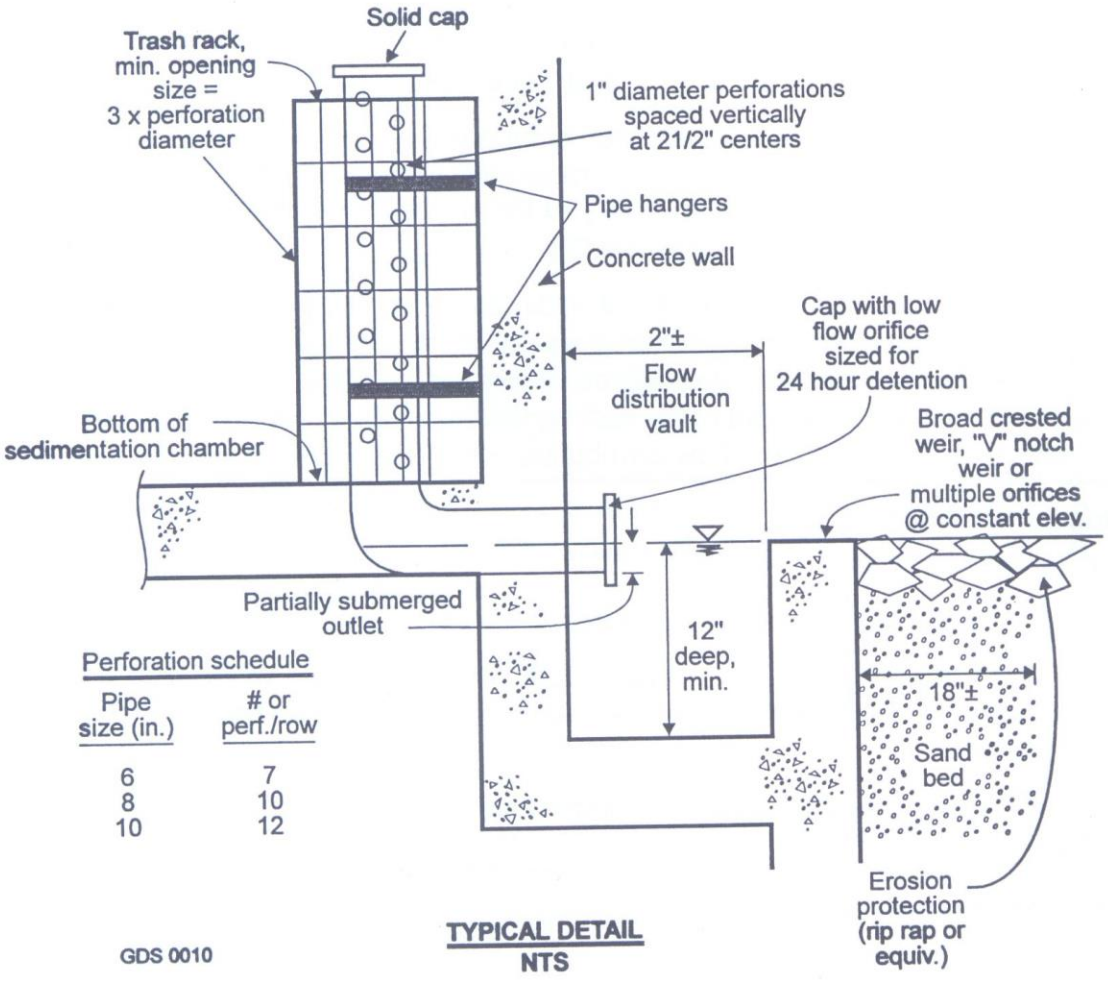


Figure 16.5 Surface Sand Filter Perforated Stand-Pipe
(Source: Claytor and Schueler, 1996)

J Additional Site-Specific Design Criteria and Issues

Physiographic Factors - Local terrain design constraints

- Low Relief – Use of surface sand filter may be limited by low head
- High Relief – Filter bed surface must be level
- Karst – Use polyliner or impermeable membrane to seal bottom of earthen surface sand filter or use watertight structure

Soils

- No restrictions

Special Downstream Watershed Considerations

- Stream Warming – Consideration should be given to the thermal influence on potential fish habitats downstream. If stream warming is significant, use shorter drain time (24 hours)
- Aquifer Protection – Use polyliner or impermeable membrane to seal bottom of earthen surface sand filter or use watertight structure; no infiltration of filter runoff into groundwater

16.6 Design Procedures

Step 1 Compute runoff control volumes from the *integrated* Design Focus Areas

Calculate the Water Quality Volume (WQ_v), Streambank Protection Volume (SP_v), On-Site Flood Control Volume (Q_p), and the Downstream Flood Control Volume (V_f).

Details on the *integrated* Design Focus Areas are found in *Section 1.0 of the Planning Technical Manual*.

Step 2 Determine if the development site and conditions are appropriate for the use of a surface or perimeter sand filter.

Consider the Application and Site Feasibility Criteria in *Sections 16.4 and16.5* (A) (Location and Siting).

Step 3 Confirm local design criteria and applicability

Consider any special site-specific design conditions/criteria from *Section 16.5* (J) (Additional Site-Specific Design Criteria and Issues).

Check with local officials and other agencies to determine if there are any additional restrictions and/or surface water or watershed requirements that may apply.

Step 4 Compute WQ_v peak discharge (Q_{wq})

The peak rate of discharge for water quality design storm is needed for sizing of off-line diversion structures (see *Section 1.0 of the Water Quality Technical Manual*).

(a) Using WQ_v, compute CN
(b) Compute time of concentration using TR-55 method
(c) Determine appropriate unit peak discharge from time of concentration
(d) Compute Q_{wq} from unit peak discharge, drainage area, and WQ_v.

Step 5 Size flow diversion structure, if needed

A flow regulator (or flow splitter diversion structure) should be supplied to divert the WQ_v to the sand filter facility.

Size low flow orifice, weir, or other device to pass Q_{wq}.

Step 6 Size filtration basin chamber

The filter area is sized using the following equation (based on Darcy's Law, Equation 2.1):

$$\mathbf{A_f = (WQ_v)\,(d_f) / [(k)\,(h_f + d_f)\,(t_f)]}$$

where:

- A_f = surface area of filter bed (ft^2)
- d_f = filter bed depth (typically 18 inches, no more than 24 inches)
- k = coefficient of permeability of filter media (ft/day) (use 3.5 ft/day for sand)
- h_f = average height of water above filter bed (ft) (1/2 h_{max}, which varies based on site but h_{max} is typically ≤ 6 feet)
- t_f = design filter bed drain time (days) (1.67 days or 40 hours is recommended maximum)

Set preliminary dimensions of filtration basin chamber.

See *Section 16.5* (C) (Physical Specifications/Geometry) for filter media specifications.

Step 7 Size sedimentation chamber

***Surface sand filter*:** The sedimentation chamber should be sized to at least 25% of the computed WQ_v and have a length-to-width ratio of 2:1. The Camp-Hazen equation is used to compute the required surface area:

$$\mathbf{A_s = -(Q_o/w) * Ln\,(1-E)} \qquad \mathbf{(16.1)}$$

where:

- A_s = sedimentation basin surface area (ft^2)
- Q_o = rate of outflow = the WQ_v over a 24-hour period
- w = particle settling velocity (ft/sec)
- E = trap efficiency

Assuming:

- 90% sediment trap efficiency (0.9)
- particle settling velocity (ft/sec) = 0.0033 ft/sec for imperviousness < 75%
- particle settling velocity (ft/sec) = 0.0004 ft/sec for imperviousness ≥ 75%
- average of 24 hour holding period

Then:

- A_s = (0.066) (WQ_v) ft^2 for I < 75%
- A_s = (0.0081) (WQ_v) ft^2 for I ≥ 75%

Set preliminary dimensions of sedimentation chamber.

***Perimeter sand filter*:** The sedimentation chamber should be sized to at least 50% of the computed WQ_v. Use same approach as for surface sand filter.

Step 8 Compute V_{min}

$V_{min} = 0.75 * WQ_v$

Step 9 Compute storage volumes within entire facility and sedimentation chamber orifice size

***Surface sand filter*:**

$$\mathbf{V_{min} = 0.75\,WQ_v = V_s + V_f + V_{f\text{-}temp}} \qquad \mathbf{(16.2)}$$

(1) Compute V_f = water volume within filter bed/gravel/pipe = $A_f * d_f * n$
where: n = porosity = 0.4 for most applications

(2) Compute $V_{f\text{-}temp}$ = temporary storage volume above the filter bed = $2 * h_f * A_f$

(3) Compute V_s = volume within sediment chamber = $V_{min} - V_f - V_{f\text{-}temp}$

(4) Compute h_s = height in sedimentation chamber = V_s/A_s
(5) Ensure h_s and h_f fit available head and other dimensions still fit – change as necessary in design iterations until all site dimensions fit.
(6) Size orifice from sediment chamber to filter chamber to release V_s within 24-hours at average release rate with 0.5 h_s as average head.
(7) Design outlet structure with perforations allowing for a safety factor of 10 (see example)
(8) Size distribution chamber to spread flow over filtration media – level spreader weir or orifices.

***Perimeter sand filter*:**

(1) Compute V_f = water volume within filter bed/gravel/pipe = $A_f * d_f * n$
where: n = porosity = 0.4 for most applications
(2) Compute V_w = wet pool storage volume $A_s * 2$ feet minimum
(3) Compute V_{temp} = temporary storage volume = $V_{min} – (V_f + V_w)$
(4) Compute h_{temp} = temporary storage height = $V_{temp} / (A_f + A_s)$
(5) Ensure $h_{temp} \geq 2 * h_f$, otherwise decrease h_f and re-compute. Ensure dimensions fit available head and area – change as necessary in design iterations until all site dimensions fit.
(6) Size distribution slots from sediment chamber to filter chamber.

Step 10 Design inlets, pretreatment facilities, underdrain system, and outlet structures

See *Section 16.5* (D) through (H) for more details.

Step 11 Compute overflow weir sizes

***Surface sand filter*:**

(1) Size overflow weir at elevation h_s in sedimentation chamber (above perforated stand pipe) to handle surcharge of flow through filter system from storms producing more than 1.5 inches (see example in *Section 29.3*).

(2) Plan inlet protection for overflow from sedimentation chamber and size overflow weir at elevation h_f in filtration chamber (above perforated stand pipe) to handle surcharge of flow through filter system from storms producing more than 1.5 inches (see example).

Perimeter sand filter: Size overflow weir at end of sedimentation chamber to handle excess inflow, set at WQ_v elevation.

See *Section 29.3* for a Sand Filter Design Example

16.7 Inspection and Maintenance Requirements

Table 16.1 Typical Maintenance Activities for Sand Filters
(Source: WMI, 1997; Pitt, 1997)

Activity	Schedule
• Ensure that contributing area, facility, inlets, and outlets are clear of debris. • Ensure that the contributing area is stabilized and mowed, with clippings removed. • Remove trash and debris. • Check to ensure that the filter surface is not clogging (also check after moderate and major storms). • Ensure that activities in the drainage area minimize oil/grease and sediment entry to the system. • If permanent water level is present (perimeter sand filter), ensure that the chamber does not leak and normal pool level is retained.	Monthly
• Check to see that the filter bed is clean of sediment, and the sediment chamber is not more than 50% full or 6 inches, whichever is less, of sediment. Remove sediment as necessary. • Stabilize disturbed area contributing to the heavy sediment load. • Make sure that there is no evidence of deterioration, spalling, or cracking of concrete. • Inspect grates (perimeter sand filter). • Inspect inlets, outlets, and overflow spillway to ensure good condition and no evidence of erosion. • Repair or replace any damaged structural parts. • Stabilize any eroded areas. • Ensure that flow is not bypassing the facility. • Ensure that no noticeable odors are detected outside the facility.	Annually
• If filter bed is clogged or partially clogged, manual manipulation of the surface layer of sand may be required. Remove the top few inches of sand, roto-till or otherwise cultivate the surface, and replace media with sand meeting the design specifications. • Replace any filter fabric that has become clogged.	As needed

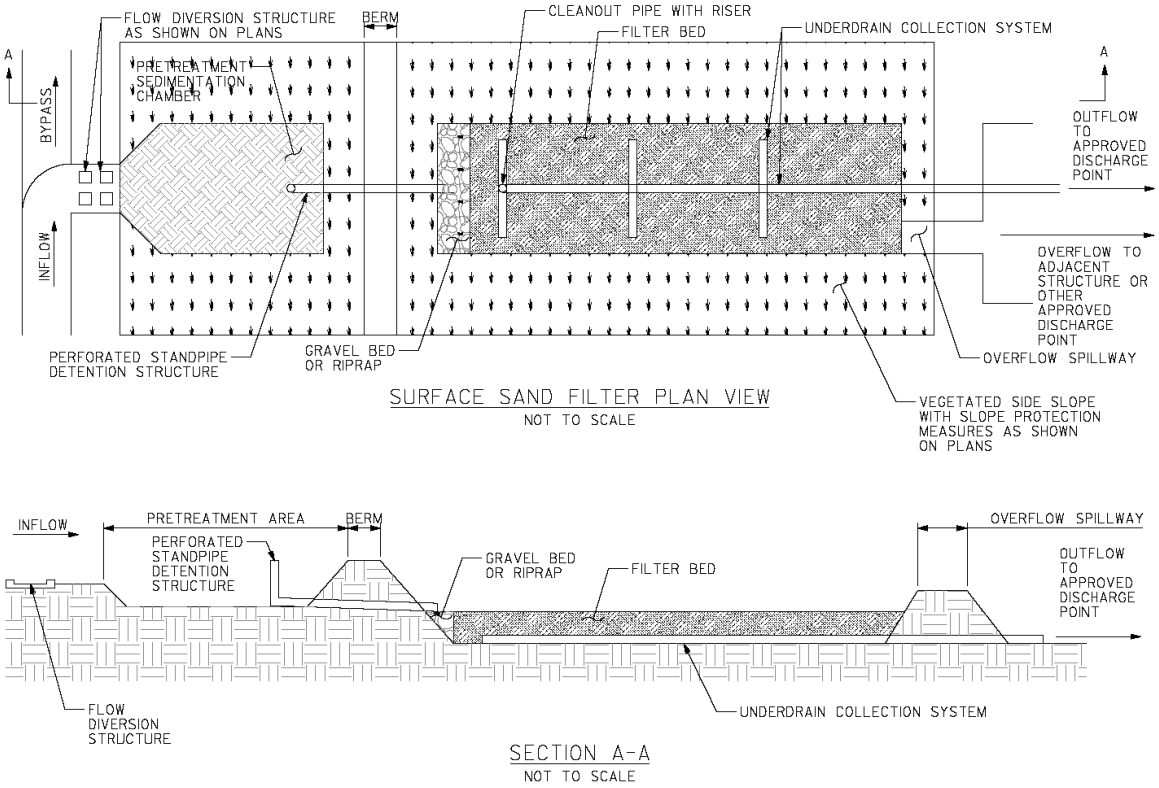
Additional Maintenance Considerations and Requirements

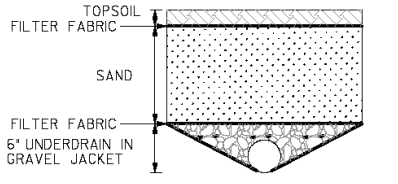
- A record should be kept of the dewatering time for a sand filter to determine if maintenance is necessary.
- When the filtering capacity of the sand filter facility diminishes substantially (i.e., when water ponds on the surface of the filter bed for more than 48 hours), then the top layers of the filter media (topsoil and 2 to 3 inches of sand) will need to be removed and replaced. This will typically need to be done every 3 to 5 years for low sediment applications, more often for areas of high sediment yield or high oil and grease.
- Removed sediment and media may usually be disposed of in a landfill.



Regular inspection and maintenance is critical to the effective operation of sand filter facilities as designed. Maintenance responsibility for a sand filter system should be vested with a responsible authority by means of a legally binding and enforceable maintenance agreement that is executed as a condition of plan approval.

16.8 Example Schematics





NOTES:

1. DIMENSIONS SHALL BE PROVIDED BY DESIGNER IN PLANS AND SHALL BE BASED ON SITE SPECIFIC CONDITIONS. DESIGN PROFESSIONAL SHALL AT A MINIMUM PROVIDE THE FOLLOWING INFORMATION:
   - HORIZONTAL CONTROL
   - VERTICAL CONTROL
   - WATER QUALITY PROTECTION VOLUME
   - STREAMBANK PROTECTION VOLUME
   - FLOOD CONTROL VOLUME
   - RIPRAP INFORMATION
   - FLOW DIVERSION STRUCTURE DETAIL(S)
   - DRAINAGE AGGREGATE AND UNDERDRAIN INFORMATION

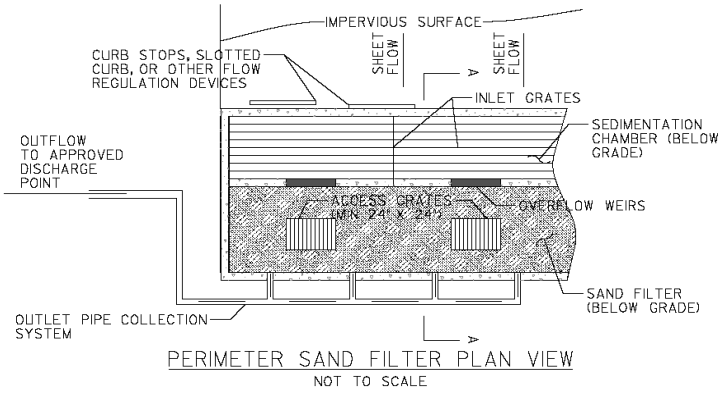
2. DESIGN ENGINEER IS RESPONSIBLE FOR PROVIDING CALCULATIONS THAT DEMONSTRATE THAT COMPOSITE MEDIA AND SYSTEM AS A WHOLE CAN EFFECTIVELY TREAT THE WATER QUALITY VOLUME (WQv) AS SPECIFIED IN SECTION 1.4 OF THE NCTCOG WATER QUALITY TECHNICAL MANUAL. FLOW IN EXCESS OF THE WQV SHALL BE DESIGNED TO BYPASS THE SYSTEM VIA THE OVERFLOW SYSTEM(S).

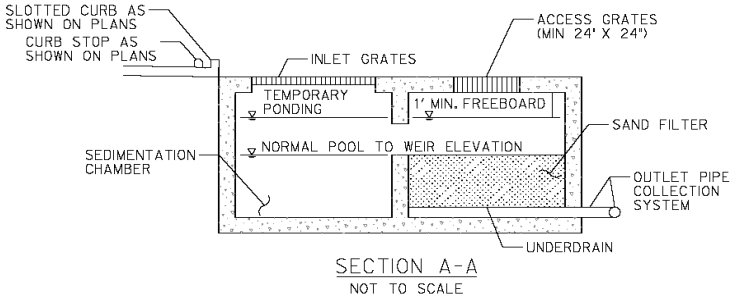
3. DESIGN OF THIS SYSTEM SHALL FOLLOW THAT OF SECTION 16 "SAND FILTERS" CONTAINED IN THE NCTCOG SITE DEVELOPMENT CONTROLS ISWM TECHNICAL MANUAL.

4. DESIGNER SHALL ENSURE THAT SYSTEM IS CAPABLE OF BEING FULLY DRAINED WITHIN 48-HOURS.

5. DESIGNER SHALL BE RESPONSIBLE FOR PREPARING AN OPERATIONS AND MAINTENANCE MANUAL OR MATRIX FOR SYSTEMS TO AID OWNER IN DETERMINING MAINTENANCE PROCEEDURES AND INTERVALS. DESIGNER SHALL ALSO CONSIDER MAINTENANCE ACCESS WHEN DESIGNING SYSTEM.

Figure 16.6 Schematic of Surface Sand Filter
(Source: Center for Watershed Protection)





NOTES:

1. DIMENSIONS SHALL BE PROVIDED BY DESIGNER IN PLANS AND SHALL BE BASED ON SITE SPECIFIC CONDITIONS. DESIGN PROFESSIONAL SHALL AT A MINIMUM PROVIDE THE FOLLOWING INFORMATION:
- HORIZONTAL CONTROL
- VERTICAL CONTROL
- WATER QUALITY PROTECTION VOLUME
- STREAMBANK PROTECTION VOLUME
- FLOOD CONTROL VOLUME
- CHAMBER STRUCTURAL AND ACCESS INFORMATION
- OUTLET PIPE INFORMATION

2. DESIGN ENGINEER IS RESPONSIBLE FOR PROVIDING CALCULATIONS THAT DEMONSTRATE THAT SYSTEM AS A WHOLE CAN EFFECTIVELY TREAT THE WATER QUALITY VOLUME (WQv) AS SPECIFIED IN SECTION 1.4 OF THE NCTCOG WATER QUALITY TECHNICAL MANUAL. FLOW IN EXCESS OF THE WQV SHALL BE DESIGNED TO BYPASS THE SYSTEM VIA THE OVERFLOW SYSTEM(S).

3. DESIGN OF THIS SYSTEM SHALL FOLLOW THAT OF SECTION 16 "SAND FILTERS" CONTAINED IN THE NCTCOG SITE DEVELOPMENT CONTROLS ISWM TECHNICAL MANUAL.

4. DESIGNER SHALL BE RESPONSIBLE FOR PREPARING AN OPERATIONS AND MAINTENANCE MANUAL OR MATRIX FOR SYSTEMS TO AID OWNER IN DETERMINING MAINTENANCE PROCEEDURES AND INTERVALS. DESIGNER SHALL ALSO CONSIDER MAINTENANCE ACCESS WHEN DESIGNING SYSTEM.

Figure 16.7 Schematic of Perimeter Sand Filter
(Source: Center for Watershed Protection)

16.9 Design Forms

Design Procedure Form: Sand Filters

PRELIMINARY HYDROLOGIC

1a.	Compute WQ_v volume requirements	
	Copute Runoff Coefficient, R_v	R_v = _____
	Compute WQ_v	WQ_v = _____ acre-ft
1b.	Compute SP_v	SP_v = _____ acre-ft
	Copute average release rate	release rate = _____ cfs
	Compute Q_p (100-year detention volume required)	Q_p = _____ acre-ft
	Compute (as necessary) Q_f	Q_f = _____ cfs

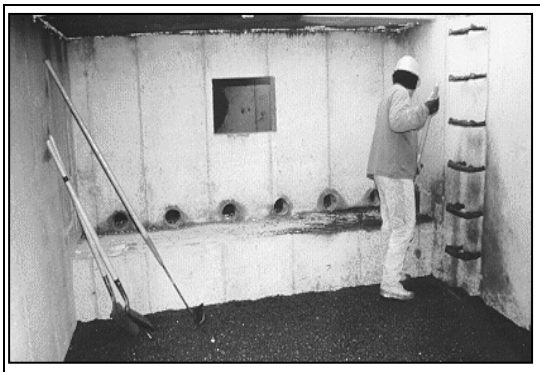
SAND FILTER DESIGN

2.	Is the use of a sand fiilter appropriate?	Low Point in development area = _____ Low Point at stream invert = _____ Total available head = _____ Average depth, h_f = _____
		See subsections 5.2.15.4 and 5.2.15.5-A
3.	Confirm localdesign criteria and applicability.	***See subsection 5.2.15.5-J***
4.	Compute WQ_v peak discharge (Q_{vq})	
	Compute Curve Number	CN = _____
	Compute Time of Concentration t_c	t_c = _____ hour
	Compute Q_{wq}	Q_{wq} = _____ cfs
5.	Size flow diversion structure	
	Low flow orifice - Orifice equation	A = _____ ft^2
		diameter = _____ in
	Overflow weir - Weir equation	Length = _____ ft
6.	Size filtration bed chamber	
	Compute area from Darcy's Law	A_f = _____ ft^2
	Using length to width (2:1) ratio	L = _____ ft
		W = _____ ft
7.	Size seidmentation chamber	
	Compute area from Camp-Hazen equation	A_f = _____ ft^2
	Given W from step 5, compute Length	L = _____ ft
8.	Compute V_{min}	V_{min} = _____ ft^3
9.	Compute volume within practice	
	<u>Surface sand filter</u>	
	Volume within filter bed	V_f = _____ ft^3
	Temporary storage above filter bed	$V_{s\text{-}temp}$ = _____ ft^3
	Sedimentation chamber (remaining volume)	V_s = _____ ft^3
	Height in sedimentation chamber	h_s = _____ ft
	Perforated stand pipe - Orifice equation	A = _____ ft^2
		diameter = _____ in
	<u>Perimeter sand filter</u>	
	Compute volume in filter bed	V_f = _____ ft^3
	Compute wet pool storage	V_w = _____ ft^3
	Compute temporary storage	V_{temp} = _____ ft^3
		h_{temp} = _____ ft
10.	Compute overflow weir sizes	
	Compute overflow - Orifice equation	Q = _____ cfs
	Weir from sedimentation chamber - Weir equation	Length = _____ ft
	Weir from filtration chamber - Weir equation	Length = _____ ft

Notes:

17.0 Underground Sand Filter

Limited Application
Structural Stormwater Control



Description: Design variant of the sand filter located in an underground vault.

KEY CONSIDERATIONS

DESIGN CRITERIA:

- Intended for space-limited applications

ADVANTAGES / BENEFITS:

- High pollutant removal capability
- High removal rates for sediment, BOD, and fecal coliform bacteria
- Precast concrete shells available, which decrease construction costs

DISADVANTAGES / LIMITATIONS:

- High maintenance requirements

MAINTENANCE REQUIREMENTS:

- Filter may require more frequent maintenance than most of the other stormwater controls

POLLUTANT REMOVAL

80%	**Total Suspended Solids**
50/25%	**Nutrients** - Total Phosphorus / Total Nitrogen removal
50%	**Metals** - Cadmium, Copper, Lead, and Zinc removal
40%	**Pathogens** - Coliform, Streptococci, E.Coli removal

STORMWATER MANAGEMENT SUITABILITY

P	**Water Quality Protection**
	Streambank Protection
	On-Site Flood Control
	Downstream Flood Control

IMPLEMENTATION CONSIDERATIONS

L	**Land Requirement**
H	**Capital Cost**
H	**Maintenance Burden**

Residential Subdivision Use: No
High Density/Ultra-Urban: Yes
Drainage Area: 5 acres max.
Soils: No restrictions
Other Considerations:

- *Hotspot areas*

L=Low **M**=Moderate **H**=High

17.1 General Description

The underground sand filter is a design variant of the sand filter located in an underground vault designed for high-density land use or ultra-urban applications where there is not enough space for a surface sand filter or other structural stormwater controls.

The underground sand filter is a three-chamber system. The initial chamber is a sedimentation (pretreatment) chamber that temporarily stores runoff and utilizes a wet pool to capture sediment. The sedimentation chamber is connected to the sand filter chamber by a submerged wall that protects the filter bed from oil and trash. The filter bed is 18 to 24 inches deep and may have a protective screen of gravel or permeable geotextile to limit clogging. The sand filter chamber also includes an underdrain system with inspection and clean out wells. Perforated drain pipes under the sand filter bed extend into a third chamber that collects filtered runoff. Flows beyond the filter capacity are diverted through an overflow weir.

Due to its location below the surface, underground sand filters have a high maintenance burden and should only be used where adequate inspection and maintenance can be ensured.

17.2 Pollutant Removal Capabilities

Underground sand filter pollutant removal rates are similar to those for surface and perimeter sand filters (see *Section 16.0*).

17.3 Design Criteria and Specifications

- Underground sand filters are typically used on highly impervious sites of 1 acre or less. The maximum drainage area that should be treated by an underground sand filter is 5 acres.
- Underground sand filters are typically constructed on-line, but can be constructed off-line. For off-line construction, the overflow between the second and third chambers is not included.
- The underground vault should be tested for water tightness prior to placement of filter layers.
- Adequate maintenance access must be provided to the sedimentation and filter bed chambers.
- Compute the minimum wet pool volume required in the sedimentation chamber as:

  $V_w = A_s$ * 3 feet minimum **(17.1)**

- Consult the design criteria for the perimeter sand filter (see *Section 16.0*) for the rest of the underground filter sizing and design steps.

17.4 Inspection and Maintenance Requirements

Table 17.1 Typical Maintenance Activities for Underground Sand Filters
(Source: CWP, 1996)

Activity	Schedule
Monitor water level in sand filter chamber.	Quarterly and following large storm events
Sedimentation chamber should be cleaned out when the sediment depth reaches 12 inches.	As needed
Remove accumulated oil and floatables in sedimentation chamber.	As needed, (typically every 6 months)

As a variant, organic material may be used instead of the sand media in the underground filter. Organic material has a higher cation exchange capacity and may remove more metals and other charged pollutants. Additional inspection and maintenance requirements for organic filters are similar to those for surface sand filter facilities (see *Section 16.0*)

17.5 Example Schematic

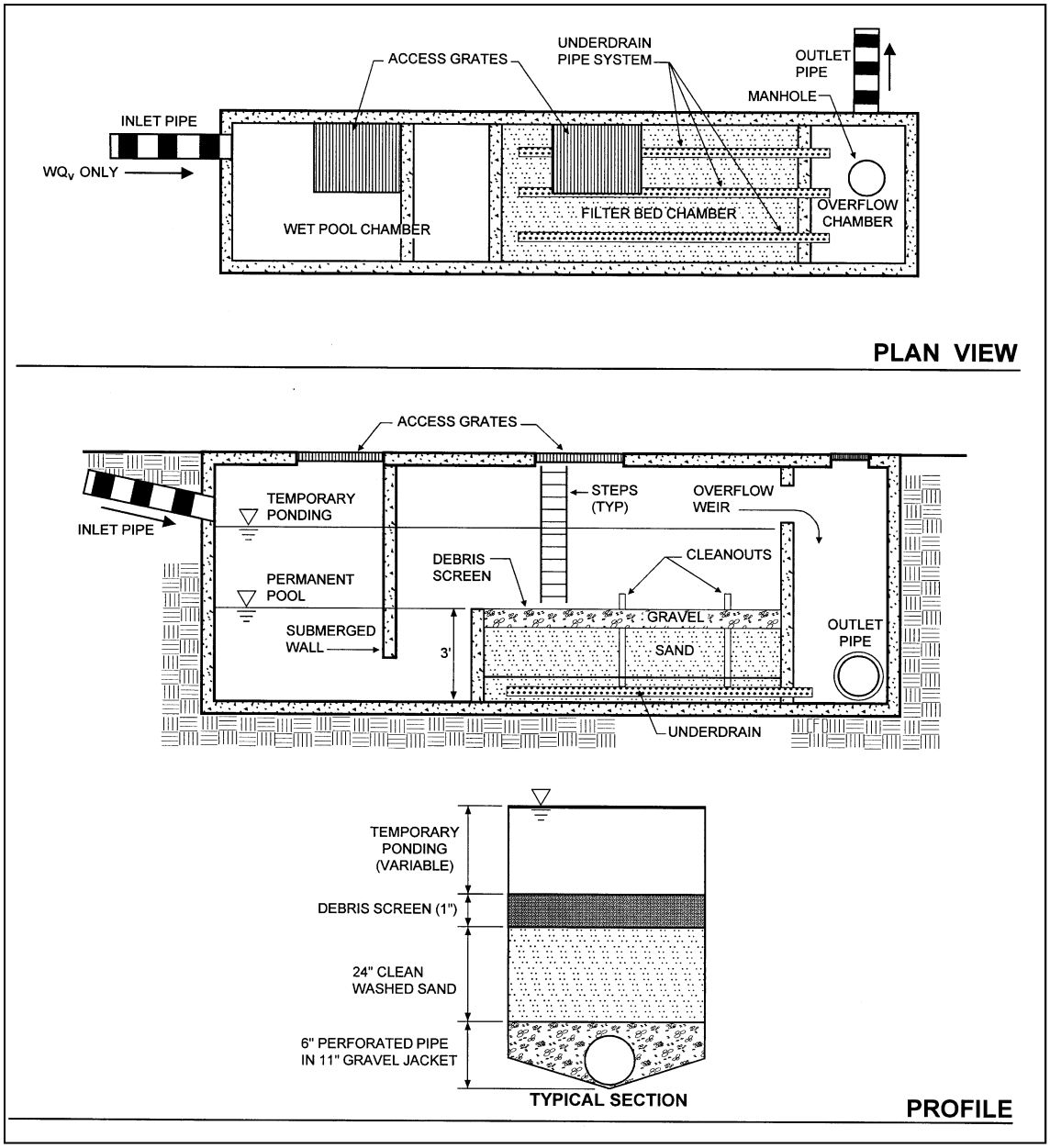
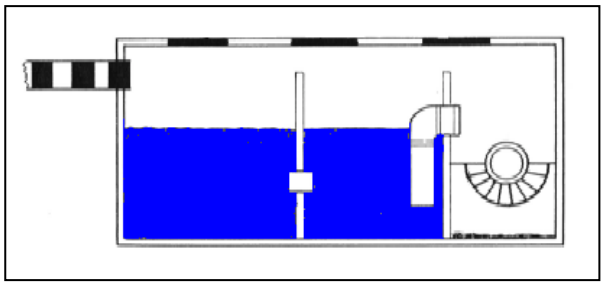


Figure 17.1 Schematic of Underground Sand Filter
(Source: Center for Watershed Protection)

18.0 Gravity (Oil Grit) Separator

Structural Stormwater Control



Description: Hydrodynamic separation device designed to remove settleable solids, oil and grease, debris, and floatables from stormwater runoff through gravitational settling and trapping of pollutants.

KEY CONSIDERATIONS

DESIGN CRITERIA:

- Intended for hotspot, space-limited, or pretreatment applications
- Intended for the removal of settleable solids (grit and sediment) and floatable matter, including oil and grease
- Performance dependent on design and frequency of inspection and cleanout of unit

DISADVANTAGES / LIMITATIONS:

- Cannot alone achieve the 80% TSS removal target
- Limited performance data
- Dissolved pollutants are not effectively removed

MAINTENANCE REQUIREMENTS:

- Frequent maintenance required

POLLUTANT REMOVAL

80%	**Total Suspended Solids**
50/25%	**Nutrients** - Total Phosphorus / Total Nitrogen removal
50%	**Metals** - Cadmium, Copper, Lead, and Zinc removal
40%	**Pathogens** - Coliform, Streptococci, E.Coli removal

STORMWATER MANAGEMENT SUITABILITY

S	**Water Quality Protection**
	Streambank Protection
	On-Site Flood Control
	Downstream Flood Control

IMPLEMENTATION CONSIDERATIONS

L	**Land Requirement**
H	**Capital Cost**
H	**Maintenance Burden**

Residential Subdivision Use: No
High Density/Ultra-Urban: Yes
Drainage Area: 1 acres max.
Soils: No restrictions
Other Considerations:

- Hotspot areas
- Pretreatment

L=Low **M**=Moderate **H**=High

18.1 General Description

Gravity separators (also known as oil-grit separators) are hydrodynamic separation devices that are designed to remove grit and heavy sediments, oil and grease, debris, and floatable matter from stormwater runoff through gravitational settling and trapping. Gravity separator units contain a permanent pool of water and typically consist of an inlet chamber, separation/storage chamber, a bypass chamber, and an access port for maintenance purposes. Runoff enters the inlet chamber where heavy sediments and solids drop out. The flow moves into the main gravity separation chamber, where further settling of suspended solids takes place. Oil and grease are skimmed and stored in a waste oil storage compartment for future removal. After moving into the outlet chamber, the clarified runoff is then discharged.

The performance of these systems is based primarily on the relatively low solubility of petroleum products in water and the difference between the specific gravity of water and the specific gravities of petroleum compounds. Gravity separators are not designed to separate other products such as solvents, detergents, or dissolved pollutants. The typical gravity separator unit may be enhanced with a pretreatment swirl concentrator chamber, oil draw-off devices that continuously remove the accumulated light liquids, and flow control valves regulating the flow rate into the unit.

Gravity separators are best used in commercial, industrial, and transportation land uses and are intended primarily as a pretreatment measure for high-density or ultra urban sites, or for use in hydrocarbon hotspots, such as gas stations and areas with high vehicular traffic. However, gravity separators cannot be used for the removal of dissolved or emulsified oils and pollutants such as coolants, soluble lubricants, glycols, and alcohols.

Since re-suspension of accumulated sediments is possible during heavy storm events, gravity separator units are typically installed off-line. Gravity separators are available as prefabricated proprietary systems from a number of different commercial vendors.

18.2 Pollutant Removal Capabilities

Testing of gravity separators has shown that they can remove between 40 and 50% of the TSS loading when used in an off-line configuration (Curran, 1996 and Henry, 1999). Gravity separators also provide removal of debris, hydrocarbons, trash and other floatables. They provide only minimal removal of nutrients and organic matter.

The following design pollutant removal rates are conservative average pollutant reduction percentages for design purposes derived from sampling data, modeling and professional judgment.

- Total Suspended Solids – 40%
- Total Phosphorus – 5%
- Total Nitrogen – 5%
- Fecal Coliform – insufficient data
- Heavy Metals – insufficient data

Actual field testing data and pollutant removal rates from an independent source should be obtained before using a proprietary gravity separator system.

18.3 Design Criteria and Specifications

The use of gravity (oil-grit) separators should be limited to the following applications:

- Pretreatment for other structural stormwater controls
- High-density, ultra urban or other space-limited development sites
- Hotspot areas where the control of grit, floatables, and/or oil and grease are required

Gravity separators are typically used for areas less than 5 acres. It is recommended that the contributing area to any individual gravity separator be limited to 1 acre or less of impervious cover.

Gravity separator systems can be installed in almost any soil or terrain. Since these devices are underground, appearance is not an issue and public safety risks are low.

Gravity separators are rate-based devices. This contrasts with most other stormwater structural controls, which are sized based on capturing and treating a specific volume.

Gravity separator units are typically designed to bypass runoff flows in excess of the design flow rate. Some designs have built-in high flow bypass mechanisms. Other designs require a diversion structure or flow splitter ahead of the device in the drainage system. An adequate outfall must be provided.

The separation chamber should provide for three separate storage volumes:

- A volume for separated oil storage at the top of the chamber
- A volume for settleable solids accumulation at the bottom of the chamber
- A volume required to give adequate flow-through detention time for separation of oil and sediment from the stormwater flow

The total wet storage of the gravity separator unit should be at least 400 cubic feet per contributing impervious acre.

The minimum depth of the permanent pools should be 4 feet.

Horizontal velocity through the separation chamber should be 1 to 3 ft/min or less. No velocities in the device should exceed the entrance velocity.

A trash rack should be included in the design to capture floating debris, preferably near the inlet chamber to prevent debris from becoming oil impregnated.

Ideally, a gravity separator design will provide an oil draw-off mechanism to a separate chamber or storage area.

Adequate maintenance access to each chamber must be provided for inspection and cleanout of a gravity separator unit.

Gravity separator units should be watertight to prevent possible groundwater contamination.

The design criteria and specifications of a proprietary gravity separator unit should be obtained from the manufacturer.

18.4 Inspection and Maintenance Requirements

Table 18.1 Typical Maintenance Activities for Gravity Separators

Activity	Schedule
• Inspect the gravity separator unit for structural problems, accumulated pollutants, and mosquito larvae.	Regularly (quarterly)
• Clean out sediment, oil and grease, and floatables, using catch basin cleaning equipment (vacuum pumps). Manual removal of pollutants may be necessary.	As Needed

Additional Maintenance Considerations and Requirements

Additional maintenance requirements for a proprietary system should be obtained from the manufacturer.

Failure to provide adequate inspection and maintenance can result in the re-suspension of accumulated solids. Frequency of inspection and maintenance is dependent on land use, climatological conditions, and the design of gravity separator.

Proper disposal of oil, solids, and floatables removed from the gravity separator must be ensured.

If mosquito larvae are present in the unit, treat with larvacide. (See *Section 19.4*)

18.5 Example Schematic

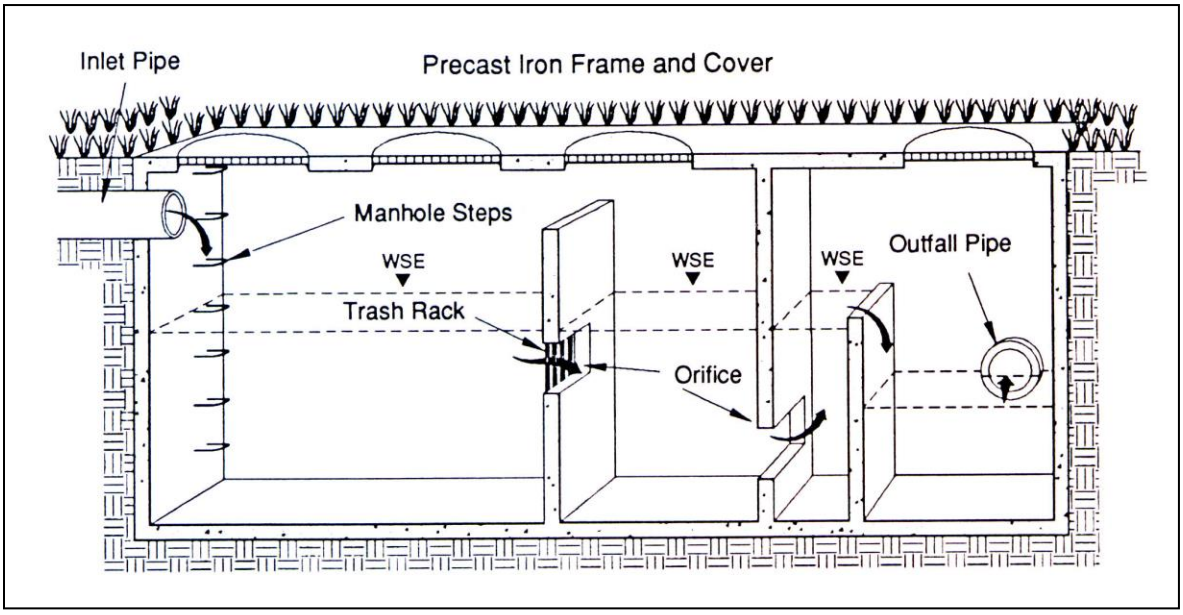


Figure 18.1 Schematic of an Example Gravity (Oil-Grit) Separator
(Source: NVRC, 1992[1])

19.0 Downspout Drywell

Structural Stormwater Control

North Central Texas Council of Governments

Integrated Stormwater Management iSWM

Description

Downspout drywells are essentially perforated manholes that can be manufactured in a variety of sizes. This BMP is used underground and allows for infiltration even in heavily urbanized areas.



Downspout Drywell in Traverse City, MI. *(Source: Tetra Tech)*

Design Considerations

- Downspout drywells should be utilized in conjunction with pretreatment devices.
- Since infiltration is a key component of the wells, they should be used in areas with minimal risk of groundwater contamination.
- Downspout drywells are intended to be used in applications with space limitations.
- Should not be used in areas with high sediment loads to minimize clogging.
- Pervious soils (over 0.5 inches per hour) are required for the infiltration process.

Key Advantages

- Pollutant removal can be obtained through filtration into adjacent soils.
- Peak flow rates are decreased through the use of a downspout drywell.
- Easily adaptable to the space restrictions and treatment requirements.

Limitations

- Since downspout drywells utilize injection via a subsurface structure, special permits may be required.
- Downspout drywells should not be used in areas near drinking water wells, high groundwater tables, or areas with industrial usage.
- Maintenance may require access limitations and therefore OSHA permits.

Target Constituent	Removal Rate (0% → 100%)
Total Suspended Solids	
Total Phosphorus	
Total Nitrogen	
Fecal Coliform	
Heavy Metals	

Implementation Considerations



Suitability

The iSWM manual has designated downspout drywells as suitable for providing:

Water Quality Protection

Maintenance

- Clean out sediment and debris from the drywell and any pretreatment devices.
- Perform pest control if rodents, mosquitos, or other vectors are found.
- Ensure that waterlogged soils do not exist in the vicinity of the downspout drywell.
- Clear pipes when clogging occurs.

Published October 2020

19.1 General Description

Drywells are infiltration devices that have historically been used to dispose of excess runoff without extensive infrastructure. Its minimal land requirements allow it to be used in highly urbanized areas. Drywells used for stormwater disposal are considered Class V injection devices by the EPA and fall under the Texas UIC program. Concerns about contaminating aquifers limit their application to "clean" runoff, such as roofdrains, and require pretreatment devices to remove sediments and other pollutants.

Drywells should not be used in areas near drinking water wells, with industrial land use, with high groundwater tables, a substrate of fractured rock, or slow-draining soils. Drywell design should be overseen by a licensed engineer.

19.2 Pollutant Removal Capabilities

Pollutant removal is similar to infiltration trenches (see *Section 20.0*), but care should be taken to avoid clogging with sediments.

- Total Suspended Solids – 80%
- Total Phosphorus – 60%
- Total Nitrogen – 60%
- Fecal Coliform – 90%
- Heavy Metals – 90%

19.3 Design Criteria and Specifications

The drywell should be located at least 5 feet from the nearest property line and 10 feet away from an occupied building.

Drywells shall be located at least 200 feet from the tops of slopes more than 10 feet high and steeper than 2h:1v.

The drywell shall be excavated in native soil, uncompacted by heavy equipment.

A qualified professional shall conduct infiltration testing. The surrounding soil should have a minimum infiltration rate of 0.5 inches per hour.

The drywell shall be surrounded by a 12 inch thick layer of ¾" to 2 ½" round rock.

There should be at least four feet between the bottom of the drywell and the seasonal high ground water table or bedrock.

A pretreatment device should be installed upstream of the drywell to remove sediments and other pollutants.

The drywell shall be sized in accordance with the simplified sizing criteria.

The drywell should not be located next to trees, since roots may penetrate drywell and clog it.

Access should be provided for drywell maintenance via a secured manhole or cleanout.

19.4 Inspection and Maintenance Requirements

The inspection and maintenance requirements for drywells are designed to maintain an adequate drainage rate through the drywell, while avoiding groundwater contamination.

Table 19.1 Typical Maintenance Activities for Drywells	
Activity	**Schedule**
Ensure that inflow is unimpeded. Clean out accumulated sediment/ debris and dispose of properly.	Quarterly and within 48 hours of major storms
Inspect pretreatment device and clean if necessary. Cleaning shall be done without the used of detergents or solvents.	As needed, based on minimum annual inspection
Inspect area surrounding the drywell for waterlogged soils at surface, indicating drywell failure. Clogged drywells must be replaced.	Inspect between 24 - 48 hours after major storms
Pest control measures shall be taken if rodents or mosquitoes are found to be present. Holes in the ground around the drywell shall be filled and a low toxicity mosquito larvacide, such as Bacillus thuringiensis (Bti), Bacillus Sphearicus (Bsph) or Methoprene (insect growth regulator) applied by a licensed individual, if necessary.	As needed

19.5 Example Schematic

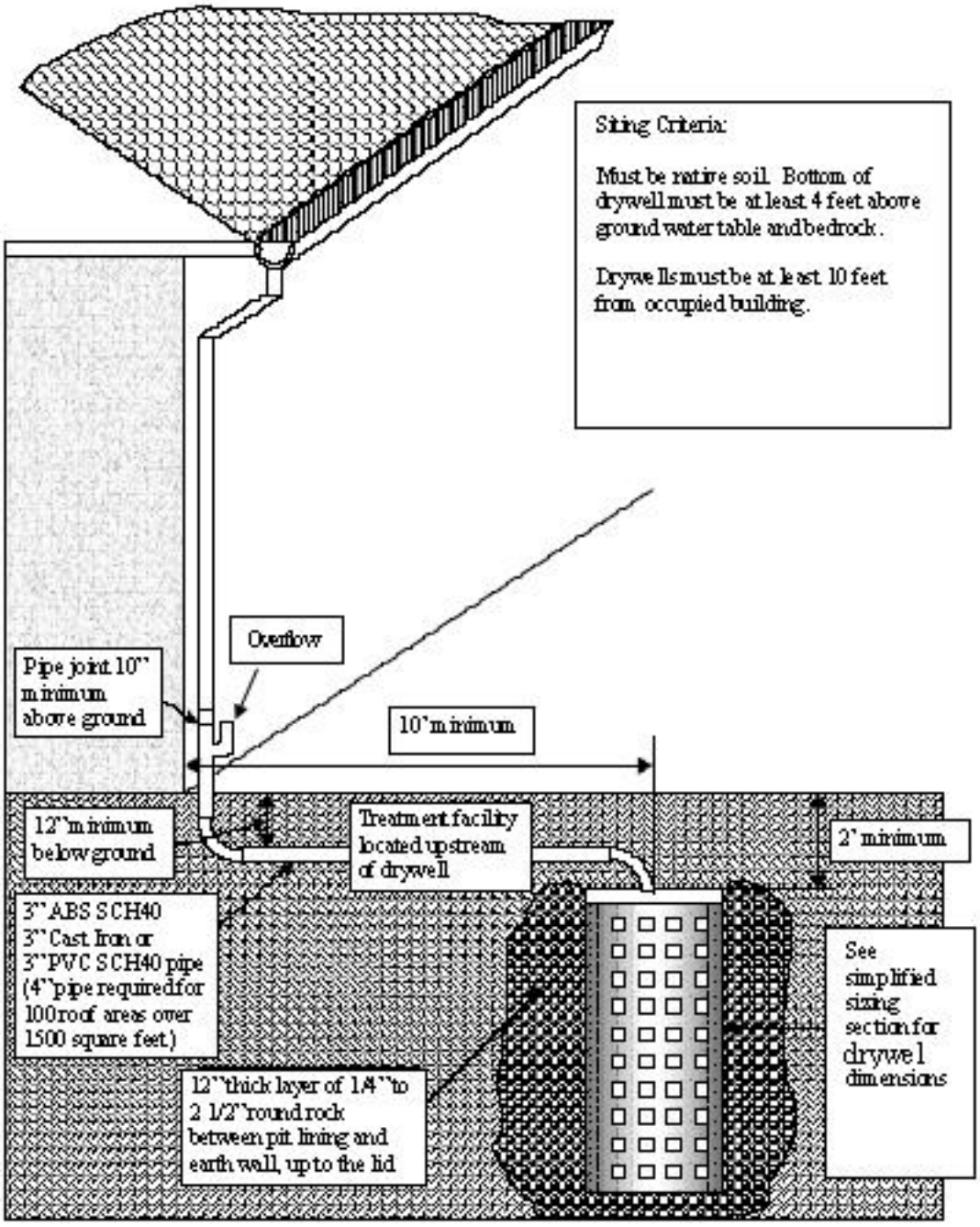


Figure 19.1 Schematic of Drywell System
(Source: City of Portland, Oregon)

20.0 Infiltration Trench

Structural Stormwater Control

North Central Texas Council of Governments

integrated Stormwater Management iSWM



Soakage Trench installation. *(Source: Wikipedia, CC BY-SA 3.0)*

Description

Infiltration trenches allow for the infiltration of stormwater runoff into the surrounding soils. They consist of an excavated trench filled with stone aggregate. This creates an underground reservoir for stormwater runoff. Soakage trenches are a variation of infiltration trenches and drain through a perforated pipe buried in gravel.

Design Considerations

- Soils must have infiltration rates of 0.5 inches per hour or higher in order to effectively accept runoff.
- Geotechnical testing is required to determine infiltration rates (two borings per facility).
- Upstream treatment (such as a sediment forebay and grass channel) is required to prevent sediment choking.
- Observation wells are required in order to ensure that the facilities are functioning as designed.
- Soakage trenches are intended to be used in space-limited applications.
- Soakage trenches can be located under pavement or impervious surfaces.

Key Advantages

- Infiltration facilities promote groundwater recharge.
- Small sites with porous soils are ideal candidates.
- Can be adapted to many types of sites due to the relatively narrow shape.
- Soakage trenches can be used in conjunction with other stormwater devices.

Limitations

These facilities should not be used where there is the potential for groundwater contamination.

There is a high clogging potential due to the reliance on these facilities on infiltration. They should not be used in areas with fine-particle soils (such as clays or silts).

- Significant setback requirements are needed.
- Restrictions for infiltration in karst areas.
- The groundwater table should be below the trench area (4 feet between the bottom of the facility and the seasonally high water table).

Target Constituent	Removal Rate 0% → 100%
Total Suspended Solids	
Total Phosphorus	
Total Nitrogen	
Fecal Coliform	
Heavy Metals	

Implementation Considerations



Suitability

The iSWM manual has designated infiltration trenches and soakage trenches as suitable for providing:



- The subsurface pipe in soakage trenches may require a special permit in some areas, as it may be considered an injection well.

Maintenance

- Inspections to determine clogging.
- Sediment accumulation in the forebay must be removed.
- The pea gravel layer should be replaced as needed.
- Mow filter strips and remove grass cuttings.
- Trash, debris and leaves need to be removed to prevent clogging.

Published October 2020

20.1 General Description

Infiltration trenches are excavations typically filled with stone to create an underground reservoir for stormwater runoff (see Figure 20.1). This runoff volume gradually filtrates through the bottom and sides of the trench into the subsoil over a 2-day period and eventually reaches the water table. By diverting runoff into the soil, an infiltration trench not only treats the water quality volume, but also helps to preserve the natural water balance on a site and can recharge groundwater and preserve baseflow. Due to this fact, infiltration systems are limited to areas with highly porous soils where the water table and/or bedrock are located well below the bottom of the trench. In addition, infiltration trenches must be carefully sited to avoid the potential of groundwater contamination.

Infiltration trenches are not intended to trap sediment and must always be designed with a sediment forebay and grass channel or filter strip, or other appropriate pretreatment measures to prevent clogging and failure. Due to their high potential for failure, these facilities must only be considered for sites where upstream sediment control can be ensured.

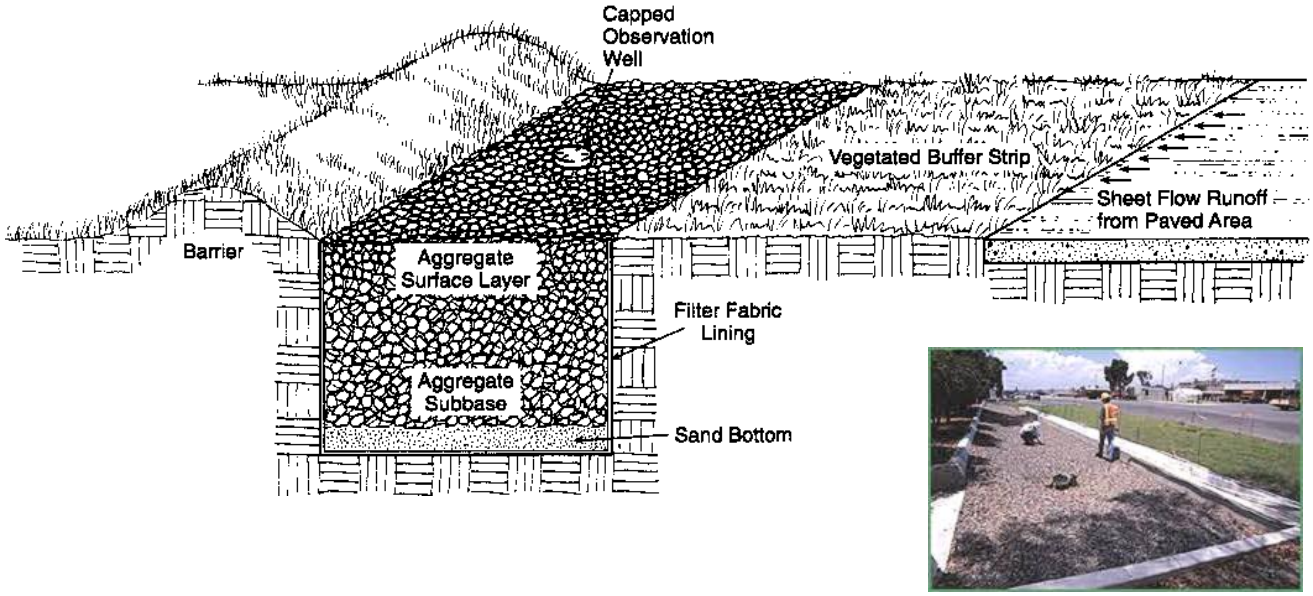


Figure 20.1 Infiltration Trench Example

20.2 Stormwater Management Suitability

Infiltration trenches are designed primarily for stormwater quality, i.e. the removal of stormwater pollutants. However, they can provide limited runoff quantity control, particularly for smaller storm events. For some smaller sites, trenches can be designed to capture and infiltrate the streambank protection volume (SP_v) in addition to WQ_v. An infiltration trench will need to be used in conjunction with another structural control to provide flood control, if required.

Water Quality Protection

Using the natural filtering properties of soil, infiltration trenches can remove a wide variety of pollutants from stormwater through sorption, precipitation, filtering, and bacterial and chemical degradation. Sediment load and other suspended solids are removed from runoff by pretreatment measures in the facility that treats flows before they reach the trench surface.

Section 20.3 provides pollutant removal efficiencies that can be used for planning and design purposes.

Streambank Protection

For smaller sites, an infiltration trench may be designed to capture and infiltrate the entire streambank protection volume SP_v in either an off- or on-line configuration. For larger sites, or where only the WQ_v is diverted to the trench, another structural control must be used to provide SP_v extended detention.

Flood Control

Infiltration trench facilities must provide flow diversion and/or be designed to safely pass extreme storm flows and protect the filter bed and facility.

The volume of runoff removed and treated by the infiltration trench may be taken in the on-site and/or downstream flood control calculations (see Section 1.0).

20.3 Pollutant Removal Capabilities

An infiltration trench is presumed to be able to remove 80% of the total suspended solids load in typical urban post-development runoff when sized, designed, constructed, and maintained in accordance with the recommended specifications. Undersized or poorly designed infiltration trenches can reduce TSS removal performance.

The following design pollutant removal rates are conservative average pollutant reduction percentages for design purposes derived from sampling data, modeling, and professional judgment. In a situation where a removal rate is not deemed sufficient, additional controls may be put in place at the given site in a series or "treatment train" approach.

- Total Suspended Solids – 80%
- Total Phosphorus – 60%
- Total Nitrogen – 60%
- Fecal Coliform – 90%
- Heavy Metals – 90%

For additional information and data on pollutant removal capabilities for infiltration trenches, see the National Pollutant Removal Performance Database (2nd Edition) available at www.cwp.org and the National Stormwater Best Management Practices (BMP) Database at www.bmpdatabase.org

20.4 Application and Site Feasibility Criteria

Infiltration trenches are generally suited for medium-to-high density residential, commercial, and institutional developments where the subsoil is sufficiently permeable to provide a reasonable infiltration rate and the water table is low enough to prevent groundwater contamination. They are applicable primarily for impervious areas where there are not high levels of fine particulates (clay/silt soils) in the runoff and should only be considered for sites where the sediment load is relatively low.

Infiltration trenches can either be used to capture sheet flow from a drainage area or function as an off-line device. Due to the relatively narrow shape, infiltration trenches can be adapted to many different types of sites and can be utilized in retrofit situations. Unlike some other structural stormwater controls, they can easily fit into the margin, perimeter, or other unused areas of developed sites.

To protect groundwater from potential contamination, runoff from designated hotspot land uses or activities must not be infiltrated. Infiltration trenches should not be used for manufacturing and industrial sites, where there is a potential for high concentrations of soluble pollutants and heavy metals. In addition, infiltration should not be considered for areas with a high pesticide concentration. Infiltration trenches are also not suitable in areas with karst geology without adequate geotechnical testing by qualified individuals and in accordance with local requirements.

The following criteria should be evaluated to ensure the suitability of an infiltration trench for meeting stormwater management objectives on a site or development.

General Feasibility

- Suitable for Residential Subdivision Usage – YES
- Suitable for High Density/Ultra Urban Areas – YES
- Regional Stormwater Control – NO

Physical Feasibility - Physical Constraints at Project Site

- Drainage Area – 5 acres maximum
- Space Required – Will vary depending on the depth of the facility
- Site Slope – No more than 6% slope (for pre-construction facility footprint)
- Minimum Head – Elevation difference needed at a site from the inflow to the outflow: 1 foot
- Minimum Depth to Water Table – 4 feet recommended between the bottom of the infiltration trench and the elevation of the seasonally high water table
- Soils – Infiltration rate greater than 0.5 inches per hour required (typically hydrologic group "A", some group "B" soils)

Other Constraints / Considerations

- Aquifer Protection – No hotspot runoff allowed; meet setback requirements in design criteria

20.5 Planning and Design Criteria

*The following criteria are to be considered **minimum** standards for the design of an infiltration trench facility. Consult with the local review authority to determine if there are any variations to these criteria or additional standards that must be followed.*

A Location and Siting

To be suitable for infiltration, underlying soils should have an infiltration rate (f_c) of 0.5 inches per hour or greater, as initially determined from NRCS soil textural classification and subsequently confirmed by field geotechnical tests. The minimum geotechnical testing is one test hole per 5,000 square feet, with a minimum of two borings per facility (taken within the proposed limits of the facility). Infiltration trenches cannot be used in fill soils.

Infiltration trenches should have a contributing drainage area of 5 acres or less.

Soils on the drainage area tributary to an infiltration trench should have a clay content of less than 20% and a silt/clay content of less than 40% to prevent clogging and failure.

There should be at least 4 feet between the bottom of the infiltration trench and the elevation of the seasonally high water table.

Clay lenses, bedrock or other restrictive layers below the bottom of the trench will reduce infiltration rates unless excavated.

Minimum setback requirements for infiltration trench facilities (when not specified by local ordinance or criteria):

From a property line – 10 feet

From a building foundation – 25 feet

From a private well – 100 feet

From a public water supply well – 1,200 feet

From a septic system tank/leach field – 100 feet

From surface waters – 100 feet

From surface drinking water sources – 400 feet (100 feet for a tributary)

When used in an off-line configuration, the water quality protection volume (WQ_v) is diverted to the infiltration trench through the use of a flow splitter. Stormwater flows greater than the WQ_v are diverted to other controls or downstream using a diversion structure or flow splitter.

To reduce the potential for costly maintenance and/or system reconstruction, it is strongly recommended that the trench be located in an open or lawn area, with the top of the structure as close to the ground surface as possible. Infiltration trenches shall not be located beneath paved surfaces, such as parking lots.

Infiltration trenches are designed for intermittent flow and must be allowed to drain and allow re-aeration of the surrounding soil between rainfall events. They must not be used on sites with a continuous flow from groundwater, sump pumps, or other sources.

B General Design

A well-designed infiltration trench consists of:

Excavated shallow trench backfilled with sand, coarse stone, and pea gravel, and lined with a filter fabric

Appropriate pretreatment measures

One or more observation wells to show how quickly the trench dewaters or to determine if the device is clogged

Figure 20.2 provides a plan view and profile schematic for the design of an off-line infiltration trench facility. An example of an on-line infiltration trench is shown in Figure 20.1.

C Physical Specifications / Geometry

The required trench storage volume is equal to the water quality protection volume (WQ_v). For smaller sites, an infiltration trench can be designed with a larger storage volume to include the streambank protection volume (SP_v).

A trench must be designed to fully dewater the entire WQ_v within 24 to 48 hours after a rainfall event. The slowest infiltration rate obtained from tests performed at the site should be used in the design calculations.

Trench depths should be between 3 and 8 feet, to provide for easier maintenance. The width of a trench must be less than 25 feet.

Broader, shallow trenches reduce the risk of clogging by spreading the flow over a larger area for infiltration.

The surface area required is calculated based on the trench depth, soil infiltration rate, aggregate void space, and fill time (assume a fill time of 2 hours for most designs).

The bottom slope of a trench should be flat across its length and width to evenly distribute flows, encourage uniform infiltration through the bottom, and reduce the risk of clogging.

The stone aggregate used in the trench should be washed, bank-run gravel, 1.5 to 2.5 inches in diameter with a void space of about 40%. Aggregate contaminated with soil shall not be used. A porosity value (void space/total volume) of 0.32 should be used in calculations, unless aggregate specific data exist.

A 6-inch layer of clean, washed sand is placed on the bottom of the trench to encourage drainage and prevent compaction of the native soil while the stone aggregate is added.

The infiltration trench is lined on the sides and top by an appropriate geotextile filter fabric that prevents soil piping but has greater permeability than the parent soil. The top layer of filter fabric is located 2 to 6 inches from the top of the trench and serves to prevent sediment from passing into the stone aggregate. Since this top layer serves as a sediment barrier, it will need to be replaced more frequently and must be readily separated from the side sections.

The top surface of the infiltration trench above the filter fabric is typically covered with pea gravel. The pea gravel layer improves sediment filtering and maximizes the pollutant removal in the top of the trench. In addition, it can easily be removed and replaced should the device begin to clog. Alternatively, the trench can be covered with permeable topsoil and planted with grass in a landscaped area.

An observation well must be installed in every infiltration trench and should consist of a perforated PVC pipe, 4 to 6 inches in diameter, extending to the bottom of the trench (see Figure 20.3 for an observation well detail). The observation well will show the rate of dewatering after a storm, as well as provide a means of determining sediment levels at the bottom and when the filter fabric at the top is clogged and maintenance is needed. It should be installed along the centerline of the structure, flush with the ground elevation of the trench. A visible floating marker should be provided to indicate the water level. The top of the well should be capped and locked to discourage vandalism and tampering.

The trench excavation should be limited to the width and depth specified in the design. Excavated material should be placed away from the open trench so as not to jeopardize the stability of the trench sidewalls. The bottom of the excavated trench shall not be loaded in a way that causes soil compaction, and should be scarified prior to placement of sand. The sides of the trench shall be trimmed of all large roots. The sidewalls shall be uniform with no voids and scarified prior to backfilling. All infiltration trench facilities should be protected during site construction and should be constructed after upstream areas have been stabilized.

D Pretreatment / Inlets

Pretreatment facilities **must always** be used in conjunction with an infiltration trench to prevent clogging and failure.

For a trench receiving sheet flow from an adjacent drainage area, the pretreatment system should consist of a vegetated filter strip with a minimum 25-foot length. A vegetated buffer strip around the entire trench is required if the facility is receiving runoff from both directions. If the infiltration rate for the underlying soils is greater than 2 inches per hour, 50% of the WQ_v should be pretreated by another method prior to reaching the infiltration trench.

For an off-line configuration, pretreatment should consist of a sediment forebay, vault, plunge pool, or similar sedimentation chamber (with energy dissipaters) sized to 25% of the water quality protection volume (WQ_v). Exit velocities from the pretreatment chamber must be nonerosive for the 2-year design storm.

E Outlet Structures

Outlet structures are not required for infiltration trenches.

F Emergency Spillway

Typically, for off-line designs, there is no need for an emergency spillway. However, a nonerosive overflow channel should be provided to pass safely flows that exceed the storage capacity of the trench to a stabilized downstream area or watercourse.

G Maintenance Access

Adequate access should be provided to an infiltration trench facility for inspection and maintenance.

H Safety Features

In general, infiltration trenches are not likely to pose a physical threat to the public and do not need to be fenced.

I Landscaping

Vegetated filter strips and buffers should fit into and blend with surrounding area. Native grasses are preferable, if compatible. The trench may be covered with permeable topsoil and planted with grass in a landscaped area

J Additional Site-Specific Design Criteria and Issues

Physiographic Factors - Local terrain design constraints

Low Relief – No additional criteria

High Relief – Maximum site slope of 6%

Karst – Not suitable without adequate geotechnical testing

Special Downstream Watershed Considerations

No additional criteria

20.6 Design Procedures

Step 1 Compute runoff control volumes from the *integrated* Design Focus Areas

Calculate the Water Quality Protection Volume (WQ_v), Streambank Protection Volume (SP_v), and the Flood mitigation storm (Q_f).

Details on the *integrated* Design Focus Areas are found in *Section 1.0 of the Planning Technical Manual*.

Step 2 Determine if the development site and conditions are appropriate for the use of an infiltration trench.

Consider the Application and Site Feasibility Criteria in *Sections 20.4 and 20.5* (A) (Location and Siting).

Step 3 Confirm local design criteria and applicability

Consider any special site-specific design conditions/criteria from *Section 20.5* (J) (Additional Site-Specific Design Criteria and Issues).

Check with local officials and other agencies to determine if there are any additional restrictions and/or surface water or watershed requirements that may apply.

Step 4 Compute WQ_v peak discharge (Q_{wq})

The peak rate of discharge for water quality design storm is needed for sizing of off-line diversion (see *Section 1.5 of the Hydrology Technical Manual*).

(a) Using WQ_v (or total volume to be infiltrated), compute CN
(b) Compute time of concentration using TR-55 method
(c) Determine appropriate unit peak discharge from time of concentration
(d) Compute Q_{wq} from unit peak discharge, drainage area, and WQ_v.

Step 5 Size flow diversion structure, if needed

A flow regulator (or flow splitter diversion structure) should be supplied to divert the WQ_v to the infiltration trench.

Size low flow orifice, weir, or other device to pass Q_{wq}.

Step 6 Size infiltration trench

The area of the trench can be determined from the following equation:

$$A = \frac{WQ_v}{(nd + kT/12)} \quad \textbf{(20.1)}$$

where:

A = Surface Area
WQ_v = Water Quality Protection Volume (or total volume to be infiltrated)
n = porosity
d = trench depth (feet)
k = percolation (inches/hour)
T = Fill Time (time for the practice to fill with water), in hours

A porosity value *n* = 0.32 should be used.

All infiltration systems should be designed to fully dewater the entire WQ_v within 24 to 48 hours after the rainfall event.

A fill time T=2 hours can be used for most designs

See *Section 20.5* (C) (Physical Specifications/Geometry) for more specifications.

Step 7 Determine pretreatment volume and design pretreatment measures

Size pretreatment facility to treat 25% of the water quality protection volume (WQ_v) for off-line configurations.

See *Section 20.5* (D) (Pretreatment / Inlets) for more details.

Step 8 Design spillway(s)

Adequate stormwater outfalls should be provided for the overflow exceeding the capacity of the trench, ensuring nonerosive velocities on the down-slope.

See *Section 29.5* for an Infiltration Trench Design Example

20.7 Inspection and Maintenance Requirements

Table 20.1 Typical Maintenance Activities for Infiltration Trenches

Activity	Schedule
• Ensure that contributing area, facility, and inlets are clear of debris. • Ensure that the contributing area is stabilized. • Remove sediment and oil/grease from pretreatment devices, as well as overflow structures. • Mow grass filter strips as necessary. Remove grass clippings.	Monthly

Table 20.1 Typical Maintenance Activities for Infiltration Trenches	
Activity	**Schedule**
• Check observation wells following 3 days of dry weather. Failure to percolate within this time period indicates clogging. • Inspect pretreatment devices and diversion structures for sediment build-up and structural damage. • Remove trees that start to grow in the vicinity of the trench.	Semi-annual Inspection
• Replace pea gravel/topsoil and top surface filter fabric (when clogged).	As needed
• Perform total rehabilitation of the trench to maintain design storage capacity. • Excavate trench walls to expose clean soil.	Upon Failure

(Source: EPA, 1999)

Additional Maintenance Considerations and Requirements

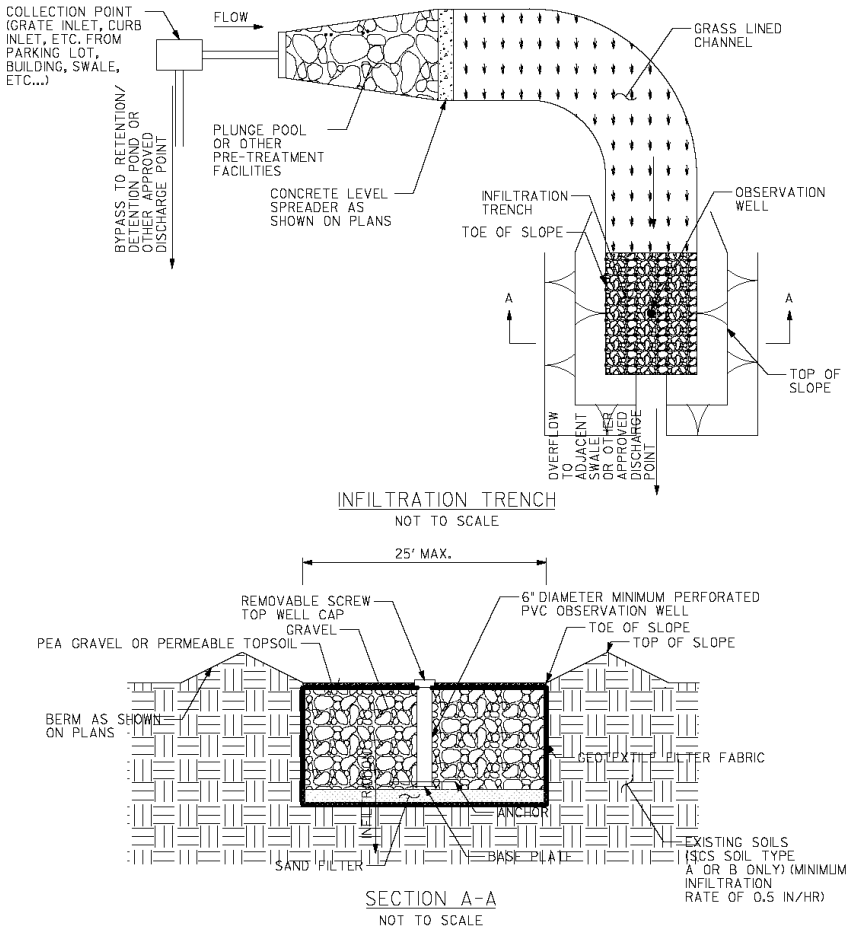
A record should be kept of the dewatering time of an infiltration trench to determine if maintenance is necessary.

Removed sediment and media may usually be disposed of in a landfill.



Regular inspection and maintenance is critical to the effective operation of infiltration trench facilities as designed. Maintenance responsibility for an infiltration trench should be vested with a responsible authority by means of a legally binding and enforceable maintenance agreement that is executed as a condition of plan approval.

20.8 Example Schematics



NOTES:

1. RECOMMENDED MAXIMUM WIDTH OF INFILTRATION TRENCH IS 25-FEET. DIMENSIONS SHALL BE PROVIDED BY DESIGNER IN PLANS AND SHALL BE BASED ON SITE SPECIFIC CONDITIONS. DESIGN PROFESSIONAL SHALL AT A MINIMUM PROVIDE THE FOLLOWING INFORMATION:
   - HORIZONTAL CONTROL
   - VERTICAL CONTROL
   - WATER QUALITY PROTECTION VOLUME
   - STREAMBANK PROTECTION VOLUME (IF NEEDED)
   - OBSERVATION WELL INFORMATION
   - PRETREATMENT INFORMATION

2. DESIGN ENGINEER IS RESPONSIBLE FOR PROVIDING CALCULATIONS THAT DEMONSTRATE THAT GROWING MEDIUM AND SYSTEM AS A WHOLE CAN EFFECTIVELY TREAT THE WATER QUALITY VOLUME (WQv) AS SPECIFIED IN SECTION 1.4 OF THE NCTCOG WATER QUALITY TECHNICAL MANUAL. FLOW IN EXCESS OF THE WQV SHALL BE DESIGNED TO BYPASS THE SYSTEM VIA THE OVERFLOW SYSTEM(S).

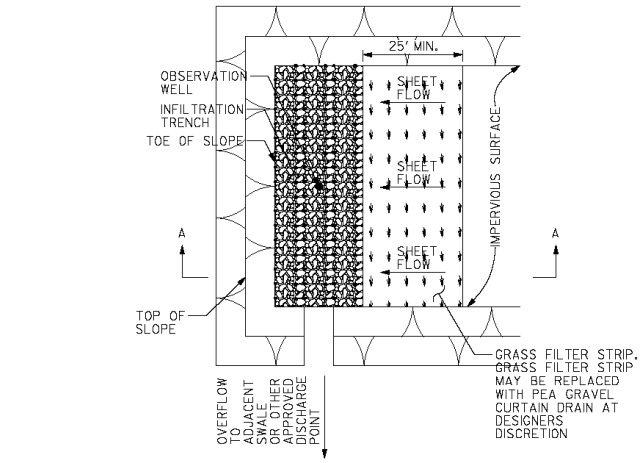
3. DESIGN OF THIS SYSTEM SHALL FOLLOW THAT OF SECTION 20 "INFILTRATION TRENCH" CONTAINED IN THE NCTCOG SITE DEVELOPMENT CONTROLS ISWM TECHNICAL MANUAL.

4. DESIGNER SHALL ENSURE THAT SYSTEM IS CAPABLE OF BEING FULLY DRAINED WITHIN 48-HOURS.

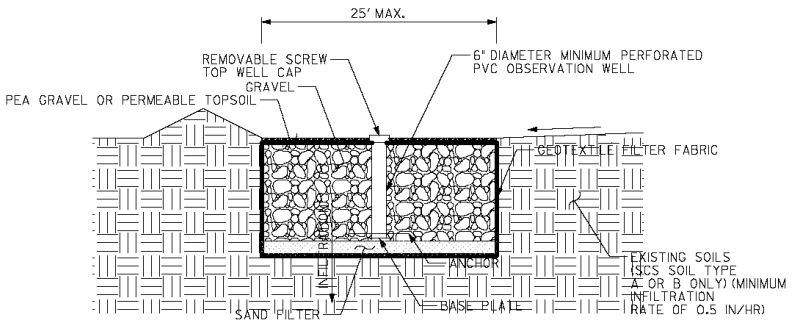
5. CARE SHOULD BE TAKEN TO ENSURE THAT EXISTING SOILS INTENDED TO FACILITATE INFILTRATION ARE NOT COMPACTED DURING ADJACENT CONSTRUCTION OPERATIONS.

6. DESIGNER SHALL BE RESPONSIBLE FOR PREPARING AN OPERATIONS AND MAINTENANCE MANUAL OR MATRIX FOR SYSTEMS TO AID OWNER IN DETERMINING MAINTENANCE PROCEEDURES AND INTERVALS. DESIGNER SHALL ALSO CONSIDER MAINTENANCE ACCESS WHEN DESIGNING SYSTEM.

Figure 20.2 Schematic of Off Line Infiltration Trench
(Source: Center for Watershed Protection)



INFILTRATION TRENCH
NOT TO SCALE



SECTION A-A
NOT TO SCALE

NOTES:

1. RECOMMENDED MAXIMUM WIDTH OF INFILTRATION TRENCH IS 25-FEET. DIMENSIONS SHALL BE PROVIDED BY DESIGNER IN PLANS AND SHALL BE BASED ON SITE SPECIFIC CONDITIONS. DESIGN PROFESSIONAL SHALL AT A MINIMUM PROVIDE THE FOLLOWING INFORMATION:
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2. DESIGN ENGINEER IS RESPONSIBLE FOR PROVIDING CALCULATIONS THAT DEMONSTRATE THAT GROWING MEDIUM AND SYSTEM AS A WHOLE CAN EFFECTIVELY TREAT THE WATER QUALITY VOLUME (WQv) AS SPECIFIED IN SECTION 1.4 OF THE NCTCOG WATER QUALITY TECHNICAL MANUAL. FLOW IN EXCESS OF THE WQv SHALL BE DESIGNED TO BYPASS THE SYSTEM VIA THE OVERFLOW SYSTEM(S).

3. DESIGN OF THIS SYSTEM SHALL FOLLOW THAT OF SECTION 20 "INFILTRATION TRENCH" CONTAINED IN THE NCTCOG SITE DEVELOPMENT CONTROLS ISWM TECHNICAL MANUAL.

4. DESIGNER SHALL ENSURE THAT SYSTEM IS CAPABLE OF BEING FULLY DRAINED WITHIN 48-HOURS.

5. CARE SHOULD BE TAKEN TO ENSURE THAT EXISTING SOILS INTENDED TO FACILITATE INFILTRATION ARE NOT COMPACTED DURING ADJACENT CONSTRUCTION OPERATIONS.

6. DESIGNER SHALL BE RESPONSIBLE FOR PREPARING AN OPERATIONS AND MAINTENANCE MANUAL OR MATRIX FOR SYSTEMS TO AID OWNER IN DETERMINING MAINTENANCE PROCEEDURES AND INTERVALS. DESIGNER SHALL ALSO CONSIDER MAINTENANCE ACCESS WHEN DESIGNING SYSTEM.

Figure 20.3 On Line Infilltration Trench

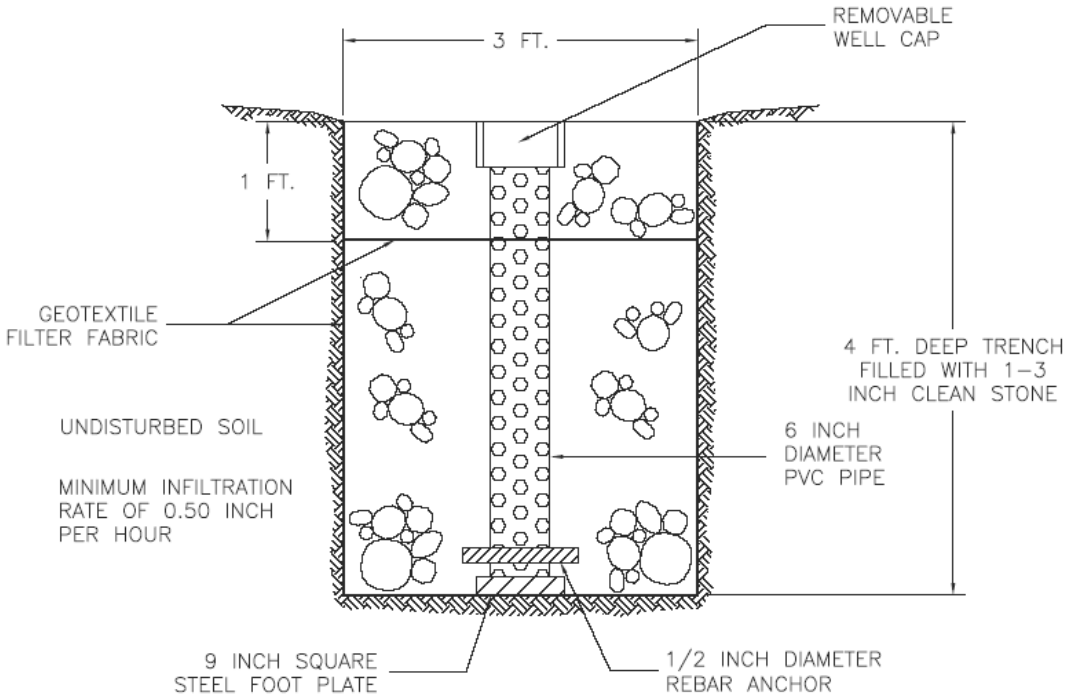


Figure 20.4 Observation Well Detail

- The aggregate material for the trench should consist of a clean aggregate with a maximum diameter of 3 inches and a minimum diameter of 1.5 inches.
- The aggregate should be graded such that there will be few aggregates smaller than the selected size. For design purposes, void space for these aggregates may be assumed to be in the range of 30 to 40%.
- A 6-inch layer of clean, washed sand is placed on the bottom of the trench to encourage drainage and prevent compaction of the native soil, while the stone aggregate is added.
- The aggregate should be completely surrounded with an engineering filter fabric. If the trench has an aggregate surface, filter fabric should surround all of the aggregate fill material except for the top 1 foot.
- The observation well should consist of perforated PVC pipe, 4 to 6 inches diameter, located in the center of the structure, and be constructed flush with the ground elevation of the trench.
- The PVC pipe should have a factory attached cast iron or high impact to prevent rotation when removing the screw top lid.
- The screw top lid should be cast iron and clearly labeled as an observation well.

20.9 Design Forms

Design Procedure Form: Infiltration Trench

PRELIMINARY HYDROLOGIC CALCULATIONS

1a. Compute WQ_v volume requirements
Compute Runoff Coefficient, R_v — R_v = _____
Compute WQv — WQ_v = _____ acre-ft

1b. Compute **SP_v** — **SP_v** = _____ acre-ft
Compute average release rate — release rate = _____ cfs
Compute Q_p (100-year detention volume required) — Q_p = _____ acre-ft
Compute (as necessary) Q_f — Q_f = _____ cfs

INFILTRATION TRENCH DESIGN

2. Is the use of a infiltration trench appropriate? — ***See subsections 5.2.19.4 and 5.2.19.5 - A***

3. Confirm local design criteria and applicability. — ***See subsection 5.2.19.5 - J***

4. Compute WQ_v peak discharge (Q_{wq})
Compute Curve Number — CN = _____
Compute Time of Concentration t_c — t_c = _____ hour
Compute Q_{wq} — Q_{wq} = _____ cfs

5. Size infiltration trench — Area = _____ ft^2
Width must be less than 25 ft — Width = _____ ft
Length = _____ ft

6. Size the flow diversion structures
Low flow orifice from orifice equation
$Q = CA(2gh)^{0.5}$ — A = _____ ft^2
diam. = _____ inch

Overflow weir from weir equation
$Q = CLH^{3/2}$ — Length = _____ ft

7. Pretreatment volume (for offine designs)
$Vol_{pre} = 0.25(WQ_v)$ — Vol_{pre} = _____ ft^3

8. Design spillway(s)

Notes: _____

21.0 Soakage Trench

Structural Stormwater Control



Description: Soakage trenches are a variation of infiltration trenches. Soakage trenches drain through a perforated pipe buried in gravel. They are used in highly impervious areas where conditions do not allow surface infiltration and where pollutant concentrations in runoff are minimal (i.e. non-industrial rooftops. They may be used in conjunction with other stormwater devices, such as draining downspouts or planter boxes.

KEY CONSIDERATIONS

DESIGN CRITERIA:

- Intended for space-limited applications
- Like other infiltration devices, soakage trenches should not be used for stormwater containing high sediment loads to minimize clogging

ADVANTAGES / BENEFITS:

- Filtration provides pollutant removal capability
- Reservoir decreases peak flow rates

DISADVANTAGES / LIMITATIONS:

- Subsurface pipe considered an injection well and may require special permit

POLLUTANT REMOVAL

80%	**Total Suspended Solids**
60/60%	**Nutrients** - Total Phosphorus / Total Nitrogen removal
90%	**Metals** - Cadmium, Copper, Lead, and Zinc removal
90%	**Pathogens** - Coliform, Streptococci, E.Coli removal

STORMWATER MANAGEMENT SUITABILITY

P	**Water Quality Protection**
S	**Streambank Protection**
	On-Site Flood Control
	Downstream Flood Control

IMPLEMENTATION CONSIDERATIONS

M	**Land Requirement**
H	**Capital Cost**
H	**Maintenance Burden**

Residential Subdivision Use: Yes
High Density/Ultra-Urban: Yes
Drainage Area: 5 acres max
Soils: Pervious soils required (0.5 in/hr or greater)

L=Low **M**=Moderate **H**=High

21.1 General Description

Soakage trenches represent a variation of infiltration trench. Regular infiltration trenches drain from the surface, but in highly urbanized areas there is not often a suitable area available for this type of setup. Soakage trenches utilize a perforated pipe embedded within the trench, thereby minimizing the surface area required for the device. They can even be located under pavement.

Soakage trenches used for stormwater disposal are considered Class V injection devices by the EPA and fall under the Texas UIC program.

21.2 Pollutant Removal Capabilities

Pollutant removal is similar to infiltration trenches (see *Section 16.0*), but care should be taken to avoid clogging with sediments.

- Total Suspended Solids – 80%
- Total Phosphorus – 60%
- Total Nitrogen – 60%
- Fecal Coliform – 90%
- Heavy Metals – 90%

21.3 Design Criteria and Specifications

- The soakage trench should be located at least 5 feet from the nearest property line and 10 feet away from an occupied building (they may be closer to other structures, such as a parking garage or other structures on piers.
- The trench shall be excavated in native soil, uncompacted by heavy equipment.
- The trench should be at least 3 feet deep and 2.5 feet wide as shown in Figure 21.1. The exact dimensions will be dependent on the drainage characteristics of the surrounding soils.
- There should be at least four feet between the bottom of the trench and the seasonal high ground water table.
- A silt trap or similar device may be installed upstream of the perforated pipe if pretreatment is needed prior to discharge.
- The bottom of the trench should be filled with at least 18 inches of medium sand meeting TxDOT Fine Aggregate Grade No 1 and covered with a layer of filter fabric.
- A minimum of six inches of ¾” – 2 ½” round or crushed rock shall be placed on top of the fabric covered sand base.
- Piping should be 3” diameter prior to the perforated drainage pipe, 4” if serving greater than 1500 square feet of roof.
- The perforated pipe shall be an approved leach field pipe with holes oriented downward. It shall be covered with filter fabric, with at least 12“ of backfill above the pipe.

21.4 Inspection and Maintenance Requirements

The inspection and maintenance requirements for soakage trenches are designed to maintain an adequate drainage rate through the trench, avoiding flooding.

Table 21.1 Typical Maintenance Activities for Soakage Trenches

Activity	Schedule
• Ensure that inflow is unimpeded.	Quarterly and within 48 hours of major storms
• Clean silt trap if it is more than 25% full of sediment	As needed, based on minimum annual inspection
• Inspect trench for waterlogged soils at surface.	Between 24 - 48 hours after major storms

21.5 Example Schematics

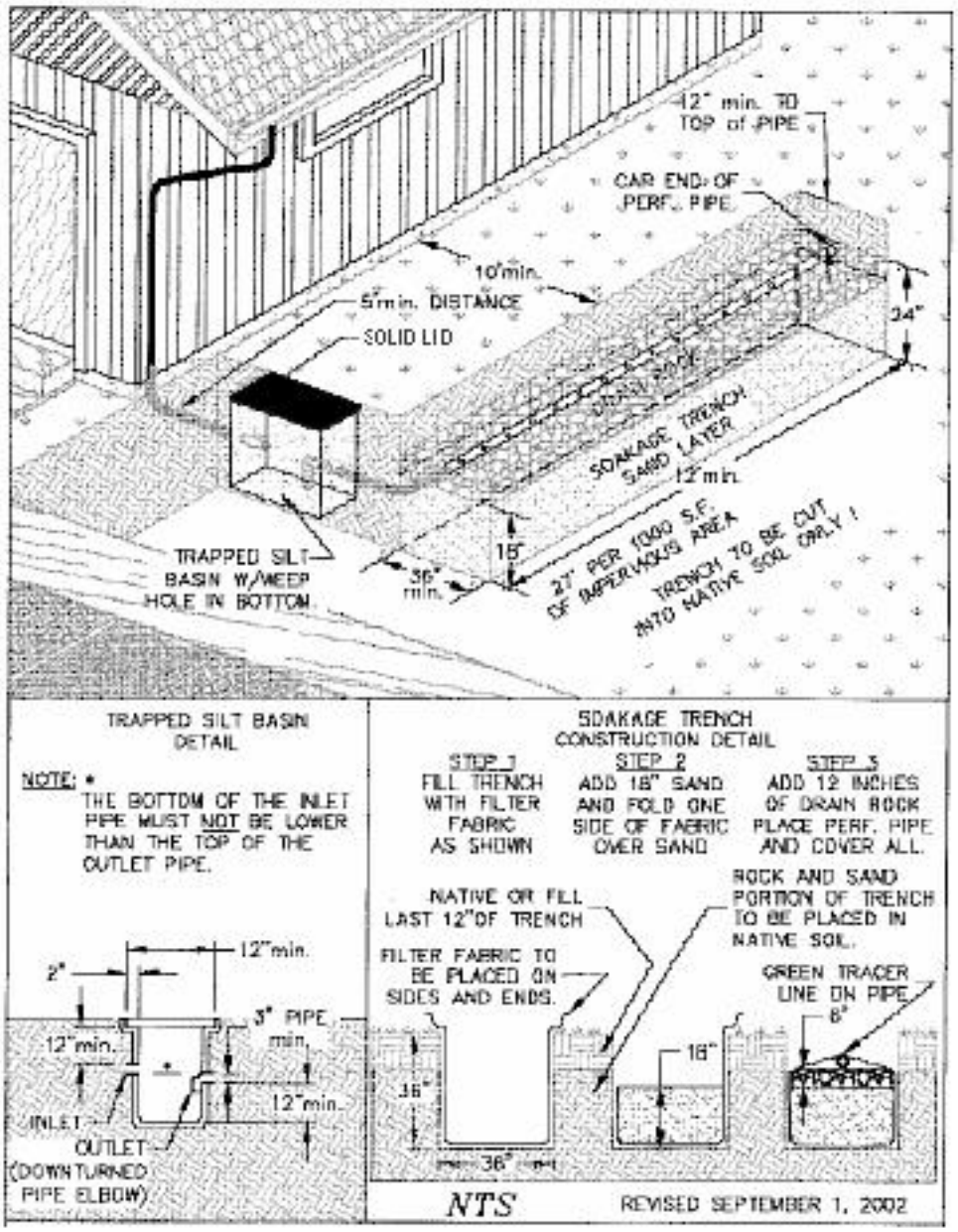


Figure 21.1 Schematic of a Soakage Trench
(Source: City of Portland, Oregon)

22.0 Stormwater Ponds

Stormwater Control

North Central Texas Council of Governments

Integrated Stormwater Management (iSWM)

Description

Stormwater ponds are constructed retention basins that contain a permanent pool or micropool. Stormwater runoff is detained in the ponds, and treatment is achieved through settling and biological uptake mechanisms. Stormwater ponds are also called retention ponds, wet ponds, or wet excavation detention ponds.

Design Considerations

- Stormwater ponds are good solutions for large drainage areas. Maximum drainage areas are 25 acres, 10 acres for the micropool.
- Upstream treatment (such as a sediment forebay or equivalent) is required.
- Permanent pools should not exceed 8 feet in depth.
- Temporary storage can be provided above the permanent storage for larger storm events.
- Vegetated side slopes are required and must be no steeper than 3:1.
- Ponds located in areas with high infiltration rates will require a pond liner to keep the permanent pool.
- Ponds can require a larger area than other stormwater facilities, but can treat large areas as well.

Key Advantages

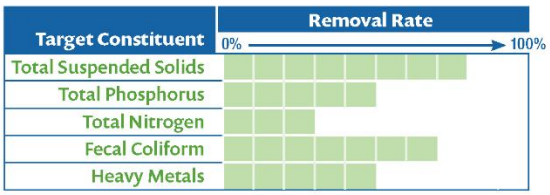
- Urban pollutants are removed at a moderate to high rate.
- Stormwater ponds can be considered amenities and generally have a high rate of community acceptance.
- Ponds provide an opportunity for wildlife habitat.
- Stormwater ponds are among the most cost-effective facilities and are widely used.
- Multiple ponds can be used in series. The series provides improved downstream protection and longer pollutant removal pathways.

Limitations

- High relief areas have dam height restrictions that need to be examined during design.
- Low relief terrain may cause poor drainage.
- Ponds may cause thermal impacts and downstream warming of stormwater.
- Fecal coliform removal rate decreases if waterfowl are present.
- Mosquito and other vectors may require treatment.



Stormwater Pond in San Antonio, TX. *(Source: Tetra Tech)*



Implementation Considerations



Suitability

The iSWM manual has designated stormwater ponds as suitable for providing:



Maintenance

- Debris needs to be removed from inlet and outlet structures.
- Invasive vegetation should be removed.
- Sediment accumulation and erosion should be monitored and remedied when issues arise.
- If dams are required, inspection and maintenance must be performed.
- Monitor for illegal dumping.
- Mosquito control and rodent repair may be required.

Published October 2020

22.1 General Description

Stormwater ponds (also referred to as *retention ponds*, *wet ponds*, *or wet extended detention ponds*) are constructed stormwater retention basins that have a permanent (dead storage) pool of water throughout the year. They can be created by excavating an already existing natural depression or through the construction of embankments.

In a stormwater pond, runoff from each rain event is detained and treated in the pool through gravitational settling and biological uptake until it is displaced by runoff from the next storm. The permanent pool also serves to protect deposited sediments from resuspension. Above the permanent pool level, additional temporary storage (live storage) is provided for runoff quantity control. The upper stages of a stormwater pond are designed to provide extended detention of the 1-year storm for downstream streambank protection, as well as normal detention of larger storm events to meet Q_f requirements.

Stormwater ponds are among the most cost-effective and widely used stormwater practices. A well-designed and landscaped pond can be an aesthetic feature on a development site when planned and located properly.

There are several different variants of stormwater pond design, the most common of which include the wet pond, the wet extended detention pond, and the micropool extended detention pond. In addition, multiple stormwater ponds can be placed in series or parallel to increase performance or meet site design constraints. Below are descriptions of each design variant:

- **Wet Pond** – Wet ponds are stormwater basins constructed with a permanent (dead storage) pool of water equal to the water quality volume. Stormwater runoff displaces the water already present in the pool. Temporary storage (live storage) can be provided above the permanent pool elevation for larger flows.
- **Wet Extended Detention (ED) Pond** – A wet extended detention pond is a wet pond where the water quality volume is split evenly between the permanent pool and extended detention (ED) storage provided above the permanent pool. During storm events, water is detained above the permanent pool and released over 24 hours. This design has similar pollutant removal to a traditional wet pond, but consumes less space.
- **Micropool Extended Detention (ED) Pond** – The micropool extended detention pond is a variation of the extended detention wet pond where only a small "micropool" is maintained at the outlet to the pond. The outlet structure is sized to detain the water quality volume for 24 hours. The micropool prevents resuspension of previously settled sediments and also prevents clogging of the low flow orifice.
- **Multiple Pond Systems** – Multiple pond systems consist of constructed facilities that provide water quality and quantity volume storage in two or more cells. The additional cells can create longer pollutant removal pathways and improved downstream protection.

Figure 22.1 shows a number of examples of stormwater pond variants. *Section 22.8* provides plan view and profile schematics for the design of a wet pond, wet extended detention pond, micropool extended detention pond, and multiple pond system.



Conventional dry detention basins do not provide a permanent pool and are **not recommended** for general application use to meet water quality criteria, as they fail to demonstrate an ability to meet the majority of the water quality goals. In addition, dry detention basins are prone to clogging and resuspension of previously settled solids and require a higher frequency of maintenance than wet ponds if used for untreated stormwater flows. These facilities can be used in combination with appropriate water quality controls to provide streambank protection, and overbank and extreme flood storage. Please see a further discussion in *Section 10.0* on Dry Detention Basins).



Figure 22.1 Stormwater Pond Examples

22.2 Stormwater Management Suitability

Stormwater ponds are designed to control both stormwater quantity and quality. Thus, a stormwater pond can be used to address all of the *integrated stormwater sizing criteria* for a given drainage area.

Water Quality

Ponds treat incoming stormwater runoff by physical, biological, and chemical processes. The primary removal mechanism is gravitational settling of particulates, organic matter, metals, bacteria, and organics as stormwater runoff resides in the pond. Another mechanism for pollutant removal is uptake by algae and wetland plants in the permanent pool – particularly of nutrients. Volatilization and chemical activity also work to break down and eliminate a number of other stormwater contaminants such as hydrocarbons.

Section 22.3 provides pollutant removal efficiencies that can be used for planning and design purposes.

Streambank Protection

A portion of the storage volume above the permanent pool in a stormwater pond can be used to provide control of the streambank protection volume (SP_v). This is accomplished by releasing the 1-year, 24-hour storm runoff volume over 24 hours (extended detention).

On-Site Flood Control

A stormwater pond can also provide detention storage above the permanent pool to reduce the post-development peak flow to pre-development levels, if required.

Downstream Flood Control

In situations where it is required, stormwater ponds can also be used to provide detention to control the flood mitigation storm peak flow downstream. Where this is not required, the pond structure is designed to safely pass extreme storm flows.

22.3 Pollutant Removal Capabilities

All of the stormwater pond design variants are presumed to be able to remove 80% of the total suspended solids load in typical urban post-development runoff when sized, designed, constructed and maintained in accordance with the recommended specifications. Undersized or poorly designed ponds can reduce TSS removal performance.

The following design pollutant removal rates are conservative average pollutant reduction percentages for design purposes derived from sampling data, modeling and professional judgment. In a situation where a removal rate is not deemed sufficient, additional controls may be put in place at the given site in a series or "treatment train" approach.

- Total Suspended Solids – 80%
- Total Phosphorus – 50%
- Total Nitrogen – 30%
- Fecal Coliform – 70% (if no resident waterfowl population present)
- Heavy Metals – 50%

For additional information and data on pollutant removal capabilities for stormwater ponds, see the National Pollutant Removal Performance Database (2nd Edition) available at www.cwp.org and the National Stormwater Best Management Practices (BMP) Database at www.bmpdatabase.org

22.4 Application and Site Feasibility Criteria

Stormwater ponds are generally applicable to most types of new development and redevelopment, and can be used in both residential and nonresidential areas. Ponds can also be used in retrofit situations. The following criteria should be evaluated to ensure the suitability of a stormwater pond for meeting stormwater management objectives on a site or development.

General Feasibility

- Suitable for Residential Subdivision Usage – YES
- Suitable for High Density/Ultra-Urban Areas – Land requirements may preclude use
- Regional Stormwater Control – YES
- Hotspot Runoff - YES

Physical Feasibility - Physical Constraints at Project Site

Drainage Area – A minimum of 25 acres is needed for wet pond and extended detention wet pond to maintain a permanent pool, 10 acres minimum for extended detention micropool pond. A smaller drainage area may be acceptable with an adequate water balance and anti-clogging device.

Space Required – Approximately 2 to 3% of the tributary drainage area

Site Slope – There should not be more than 15% slope across the pond site.

Minimum Head – Elevation difference needed at a site from the inflow to the outflow: 6 to 8 feet

Minimum Depth to Water Table – If used on a site with an underlying water supply aquifer or when treating a hotspot, a separation distance of 2 feet is required between the bottom of the pond and the elevation of the seasonally high water table.

Soils – Underlying soils of hydrologic group "C" or "D" should be adequate to maintain a permanent pool. Most group "A" soils and some group "B" soils will require a pond liner. *Evaluation of soils should be based upon an actual subsurface analysis and permeability tests*.

Other Constraints / Considerations

- Local Aquatic Habitat – Consideration should be given to the thermal influence of stormwater pond outflows on downstream local aquatic habitats.

22.5 Planning and Design Criteria

*The following criteria are to be considered **minimum** standards for the design of a stormwater pond facility. Consult with the local review authority to determine if there are any variations to these criteria or additional standards that must be followed.*

A Location and Siting

Stormwater ponds should have a minimum contributing drainage area of 25 acres or more for wet pond or extended detention wet pond to maintain a permanent pool. For an extended detention micropool pond, the minimum drainage area is 10 acres. A smaller drainage area can be considered when water availability can be confirmed (such as from a groundwater source or areas with a high water table). In these cases a water balance may be performed (see *Section 4.0 of the Hydrology Technical Manual* for details). Ensure that an appropriate anti-clogging device is provided for the pond outlet.

A stormwater pond should be sited such that the topography allows for maximum runoff storage at minimum excavation or construction costs. Pond siting should also take into account the location and use of other site features such as buffers and undisturbed natural areas and should attempt to aesthetically "fit" the facility into the landscape. Bedrock close to the surface may prevent excavation.

Stormwater ponds should not be located on steep (>15%) or unstable slopes.

Stormwater ponds cannot be located within a stream or any other navigable waters of the U.S., including wetlands, without obtaining a Section 404 permit under the Clean Water Act, and any other applicable State permit.

Minimum setback requirements for stormwater pond facilities measured from the easement line that defines the pond site (when not specified by local ordinance or criteria):

- From a property line – 10 feet
- From a private well – 100 feet; if well is downgradient from a hotspot land use then the minimum setback is 250 feet
- From a septic system tank/leach field/spray area – 50 feet

All utilities should be located outside of the pond/basin site.

B General Design

A well-designed stormwater pond consists of:

1. Permanent pool of water,
2. Overlying zone in which runoff control volumes are stored, and
3. Shallow littoral zone (aquatic bench) along the edge of the permanent pool that acts as a biological filter.

In addition, all stormwater pond designs need to include a sediment forebay at all major inflows to the basin to allow heavier sediments to drop out of suspension before the runoff enters the permanent pool. (A sediment forebay schematic can be found in *iSWM Program Guidance – Federal, State and Regional Initiatives*)

Additional pond design features include an emergency spillway, maintenance access, safety bench, pond buffer, and appropriate native landscaping.

Figures 22.4 thru 22.7 in *Section 22.8* provide plan view and profile schematics for the design of a wet pond, extended detention wet pond, extended detention micropool pond and multiple pond system.

C Physical Specifications / Geometry

In general, pond designs are unique for each site and application. However, there are number of geometric ratios and limiting depths for pond design that must be observed for adequate pollutant removal, ease of maintenance, and improved safety.

Permanent pool volume is typically sized as follows:

- Standard wet ponds: 100% of the water quality treatment volume (1.0 WQ_v)
- Extended detention wet ponds: 50% of the water quality treatment volume (0.5 WQ_v)
- extended detention micropool ponds: Approximately 0.1 inch per impervious acre

Proper geometric design is essential to prevent hydraulic short-circuiting (unequal distribution of inflow), which results in the failure of the pond to achieve adequate levels of pollutant removal. The minimum length-to-width ratio for the permanent pool shape is 1.5:1, and should ideally be greater than 3:1 to avoid short-circuiting. In addition, ponds should be wedge-shaped when possible so that flow enters the pond and gradually spreads out, improving the sedimentation process. Baffles, pond shaping or islands can be added within the permanent pool to increase the flow path.

Maximum depth of the permanent pool should generally not exceed 8 feet to avoid stratification and anoxic conditions. Minimum depth for the pond bottom should be 3 to 4 feet. Deeper depths near the outlet will yield cooler bottom water discharges that may mitigate downstream thermal effects.

Side slopes to the pond should not usually exceed 3:1 (h:v) without safety precautions or if mowing is anticipated and should terminate on a safety bench (see Figure 22.2). The safety bench requirement may be waived if slopes are 4:1 or gentler. All side slopes should be verified with a geotechnical evaluation to ensure slope stability.

The perimeter of all deep pool areas (4 feet or greater in depth) should be surrounded by two benches: safety and aquatic. For larger ponds, a safety bench extends approximately 15 feet outward from the normal water edge to the toe of the pond side slope. The maximum slope of the safety bench should be 6%. An aquatic bench extends inward from the normal pool edge (15 feet on average) and has a maximum depth of 18 inches below the normal pool water surface elevation (see Figure 22.2).

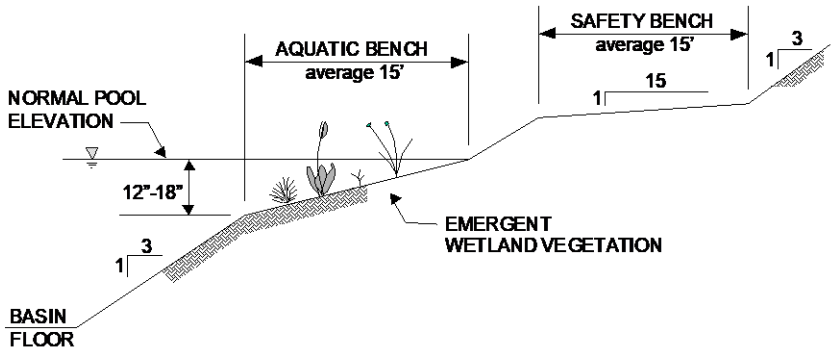


Figure 22.2 Typical Stormwater Pond Geometry Criteria

The contours and shape of the permanent pool should be irregular to provide a more natural landscaping effect.

D Pretreatment / Inlets

Each pond should have a sediment forebay or equivalent upstream pretreatment. A sediment forebay is designed to remove incoming sediment from the stormwater flow prior to dispersal in a larger permanent pool. The forebay should consist of a separate cell, formed by an acceptable barrier. A forebay is to be provided at each inlet, unless the inlet provides less than 10% of the total design storm inflow to the pond. In some design configurations, the pretreatment volume may be located within the permanent pool.

The forebay is sized to contain 0.1 inches per impervious acre of contributing drainage and should be 4 to 6 feet deep. The pretreatment storage volume is part of the total WQ_v requirement and may be subtracted from WQ_v for permanent pool sizing.

A fixed vertical sediment depth marker shall be installed in the forebay to measure sediment deposition over time. The bottom of the forebay may be hardened (e.g., using concrete, paver blocks, etc.) to make sediment removal easier.

Inflow channels are to be stabilized with flared riprap aprons, or the equivalent. Inlet pipes to the pond can be partially submerged. Inflow pipe, channel velocities, and exit velocities from the forebay must be nonerosive.

E Outlet Structures

Flow control from a stormwater pond is typically accomplished with the use of a concrete or corrugated aluminum, aluminized steel, or HDPE riser and barrel. The riser is a vertical pipe or inlet structure that is attached to the bottom of the pond with a watertight connection. The outlet barrel is a horizontal pipe attached to the riser that conveys flow under the embankment (see Figure 22.3). The riser should be located within the embankment for maintenance access, safety and aesthetics.

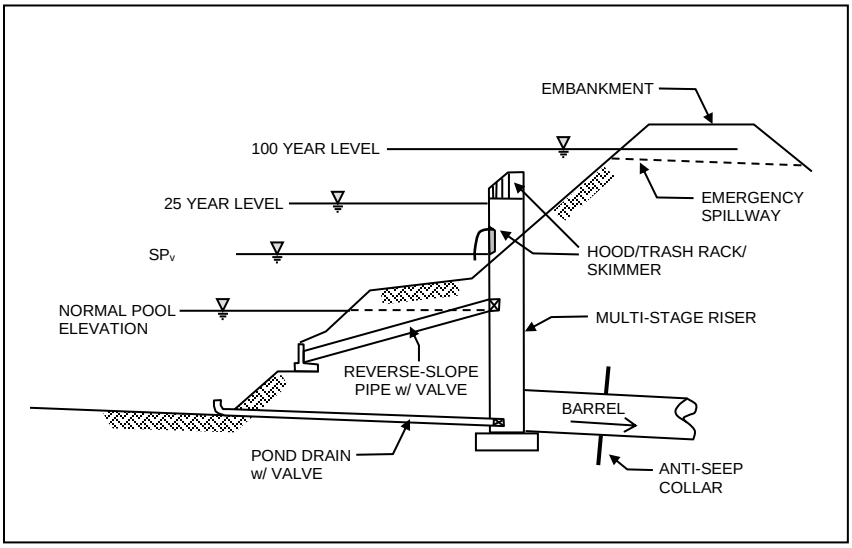


Figure 22.3 Typical Pond Outlet Structure

A number of outlets at varying depths in the riser provide internal flow control for routing of the water quality, streambank protection, and on-site flood control runoff volumes. The number of orifices can vary and is usually a function of the pond design.

Embankments 6 feet in height or greater shall be designed per Texas Commission on Environmental Quality guidelines for Dam Safety. See *iSWM Program Guidance – Dams and Reservoirs in Texas*.

For example, a wet pond riser configuration is typically comprised of a streambank protection outlet (usually an orifice) and on-site flood control outlet (often a slot or weir). The streambank protection orifice is sized to release the streambank protection storage volume over a 24-hour period (12-hour extended detention

may be warranted in some cold water streams). Since the water quality volume is fully contained in the permanent pool, no orifice sizing is necessary for this volume. As runoff from a water quality event enters the wet pond, it simply displaces that same volume through the streambank protection orifice. Thus an off-line wet pond providing *only* water quality treatment can use a simple overflow weir as the outlet structure.

In the case of an extended detention wet pond or extended detention micropool pond, there is generally a need for an additional outlet (usually an orifice) that is sized to pass the extended detention water quality volume that is surcharged on top of the permanent pool. Flow will first pass through this orifice, which is sized to release the water quality extended detention volume in 24 hours. The preferred design is a reverse slope pipe attached to the riser, with its inlet submerged 1 foot below the elevation of the permanent pool to prevent floatables from clogging the pipe and to avoid discharging warmer water at the surface of the pond. The next outlet is sized for the release of the streambank protection storage volume. The outlet (often an orifice) invert is located at the maximum elevation associated with the extended detention water quality volume and is sized to release the streambank protection storage volume over a 24-hour period.

Alternative hydraulic control methods to an orifice can be used and include the use of a broad-crested rectangular, V-notch, proportional weir, or an outlet pipe protected by a hood that extends at least 12 inches below the normal pool.

The water quality outlet (if design is for an extended detention wet or extended detention micropool pond) and streambank protection outlet should be fitted with adjustable gate valves or other mechanism that can be used to adjust detention time.

Higher flows (On-Site and Downstream Flood Control) pass through openings or slots protected by trash racks further up on the riser.

After entering the riser, flow is conveyed through the barrel and is discharged downstream. Anti-seep collars should be installed on the outlet barrel to reduce the potential for pipe failure.

Riprap, plunge pools, or pads, or other energy dissipators are to be placed at the outlet of the barrel to prevent scouring and erosion. If a pond daylights to a channel with dry weather flow, care should be taken to minimize tree clearing along the downstream channel, and to reestablish a forested riparian zone in the shortest possible distance. See *Section 4.0 of the Hydraulics Technical Manual* for more guidance.

Each pond must have a bottom drain pipe with an adjustable valve that can completely or partially drain the pond within 24 hours.

The pond drain should be sized one pipe size greater than the calculated design diameter. The drain valve is typically a handwheel activated knife or gate valve. Valve controls shall be located inside of the riser at a point where they (a) will not normally be inundated and (b) can be operated in a safe manner.

See the design procedures in *Section 22.6* as well as *Sections 2.0 and 2.2 of the Hydraulics Technical Manual* for additional information and specifications on pond routing and outlet works.

F Emergency Spillway

An emergency spillway is to be included in the stormwater pond design to safely pass the extreme flood flow. The spillway prevents pond water levels from overtopping the embankment and causing structural damage. The emergency spillway must be located so that downstream structures will not be impacted by spillway discharges. All local and state dam safety requirements should be met.

A minimum of 1 foot of freeboard must be provided, measured from the top of the water surface elevation for the extreme flood to the lowest point of the dam embankment, not counting the emergency spillway.

G Maintenance Access

A maintenance right of way or easement must be provided to a pond from a public road or easement. Maintenance access should be at least 12 feet wide, have a maximum slope of no more than 15%, and be appropriately stabilized to withstand maintenance equipment and vehicles.

The maintenance access must extend to the forebay, safety bench, riser, and outlet and, to the extent feasible, be designed to allow vehicles to turn around.

Access to the riser is to be provided by lockable manhole covers, and manhole steps should be within easy reach of valves and other controls.

H Safety Features

All embankments and spillways must be designed to State of Texas guidelines for dam safety (see *iSWM Program Guidance – Dams and Reservoirs in Texas).*

Fencing of ponds is not generally desirable, but may be required by the local review authority. A preferred method is to manage the contours of the pond through the inclusion of a safety bench (see above) to eliminate dropoffs and reduce the potential for accidental drowning. In addition, the safety bench may be landscaped to deter access to the pool.

The principal spillway opening should not permit access by small children, and endwalls above pipe outfalls greater than 48 inches in diameter should be fenced to prevent access. Warning signs should be posted near the pond to prohibit swimming and fishing in the facility.

I Landscaping

- Aquatic vegetation can play an important role in pollutant removal in a stormwater pond. In addition, vegetation can enhance the appearance of the pond, stabilize side slopes, serve as wildlife habitat, and can temporarily conceal unsightly trash and debris. Therefore, wetland plants should be encouraged in a pond design, along the aquatic bench (fringe wetlands), the safety bench and side slopes (ED ponds), and within shallow areas of the pool itself. The best elevations for establishing wetland plants, either through transplantation or volunteer colonization, are within 6 inches (plus or minus) of the normal pool elevation. Additional information on establishing wetland vegetation and appropriate wetland species for North Central Texas can be found in the *Landscape Technical Manual.*
- Woody vegetation may not be planted on the embankment or allowed to grow within 15 feet of the toe of the embankment and 25 feet from the principal spillway structure.
- A pond buffer should be provided that extends 25 feet outward from the maximum water surface elevation of the pond. The pond buffer should be contiguous with other buffer areas that are required by existing regulations (e.g., stream buffers) or that are part of the overall stormwater management concept plan. No structures should be located within the buffer, and an additional setback to permanent structures may be provided.
- Existing trees should be preserved in the buffer area during construction. It is desirable to locate forest conservation areas adjacent to ponds. To discourage resident geese populations, the buffer can be planted with trees, shrubs and native ground covers.
- The soils of a pond buffer are often severely compacted during the construction process to ensure stability. The density of these compacted soils is so great that it effectively prevents root penetration and therefore may lead to premature mortality or loss of vigor. Consequently, it is advisable to excavate large and deep holes around the proposed planting sites and backfill these with uncompacted topsoil.
- Fish such as Gambusia affinis can be stocked in a pond to aid in mosquito prevention.
- A fountain or solar-powered aerator may be used for oxygenation of water in the permanent pool.
- Compatible multi-objective use of stormwater pond locations is strongly encouraged.

J Additional Site-Specific Design Criteria and Issues

Physiographic Factors - Local terrain design constraints

- Low Relief – Maximum normal pool depth is limited; providing pond drain can be problematic
- High Relief – Embankment heights restricted
- Karst – Requires poly or clay liner to sustain a permanent pool of water and protect aquifers; limits on ponding depth; geotechnical tests may be required

Soils

- Hydrologic group “A” soils generally require pond liner; group “B” soils may require infiltration testing

Special Downstream Watershed Considerations

- Local Aquatic Habitat – extended detention micropool pond best alternative; design wet ponds and extended detention wet ponds offline and provide shading to minimize thermal impact; limit WQ_v-ED to 12 hours
- Aquifer Protection – Reduce potential groundwater contamination by preventing infiltration of hotspot runoff. May require liner for type “A” and “B” soils; pretreat hotspots; 2 to 4 foot separation distance from water table
- Swimming Area/Shellfish – Design for geese prevention (see the *Landscape Technical Manual*); provide 48-hour extended detention for maximum coliform dieoff.

Dams

Dam construction for stormwater ponds can take a variety of forms. Large dams that are over six feet in height are regulated by the State of Texas (See *iSWM Program Guidance – Dams and Reservoirs in Texas*). Small dams are not as tightly regulated, but require careful attention to design and construction details to ensure that they function properly throughout their designed economic life.

The most commonly used material for small dam construction is earth fill, but structural concrete can also be used. For on-site stormwater controls in high density areas of development or where land values are very costly, the use of a structural concrete dam can save significant amounts of land while making a much more aesthetically appealing outfall structure that the typical riser and barrel assembly.

General

- The dam area shall be cleared, grubbed and stripped of all vegetative material and topsoil prior to dam construction.

Earth Dams

- The dam construction plans shall indicate allowable soil materials to be used, compaction required, locations of core trenches if used, any sub-drainage facilities to be installed to control seepage, plus horizontal and vertical dimensions of the earthen structure.
- The sub-grade of the dam shall be scarified prior to the placement and compaction of the first lift of soil backfill to ensure a good bond between the existing soil and the earthen dam.
- Placement of earth fill shall be in controlled lifts with proper compaction.
- Placement of spillway or outflow pipes through the dam shall be per the plan details, with proper backfill and compaction of any excavated trenches. Hydraulic flooding or other compaction methods of saturated soil shall not be allowed.
- Topsoil and soil additives necessary for the establishment of permanent ground cover above the normal water surface elevation and on the downstream side of the dam shall be installed and seeded as soon as practical to avoid rilling and erosion of the dam’s earthen embankment.
- Do not plant trees or large shrubs on the earth dam, as their root systems cause seepage and damage to the structure.

Concrete Dams

- Concrete dams shall be designed and built in accordance with the American Concrete Institute’s (ACI) latest guidelines for Environmental Engineering Concrete Structures. Particular attention shall be paid to water tightness, crack control, concrete materials and construction practices.
- The construction plans shall indicate materials, plus horizontal and vertical dimensions necessary for the construction of the dam. Details and information shall be provided on joint types and spacing to be used.

- At least one-half of the water surface perimeter of the pond at normal pool elevation shall be constructed with a vegetated earthen embankment.
- Principal and emergency spillways can be incorporated into a weir overflow over the dam if splash pads or another type of control structure is provided to protect the downstream toe of the concrete structure.
- Placement of drain valves, overflow controls and other penetrations of the concrete wall shall not be located on the same vertical line to prevent creating a weakened plane where uncontrolled cracks can form. Locations should also anticipate operation during storm events when overflow weirs will be operating.

22.6 Design Procedures

Step 1 Compute runoff control volumes from the *integrated* Design Focus Areas

Calculate the Water Quality Volume (WQ_v), Streambank Protection Volume (SP_v), and the Flood Protection Storm (Q_f). Design volume should be increased by 15% for extended detention ponds.

Details on the *integrated* Design Focus Areas are found in *Section 1.0 of the Planning Technical Manual.*

Step 2 Determine if the development site and conditions are appropriate for the use of a stormwater pond

Consider the Application and Site Feasibility Criteria in *Sections 22.4 and 22.5* (A) (Location and Siting).

Step 3 Confirm local design criteria and applicability

Consider any special site-specific design conditions/criteria from *Section 22.5 (J)*. (Additional Site-Specific Design Criteria and Issues).

Check with local officials and other agencies to determine if there are any additional restrictions and/or surface water or watershed requirements that may apply.

Step 4 Determine pretreatment volume

A sediment forebay is provided at each inlet, unless the inlet provides less than 10% of the total design storm inflow to the pond. The forebay should be sized to contain 0.1 inches per impervious acre of contributing drainage and should be 4 to 6 feet deep. The forebay storage volume counts toward the total WQ_v requirement and may be subtracted from the WQ_v for subsequent calculations.

Step 5 Determine permanent pool volume (and water quality extended detention volume)

Wet Pond: Size permanent pool volume to 1.0 WQ_v

Extended Detention Wet Pond: Size permanent pool volume to 0.5 WQ_v. Size extended detention volume to 0.5 WQ_v.

Extended Detention Micropool Pond: Size permanent pool volume to 25 to 30% of WQ_v. Size extended detention volume to remainder of WQ_v.

Step 6 Determine pond location and preliminary geometry. Conduct pond grading and determine storage available for permanent pool (and water quality extended detention if extended detention wet pond or extended detention micropool pond)

This step involves initially grading the pond (establishing contours) and determining the elevation-storage relationship for the pond.

- Include safety and aquatic benches.
- Set WQ_v permanent pool elevation (and WQ_v-ED elevation for extended detention wet and extended detention micropool pond) based on volumes calculated earlier.

See *Section 22.5* (C) (Physical Specifications / Geometry) for more details.

Step 7 Compute extended detention orifice release rate(s) and size(s), and establish SP_v elevation

Wet Pond: The SP_v elevation is determined from the stage-storage relationship and the orifice is then sized to release the streambank protection storage volume over a 24-hour period (12-hour extended detention may be warranted in some cold water streams). The streambank protection orifice should have a minimum diameter of 3 inches and should be adequately protected from clogging by an acceptable external trash rack. A reverse slope pipe attached to the riser, with its inlet submerged 1 foot below the elevation of the permanent pool, is a recommended design. The orifice diameter may be reduced to 1 inch if internal orifice protection is used (i.e., an over-perforated vertical stand pipe with ½-inch orifices or slots that are protected by wirecloth and a stone filtering jacket). Adjustable gate valves can also be used to achieve this equivalent diameter.

Extended Detention Wet Pond and Extended Detention Micropool Pond: Based on the elevations established in Step 6 for the extended detention portion of the water quality volume, the water quality orifice is sized to release this extended detention volume in 24 hours. The water quality orifice should have a minimum diameter of 3 inches and should be adequately protected from clogging by an acceptable external trash rack. A reverse slope pipe attached to the riser, with its inlet submerged 1 foot below the elevation of the permanent pool, is a recommended design. Adjustable gate valves can also be used to achieve this equivalent diameter. The SP_v elevation is then determined from the stage-storage relationship. The invert of the streambank protection orifice is located at the water quality extended detention elevation, and the orifice is sized to release the streambank protection storage volume over a 24-hour period (12-hour extended detention may be warranted in some cold water streams).

Step 8 Calculate Q_p release rate and water surface elevation

Set up a stage-storage-discharge relationship for the control structure for the extended detention orifice(s) and the deisgn storm.

Step 9 Design embankment(s) and spillway(s)

Size emergency spillway, calculate flood mitigation stormwater surface elevation, set top of embankment elevation, and analyze safe passage of the flood mitigation storm.

At final design, provide safe passage for the flood mitigation storm event.

Step 10 Investigate potential pond hazard classification

The design and construction of stormwater management ponds are required to follow the latest version of the State of Texas Administrative Code for and reservoirs (see *iSWM Program Guidance - Dams and Reservoirs in Texas*).

Step 11 Design inlets, sediment forebay(s), outlet structures, maintenance access, and safety features.

See *Section 22.5* (D) through (H) for more details.

Step 12 Prepare Vegetation and Landscaping Plan

A landscaping plan for a stormwater pond and its buffer should be prepared to indicate how aquatic and terrestrial areas will be stabilized and established with vegetation.

See *Section 22.5* (I) (Landscaping) and the *Landscape Technical Manual* for more details.

See *Section 29.2* for a Stormwater Pond Design Example

22.7 Inspection and Maintenance Requirements

Table 22.1 Typical Maintenance Activities for Ponds
(Source: WMI, 1997)

Activity	Schedule
• Clean and remove debris from inlet and outlet structures. • Mow side slopes. • Check visually for illegal dumping or other pollutants.	Monthly
• If wetland components are included, inspect for invasive vegetation.	Semiannual Inspection
• Inspect for damage, paying particular attention to the control structure. • Check for signs of eutrophic conditions. • Note signs of hydrocarbon build-up, and remove appropriately. • Monitor for sediment accumulation in the facility and forebay. • Examine to ensure that inlet and outlet devices are free of debris and operational. • Check all control gates, valves or other mechanical devices. • Check downstream face of dam for seepage (earth and concrete), settling (earth) and cracking (concrete).	Annual Inspection
• Repair undercut or eroded areas.	As Needed
• Perform wetland plant management and harvesting.	Annually (if needed)
• Remove sediment from the forebay.	5 to 7 years or after 50% of the total forebay capacity has been lost
• Monitor sediment accumulations, and remove sediment when the pool volume has become reduced significantly, or the pond becomes eutrophic.	10 to 20 years or after 25% of the permanent pool volume has been lost

Additional Maintenance Considerations and Requirements

- A sediment marker should be located in the forebay to determine when sediment removal is required.
- Sediments excavated from stormwater ponds that do not receive runoff from designated hotspots are not considered toxic or hazardous material and can be safely disposed of by either land application or landfilling. Sediment testing may be required prior to sediment disposal when a hotspot land use is present.
- Periodic mowing of the pond buffer is only required along maintenance rights-of-way and the embankment. The remaining buffer can be managed as a meadow (mowing every other year) or forest.
- Care should be exercised during pond drawdowns to prevent downstream discharge of sediments, anoxic water, or high flows with erosive velocities. The approving jurisdiction should be notified before draining a stormwater pond.



Regular inspection and maintenance is critical to the effective operation of stormwater ponds as designed. Maintenance responsibility for a pond and its buffer should be vested with a responsible authority by means of a legally binding and enforceable maintenance agreement that is executed as a condition of plan approval.

22.8 Example Schematics

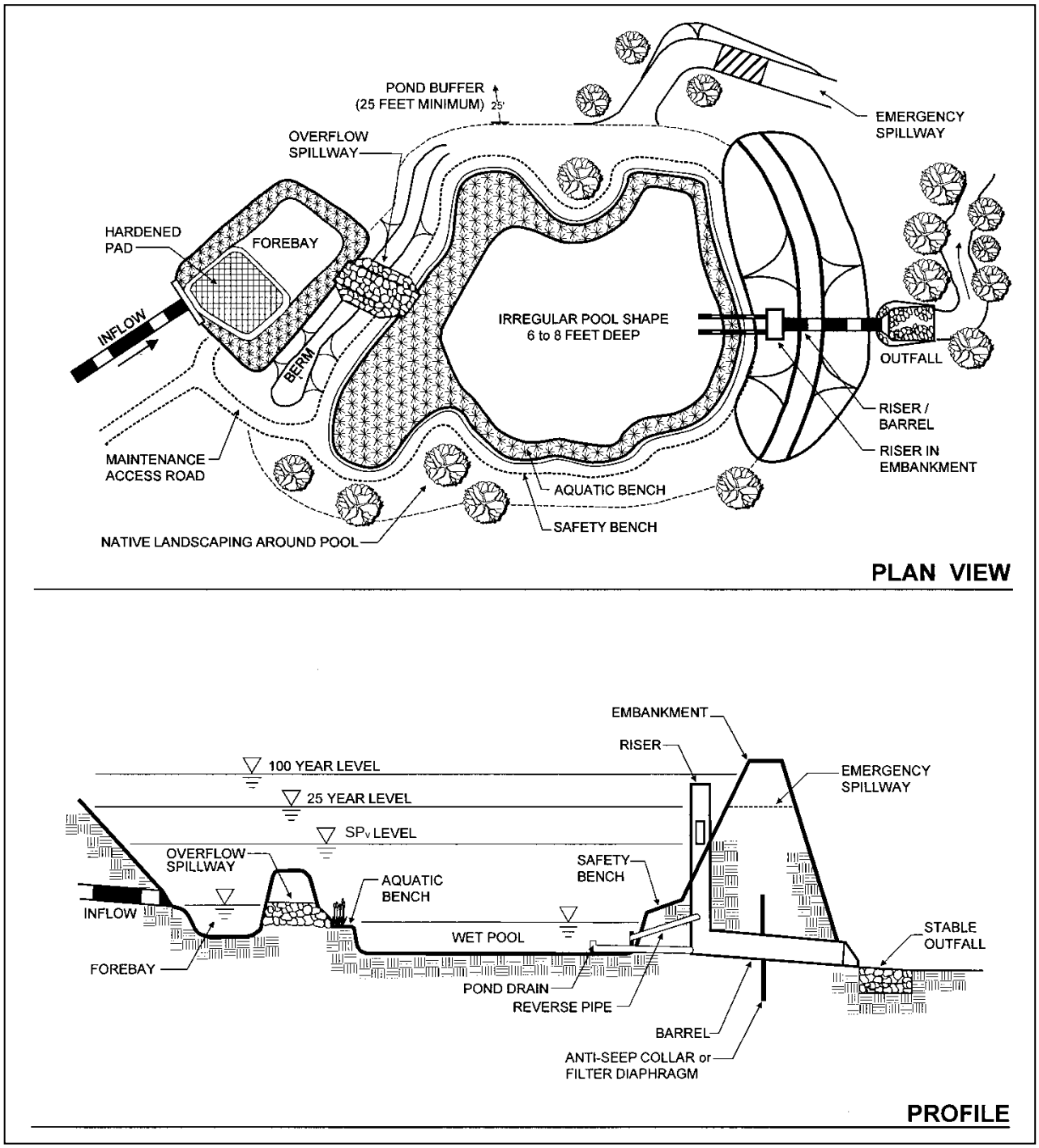


Figure 22.4 Schematic of Wet Pond
(Source: Center for Watershed Protection)

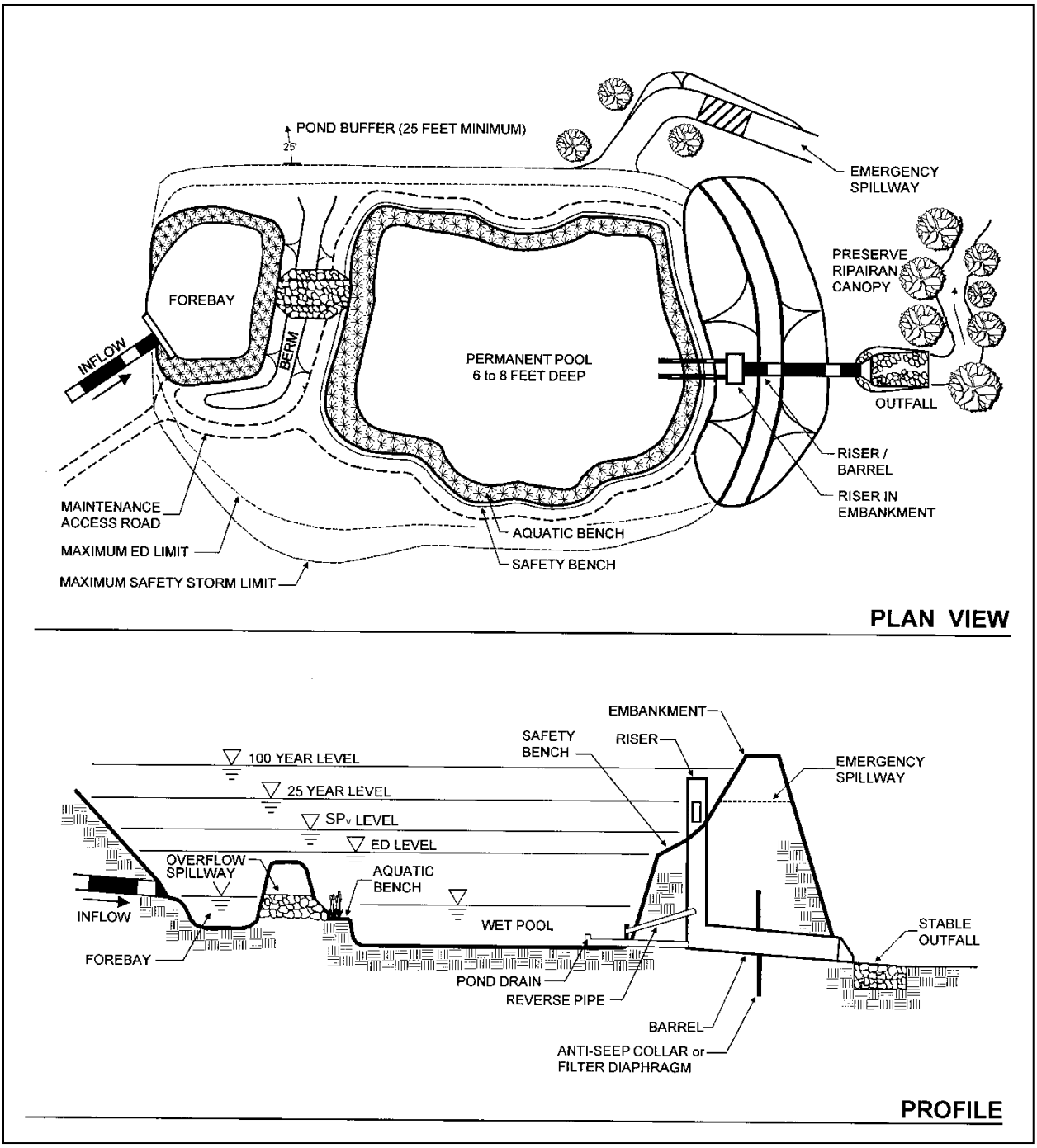


Figure 22.5 Schematic of Wet Extended Detention Pond
(Source: Center for Watershed Protection)

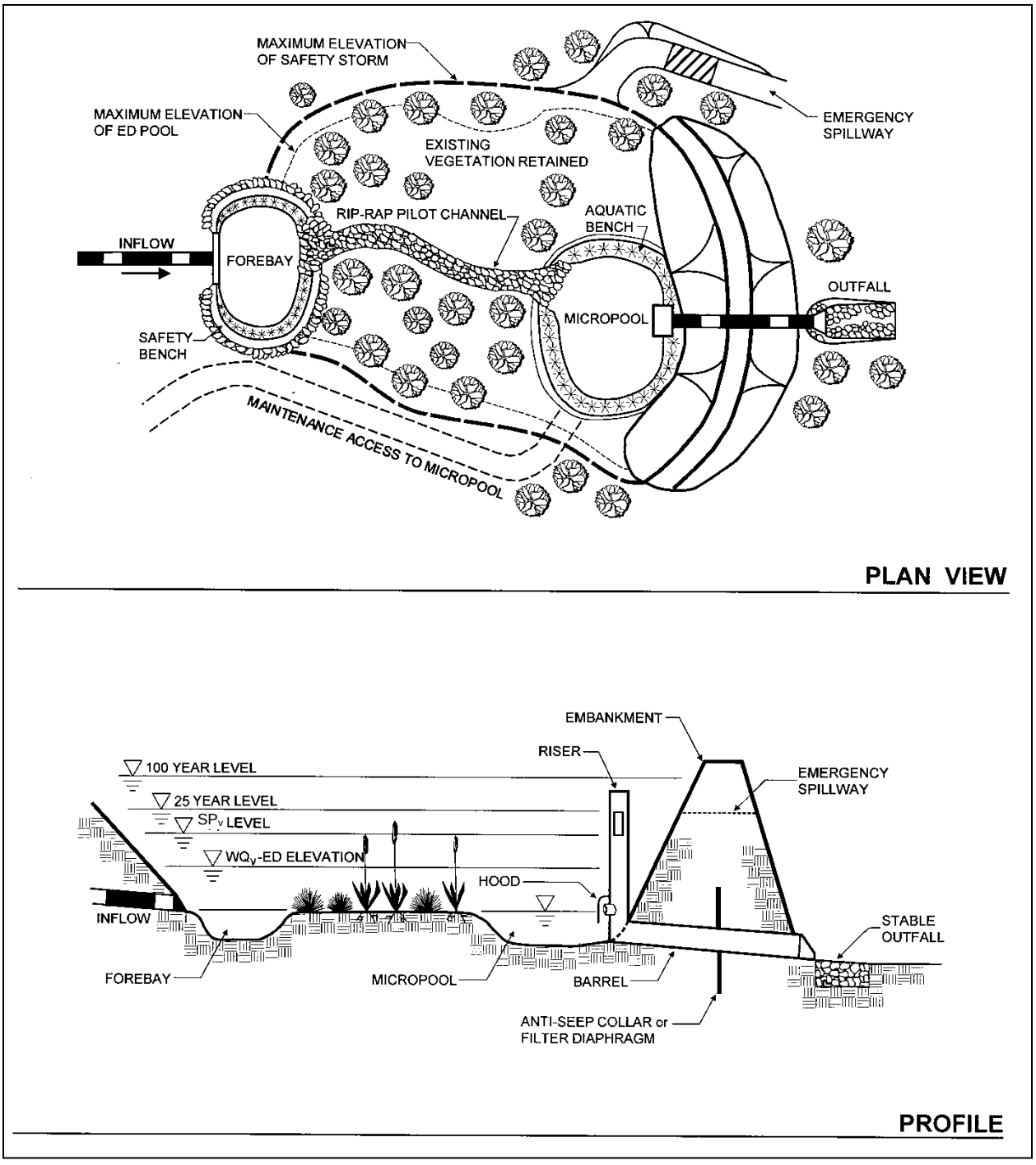


Figure 22.6 Schematic of Micropool Extended Detention Pond
(Source: Center for Watershed Protection)

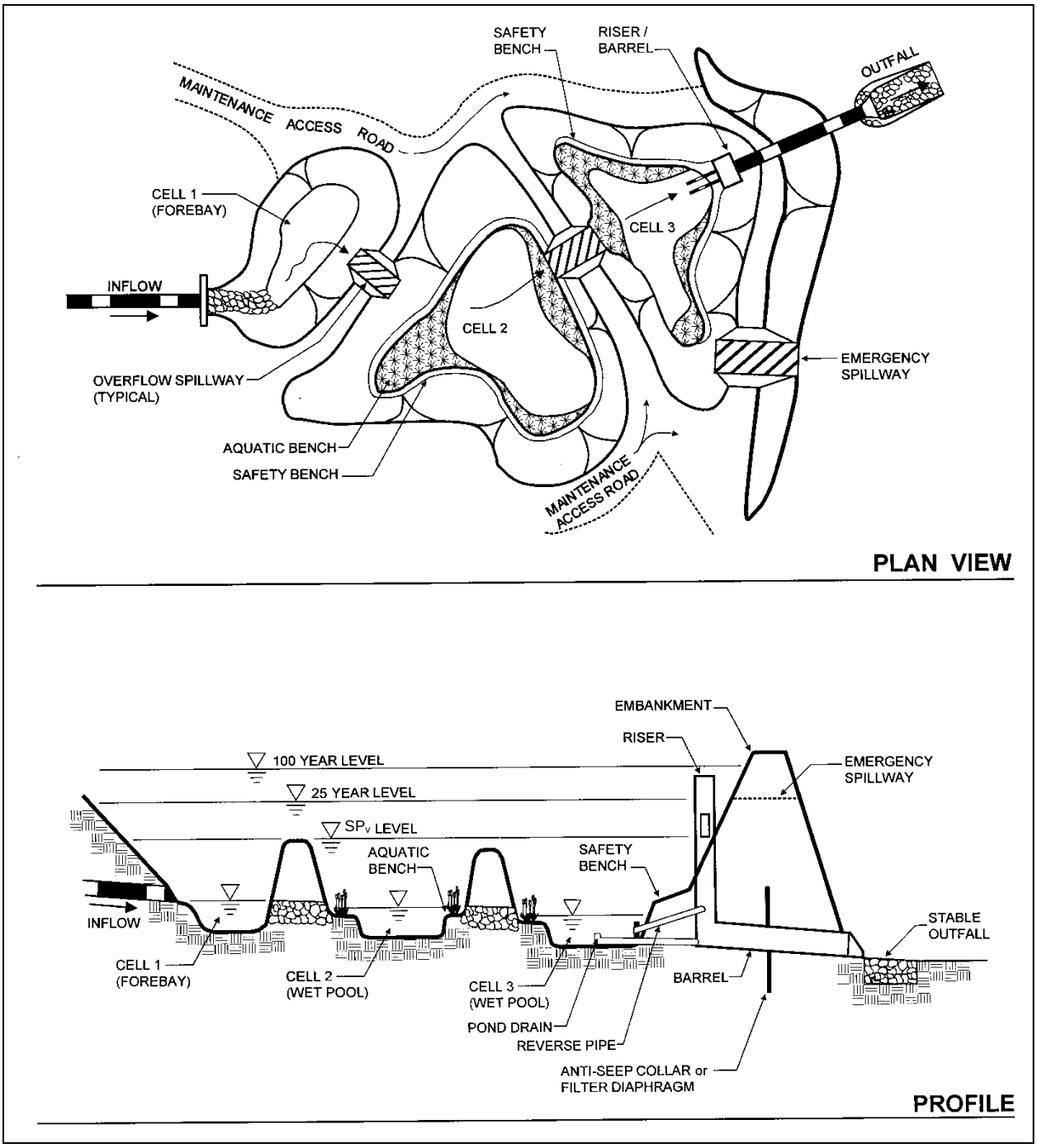


Figure 22.7 Schematic of Multiple Pond System
(Source: Center for Watershed Protection)

22.9 Design Forms

Design Procedure Form: Storm Water Ponds

PRELIMINARY HYDROLOGIC CALCULATIONS

1a. Compute WQ_v Volume requirements
Compute Runoff Coefficient, R_v — R_v = _____
Compute WQ_v Volume requirements — WQ_v = _____ acre-ft

1b. Compute SP_v — SP_v = _____ acre-ft
Compute average release rate — release rate = _____ cfs
Compute Q_p (Required 100-year detention volume) — Q_p = _____ acre-ft
Add 15% to the required Q_p volume (if ED) — Q_p* 15% = _____ acre-ft
Compute (as necessary) Q_f — Q_f = _____ cfs

STORM WATER POND DESIGN

2. Is the use of storm water pond appropriate? — **See subsection 5.2.21.4 and 5.2.21.5-A**

3. Confirm local design criteria and applicability

4. Pretretament volume
$Vol_{pre} = I(0.1")(1'/12")$ — Vol_{pre} = _____ acre-ft

5. Allocation of Permanent Pool Volume and ED Volume

Wet Pond:	$Vol_{pool} = WQ_v$	Vol_{pool} = _____ acre-ft
Wet ED Pond:	$Vol_{pool} = 0.5(WQ_v)$	Vol_{pool} = _____ acre-ft
	$Vol_{ED} = 0.5(WQ_v)$	Vol_{ED} = _____ acre-ft
Micropool ED Pond:	$Vol_{pool} = 0.25(WQ_v)$	Vol_{pool} = _____ acre-ft
	$Vol_{ED} = 0.75(WQ_v)$	Vol_{ED} = _____ acre-ft

6. Conduct grading and determine storage available for permanent pool (and WQ_v-ED volume if applicable) — Prepare an elevation-storage table and curve using the average area method for computing volumes.

Elevation MSL	Area ft^2	Average Area ft^2	Depth ft	Volume ft^3	Cumulative Volume ft^3	Cumulative Volume acre-ft	Volume above Permanent Pool acre-ft

7.	WQ_v Orifice Computations	release rate = _____ cfs
	Average ED release rate (if applicable)	h = _____ ft
	Average head, h = (ED elev. - Permanent pool elev.) / 2	
	Area of orifice from orifice equation	A = _____ ft^2
	$Q = CA(2gh)^{0.5}$	diameter = _____ in
		factor = _____ $(h)^{0.5}$
	Discharge equation $Q = (h)^{0.5}$	
	Compute release rate for SP_v-ED control and establish SP_v elevation	WSEL = _____ ft-NGVD
		release rate = _____ cfs
	Release rate =	h = _____ ft-NGVD
	Aerage head, h = SP_v elev. - Permanent pool elev.) / 2	A = _____ ft^2
	Area of orifice from orifice equatin	diameter = _____ in
	$Q = CA(2gh)^{0.5}$	factor = _____ $(h)^{0.5}$
	Discharge equation $Q = (h)^{0.5}$	
8.	Calculate Qp release rate and WSEL	**Set up a stage-storage-discharge relationship**

Elevation	Storage	Low Flow	Riser			Barrel		Emergency	Total
		WQv-ED	SPv-ED	High Storage		Inlet	Pipe	Spillway	Storage
				Orif.	Weir				
MSL	ac-ft	H(ft) Q(cfs)	H(ft) Q(cfs)	HQ	HQ	H(ft) Q(cfs)	H(ft) Q(cfs)	H(ft) Q(cfs)	Q(cfs)

	Q_p = pre-dev. Peak discharge - (WQ_v-ED release + SP_v-ED release)	Q_p = _____ cfs
	Maximum head =	H = _____ ft
	Use weir equation for slot length ($Q = CLH^{3/2}$)	L = _____ ft
	Check inlet condition Check outlet condition	**Use culvert charts (section 4.3)**
9.	Size emergency spillway, calculate 100-year WSEL and set top of embankment elevation	$WSEL_{25}$ = _____ ft $WSEL_{100}$ = _____ ft Q_{ES} = _____ cfs Q_{PS} = _____ cfs
10.	Investigate potential pond hazard classification	**See Appendix H**
11.	Design inlets, sediment forebays, outlet structures, maintenance access, and safety features.	**See subsection 3.2.15 - D through H**
12.	Attach landscaping plan	**See Appendix F**

23.0 Green Roof

Structural Stormwater Control



Description: A green roof uses a small amount of substrate over an impermeable membrane to support a covering of plants. The green roof slows down runoff from the otherwise impervious roof surface as well as moderating rooftop temperatures. With the right plants, a green roof will also provide aesthetic or habitat benefits. Green roofs have been used in Europe for decades.

KEY CONSIDERATIONS

DESIGN CRITERIA:

- Relatively new in North America
- Potential for high failure rate if poorly designed, poorly constructed, not adequately maintained Minimum length to width ratio for the pond is 1.5:1

ADVANTAGES / BENEFITS:

- Provides reduction in runoff volume
- Higher initial cost when compared to conventional roofs, but potential for lower life cycle costs through longevity

DISADVANTAGES / LIMITATIONS:

- Requires additional roof support
- Requires more maintenance than regular roofs
- Special attention to design and construction needed
- Requires close coordination with plant specialists
- Potential for leakage due to plant roots penetrating membrane.

POLLUTANT REMOVAL

85%	**Total Suspended Solids**
95/16%	**Nutrients** - Total Phosphorus / Total Nitrogen removal
25%	**Metals** - Cadmium, Copper, Lead, and Zinc removal
No Data	**Pathogens** - Coliform, Streptococci, E. Coli removal

STORMWATER MANAGEMENT SUITABILITY

P	**Water Quality Protection**
S	**Streambank Protection**
	On-Site Flood Control
	Downstream Flood Control

IMPLEMENTATION CONSIDERATIONS

L	**Land Requirement**
L	**Capital Cost**
H	**Maintenance Burden**

Residential Subdivision Use: No
High Density/Ultra-Urban: Yes
Drainage Area: No restrictions.
Soils: No restrictions.
Other Considerations:

- Hotspot Areas

L=Low **M**=Moderate **H**=High

23.1 General Description

Green roofs (also referred to as *ecoroofs*, *roof gardens, or roof meadows*) are vegetated roofs used in place of conventional roofing, such as gravel-ballasted roofs. They are used as part of sustainable development initiatives, along with narrow streets, permeable pavement, and various infiltration devices. There are two main types of green roofs. The first is what is called roof gardens or intensive green roofs. They may be thought of as a garden on the roof. They have a greater diversity of plants, including trees and shrubs, but require deeper soil, increased load bearing capacity, and require more maintenance. The second has been referred to as roof meadows or extensive green roofs. The vegetation is limited and similar to an alpine meadow, requiring less soil depth and minimal maintenance. Due to the considerably greater costs and structural design requirements, only the second type of green roof, the roof meadow or extensive type is discussed in this manual.

The extensive green roof is designed to control low-intensity storms by intercepting and retaining or storing water until the peak storm event has passed. The plants intercept and delay runoff by capturing and holding precipitation in the foliage, absorbing water in the root zone, and slowing the velocity of direct runoff by increasing retardance to flow and extending the flowpath through the vegetation. Water is also stored and evaporated from the growing media. Green roofs can capture and evaporate up to 100 percent of the incident precipitation, depending on the roof design and the storm characteristics.

Monitoring in Pennsylvania, for instance, showed reductions of approximately 2/3 in runoff from a green roof (15.5 inches runoff from 44 inches of rainfall). Furthermore, runoff was negligible for storm events of less than 0.6 inches. A study done for Portland, Oregon, indicated a reduction in stormwater discharges from the downtown area of between 11 and 15% annually if half of the roofs in the downtown area were retrofitted as green roofs.

Green roofs also:

- reduce the temperature of runoff,
- reduce the “heat island” effect of urban buildings,
- help insulate the building,
- improve visual aesthetics,
- protect roofs from weather,
- improve building insulation,
- reduce noise,
- and provide habitat for wildlife.

As with a conventional roof, a green roof must safely drain runoff from the roof. It may be desirable to drain the runoff to a rainwater harvesting system such as (rainbarrels or cisterns), or other stormwater facilities such as planters and swales.

Significant removals of heavy metals by green roofs have been reported, but there is not enough evidence to include removal rates at this time.

23.2 Design Criteria and Specifications

For either new installations or retrofits, an architect or structural engineer must be consulted to determine whether the building can provide the structural support needed for a green roof.

Generally, the building structure must be adequate to hold an additional 10 to 25 pounds per square foot (psf) saturated weight, depending on the vegetation and growth medium that will be used. (This is in addition to snow load requirements.) An existing rock ballast roof may be structurally sufficient to hold a 10-12 psf green roof, since ballast typically weighs 10-12 psf.

Green roofs can be used on flat or pitched roofs up to 40 percent. Although, on a roof slope greater than 20 degrees, the green roof installer needs to ensure that the plant layer does not slip or slump through its

own weight, especially when it becomes wet. Horizontal strapping, wood, plastic, or metal, may be necessary. Some commercial support grid systems are also available for this purpose.

A green roof typically consists of several layers, as shown in Figure 23.1. A waterproof membrane is placed over the roof's structure. A root barrier is placed on top of the membrane to prevent roots from penetrating the membrane and causing leaks. A layer for drainage is installed above this, followed by the growth media. The vegetation is then planted to form the top layer. Details of the various layers are given below.

Waterproof membranes are made of various materials, such as synthetic rubber (EPDM), hypolan (CPSE), reinforced PVC, or modified asphalts (bitumens). The membranes are available in various forms, liquid, sheets, or rolls. Check with the manufacturer to determine their strength and functional characteristics of the membrane under consideration.

Root barriers are made of dense materials or are treated with copper or other materials that inhibit root penetration, protecting the waterproof membrane from being breached. A root barrier may not be necessary for synthetic rubber or reinforced PVC membranes, but will likely be needed for asphalt mixtures. Check with the manufacturer to determine if a root barrier is required for a particular product.

The drainage layer of a green roof is usually constructed of various forms of plastic sheeting, a layer of gravel, or in some cases, the growth medium.

The growth medium is generally 2 to 6 inches thick and made of a material that drains relatively quickly. Commercial mixtures containing coir (coconut fiber), pumice, or expanded clay are available. Sand, gravel, crushed brick, and peat are also commonly used. Suppliers recommend limiting organic material to less than 33% to reduce fire hazards. The City of Portland, Oregon has found a mix of 1/3 topsoil, 1/3 compost, and 1/3 perlite may be sufficient for many applications. Growth media can weigh from 16 to 35 psf when saturated depending on the type (intensive/extensive), with the most typical range being from 10-25 psf.

When dry, all of the growth media are light-weight and prone to wind erosion. It is important to keep media covered before planting and ensure good coverage after vegetation is established.

Selecting the right vegetation is critical to minimize maintenance requirements. Due to the shallowness of the growing medium and the extreme desert-like microclimate on many roofs, plants are typically alpine, dryland, or indigenous. Ideally, the vegetation should be:

- Drought-tolerant, requiring little or no irrigation after establishment
- Self-sustaining, without fertilizers, pesticides, or herbicides
- Able to withstand heat, cold, and high winds
- Shallow root structure
- Low growing, needing little or no mowing or trimming
- Fire resistant
- Perennial or self propagating, able to spread and cover blank spots by itself

Visit www.txsmartscape.com to look up plants meeting the above criteria.

A mix of sedum/succulent plant communities is recommended because they possess many of these attributes. Certain wildflowers, herbs, forbs, grasses, mosses, and other low groundcovers can also be used to provide additional habitat benefits or aesthetics; however, these plants need more watering and maintenance to survive and keep their appearance.

Green roof vegetation is usually established by one or more of the following methods: seeding, cuttings, vegetation mats, and plugs/potted plants.

- Seeds can be either hand sown or broadcast in a slurry (hydraseeded). Seeding takes longer to establish and requires more weeding, erosion control, and watering than the other methods.
- Cuttings or sprigs are small plant sections. They are hand sown and require more weeding, erosion control, and watering than mats.

- Vegetation mats are sod-like mats that achieve full plant coverage very quickly. They provide immediate erosion control, do not need mulch, and minimize weed intrusion. They generally require less ongoing maintenance than the other methods.
- Plugs or potted plants may provide more design flexibility than mats. However, they take longer to achieve full coverage, are more prone to erosion, need more watering during establishment, require mulching, and more weeding.

Green roof vegetation is most easily established during the spring or fall.

Irrigation is necessary during the establishment period and possibly during drought conditions, regardless of the planting method used. The goal is to minimize the need for irrigation by paying close attention to plant selection, soil, and various roof characteristics.

Installation costs for green roofs generally run from $10 to $25 per square foot, as compared to $3 to $20 per square foot for a conventional roof. However, the longer lifespan of a green roof (reportedly 40 years or up to twice as long as a conventional roof) and lower maintenance costs offset this.

Provide controlled overflow point(s) to prevent overloading of roof.

23.3 Inspection and Maintenance Requirements

Table 23.1 Typical Maintenance Activities for Green Roofs

Activity	Schedule
Watering to help establish vegetation	As needed
Replant to cover bare spots or dead plants	Monthly
Weeding (as needed, based on inspection)	Two or three times yearly
Water and mowing to prevent fire hazards (if grasses or similar plants are used)	As needed
Inspect drains for clogging	Twice per year
Inspect the roof for leakage	Annually, or as needed
If leaks occur, remove and stockpile vegetation, growth media, and drainage layer. Replace membrane and root barrier, followed by stockpiled material.	Upon failure

23.4 Example Schematic

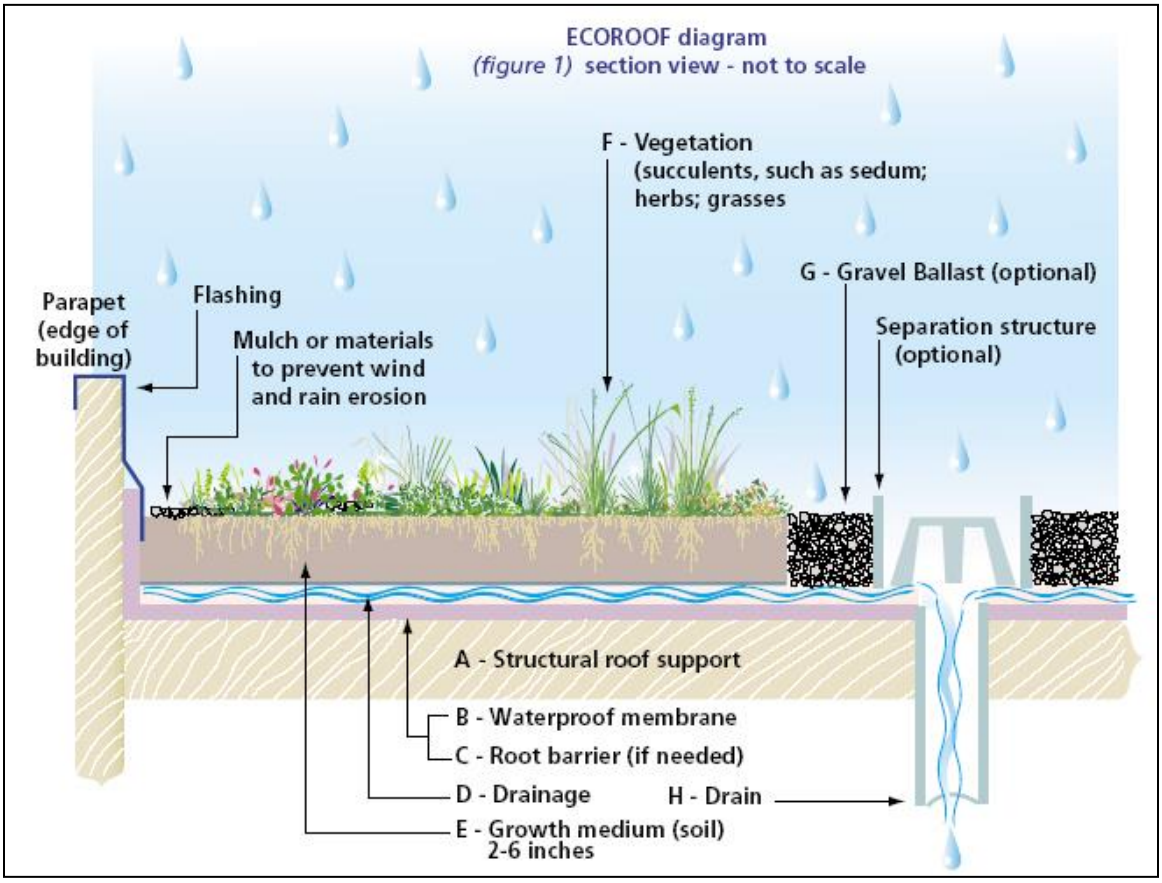


Figure 23.1 Green Roof Cross Section (from City of Portland, Oregon)

24.0 Modular Porous Paver Systems

Structural Stormwater Control

North Central Texas Council of Governments

integrated Stormwater Management iSWM

Description

Permeable pavement is a structural alternative to a paved surface that allows for the infiltration of stormwater runoff through void spaces into a stone bed and the soil or an under-drain below. Permeable pavement can refer to a variety of surfaces, including porous asphalt, pervious concrete, and permeable interlocking concrete pavers. It is intended for use in lightly trafficked areas, such as parking lots, driveways, plazas, and rights-of-way.

Design Considerations

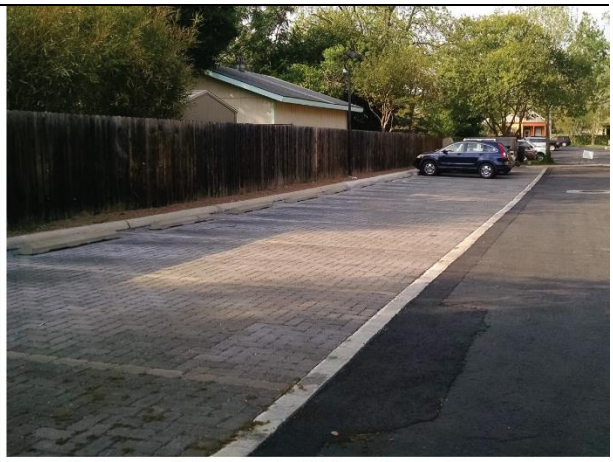
- Consists of structural units with void areas that are typically filled with pervious materials such as course sand, gravel, or turf
- Intended for low traffic areas, or for residential or overflow parking applications
- Soil types need to be considered—an infiltration rate of 0.5 to 3 inches/hour is required
- The ratio of the contributing impervious area to the porous paver surface should be no more than 3:1
- Slopes should be less than 5%, but preferably less than 2%
- A minimum of 2 feet of clearance between the bottom of the gravel and the seasonally high groundwater table is required

Key Advantages

- Permeable pavement provides a reduction in runoff volume
- There is a high level of pollutant removal with these facilities
- Some types of permeable pavement can be purchased from commercial vendors

Limitations

- There are high maintenance requirements associated with permeable pavement
- Permeable pavement can fail if designed incorrectly, placed in unstabilized areas, or if maintenance is not properly done
- Permeable pavement has the potential for groundwater contamination
- Cannot be used in areas where contamination is possible (ex. industrial sites)



Permeable Pavement in San Antonio, TX. *Source: Tetra Tech*

Target Constituent	Removal Rate (0% → 100%)
Total Suspended Solids	*not applicable*
Total Phosphorus	
Total Nitrogen	
Fecal Coliform	*insufficient data*
Heavy Metals	

Implementation Considerations



Suitability

The iSWM manual has designated that permeable pavement facilities are suitable for providing:



Maintenance

- Trash, leaf, debris and sediment removal
- Vacuum or sweep the surface
- Re-chip or reseal pavement when appropriate
- Replace fill material as needed
- Clear underdrain pipes of debris
- Perform structural repairs as needed
- Mow grass when using a permeable paver grid system

Published April 2020

24.1 General Description

Modular porous pavers are structural units, such as concrete blocks, bricks, or reinforced plastic mats, with regularly interspersed void areas used to create a load-bearing pavement surface. The void areas are filled with pervious materials (gravel, sand, or grass turf) to create a system that allows for the infiltration of stormwater runoff. Porous paver systems provide water quality benefits in addition to groundwater recharge and a reduction in stormwater volume. The use of porous paver systems results in a reduction of the effective impervious area on a site.

There are many different types of modular porous pavers available from different manufacturers, including both pre-cast and mold in-place concrete blocks, concrete grids, interlocking bricks, and plastic mats with hollow rings or hexagonal cells (see Figure 24.1).

Modular porous pavers are typically placed on a gravel (stone aggregate) base course. Runoff infiltrates through the porous paver surface into the gravel base course, which acts as a storage reservoir as it infiltrates to the underlying soil. The infiltration rate of the soils in the subgrade must be adequate to support drawdown of the entire runoff capture volume within 24 to 48 hours. Special care must be taken during construction to avoid undue compaction of the underlying soils, which could affect the soils' infiltration capability.

Modular porous paver systems are typically used in low-traffic areas such as the following types of applications:

- Parking pads in parking lots
- Overflow parking areas
- Residential driveways
- Residential street parking lanes
- Recreational trails
- Golf cart and pedestrian paths
- Emergency vehicle and fire access lanes

A major drawback is the cost and complexity of modular porous paver systems compared to conventional pavements. Porous paver systems require a very high level of construction workmanship to ensure that they function as designed and do not settle unevenly. In addition, there is the difficulty and cost of rehabilitating the surfaces should they become clogged. Therefore, consideration of porous paver systems should include the construction and maintenance requirements and costs.

24.2 Pollutant Removal Capabilities

As they provide for the infiltration of stormwater runoff, porous paver systems have a high removal of both soluble and particulate pollutants, where they become trapped, absorbed, or broken down in the underlying soil layers. Due to the potential for clogging, porous paver surfaces should not be used for the removal of sediment or other coarse particulate pollutants.

The following design pollutant removal rates are conservative average pollutant reduction percentages for design purposes derived from sampling data, modeling, and professional judgment.

- Total Suspended Solids – not applicable
- Total Phosphorus – 80%
- Total Nitrogen – 80%
- Fecal Coliform – insufficient data
- Heavy Metals – 90%

24.3 Design Criteria and Specifications

Porous paver systems can be used where the underlying in-situ subsoils have an infiltration rate of between 0.5 and 3.0 inches per hour. Therefore, porous paver systems are not suitable on sites with hydrologic group D or most group C soils, or soils with a high (>30%) clay content. During construction and preparation of the subgrade, special care must be taken to avoid compaction of the soils.

Porous paver systems should ideally be used in applications where the pavement receives tributary runoff only from impervious areas. The ratio of the contributing impervious area to the porous paver surface area should be no greater than 3:1.

If runoff is coming from adjacent pervious areas, it is important that those areas be fully stabilized to reduce sediment loads and prevent clogging of the porous paver surface.

Porous paver systems are not recommended on sites with a slope greater than 2%.

A minimum of 2 feet of clearance is required between the bottom of the gravel base course and underlying bedrock or the seasonally high groundwater table.

Porous paver systems should be sited at least 10 feet downgradient from buildings and 100 feet away from drinking water wells.

An appropriate modular porous paver should be selected for the intended application. A minimum of 40% of the surface area should consist of open void space. If it is a load bearing surface, then the pavers should be able to support the maximum load.

The porous paver infill is selected based upon the intended application and required infiltration rate. Masonry sand (such as ASTM C-33 concrete sand or TxDOT item 421 Fine Aggregate) has a high infiltration rate (8 in/hr) and should be used in applications where no vegetation is desired. A sandy loam soil has a substantially lower infiltration rate (1 in/hr), but will provide for growth of a grass ground cover.

A 1-inch top course (filter layer) of sand (ASTM C-33 concrete sand or TxDOT item 421 Fine Aggregate) underlain by filter fabric is placed under the porous pavers and above the gravel base course.

The gravel base course should be designed to store at a minimum the water quality protection volume (WQ_v). The stone aggregate used should be washed, bank-run gravel, 1.5 to 2.5 inches in diameter with a void space of about 40% (ASTM C-33 Size No. 3 Coarse Aggregate). Aggregate contaminated with soil shall not be used. A porosity value (void space/total volume) of 0.32 should be used in calculations.

The gravel base course must have a minimum depth of 9 inches. The following equation can be used to determine if the depth of the storage layer (gravel base course) needs to be greater than the minimum depth:

$$\mathbf{d = V / A * n} \qquad \mathbf{(24.1)}$$

where:

- d = Gravel Layer Depth (feet)
- V = Water Quality Protection Volume –or– Total Volume to be Infiltrated (cubic feet)
- A = Surface Area (square feet)
- n = Porosity (use n=0.32)

The surface of the subgrade should be lined with filter fabric or an 8-inch layer of sand (ASTM C-33 concrete sand or TxDOT item 421 Fine Aggregate) and be completely flat to promote infiltration across the entire surface.

Porous paver system designs must use some method to convey larger storm event flows to the conveyance system. One option is to use storm drain inlets set slightly above the elevation of the pavement. This would allow for some ponding above the surface, but would accept bypass flows that are too large to be infiltrated by the porous paver system, or if the surface clogs.

For the purpose of sizing downstream conveyance and structural control system, porous paver surface areas can be assumed to be 35% impervious. In addition, a reduction in water quality volume requirements can be obtained for the runoff volume infiltrated from other impervious areas using the methodology in *Section 1.0 of the Planning Technical Manual*.

24.4 Inspection and Maintenance Requirements

Table 24.1 Typical Maintenance Activities for Modular Porous Paver Systems	
Activity	**Schedule**
• Ensure that the porous paver surface is free of extraneous sediment. • Check to make sure that the system dewaters between storms.	Monthly
• Clear debris from contributing area and porous paver surface. • Stabilize and mow contributing adjacent areas and remove clippings.	As needed, based on inspection
• Vacuum sweep porous paver surface to keep free of sediment.	Typically three to four times a year
• Inspect the surface for deterioration or spalling.	Annually
• Totally rehabilitate the porous paver system, including the top and base course.	Upon failure

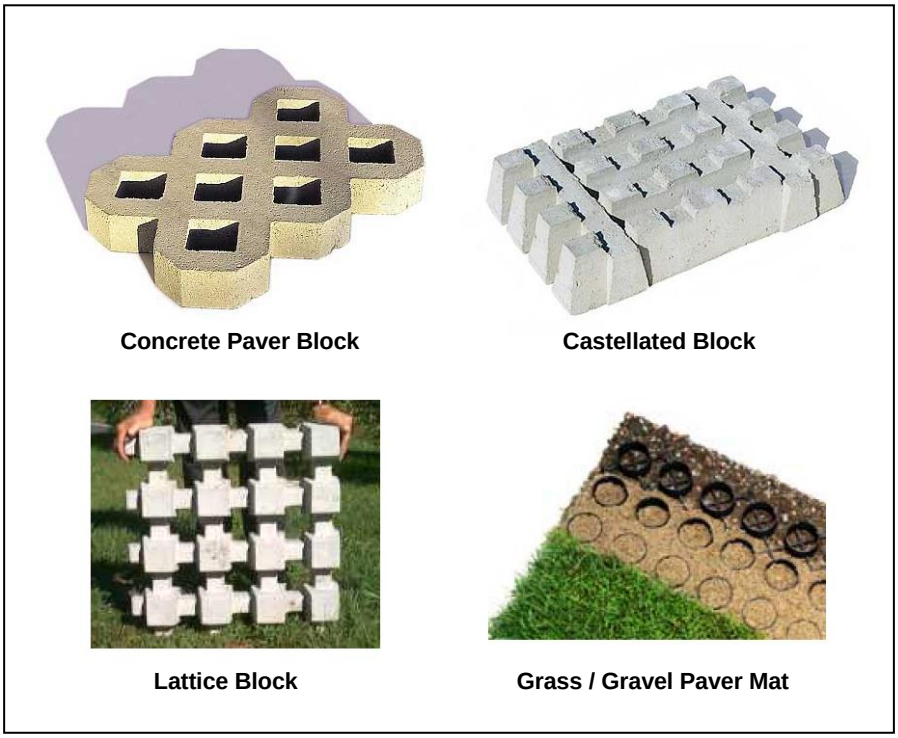


Figure 24.1 Examples of Modular Porous Pavers

24.5 Example Schematics

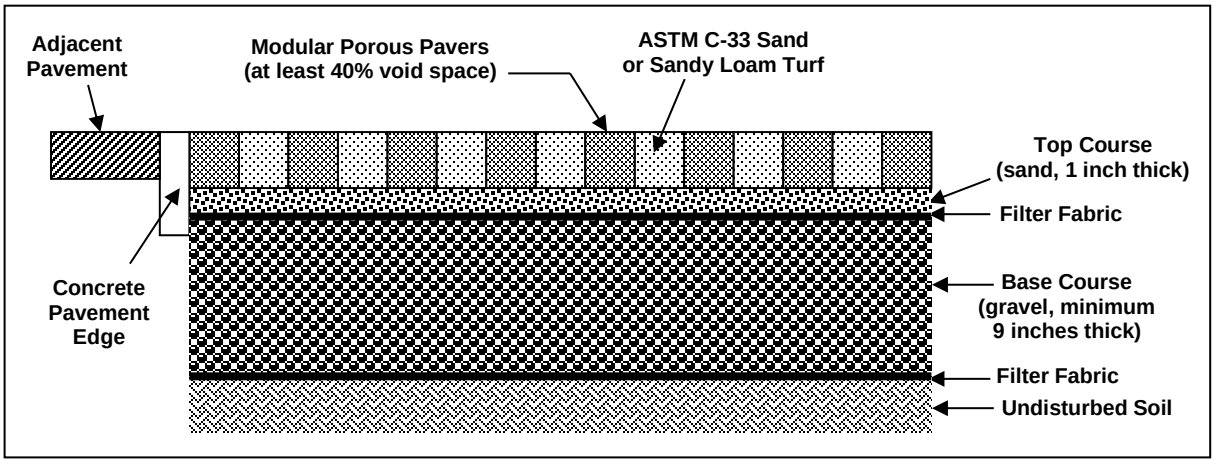


Figure 24.2 Modular Porous Paver System Section

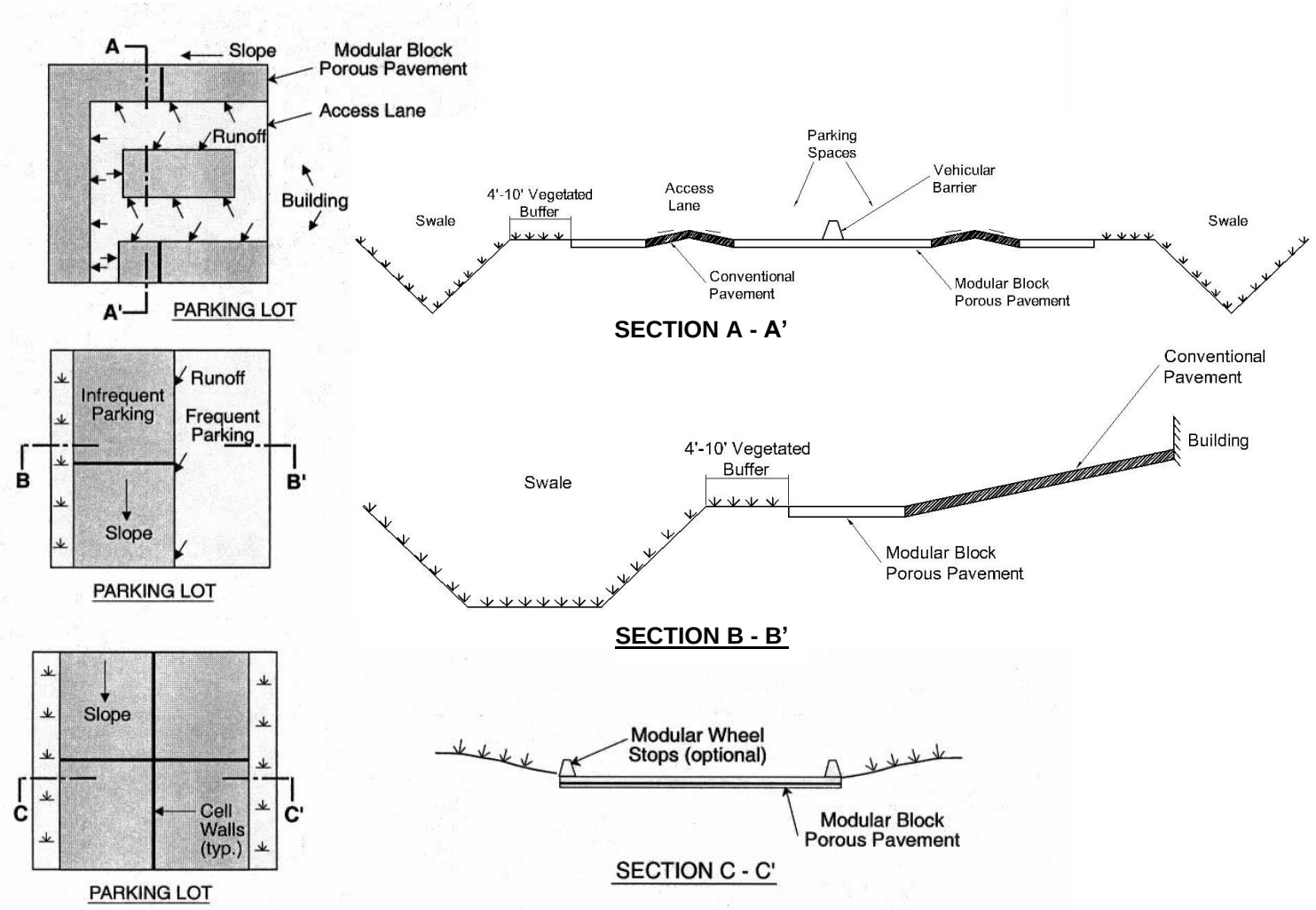


Figure 24.3 Typical Modular Porous Paver System Applications
(Source: UDFCD, 1999)



Figure 24.4 Examples of Porous Paver Surfaces
(Sources: Invisible Structures, Inc.; EP Henry Corp.)

25.0 Porous Concrete

Limited Application
Structural Stormwater Control

North Central Texas Council of Governments
integrated Stormwater Management iSWM



Porous Concrete in San Diego, CA. *(Source: Tetra Tech)*

Description

Porous concrete is a specific type of Permeable Pavement that contains a mixture of coarse aggregate, Portland cement and water overlaying a stone aggregate reservoir. The mixture allows for rapid infiltration of water by providing temporary storage. Runoff infiltrates into an underdrain system or underlying permeable soils.

Design Considerations

- Porous concrete is intended for low volume automobile traffic areas, including overflow parking areas.
- Soil types need to be considered—an infiltration rate of 0.5 to 3 inches/hour is required (unless an underdrain is installed).
- Should not be used in wellhead protection zones or recharge areas of water supply aquifer recharge zones.
- The BMP consists of an excavated area with stone media, gravel and sand filter layers, and typically includes an observation well.
- This BMP should be monitored over the entire lifespan.
- The facility should infiltrate the runoff capture volume in 24–48 hours.

Key Advantages

- Can be used in place of traditional paved surfaces for many applications.
- Porous concrete provides a reduction in runoff volume.
- There is a high level of pollutant removal with these facilities.
- Provides ancillary benefits, such as reduced heat island effect.

Limitations

- There are restrictions on heavy vehicle loading.
- There are high maintenance requirements associated with porous concrete.
- Design and construction may require special attention.
- The cost compared to conventional pavements is high.
- Porous concrete can fail if designed incorrectly, placed in unstabilized areas, or if maintenance is not properly performed.
- Porous concrete has the potential for groundwater contamination.
- Cannot be used in areas where contamination is possible (ex. industrial sites).

Target Constituent	Removal Rate (0% → 100%)
Total Suspended Solids	not applicable
Total Phosphorus	
Total Nitrogen	
Fecal Coliform	0%
Heavy Metals	

Implementation Considerations



Suitability

The iSWM manual has designated porous concrete as suitable for providing:



Maintenance

- Trash, leaf, debris and sediment removal.
- Vacuum or sweep the surface.
- Clear underdrain pipes of debris.
- Perform structural repairs as needed.
- Porous concrete must never be seal coated.

For a similar design, see the Summary Page for Modular Porous Pavement Systems

Published October 2020

25.1 General Description

Porous concrete (also referred to as *enhanced porosity concrete*, *porous concrete*, *portland cement pervious pavement,* and *pervious pavement*) is a subset of a broader family of pervious pavements including porous asphalt, and various kinds of grids and paver systems. Porous concrete is thought to have a greater ability than porous asphalt to maintain its porosity in hot weather and thus is provided as a limited application control in this manual. Although, porous concrete has seen growing use, there is still very limited practical experience with this measure. According to the U.S. EPA, porous pavement sites have had a high failure rate – approximately 75 percent. Failure has been attributed to poor design, inadequate construction techniques, soils with low permeability, heavy vehicular traffic, and poor maintenance. This measure, if used, should be carefully monitored over the life of the development.

Porous concrete consists of a specially formulated mixture of portland cement; uniform, open graded course aggregate; and water. The concrete layer has a high permeability often many times that of the underlying permeable soil layer which allows rapid percolation of rainwater through the surface and into the layers beneath. The void space in porous concrete is in the 15% to 22% range compared to three to five percent for conventional pavements. The permeable surface is placed over a layer of open-graded gravel and crushed stone. The void spaces in the stone act as a storage reservoir for runoff.

Porous concrete is designed primarily for stormwater quality, i.e. the removal of stormwater pollutants. However, it can provide limited runoff quantity control, particularly for smaller storm events. For some smaller sites, trenches can be designed to capture and infiltrate the streambank protection volume (SP_v) in addition to WQ_v. Porous concrete will need to be used in conjunction with another structural control to provide downstream flood control, if required.

Modifications or additions to the standard design have been used to pass flows and volumes in excess of the water quality volume, or to increase storage capacity or treatment. These include:

- Placing a perforated pipe near the top of the crushed stone reservoir to pass excess flows after the reservoir is filled
- Providing surface detention storage in a parking lot, adjacent swale, or detention pond with suitable overflow conveyance
- Connecting the stone reservoir layer to a stone filled trench
- Adding a sand layer and perforated pipe beneath the stone layer for filtration of the water quality volume
- Placing an underground detention tank or vault system beneath the layers

The infiltration rate of the soils in the subgrade should be adequate to support drawdown of the entire runoff capture volume within 24 to 48 hours. Special care must be taken during construction to avoid undue compaction of the underlying soils which could affect the soils' infiltration capability.

Porous concrete systems are typically used in low-traffic areas such as the following types of applications:

- Parking pads in parking lots
- Overflow parking areas
- Residential street parking lanes
- Recreational trails
- Golf cart and pedestrian paths
- Emergency vehicle and fire access lanes

Slopes should be flat or gentle to facilitate infiltration versus runoff and the seasonally high water table or bedrock should be a minimum of two feet below the bottom of the gravel layer if infiltration is to be relied on to remove the stored volume.

Porous concrete has the positive characteristics of volume reduction due to infiltration, groundwater recharge, and an ability to blend into the normal urban landscape relatively unnoticed. It also allows a

reduction in the cost of other stormwater infrastructure, a fact that may offset the greater placement cost somewhat.

A drawback is the cost and complexity of porous concrete systems compared to conventional pavements. Porous concrete systems require a very high level of construction workmanship to ensure that they function as designed. They experience a high failure rate if they are not designed, constructed, and maintained properly.

Like other infiltration controls, porous concrete should not be used in areas that experience high rates of wind erosion, where highly erosive soils are present, or in drinking water aquifer recharge areas. Also it cannot be used in traffic areas where sanding is used during winter weather.

25.2 Pollutant Removal Capabilities

As they provide for the infiltration of stormwater runoff, porous concrete systems have a high removal of both soluble and particulate pollutants. These pollutants become trapped, absorbed, or broken down in the underlying soil layers. Due to the potential for clogging, porous concrete surfaces should not be used for the removal of sediment or other coarse particulate pollutants.

The following design pollutant removal rates are conservative average pollutant reduction percentages for design purposes derived from sampling data, modeling and professional judgment.

- Total Suspended Solids – not applicable
- Total Phosphorus – 50%
- Total Nitrogen – 65%
- Fecal Coliform – insufficient data
- Heavy Metals – 60%

Pollutant removal can be improved through routine vacuum sweeping and high pressure washing, insuring a drainage time of at least 24 hours, pretreating the runoff, having organic material in the subsoil, and using clean washed aggregate (EPA, 1999).

25.3 Design Criteria and Specifications

Porous concrete systems can be used where the underlying in-situ subsoils have an infiltration rate greater than 0.5 inches per hour. Therefore, porous concrete systems are not suitable on sites with hydrologic group D or most group C soils, or soils with a high (>30%) clay content. During construction and preparation of the subgrade, special care must be taken to avoid compaction of the underlying soils.

Porous concrete systems should typically be used in applications where the pavement receives tributary runoff only from impervious areas. Actual pervious surface area sizing will depend on achieving a 24 hour minimum and 48 hour maximum draw down time for the design storm volume.

If runoff is coming from adjacent pervious areas, it is important that those areas be fully stabilized to reduce sediment loads and prevent clogging of the porous paver surface. Pretreatment using filter strips or vegetated swales for removal of course sediments is recommended. (see *Sections 4.0 and 13.0*)

Porous concrete systems should not be used on slopes greater than 5% with slopes of no greater than 2% recommended. For slopes greater than 1% barriers perpendicular to the direction of drainage should be installed in sub-grade material to keep it from washing away, or filter fabric should be placed at the bottom and sides of the aggregate to keep soil from migrating into the aggregate and reducing porosity.

A minimum of four feet of clearance is recommended between the bottom of the gravel base course and underlying bedrock or the seasonally high groundwater table.

Porous concrete systems should be sited at least 10 feet down-gradient from buildings and 100 feet away from drinking water wells.

To protect groundwater from potential contamination, runoff from designated hotspot land uses or activities must not be infiltrated. Porous concrete should not be used for manufacturing and industrial sites, where

there is a potential for high concentrations of soluble pollutants and heavy metals. In addition, porous concrete should not be considered for areas with a high pesticide concentration. Porous concrete is also not suitable in areas with karst geology without adequate geotechnical testing by qualified individuals and in accordance with local requirements.

Porous concrete system designs must use some method to convey larger storm event flows to the conveyance system. One option is to use storm drain inlets set slightly above the elevation of the pavement. This would allow for some ponding above the surface, but would accept bypass flows that are too large to be infiltrated by the porous concrete system, or if the surface clogs.

For the purpose of sizing downstream conveyance and structural control system, porous concrete surface areas can be assumed to 35% impervious. In addition, reduction in water quality volume requirements can be obtained for the runoff volume infiltrated from other impervious areas using the methodology in *Section 1.0 of the Planning Technical Manual.*

For treatment control, the design volume should be, at a minimum, equal to the water quality volume. The water quality storage volume is contained in the surface layer, the aggregate reservoir, and the sub-grade above the seasonal high water table – if the sub-grade is sandy. The storm duration (fill time) is normally short compared to the infiltration rate of the sub-grade, a duration of two hours can be used for design purposes. The total storage volume in a layer is equal to the percent of voids times the volume of the layer. Alternately storage may be created on the surface through temporary ponding, though this would tend to accelerate clogging if course sediment or mud settles out on the surface.

The cross-section typically consists of four layers, as shown in Figure 25.1. The aggregate reservoir can sometimes be avoided or minimized if the sub-grade is sandy and there is adequate time to infiltrate the necessary runoff volume into the sandy soil without by-passing the water quality volume. Descriptions of each of the layers is presented below:

- *Porous Concrete Layer* – The porous concrete layer consists of an open-graded concrete mixture usually ranging from depths of 2 to 4 inches depending on required bearing strength and pavement design requirements. Porous concrete can be assumed to contain 18 percent voids (porosity = 0.18) for design purposes. Thus, for example, a 4 inch thick porous concrete layer would hold 0.72 inches of rainfall. The omission of the fine aggregate provides the porosity of the porous pavement. To provide a smooth riding surface and to enhance handling and placement a coarse aggregate of 3/8 inch maximum size is normally used. Use coarse aggregate (3/8 to No. 16) per ASTM C 33 or No. 89 coarse aggregate (3/8 to No. 50) per ASTM D 448.
- *Top Filter Layer* – Consists of a 0.5 inch diameter crushed stone to a depth of 1 to 2 inches. This layer serves to stabilize the porous concrete layer. It can be combined with reservoir layer using suitable stone.
- *Reservoir Layer* – The reservoir gravel base course consists of washed, bank-run gravel, 1.5 to 2.5 inches in diameter with a void space of about 40% meeting the gradation listed below. The depth of this layer depends on the desired storage volume, which is a function of the soil infiltration rate and void spaces, but typically ranges from two to four feet. The layer must have a minimum depth of nine inches. The layer should be designed to drain completely in 48 hours. and stored at a minimum the water quality volume (WQ_v). Aggregate contaminated with soil shall not be used. A porosity value (void space/total volume) of 0.32 should be used in calculations unless aggregate specific data exist.

Gradation

Sieve Size	% Passing
2 ½”	100
2”	90 – 100
1 ½”	35 – 70
1”	0 – 15
½”	0 - 5

- *Bottom Filter Layer* – The surface of the subgrade should be an 6 inch layer of sand (ASTM C-33 concrete sand or TxDOT Fine Aggregate Grade No. 1) or a 2 inch thick layer of 0.5 inch crushed stone, and be completely flat to promote infiltration across the entire surface. This layer serves to stabilize the

reservoir layer, to protect the underlying soil from compaction, and act as the interface between the reservoir layer and the filter fabric covering the underlying soil.

- *Filter Fabric* – It is very important to line the entire trench area, including the sides, with filter fabric prior to placement of the aggregate. The filter fabric serves a very important function by inhibiting soil from migrating into the reservoir layer and reducing storage capacity. Fabric should be MIRIFI # 14 N or equivalent.
- *Underlying Soil* – The underlying soil should have an infiltration capacity of at least 0.5 in/hr, but preferably greater than 0.50 in/hr. as initially determined from NRCS soil textural classification, and subsequently confirmed by field geotechnical tests. The minimum geotechnical testing is one test hole per 5000 square feet, with a minimum of two borings per facility (taken within the proposed limits of the facility). Infiltration trenches cannot be used in fill soils. Soils at the lower end of this range may not be suited for a full infiltration system. Test borings are recommended to determine the soil classification, seasonal high ground water table elevation, and impervious substrata, and an initial estimate of permeability. Often a double-ring infiltrometer test is done at subgrade elevation to determine the infiltration rate of the least permeable layer, and, for safety, one-half that measured value is taken for infiltration calculations.

The pit excavation should be limited to the width and depth specified in the design. Excavated material should be placed away from the open trench as not to jeopardize the stability of the trench sidewalls. The bottom of the excavated trench should not be loaded so as to cause compaction, and should be scarified prior to placement of sand. The sides of the trench shall be trimmed of all large roots. The sidewalls shall be uniform with no voids and scarified prior to backfilling. All infiltration trench facilities should be protected during site construction, and should be constructed after upstream areas have been stabilized.

An observation well consisting of perforated PVC pipe 4 to 6 inches in diameter should be placed at the downstream end of the facility and protected. The well should be used to determine actual infiltration rates.

A warning sign should be placed at the facility that states, “Porous Paving used on this site to reduce pollution. Do not resurface with non-porous material or sand during icy weather. Call the local jurisdiction for more information.”

Details of construction of the concrete layer are beyond the scope of this manual. However, construction of porous concrete is exacting, and requires special handling, timing, and placement to perform adequately (LACDPW, 2000, Paine, 1992, Maryland, 1984).

25.4 Inspection and Maintenance Requirements

Table 25.1 Typical Maintenance Activities for Porous Concrete Systems

Activity	Schedule
• Initial inspection	Monthly for three months after installation
• Ensure that the porous paver surface is free of sediment	Monthly
• Ensure that the contributing and adjacent area is stabilized and mowed, with clippings removed	As needed, based on inspection
• Vacuum sweep porous concrete surface followed by high pressure hosing to keep pores free of sediment	Four times a year
• Inspect the surface for deterioration or spalling • Check to make sure that the system dewaters between storms	Annually

Table 25.1 Typical Maintenance Activities for Porous Concrete Systems	
Activity	**Schedule**
• Spot clogging can be handled by drilling half-inch holes through the pavement every few feet • Rehabilitation of the porous concrete system, including the top and base course as needed	Upon failure

To ensure proper maintenance of porous pavement, a carefully worded maintenance agreement is essential. It should include specific the specific requirements and establish the responsibilities of the property owner and provide for enforcement.

25.5 Example Schematics

Figure 25.1 Porous Concrete System Section
(Modified From: LAC 2000)

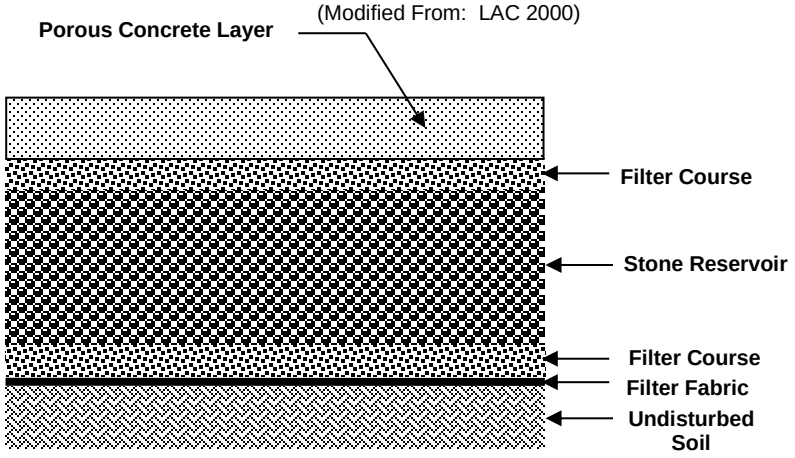




Figure 25.2 Porous Concrete System Installation



Figure 25.3 Typical Porous Concrete System Applications
(Photos by Bruce Ferguson, Don Wade)

25.6 Design Example

Data

A 1.5 acre overflow parking area is to be designed to provide water quality treatment using porous concrete to handle the runoff from the whole overflow parking area. Initial data shows:

- Rainfall depth for treatment is up to 1.5 inches
- Borings show depth to water table is 5.0 feet
- Boring and infiltrometer tests show sand-loam with percolation rate (k) of 1.02 inches/hr
- Structural design indicates the thickness of the porous concrete must be at least three inches

Water Quality Volume

R_v = 0.05 + 0.009 I (where I = 100 percent)
= 0.95

WQ_v = 1.5 R_v A / 12 = 1.5 * 0.95 * 1.5/12 = 0.178 acre-feet
= (0.178 ac-ft) (43,560 cu-ft/ac-ft) = 7,759 cubic feet

Surface Area

A porosity value n = 0.32 should be used for the gravel and 0.18 for the concrete layer.

All infiltration systems should be designed to fully de-water the entire WQ_v within 24 to 48 hours after the rainfall event at the design percolation rate.

A fill time T=2 hours can be used for most designs.

Chose a depth of gravel pit of three feet (including layer under concrete) which fits the site with a two foot minimum to water table (other lesser depths could be chosen, making the surface area larger). The minimum surface area of the trench can be determined, in a manner similar to the infiltration trench, from the Equation 20.1:

$A = WQ_v/(n_g d_g + kT/12 + n_p d_p)$

= 7,759/(0.32*3 + 1.02*2/12 + 0.18 * 3/12)

= 6,604 square feet

Where:

A = Surface Area
WQv = Water Quality Volume (or total volume to be infiltrated)
n = porosity (g of the gravel, p of the concrete layer)
d = depth or gravel layer (feet) (g of the gravel, p of the concrete layer)
k = percolation (inches/hour)
T = Fill Time (time for the practice to fill with water), in hours

Check of drain time:

depth = 3*12 + 3 inches to sand layer = 39 inches @ 1.02 in/hr = 38 hours (ok)

Overflow will be carried across the porous concrete and tied into the drainage system for the rest of the site.

26.0 Proprietary Structural Controls

Limited Application
Structural Stormwater Control

Description: Manufactured structural control systems available from commercial vendors designed to treat stormwater runoff and/or provide water quantity control

KEY CONSIDERATIONS

DESIGN CRITERIA:

- Independent performance data must be available to prove a demonstrated capability of meeting stormwater management goal(s)
- System or device must be appropriate for use in North Central Texas conditions, and specifically for the community in question
- Pre-treat runoff if sediment present

ADVANTAGES / BENEFITS:

- Provides reduction in runoff volume

DISADVANTAGES / LIMITATIONS:

- Depending on the proprietary system, there may be:
  - Limited performance data
  - Application constraints
  - High maintenance requirements
  - Higher costs than other structural control alternatives
- Installation and operations/maintenance requirements must be understood by all parties approving and using the system or device in question

STORMWATER MANAGEMENT SUITABILITY

S	**Water Quality Protection**
S	**Streambank Protection**
S	**On-Site Flood Control**
S	**Downstream Flood Control**

IMPLEMENTATION CONSIDERATIONS

L	**Land Requirement**
H	**Capital Cost**
H	**Maintenance Burden**

Residential
Subdivision Use: Depends on the specific proprietary structural control

High Density/Ultra-Urban: Yes

Drainage Area: Depends on the specific proprietary structural control.

Soils: No restrictions

L=Low **M**=Moderate **H**=High

Note: It is the policy of this Manual not to recommend any specific commercial vendors for proprietary systems. However, this section is being included in order to provide communities with a rationale for approving the use of a proprietary system or practice in their jurisdictions.

26.1 General Description

There are many types of commercially-available proprietary stormwater structural controls available for both water quality treatment and quantity control. These systems include:

- Hydrodynamic systems such as gravity and vortex separators
- Filtration systems
- Catch basin media inserts
- Chemical treatment systems
- Package treatment plants
- Prefabricated detention structures

Many proprietary systems are useful on small sites and space-limited areas where there is not enough land or room for other structural control alternatives. Proprietary systems can often be used in pretreatment applications in a treatment train. However, proprietary systems are often more costly than other alternatives and may have high maintenance requirements. Perhaps the largest difficulty in using a proprietary system is the lack of adequate independent performance data, particularly for use in North Central Texas conditions. Below are general guidelines that should be followed before considering the use of a proprietary commercial system.

26.2 Guidelines for Using Proprietary Systems

In order for use as a limited application control, a proprietary system must have a demonstrated capability of meeting the stormwater management goals for which it is being intended. This means that the system must provide:

1. Independent third-party scientific verification of the ability of the proprietary system to meet water quality treatment objectives and/or to provide water quantity control (streambank or flood control)
2. Proven record of longevity in the field
3. Proven ability to function in North Central Texas conditions (e.g., climate, rainfall patterns, soil types, etc.)
4. Maintainability - Documented procedures for required maintenance including collection and removal of pollutants or debris.

For a proprietary system to meet (1) above for water quality goals, the following monitoring criteria should be met for supporting studies:

- At least 15 storm events must be sampled
- The study must be independent or independently verified (i.e., may not be conducted by the vendor or designer without third-party verification)
- The study must be conducted in the field, as opposed to laboratory testing
- Field monitoring must be conducted using standard protocols which require proportional sampling both upstream and downstream of the device
- Concentrations reported in the study must be flow-weighted
- The propriety system or device must have been in place for at least one year at the time of monitoring

Although local data is preferred, data from other regions can be accepted as long as the design accounts for the local conditions. Local governments may submit a proprietary system to further scrutiny based on the performance of similar practices. A poor performance record or high failure rate is valid justification for not allowing the use of a proprietary system or device. Consult your local review authority for more information in regards to the use of proprietary structural stormwater controls.

27.0 Rain Harvesting (Tanks/Barrels)

Stormwater Control

North Central Texas Council of Governments

integrated Stormwater Management iSWM

Description

Rainwater harvesting (tanks or barrels) are designed to capture stormwater runoff from roofed structures. The system typically consists of a storage container, a downspout diversion, a sealed lid, and an overflow system. Rainwater harvesting may also be called rain pails, rain savers, or cisterns.

Design Considerations

- Typical systems hold between 50 and several thousand gallons of water and may operate in series to provide additional storage volume.
- Highly applicable to residential areas.
- Screens should be considered on gutters and downspouts to remove sediment and other particles.
- First flush diverters should be added to reduce the sediment and debris loading to the rainwater harvesting device.
- Can be used in conjunction with additional BMPs as part of a treatment train system.
- Overflow mechanisms are required to prevent water from backing up in the system.

Key Advantages

- Rainwater harvesting provides a reduction in runoff volume.
- Systems are low-cost, effective, and easy to maintain.
- Since various sizes exist, there is flexibility in the desired capture volume.
- Stormwater can be saved and reused, reducing the need for irrigation at a site. This can lead to healthier plants due to the lack of chlorination in the water.
- Rainwater harvesting systems can conserve well or municipal water during the dry season.
- Systems are available in a multitude of sizes, shapes, and materials to suit the needs of the user.

Limitations

- Rainwater harvesting systems may have a small storage capacity.
- Some maintenance needs to be provided to the systems.
- Attention should be given to the systems after a rainfall event to prevent leaking from the system and damage to adjacent building foundations.
- Compared to the municipal water supply, the construction cost is high.
- If reuse is a possibility, roof materials need to be considered, since there is the potential for runoff contamination.



Rainwater Harvesting in Los Angeles, CA. *(Source: Tetra Tech)*

Target Constituent	Removal Rate (0% → 100%)
Total Suspended Solids	not applicable
Total Phosphorus	not applicable
Total Nitrogen	not applicable
Fecal Coliform	not applicable
Heavy Metals	not applicable

Implementation Considerations



Suitability

The iSWM manual has designated rainwater harvesting systems as suitable for providing:

Water Quality Protection

Maintenance

- Check gutters and rooftops for debris that can get into the rainwater harvesting system.
- Inspect the area around the system to ensure it is not leaking and causing damage to the adjacent foundation.
- Clear the screens, first flush diverters, and system of sediment and other debris.
- Clear pipes when clogging occurs.
- Provide algae, mosquito, and other vector control when required.

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27.1 General Description

Rain harvesting (also referred to as *rain pails* and *rain savers*) are used as a water conservation practice and a stormwater management strategy. Capturing water in a rain tank/barrel prevents runoff from flowing down a driveway or across a parking lot and picking up soil, pesticides, and other pollutants before entering the storm sewer system.

Rain tanks/barrels not only store water, but also help to decrease the water supply demand during the sweltering summer months. Only 1/4 inch of rainfall runoff from the average roof will completely fill the typical barrel. Collection of water from rooftop runoff can provide an ample supply of free 'soft water' containing no chlorine, lime, or calcium. Because it tends to have fewer sediments and dissolved salts than municipal water, rain water is ideal for a multitude of applications, including organic vegetable gardens, planter beds for botanicals, indoor plants, automobile washing, and cleaning household windows. Saving water in this manner will reduce the demand for treated tap water, and save money by lowering the homeowner's monthly bill. Rain water diversion will also help decrease the burden on water treatment facilities and municipal drainage systems during storm events.

A typical rain harvesting design will include a storage container with a hole at the top to allow for flow from a downspout, a sealed lid, an overflow pipe and a spigot at or near the bottom of the barrel. The spigot can be left partially open to detain water or closed to fill the barrel. A screen is often included to control mosquitoes and other insects. Rain tanks/barrels can be connected in series to provide larger volumes of storage. Larger systems for commercial or industrial use can include pumps and filtration systems.

For every inch of rain that falls on a catchment area of 1,000 square feet, approximately 600 gallons of rainwater can be collected. Ten inches of rain falling on a 1,000 square foot catchment area will generate approximately 6,000 gallons of rainwater.

27.2 Design Criteria and Specifications

- The required capacity of a rain barrel is a function of the rooftop surface area that drains to it, the inches of rainfall required to fill the barrel, and water losses due primarily to evaporation. The general rule of thumb to utilize in the sizing of rain barrels is 1 inch of rainfall on a 1000 square foot roof will yield approximately 600 gallons.

Sample Calculation
Rain barrel volume can be determined by calculating the roof top water yield for any given rainfall, using the following general equation:
Equation 27.1 $V = A^2 \times R \times 0.90 \times 7.5\ gals./\ ft.^3$ where:
V = volume of rain barrel (gallons) A^2 = surface area roof (square feet) R = rainfall (feet) 0.90 = losses to system (no units) 7.5 = conversion factor (gallons per cubic foot)
Example: one 60-gallon barrel would provide runoff storage from a rooftop area of approximately 212 square feet for a 1.5 inch (0.125 ft.) of rainfall. $V = 212\ ft.^2 \times 0.125\ ft. \times 0.90 \times 7.5\ gallons/ft.^3 = 179\ gallons$

- Homeowners and manufacturers typically use a flexible plastic downspout elbow to direct water from the downspout into the rain tank/barrel. It is best to use the downspout connector only for smaller drainage areas or to use a diverter that can be engaged either automatically or with physical contact.
- The use of screens should be considered on gutters and downspouts to remove sediment and particles as the water enters the barrel.
- Containers should be opaque to discourage bacteria/algae growth.
- Most rain barrels are equipped with tight covers and screens to prevent accidents involving children and pets and to keep debris out of the barrels. To combat concerns regarding mosquitoes and West Nile virus, tight fitting screens should be inspected and maintained on a routine basis or nontoxic mosquito control agent (Bti) placed in water.
- A half-barrel design will allow the barrel to sit flush against a building and may prove to be more aesthetically pleasing.
- A typical rain barrel will include spigot at the top to accommodate overflow (this should be directed away from the foundation of the building) and a gravity-fed hose bib at the bottom to connect a hose for redistribution of the rainwater.
- Inexpensive rain barrels can be made from food grade plastic barrels or heavy-duty trash cans, for as little as $15 or they can be purchased pre-made from numerous non-profit organizations, commercial manufacturers, and retailers, in prices ranging from $25 to $150.

27.3 Example Schematics

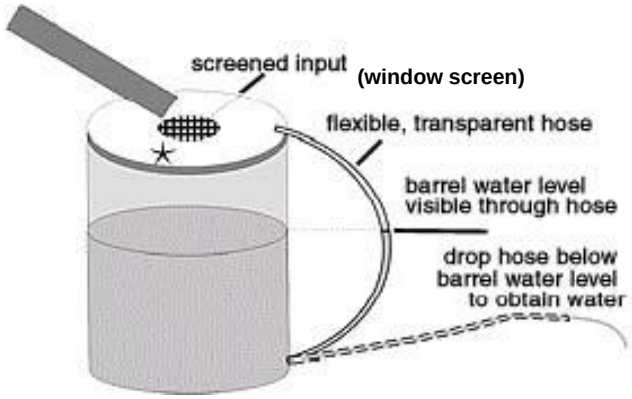


Figure 27.1 Simple Rain Barrel Design (from Maryland DNR Green Building Program)



Figure 27.2 Rain barrels in series

28.0 Stormwater Wetlands

Stormwater Control

North Central Texas Council of Governments

Integrated Stormwater Management iSWM

Description

Stormwater wetland systems are constructed to hold a permanent body of water for stormwater management. Stormwater runoff volume is stored and treated within the facility. Treatment is provided by settling and uptake by marsh vegetation. These wetlands are also called constructed wetlands.

Design Considerations

- Wetlands are designed to treat both stormwater quantity and quality.
- Variations of wetlands include shallow, extended detention (ED) shallow, pocket/wetland systems, pocket, and submerged gravel wetlands.
- Soil groups 'A' and 'B' may require a liner to keep a permanent pool elevation.
- Wetlands require a continuous base flow or high water table to support vegetation.
- Drainage area requirements are a minimum of 25 acres. Pocket wetlands have a minimum drainage area of 5 acres.
- The depth of the ponded water should vary, but 35% of the area or more should be 6" or less in depth. A deep pool (1.5 to 6 feet in depth) should encompass 10–20% of the surface area.
- Pretreatment is required to reduce the sediment load to stormwater wetlands.

Key Advantages

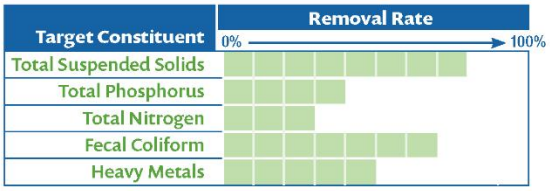
- Stormwater wetlands have high levels of nutrient removal and are considered to be one of the most effective stormwater practices.
- Wetlands provide great wildlife habitat, aesthetic value, and other amenities.
- Maintenance costs are relatively low.

Limitations

- Large land areas are required to fully implement a stormwater wetland.
- Viable wetlands require continuous base flow.
- In order to keep the wetlands viable, sediment control into the facilities is required.
- Vegetation is crucial to the success of the wetlands and must be fully established within the first three years.



Stormwater Wetland in Nashville, NC. *(Source: Tetra Tech)*



Implementation Considerations



**Maintenance Burden is high for a pocket wetland*

Suitability

The iSWM manual has designated stormwater wetlands as suitable for providing:



***Does not apply to submerged gravel wetland systems*

Maintenance

- Wetland vegetation must be maintained and replacements should be made to maintain at least 50% surface area coverage.
- Invasive vegetation needs to be removed.
- Sediment accumulation and debris must be removed periodically (After 50% of the forebay capacity is lost).
- Eroded areas require repair.
- Algae, mosquito, and other vectors need to be controlled as issues arise.
- Rodent and other animal damage require repair.

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28.1 General Description

Stormwater wetlands (also referred to as constructed wetlands) are constructed shallow marsh systems that are designed to both treat urban stormwater and control runoff volumes. As stormwater runoff flows through the wetland facility, pollutant removal is achieved through settling and uptake by marsh vegetation.

Wetlands are among the most effective stormwater practices in terms of pollutant removal and also offer aesthetic value and wildlife habitat. Constructed stormwater wetlands differ from natural wetland systems in that they are engineered facilities designed specifically for the purpose of treating stormwater runoff and typically have less biodiversity than natural wetlands both in terms of plant and animal life. However, as with natural wetlands, stormwater wetlands require a continuous base flow or a high water table to support aquatic vegetation.

There are several design variations of the stormwater wetland, each design differing in the relative amounts of shallow and deep water, and dry storage above the wetland. These include the shallow wetland, the extended detention shallow wetland, pond/wetland system, and pocket wetland. Figure 28.1 contains photos of various wetlands. Below are descriptions of each design variant:

- **Shallow Wetland** – In the shallow wetland design, most of the water quality treatment volume is in the relatively shallow high marsh or low marsh depths. The only deep portions of the shallow wetland design are the forebay at the inlet to the wetland, and the micropool at the outlet. One disadvantage of this design is that, since the pool is very shallow, a relatively large amount of land is typically needed to store the water quality volume.
- **Extended Detention (ED) Shallow Wetland** – The extended detention (ED) shallow wetland design is the same as the shallow wetland; however, part of the water quality treatment volume is provided as extended detention above the surface of the marsh and released over a period of 24 hours. This design can treat a greater volume of stormwater in a smaller space than the shallow wetland design. In the extended detention wetland option, plants that can tolerate both wet and dry periods need to be specified in the extended detention zone.
- **Pond/Wetland Systems** – The pond/wetland system has two separate cells: a wet pond and a shallow marsh. The wet pond traps sediments and reduces runoff velocities prior to entry into the wetland where stormwater flows receive additional treatment. Less land is required for a pond/wetland system than for the shallow wetland or the extended detention shallow wetland systems.
- **Pocket Wetland** – A pocket wetland is intended for smaller drainage areas of 5 to 10 acres and typically requires excavation down to the water table for a reliable water source to support the wetland system.
- **Submerged Gravel -** Also known as subsurface flow wetlands, this wetland consists of one or more cells filled with crushed rock designed to support wetland plants. Stormwater flows subsurface through the root zone of the constructed wetland where pollutant removal takes place. *This type of wetland is not recommended for use to meet stormwater management goals due to limited performance data.* They may be applicable in special or retrofit situations where there are severe limitations on what can be implemented.



Figure 28.1 Stormwater Wetland Examples

28.2 Stormwater Management Suitability

Similar to stormwater ponds, stormwater wetlands are designed to control both stormwater quantity and quality. Thus, a stormwater wetland can be used to address all of the integrated stormwater sizing criteria for a given drainage area.

Water Quality

Pollutants are removed from stormwater runoff in a wetland through uptake by wetland vegetation and algae, vegetative filtering, and through gravitational settling in the slow moving marsh flow. Other pollutant removal mechanisms are also at work in a stormwater wetland including chemical and biological decomposition and volatilization. *Section 28.3* provides pollutant removal efficiencies that can be used for planning and design purposes.

Streambank Protection

The storage volume above the permanent pool/water surface level in a stormwater wetland is used to provide control of the streambank protection volume (SP_v). This is accomplished by releasing the 1-year, 24-hour storm runoff volume over 24 hours (extended detention). It is best to do this with minimum vertical water level fluctuation, as extreme fluctuation may stress vegetation.

Flood Control

In situations where it is required, stormwater wetlands can also be used to provide detention to control the flood mitigation storm peak flow. Where flood mitigation storm peak control is not required, a stormwater wetland must be designed to safely pass the flood mitigation storm flows.

28.3 Pollutant Removal Capabilities

All of the stormwater wetland design variants are presumed to be able to remove 80% of the total suspended solids load in typical urban post-development runoff when sized, designed, constructed and maintained in accordance with the recommended specifications. Undersized or poorly designed wetland facilities can reduce TSS removal performance.

The following design pollutant removal rates are conservative average pollutant reduction percentages for design purposes derived from sampling data, modeling and professional judgment. In a situation where a removal rate is not deemed sufficient, additional controls may be put in place at the given site in a series or "treatment train" approach.

- Total Suspended Solids – 80%
- Total Phosphorus – 40%
- Total Nitrogen – 30%
- Fecal Coliform – 70% (if no resident waterfowl population present)
- Heavy Metals – 50%

For additional information and data on pollutant removal capabilities for stormwater wetlands, see the National Pollutant Removal Performance Database (2nd Edition) available at www.cwp.org and the National Stormwater Best Management Practices (BMP) Database at www.bmpdatabase.org

Submerged Gravel Wetland

The pollution removal efficiency of the submerged gravel wetland is similar to a typical wetland. Recent data show a TSS removal rate in excess of the 80% goal. This reflects the settling environment of the gravel media. These systems also exhibit removals of about 60% TP, 20% TN, and 50% Zn. The growth of algae and microbes among the gravel media has been determined to be the primary removal mechanism of the submerged gravel wetland.

The following design pollutant removal rates are conservative average pollutant reduction percentages for design purposes derived from sampling data, modeling and professional judgment.

- Total Suspended Solids – 80%
- Total Phosphorus – 50%
- Total Nitrogen – 20%
- Fecal Coliform – 70%
- Heavy Metals – 50%

Although gravel wetlands are fairly effective at removing total phosphorus, they have a tendency to contribute small amounts of soluble phosphorus.

28.4 Application and Site Feasibility Criteria

Stormwater wetlands are generally applicable to most types of new development and redevelopment, and can be utilized in both residential and nonresidential areas. However, due to the large land requirements, wetlands may not be practical in higher density areas. The following criteria should be evaluated to ensure the suitability of a stormwater wetland for meeting stormwater management objectives on a site or development.

General Feasibility

- Suitable for Residential Subdivision Usage – YES
- Suitable for High Density/Ultra Urban Areas – Land requirements may preclude use
- Regional Stormwater Control – YES
- Hot Spot Runoff - YES

Physical Feasibility - Physical Constraints at Project Site

- Drainage Area – A minimum of 25 acres and a positive water balance is needed to maintain wetland conditions; 5 acres for pocket wetland
- Space Required – Approximately 3 to 5% of the tributary drainage area
- Site Slope – There should be no more than 8% slope across the wetland site
- Minimum Head – Elevation difference needed at a site from the inflow to the outflow: 3 to 5 feet; 2 to 3 feet for pocket wetland
- Minimum Depth to Water Table – If used on a site with an underlying water supply aquifer or when treating a hotspot, a separation distance of 2 feet is recommended between the bottom of the wetland and the elevation of the seasonally high water table; pocket wetland is typically below water table.
- Soils – Permeable soils are not well suited for a constructed stormwater wetland without a high water table. Underlying soils of hydrologic group “C” or “D” should be adequate to maintain wetland conditions. Most group “A” soils and some group “B” soils will require a liner. *Evaluation of soils should be based upon an actual subsurface analysis and permeability tests*.

28.5 Planning and Design Criteria

*The following criteria are to be considered **minimum** standards for the design of a stormwater wetland facility. Consult with the local review authority to determine if there are any variations to these criteria or additional standards that must be followed.*

A Location and Siting

Stormwater wetlands should normally have a minimum contributing drainage area of 25 acres or more. For a pocket wetland, the minimum drainage area is 5 acres.

A continuous base flow or high water table is required to support wetland vegetation. A water balance must be performed to demonstrate that a stormwater wetland can withstand a 30-day drought at summer

evaporation rates without completely drawing down (see *Section 4.0 of the Hydrology Technical Manual* for details).

Wetland siting should also take into account the location and use of other site features such as natural depressions, buffers, and undisturbed natural areas, and should attempt to aesthetically "fit" the facility into the landscape. Bedrock close to the surface may prevent excavation.

Stormwater wetlands cannot be located within navigable waters of the U.S., including natural wetlands, without obtaining a Section 404 permit under the Clean Water Act, and any other applicable State permit. In some isolated cases, a wetlands permit may be granted to convert an existing degraded wetland in the context of local watershed restoration efforts.

If a wetland facility is not used for flood control less than the 100 year event, it should be designed as an off-line system to bypass higher flows rather than passing them through the wetland system.

Minimum setback requirements for stormwater wetland facilities (when not specified by local ordinance or criteria):

- From a property line – 10 feet
- From a private well – 100 feet; if well is downgradient from a hotspot land use then the minimum setback is 250 feet
- From a septic system tank/leach field/spray area – 50 feet

All utilities should be located outside of the wetland site.

B General Design

A well-designed stormwater wetland consists of:

1. Shallow marsh areas of varying depths with wetland vegetation,
2. Permanent micropool, and
3. Overlying zone in which runoff control volumes are stored.

Pond/wetland systems also include a stormwater pond facility (see *Section 22.0*, for pond design information).

In addition, all wetland designs must include a sediment forebay at the inflow to the facility to allow heavier sediments to drop out of suspension before the runoff enters the wetland marsh.

Additional wetland design features include an emergency spillway, maintenance access, safety bench, wetland buffer, and appropriate wetland vegetation and native landscaping.

Figures 28.3 through 28.6 in *Section 28.8* provide plan view and profile schematics for the design of a shallow wetland, extended detention shallow wetland, pond/wetland system, and pocket wetland, respectively.

C Physical Specifications / Geometry

In general, wetland designs are unique for each site and application. However, there are a number of geometric ratios and limiting depths for the design of a stormwater wetland that must be observed for adequate pollutant removal, ease of maintenance, and improved safety. Table 28.1 provides the recommended physical specifications and geometry for the various stormwater wetland design variants.

Table 28.1 Recommended Design Criteria for Stormwater Wetlands
Modified from Massachusetts DEP, 1997; Schueler, 1992

Design Criteria	Shallow Wetland	ED Shallow Wetland	Pond/ Wetland	Pocket Wetland
Length to Width Ratio (minimum)	2:1	2:1	2:1	2:1
Extended Detention (ED)	No	Yes	Optional	Optional
Allocation of WQ_v Volume (pool/marsh/ED) in %	25/75/0	25/25/50	70/30/0 (includes pond volume)	25/75/0
Allocation of Surface Area (deepwater/low marsh/high marsh/semi-wet) in %	20/35/40/5	10/35/45/10	45/25/25/5 (includes pond surface area)	10/45/40/5
Forebay	Required	Required	Required	Required
Micropool	Required	Required	Required	Required
Outlet Configuration	Reverse-slope pipe or hooded broad-crested weir	Reverse-slope pipe or hooded broad-crested weir	Reverse-slope pipe or hooded broad-crested weir	Hooded broad-crested weir

The stormwater wetland should be designed with the recommended proportion of "depth zones." Each of the four wetland design variants has depth zone allocations which are given as a percentage of the stormwater wetland surface area. Target allocations are found in Table 28.1. The four basic depth zones are:

- **Deepwater zone**
  From 1.5 to 6 feet deep. Includes the outlet micropool and deepwater channels through the wetland facility. This zone supports little emergent wetland vegetation, but may support submerged or floating vegetation.
- **Low marsh zone**
  From 6 to 18 inches below the normal permanent pool or water surface elevation. This zone is suitable for the growth of several emergent wetland plant species.
- **High marsh zone**
  From 6 inches below the pool to the normal pool elevation. This zone will support a greater density and diversity of wetland species than the low marsh zone. The high marsh zone should have a higher surface area to volume ratio than the low marsh zone.
- **Semi-wet zone**
  Those areas above the permanent pool that are inundated during larger storm events. This zone supports a number of species that can survive flooding.

A minimum dry weather flow path of 2:1 (length to width) is required from inflow to outlet across the stormwater wetland and should ideally be greater than 3:1. This path may be achieved by constructing internal dikes or berms, using marsh plantings, and by using multiple cells. Finger dikes are commonly used in surface flow systems to create serpentine configurations and prevent short-circuiting. Microtopography (contours along the bottom of a wetland or marsh that provide a variety of conditions for different species needs and increases the surface area to volume ratio) is encouraged to enhance wetland diversity.

A 4- to 6-foot deep micropool must be included in the design at the outlet to prevent the outlet from clogging and resuspension of sediments, and to mitigate thermal effects.

Maximum depth of any permanent pool areas should generally not exceed 6 feet.

The volume of the extended detention must not comprise more than 50% of the total WQ_v, and its maximum water surface elevation must not extend more than 3 feet above the normal pool. Q_p and/or SP_v storage can be provided above the maximum WQ_v elevation within the wetland.

The perimeter of all deep pool areas (4 feet or greater in depth) should be surrounded by safety and aquatic benches similar to those for stormwater ponds (see *Section 22.0*).

The contours of the wetland should be irregular to provide a more natural landscaping effect.

D Submerged Gravel Wetlands

Submerged gravel wetlands should be designed as off-line systems designed to handle only water quality volume.

Submerged gravel wetland systems need sufficient drainage area to maintain vegetation. See *Section 4.0 of the Hydrology Technical Manual* for guidance on performing a water balance calculation.

The local slope should be relatively flat (<2%). While there is no minimum slope requirement, there does need to be enough elevation drop from the inlet to the outlet to ensure that hydraulic conveyance by gravity is feasible (generally about 3 to 5 feet).

A design maximum depth of 16 inches of water at the inlet is recommended, with a total gravel depth of 20 inches.

Gravel should be 0.5-1.0 inch in size.

Darcy's Law may be used to estimate flows in the gravel media, although the use of predesign tests with the actual gravel will refine the "effective" hydraulic conductivity.

The initial design should not utilize more than 70 percent of the potential hydraulic gradient available in the proposed bed to allow a safety factor for clogging.

Using a value of < 113 $m^3/m^2/d$ for the "effective" hydraulic conductivity (k_s) in the design will also help account for potential clogging.

An adjustable outlet is recommended to ensure adequate hydraulic gradient and prevent surface flow from occurring and shortcircuiting treatment within the gravel media.

Washed stone or gravel, should be specified to protect against an accumulation of fine material that could cause hydraulic blockages.

All submerged gravel wetland designs should include a sediment forebay or other equivalent pretreatment measures to prevent sediment or debris from entering and clogging the gravel bed.

Unless they receive hotspot runoff, submerged gravel wetland systems can be allowed to intersect the groundwater table.

Guidance on establishing wetland vegetation can be found in the *Landscape Technical Manual*.

E Pretreatment / Inlets

Sediment regulation is critical to sustain stormwater wetlands. A wetland facility should have a sediment forebay or equivalent upstream pretreatment. A sediment forebay is designed to remove incoming sediment from the stormwater flow prior to dispersal into the wetland. The forebay should consist of a separate cell, formed by an acceptable barrier. A forebay is to be provided at each inlet, unless the inlet provides less than 10% of the total design storm inflow to the wetland facility.

The forebay is sized to contain 0.1 inches per impervious acre of contributing drainage and should be 4 to 6 feet deep. The pretreatment storage volume is part of the total WQ_v requirement and may be subtracted from WQ_v for wetland storage sizing.

A fixed vertical sediment depth marker shall be installed in the forebay to measure sediment deposition over time. The bottom of the forebay may be hardened (e.g., using concrete, paver blocks, etc.) to make sediment removal easier.

Inflow channels are to be stabilized with flared riprap aprons, or the equivalent. Inlet pipes to the pond can be partially submerged. Inflow pipe, channel velocities and exit velocities from the forebay must be nonerosive.

F Outlet Structures

- Flow control from a stormwater wetland is typically accomplished with the use of a concrete or corrugated aluminum, aluminized steel, or HDPE riser and barrel. The riser is a vertical pipe or inlet structure that is attached to the base of the micropool with a watertight connection. The outlet barrel is a horizontal pipe attached to the riser that conveys flow under the embankment (see Figure 28.2). The riser should be located within the embankment for maintenance access, safety, and aesthetics.

- A number of outlets at varying depths in the riser provide internal flow control for routing of the water quality protection, streambank protection, and flood control runoff volumes. The number of orifices can vary and is usually a function of the pond design.

  For shallow and pocket wetlands, the riser configuration is typically comprised of a streambank protection outlet (usually an orifice) and flood control outlet (often a slot or weir). The streambank protection orifice is sized to release the streambank protection storage volume over a 24-hour period (12-hour extended detention may be warranted in some cold water streams). Since the water quality volume is fully contained in the permanent pool, no orifice sizing is necessary for this volume. As runoff from a water quality event enters the wet pond, it simply displaces that same volume through the streambank protection orifice. Thus an off-line shallow or pocket wetland providing *only* water quality treatment can use a simple overflow weir as the outlet structure.

  In the case of a extended detention (ED) shallow wetland, there is generally a need for an additional outlet (usually an orifice) that is sized to pass the extended detention water quality volume that is surcharged on top of the permanent pool. Flow will first pass through this orifice, which is sized to release the water quality extended detention volume in 24 hours. The preferred design is a reverse slope pipe attached to the riser, with its inlet submerged 1 foot below the elevation of the permanent pool to prevent floatables from clogging the pipe and to avoid discharging warmer water at the surface of the pond. The next outlet is sized for the release of the streambank protection storage volume. The outlet (often an orifice) invert is located at the maximum elevation associated with the extended detention water quality volume and is sized to release the streambank protection storage volume over a 24-hour period (12-hour extended detention may be warranted in some cold water streams).

  Alternative hydraulic control methods to an orifice can be used and include the use of a broad-crested rectangular, V-notch, proportional weir, or an outlet pipe protected by a hood that extends at least 12 inches below the normal pool.

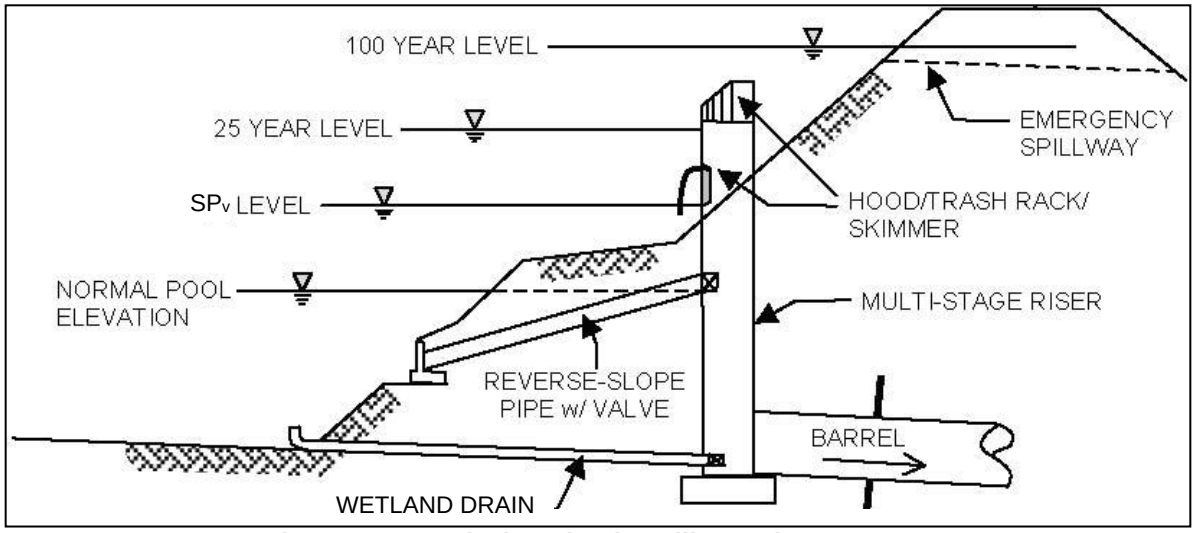


Figure 28.2 Typical Wetland Facility Outlet Structure

- The water quality outlet (if design is for an extended detention shallow wetland) and streambank protection outlet should be fitted with adjustable gate valves or other mechanism that can be used to adjust detention time.
- Higher flows pass through openings or slots protected by trash racks further up on the riser.
- After entering the riser, flow is conveyed through the barrel and is discharged downstream. Anti-seep collars should be installed on the outlet barrel to reduce the potential for pipe failure.
- Riprap, plunge pools or pads, or other energy dissipators are to be placed at the outlet of the barrel to prevent scouring and erosion. If a wetland facility daylights to a channel with dry weather flow, care should be taken to minimize tree clearing along the downstream channel, and to reestablish a forested riparian zone in the shortest possible distance. See *Section 6.0 of the Hydraulic Technical Manual* for more guidance.
- The wetland facility must have a bottom drain pipe located in the micropool with an adjustable valve that can completely or partially dewater the wetland within 24 hours.
- The wetland drain should be sized one pipe size greater than the calculated design diameter. The drain valve is typically a handwheel activated knife or gate valve. Valve controls shall be located inside of the riser at a point where they (a) will not normally be inundated and (b) can be operated in a safe manner.

See the design procedures in *Sections 2.0 and 2.2 of the Hydraulics Technical Manual* for additional information and specifications on pond routing and outlet works.

G. Emergency Spillway

An emergency spillway is to be included in the stormwater wetland design to safely pass flows that exceed the design storm flows. The spillway prevents the wetland's water levels from overtopping the embankment and causing structural damage. The emergency spillway must be located so that downstream structures will not be impacted by spillway discharges.

A minimum of 1 foot of freeboard must be provided, measured from the top of the water surface elevation for the flood mitigation storm to the lowest point on top of the dam, not counting the emergency spillway.

H Maintenance Access

A maintenance right of way or easement must be provided to the wetland facility from a public road or easement. Maintenance access should be at least 12 feet wide, have a maximum slope of no more than 15%, and be appropriately stabilized to withstand maintenance equipment and vehicles.

The maintenance access must extend to the forebay, safety bench, riser, and outlet and, to the extent feasible, be designed to allow vehicles to turn around.

Access to the riser is to be provided by lockable manhole covers, and manhole steps within easy reach of valves and other controls.

I Safety Features

All embankments and spillways must be designed to State of Texas Administrative Code for dams and reservoirs (see *iSWM Program Guidance – Dams and Reservoirs in Texas*).

Fencing of wetlands is not generally desirable, but may be required by the local review authority. A preferred method is to manage the contours of deep pool areas through the inclusion of a safety bench (see above) to eliminate dropoffs and reduce the potential for accidental drowning.

The principal spillway opening should not permit access by small children, and endwalls above pipe outfalls greater than 48 inches in diameter should be fenced to prevent a hazard.

J Landscaping

A landscaping plan should be provided that indicates the methods used to establish and maintain wetland coverage. Minimum elements of a plan include: delineation of landscaping zones, selection of corresponding plant species, planting plan, sequence for preparing wetland bed (including soil amendments, if needed), and sources of plant material.

Landscaping zones include low marsh, high marsh, and semi-wet zones. The low marsh zone ranges from 6 to 18 inches below the normal pool. This zone is suitable for the growth of several emergent plant species. The high marsh zone ranges from 6 inches below the pool up to the normal pool. This zone will support greater density and diversity of emergent wetland plant species. The high marsh zone should have a higher surface area to volume ratio than the low marsh zone. The semi-wet zone refers to those areas above the permanent pool that are inundated on an irregular basis and can be expected to support wetland plants.

The landscaping plan should provide elements that promote greater wildlife and waterfowl use within the wetland and buffers.

Woody vegetation may not be planted on a dam embankment or allowed to grow within 15 feet of the toe of the dam and 25 feet from the principal spillway structure.

A wetland buffer shall extend 25 feet outward from the maximum water surface elevation, with an additional 15-foot setback to structures. The wetland buffer should be contiguous with other buffer areas that are required by existing regulations (e.g., stream buffers) or that are part of the overall stormwater management concept plan. No structures should be located within the buffer, and an additional setback to permanent structures may be provided.

Existing trees should be preserved in the buffer area during construction. It is desirable to locate forest conservation areas adjacent to ponds. To discourage resident water fowl populations, the buffer can be planted with trees, shrubs and native ground covers.

The soils of a wetland buffer are often severely compacted during the construction process to ensure stability. The density of these compacted soils is so great that it effectively prevents root penetration and therefore may lead to premature mortality or loss of vigor. Consequently, it is advisable to excavate large and deep holes around the proposed planting sites and backfill these with uncompacted topsoil.

Guidance on establishing wetland vegetation can be found in the *Landscape Technical Manual.*

K Additional Site-Specific Design Criteria and Issues

Physiographic Factors - Local terrain design constraints

- Low Relief – Providing wetland drain can be problematic
- High Relief – Embankment heights restricted
- Karst – Requires poly or clay liner to sustain a permanent pool of water and protect aquifers; limits on ponding depth; geotechnical tests may be required

Soils

Hydrologic group "A" soils and some group "B" soils may require liner (not relevant for pocket wetland)

Special Downstream Watershed Considerations

- Local Aquatic Habitat – Design wetland offline and provide shading to reduce thermal impact; limit WQ_v-ED to 12 hours
- Aquifer Protection – Prevent possible groundwater contamination by preventing infiltration of hotspot runoff. May require liner for type "A" soils; Pretreat hotspots; 2 to 4 foot separation distance from water table.

28.6 Design Procedures

Step 1 Compute runoff control volumes from the *integrated* Design Focus Areas

Calculate the Water Quality Volume (WQ_v), Streambank protection Volume (SP_v), and the flood mitigation storm Flood Discharge, (for ED wetlands the design volume should be increased by 15%).

Details on the *integrated* Design Focus Areas are found in *Section 1.0 of the Planning Technical Manual.*

Step 2 Determine if the development site and conditions are appropriate for the use of a stormwater wetland

Consider the Application and Site Feasibility Criteria in *Sections 28.4 and 28.5* (A) (Location and Siting).

Step 3 Confirm local design criteria and applicability

Consider any special site-specific design conditions/criteria from *Section 28.5* (K) (Additional Site-Specific Design Criteria and Issues).

Check with local officials and other agencies to determine if there are any additional restrictions and/or surface water or watershed requirements that may apply.

Step 4 Determine pretreatment volume

A sediment forebay is provided at each inlet, unless the inlet provides less than 10% of the total design storm inflow to the pond. The forebay should be sized to contain 0.1 inches per impervious acre of contributing drainage and should be 4 to 6 feet deep. The forebay storage volume counts toward the total WQ_v requirement and may be subtracted from the WQ_v for subsequent calculations.

Step 5 Allocate the WQ_v volume among marsh, micropool, and extended detention volumes

Use recommended criteria from Table 28.1.

Step 6 Determine wetland location and preliminary geometry, including distribution of wetland depth zones

This step involves initially laying out the wetland design and determining the distribution of wetland surface area among the various depth zones (high marsh, low marsh, and deepwater). Set WQ_v permanent pool elevation (and WQ_v-ED elevation for extended detention shallow wetland) based on volumes calculated earlier.

See *Section 28.5* (C) (Physical Specification / Geometry) for more details.

Step 7 Compute extended detention orifice release rate(s) and size(s), and establish SP_v elevation

Shallow Wetland and Pocket Wetland: The SP_v elevation is determined from the stage-storage relationship and the orifice is then sized to release the streambank protection storage volume over a 24-hour period (12-hour extended detention may be warranted in some cold water

streams). The streambank protection orifice should have a minimum diameter of 3 inches and should be adequately protected from clogging by an acceptable external trash rack. A reverse slope pipe attached to the riser, with its inlet submerged 1 foot below the elevation of the permanent pool is a recommended design. The orifice diameter may be reduced to 1 inch if internal orifice protection is used (i.e., an over-perforated vertical stand pipe with ½-inch orifices or slots that are protected by wirecloth and a stone filtering jacket). Adjustable gate valves can also be used to achieve this equivalent diameter.

ED Shallow Wetland: Based on the elevations established in Step 6 for the extended detention portion of the water quality volume, the water quality orifice is sized to release this extended detention volume in 24 hours. The water quality orifice should have a minimum diameter of 3 inches, and should be adequately protected from clogging by an acceptable external trash rack. A reverse slope pipe attached to the riser, with its inlet submerged one foot below the elevation of the permanent pool, is a recommended design. Adjustable gate valves can also be used to achieve this equivalent diameter. The SP_v elevation is then determined from the stage-storage relationship. The invert of the streambank protection orifice is located at the water quality extended detention elevation, and the orifice is sized to release the streambank protection storage volume over a 24-hour period (12-hour extended detention may be warranted in some cold water streams).

Step 8 Calculate the intermediate flood control release rate and water surface elevation

Set up a stage-storage-discharge relationship for the control structure for the extended detention orifice(s) and the flood control storm.

Step 9 Design embankment(s) and spillway(s)

Size emergency spillway, calculate flood mitigation stormwater surface elevation, set top of embankment elevation, and analyze safe passage of the flood mitigation storm event (Q_f).

At final design, provide safe passage for the flood mitigation storm event. Attenuation may not be required.

Step 10 Investigate potential pond/wetland hazard classification

The design and construction of stormwater management ponds and wetlands are required to follow the latest version of the State of Texas Administrative Code for dams and reservoirs (see *iSWM Program Guidance – Dams and Reservoirs in Texas*).

Step 11 Design inlets, sediment forebay(s), outlet structures, maintenance access, and safety features.

See *Section 28.5* (E) through (I) for more details.

Step 12 Prepare Vegetation and Landscaping Plan

A landscaping plan for the wetland facility and its buffer should be prepared to indicate how aquatic and terrestrial areas will be stabilized and established with vegetation.

See *Section 28.5* (J) (Landscaping) and the *Landscape Technical Manual* for more details.

28.7 Inspection and Maintenance Requirements

Table 28.2 Typical Maintenance Activities for Wetlands
(Adapted from WMI, 1997 and CWP, 1998)

Constructed Wetland Systems	
Activity	**Schedule**
• Replace wetland vegetation to maintain at least 50% surface area coverage in wetland plants after the second growing season.	One-Time Activity
• Clean and remove debris from inlet and outlet structures. • Mow side slopes.	Frequently (3 to 4 times/year)
• Monitor wetland vegetation and perform replacement planting as necessary.	Semi-annual Inspection (first 3 years)
• Examine stability of the original depth zones and microtopographical features. • Inspect for invasive vegetation, and remove where possible. • Inspect for damage to the dam and inlet/outlet structures. Repair as necessary. • Note signs of hydrocarbon build-up, and remove appropriately • Monitor for sediment accumulation in the facility and forebay. • Examine to ensure that inlet and outlet devices are free of debris and operational.	Annual Inspection
• Repair undercut or eroded areas.	As Needed
• Harvest wetland plants that have been “choked out” by sediment build-up.	Annually
• Removal of sediment from the forebay	5 to 7 years or after 50% of the total forebay capacity has been lost
• Monitor sediment accumulations, and remove sediment when the pool volume has become reduced significantly, plants are “choked” with sediment, or the wetland becomes eutrophic.	10 to 20 years or after 25% of the wetland volume has been lost
• Ensure that inlets and outlets to each submerged gravel wetland cell are free from debris and not clogged. • Check for sediment buildup in gravel bed.	Monthly Annual inspection
• If sediment buildup is preventing flow through the wetland, remove gravel and sediment from cell. Replace with clean gravel and replant vegetation.	As needed
• Although there is less evaporation with a submerged as opposed to a surface wetland, supplemental water may be required during long periods without stormwater input.	As needed
• Routine maintenance of the vegetation (including harvesting) is not required, although weeds can be controlled by flooding the surface after planting and during early part of growing season.	As needed

Additional Maintenance Considerations and Requirements

- Maintenance requirements for constructed wetlands are particularly high while vegetation is being established. Monitoring during these first years is crucial to the future success of the wetland as a stormwater structural control. Wetland facilities should be inspected after major storms (greater than 2 inches of rainfall) during the first year of establishment to assess bank stability, erosion damage, flow channelization, and sediment accumulation within the wetland. For the first 3 years, inspections should be conducted at least twice a year.

- A sediment marker should be located in the forebay to determine when sediment removal is required.
- Accumulated sediments will gradually decrease wetland storage and performance. The effects of sediment deposition can be mitigated by the removal of the sediments.
- Sediments excavated from stormwater wetlands that do not receive runoff from designated hotspots are not considered toxic or hazardous material and can be safely disposed of by either land application or landfilling. Sediment testing may be required prior to sediment disposal when a hotspot land use is present. Sediment removed from stormwater wetlands should be disposed of according to an approved erosion and sediment control plan.
- Periodic mowing of the wetland buffer is only required along maintenance rights-of-way and the embankment. The remaining buffer can be managed as a meadow (mowing every other year) or forest.



Regular inspection and maintenance is critical to the effective operation of stormwater wetlands as designed. Maintenance responsibility for a wetland facility and its buffer should be vested with a responsible authority by means of a legally binding and enforceable maintenance agreement that is executed as a condition of plan approval.

28.8 Example Schematics

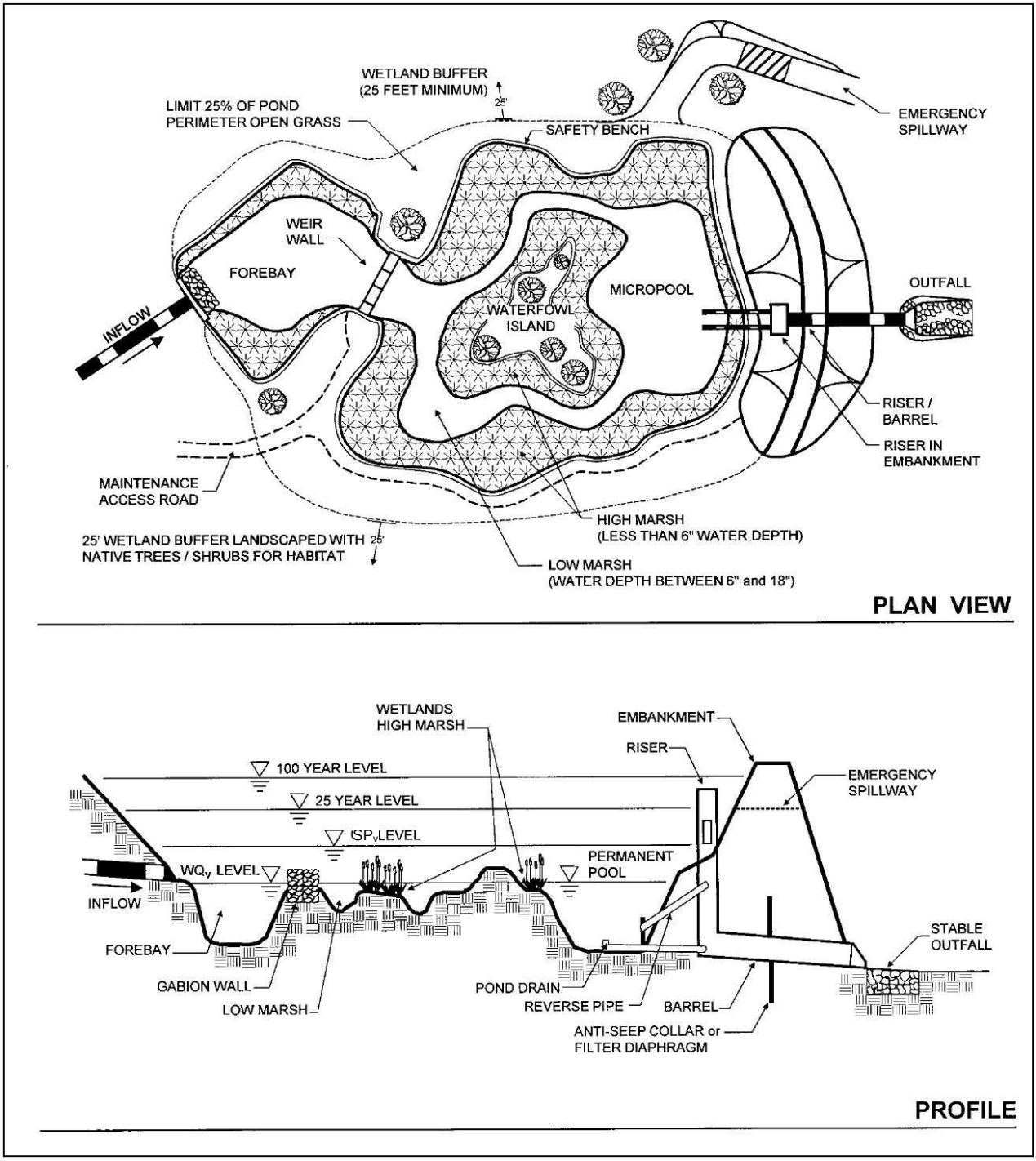


Figure 28.3 Schematic of Shallow Wetland
(Source: Center for Watershed Protection)

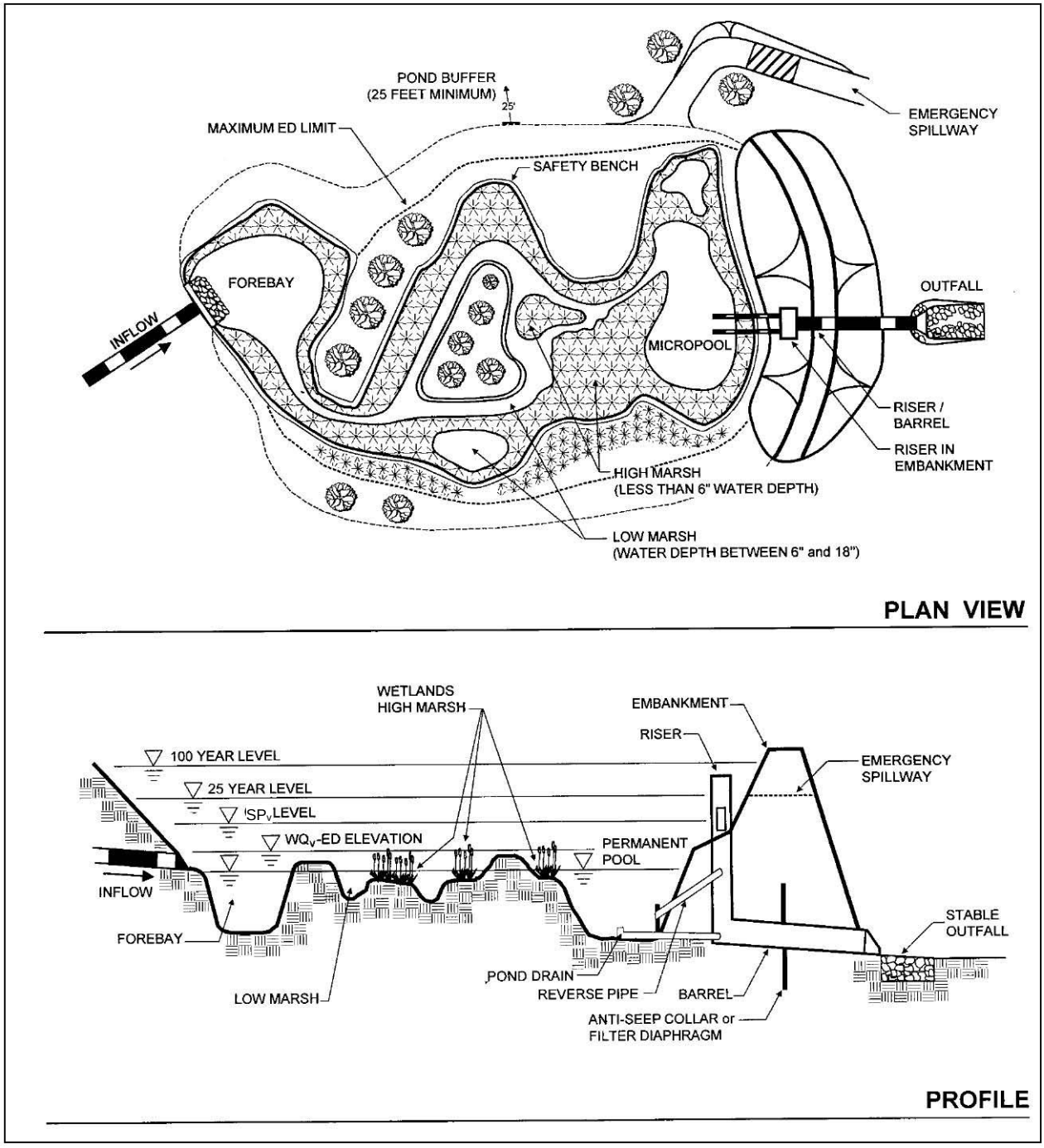


Figure 28.4 Schematic of Extended Detention Shallow Wetland
(Source: Center for Watershed Protection)

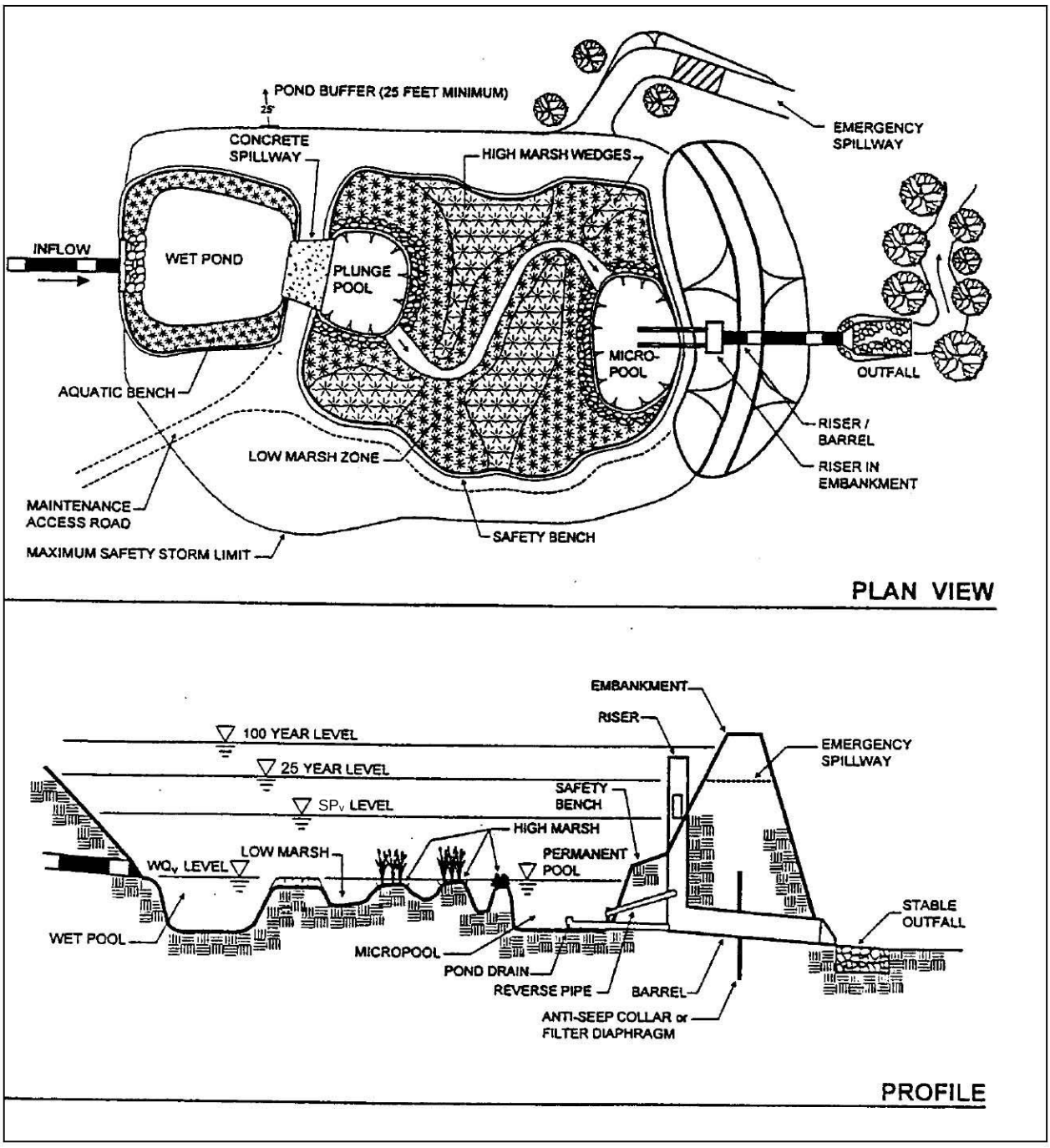


Figure 28.5 Schematic of Pond/Wetland System
(Source: Center for Watershed Protection)

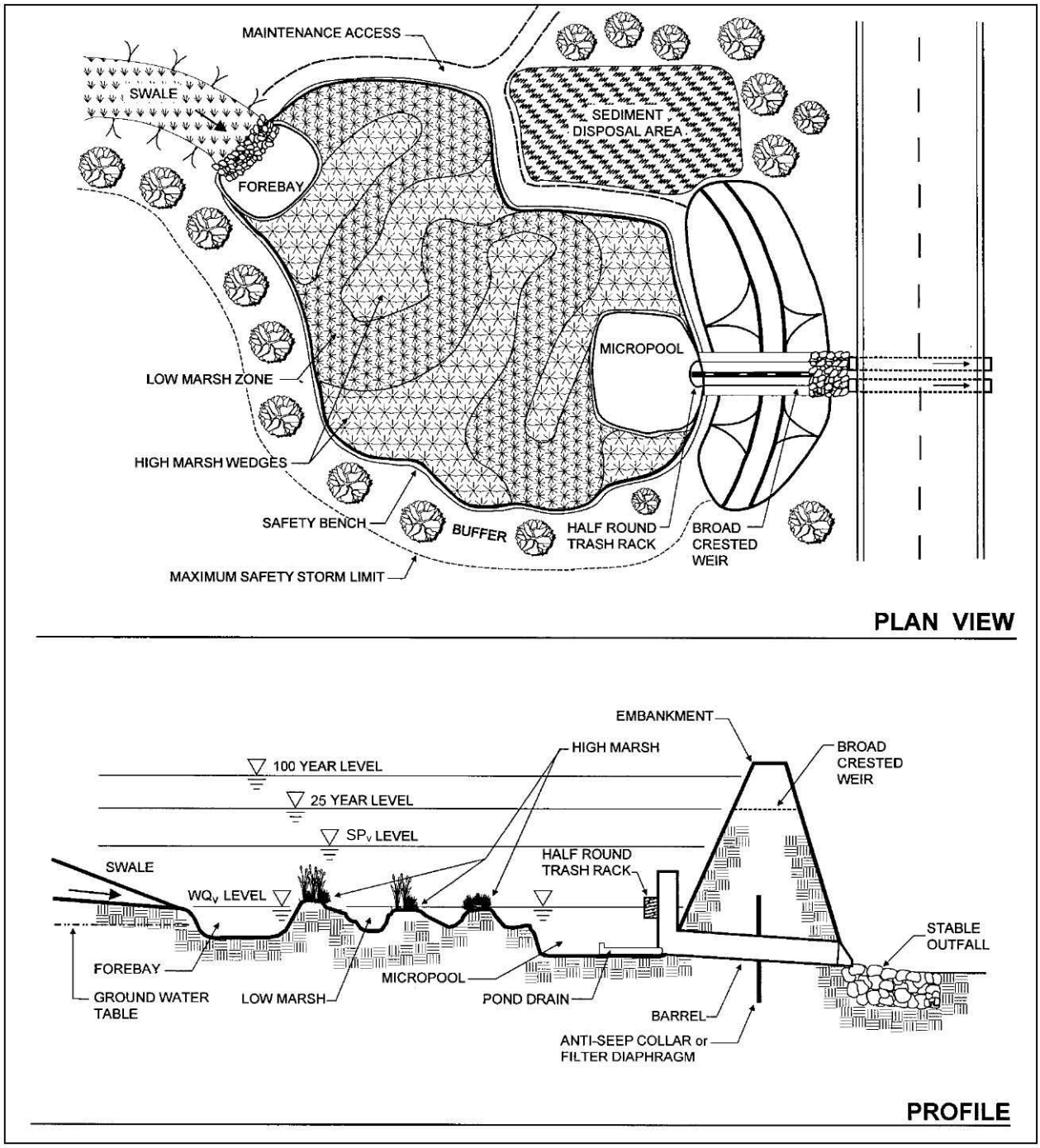
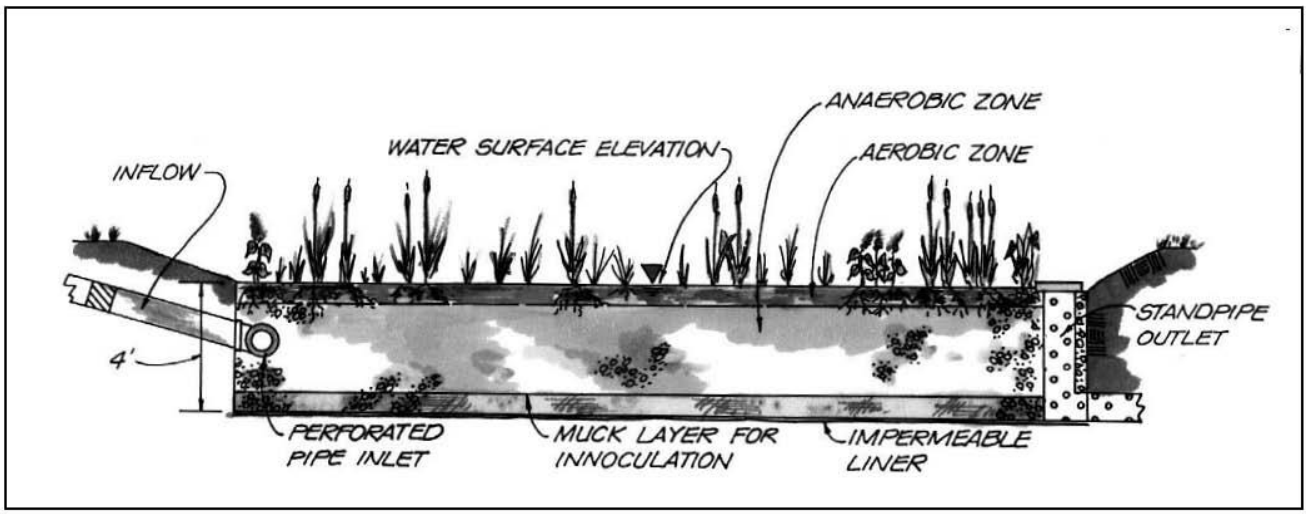


Figure 28.6 Schematic of Pocket Wetland
(Source: Center for Watershed Protection)



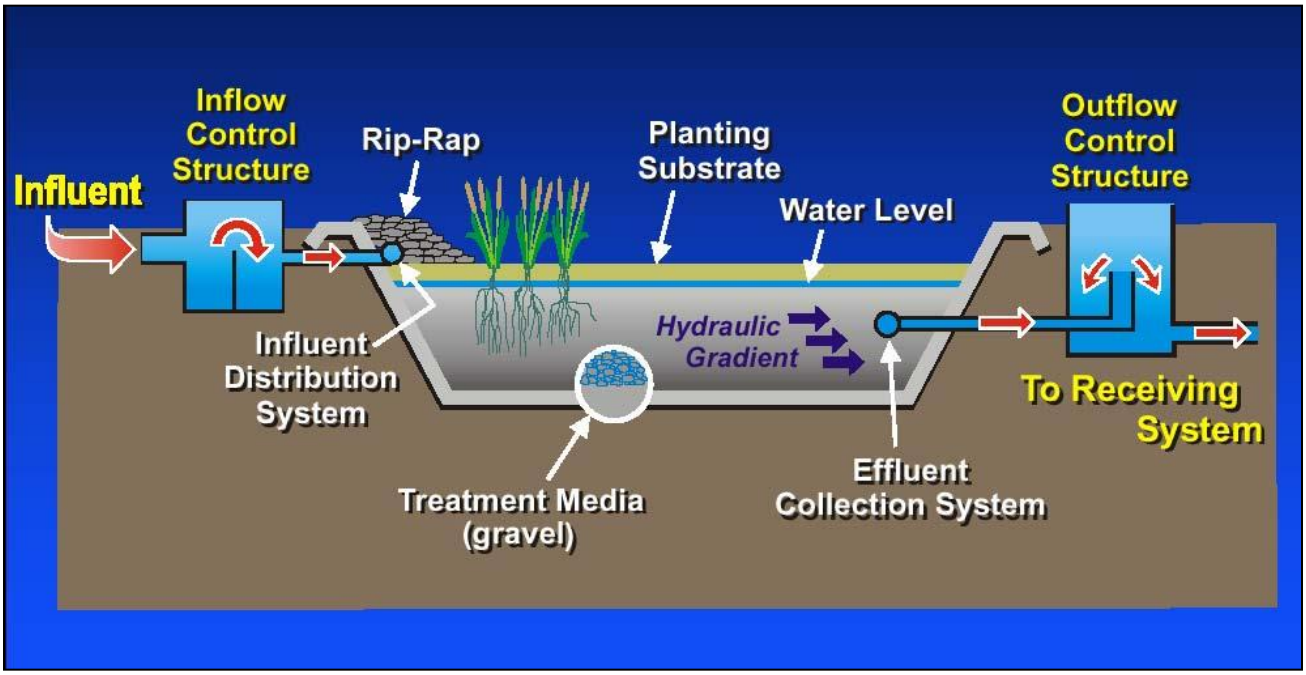


Figure 28.7 Schematics of Submerged Gravel Wetland System
(Sources: Center for Watershed Protection; Roux Associates Inc.)

28.9 Design Forms

Design Procedure Form: Storm Water Wetlands

PRELIMINARY HYDROLOGIC CALCULATIONS

1a. Compute WQ_v volume requirements
Compute Runoff Coefficient, R_v
Compute WQ_v

R_v = _____
WQ_v = _____ acre-ft

1b. Compute SP_v
Compute average release rate
Compute Q_p (100-year detention volume required)
Add 15% to the required Q_p volume (if ED)
Compute (as necessary) Q_f

SP_v = _____ acre-ft
release rate = _____ cfs
Q_p = _____ acre-ft
Q_p * 15% = _____ acre-ft
Q_f = _____ cfs

STORM WATER WETLAND DESIGN

2. Is the use of a storm water wetland appropriate?

See subsections 5.2.27.4 and 5.2.27.5-A

3. Confirm local design criteria and applicability.

4. Pretreatment volume
Vol_{pre} = 1 (0.1")(1'/12")

Vol_{pre} = _____ acre-ft

5. Allocation of Pool, Marsh, and ED Volumes

Shallow Wetland:
Vol_{pool} = 0.25 (WQ_v)
Vol_{marsh} = 0.75 (WQ_v)

Vol_{pool} = _____ acre-ft
Vol_{marsh} = _____ acre-ft

Shallow ED Wetland:
Vol_{pool} = 0.25 (WQ_v)
Vol_{marsh} = 0.25 (WQ_v)
Vol_{ED} = 0.5 (WQ_v)

Vol_{pool} = _____ acre-ft
Vol_{marsh} = _____ acre-ft
Vol_{ED} = _____ acre-ft

Pocket Wetland:
Vol_{pool} = 0.25 (WQ_v)
Vol_{marsh} = 0.75 (WQ_v)

Vol_{pool} = _____ acre-ft
Vol_{marsh} = _____ acre-ft

6. Allocation of Surface Area
(choose from Table 5.2.27-1 based on wetland variant)
Pool/Deepwater Wetland Zone (1.5-6 feet deep)
Low Marsh Wetland Zone (6-18 inches deep)
High Marsh Wetland Zone (0-6 inches deep)
Semi-Wet Wetland Zone (above pool depth)

$Area_{water}$ = _____ acres, % = _____
$Area_{low}$ = _____ acres, % = _____
$Area_{high}$ = _____ acres, % = _____
$Area_{semi}$ = _____ acres, % = _____

Conduct grading and determine storage available for marsh zones (and ED if applicable), and compute orifice size

Prepare an elevation-storage table and curve using the average area method for computing volumes.

Elevation MSL	Area ft^2	Average Area ft^2	Depth ft	Volume ft^3	Cummulative Volume ft^3	Cummulative Volume ac-ft	Volume above Permanent Pool ac-ft

Notes:

7. WQ_v Orifice Computations
   Average ED release rate (if applicable) — release rate _____ cfs
   Average head, h = (ED elev. - Permanent pool elev.)/2 — h = _____ ft
   Area of orifice from orifice equation — A = _____ ft^2
   $Q = CA(2gh)^{0.5}$ — diameter = _____ in
   Discharge equation $Q = (h)^{0.5}$ — factor = _____ $(h)^{0.5}$

   Compute release rate for SP_v-ED control and establish SP_v elevation — WSEL = _____ ft-NGVD
   Release rate = — release rate = _____ cfs
   Average head, h = (SP_v elev. - Permanent pool elev.)/2 — h = _____ ft
   Area of orifice from orifice equation — A = _____ ft^2
   $Q = CA(2gh)^{0.5}$ — diameter = _____ in
   Discharge equation $Q = (h)^{0.5}$ — factor = _____ $(h)^{0.5}$

8. Calculate Q_p release rate and WSEL — ***Set up a stage-storage-discharge relationship***

Elevation	Storage	Low Flow	Riser			Barrier		Emergency	Total
		WQ_v-ED	SPv-ED	High Storage		Inlet	Pipe	Spillway	Storage
MSL	ac-ft	Ft(ft) Q(cfs)	Ft(ft) Q(cfs)	Orif. Ft Q	Weir Ft Q	Ft(ft) Q(cfs)	Ft(ft) Q(cfs)	Ft(ft) Q(cfs)	Q(cfs)

   Q_p = pre-dev. peak discharge - (WQ_v-ED release + SP_v-ED release) — Q_p = _____ cfs

   Maximum head = — H = _____ ft
   Use weir equation for slot length ($Q = CLH^{3/2}$) — L = _____ ft

   Check inlet condition — ***Use culvert charts***
   Check outlet condition — ***(Section 4.2)***

9. Size emergency spillway, calculate 100-year WSEL and set top of embankment elevation — $WSEL_{25}$ = _____ ft; $WSEL_{100}$ = _____ ft; Q_{ES} = _____ cfs; Q_{ps} = _____ cfs

10. Investigate potential pond hazard classification — ***See Appendix H***

11. Design inlets, sediment forebays, outlet structures, maintenance access, and safety features. — ***See subsection 5.2.27.5-D through H***

12. Attach landscaping plan (including wetland vegetatoin) — ***See Appendix F***

Notes: _____

29.0 Stormwater Control Design Examples

29.1 Introduction

The five stormwater control examples included in this section are presented to provide example computations using the most commonly used design standards. All of the possible options that might be incorporated into a stormwater control structure are not covered in these examples.

For all possible design standards and options that might be incorporated into the design of a stormwater control structure, refer to *Table 1.1 of the Planning Technical Manual*. In order to apply design standards to a given stormwater control structure, additions or modifications to *Section 1.0 and 2.0 of the Planning Technical Manual* must also be considered. These are found in the Criteria Manual.

The Criteria Manual may determine the extent of downstream assessments; the return periods to be used for the "Streambank Protection" and "Conveyance" storms; and the acceptable design focus area to be used for Water Quality Protection, Streambank Protection, and Flood Control. It is evident that there are many combinations of design options that might apply to a given stormwater control structure.

29.2 Stormwater Pond Design Example

The following design example is for a wet extended detention (ED) stormwater pond. The design options chosen for this example are Option 1 for Water Quality Protection (WQ_v), Option 4 for Streambank Protection (SP_v), and Option 4 for Flood Control (Q_{p100}). The layout of the Rolling Meadows Subdivision is shown in Figure 29.1.

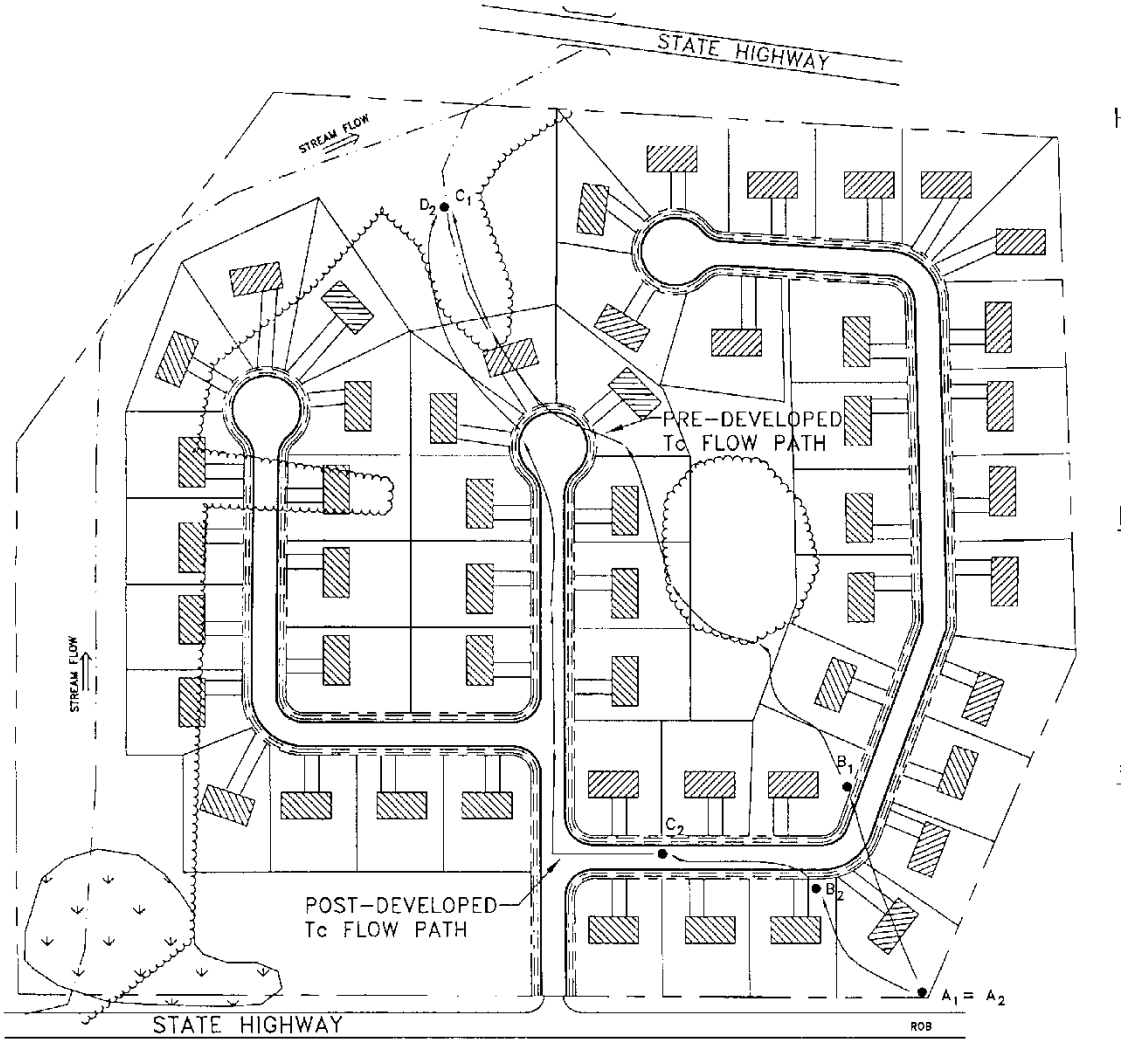


Figure 29.1 Rolling Meadows Site Plan

Base Data	Hydrologic Data		
Site Area = Total Drainage Area (A) = 38.0 ac		Pre	Post
Measured Impervious Area=13.8 ac; or I=13.8/38=36.3%	CN	76	85
Soils Types: 20% "C", 80% "B"	t_c	0.33 hr	0.19 hr
Zoning: Residential (½ acre lots) Denton County			

Computation of Preliminary Stormwater Storage Volumes and Peak Discharges

Step 1: Compute runoff control volumes from the integrated Design Focus Areas

More details hydrologic calculations will be required during the design step – these numbers are preliminary.

Compute Water Quality Volume, WQ_v

- Compute Runoff Coefficient, R_v

  R_v = 0.05 + (I) (0.009)
  = 0.05 + (36.3) (0.009) = 0.38

- Compute WQ_v

  WQ_v = (1.5") (R_v) (A)
  = (1.5") (0.38) (38.0 ac) (1ft/12in)
  = 1.44 ac-ft

Develop Site Hydrologic and Hydrologic Input Parameters

Per Figures 29.2 and 29.3. Note that any hydrologic models using SCS procedures, such as TR-20, HEC-HMS, or HEC-1, can be used to perform preliminary hydrologic calculations

Condition	Area	CN	Tc
	Ac		*hrs*
pre-developed	38	76	0.33
post-developed	38	85	0.19

Perform Preliminary Hydrologic Calculations

Condition	$Q_{1\text{-yr}}$	$Q_{1\text{-yr}}$	$Q_{100\text{-yr}}$
Runoff	*Inches*	*cfs*	*cfs*
pre-developed	0.78	26.9	266
post-developed	1.29	61.3	402

Compute Streambank Protection Volume, (SP_v)

For stream streambank protection, provide 24 hours of extended detention for the 1-year event.

Utilize SCS approach to Compute Streambank Protection Storage Volume

See *Section 1.1 of the Hydrology Technical Manual.*

- Initial abstraction (I_a) for CN of 85 is 0.353: [I_a = (200/CN - 2)]
- I_a/P = (0.353)/ 2.64 inches = 0.13
- T_c = 0.19 hours
- q_u = 800 csm/in (Type II Storm)

Knowing q_u and T (extended detention time), find q_o/q_i. For a Type II rainfall distribution.

- Peak outflow discharge/peak inflow discharge (q_o/q_i) = 0.022
- $V_s/V_r = 0.683 - 1.43(q_o/q_i) + 1.64(q_o/q_i)^2 - 0.804(q_o/q_i)^3$
- Where V_s equals streambank protection storage (SP_v) and V_r equals the volume of runoff in inches.
- V_s/V_r = 0.65

- Therefore, $V_s = SP_v$ = 0.65(1.29")(1/12)(38 ac) = 2.66 ac-ft (116,077 cubic feet)

Define the average SP_v-ED Release Rate

- The above volume, 2.66 ac-ft, is to be released over 24 hours.
- (2.66 ac-ft × 43,560 ft^2/ac) / (24 hrs × 3,600 sec/hr) = 1.34 cfs

Analyze Safe Passage of 100 Year Design Storm (Q_f)

At final design, provide safe passage for the 100-year event, or detain it, depending on downstream conditions and local policy. Based on field observation and review of local requirements no control of the 100-year storm is necessary. If it were storage estimates would have been made similar to the Q_p Volume in the previous sub-step.

Table 29.1 Summary of General Storage Requirements for Rolling Meadows

Symbol	Control Volume	Volume Required (ac- ft)	Notes
WQ_v	Water Quality	1.44	
SP_v	Streambank Protection	2.66	Average extended detention release rate is 1.34 cfs over 24 hours
Q_f	Extreme Flood Protection	4.88	Detain to pre-developed conditions; Provide safe passage for the 100-year event in final design

The Modified Rational Method is used to estimate the storage volume.

Determine the Allowable Release Rate, Q_a

- Predevelopment Rational Coefficient, c_a= 0.45
- For t_c=0.33 hr, from *Table 5.3 of the Hydrology Technical Manual*, i_{100} =6.99 in/hr
- From *Equation 1.26 of the Hydrology Technical Manual*, $Q_a = c_a\ i\ A$= (0.45) (6.99) (38)=119.5 cfs

Determine the Critical Duration of the Storm T_d

- From *Table 1.18 of the Hydrology Technical Manual*, a=325.18; b=24.822
- Post-developed Rational Coefficient, c=0.61
- From *Equation 1.27 of the Hydrology Technical Manual*,

  $Td=\sqrt{(2cAab/Q_a)} - b$

  $Td=\sqrt{(2(0.61)(38)\ (325.18)\ (24.822)/(119.5)\ )} - (24.822)$ = 31.1 min
- From *Table 5.3 of the Hydrology Technical Manual*, for T_d= 31.1min, i_{Td} = 5.55 in/hr

  P_{Td} = 5.55 in/hr (31.1 min) (hr/ 60min) = 2.88 in

Determine the Allowable Release Rate, Q_a

- Predevelopment Rational Coefficient, c_a= 0.45
- For t_c=0.33 hr, from *Table 5.3 of the Hydrology Technical Manual*, i_{100} =6.99 in/hr
- From *Equation 1.26 of the Hydrology Technical Manual*,

  $Q_a = c_a\ i\ A$= (0.45) (6.99) (38)=119.5 cfs

Compute Storage Volume

- Post-developed Time of concentration, t_c = 0.19 hr (60 min/hr) = 11.4 min
- From *Equation 1.28a of the Hydrology Technical Manual*,

  $V_{pre} = 60\ [cAa-(2cabAQ_a)^{1/2} + (Q_a/2)\ (b-t_c)]$

  $V_{pre} = 60\ \{[0.61(38)(325.18)- 2\ (0.61)(24.822)(38)(119.5)]^{1/2} + (119.5/2)\ (24.822-11.4)\}$
  $= 99154\ ft^3$
- From *Equation 1.28b of the Hydrology Technical Manual*,

  $V_{max} = V_{pre} * P_{180}/P_{Td}$

  P_{180} = 1.79 in/hr (3 hr) in *Table 5.3 of the Hydrology Technical Manual.*

  V_{max} = 99154 (5.37/ 2.88) = 184881 ft^3 = 4.24 ac-ft

Experience has shown that additional 10-15% storage is required when multiple levels of extended detention are provided. So, for preliminary sizing purposes added 15% to the required volume for downstream flood control. O_f= 1.15 (4.24) = 4.88 ac-ft

PEAK DISCHARGE SUMMARY				
JOB:	**P'TREE MEADOWS**			**EWB**
DRAINAGE AREA NAME:	**PRE-DEVELOPED CONDITIONS**			**3-Jan-00**
COVER DESCRIPTION	**SOIL GROUP A, B, C, D?**	**C from TABLE 1.6 Hydrology Section**	**CN from TABLE 1.9 Hydrology Section**	**AREA (in acres)**
meadow (good cond.)	D	0.5	78	30.40
meadow (good cond.)	C	0.5	71	2.60
woods (good cond.)	C	0.15	70	5.00
			AREA SUBTOTALS:	**38.00**
Time of Concentration	**Surface Cover**	**Manning 'n'**	**Flow Length**	**Slope**
2-Yr 24 Hr Rainfall = 3.36"	**Cross Section**	**Wetted Per**	**Avg Velocity**	**Tt (hrs)**
Sheet Flow	**dense grass**	**'n'= 0.24**	**150ft**	**2.50%**
				0.29 hrs
Shallow Flow	**unpaved**		**500 ft**	**4.00%**
			3.23 fps	**0.04 hrs**
Channel Flow				
Total Area in Acres =	**38.00**	Total Sheet Flow = 0.29 hrs.	Total Shallow Flow = 0.04 hrs.	Total Channel Flow = 0.00 hrs.
Weighted CN =	**76**			
Time of Concentration =	**0.33 hrs**			
Pond Factor =	**1**	**RAINFALL TYPE II**		

STORM	Precipitation (P) inches	Runoff (Q)	Qp, PEAK DISCHARGE (cfs)
1 Year	2.64	0.78	26.9
2 Year	3.36	1.26	45.1
5 Year	4.8	2.37	88
10 Year	5.52	2.97	113
25 Year	6.96	4.22	165
50 Year	7.92	5.09	205
100 Year	9.36	6.41	266

Figure 29.2 Rolling Meadows Pre-Development Conditions

PEAK DISCHARGE SUMMARY				
JOB:	**P'TREE MEADOWS**			**EWB**
DRAINAGE AREA NAME:	**POST-DEVELOPED CONDITIONS**			**3-Jan-00**
COVER DESCRIPTION	**SOIL GROUP A, B, C, D?**	**C from TABLE 1.6 Hydrology Section**	**CN from TABLE 1.9 Hydrology Section**	**AREA (in acres)**
open space (good cond.)	D	0.5	80	20.00
woods (good cond.)	C	0.15	70	5.0
impervious area	D	.95	98	10.4
Impervious area	C	.95	98	2.6
			AREA SUBTOTALS:	**38.00**
Time of Concentration	**Surface Cover**	**Manning 'n'**	**Flow Length**	**Slope**
2-Yr 24 Hr Rainfall = 3.36"	**Cross Section**	**Wetted Per**	**Avg Velocity**	**Tt (hrs)**
Sheet Flow	**short grass**	**'n'= 0.15**	**100 ft**	**2.50%**
				0.15 hrs
Shallow Flow	**paved**		**300 ft**	**2.00%**
			2.87 fps	**0.03 hrs**
Channel Flow		**'n'=0.013**	**600 ft**	**2.00%**
Hydraulic Radius	X-S estimated	WP estimated	**16.21 fps**	**0.01 hrs**
Total Area in Acres =	38.00	Total Sheet Flow =	Total Shallow Flow =	Total Channel Flow =
Weighted CN =	85			
Time of Concentration =	0.19 hrs	0.15 hrs.	0.03 hrs.	0.01 hrs.
Pond Factor =	1	RAINFALL TYPE II		

STORM	Precipitation (P) inches	Runoff (Q)	Qp, PEAK DISCHARGE (cfs)
1 Year	2.64	1.29	61.3
2 Year	3.36	1.89	92.3
5 Year	4.8	3.18	159
10 Year	5.52	3.85	196
25 Year	6.96	5.21	269
50 Year	7.92	6.14	322
100 Year	9.36	7.53	402

Figure 29.3 Rolling Meadows Post-Development Conditions

Step 2: Determine if the development site and conditions are appropriate for the use of a stormwater pond

Site Specific Data:

The site area and drainage area to the pond is 38.0 acres. Existing ground at the pond outlet is 919 MSL. Soil boring observations reveal that the seasonally high water table is at elevation 918. The underlying soils are predominantly clay and are suitable for earthen embankments and to support a wet pond without a liner. The stream invert at the adjacent stream is at elevation 916.

Other site screening aspects listed in *Section 1.1 and 1.2 of the Hydraulics Technical Manual* were assessed and a pond was found to be suitable.

Step 3: Confirm local design criteria and applicability

There are no additional requirements for this site.

Step 4: Determine pretreatment volume

Size wet forebay to treat 0.1"/impervious acre. (13.8 ac) (0.1") (1'/12") = **0.12 ac-ft**
(forebay volume is included in WQ_v as part of permanent pool volume)

Step 5: Determine permanent pool volume (and water quality extended detention volume)

Size permanent pool volume to contain 50% of WQ_v:

0.5 × (1.44 ac-ft) = **0.72 ac-ft**. (includes 0.12 ac-ft of forebay volume)

Size extended detention volume to contain 50% of WQ_v: 0.5 × (1.44 ac-ft) = **0.72 ac-ft**

Note: This design focus area assumes that all of the extended detention volume will be in the pond at once. While this will not be the case, since there is a discharge during the early stages of storms, this conservative approach allows for extended detention control over a wider range of storms, not just the target rainfall.

Step 6: Determine pond location and preliminary geometry. Conduct pond grading and determine storage available for permanent pool and water quality extended detention

This step involves initially grading the pond (establishing contours) and determining the elevation-storage relationship for the pond. Storage must be provided for the permanent pool (including sediment forebays), extended detention (WQ_v-ED), SP_v-ED, downstream flood protection, plus sufficient additional storage to pass the 100-year storm with minimum freeboard. An elevation-storage table and curve is prepared using the average area method for computing volumes. See Figure 29.4 for pond location on site, Figure 29.5 grading and Figure 29.6 for Elevation-Storage Data.

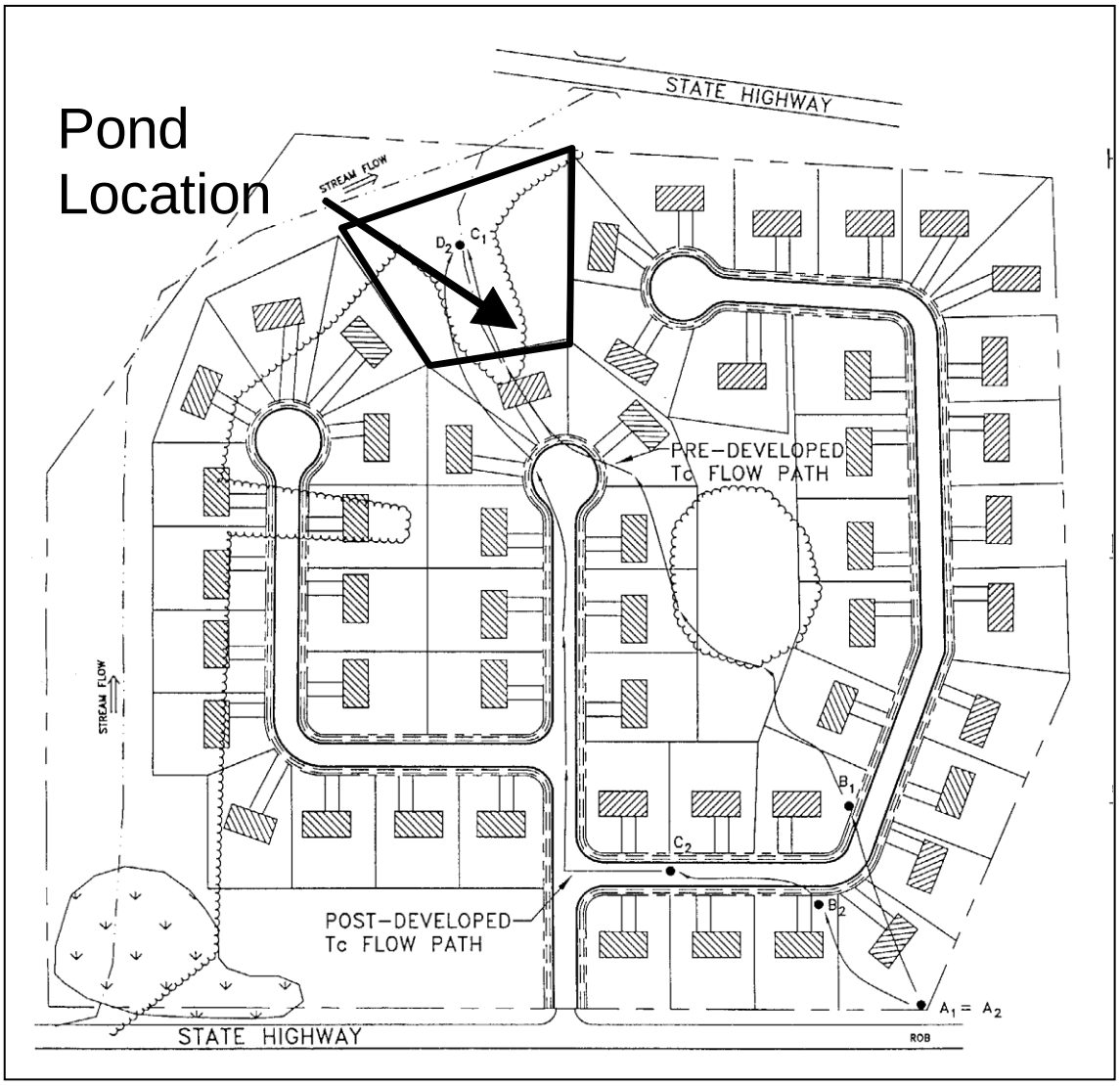


Figure 29.4 Pond Location on Site

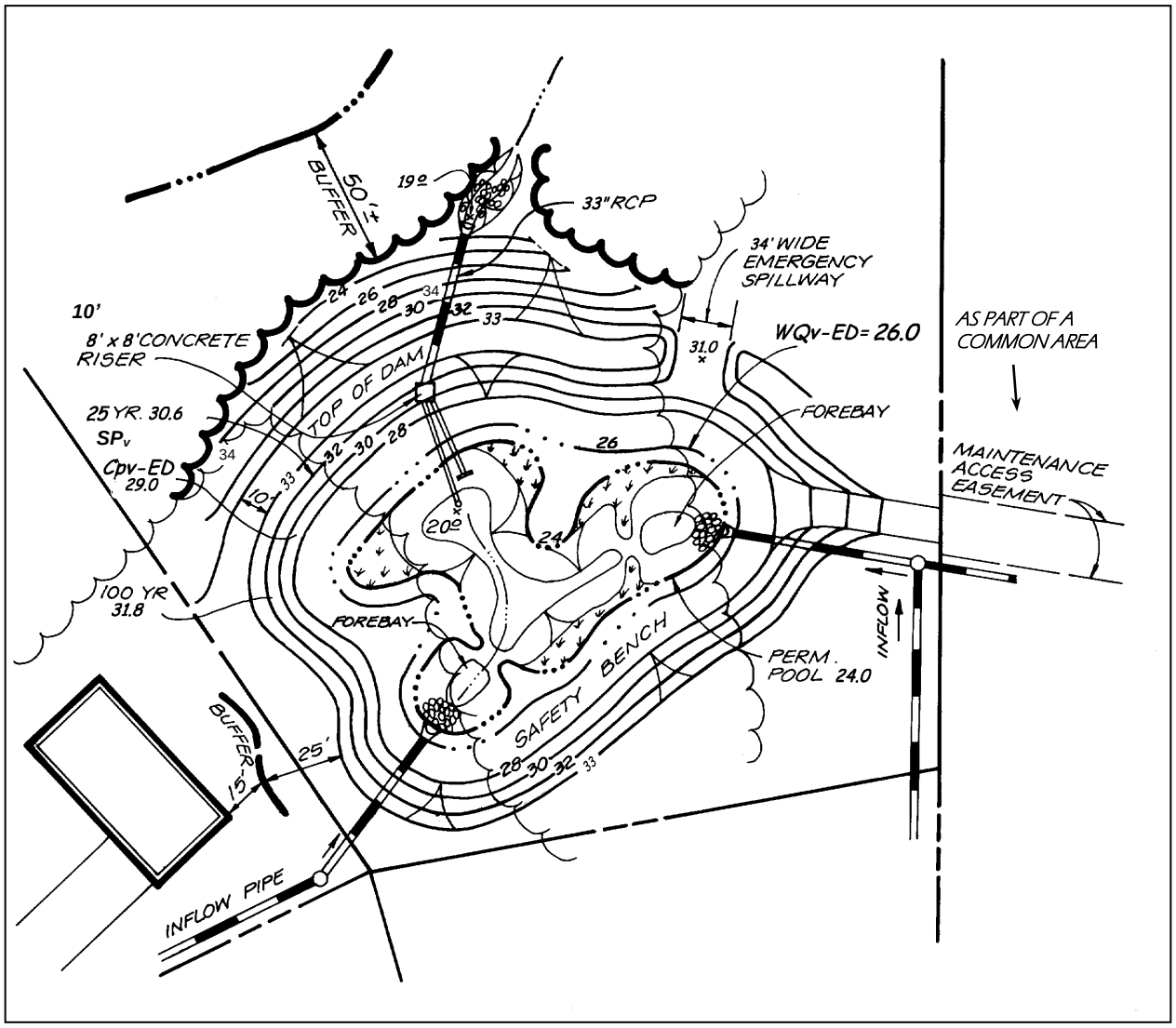


Figure 29.5 Plan View of Pond Grading (Not to Scale)

Elevation MSL	Average Area ft^2	Depth ft	Volume ft^3	Cumulative Volume ft^3	Cumulative Volume ac-ft	Volume Above Permanent Pool ac-ft
920.0						
921.0	7838	1	7838	7838	0.18	
923.0	11450	2	22900	30738	0.71	
924.0	14538	1	14538	45275	1.04	0
925.0	15075	1	15075	60350	1.39	0.35
925.5	16655	0.5	8328	68678	1.58	0.54
926.0	17118	0.5	8559	77236	1.77	0.73
926.5	21000	0.5	10500	87736	2.01	0.97
927.0	25000	0.5	12500	100236	2.30	1.26
927.5	30000	0.5	15000	115236	2.65	1.61
928.0	36000	0.5	18000	133236	3.06	2.02
928.5	38000	0.5	19000	152236	3.49	2.46
929.0	41000	0.5	20500	172736	3.97	2.93
929.5	43000	0.5	21500	194236	4.46	3.42
930.0	45000	0.5	22500	216736	4.98	3.94
930.5	47000	0.5	23500	240236	5.52	4.48
931.0	49000	0.5	24500	264736	6.08	5.04
931.5	52000	0.5	26000	290736	6.67	5.64
932.0	55000	0.5	27500	318236	7.31	6.27
932.5	58000	0.5	29000	347236	7.97	6.93
933.0	61000	0.5	30500	377736	8.67	7.63
933.5	65000	0.5	32500	410236	9.42	8.38
934.0	69000	0.5	34500	444736	10.21	9.17
935.0	74000	1	74000	518736	11.91	10.87

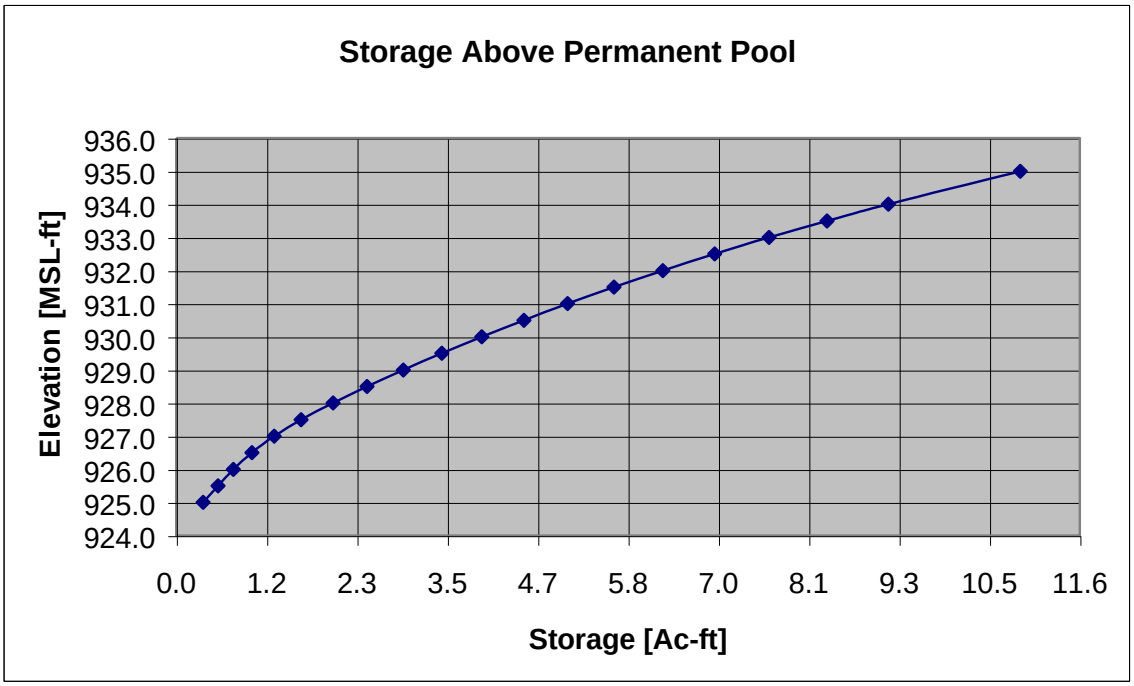


Figure 29.6 Storage-Elevation Table and Curve

Set basic elevations for pond structures

- The pond bottom is set at elevation 920.0.
- Provide gravity flow to allow for pond drain, set riser invert at 919.5.
- Set barrel outlet elevation at 919.0.

Set water surface and other elevations

- Required permanent pool volume = 50% of WQ_v = 0.72 ac-ft. From the elevation-storage table, read elevation 924.0 (1.04 ac-ft > 0.72 ac-ft). The site can accommodate it and it allows a small safety factor for fine sediment accumulation - OK
- Forebay volume provided in two pools with avg. vol. = 0.08 ac-ft each (0.16 ac-ft > 0.12 ac-ft) OK
- Required extended detention volume (WQ_v-ED) = 0.72 ac-ft. From the elevation-storage table (volume above permanent pool), read elevation 926.0 (0.73 ac-ft > 0.72 ac-ft) OK. Set extended detention wsel = 926.0

Note: Total storage at elevation 926.0 = 1.77 ac-ft (greater than required WQ_v of 1.44 ac-ft)

Compute the required WQ_v-ED orifice diameter to release 0.72 ac-ft over 24 hours

- Avg. extended detention release rate = (0.72 ac-ft)(43,560 ft^2/ac)/(24 hr)(3600 sec/hr) = 0.36 cfs
- Average head = (926.0 - 924.0)/ 2 = 1.0'
- Use orifice equation to compute cross-sectional area and diameter
  - $Q = CA(2gh)^{0.5}$, for Q = 0.36 cfs, h = 1.0 ft, C = 0.6 = discharge coefficient) solve for A
  - $A = 0.36 \text{ cfs} / [(0.6)((2)32.2 \text{ ft/s}^2)(1.0 \text{ ft}))^{0.5}]$ A = 0.075 ft^2, $A = \pi d^2 / 4$; dia. = 0.31 ft = 3.7"
  - Use 4" pipe with 4" gate valve to achieve equivalent diameter

Compute the stage-discharge equation for the 3.7" dia. WQv orifice

- $Q_{WQv\text{-}ED} = CA(2gh)^{0.5} = (0.6)\ (0.075 \text{ ft}^2)\ [((2)(32.2 \text{ ft/s}^2))^{0.5}]\ (h^{0.5})$,
- $Q_{WQv\text{-}ED} = (0.36)\ h^{0.5}$, where: h = wsel - 924.16

(Note: account for one half of orifice diameter when calculating head)

Step 7: Compute extended detention orifice release rate(s) and size(s), and establish SP_v elevation

Set the SP_v pool elevation

- Required SP_v storage = 2.66 ac-ft (see Table 29.1).
- From the elevation-storage table, read elevation 929 (this includes the WQ_v).
- Set SP_v wsel = 929

Size SP_v orifice

- Size to release average of 1.34 cfs.
  - Average WQ_v-ED orifice release rate is 0.66 cfs, based on average head of 3.34' (926 – 924.16 + (929 – 926)/2)
  - SP_v-ED orifice release = 1.34 -0.66 = 0.66 cfs
- Head = (929 - 926.0)/2 = 1.5'

Use orifice equation to compute cross-sectional area and diameter

- $Q = CA(2gh)^{0.5}$, for h = 1.5'
  - $A = 0.68 \text{ cfs} / [(0.6)((2)(32.2'/s^2)(1.5'))^{0.5}]$
  - $A = 0.12 \text{ ft}^2$, $A = \pi d^2 / 4$;
  - dia. = 0.38 ft = 4.6"
  - Use PVC pipe to the nearest 1" (in this case 5" PVC pipe)

Compute the stage-discharge equation for the 4.6" dia. SP_v orifice

- $Q_{SPv\text{-}ED} = CA(2gh)^{0.5} = (0.6)\ (0.12 \text{ ft}^2)\ [((2)\ (32.2'/s^2))^{0.5}]\ (h^{0.5})$,
- $Q_{SPv\text{-}ED} = (0.55)\ (h^{0.5})$, where: h = wsel - 926.19

(Note: Use the distance form the water surface to the center of the orifice when calculating head)

Step 8: Calculate Q_f (100-year storm) release rate and water surface elevation

In order to calculate the release rate and water surface elevation, the designer must set up a stage-storage-discharge relationship for the control structure for each of the low flow release pipes (WQ_v-ED and SP_v-ED) plus the 100 year storm.

Develop basic data and information

- The 100 year pre-developed peak discharge = 266 cfs,
- The post developed inflow = 402 cfs, from Table 29.1,
- From previous estimate Q_f = 4.88 ac-ft.
- From elevation-storage table (Figure 29.6), read elevation 930.9.

Size 100-year slot to release 266 cfs at elevation 930.9.

- @ wsel 930.9:
  - WQ_v-ED orifice releases 0.93 cfs,
  - SP_v-ED orifice releases 1.19 cfs, therefore;
  - Allowable Q_p = 266 cfs - (.93 + 1.19) = 263.9 cfs, say 264 cfs.
- Max head = (930.9 – 929) = 1.9'
- Use weir equation to compute slot length
  - $Q = CLH^{3/2}$
  - $L = 264 \text{ cfs} / (3.1)\ (1.9^{3/2}) = 32.5 \text{ ft}$
- Use four 8.5 ft x 2 ft slots for 100-year release (opening should be slightly larger than needed so as to have the barrel control before slot goes from weir flow to orifice flow).

Check orifice equation using cross-sectional area of opening

- $Q = CA(2gh)^{0.5}$, for h = 1.0' (For orifice equation, h is from midpoint of slot)
- $A = 4\ (8.5')\ (2') = 68.0 \text{ ft}^2$
- $Q = 0.6\ (68.0\text{ft}^2)\ [(64.4)(1.0)]^{0.5} = 327 \text{ cfs} > 266 \text{ cfs}$, so use weir equation

$Q_{100} = (3.1)\ (34')\ H^{3/2}$, $Q_{100} = (105.8)\ H^{3/2}$, where H = wsel – 929.0

Size barrel to release approximately 266 cfs at elevation 930.9

- Check inlet condition: (use *Section 3.3 of the Hydraulic Technical Manual*)
  - H_w = 930.9-919.5 = 11.4 ft
  - Try 60" diameter RCP, Using *Figure 1.19a of the Hydraulics Technical Manual* with entrance condition 1
  - H_w / D = 11.4/5 = 2.28, Discharge = 280 cfs
- Check outlet condition:
  - $Q = a\,[(2gH)/(1+k_m+k_pL)]^{0.5}$

    where:

    Q = discharge in cfs

    a = pipe cross sectional area in ft^2

    g = acceleration of gravity in ft/sec^2

    H = head differential (wsel - downstream centerline of pipe or tailwater elev.)

    k_m = coefficient of minor losses (use 1.0)

    k_p = pipe friction loss coef. (= $5087n^2/d^{4/3}$, d in ", n is Manning's n)

    L = pipe length in ft
  - H = 930.9 - (919.0 + 2.5) = 9.4'
  - for 60" RCP, 70 feet long:
  - $Q = 19.63\,[(64.4)\,(9.4)\,/\,1+1+(.003)\,(70))]^{0.5}$ = 324.9 cfs
  - 280 cfs < 325 cfs, so barrel is inlet controlled.

Note: Pipe will control flow before high stage inlet reaches max head.

Complete stage-storage-discharge summary (Figure 29.7) up to preliminary 100-year wsel (930.9) and route 100 year post-developed condition inflow using computer software.

Elevation MSL	Storage ac-ft	Flow Flow WQv-ED 3.7" eq dia		Riser						Barrel				Emergency Spillway		Total Discharge
				SPv-ED 4.7" eq dia		High Stage Slot				Inlet		Pipe				
						Orifice		Weir								
		H ft	Q cfs	H ft	Q cfs	H ft	Q cfs	H ft	Q cfs	H ft	Q cfs	H ft	Q cfs	H ft	Q cfs	Q cfs
924.0	0.00	0	0													0
925.0	0.35	0.8	0.33													0.33
925.5	0.54	1.3	0.42													0.42
926.0	0.73	1.8	0.49	0	0											0.49
926.5	0.97	2.3	0.55	0.3	0.31											0.86
927.0	1.26	2.8	0.61	0.8	0.50											1.11
927.5	1.61	3.3	0.66	1.3	0.63											1.29
928.0	2.02	3.8	0.71	1.8	0.74											1.45
928.5	2.46	4.3	0.75	2.3	0.84											1.59
929.0	2.93	4.8	0.79	2.8	0.92	N/A		0.0	0.0							1.71
929.5	3.42	5.3	0.83	3.3	1.00			0.5	37.4							39.2
930.0	3.94	5.8	0.87	3.8	1.07			1.0	105.8							107.7
930.5	4.48	6.3	0.91	4.3	1.14			1.5	194.4							196.5
930.9	4.93	6.7	0.93	4.7	1.19			1.9	277.1	11.4	280.0	9.4	324.9			279.2
931.0	5.04	-	-	-	-	1.0	327.0	2.0	299.2	11.5	280.0	9.5	326.6	0.0	0.0	280.0
931.5	5.64	-	-	-	-					12.0	285.0	10.0	335.1	0.5	24.0	309.0
932.0	6.27	-	-	-	-					13.0	290.0	10.5	343.4	1.0	79.0	369.0
932.5	6.93	-	-	-	-					13.5	300.0	11.0	351.4	1.5	154.0	454.0

Figure 29.7 Stage-Storage-Discharge Summary

Note: Adequate outfall protection must be provided in the form of a riprap channel, plunge pool, or combination to ensure non-erosive velocities. Plans must indicate pipe class, joint type, and bedding.

Step 9: Design embankment(s) and spillway(s)

Set the emergency spillway at elevation 931.0 and use design information and criteria Earth Spillways (not included in this manual)

- Q_{100} inflow = 402 cfs.
- Try 34' wide vegetated emergency spillway with 3:1 side slopes.
  - @ elevation 932.6, H = 1.5', Emergency spillway, Q_{ES} = 154 cfs. Primary spillway, Q_{PS} . 300 cfs
  - Q_{ES} + Q_{PS} = 454 cfs, will be able to safely convey Q_f = 402. (use computer routing for exact elevations and discharges).
  - 100 year wsel = 932.2, say 932.5, so set top of embankment with 1 foot of freeboard at elevation 933.5.

Step 10: Investigate potential pond hazard classification

Refer to Texas Commission on Environmental Quality (TCEQ) Dam Safety Program to establish preliminary classification of embankment and whether special design criteria need to be met. Their regulations apply for the construction of dams that are six feet or more in height.

Check pond classification: Height = 931 -919 = 12', equals assumed embankment height, Pond will remain Category II or lower.

As reported in Table 29.1, the preliminary maximum storage volume required is about 4.24 acre-feet. Therefore, for initial design considerations, no additional dam safety requirements will apply. Once final design elevations and storage volumes have been determined, a final check for dam rules exemption should be made by the designer.

Step 11: Design inlets, sediment forebay(s), outlet structures, maintenance access, and safety features.

Table 29.2 Summary of Controls Provided

Control Element	Type/Size of Control	Storage Provided	Elevation	Discharge	Remarks
Units		*Acre-feet*	*MSL*	*cfs*	
Permanent Pool		0.86	924.0	0	part of WQ_v
Forebay	submerged berm	0.12	924.0	0	included in permanent pool volume
Water Quality Extended Detention (WQ_v-ED)	4" pipe, sized to 3.7" equivalent diameter	0.72	926.0	0.36	part of WQ_v above perm. pool, discharge is average release rate over 24 hours
Streambank Protection (SP_v-ED)	6" pipe sized to 5.0" equivalent diameter	2.66	929.0	1.34	volume above perm. pool, discharge is average release rate over 24 hours
Downstream Flood Protection (Q_p)	Four 8.5' x 2' slots on a 10' x 8' riser, 60" barrel.	4.88	931.0	280	volume above perm. pool
Extreme Flood Protection ($Q_{f\text{-}100}$)	34' wide earth spillway	6.53	932.2	207	volume above perm. pool

See Figure 29.8 for profile through principal spillway of the facility.

See Figure 29.9 for a schematic of the riser.

Step 12: Prepare Vegetation and Landscaping Plan

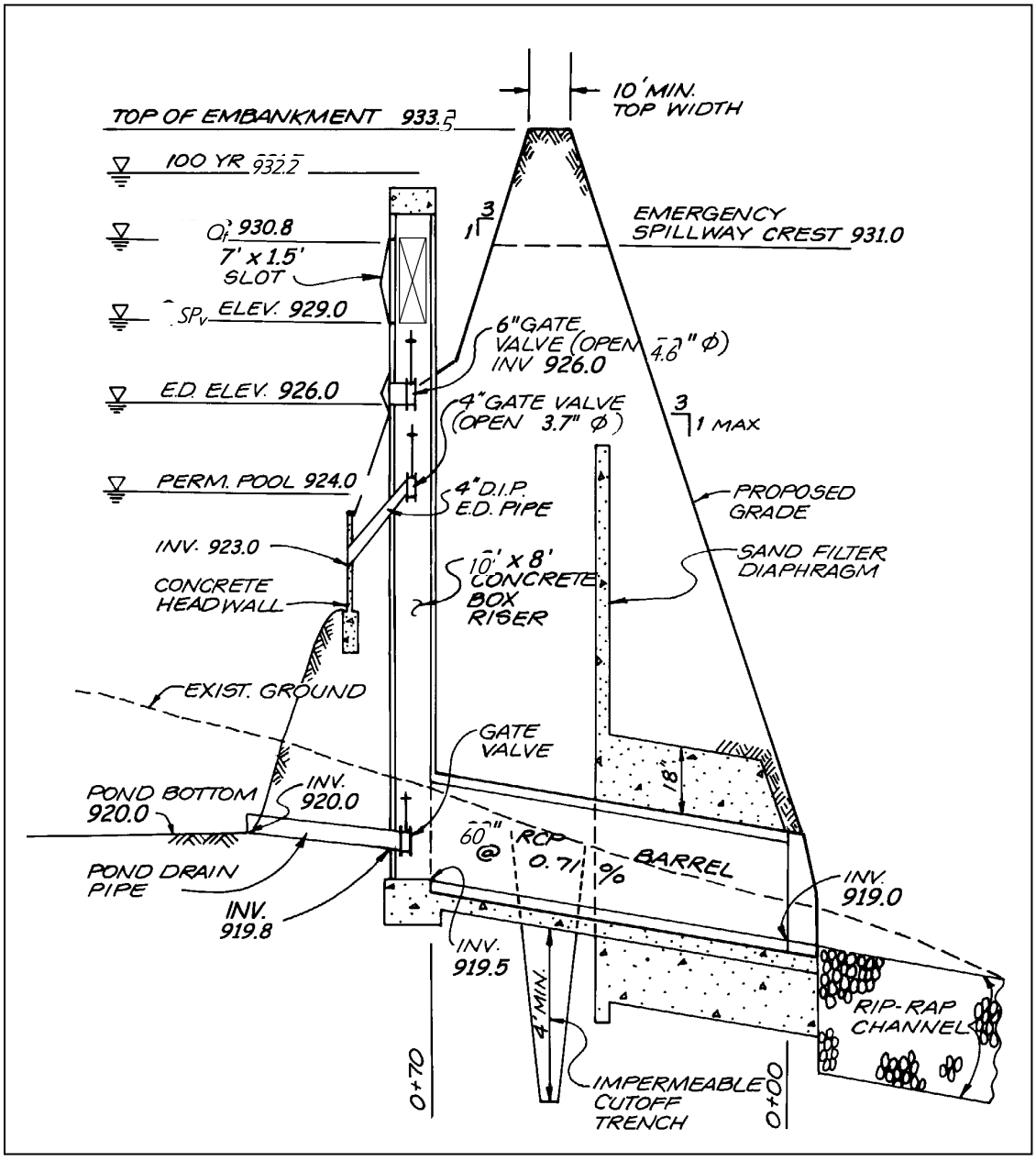


Figure 29.8 Profile of Principal Spillway

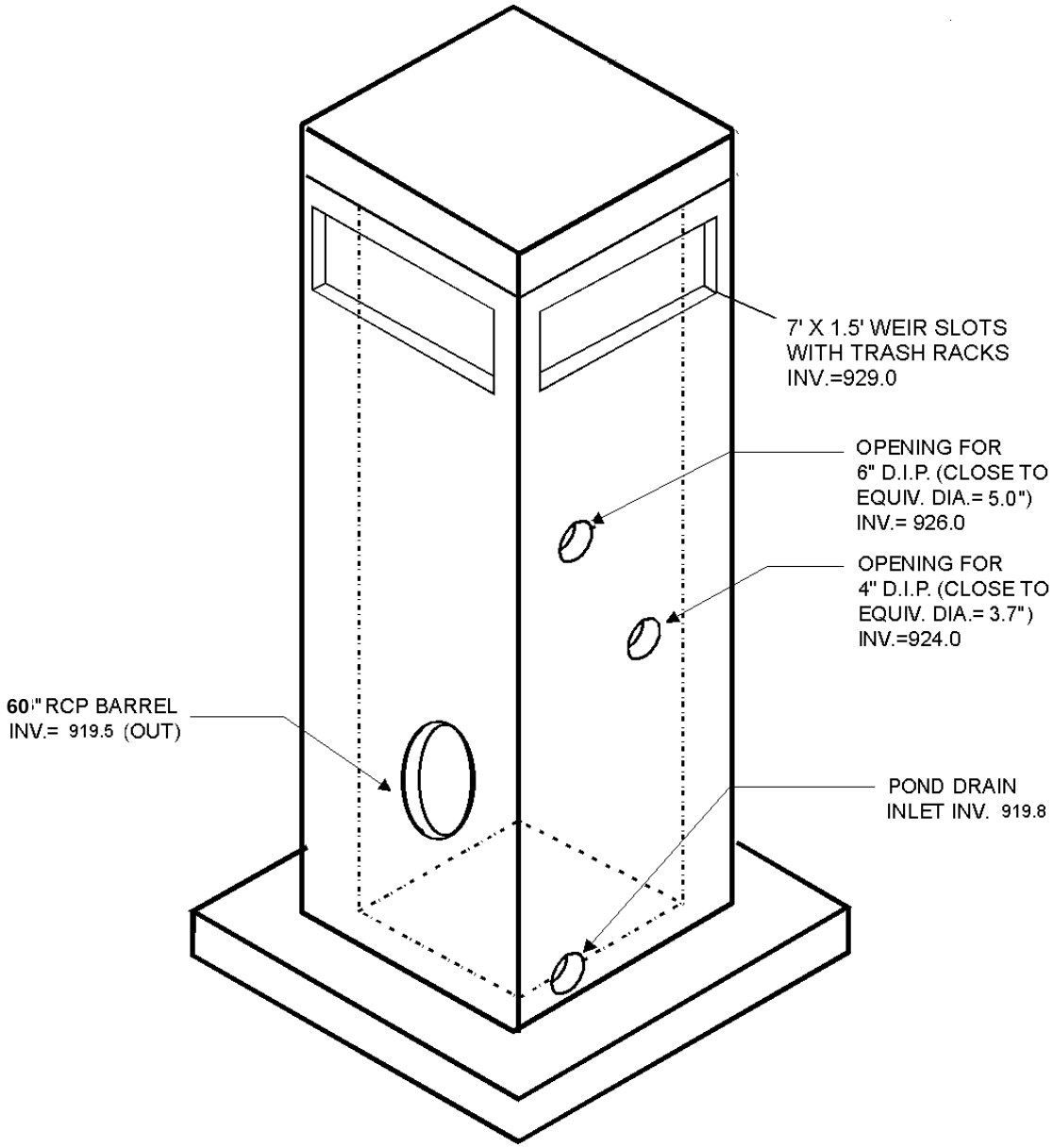


Figure 29.9 Schematic of Riser Detail

29.3 Bioretention Area Design Example

This example focuses on the design of a bioretention facility to meet the water quality treatment requirements of the Wellington Recreation Center. The design options chosen for this example are Option 1 for Water Quality Protection (WQ_v), Option 4 for Streambank Protection (SP_v), and Option 4 for Flood Control (Q_{p100}). Streambank Protection and Flood Control are not addressed in this example other than quantification of preliminary storage volume and peak discharge requirements. It is assumed that the designer can refer to the previous pond example in order to extrapolate the necessary information to determine and design the required storage and outlet structures to meet these criteria. In general, the primary function of bioretention is to provide water quality treatment and not large storm attenuation. As such, flows in excess of the water quality volume are typically routed to bypass the facility or pass through the facility. Where quantity control is required, the bypassed flows can be routed to conventional detention basins (or some other facility such as underground storage vaults). Under some conditions, Streambank Protection storage can be provided by bioretention facilities. The layout of the Wellington Recreation Center is shown in Figure 29.10.

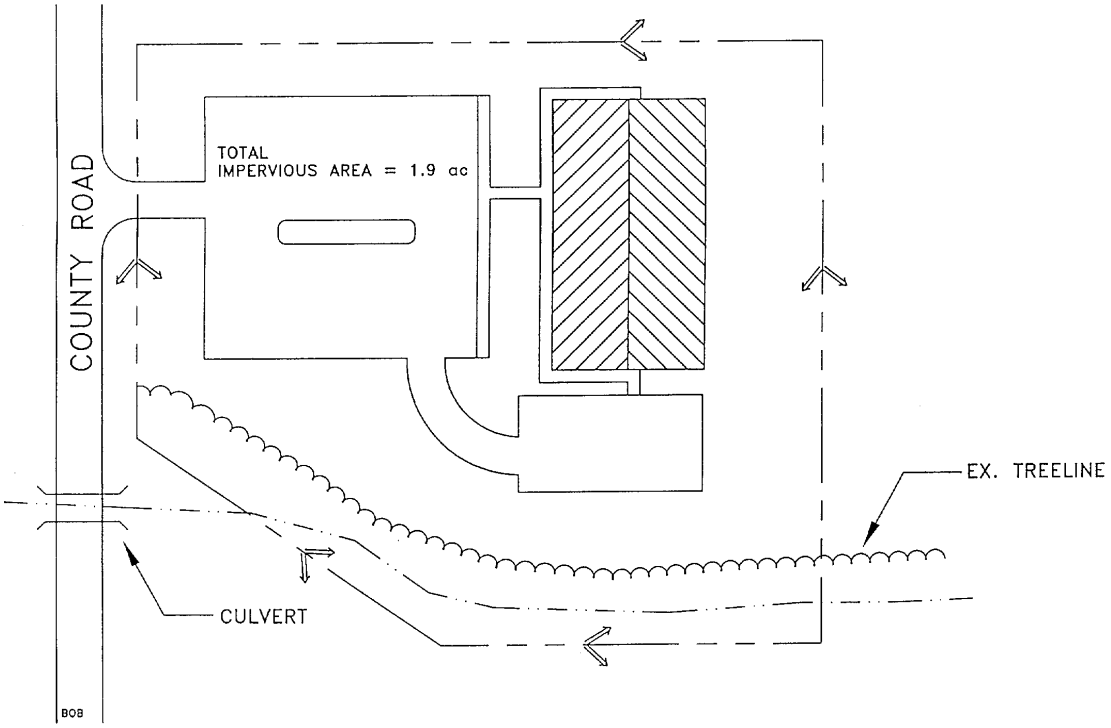


Figure 29.10 Wellington Recreation Center Site Plan

Base Data	Hydrologic Data
Site Area = Total Drainage Area (A) = 3.0 ac Measured Impervious Area = 1.9 ac; or I =1.9/3.0 = 63.3% Soils Type "D" Collin County	Pre / Post CN 77 / 91 t_c 0.41 hr / 0.20 hr

Computation of Preliminary Stormwater Storage Volumes and Peak Discharges

Step 1: Compute runoff control volumes from the integrated Design Focus Areas

Compute Water Quality Volume (WQ_v):

- Compute Runoff Coefficient, R_v

  $R_v = 0.05 + (63.3)\ (0.009) = 0.62$

- Compute WQ_v

  $WQ_v = (1.5")\ (R_v)\ (A) / 12$

  $= (1.5")\ (0.62)\ (3.0ac)\ (43{,}560ft^2/ac)\ (1ft/12in)$

  $= 8{,}102\ ft^3$

Compute Stream Streambank Protection Volume (SP_v):

For stream streambank protection, provide 24 hours of extended detention for the 1-year event.

In order to determine a preliminary estimate of storage volume for streambank protection and flood control, it will be necessary to perform hydrologic calculations using approved methodologies. This example uses the NRCS TR-55 methodology presented in *Section 1.1 of the Hydrology Technical Manual* to determine pre- and post-development peak discharges for the 1-yr, and 100-yr 24-hour return frequency storms.

- Per attached TR-55 calculations (Figures 29.11 and 29.12)

Condition	CN	$Q_{1\text{-year}}$	$Q_{1\text{-year}}$	$Q_{100\ year}$
		Inches	*cfs*	*cfs*
Pre-developed	77	0.83	2.1	21
Post-Developed	91	1.79	6.7	37

- Utilize modified TR-55 approach to compute streambank protection storage volume

  Initial abstraction (I_a) for CN of 91 is 0.27: [$I_a = (200/CN - 2)$]

  $I_a/P = (0.198)/ 2.64$ inches $= 0.075$

  $T_c = 0.20$ hours

  $q_u = 820$ csm/in

  Knowing q_u and T (extended detention time), find q_o/q_i for a Type II rainfall distribution.

  Peak outflow discharge/peak inflow discharge (q_o/q_i) = 0.022

  For a Type II rainfall distribution,

  $V_s/V_r = 0.683 - 1.43(q_o/q_i) + 1.64(q_o/q_i)^2 - 0.804(q_o/q_i)^3$

  Where Vs equals streambank protection storage (SP_v) and V_r equals the volume of runoff in inches.

$V_s/V_r = 0.65$

Therefore, $V_s = SP_v = 0.65(1.74")(1/12)(3\ ac) = 0.28\ ac\text{-}ft = 12{,}317\ ft^3$

Analyze for Safe Passage of 100 Year Design Storm (Q_f):

At final design, prove that discharge conveyance channel is adequate to convey the 100-year event and discharge to receiving waters, or handle it with a peak flow control structure, typically the same one used for the 25 year storm flood protection control.

Table 29.3 Summary of General Design Information for Wellington Recreation Center

Symbol	Control Volume	Volume Required (cubic feet)	Notes
WQ_v	Water Quality	8,102	
SP_v	Streambank Protection	12,317	
Q_f	Extreme Flood Protection	NA	Provide safe passage for the 100-year event in final design

PEAK DISCHARGE SUMMARY				
JOB:	**Wellington Recreation Center**			**EWB**
DRAINAGE AREA NAME:	**Pre-Developed Conditions**			
COVER DESCRIPTION	**SOIL GROUP A, B, C, D?**	**C from TABLE 1.6 Hydrology Section**	**CN from TABLE 1.9 Hydrology Section**	**AREA (in acres)**
woods (good cond.)	D		77	3.0
			AREA SUBTOTALS:	**3.0**
Time of Concentration	**Surface Cover**	**Manning 'n'**	**Flow Length**	**Slope**
2-Yr 24 Hr Rainfall = 3.6"	**Cross Section**	**Wetted Per**	**Avg Velocity**	**Tt (hrs)**
Sheet Flow	**dense grass**	**'n'= 0.24**	**150 ft**	**1.50%**
				0.35 hrs
Shallow Flow	**unpaved**		**500 ft**	**2.00%**
			2.28 fps	**0.06 hrs**
Channel Flow				
Total Area in Acres =	3.00	Total Sheet Flow = 0.35 hrs.	Total Shallow Flow = 0.06 hrs.	Total Channel Flow = 0.00 hrs.
Weighted CN =	77			
Time of Concentration =	0.41 hrs			
Pond Factor =	1	RAINFALL TYPE II		

STORM	Precipitation (P) inches	Runoff (Q)	Qp, PEAK DISCHARGE (cfs)
1 Year	2.64	0.83	2.1
2 Year	3.60	1.51	4.0
5 Year	5.04	2.66	7.2
10 Year	6.00	3.48	9.6
25 Year	7.20	4.55	13.0
50 Year	8.40	5.64	16.0
100 Year	9.84	6.99	21.0

Figure 29.11 Wellington Recreation Center Pre-Developed Conditions

PEAK DISCHARGE SUMMARY				
JOB:	**Wellington Recreation Center**			**EWB**
DRAINAGE AREA NAME:	**Post-Development Conditions**			**3-Jan-00**
COVER DESCRIPTION	**SOIL GROUP A, B, C, D?**	**C from TABLE 1.6 Hydrology Section**	**CN from TABLE 1.9 Hydrology Section**	**AREA (in acres)**
open space (good cond.)	D		80	0.50
woods (good cond.)	D		70	0.60
impervious area	D		98	1.90
			AREA SUBTOTALS:	**38.00**
Time of Concentration	**Surface Cover**	**Manning 'n'**	**Flow Length**	**Slope**
2-Yr 24 Hr Rainfall = 3.6"	**Cross Section**	**Wetted Per**	**Avg Velocity**	**Tt (hrs)**
Sheet Flow	**dense grass**	**'n'= 0.24**	**50 ft**	**1.50%**
				0.14 hrs
Shallow Flow	**paved**		**600 ft**	**2.00%**
			2.87 fps	**0.06 hrs**
Channel Flow		**'n'=0.024**	**50 ft**	**2.00%**
Hydraulic Radius	X-S estimated	WP estimated	**7.25 fps**	**0.00 hrs**
Total Area in Acres =	3.00	Total Sheet Flow =	Total Shallow Flow =	Total Channel Flow =
Weighted CN =	91			
Time of Concentration =	0.20 hrs	0.14 hrs.	0.06 hrs.	0.00 hrs.
Pond Factor =	1	RAINFALL TYPE II		

STORM	Precipitation (P) inches	Runoff (Q)	Qp, PEAK DISCHARGE (cfs)
1 Year	2.64	1.74	6.7
2 Year	3.60	2.64	10.4
5 Year	5.04	4.02	16.0
10 Year	6.00	4.96	20.0
25 Year	7.20	6.14	25.0
50 Year	8.40	7.32	31.0
100 Year	9.84	8.75	37.0

Figure 29.12 Wellington Recreation Center Post-Developed Conditions

Step 2: Determine if the development site and conditions are appropriate for the use of a bioretention area.

Site Specific Data:

Existing ground elevation at the facility location is 922.0 feet, mean sea level. Soil boring observations reveal that the seasonally high water table is at 913.0 feet and underlying soil is predominately clay. Adjacent creek invert is at 912.0 feet.

Step 3: Confirm local design criteria and applicability

There are no additional criteria that must be met for this design.

Step 4: Compute WQ_v peak discharge (Q_{wq})

Step 5: Size flow diversion structure, if needed

Bioretention areas can be either on or off-line. On-line facilities are generally sized to receive, but not necessarily treat, the 25-year event. Off-line facilities are designed to receive a more or less exact flow rate through a weir, channel, manhole, "flow splitter", etc. This facility is situated to receive direct runoff from grass areas and parking lot curb openings and piping for the 25-year event (25.0 cfs), and *no special flow diversion structure is incorporated.*

Step 6: Determine size of bioretention ponding / filter area

$A_f = (WQ_v)\ (d_f) / [\ (k)\ (h_f + d_f)\ (t_f)]$

where:

- A_f = surface area of filter bed (ft^2)
- d_f = filter bed depth (ft)
- k = coefficient of permeability of filter media (ft/day)
- h_f = average height of water above filter bed (ft)
- t_f = design filter bed drain time (days) (48 hours is recommended)

A_f = (8,102 ft^3)(5') / [(0.5'/day) (0.25' + 5') (2 days)] (With k = 0.5'/day, h_f = 0.25', t_f = 2 days)

A_f = 7,716 sq ft

Step 7: Set design elevations and dimensions of facility

Assume a roughly 2 to 1 rectangular shape. Given a filter area requirement of 7,716 sq ft, say facility is roughly 65' by 120'. See Figure 29.13. Set top of facility at 921.0 feet, with the berm at 922.0 feet. The facility is 5' deep, which will allow 3' of freeboard over the seasonally high water table. See Figure 29.14 for a typical section of the facility.

Step 8: Design conveyance to facility (off-line systems)

This facility is not designed as an off-line system.

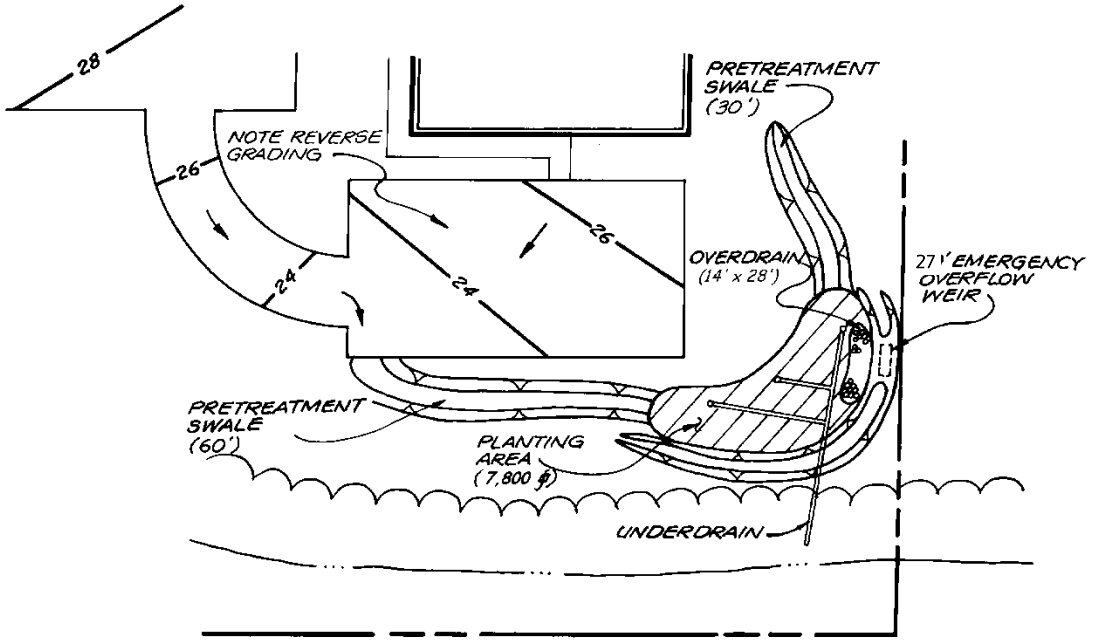


Figure 29.13 Plan View of Bioretention Facility

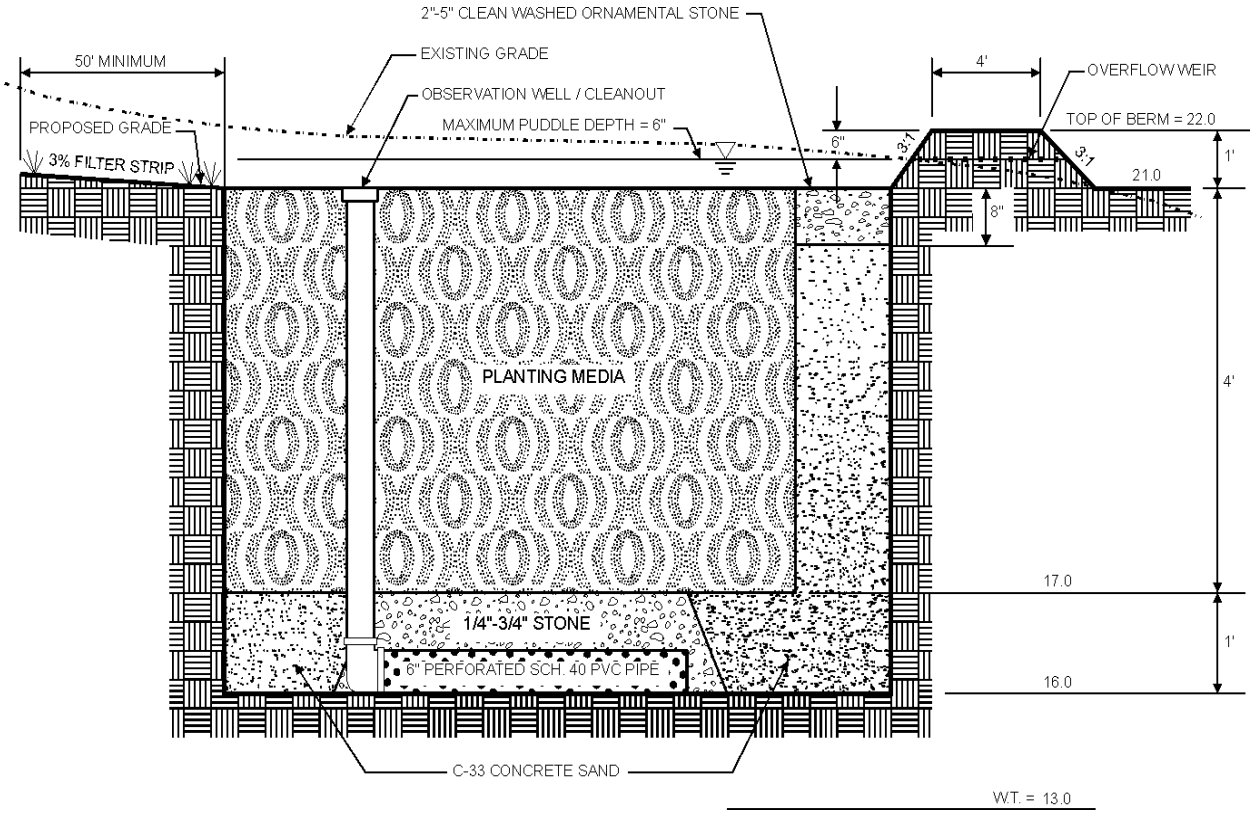


Figure 29.14 Typical Section of Bioretention Facility

Step 9: Design pretreatment

Pretreat with a grass channel, based on guidance provided in Table 29.4, below. For a 3.0 acre drainage area, 63% imperviousness, and slope less than 2.0%, provide a 90' grass channel at 1.5% slope. The value from Table 29.4 is 30' for a one acre drainage area.

Table 29.4 Pretreatment Grass Channel Guidance for 1.0 Acre Drainage Area
(Adapted from Claytor and Schueler, 1996)

Parameter	≤ 33% Impervious		Between 34% & 66% Impervious		≥ 67% Impervious		Notes
Slope	≤2%	≥2%	≤2%	≥2%	≤2%	≥2%	Max slope = 4%
Grassed channel min. length (feet)	25	40	30	45	35	50	Assumes a 2' wide bottom width

Step 10: Size underdrain area

Base underdrain design on 10% of the A_f or 772 sq ft. Using 6" perforated plastic pipes surrounded by a three-foot-wide gravel bed, 10' on center (o.c.). See Figures 29.5 and 29.6.

(772 sq ft)/3' per foot of underdrain = 257', say 260' of perforated underdrain

Step 11: Design emergency overflow

To ensure against the planting media clogging, design a small ornamental stone window of 2" to 5" stone connected directly to the sand filter layer. This area is based on 5% of the A_f or 386 sq ft. Say 14' by 28'. See Figures 29.5 and 29.6.

The parking area, curb and gutter are sized to convey the 25-year event to the facility. Should filtering rates become reduced due to facility age or poor maintenance, an overflow weir is provided to pass the 25-year event. Size this weir with 6" of head, using the weir equation.

$Q = CLH^{3/2}$

Where C = 2.65 (smooth crested grass weir)
Q = 25.0 cfs
H = 6"

Solve for L: $L = Q / [(C) (H^{3/2})]$ or (25.0 cfs) / $[(2.65) (.5)^{1.5}]$ = 26.7' (say 27')

Outlet protection in the form of riprap or a plunge pool/stilling basin should be provided to ensure non-erosive velocities. See Figures 29.5 and 29.6.

Step 12: Prepare Vegetation and Landscaping Plan

Choose plants based on factors such as whether native or not, resistance to drought and inundation, cost, aesthetics, maintenance, etc. Select species locations (i.e., on center planting distances) so species will not "shade out" one another. Do not plant trees and shrubs with extensive root systems near pipe work. A potential plant list is presented in the *Landscape Technical Manual*.

29.4 Sand Filter Design Example

This example focuses on the design of a surface sand filter to meet the water quality treatment requirements of the Falcon Creek Community Center. The design options chosen for this example are Option 1 for Water Quality Protection (WQ_v), Option 4 for Streambank Protection (SP_v), and Option 4 for Flood Control (Q_{p100}). Streambank Protection and Flood Control are not addressed in this example other than quantification of preliminary storage volume and peak discharge requirements. It is assumed that the designer can refer to the previous pond example in order to extrapolate the necessary information to determine and design the required storage and outlet structures to meet these criteria. In general, the primary function of sand filters is to provide water quality treatment and not large storm attenuation. As such, flows in excess of the water quality volume are typically routed to bypass the facility. Where quantity control is required, the bypassed flows can be routed to conventional detention basins (or some other facility such as underground storage vaults). The layout of the Falcon Creek Community Center is shown in Figure 29.15.

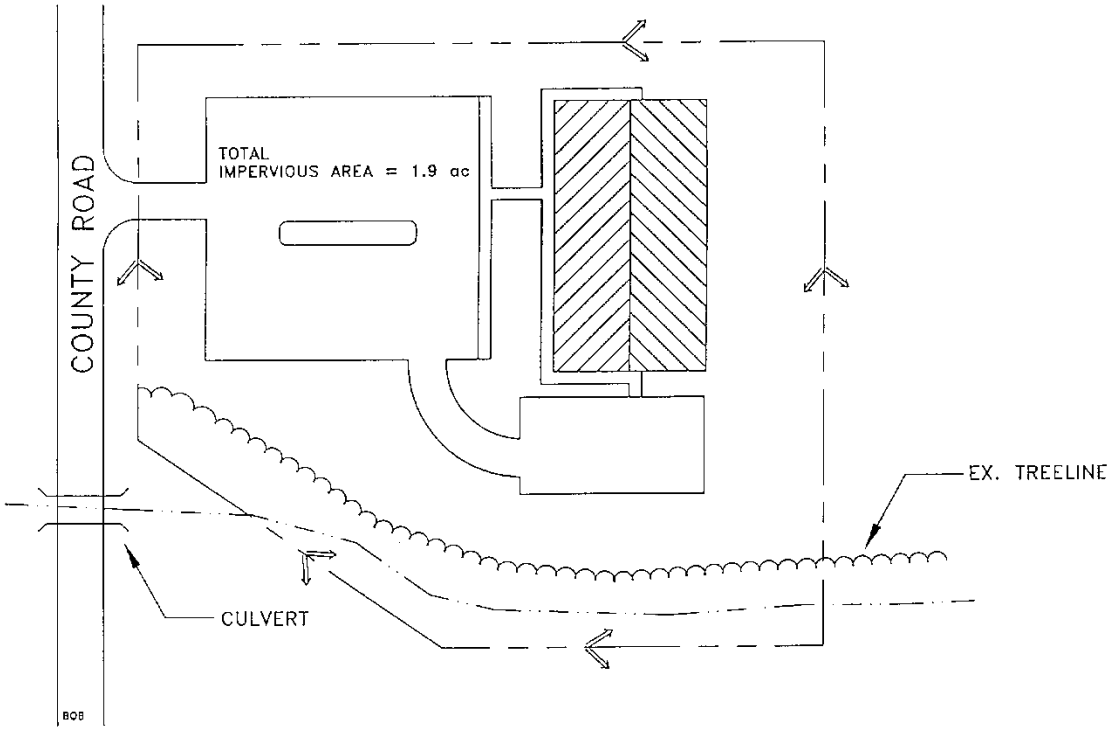


Figure 29.15 Falcon Creek Community Center Site Plan

Base Data		Hydrologic Data		
Site Area = Total Drainage Area (A) = 3.0 ac			Pre	Post
Impervious Area = 1.9 ac; or I =1.9/3.0 = 63.3%		CN	78	91
Soils Type “D”	Dallas County	t_c	0.41hr	0.16 hr

Computation of Preliminary Stormwater Storage Volumes and Peak Discharges

Step 1: Compute runoff control volumes from the *integrated* Design Focus Areas

Compute Water Quality Volume (WQ_v):

- Compute Runoff Coefficient, R_v

  $R_v = 0.05 + (63.3)\ (0.009) = 0.62$

- Compute WQ_v

  WQ_v = (1.5") (R_v) (A) / 12
  = (1.5") (0.62) (3.0 ac) (43,560 ft^2/ac) (1ft/12in)
  = 8,102 ft^3 = 0.186 ac-ft

Compute Stream Streambank Protection Volume, (SP_v):

For stream streambank protection, provide 24 hours of extended detention for the 1-year event.

- **Develop Site Hydrologic and Hydrologic Input Parameters and Perform Preliminary Hydrologic Calculations**

  Per Figures 29.16 and 29.17. Note that any hydrologic models using SCS procedures, such as TR-20, HEC-HMS, or HEC-1, can be used to perform preliminary hydrologic calculations

Condition	CN	$Q_{1\text{-year}}$	$Q_{1\text{-year}}$	$Q_{100\text{ year}}$
		Inches	*cfs*	*cfs*
Pre-developed	78	0.88	2.2	20.0
Post-Developed	91	1.74	7.3	39.0

- Utilize modified TR-55 approach to compute streambank protection storage volume

  Initial abstraction (I_a) for CN of 91 is 0.198: (TR-55) [I_a = (200/CN - 2)]

  Ia/P = (0.198)/ 2.64 inches = 0.075
  T_c = 0.16 hours
  q_u = 900 csm/in

  Knowing q_u and T (extended detention time), find q_o/q_i for a Type II rainfall distribution.

  Peak outflow discharge/peak inflow discharge (q_o/q_i) = 0.02

  $V_s/V_r = 0.683 - 1.43(q_o/q_i) + 1.64(q_o/q_i)^2 - 0.804(q_o/q_i)^3$

  Where Vs equals streambank protection storage (SP_v) and V_r equals the volume of runoff in inches.

  Vs/Vr = 0.655

  Therefore, $V_s = SP_v = 0.655(1.74")(1/12)(3\text{ ac}) = 0.285$ ac-ft = 12,415 ft^3

- Define the average extended detention Release Rate

  The above volume, 0.30 ac-ft, is to be released over 24 hours.
  (0.30 ac-ft × 43,560 ft^2/ac) / (24 hrs × 3,600 sec/hr) = 0.15 cfs

Analyze for Safe Passage of 100 Year Design Storm (Q_f):

At final design, prove that discharge conveyance channel is adequate to convey the 100-year event and discharge to receiving waters, or handle it with a peak flow control structure, typically the same one used for the 25-year storm flood protection control.

Table 29.5 Summary of General Design Information for Falcon Creek Community Center			
Symbol	**Control Volume**	**Volume Required** (cubic feet)	**Notes**
WQ_v	Water Quality	8,102	
SP_v	Streambank Protection	12,415	
Q_f	Extreme Flood Protection	NA	Provide safe passage for the 100-year event in final design

PEAK DISCHARGE SUMMARY				
JOB:	**Falcon Creek Center**			**EWB**
DRAINAGE AREA NAME:	**Pre-Developed Conditions**			**3-Jan-00**
COVER DESCRIPTION	**SOIL GROUP A, B, C, D?**	**C from TABLE 1.6 Hydrology Section**	**CN from TABLE 1.9 Hydrology Section**	**AREA (in acres)**
meadows (good cond.)	D		78	2.40
woods (good cond.)	D		77	0.60
			AREA SUBTOTALS:	**3.00**
Time of Concentration	**Surface Cover**	**Manning 'n'**	**Flow Length**	**Slope**
2-Yr 24 Hr Rainfall = 3.6"	**Cross Section**	**Wetted Per**	**Avg Velocity**	**Tt (hrs)**
Sheet Flow	**Dense grass**	**'n'= 0.24**	**150 ft**	**2.50%**
				0.35 hrs
Shallow Flow	**unpaved**		**500 ft**	**2.00%**
			2.28 fps	**0.06 hrs**
Channel Flow				
Hydraulic Radius				

Total Area in Acres =	3.00	Total Sheet Flow = 0.35 hrs.	Total Shallow Flow = 0.06 hrs.	Total Channel Flow = 0.00 hrs.
Weighted CN =	78			
Time of Concentration =	0.41 hrs			
Pond Factor =	1	RAINFALL TYPE II		

STORM	Precipitation (P) inches	Runoff (Q)	Qp, PEAK DISCHARGE (cfs)
1 Year	2.64	0.88	2.2
2 Year	3.60	1.57	4.2
5 Year	4.8	2.54	6.9
10 Year	5.76	3.37	9.3
25 Year	7.20	4.66	13.0
50 Year	8.40	5.76	17.0
100 Year	9.6	6.98	20.0

Figure 29.16 Falcon Creek Community Center Pre-Developed Conditions

PEAK DISCHARGE SUMMARY				
JOB:	**Falcon Creek Center**			**EWB**
DRAINAGE AREA NAME:	**Post-Developed Conditions**			**3-Jan-00**
COVER DESCRIPTION	**SOIL GROUP A, B, C, D?**	**C from TABLE 1.6 Hydrology Section**	**CN from TABLE 1.9 Hydrology Section**	**AREA (in acres)**
open space (good cond.)	D		78	0.50
woods (good cond.)	D		77	0.60
impervious	D		98	1.90
			AREA SUBTOTALS:	**3.00**
Time of Concentration	**Surface Cover**	**Manning 'n'**	**Flow Length**	**Slope**
2-Yr 24 Hr Rainfall = 3.6"	**Cross Section**	**Wetted Per**	**Avg Velocity**	**Tt (hrs)**
Sheet Flow	**short grass**	**'n'= 0.15**	**50 ft**	**1.50%**
				0.10 hrs
Shallow Flow	**paved**		**600 ft**	**2.00%**
			2.87 fps	**0.06 hrs**
Channel Flow		**'n'= 0.024**	**50 ft**	**2.00%**
Hydraulic Radius= 0.75	X-S estimated	WP estimated	**7.25 fps**	**0.00hrs**
Total Area in Acres =	3.00	Total Sheet Flow = 0.10 hrs.	Total Shallow Flow = 0.06 hrs.	Total Channel Flow = 0.00 hrs.
Weighted CN =	91			
Time of Concentration =	0.16 hrs			
Pond Factor =	1	RAINFALL TYPE II		

STORM	Precipitation (P) inches	Runoff (Q)	Qp, PEAK DISCHARGE (cfs)
1 Year	2.64	1.74	7.3
2 Year	3.60	2.64	11.0
5 Year	4.8	3.79	17.0
10 Year	5.76	4.72	21.0
25 Year	7.20	6.14	27.0
50 Year	8.40	7.32	33.0
100 Year	9.6	8.51	39.0

Figure 29.17 Falcon Creek Community Center Post-Developed Conditions

Step 2: Determine if the development site and conditions are appropriate for the use of a surface sand filter.

Site Specific Data:

Existing ground elevation at the facililty location is 22.0 feet, mean sea level. Soil boring observations reveal that the seasonally high water table is at 13.0 feet. Adjacent creek invert is at 12.0.

Step 3: Confirm local design criteria and applicability.

There are no additional requirements for this site.

Step 4: Compute WQ_v peak discharge (Q_{wq}) & Head

- Water Quality Volume:

  WQ_v previously determined to be 8,102 cubic feet.

- Determine available head (See Figure 29.18)

  Low point at parking lot is 23.5. Subtract 2' to pass Q_{100} discharge (39) and a half foot for channel to facility (21.0). Low point at stream invert is 12.0. Set outfall underdrain pipe 2' above stream invert and add 0.5' to this value for drain (14.5). Add to this value 8" for the gravel blanket over the underdrains, and 18" for the sand bed (16.67). The total available head is 21.0 - 16.67 or 4.33 feet. Therefore, the average depth, h_f, is (h_f) = 4.33' / 2, and h_f = 2.17'.

  The peak rate of discharge for the water quality design storm is needed for the sizing of off-line diversion structures, such as sand filters and grass channels. Conventional SCS methods have been found to underestimate the volume and rate of runoff for rainfall events less than 2". This discrepancy in estimating runoff and discharge rates can lead to situations where a significant amount of runoff by-passes the filtering treatment practice due to an inadequately sized diversion structure and leads to the design of undersized bypass channels.

  The following procedure can be used to estimate peak discharges for small storm events. It relies on the volume of runoff computed using the Small Storm Hydrology Method (Pitt, 1994) and utilizes the NRCS, TR-55 Graphical Peak Discharge Method (USDA, 1986). A brief description of the calculation procedure is presented below.

- Using the water quality volume (WQ_v), a corresponding Curve Number (CN) is computed utilizing *Equation 1.8 of the Hydrology Technical Manual*:

  $$\mathbf{CN = 1000/[10 + 5P + 10Q - 10(Q^2 + 1.25\ QP)^{½}]}$$

  where P = rainfall, in inches (use 1.5" for the Water Quality Storm)
  and Q = runoff volume, in inches (equal to $WQ_V \div$ area)

- Once a CN is computed, the time of concentration (t_c) is computed

- Using the computed CN, t_c and drainage area (A), in acres; the peak discharge (Q_{wq}) for the Water Quality Storm is computed (based on the procedures identified in *Section 1.4 of the Water Quality Technical Manual* (typically Type II in the North Central Texas region).

- Read initial abstraction (I_a), compute I_a/P

- Read the unit peak discharge (q_u) for appropriate t_c

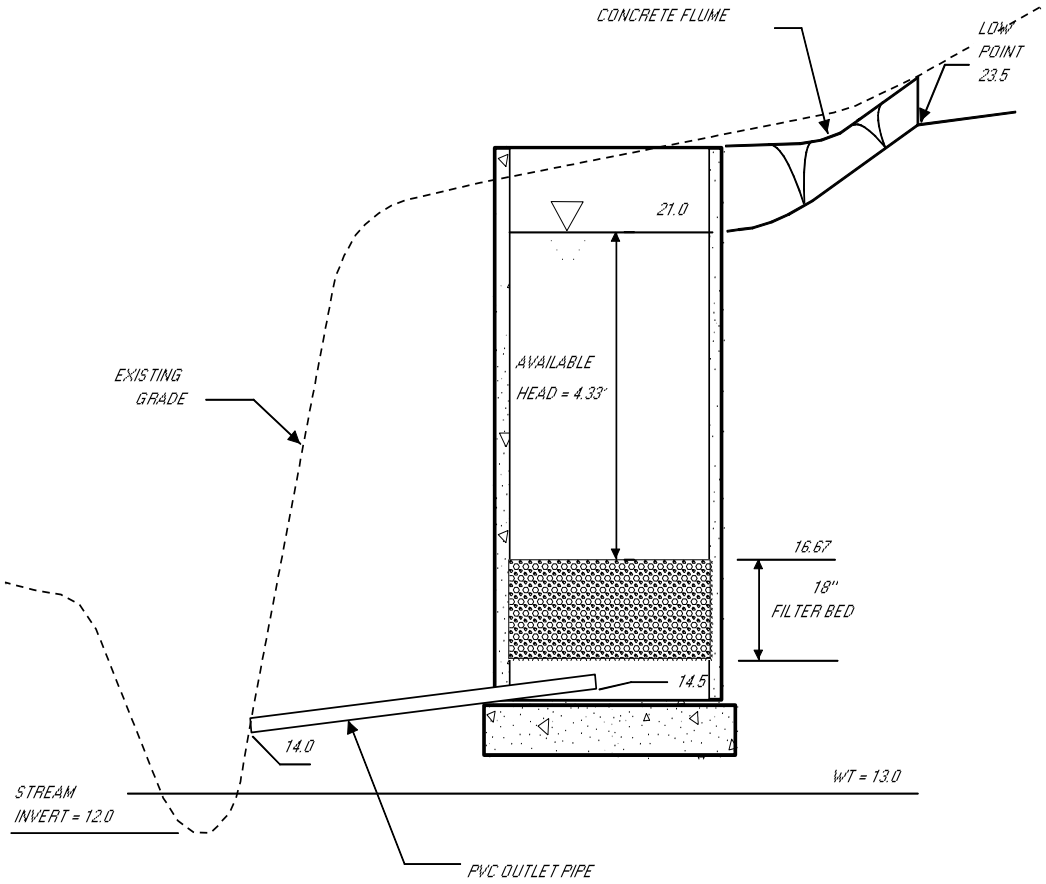


Figure 29.18 Available Head Diagram

- Using the water quality volume (WQ_v), compute the water quality peak discharge (Q_{wq})

$$\mathbf{Q_{wq} = q_u * A * WQ_V} \qquad \textbf{(29.2)}$$

where

Q_{wq} = the peak discharge, in cfs

q_u = the unit peak discharge, in cfs/mi²/inch

A = drainage area, in square miles

WQ_v = Water Quality Volume, in watershed inches

For this example, the steps are as follows:

Compute modified CN for 1.5" rainfall

P = 1.5"

Q = $WQ_v \div$ area = (8,102 ft^3 ÷ 3 ac ÷ 43,560 ft^2/ac × 12 in/ft) = 0.74"

CN = $1000/[10+5P+10Q-10(Q^2+1.25*Q*P)^{½}]$

= $1000/[10+5*1.5+10*0.74-10(0.74^2+1.25*0.74*1.5)^{½}]$

= 95.01

Use CN = 95

For CN = 95 and the T_c = 0.16 hours, compute the Q_p for a 1.5" storm. With the CN = 95, a 1.5" storm will produce 0.74" of runoff. I_a = 0.105, therefore I_a/P = 0.105/1.5 = 0.088. From *Section 1.0 of the Hydrology*

Technical Manual, q_u = 625 csm/in, and therefore Q_{wq} = (900 csm/in) (3.0 ac/640ac/sq mi.) (0.74") = 3.1 cfs.

Step 5: Size flow diversion structure (see Figure 29.19):

Size a low flow orifice to pass 3.1 cfs with 1.5' of head using the Orifice equation.

$Q = CA(2gh)^{1/2}$; 3.1 cfs = (0.6) (A) $[(2) (32.2 \text{ ft/s}^2) (1.5')]^{1/2}$

A = 0.53 sq ft = $\pi d^2/4$: d = 0.8' or 9.8"; use 10 inches

Size the 100-year overflow as follows: the 100-year wsel is set at 23.0. Use a concrete weir to pass the 100-year flow (39.0 cfs) into a grassed overflow channel using the Weir equation. Assume 2' of head to pass this event. Overflow channel should be designed to provide sufficient energy dissipation (e.g., riprap, plunge pool, etc.) so that there will be non-erosive velocities.

$Q = CLH^{3/2}$

$39 = 3.1 (L) (2')^{1.5}$

L = 4.45'; use L = 4'-5" which sets flow diversion chamber dimension.

Weir wall elev. = 21.0. Set low flow invert at 21.0 - [1.5' + (0.5*10"*1ft/12")] = 19.08.

Step 6: Size filtration bed chamber (see Figure 29.20):

From Darcy's Law: $A_f = WQ_v (d_f) / [k (h_f + d_f) (t_f)]$

where d_f = 18"
k = 3.5 ft/day
h_f = 2.17'
t_f = 40 hours

A_f = (8,102 cubic feet) (1.5') / [3.5 (2.17' + 1.5') (40hr/(24hr/day))]

A_f = 567.7 sq ft; using a 2:1 ratio, say filter is 17' by 34' (= 578 sq ft)

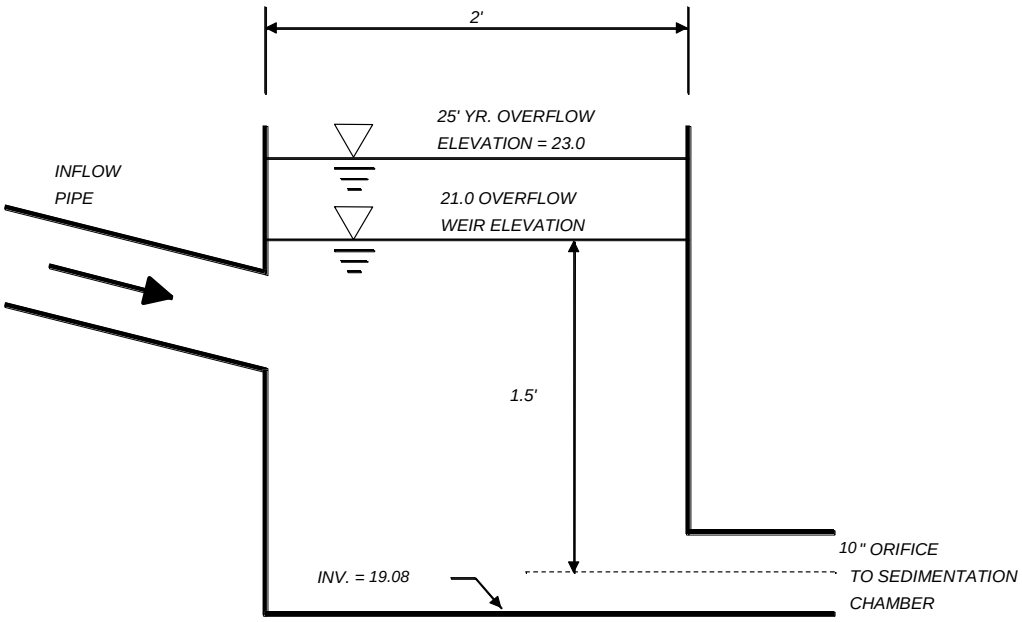


Figure 29.19 Flow Diversion Structure

Step 7: Size sedimentation chamber

From Camp-Hazen equation, for I < 75%: $A_s = 0.066 (WQ_v)$

A_s = 0.066 (8,102 cubic ft) or 535 sq ft

given a width of 17 feet, the length will be 535'/17' or 31.5 feet (use 17' x 32')

Step 8: Compute V_{min}

V_{min} = ¾(WQ_v) or 0.75 (8,102 cubic feet) = 6,077 cubic feet

Step 9: Compute storage volumes within entire facility and sedimentation chamber orifice size:

Volume within filter bed (V_f): $V_f = A_f (d_f) (n)$; n = 0.4 for sand
V_f = (578 sq ft) (1.5') (0.4) = 347 cubic feet

Temporary storage above filter bed ($V_{f\text{-}temp}$): $V_{f\text{-}temp} = 2\ h_f\ A_f$
$V_{f\text{-}temp}$ = 2 (2.17') (578 sq ft) = 2,509 cubic feet

Compute remaining volume for sedimentation chamber (V_s):
$V_s = V_{min} - [V_f + V_{f\text{-}temp}]$ or 6,077 - [347 + 2,509] = 3,221 cubic feet

Compute height in sedimentation chamber (h_s): $h_s = V_s/A_s$

(3,221 cubic ft)/(17' x 32') = 5.9' which is larger than the head available (4.33'); increase the size of the settling chamber, using 4.33' as the design height;

(3,221 cubic ft)/4.33' = 744 sq ft; 744'/17' yields a length of 43.8 feet (say 44')

New sedimentation chamber dimensions are 17' by 44'

With adequate preparation of the bottom of the settling chamber (rototil earth, place gravel, then surge stone), the bottom can infiltrate water into the substrate. The runoff will enter the groundwater directly without treatment. The stone will eventually clog without protection from settling solids, so use a removable

geotextile to facilitate maintenance. Note that there is 2.17' of freeboard between bottom of recharge filter and water table.

Provide perforated standpipe with orifice sized to release volume (within sedimentation basin) over a 24 hr period (see Figure 29.21). Average release rate equals 3,221 ft^3/24 hr = 0.04 cfs

Equivalent orifice size can be calculated using orifice equation:

$Q = CA(2gh)^{1/2}$, where h is average head, or 4.33'/2 = 2.17'.

$0.04 \text{ cfs} = 0.6*A*(2*32.2 \text{ ft/s}^2*2.17 \text{ ft})^{1/2}$

$A = 0.005 \text{ ft}^2 = \pi D^2/4$: therefore equivalent orifice diameter equals 1".

Recommended design is to cap stand pipe with low flow orifice sized for 24 hr detention. Over-perforate pipe by a safety factor of 10 to account for clogging. Note that the size and number of perforations will depend on the release rate needed to achieve 24 hr detention. A multiple orifice stage-discharge relation needs to be developed for the proposed perforation configuration. Stand pipe should discharge into a flow distribution chamber prior to filter bed. Distribution chamber should be between 2 and 4 feet in length and same width as filter bed. Flow distribution to the filter bed can be achieved either with a weir or multiple orifices at constant elevation. See Figure 29.9 for stand pipe details.

Step 10: Design inlets, pretreatment facilities, underdrain system, and outlet structures

Step 11: Compute overflow weir sizes

Assume overflow that needs to be handled is equivalent to the 10" orifice discharge under a head of 3.5 ft (i.e., the head in the diversion chamber associated with the 100-year peak discharge).

$Q = CA(2gh)^{½}$

$Q = 0.6(0.55 \text{ ft}^2)[(2)(32.2 \text{ ft/s}^2)(3.5 \text{ ft})]^{½}$

Q = 4.91 cfs, say 5.0 cfs

For the overflow from the sediment chamber to the filter bed, size to pass 5 cfs.

Weir equation: $Q = CLh^{3/2}$, assume a maximum allowable head of 0.5'

$5.0 = 3.1 * L * (0.5 \text{ ft})^{3/2}$

L = 4.56 ft, Use L = 4.75 ft.

Similarly, for the overflow from the filtration chamber to the outlet of the facility, size to pass 5.0 cfs.

Weir equation: $Q = CLh^{3/2}$, assume a maximum allowable head of 0.5'

$5.0 = 3.1 * L * (0.5 \text{ ft})^{3/2}$

L = 4.56 ft, Use L = 4.75 ft.

Adequate outlet protection and energy dissipation (e.g., riprap, plunge pool, etc.) should be provided for the downstream overflow channel.

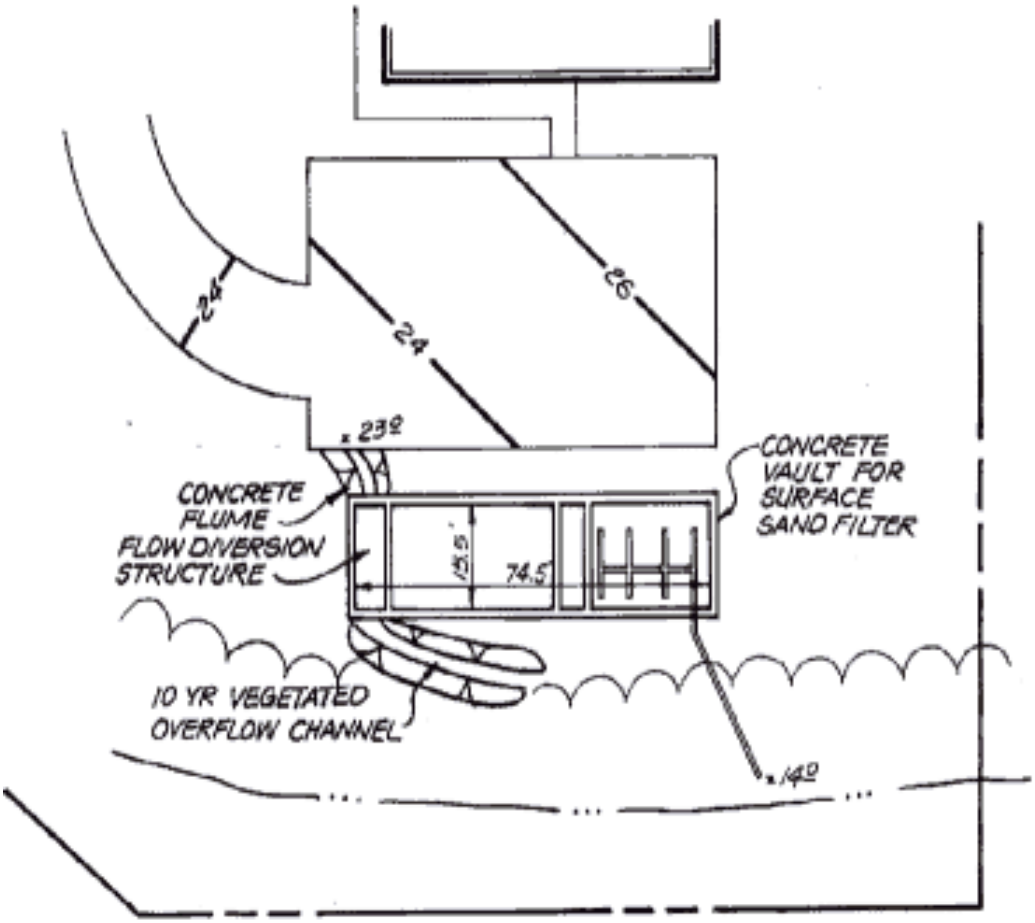


Figure 29.20 Surface Sand Filter Site Plan

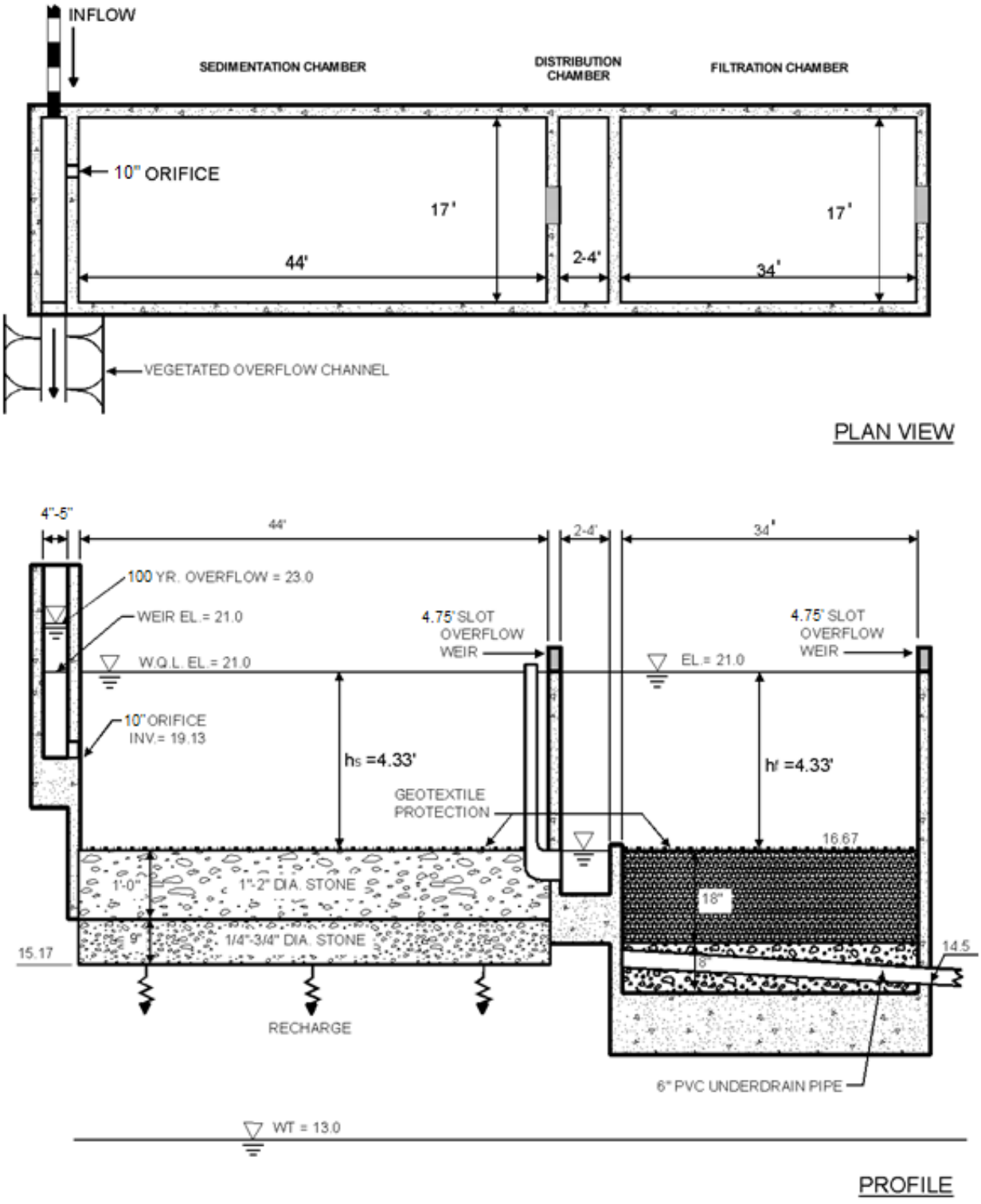


Figure 29.21 Plan and Profile of Surface Sand Filter

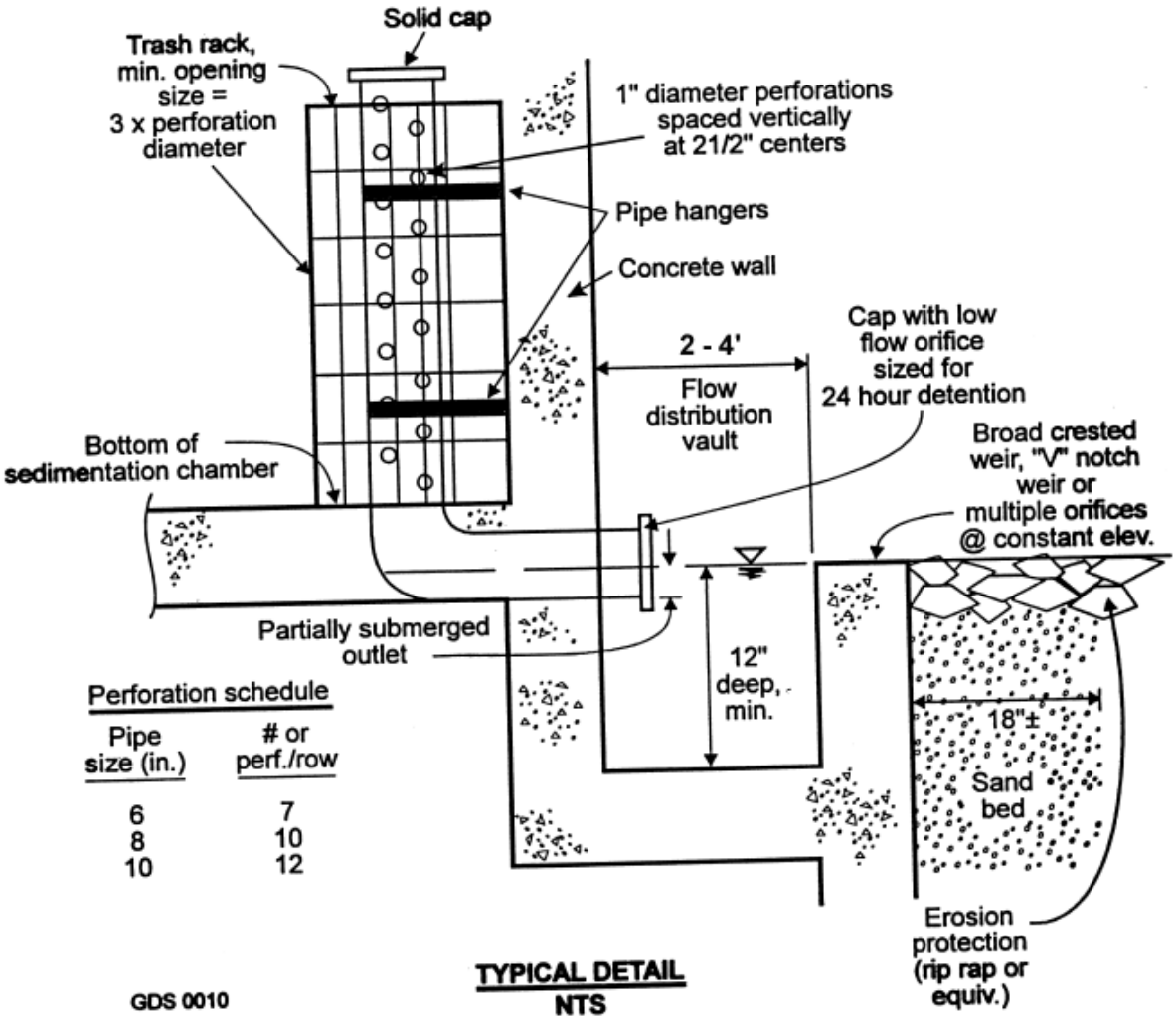


Figure 29.22 Perforated Stand Pipe Detail

29.5 Infiltration Trench Design Example

This example focuses on the design of an infiltration trench to meet the water quality treatment requirements of the site. The design options chosen for this example are Option 1 for Water Quality Protection (WQ_v), Option 4 for Streambank Protection (SP_v), and Option 4 for Flood Control (Q_{p100}). Streambank Protection and Flood Control are not addressed in this example other than quantification of preliminary storage volume and peak discharge requirements. It is assumed that the designer can refer to the previous pond example in order to extrapolate the necessary information to determine and design the required storage and outlet structures to meet these criteria. In general, the primary function of infiltration trenches is to provide water quality treatment and groundwater recharge and not large storm attenuation. As such, flows in excess of the water quality volume are typically routed to bypass the facility. Where quantity control is required, the bypassed flows can be routed to conventional detention basins (or some other facility such as underground storage vaults). The layout of the Cottonwood Creek Community Center is shown in Figure 29.23.

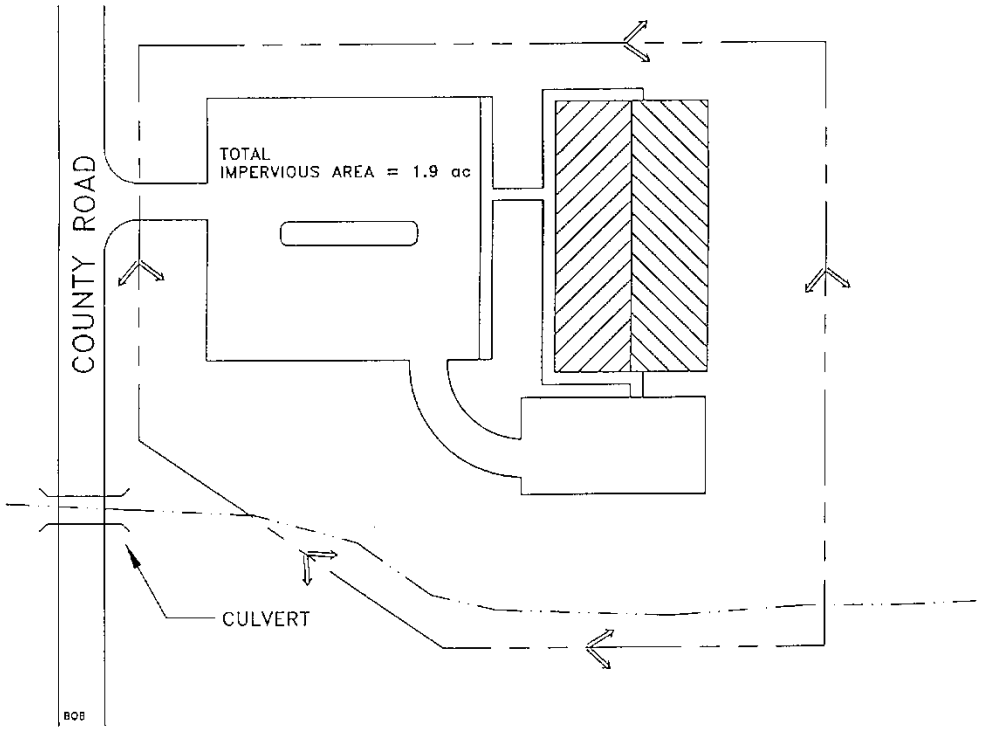


Figure 29.23 Cottonwood Creek Community Center Site Plan

Base Data		Hydrologic Data	
Site Area = Total Drainage Area (A) = 3.0 ac		Pre	Post
Impervious Area = 1.9 ac; or I =1.9/3.0 = 63.3%	CN	55	82
Soils Type “B” Wise County	t_c	0.42 hr	0.16 hr

Step 1: Compute runoff control volumes from the *integrated* Design Focus Areas

Compute Water Quality Volume (WQ_v):

- Compute Runoff Coefficient, R_v

  $R_v = 0.05 + (63.3)\ (0.009) = 0.62$

- Compute WQ_v

  WQ_v = (1.5") (R_v) (A) / 12
  = (1.5") (0.62) (3.0 ac) (43,560 ft^2/ac) (1ft/12in)
  = 8,102 ft^3 = 0.186 ac-ft

Compute Stream Streambank Protection Volume, (SP_v):

For stream streambank protection, provide 24 hours of extended detention for the 1-year event.

- **Develop Site Hydrologic and Hydrologic Input Parameters and Perform Preliminary Hydrologic Calculations**

  Per Figures 29.24 and 29.25. Note that any hydrologic models using SCS procedures, such as TR-20, HEC-HMS, or HEC-1, can be used to perform preliminary hydrologic calculations

Condition	CN	$Q_{1\text{-year}}$	$Q_{1\text{-year}}$	$Q_{100\ year}$
		Inches	*cfs*	*cfs*
Pre-developed	55	0.11	0.1	8.9
Post-Developed	82	1.1	4.4	29.0

- Utilize modified TR-55 approach to compute streambank protection storage volume

  Initial abstraction (I_a) for CN of 82 is 0.44: (TR-55) [I_a = (200/CN - 2)]

  I_a/P = (0.44)/ 2.64 inches = 0.17

  T_c = 0.17 hours

  q_u = 850 csm/in

  Knowing q_u and T (extended detention time), find q_o/q_i for a Type II rainfall distribution.

  Peak outflow discharge/peak inflow discharge (q_o/q_i) = 0.02

  $V_s/V_r = 0.683 - 1.43(q_o/q_i) + 1.64(q_o/q_i)^2 - 0.804(q_o/q_i)^3$

  Where Vs equals streambank protection storage (SP_v) and V_r equals the volume of runoff in inches.

  $V_s/V_r = 0.655$

  Therefore, $V_s = SP_v = 0.655(1.10")(1/12)(3\ ac) = 0.18$ ac-ft = 7,841 ft^3

- Define the average extended detention Release Rate

  The above volume, 0.18 ac-ft, is to be released over 24 hours.
  (0.18 ac-ft × 43,560 ft^2/ac) / (24 hrs × 3,600 sec/hr) = 0.09 cfs

Analyze for Safe Passage of 100 Year Design Storm (Q_f):

At final design, prove that discharge conveyance channel is adequate to convey the 100-year event and discharge to receiving waters, or handle it with a peak flow control structure.

Table 29.6 Summary of General Design Info for Falcon Creek Community Center			
Symbol	**Control Volume**	**Volume Required** (cubic feet)	**Notes**
WQ_v	Water Quality	8,102	
SP_v	Streambank Protection	7,841	
Q_f	Flood Protection	NA	Provide safe passage for the 100-year event in final design

PEAK DISCHARGE SUMMARY				
JOB:	**Cottonwood Creek**			**EWB**
DRAINAGE AREA NAME:	**Pre-Developed Conditions**			**3-Jan-00**
COVER DESCRIPTION	**SOIL GROUP A, B, C, D?**	**C from TABLE 1.6 Hydrology Section**	**CN from TABLE 1.9 Hydrology Section**	**AREA (in acres)**
meadow (good cond.)	B		55	3.00
		AREA SUBTOTALS:		**3.00**
Time of Concentration	**Surface Cover**	**Manning 'n'**	**Flow Length**	**Slope**
2-Yr 24 Hr Rainfall = 3.36"	**Cross Section**	**Wetted Per**	**Avg Velocity**	**Tt (hrs)**
Sheet Flow	**dense grass**	**'n'= 0.24**	**150ft**	**1.50%**
				0.36 hrs
Shallow Flow	**unpaved**		**500 ft**	**2.00%**
			2.28 fps	**0.06 hrs**
Channel Flow				
Total Area in Acres =	**3.00**	Total Sheet Flow = 0.36 hrs.	Total Shallow Flow = 0.06 hrs.	Total Channel Flow = 0.00 hrs.
Weighted CN =	**55**			
Time of Concentration =	**0.42 hrs**			
Pond Factor =	**1**	**RAINFALL TYPE II**		

STORM	Precipitation (P) inches	Runoff (Q)	Qp, PEAK DISCHARGE (cfs)
1 Year	2.64	0.11	0.10
2 Year	3.36	0.30	0.39
5 Year	4.56	0.77	1.5
10 Year	5.28	1.12	2.5
25 Year	6.72	1.95	4.8
50 Year	7.68	2.57	6.5
100 Year	8.88	3.40	8.9

Figure 29.24 Cottonwood Creek Community Center Pre-Developed Conditions

PEAK DISCHARGE SUMMARY				
JOB:	**Cottonwood Creek**			**EWB**
DRAINAGE AREA NAME:	**Post-Developed Conditions**			**3-Jan-00**
COVER DESCRIPTION	**SOIL GROUP A, B, C, D?**	**C from TABLE 1.6 Hydrology Section**	**CN from TABLE 1.9 Hydrology Section**	**AREA (in acres)**
meadow (good cond.)	B		55	1.10
impervious	B		98	1.90
			AREA SUBTOTALS:	**38.00**
Time of Concentration	**Surface Cover**	**Manning 'n'**	**Flow Length**	**Slope**
2-Yr 24 Hr Rainfall = 3.36"	**Cross Section**	**Wetted Per**	**Avg Velocity**	**Tt (hrs)**
Sheet Flow	**short grass**	**'n'= 0.15**	**50 ft**	**1.50%**
				0.10 hrs
Shallow Flow	**paved**		**600 ft**	**2.00%**
			2.88 fps	**0.06 hrs**
Channel Flow		**'n'= 0.24**	**50 ft**	**2.00%**
Hydraulic Radius = 0.75	X-S estimated	**WP estimated**	**7.25 fps**	**0.00 hrs**
Total Area in Acres =	**3.00**	Total Sheet Flow = 0.10 hrs.	Total Shallow Flow = 0.06 hrs.	Total Channel Flow = 0.00 hrs.
Weighted CN =	**82**			
Time of Concentration =	**0.16 hrs**			
Pond Factor =	**1**	**RAINFALL TYPE II**		

STORM	Precipitation (P) inches	Runoff (Q)	Qp, PEAK DISCHARGE (cfs)
1 Year	2.64	1.10	4.4
2 Year	3.36	1.67	6.8
5 Year	4.56	2.69	11
10 Year	5.28	3.33	14
25 Year	6.72	4.65	20
50 Year	7.68	5.56	24
100 Year	8.88	6.70	29

Figure 29.25 Cottonwood Creek Community Center Post-Developed Conditions

Step 2: Determine if the development site and conditions are appropriate for the use of an infiltration trench

Site Specific Data:

Table 29.7 presents site-specific data, such as soil type, percolation rate, and slope, for consideration in the design of the infiltration trench.

Table 29.7 Site Specific Data

Criteria	Value
Soil	Sandy Loam
Percolation Rate	1"/hour
Ground Elevation at BMP	20'
Seasonally High Water Table	13'
Stream Invert	12'
Soil slopes	<1%

Step 3: Confirm local design criteria and applicability

Table 29.8, below, summarizes the requirements that need to be met to successfully implement infiltration practices. On this site, infiltration is feasible, with restrictions on the depth and width of the trench.

Table 29.8 Infiltration Feasibility

Criteria	Status
Infiltration rate (f_c) greater than or equal to 0.5 inches/hour.	Infiltration rate is 1.0 inches/hour. OK.
Soils have a clay content of less than 20% and a silt/clay content of less than 40%.	Sandy Loam meets both criteria.
Infiltration cannot be located on slopes greater than 6% or in fill soils.	Slope is <1%; not fill soils. OK.
Hotspot runoff should not be infiltrated.	Not a hotspot land use. OK.
Infiltration is prohibited in karst topography.	Not in karst. OK.
The bottom of the infiltration facility must be separated by at least two feet vertically from the seasonally high water table.	Elevation of seasonally high water table: 13' Elevation of BMP location: 20'. The difference is 7'. Thus, the trench can be up to 5' deep. OK.
Infiltration facilities must be located 100 feet horizontally from any water supply well.	No water supply wells nearby. OK.
Maximum contributing area generally less than 5 acres. (Optional)	3 acres. OK.
Setback 25 feet down-gradient from structures.	Fifty feet straight-line distance between the parking lot and the tree line. OK if the trench is 25' wide or narrower.

Step 4: Compute WQ_v peak discharge (Q_{wq})

- Compute Water Quality Volume:

  WQ_v previously determined to be 8,102 cubic feet.

The peak rate of discharge for the water quality design storm is needed for the sizing of off-line diversion structures, such as sand filters and grass channels. Conventional SCS methods have been found to underestimate the volume and rate of runoff for rainfall events less than 2". This discrepancy in estimating runoff and discharge rates can lead to situations where a significant amount of runoff by-passes the filtering treatment practice due to an inadequately sized diversion structure or leads to the design of undersized grass channels.

The following procedure can be used to estimate peak discharges for small storm events. It relies on the volume of runoff computed using the Small Storm Hydrology Method (Pitt, 1994) and utilizes the NRCS, TR-55 Graphical Peak Discharge Method (USDA, 1986). A brief description of the calculation procedure is presented below.

$$\mathbf{CN = 1000/[10 + 5P + 10Q - 10(Q^2 + 1.25\,QP)^{½}]}$$

where P = rainfall, in inches (use 1.5" for the Water Quality Storm)
and Q = runoff volume, in inches (equal to $WQ_V \div$ area)

- Once a CN is computed, the time of concentration (t_c) is computed (based on the methods identified in TR-55, Chapter 3: "Time of concentration and travel time").
- Using the computed CN, t_c and drainage area (A), in acres; the peak discharge (Q_{wq}) for the Water Quality Storm is computed (based on the procedures identified in TR-55, Chapter 4: "Graphical Peak Discharge Method"). Use appropriate rainfall distribution type (typically Type II in the North Central Texas region).

  Read initial abstraction (I_a), compute I_a/P

  Read the unit peak discharge (q_u) for appropriate t_c

  Using the water quality volume (WQ_v), compute the peak discharge (Q_{wq})

  $$\mathbf{Q_{wq} = q_u * A * WQ_V}$$

  where

  Q_{wq}= the peak discharge, in cfs
  q_u = the unit peak discharge, in cfs/mi²/inch
  A = drainage area, in square miles
  WQ_v = Water Quality Volume, in watershed inches

For this example, the steps are as follows:

Compute modified CN for 1.5" rainfall

P = 1.5"
Q = $WQ_v \div$ area = (8,102 ft³ ÷ 3 ac ÷ 43,560 ft²/ac × 12 in/ft) = 0.74"

$$CN = 1000/[10 + 5P + 10Q - 10\,(Q^2 + 1.25*Q*P)^{½}]$$
$$= 1000/[10 + 5*1.5 + 10*0.74 - 10(0.74^2 + 1.25*0.74*1.5)^{½}]$$
$$= 91.1$$

Use CN = 91

For CN = 91 and the T_c = 0.16 hours, compute the Q_{wq} for a 1.5" storm. With the CN = 91, a 1.5" storm will produce 0.74" of runoff. I_a = 0.198, therefore I_a/P = 0.198/1.5 = 0.132. q_u = 825 csm/in, and therefore:

Q_{wq} = (825 csm/in) (3.0 ac/640ac/sq mi.) (0.74") = 2.86 cfs.

Step 5: Size the infiltration trench

The area of the trench can be determined by the following equation (Equation 20.1):

$$A = \frac{WQ_v}{(nd + kT/12)}$$

Where:

A = Surface Area
n = porosity
d = trench depth (feet)
k = percolation (inches/hour)
T= Fill Time (time for the practice to fill with water), in hours

Assume that:

n = 0.32
d = 5 feet (see above; feasibility criteria)
k = 1 inch/hour (see above; site data)
T = 2 hours

Therefore:

$A = 8{,}102\ ft^3 / (0.32 \times 5 + 1 \times 2/12)ft$
$A = 4{,}586\ ft^2$

Since the width can be no greater than 25' (see above; feasibility), determine the length:

$L = 4{,}586\ ft^2 / 25\ ft$
L = 183 feet

Assume that 1/3 of the runoff from the site drains to Point A and 2/3 drains to Point B. Use an L-shaped trench in the corner of the site (see Figure 29.4 for a site plan view). The surface area of the trench is proportional to the amount of runoff it drains (e.g., the portion draining from Point A is half as large as the portion draining Point B).

Step 6: Size the flow diversion structures

Since two entrances are used, two flow diversions are needed.

For the entire site:

$Q_{100\text{-year}}$ = 29 cfs (See Figure 29.3)
Peak flow for WQ_v = 2.86 cfs. (Step 3).

For the first diversion (Point A)

Assume peak flow equals 1/3 of the value for the entire site.
Thus, $Q_{100\text{-year}}$ = 29/3 = 9.7 cfs
Peak flow for WQ_v = 2.86/3 = 0.95 cfs

Size the low flow orifice to pass 0.95 cfs with 1.5' of head using the Orifice equation.

$Q=CA(2gh)^{½}$; $0.95\ cfs = 0.6A(2 \times 32.2\ ft/s^2 \times 1.5')^{½}$
$A=0.16$ sq. ft. $= \pi d^2/4$; d = 0.45'; use 6" pipe with 6" gate valve

Size the 100-year overflow weir crest at 22.5'. Use a concrete weir to pass the 100-year flow (9.7 – 0.95 = 8.75 cfs). Assume 1 foot of head to pass this event. Size using the weir equation.

$Q = CLH^{1.5}$; $L= Q/(CH^{1.5})$
$L = 8.75\ cfs/ (3.1)(1)^{1.5} = 2.8'$; use 2.8' (see Figure 29.27)

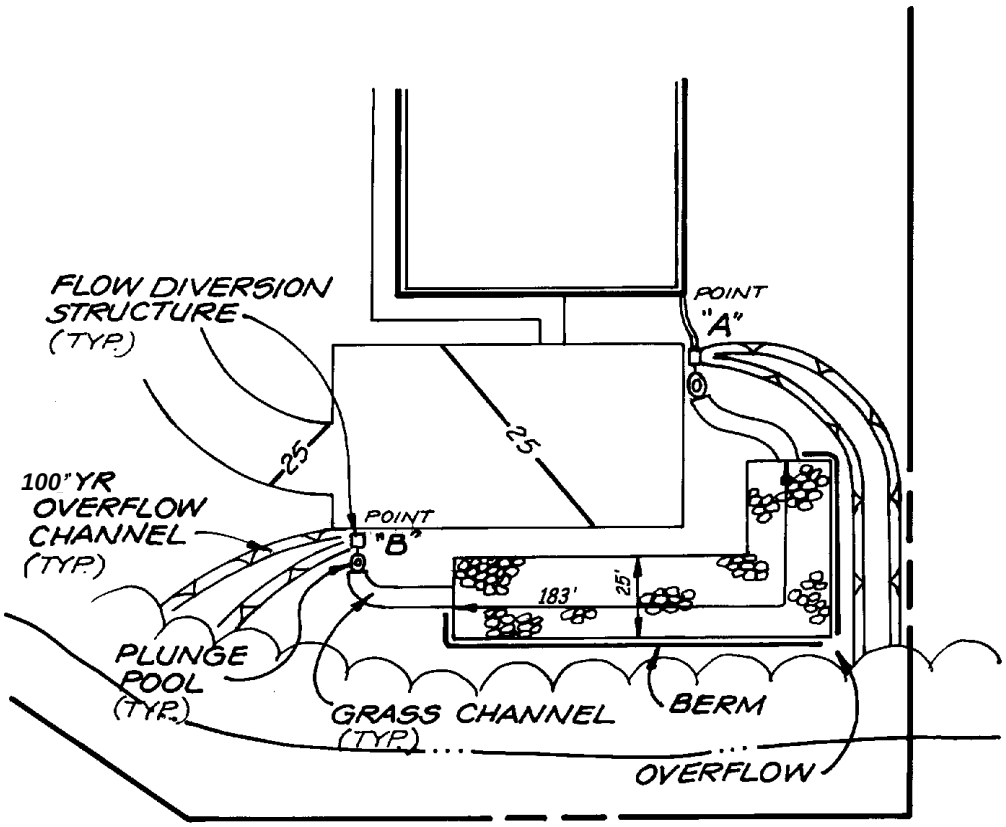


Figure 29.26 Infiltration Trench Site Plan

Size the second diversion (Point B) using the same techniques.

Peak flow equal 2/3 of the value for the entire site. Thus:

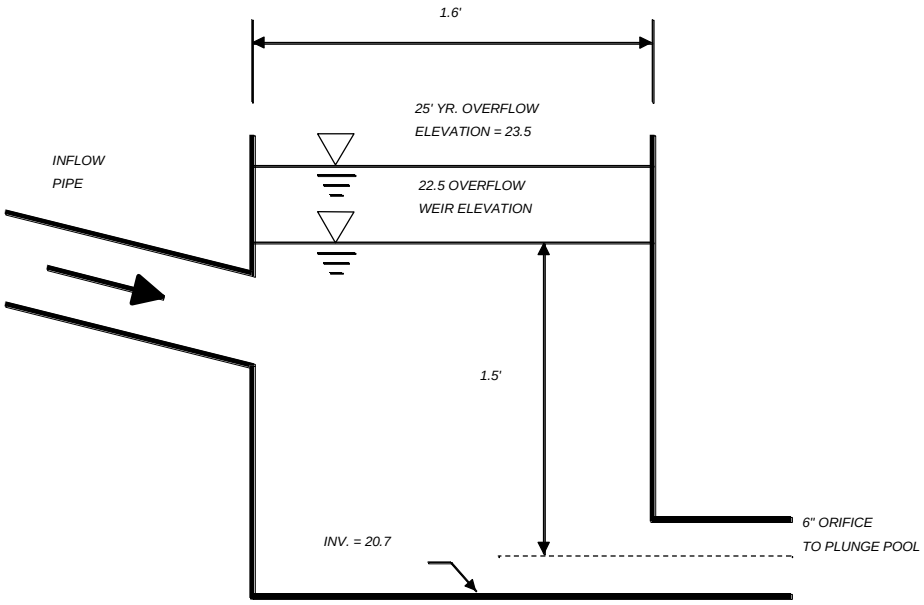
$Q_{100\text{-year}}$ = 29*0.67 = 19.3 cfs
Peak flow for WQ_v = 2.86*0.67 = 1.47 cfs

Size the low flow orifice to pass 1.47 cfs with 1.5' of head using the Orifice equation.

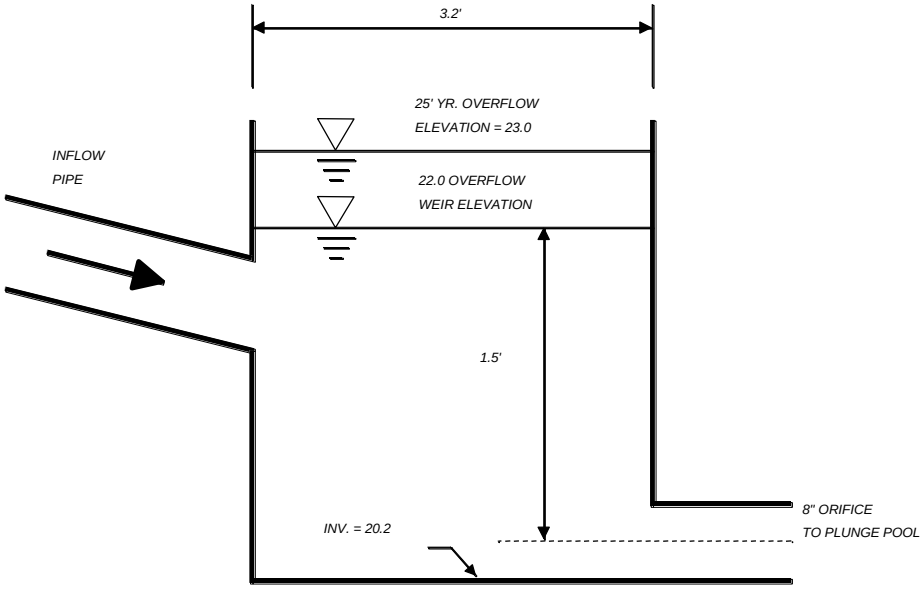
$Q=CA(2gh)^{½}$; 1.91 cfs = $0.6A(2 \times 32.2 \text{ ft/s}^2 \times 1.5')^{½}$
A=0.32 sq. ft. = $\pi d^2/4$; d = 0.64'; use 8" pipe with 8" gate valve

Size the 100-year overflow weir crest at 22.0'. Use a concrete weir to pass the 100-year flow (19.3 –1.91 = 17.39 cfs). Assume 1 foot of head to pass this event. Size using the weir equation.

$Q = CLH^{1.5}$; $L= Q/(CH^{1.5})$
$L = 17.39 \text{ cfs}/ (3.1)(1)^{1.5} = 5.6$; use 5.6' (see Figure 29.27)



POINT "A"



POINT "B"

Figure 29.27 Flow Diversion Structures

Step 7: Size pretreatment volume and design pretreatment measures

As rule of thumb, size pretreatment to treat 25% of the WQ_v. Therefore, treat 8,102 × 0.25 = 2,026 ft^3.

For pretreatment, use a pea gravel filter layer with filter fabric, a plunge pool, and a grass channel.

Pea Gravel Filter

The pea gravel filter layer covers the entire trench with 2" (see Figure 29.28). Assuming a porosity of 0.32, the water quality treatment in the pea gravel filter layer is:

WQ_{filter}= (0.32)(2")(1 ft/12 inches)(4,586 ft^2) = 245 ft^3

*Plunge Pool*s

Use a 5'X10' plunge pool at Point A and a 10'X10' plunge pool at Point B with average depths of 2'.

Total WQ_{pool}= (10 ft)(10+5 ft)(2 ft) = 300 ft^3

Grass Channel

Thus, the grass channel needs to treat at least (2,026 - 245 - 300)ft^3 = 1,481 ft^3

Use a Manning's Equation nomograph or software to size the swale.

The channel at point A should treat one third of 1,481 ft^3 or 494 ft^3

- Assume a trapezoidal channel with 4' channel bottom, 3H:1V side slopes, and a Manning's n value of 0.15. Use a nomograph to size the swale; assume a 1% slope.
- Use a peak discharge of 0.95 cfs (Peak flow for one third of WQ_v, or 2,700 ft^3)
- Compute velocity: V=0.47 ft/s
- To retain the 1/3 of the WQ_v (2,700 ft^3) for 10 minutes, the length would be 282 feet.
- Since the swale only needs to treat 25% of the water quality volume minus the treatment provided by the plunge pool and the gravel layer, or 494 ft^3, the length should be pro-rated to reflect this reduction.

  Therefore, adjust length:

  L= (282 ft)(494 ft^3/2,700 ft^3) =52 feet. Use 55 feet.

  The channel at point B should treat two thirds of 1,481 ft^3, or 988 ft^3
- Assume a trapezoidal channel with 5' channel bottom, 3H:1V side slopes, and a Manning's n value of 0.12. Use a nomograph to size the swale; assume a 0.5% slope.
- Use a peak discharge of 1.91 cfs (Peak flow for two thirds of WQ_v, or 5401 ft^3)
- Compute velocity: V=0.51 ft/s
- To retain the 2/3 of the WQ_v (5,401 ft^3) for 10 minutes, the length would be 306 feet.
- Since the swale only needs to treat 25% of the water quality volume minus the treatment provided by the plunge pool and the gravel layer, or 988 ft^3, the length should be pro-rated to reflect this reduction.

  Therefore, adjust length:

  L= (306 ft)(988 ft^3/5,401 ft^3) = 55 feet. Use 55 feet.

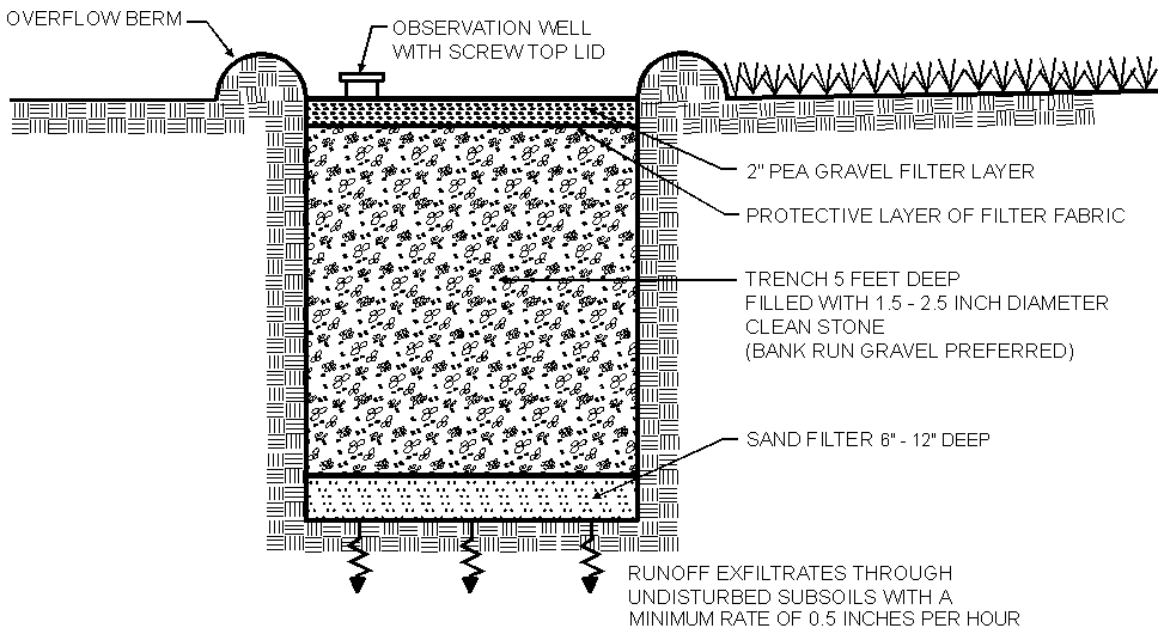


Figure 29.28 Infiltration Trench Cross Section

Step 8: Design Spillway(s)

Adequate stormwater outfalls should be provided for the overflow associated with the 100-year and larger design storm events, ensuring non-erosive velocities on the down slope.

29.6 Enhanced Swale Design Example

This example focuses on the design of a dry swale to meet the water quality treatment requirements of the site. The design options chosen for this example are Option 1 for Water Quality Protection (WQ_v), Option 4 for Streambank Protection (SP_v), and Option 4 for Flood Control (Q_{p100}). It is assumed that the criteria requires enhanced swales to adequately convey the 25-year peak flow. Streambank Protection and Flood Control are not addressed in this example other than quantification of preliminary storage volume and peak discharge requirements. It is assumed that the designer can refer to the previous pond example in order to extrapolate the necessary information to determine and design the required storage and outlet structures to meet these criteria. In general, the primary function of dry swales is to provide water quality treatment and groundwater recharge and not large storm attenuation. As such, flows in excess of the water quality volume are typically routed to bypass the facility. Where quantity control is required, the bypassed flows can be routed to conventional detention basins (or some other facility such as underground storage vaults). The layout of the Wellington Recreation Center is shown in Figure 29.29.

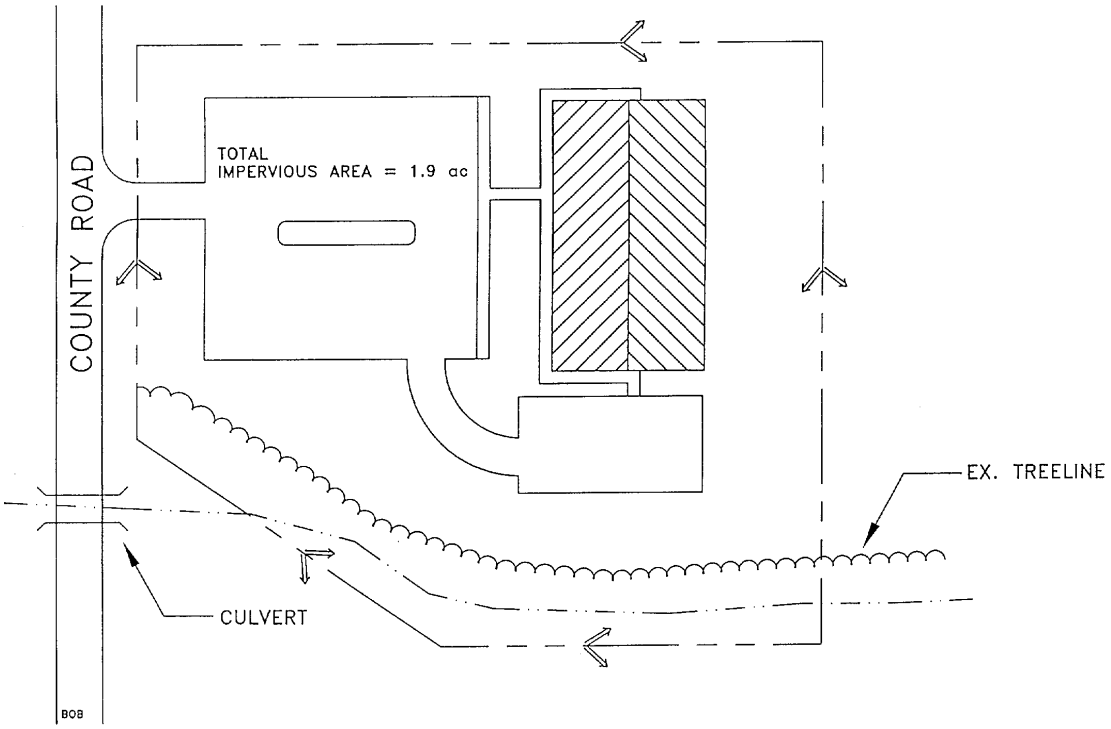


Figure 29.29 Wellington Recreation Center Site Plan

Base Data		Hydrologic Data	
Site Area = Total Drainage Area (A) = 3.0 ac		Pre	Post
Impervious Area = 1.9 ac; or I =1.9/3.0 = 63.3%	CN	77	91
Soils Type 50% "C", 50% "D" Tarrant County	t_c	.41	.20

Computation of Preliminary Stormwater Storage Volumes and Peak Discharges

Two swales will be designed to carry flow to the existing stream, one around each side of the development.

Step 1: Compute runoff control volumes from the *integrated* Design Focus Areas

Compute Water Quality Volume (WQ_v):

- Compute Runoff Coefficient, R_v

  $R_v = 0.05 + (63.3)\ (0.009) = 0.62$

- Compute WQ_v

  $WQ_v = (1.5")\ (R_v)\ (A) / 12$
  $= (1.5")\ (0.62)\ (3.0ac)\ (43{,}560ft^2/ac)\ (1ft/12in)$
  $= 8{,}102\ ft^3 = 0.19\ ac\text{-}ft$

Compute Stream Streambank Protection Volume (SP_v):

For stream streambank protection, provide 24 hours of extended detention for the 1-year event.

In order to determine a preliminary estimate of storage volume for streambank protection and flood control, it will be necessary to perform hydrologic calculations using approved methodologies. This example uses

the NRCS TR-55 methodology presented in *Section 1.1 of the Hydrology Technical Manual* to determine pre- and post-development peak discharges for the 1-yr, 25-yr, and 100-yr 24-hour return frequency storms.

- Per attached TR-55 calculations (Figures 29.30 and 29.31)

Condition	CN	$Q_{1\text{-year}}$	$Q_{1\text{-year}}$	$Q_{100\text{ year}}$
		Inches	*cfs*	*cfs*
Pre-developed	74	0.69	1.7	17.0
Post-Developed	90	1.66	6.5	33.0

- Utilize modified TR-55 approach to compute streambank protection storage volume

  Initial abstraction (Ia) for CN of 90 is 0.222: [I_a = (200/CN - 2)]

  I_a/P = (0.222)/ 2.64 inches = 0.08
  T_c = 0.21 hours
  q_u = 840 csm/in

  Knowing q_u and T (extended detention time), find q_o/q_I for a Type II rainfall distribution.

  Peak outflow discharge/peak inflow discharge (q_o/q_i) = 0.022

  For a Type II rainfall distribution,

  $V_s/V_r = 0.683 - 1.43(q_o/q_i) + 1.64(q_o/q_i)^2 - 0.804(q_o/q_i)^3$

  Where Vs equals streambank protection storage (SP_v) and V_r equals the volume of runoff in inches.

  V_s/V_r = 0.65

  Therefore, $V_s = SP_v$ = 0.65(1.66")(1/12)(3 ac) = 0.27 ac-ft = 11,761 ft^3

Analyze for Safe Passage of 100 Year Design Storm (Q_f):

At final design, prove that discharge conveyance channel is adequate to convey the 100-year event and discharge to receiving waters, or handle it with a peak flow control structure.

Table 29.9 Summary of General Design Information for Wellington Recreation Center

Symbol	Control Volume	Volume Required (cubic feet)	Notes
WQ_v	Water Quality	8,102	
SP_v	Streambank Protection	11,761	
Q_f	Flood Protection	NA	Provide safe passage for the 100-year event in final design

PEAK DISCHARGE SUMMARY				
JOB:	**Wellington of Recreation Center**			**EWB**
DRAINAGE AREA NAME:	**Pre-Developed Conditions**			
COVER DESCRIPTION	**SOIL GROUP A, B, C, D?**	**C from TABLE 1.6 Hydrology Section**	**CN from TABLE 1.9 Hydrology Section**	**AREA (in acres)**
woods (good cond.)	C		70	1.5
woods (good cond.)	D		78	1.5
		AREA SUBTOTALS:		**3.00**
Time of Concentration	**Surface Cover**	**Manning 'n'**	**Flow Length**	**Slope**
2-Yr 24 Hr Rainfall = 3.36"	**Cross Section**	**Wetted Per**	**Avg Velocity**	**Tt (hrs)**
Sheet Flow	**dense grass**	**'n'= 0.24**	**150 ft**	**1.50%**
				0.36 hrs
Shallow Flow	**unpaved**		**500 ft**	**2.00%**
			2.28 fps	**0.06 hrs**
Channel Flow				
Total Area in Acres =	3.00	Total Sheet Flow = 0.36 hrs.	Total Shallow Flow = 0.06 hrs.	Total Channel Flow = 0.00 hrs.
Weighted CN =	74			
Time of Concentration =	0.42 hrs			
Pond Factor =	1	RAINFALL TYPE II		

STORM	Precipitation (P) inches	Runoff (Q)	Qp, PEAK DISCHARGE (cfs)
1 Year	2.64	0.69	1.7
2 Year	3.36	1.14	3.0
5 Year	4.56	2.02	5.4
10 Year	5.52	2.79	7.6
25 Year	6.72	3.80	11.0
50 Year	7.92	4.85	14.0
100 Year	9.12	5.94	17.0

Figure 29.30 Wellington Recreation Center Pre-Developed Conditions

PEAK DISCHARGE SUMMARY				
JOB:	**Wellington on Recreation Center**			**EWB**
DRAINAGE AREA NAME:	**Post-Developed Conditions**			**3-Jan-00**
COVER DESCRIPTION	**SOIL GROUP A, B, C, D?**	**C from TABLE 1.6 Hydrology Section**	**CN from TABLE 1.9 Hydrology Section**	**AREA (in acres)**
open space (good cond.)	C		74	0.25
woods (good cond.)	C		70	0.30
impervious	C		98	1.90
open space (good cond.)	D		80	0.25
woods (good cond.)	D		77	0.30
			AREA SUBTOTALS:	**3.00**
Time of Concentration	**Surface Cover**	**Manning 'n'**	**Flow Length**	**Slope**
2-Yr 24 Hr Rainfall = 3.36"	**Cross Section**	**Wetted Per**	**Avg Velocity**	**Tt (hrs)**
Sheet Flow	**dense grass**	**'n'= 0.24**	**50 ft**	**1.50%**
				0.15 hrs
Shallow Flow	**paved**		**600 ft**	**2.00%**
			2.87 fps	**0.06 hrs**
Channel Flow		**'n'= 0.024**	**50 ft**	**2.00%**
Hydraulic Radius= 0.75	X-S estimated	WP estimated	**7.25 fps**	**0.00hrs**
Total Area in Acres =	3.00	Total Sheet Flow = 0.15 hrs.	Total Shallow Flow = 0.06 hrs.	Total Channel Flow = 0.00 hrs.
Weighted CN =	90			
Time of Concentration =	0.21 hrs			
Pond Factor =	1	RAINFALL TYPE II		

STORM	Precipitation (P) inches	Runoff (Q)	Qp, PEAK DISCHARGE (cfs)
1 Year	2.64	1.66	6.5
2 Year	3.36	2.32	9.3
5 Year	4.56	3.45	14.0
10 Year	5.52	4.38	18.0
25 Year	6.72	5.55	23.0
50 Year	7.92	6.73	28.0
100 Year	9.12	7.91	33.0

Figure 29.31 Wellington Recreation Center Post-Developed Conditions

Step 2: Determine if the development site and conditions are appropriate for the use of an enhanced dry swale system

Existing ground elevation at the facility location is 922.0 feet, mean sea level. Soil boring observations reveal that the seasonally high water table is at 913.0 feet and underlying soils are predominately clay. Adjacent creek invert is at 912.0 feet.

Step 3: Confirm local design criteria and applicability

There is a local requirement that the 25-year storm is contained within the top of banks of all channels, including these enhanced swale controls.

No additional criteria are applicable.

Step 4: Determine pretreatment volume

Size two shallow forebays at the head of the swales equal to 0.05" per impervious acre of drainage (each) (Note, total recommended pretreatment requirement is 0.1"/imp acre). (1.9 ac) (0.05") (1ft/12") (43,560 sq ft/ac) = 344.9 ft^3

Use a 2' deep pea gravel drain at the head of the swale to provide erosion protection and to assist in the distribution of the inflow. There will be no side inflow nor need for pea gravel diaphragm along the sides.

Step 5: Determine swale dimensions

Required: bottom width, depth, length, and slope necessary to store WQ_v with less than 18" of ponding (see Figure 29.32 for representative site plan).

Unit Peak Discharge (q_u), csm/in

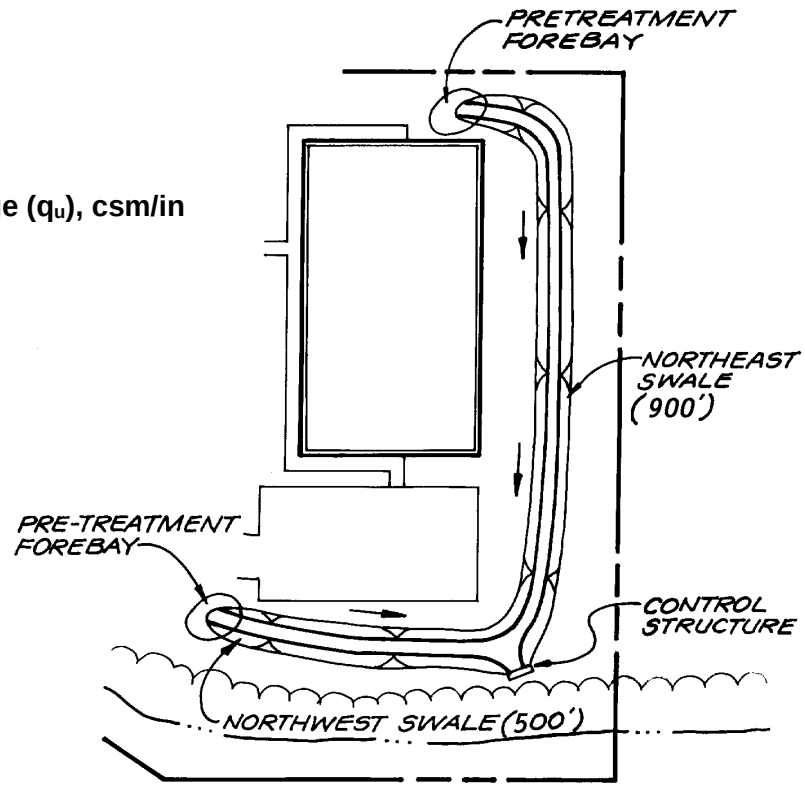


Figure 29.32 Enhanced Dry Swale Site Plan

Assume a trapezoidal channel with a maximum WQ_v depth of 18". Control for this swale will be a shallow concrete wall with a low flow orifice, trash rack located per Figures 29.5 and 29.6. Per the site plan, we have about 1,400' of swale available, if the swale is put in with two tails. The outlet control will be set at the existing invert minus three feet (922.0 - 3.0 = 919.0). The existing uphill invert for the northwest fork is 924.0 (length of 500'), the invert for the northeast fork is 928.0 (at a length of 900').

Slope of northwest fork is (924 - 919)/500' = 0.01 or 1.0%

Slope of northeast fork is (928 - 919)/900' = 0.01 or 1.0%

Minimum slope is 1.0 % [okay]

For a trapezoidal section with a bottom width of 6', a WQ_v average depth of 9", 3:1 side slopes, compute a cross sectional area of (6') (0.75') + (0.75') (2.25') = 6.2 ft^2 (see Figure 29.34).

(6.2 sq ft) (1,400 ft) = 8,680cubic feet [> WQ_v of 8,102 ft^3; OK]

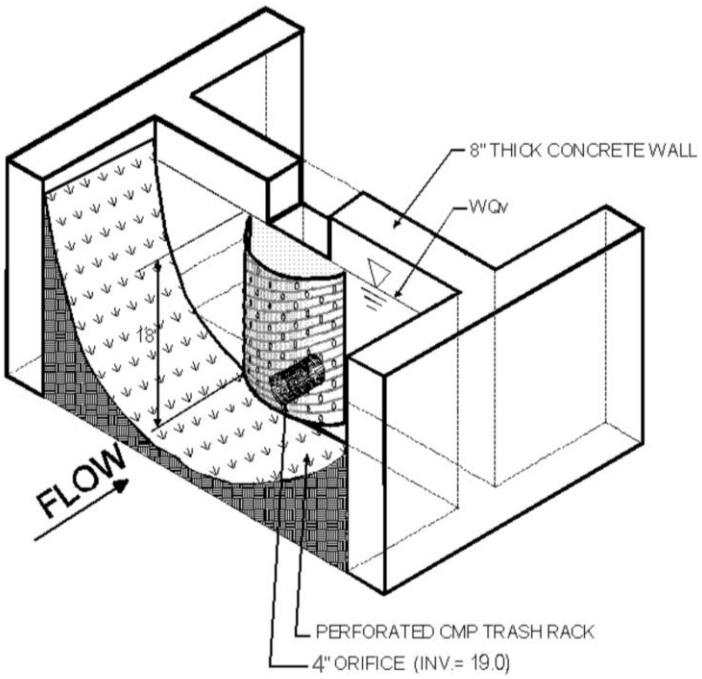


Figure 29.33 Control Structure at End of Swale

Step 6: Compute number of check dams (or similar structure) required to detain WQ_v (see Figure 29.7)

For the northwest fork, 500 ft @ 1.0% slope, and maximum depth at 18", place checkdams at:

1.5'/0.01 = 150' place at 150', 4 required

For the northeast fork, 900 ft @ 1.0% slope, and maximum 18" depth, place checkdams at:

1.5'/0.01 = 150' place at 150', 6 required

Step 7: Calculate draw-down time

In order to ensure that the swale will draw down within 24 hours, the planting soil will need to pass a maximum rate of 1.5' in 24 hours (k = 1.5' per day). Provide 6" perforated underdrain pipe and gravel system below soil bed (see Figure 29.34).

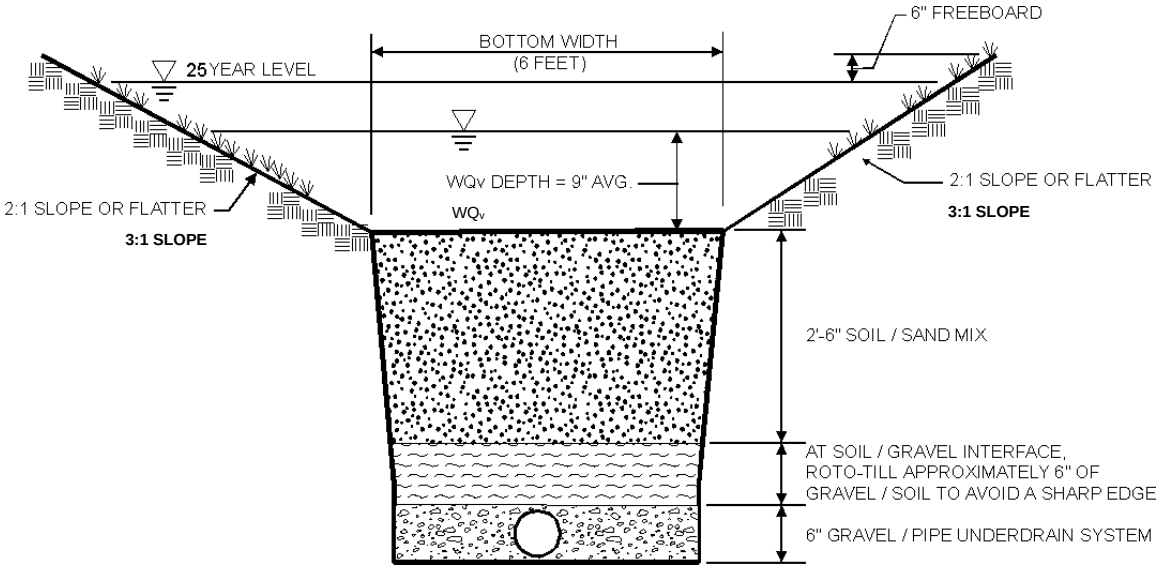


Figure 29.34 Trapezoidal Dry Swale Section

Step 8: Check 25-year flows for velocity erosion potential and freeboard

Given the local requirements to contain the 25-year flow within banks with freeboard. In this example only the 25-year flow will be checked assuming that lower flows will be handled. The 25-year flow is 23.0 cfs, assume that 30% goes through northwestern swale (6.9 cfs) and 70% goes through the northeastern swale (9.3 cfs). Design for the larger amount (13.3 cfs). From separate computer analysis, with a slope of 1.0%, the 25-year velocity will be 2.7 feet-per-second at a depth of .63 feet, provide an additional .5' of freeboard above top of checkdams or about 1.2' (total channel depth = 2.7').

Find 25-year overflow weir length required: (weir eq. $Q= CLH^{3/2}$), where C = 3.1, Q_{25} = 23 cfs, H =1.2; Rearranging the equation yields:

L = 23 cfs/ $(3.1*1.2^{1.5})$ = 5.6' Use 5 ft

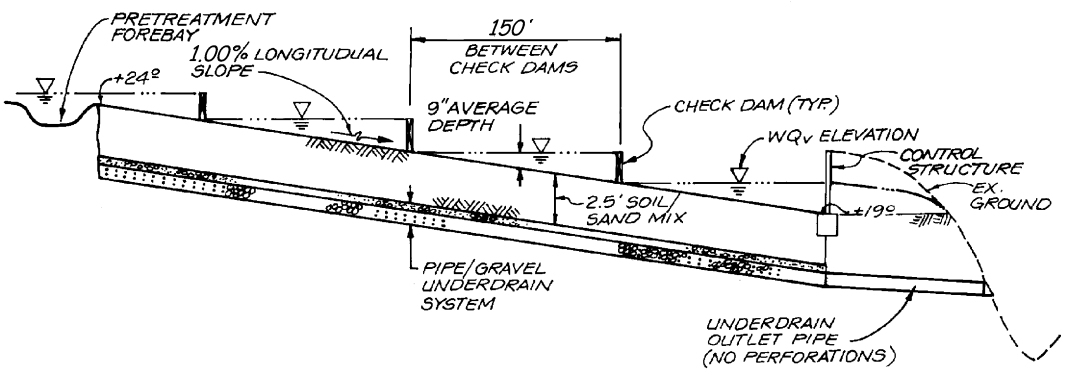


Figure 29.35 Profile of Northwest Fork Dry Swale

Step 9: Design low flow orifice at downstream headwall and checkdams (See Figure 29.33)

Design orifice to pass 8,102 cubic feet in 6 hours.

8,102 cubic feet/ [(6 hours) (3600 sec/hour)] = 0.4 cfs

Use Orifice equation: $Q = CA(2gh)^{1/2}$

Assume h = 1.5'

$A = (0.4 \text{ cfs}) / [(0.6) ((2) (32.2 \text{ ft/s}^2) (1.5'))^{1/2}]$

A = 0.068 sq ft, dia = 0.29 feet or 3.6" Use 4" orifice.

Provide 3" v-notch slot in each check dam.

Step 10: Design inlets, sediment forebay(s), and underdrain system

See Figure 29.35

Step 11: Prepare Vegetation and Landscaping Plan

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Hydraulics:

1.0 Streets and Closed Conduits
2.0 Storage Design
3.0 Open Channels, Culverts, and Bridges
4.0 Energy Dissipation

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1.0 Streets and Closed Conduits

1.1 Stormwater Street and Closed Conduit Design Overview

1.1.1 Stormwater System Design

Stormwater system design is an integral component of both site and overall stormwater management design. Good drainage design must strive to maintain compatibility and minimize interference with existing drainage patterns; control flooding of property, structures, and roadways for design flood events; and minimize potential environmental impacts on stormwater runoff.

Stormwater collection systems must be designed to provide adequate surface drainage while at the same time meeting other stormwater management goals such as water quality, streambank protection, habitat protection, and groundwater recharge.

1.1.2 System Components

The stormwater system components consist of all the *integrated* site design practices and stormwater controls utilized on the site. Three considerations largely shape the design of the stormwater systems: water quality, streambank protection, and flood control.

The on-site flood control systems are designed to remove stormwater from areas such as streets and sidewalks for public safety reasons. The drainage system can consists of inlets, street and roadway gutters, roadside ditches, small channels and swales, stormwater ponds and wetlands, and small underground pipe systems which collect stormwater runoff from mid-frequency storms and transport it to structural control facilities, pervious areas, and/or the larger stormwater systems (i.e., natural waterways, large man-made conduits, and large water impoundments).

The stormwater (major) system consists of natural waterways, large man-made conduits, and large water impoundments. In addition, the major system includes some less obvious drainageways such as overload relief swales and infrequent temporary ponding areas. The stormwater system includes not only the trunk line system that receives the water, but also the natural overland relief which functions in case of overflow from or failure of the on-site flood control system. Overland relief must not flood or damage houses, buildings or other property.

This section is intended to provide design criteria and guidance on several on-site flood control system components; including street and roadway gutters, inlets, and storm drain pipe systems. *Section 2.0* covers storage design and outlet structures. *Section 3.0* covers the design of culverts, vegetated and lined open channels, and bridges. *Section 4.0* covers energy dissipation devices for outlet protection. The rest of this section covers important considerations to keep in mind in the planning and design of stormwater drainage facilities.

1.1.3 Checklist for Planning and Design

The following is a general procedure for drainage system design on a development site.

A. Analyze topography, including:
   1. Check off-site drainage pattern. Where is water coming onto the site? Where is water leaving the site?
   2. Check on-site topography for surface runoff and storage, and infiltration
      a. Determine runoff pattern: high points, ridges, valleys, streams, and swales. Where is the water going?
      b. Overlay the grading plan and indicate watershed areas: calculate square footage (acreage), points of concentration, low points, etc.

B. Analyze other site conditions, including:
   1. Land use and physical obstructions such as walks, drives, parking, patios, landscape edging, fencing, grassed area, landscaped area, tree roots, etc.
   2. Soil type (infiltration rates).
   3. Vegetative cover (slope protection).

C. Check potential drainage outlets and methods, including:
   On-site (structural control, receiving water)
   Off-site (highway, storm drain, receiving water, regional control)
   Natural drainage system (swales)
   Existing drainage system (drain pipe)

D. Analyze areas for probable location of drainage structures and facilities.

E. Identify the type and size of drainage system components required. Design the drainage system and integrate with the overall stormwater management system and plan.

1.1.4 Key Issues in Stormwater System Design

The traditional design of stormwater systems has been to collect and convey stormwater runoff as rapidly as possible to a suitable location where it can be discharged. This manual takes a different approach wherein the design methodologies and concepts of drainage design are to be integrated with the objectives for water quantity and quality control. This means:

- Stormwater systems are to remove water efficiently enough to meet flood protection criteria and level of service requirements, and
- These systems are to complement the ability of the site design and structural stormwater controls to mitigate the major stormwater impacts of urban development.

The following are some of the key issues in integrating water quantity and quality control consideration in stormwater system design.

General Design Considerations

- Stormwater systems should be planned and designed so as to generally conform to natural drainage patterns and discharge to natural drainage pathways within a drainage basin. These natural drainage pathways should only be modified as a last resort to contain and safely convey the peak flows generated by the development.
- Runoff must be discharged in a manner that will not cause adverse impacts on downstream properties or stormwater systems. In general, runoff from development sites within a drainage basin should be discharged at the existing natural drainage outlet or outlets. If the developer wishes to change discharge points he or she must demonstrate that the change will not have any adverse impacts on downstream properties or stormwater (minor) systems.
- It is important to ensure that the combined on-site flood control system and major stormwater system can handle blockages and flows in excess of the design capacity to minimize the likelihood of nuisance flooding or damage to private properties. If failure of minor stormwater systems and/or major stormwater structures occurs during these periods, the risk to life and property could be significantly increased.
- In establishing the layout of stormwater systems, it is essential to ensure that flows are not diverted onto private property during flows up to the major stormwater system design capacity.

Street and Roadway Gutters

- Gutters are efficient flow conveyance structures. This is not always an advantage if removal of pollutants and reduction of runoff is an objective. Therefore, impervious surfaces should be disconnected hydrologically where possible, and runoff should be allowed to flow across pervious surfaces or through vegetated channels. Gutters should be used only after other options have been investigated and only after runoff has had as much chance as possible to infiltrate and filter through vegetated areas.
- It may be possible not to use gutters at all, or to modify them to channel runoff to off-road pervious areas or open channels. For example, curb opening type designs take roadway runoff to smaller feeder grass channels. Care should be taken not to create erosion problems in off-road areas. Protection during construction, establishment of strong stands of vegetation, and active maintenance may be necessary in some areas.
- Use typical road sections that use grass channels or swales instead of gutters to provide for pollution reduction and reduce the impervious area required. Figure 1.1 illustrates a roadway cross section that eliminates gutters for residential neighborhoods. Flow can also be directed to center median strips in divided roadway designs. To protect the edge of pavement, ribbons of concrete can be used along the outer edges of asphalt roads.

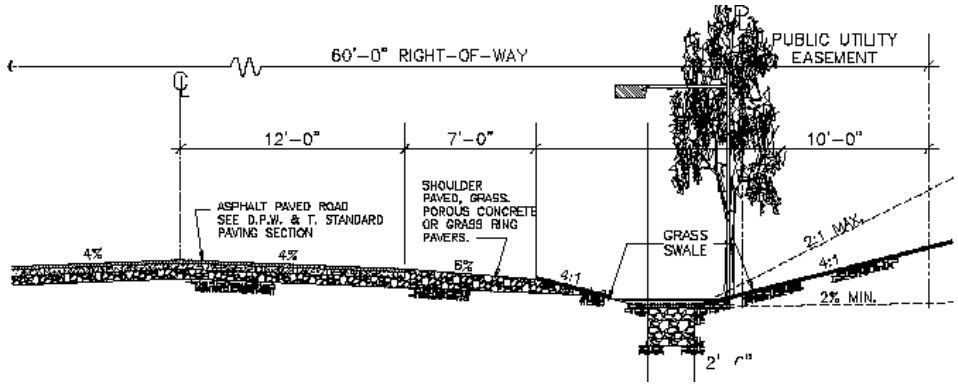


Figure 1.1 Alternate Roadway Section without Gutters
(Source: Prince George's County, MD, 1999)

Inlets and Drains

- Inlets should be located to maximize the overland flow path, take advantage of pervious areas, and seek to maximize vegetative filtering and infiltration. For example, it might be possible to design a parking lot so water flows into vegetated areas prior to entering the nearest inlet.
- Inlet location should not compromise safety or aesthetics. It should not allow for standing water in areas of vehicular or pedestrian traffic, but should take advantage of natural depression storage where possible.
- Inlets should be located to serve as overflows for structural stormwater controls. For example, a bioretention device in a commercial area could be designed to overflow to a catch basin for larger storm events.
- The choice of inlet type should match its intended use. A sumped inlet may be more effective supporting water quality objectives.
- Use several smaller inlets instead of one large inlet in order to:
    - Prevent erosion on steep landscapes by intercepting water before it accumulates too much volume and velocity.

- Provide a safety factor. If a drain inlet clogs, the other surface drains may pick up the water.
- Improve aesthetics. Several smaller drains will be less obvious than one large drain.
- Spacing smaller drain inlets will give surface runoff a better chance of reaching the drain. Water will have to travel farther to reach one large drain inlet.

Closed Conduit Systems (Storm Drains/Sewers)

- The use of *integrated* site design practices (and corresponding site design credits) should be considered to reduce the overall length of a closed conduit stormwater system.
- Shorter and smaller conveyances can be designed to carry runoff to nearby holding areas, natural conservation areas, or filter strips (with spreaders at the end of the pipe).
- Ensure that storms in excess of closed conduit design flows can be safely conveyed through a development without damaging structures or flooding major roadways. This is often done through design of both a major and minor drainage system. The on-site flood control system carries the mid-frequency design flows while larger runoff events may flow across lots and along streets as long as it will not cause property damage or impact public safety.

1.2 On-Site Flood Control System Design

1.2.1 Introduction

On-Site Flood Control Systems, also known as minor drainage systems, quickly remove runoff from areas such as streets and sidewalks for public safety purposes. The on-site flood control system consists of inlets, street and roadway gutters, roadside ditches, small channels and swales, and small underground pipe systems which collect stormwater runoff and transport it to structural control facilities, pervious areas, and/or the larger stormwater system (i.e., natural waterways, large man-made conduits, and large water impoundments).

This section is intended to provide criteria and guidance for the design of on-site flood control system components including:

- Street and roadway gutters
- Stormwater inlets
- Storm drain pipe systems

Ditch, channel and swale design criteria and guidance are covered in *Section 3.0.*

Procedures for performing gutter flow calculations are based on a modification of Manning's Equation. Inlet capacity calculations for grate, curb, and combination inlets are based on information contained in HEC-12 (USDOT, FHWA, 1984). Storm drain system design is based on the use of the Rational Method Formula.

1.2.3 Symbols and Definitions

To provide consistency within this section as well as throughout this Manual, the symbols listed in Table 1.1 will be used. These symbols were selected because of their wide use. In some cases, the same symbol is used in existing publications for more than one definition. Where this occurs in this section, the symbol will be defined where it occurs in the text or equations.

Table 1.1 Symbols and Definitions

Symbol	Definition	Units
a	Gutter depression	in
A	Area of cross section	ft^2
d or D	Depth of gutter flow at the curb line	ft
D	Diameter of pipe	ft
E_o	Ratio of frontal flow to total gutter flow Q_w/Q	-
g	Acceleration due to gravity (32.2 ft/s^2)	ft/s^2
h	Height of curb opening inlet	ft
H	Head loss	ft
K	Loss coefficient	-
L or L_T	Length of curb opening inlet	ft
L	Pipe length	ft
n	Roughness coefficient in the modified Manning's formula for triangular gutter flow	-
P	Perimeter of grate opening, neglecting bars and side against curb	ft
Q	Rate of discharge in gutter	cfs
Q_i	Intercepted flow	cfs
Q_S	Gutter capacity above the depressed section	cfs
S or S_x	Cross Slope - Traverse slope	ft/ft
S or S_L	Longitudinal slope	ft/ft
S_f	Friction slope	ft/ft
S'_w	Depression section slope	ft/ft
T	Top width of water surface (spread on pavement)	ft
T_s	Spread above depressed section	ft
V	Velocity of flow	ft/s
W	Width of depression for curb opening inlets	ft
Z	T/d, reciprocal of the cross slope	-

1.2.4 Street and Roadway Gutters

Effective drainage of street and roadway pavements is essential to the maintenance of the roadway service level and to traffic safety. Water on the pavement can interrupt traffic flow, reduce skid resistance, increase potential for hydroplaning, limit visibility due to splash and spray, and cause difficulty in steering a vehicle when the front wheels encounter puddles. Surface drainage is a function of transverse and longitudinal pavement slope, pavement roughness, inlet spacing, and inlet capacity. The design of these elements is dependent on storm frequency and the allowable spread of stormwater on the pavement surface.

This section presents design guidance for gutter flow hydraulics originally published in HEC-12, Drainage of Highway Pavements and AASHTO's Model Drainage Manual.

Formula

The following form of Manning's Equation should be used to evaluate gutter flow hydraulics:

$$Q = [0.56/n]\ S_x^{5/3}\ S^{1/2}\ T^{8/3} \qquad (1.1)$$

where:

- Q = gutter flow rate, cfs
- S_x = pavement cross slope, ft/ft
- n = Manning's roughness coefficient
- S = longitudinal slope, ft/ft
- T = width of flow or spread, ft

Equation 1.1 was first presented by C.F. Izzard in 1946.

Nomograph

Figure 1.2 is a nomograph for solving Equation 1.1. Manning's n values for various pavement surfaces are presented in Table 1.2 below. Note: the nomograph will not work on slopes steeper than 3% - 4% for 2 and 3 lane thoroughfares. Also, the "Q" in the nomograph is only for n = 0.016.

Manning's n Table

Table 1.2 Manning's n Values for Street and Pavement Gutters

Type of Gutter or Pavement	Manning's n
Concrete gutter, troweled finish	0.014
Asphalt pavement: Smooth texture Rough texture	 0.015 0.019
Concrete gutter with asphalt pavement: Smooth Rough	 0.015 0.018
Concrete pavement: Float finish Broom finish	 0.017 0.019
For gutters with small slopes, where sediment may accumulate, increase above values of n by	0.002

Note: Based on the statement of Izzard (1946) and confirmation by model studies (Ickert and Crosby, 2003), the n-values given in Table 4-3 of HEC No. 22, 2001, were increased by 18% to derive the n-values in this table.

Uniform Cross Slope

The nomograph in Figure 1.2 is used with the following procedures to find gutter capacity for uniform cross slopes:

Condition 1: Find spread, given gutter flow.

Step 1 Determine input parameters, including longitudinal slope (S), cross slope (S_x), gutter flow (Q), and Manning's n.

Step 2 Draw a line between the S and S_x scales and note where it intersects the turning line.

Step 3 Draw a line between the intersection point from Step 2 and the appropriate gutter flow value on the capacity scale. If Manning's n is 0.016, use Q from Step 1; if not; use the product of Q and n (Qn).

Step 4 Read the value of the spread (T) at the intersection of the line from Step 3 and the spread scale.

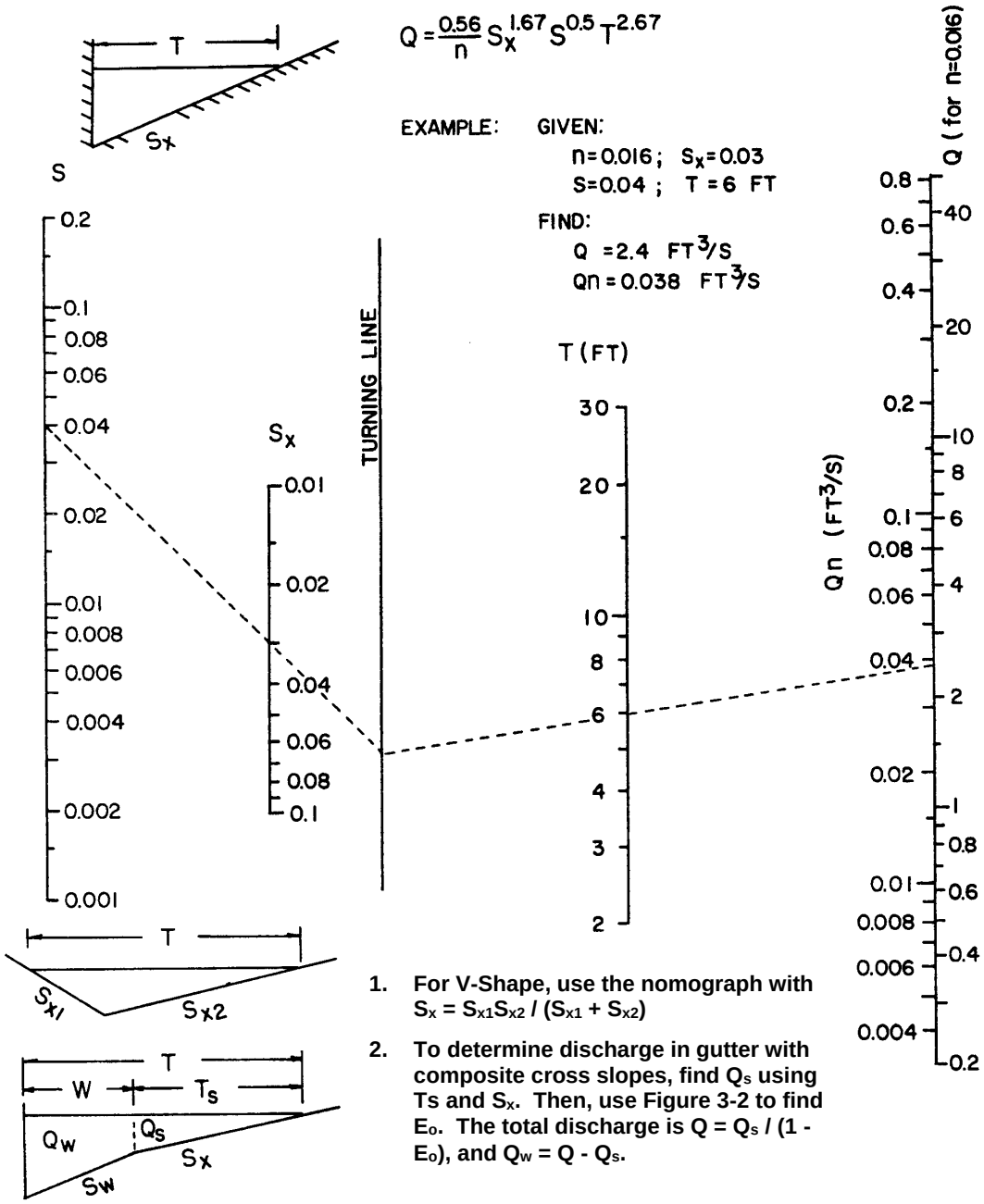
Condition 2: Find gutter flow, given spread.

Step 1 Determine input parameters, including longitudinal slope (S), cross slope (S_x), spread (T), and Manning's n.

Step 2 Draw a line between the S and S_x scales and note where it intersects the turning line.

Step 3 Draw a line between the intersection point from Step 2 and the appropriate value on the T scale. Read the value of Q or Qn from the intersection of that line on the capacity scale.

Step 4 For Manning's n values of 0.016, the gutter capacity (Q) from Step 3 is selected. For other Manning's n values, the gutter capacity times n (Qn) is selected from Step 3 and divided by the appropriate n value to give the gutter capacity.



1. **For V-Shape, use the nomograph with $S_x = S_{x1}S_{x2} / (S_{x1} + S_{x2})$**
2. **To determine discharge in gutter with composite cross slopes, find Q_s using Ts and S_x. Then, use Figure 3-2 to find E_o. The total discharge is $Q = Q_s / (1 - E_o)$, and $Q_w = Q - Q_s$.**

Figure 1.2 Flow in Triangular Gutter Sections
(Source: AASHTO Model Drainage Manual, 1991)

Composite Gutter Sections

Figure 1.3 in combination with Figure 1.2 can be used to find the flow in a gutter with width (W) less than the total spread (T). Such calculations are generally used for evaluating composite gutter sections or frontal flow for grate inlets.

Figure 1.4 provides a direct solution of gutter flow in a composite gutter section. The flow rate at a given spread or the spread at a known flow rate can be found from this figure. Figure 1.4 involves a complex graphical solution of the equation for flow in a composite gutter section. Typical of graphical solutions, extreme care in using the figure is necessary to obtain accurate results.

Condition 1: Find spread, given gutter flow.

Step 1 Determine input parameters, including longitudinal slope (S), cross slope (S_x), depressed section slope (S_w), depressed section width (W), Manning's n, gutter flow (Q), and a trial value of gutter capacity above the depressed section (Q_s).

Step 2 Calculate the gutter flow in W (Q_w), using the equation:

$$\mathbf{Q_w = Q - Q_s} \qquad \mathbf{(1.2)}$$

Step 3 Calculate the ratios Q_w/Q or E_o and S_w/S_x and use Figure 1.3 to find an appropriate value of W/T.

Step 4 Calculate the spread (T) by dividing the depressed section width (W) by the value of W/T from Step 3.

Step 5 Find the spread above the depressed section (T_s) by subtracting W from the value of T obtained in Step 4.

Step 6 Use the value of T_s from Step 5 along with Manning's n, S, and S_x to find the actual value of Q_s from Figure 1.2.

Step 7 Compare the value of Q_s from Step 6 to the trial value from Step 1. If values are not comparable, select a new value of Q_s and return to Step 1.

Condition 2: Find gutter flow, given spread.

Step 1 Determine input parameters, including spread (T), spread above the depressed section (T_s), cross slope (S_x), longitudinal slope (S), depressed section slope (S_w), depressed section width (W), Manning's n, and depth of gutter flow (d).

Step 2 Use Figure 1.2 to determine the capacity of the gutter section above the depressed section (Q_s). Use the procedure for uniform cross slopes, substituting T_s for T.

Step 3 Calculate the ratios W/T and S_w/S_x, and, from Figure 1.3, find the appropriate value of E_o (the ratio of Q_w/Q).

Step 4 Calculate the total gutter flow using the equation:

$$\mathbf{Q = Q_s / (1 - E_o)} \qquad \mathbf{(1.3)}$$

where:

- Q = gutter flow rate, cfs
- Q_s = flow capacity of the gutter section above the depressed section, cfs
- E_o = ratio of frontal flow to total gutter flow (Q_w/Q)

Step 5 Calculate the gutter flow in width (W), using Equation 1.2.

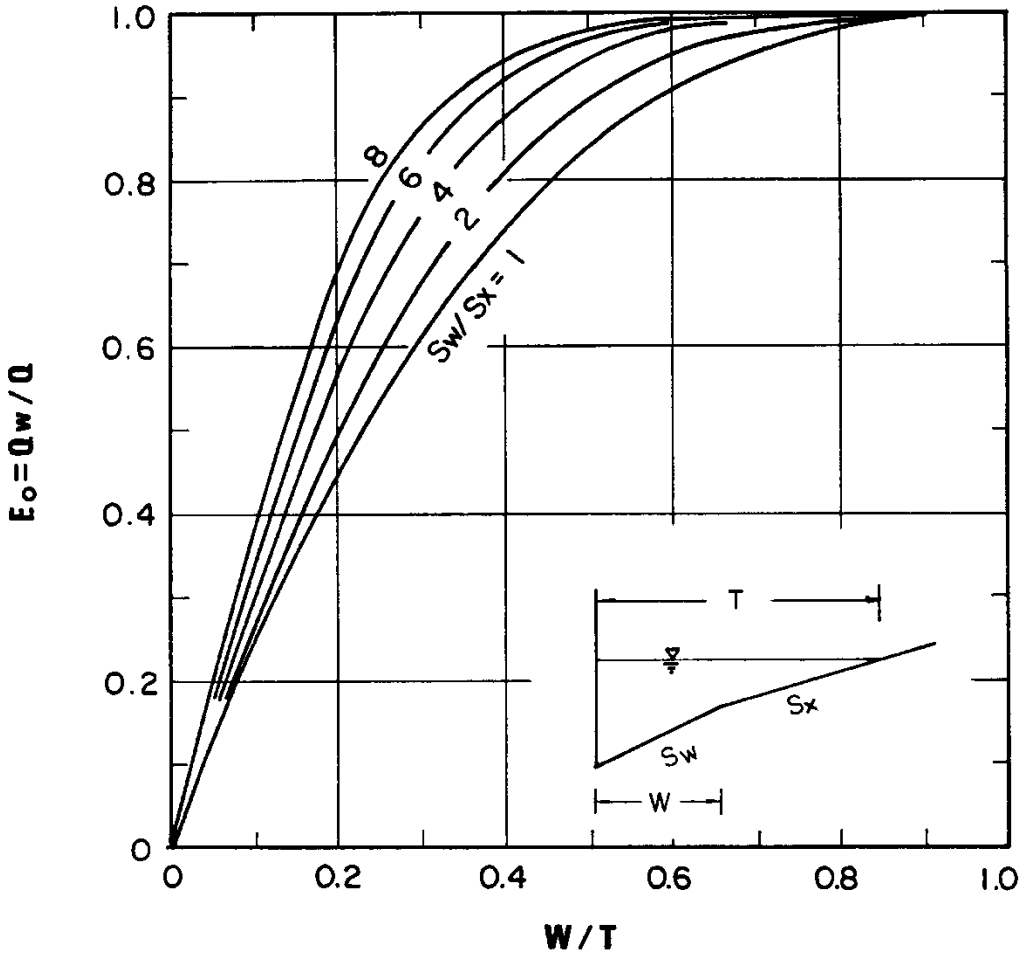


Figure 1.3 Ratio of Frontal Flow to Total Gutter Flow
(Source: AASHTO Model Drainage Manual, 1991)

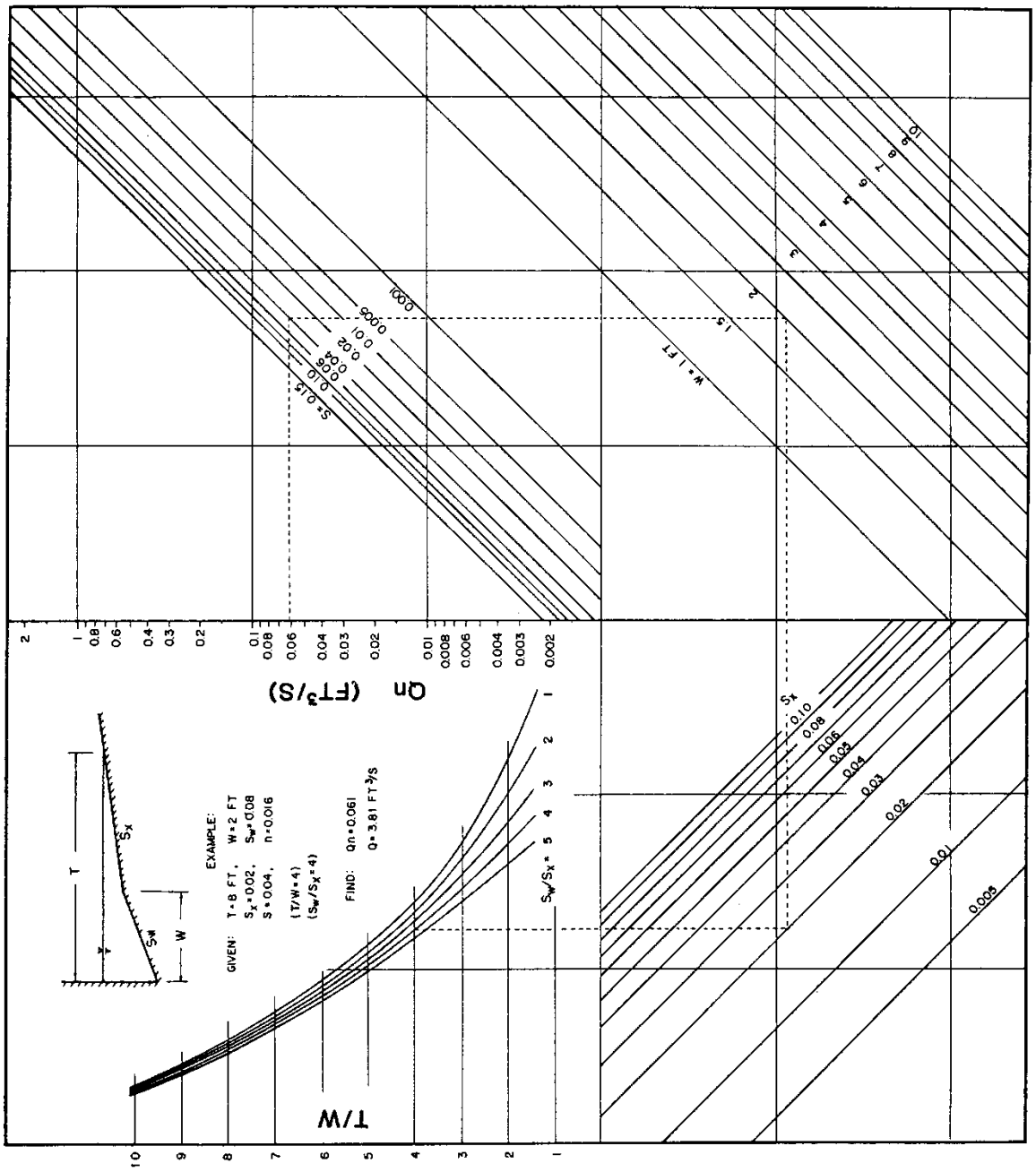


Figure 1.4 Flow in Composite Gutter Sections
(Source: AASHTO Model Drainage Manual, 1991)

Examples

Example 1

Given:

T = 8 ft
S_x = 0.025 ft/ft
n = 0.015
S = 0.01 ft/ft

Find:

1. Flow in gutter at design spread
2. Flow in width (W = 2 ft) adjacent to the curb

Solution:

a. From Figure 1.2, Qn = 0.03
Q = Qn/n = 0.03/0.015 = 2.0 cfs

b. T = 8 - 2 = 6 ft
$(Qn)_2$ = 0.014 (Figure 1.2) (flow in 6-foot width outside of width (W))
Q = 0.014/0.015 = 0.9 cfs
Q_w = 2.0 - 0.9 = 1.1 cfs

Flow in the first 2 ft adjacent to the curb is 1.1 cfs and 0.9 cfs in the remainder of the gutter.

Example 2

Given:

T = 6 ft
S_w = 0.0833 ft/ft
T_s = 6 - 1.5 = 4.5 ft
W = 1.5 ft
S_x = 0.03 ft/ft
n = 0.014
S = 0.04 ft/ft

Find:

Flow in the composite gutter

Solution:

1. Use Figure 1.2 to find the gutter section capacity above the depressed section.
Q_sn = 0.038
Q_s = 0.038/0.014 = 2.7 cfs
2. Calculate W/T = 1.5/6 = 0.25 and
S_w/S_x = 0.0833/0.03 = 2.78
Use Figure 1.3 to find E_o = 0.64
3. Calculate the gutter flow using Equation 1.3
Q = 2.7/(1 - 0.64) = 7.5 cfs
4. Calculate the gutter flow in width, W, using Equation 1.2
Q_W = 7.5 - 2.7 = 4.8 cfs

Parabolic Cross Slope

The following methodology regarding Parabolic cross-slopes was excerpted from the City of Austin Drainage Criteria Manual dated July 2003.

Flows in the gutter of a parabolically crowned pavement are calculated from a variation of Manning's Equation, which assumes steady flow in a prismatic open channel. However, this equation is complicated and difficult to solve for each design case.

To provide a means of determining the flow in the gutter, generalized gutter flow equations for combinations of parabolic crown heights, curb splits and street grades of different street widths have been prepared. All of these equations have a logarithmic form.

Note: The street width used in this section is measured from face of curb to face of curb.

Streets Without Curb Split

Curb split is the vertical difference in elevation between curbs at a given street cross section. The gutter flow equation for parabolic crown streets without any curb split is:

$$\log Q = K_0 + K_1 \log S_0 + K_2 \log y_0 \qquad (1.4)$$

where:

Q = Gutter flow rate, cfs

S_0 = Street grade, ft/ft

y_0 = Water depth in the gutter, feet

K_0, K_1, K_2 = Constant coefficients shown in Table 1.3 for different street widths

Table 1.3 Coefficients for Equation 1.4, Streets Without Curb Split

Street Width* (ft)	Coefficients		
	K_0	K_1	K_2
30	2.85	0.50	3.03
36	2.89	0.50	2.99
40	2.85	0.50	2.89
44	2.84	0.50	2.83
48	2.83	0.50	2.78
60	2.85	0.50	2.74

***Note:** Based on the Transportation Criteria Manual the street width is measured from face of curb to face of curb (FOC-FOC).

Source: City of Austin, Watershed Engineering Division

Streets With Curb Split

The gutter flow equation for parabolic crown streets with curb split is:

$$\log Q = K_0 + K_1 \log S_0 + K_2 \log y_0 + K_3 (CS) \qquad (1.5)$$

where:

Q = Gutter flow rate, cfs
S_0 = Street grade, ft/ft
y_0 = Water depth in the gutter, feet
CS = Curb split, feet
K_0, K_1, K_2, K_3 = Constant coefficients shown in Tables 1.4 and 1.5

The K values in Equation 1.5 are found in Tables 1.4 and 1.5 for different street widths. Table 1.4 is used when calculating the higher gutter flows, and Table 1.5 is used when calculating the lower gutter flows.

Table 1.4 Coefficients for Equation 1.5, Streets With Curb Split – Higher Gutter

Street Width* (ft)	Coefficients				Curb Split Range (ft)
	K_0	K_1	K_2	K_3	
30	2.85	0.50	3.03	-0.131	0.0-0.6
36	2.89	0.50	2.99	-0.140	0.0-0.8
40	2.85	0.50	2.89	-0.084	0.0-0.8
44	2.84	0.50	2.83	-0.091	0.0-0.9
48	2.83	0.50	2.78	-0.095	0.0-1.0
60	2.85	0.50	2.74	-0.043	0.0-1.2

Source: City of Austin, Watershed Engineering Division

Table 1.5 Coefficients for Equation 1.5, Streets With Curb Split – Lower Gutter

Street Width* (ft)	Coefficients				Curb Split Range (ft)
	K_0	K_1	K_2	K_3	
30	2.70	0.50	2.74	-0.215	0.0-0.6
36	2.74	0.50	2.73	-0.214	0.0-0.8
40	2.75	0.50	2.73	-0.198	0.0-0.8
44	2.76	0.50	2.73	-0.186	0.0-0.9
48	2.77	0.50	2.72	-0.175	0.0-1.0
60	2.80	0.50	2.71	-0.159	0.0-1.2

Source: City of Austin, Watershed Engineering Division

All the crown heights for different street widths are calculated by the following equation:

Crown Height (feet) = 0.5 * W * S_x

where,

W = street width, feet
S_x = road cross slope, ft/ft

Parabolic Crown Location

The gutter flow equation presented for parabolic crowns with split curb heights is based on a procedure for locating the street crown. The procedure allows the street crown to shift from the street center line toward the high ¼ point of the street in direct proportion to the amount of curb split. The maximum curb split occurs with the crown at the ¼ point of the street. The maximum allowable curb split for a street with parabolic crowns is 0.02 feet per foot of street width, except for 30-foot side streets where the maximum curb split is considered to be 5 feet..

Example: Determination of Crown Location

Given: 0.4 feet Design split on 30-foot wide street.

Maximum Curb Split = 0.02 x street width
= 0.2 x 30 feet = 0.6 feet
Maximum Movement = ¼ street width for 30-foot street
= ¼ x 30 feet = 7.5 feet

Split Movement = (Design Split x W/Maximum Split x 4)
= (0.4 x 30/0.6 x 4) = 5 feet

Curb splits that are determined by field survey, whether built intentionally or not, should be considered when determining the capacity of the curb flow.

Special consideration should be given when working with cross sections which have the pavement crown above the top of curb. When the crown exceeds the height of the curb the maximum depth of water is equal to the height of the curb, not the crown height. It should be noted that a parabolic section where the crown equals the top of curb will carry more water than a section which has the crown one (1) inch above the top of curb. For other parabolic roadway sections not included in Tables 1.3, 1.4, and 1.5, see HEC-22 Urban Design Manual by the Federal Highway Administration, dated August 2001.

1.2.5 Stormwater Inlets

Inlets are drainage structures used to collect surface water through grate or curb openings and convey it to storm drains or direct outlet to culverts. Grate inlets subject to traffic should be bicycle safe and be load-bearing adequate. Appropriate frames should be provided.

Inlets used for the drainage of highway surfaces can be divided into three major classes:

- Grate Inlets – These inlets include grate inlets consisting of an opening in the gutter covered by one or more grates, and slotted inlets consisting of a pipe cut along the longitudinal axis with a grate or spacer bars to form slot openings.
- Curb-Opening Inlets – These inlets are vertical openings in the curb covered by a top slab.
- Combination Inlets – These inlets usually consist of both a curb-opening inlet and a grate inlet placed in a side-by-side configuration, but the curb opening may be located in part upstream of the grate.

Inlets may be classified as being on a continuous grade or in a sump. The term "continuous grade" refers to an inlet located on the street with a continuous slope past the inlet with water entering from one direction. The "sump" condition exists when the inlet is located at a low point and water enters from both directions. Sump areas should have an overflow route or channel.

Where significant ponding can occur, in locations such as underpasses and in sag vertical curves in depressed sections, it is good engineering practice to place flanking inlets on each side of the inlet at the low point in the sag. The flanking inlets should be placed so they will limit spread on low gradient

approaches to the level point and act in relief of the inlet at the low point if it should become clogged or if the design spread is exceeded.

The design of grate inlets will be discussed in *Section 1.2.6*, curb inlet design in *Section 1.2.7*, and combination inlets in *Section 1.2.8*.

1.2.6 Grate Inlet Design

Grate Inlets on Grade

The capacity of an inlet depends upon its geometry and the cross slope, longitudinal slope, total gutter flow, depth of flow, and pavement roughness. The depth of water next to the curb is the major factor in the interception capacity of both gutter inlets and curb opening inlets. At low velocities, all of the water flowing in the section of gutter occupied by the grate, called frontal flow, is intercepted by grate inlets, and a small portion of the flow along the length of the grate, termed side flow, is intercepted. On steep slopes, only a portion of the frontal flow will be intercepted if the velocity is high or the grate is short and splash-over occurs. For grates less than 2 feet long, intercepted flow is small.

A parallel bar grate is the most efficient type of gutter inlet; however, when crossbars are added for bicycle safety, the efficiency is greatly reduced. Where bicycle traffic is a design consideration, the curved vane grate and the tilt bar grate are recommended for both their hydraulic capacity and bicycle safety features. They also handle debris better than other grate inlets but the vanes of the grate must be turned in the proper direction. Where debris is a problem, consideration should be given to debris handling efficiency rankings of grate inlets from laboratory tests in which an attempt was made to qualitatively simulate field conditions. Table 1.6 presents the results of debris handling efficiencies of several grates. Debris handling efficiencies were based on the total number of simulated leaves arriving at the grate and the number passed.

The ratio of frontal flow to total gutter flow, E_O, for straight cross slope is expressed by the following equation:

$$\mathbf{E_o = Q_w/Q = 1 - (1 - W/T)^{2.67}} \qquad \mathbf{(1.6)}$$

where:

Q = total gutter flow, cfs

Q_W = flow in width W, cfs

W = width of depressed gutter or grate, ft

T = total spread of water in the gutter, ft

Table 1.6 Grate Debris Handling Efficiencies

Rank	Grate	Longitudinal Slope	
		(0.005)	(0.04)
1	CV - 3-1/4 - 4-1/4	46	61
2	30 - 3-1/4 - 4	44	55
3	45 - 3-1/4 - 4	43	48
4	P - 1-7/8	32	32
5	P - 1-7/8 - 4	18	28
6	45 - 2-1/4 - 4	16	23
7	Reticuline	12	16
8	P - 1-1/8	9	20

Source: "Drainage of Highway Pavements" (HEC-12), Federal Highway Administration, 1984.

Figure 1.3 provides a graphical solution of E_o for either depressed gutter sections or straight cross slopes. The ratio of side flow, Q_s, to total gutter flow is:

$$\mathbf{Q_s/Q = 1 - Q_w/Q = 1 - E_o} \quad \textbf{(1.7)}$$

The ratio of frontal flow intercepted to total frontal flow, R_f, is expressed by the following equation:

$$\mathbf{R_f = 1 - 0.09\,(V - V_0)} \quad \textbf{(1.8)}$$

where:

V = velocity of flow in the gutter, ft/s (using Q from Figure 1.2)

V_o = gutter velocity where splash-over first occurs, ft/s (from Figure 1.5)

This ratio is equivalent to frontal flow interception efficiency. Figure 1.5 provides a solution of Equation 1.8, which takes into account grate length, bar configuration and gutter velocity at which splash-over occurs. The gutter velocity needed to use Figure 1.5 is total gutter flow divided by the area of flow. The ratio of side flow intercepted to total side flow, R_s, or side flow interception efficiency, is expressed by:

$$\mathbf{R_s = 1 / [1 + (0.15V^{1.8}/S_xL^{2.3})]} \quad \textbf{(1.9)}$$

where:

L = length of the grate, ft

Figure 1.5 provides a solution to Equation 1.9.

The efficiency, E, of a grate is expressed as:

$$\mathbf{E = R_fE_o + R_s(1 - E_o)} \quad \textbf{(1.10)}$$

The interception capacity of a grate inlet on grade is equal to the efficiency of the grate multiplied by the total gutter flow:

$$\mathbf{Q_i = EQ = Q[R_fE_o + R_s(1 - E_o)]} \quad \textbf{(1.11)}$$

The following example illustrates the use of this procedure.

Given:

W = 2 ft

T = 8 ft

S_x = 0.025 ft/ft

S = 0.01 ft/ft

E_o = 0.69

Q = 3.0 cfs

V = 3.1 ft/s

Gutter depression = 2 in

Find:

Interception capacity of:

1. a curved vane grate, and
2. a reticuline grate 2-ft long and 2-ft wide

Solution:

From Figure 1.5 for Curved Vane Grate, R_f = 1.0

From Figure 1.5 for Reticuline Grate, R_f = 1.0

From Figure 1.6 R_s = 0.1 for both grates

From Equation 1.11:

$Q_i = 3.0[1.0 \text{ X } 0.69 + 0.1(1 - 0.69)] = 2.2$ cfs

For this example, the interception capacity of a curved vane grate is the same as that for a reticuline grate for the sited conditions.

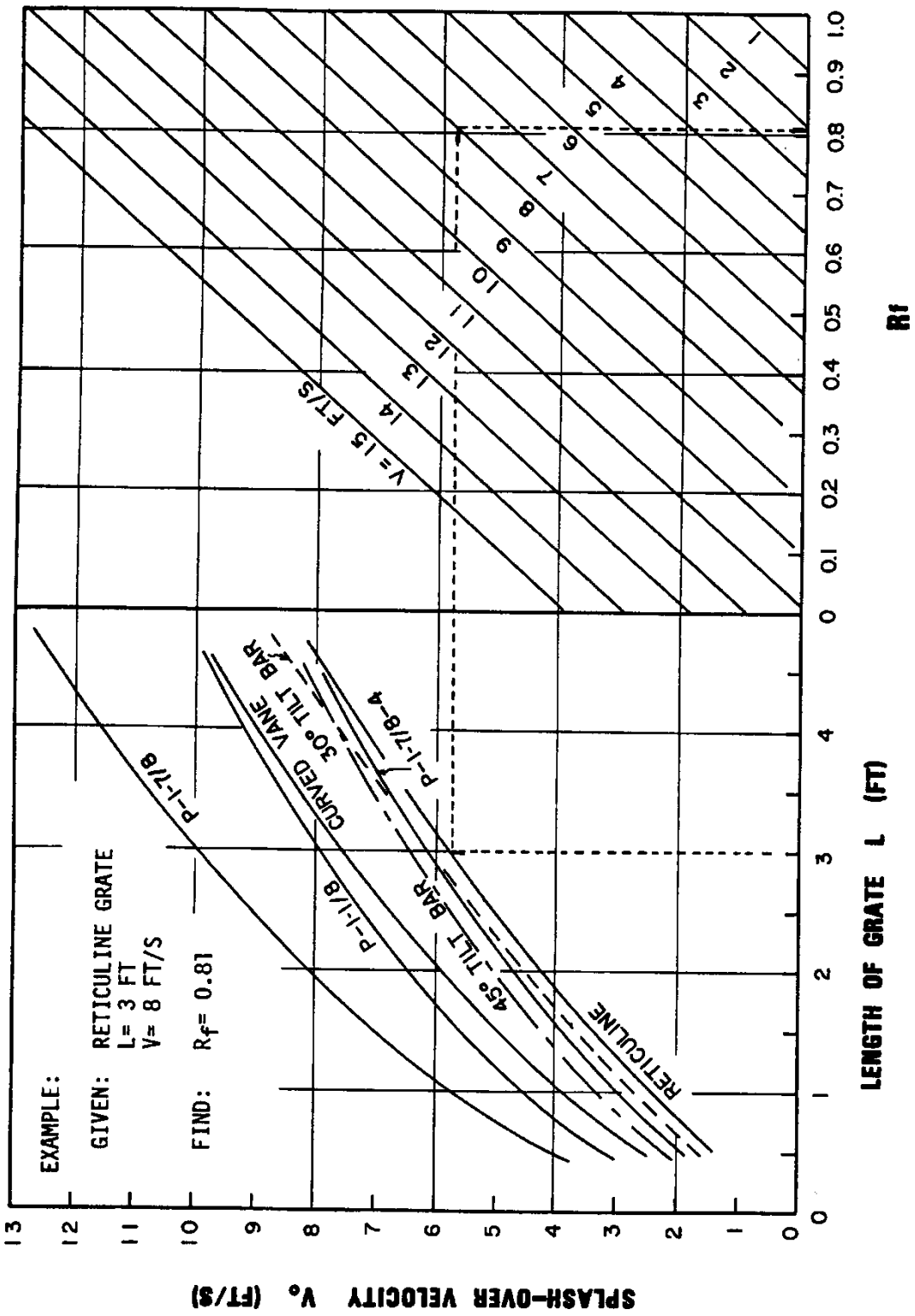


Figure 1.5 Grate Inlet Frontal Flow Interception Efficiency
(Source: HEC-12, 1984)

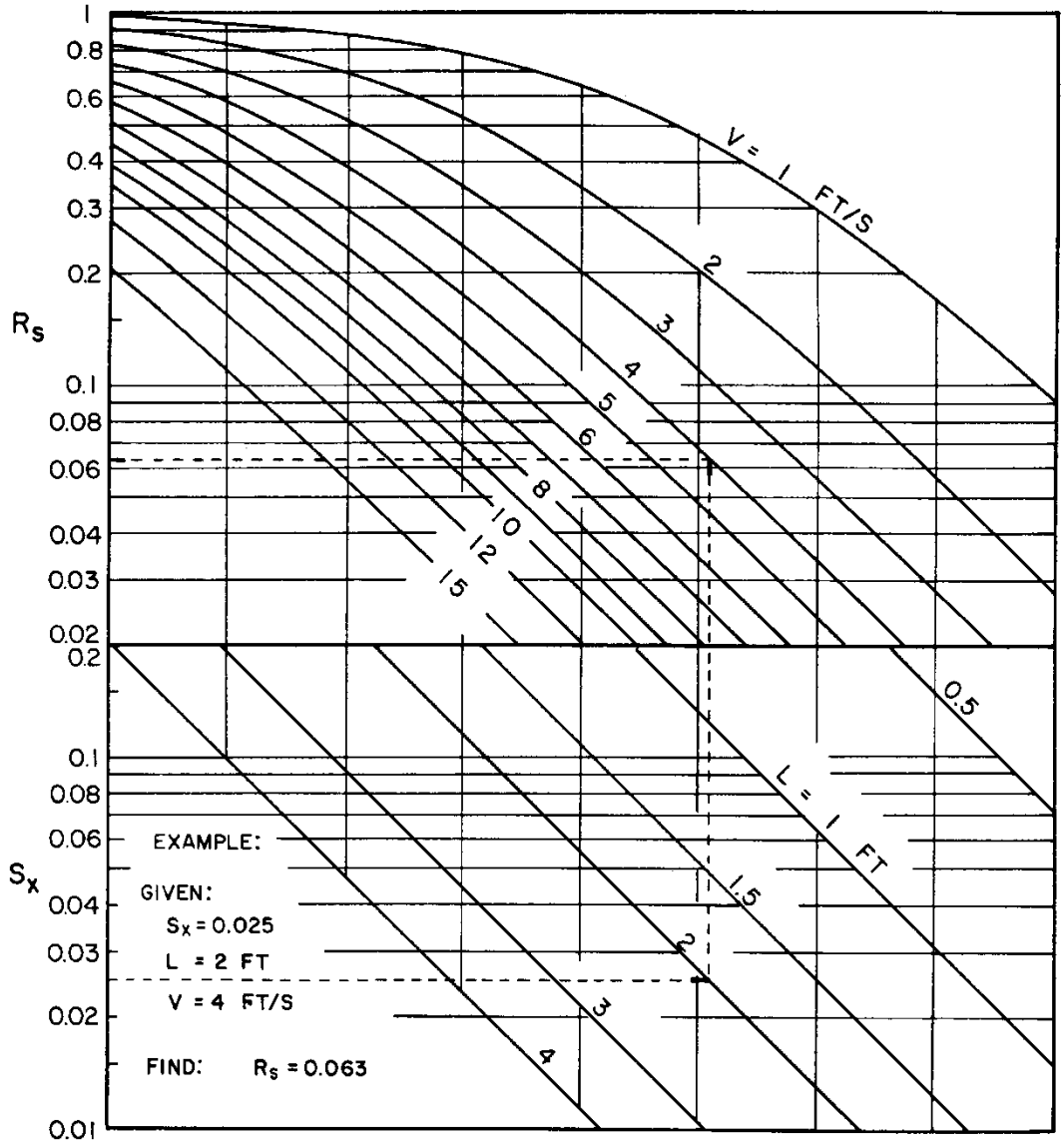


Figure 1.6 Grate Inlet Side Flow Interception Efficiency
(Source: HEC-12, 1984)

Grate Inlets in Sag

A grate inlet in a sag operates as a weir up to a certain depth, depending on the bar configuration and size of the grate, and as an orifice at greater depths. For a standard gutter inlet grate, weir operation continues to a depth of about 0.4 feet above the top of grate and when depth of water exceeds about 1.4 feet, the grate begins to operate as an orifice. Between depths of about 0.4 feet and about 1.4 feet, a transition from weir to orifice flow occurs.

The capacity of grate inlets operating as a weir is:

$$\mathbf{Q_i = CPd^{1.5}} \qquad \textbf{(1.12)}$$

where:

P = perimeter of grate excluding bar widths and the side against the curb, ft

C = 3.0

d = depth of water above grate, ft

and as an orifice is:

$$\mathbf{Q_i = CA(2gd)^{0.5}} \qquad \textbf{(1.13)}$$

where:

C = 0.67 orifice coefficient

A = clear opening area of the grate, ft^2

g = 32.2 ft/s^2

Figure 1.7 is a plot of Equations 1.12 and 1.13 for various grate sizes. The effect of grate size on the depth at which a grate operates as an orifice is apparent from the chart. Transition from weir to orifice flow results in interception capacity less than that computed by either weir or the orifice equation. This capacity can be approximated by drawing in a curve between the lines representing the perimeter and net area of the grate to be used. The following example illustrates the use of this figure.

Given:

A symmetrical sag vertical curve with equal bypass from inlets upgrade of the low point; allow for 50% clogging of the grate.

Q_b = 3.6 cfs

Q = 8 cfs, 25-year storm

T = 10 ft, design

S_x = 0.05 ft/ft

d = TS_x = 0.5 ft

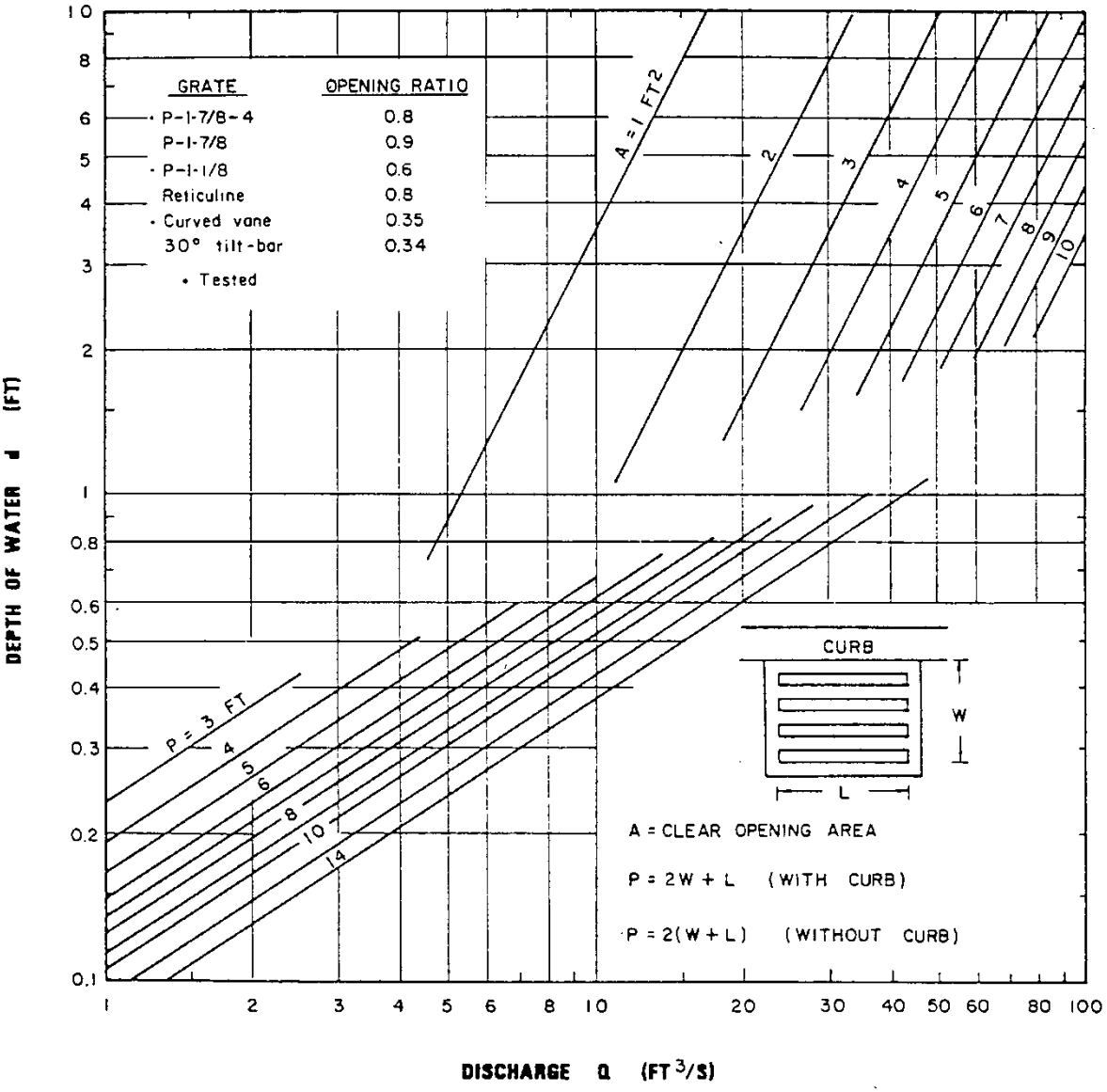
Find:

Grate size for design Q. Check spread at S = 0.003 on approaches to the low point.

Solution:

From Figure 1.7, a grate must have a perimeter of 8 ft to intercept 8 cfs at a depth of 0.5 ft.

Some assumptions must be made regarding the nature of the clogging in order to compute the capacity of a partially clogged grate. If the area of a grate is 50% covered by debris so that the debris-covered portion does not contribute to interception, the effective perimeter will be reduced by a lesser amount than 50%. For example if a 2-ft x 4-ft grate is clogged so that the effective width is 1 ft, then the perimeter, P = 1 + 4 + 1 = 6 ft, rather than 8 ft, the total perimeter, or 4 ft, half of the total perimeter. The area of the opening would be reduced by 50% and the perimeter by 25%.



Reference: USDOT, FHWA, HEC-12 (1984).

Figure 1.7 Grate Inlet Capacity in Sag Conditions
(Source: HEC-12, 1984)

Therefore, assuming 50% clogging along the length of the grate, a 4 x 4, a 2 x 6, or a 3 x 5 grate would meet requirements of an 8-ft perimeter 50% clogged.

Assuming that the installation chosen to meet design conditions is a double 2 x 3 ft grate, for 50% clogged conditions: P = 1 + 6 + 1 = 8 ft

For 25-year flow: d = 0.5 ft (from Figure 1.7)

The American Society of State Highway and Transportation Officials (AASHTO) geometric policy recommends a gradient of 0.3% within 50 ft of the level point in a sag vertical curve.

Check T at S = 0.003 for the design and check flow:

Q = 3.6 cfs, T = 8.2 ft (25-year storm) (from Figure 1.2)

Thus a double 2 x 3-ft grate inlet with 50% clogging is adequate to intercept the design flow at a spread that does not exceed design spread, and to ensure the spread on the approaches to the low point will not exceed design spread. However, the tendency of grate inlets to clog completely warrants consideration of a combination inlet or curb-opening inlet in sag where ponding can occur, as well as flanking inlets on the low gradient approaches.

1.2.7 Curb Inlet Design

Curb Inlets on Grade

Following is a discussion of the procedures for the design of curb inlets on grade. Curb-opening inlets are effective in the drainage of pavements where flow depth at the curb is sufficient for the inlet to perform efficiently. Curb openings are relatively free of clogging tendencies and offer little interference to traffic operation. They are a viable alternative to grates in many locations where grates would be in traffic lanes or would be hazardous for pedestrians or bicyclists.

The length of curb-opening inlet required for total interception of gutter flow on a pavement section with a straight cross slope is determined using Figure 1.8. The efficiency of curb-opening inlets shorter than the length required for total interception is determined using Figure 1.9.

The length of inlet required for total interception by depressed curb-opening inlets or curb-openings in depressed gutter sections can be found by the use of an equivalent cross slope, S_e, in the following equation:

$$\mathbf{S_e = S_x + S'_w E_o} \qquad \mathbf{(1.14)}$$

where:

E_o = ratio of flow in the depressed section to total gutter flow

S'_w = cross slope of gutter measured from the cross slope of the pavement, S_x (ft/ft)

$S'_w = (a/12W)$

where:

a = gutter depression, in

W = width of depressed gutter, ft

It is apparent from examination of Figure 1.8 that the length of curb opening required for total interception can be significantly reduced by increasing the cross slope or the equivalent cross slope. The equivalent cross slope can be increased by use of a continuously depressed gutter section or a locally depressed gutter section.

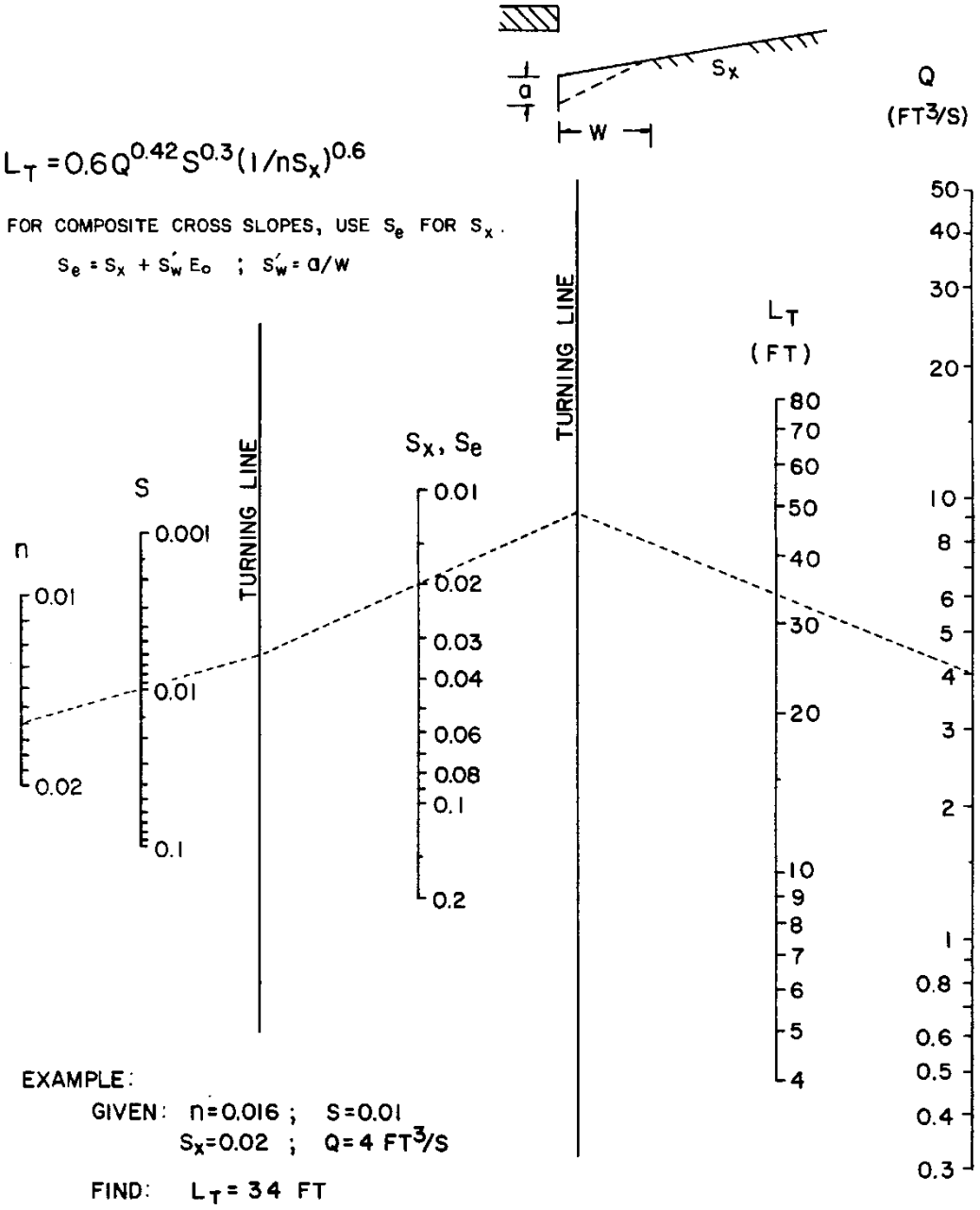


Figure 1.8 Curb-Opening and Slotted Drain Inlet Length for Total Interception
(Source: HEC-12, 1984)

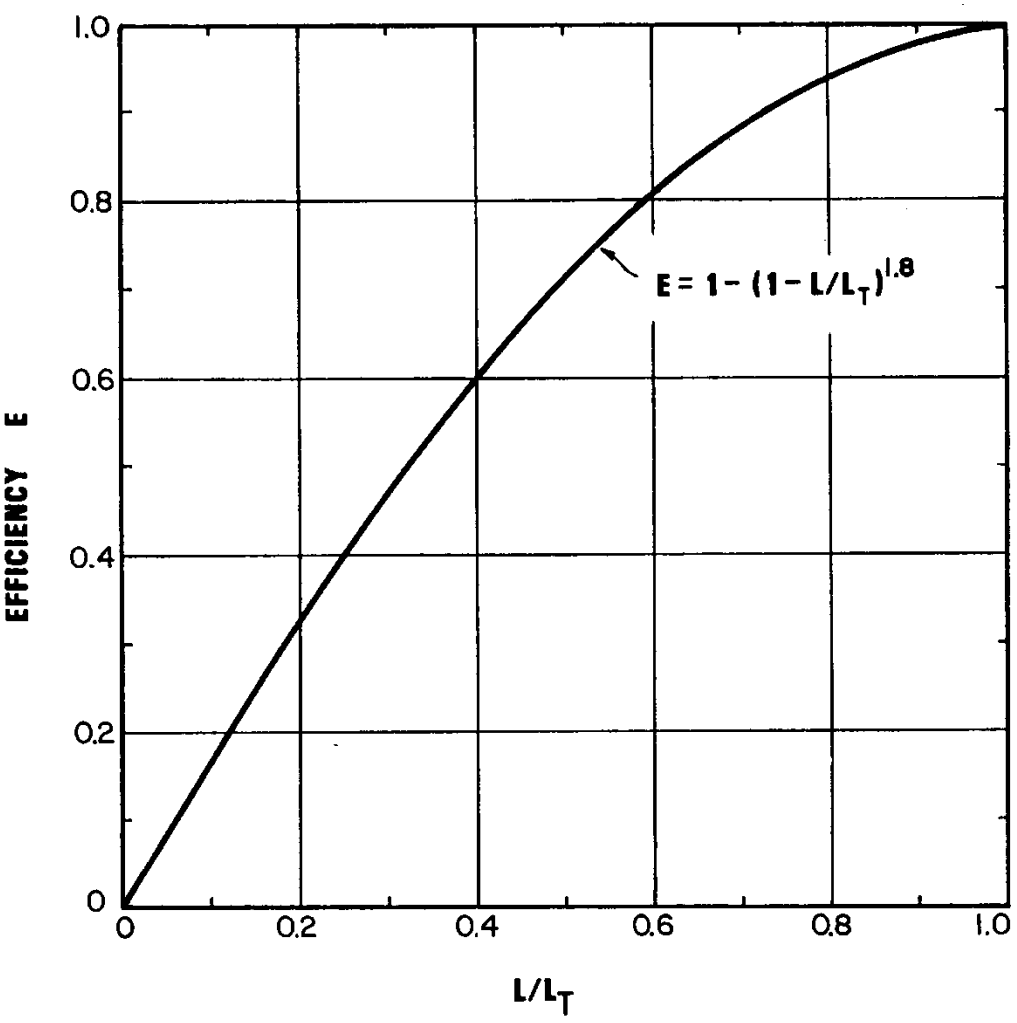


Figure 1.9 Curb-Opening and Slotted Drain Inlet Interception Efficiency
(Source: HEC-12, 1984)

Design Steps

Steps for using Figures 1.8 and 1.9 in the design of curb inlets on grade are given below.

Step 1 Determine the following input parameters:

Cross slope = S_x (ft/ft)

Longitudinal slope = S (ft/ft)

Gutter flow rate = Q (cfs)

Manning's n = n

Spread of water on pavement = T (ft) from Figure 1.2

Step 2 Enter Figure 1.8 using the two vertical lines on the left side labeled n and S. Locate the value for Manning's n and longitudinal slope and draw a line connecting these points and extend this line to the first turning line.

Step 3 Locate the value for the cross slope (or equivalent cross slope) and draw a line from the point on the first turning line through the cross slope value and extend this line to the second turning line.

Step 4 Using the far right vertical line labeled Q locate the gutter flow rate. Draw a line from this value to the point on the second turning line. Read the length required from the vertical line labeled L_T.

Step 5 If the curb-opening inlet is shorter than the value obtained in Step 4, Figure 1.9 can be used to calculate the efficiency. Enter the x-axis with the L/L_T ratio and draw a vertical line upward to the E curve. From the point of intersection, draw a line horizontally to the intersection with the y-axis and read the efficiency value.

Example

Given:

S_x = 0.03 ft/ft
n = 0.016
S = 0.035 ft/ft
Q = 5 cfs
S'_w = 0.083 (a = 2 in, W = 2 ft)

Find:

1. Q_i for a 10-ft curb-opening inlet
2. Q_i for a depressed 10-ft curb-opening inlet with a = 2 in, W = 2 ft, T = 8 ft (Figure 1.2)

Solution:

1. From Figure 1.8, L_T = 41 ft, L/L_T = 10/41 = 0.24
   From Figure 1.9, E = 0.39, Q_i = EQ = 0.39 x 5 = 2 cfs
2. Qn = 5.0 x 0.016 = 0.08 cfs
   S_w/S_x = (0.03 + 0.083)/0.03 = 3.77
   T/W = 3.5 (from Figure 1.4)
   T = 3.5 x 2 = 7 ft
   W/T = 2/7 = 0.29 ft
   E_o = 0.72 (from Figure 1.3)
   Therefore, $S_e = S_x + S'_wE_o$ = 0.03 + 0.083(0.72) = 0.09
   From Figure 1.8, L_T = 23 ft, L/L_T = 10/23 = 0.43
   From Figure 1.9, E = 0.64, Q_i = 0.64 x 5 = 3.2 cfs

The depressed curb-opening inlet will intercept 1.6 times the flow intercepted by the undepressed curb opening and over 60% of the total flow.

Curb Inlets in Sump

For the design of a curb-opening inlet in a sump location, the inlet operates as a weir to depths equal to the curb opening height and as an orifice at depths greater than 1.4 times the opening height. At depths between 1.0 and 1.4 times the opening height, flow is in a transition stage.

The capacity of curb-opening inlets in a sump location can be determined from Figure 1.10, which accounts for the operation of the inlet as a weir and as an orifice at depths greater than 1.4h. This figure is applicable to depressed curb-opening inlets and the depth at the inlet includes any gutter depression. The height (h) in the figure assumes a vertical orifice opening (see sketch on Figure 1.10). The weir portion of Figure 1.10 is valid for a depressed curb-opening inlet when $d \leq (h + a/12)$.

The capacity of curb-opening inlets in a sump location with a vertical orifice opening but without any depression can be determined from Figure 1.11. The capacity of curb-opening inlets in a sump location with other than vertical orifice openings can be determined by using Figure 1.12.

Design Steps

Steps for using Figures 1.10, 1.11, and 1.12 in the design of curb-opening inlets in sump locations are given below.

Step 1 Determine the following input parameters:
Cross slope = S_x (ft/ft)
Spread of water on pavement = T (ft) from Figure 1.2
Gutter flow rate = Q (cfs) or dimensions of curb-opening inlet [L (ft) and H (in)]
Dimensions of depression if any [a (in) and W (ft)]

Step 2 To determine discharge given the other input parameters, select the appropriate Figure (1.10, 1.11, or 1.12 depending on whether the inlet is in a depression and if the orifice opening is vertical).

Step 3 To determine the discharge (Q), given the water depth (d), locate the water depth value on the y-axis and draw a horizontal line to the appropriate perimeter (p), height (h), length (L), or width x length (hL) line. At this intersection draw a vertical line down to the x-axis and read the discharge value.

Step 4 To determine the water depth given the discharge, use the procedure described in Step 3 except enter the figure at the value for the discharge on the x-axis.

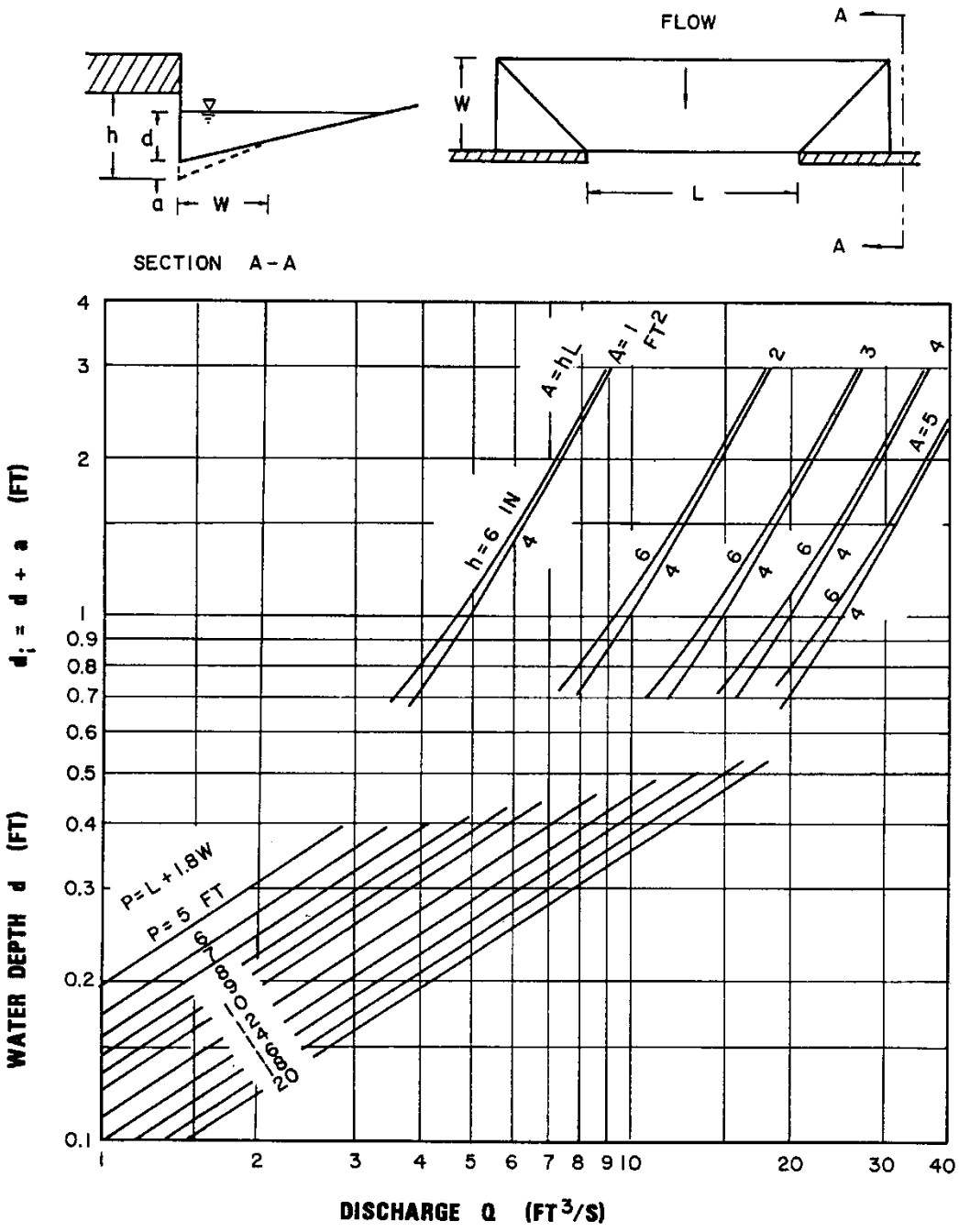


Figure 1.10 Depressed Curb-Opening Inlet Capacity in Sump Locations
(Source: AASHTO Model Drainage Manual, 1991)

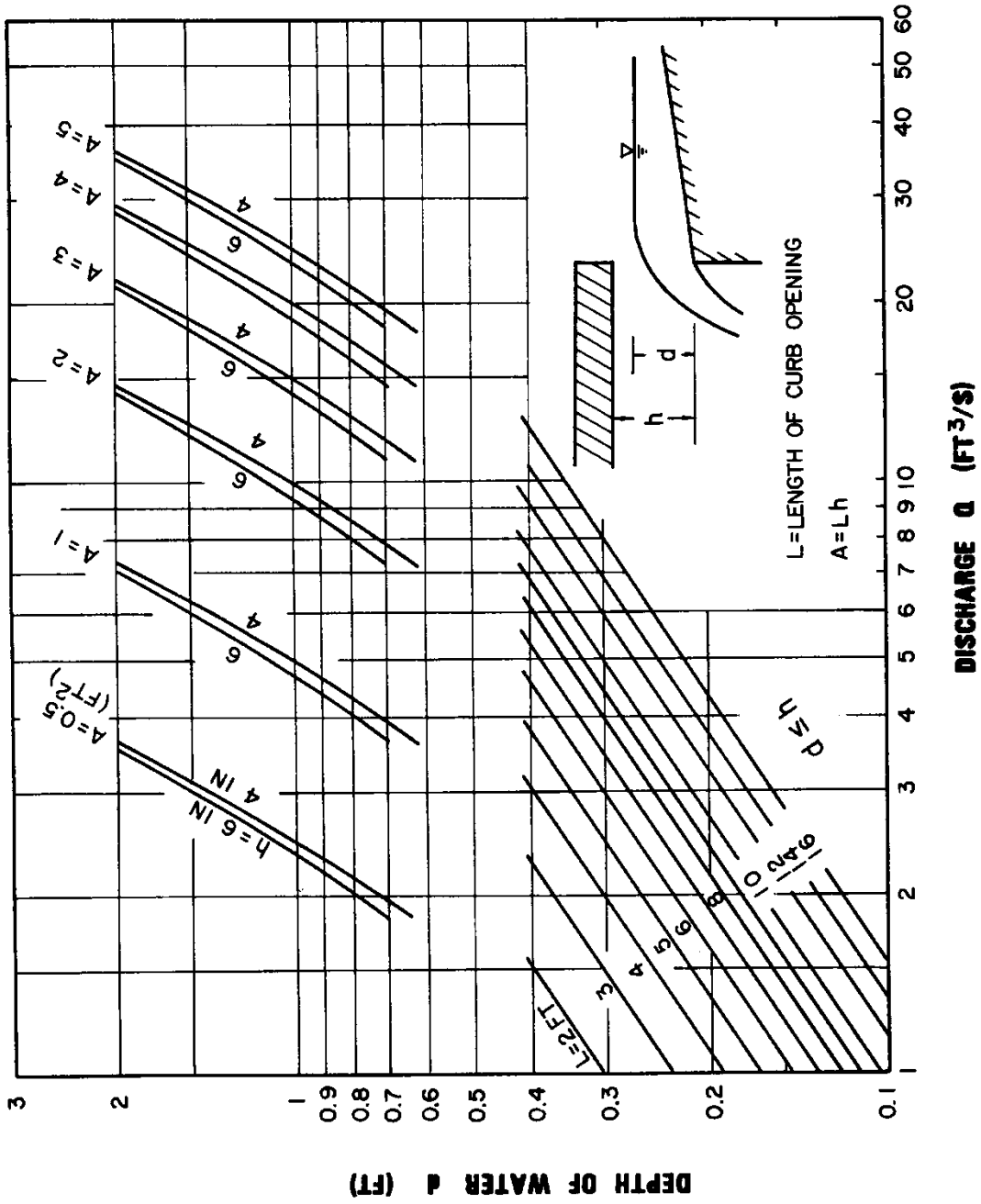


Figure 1.11 Curb-Opening Inlet Capacity in Sump Locations
(Source: AASHTO Model Drainage Manual, 1991)

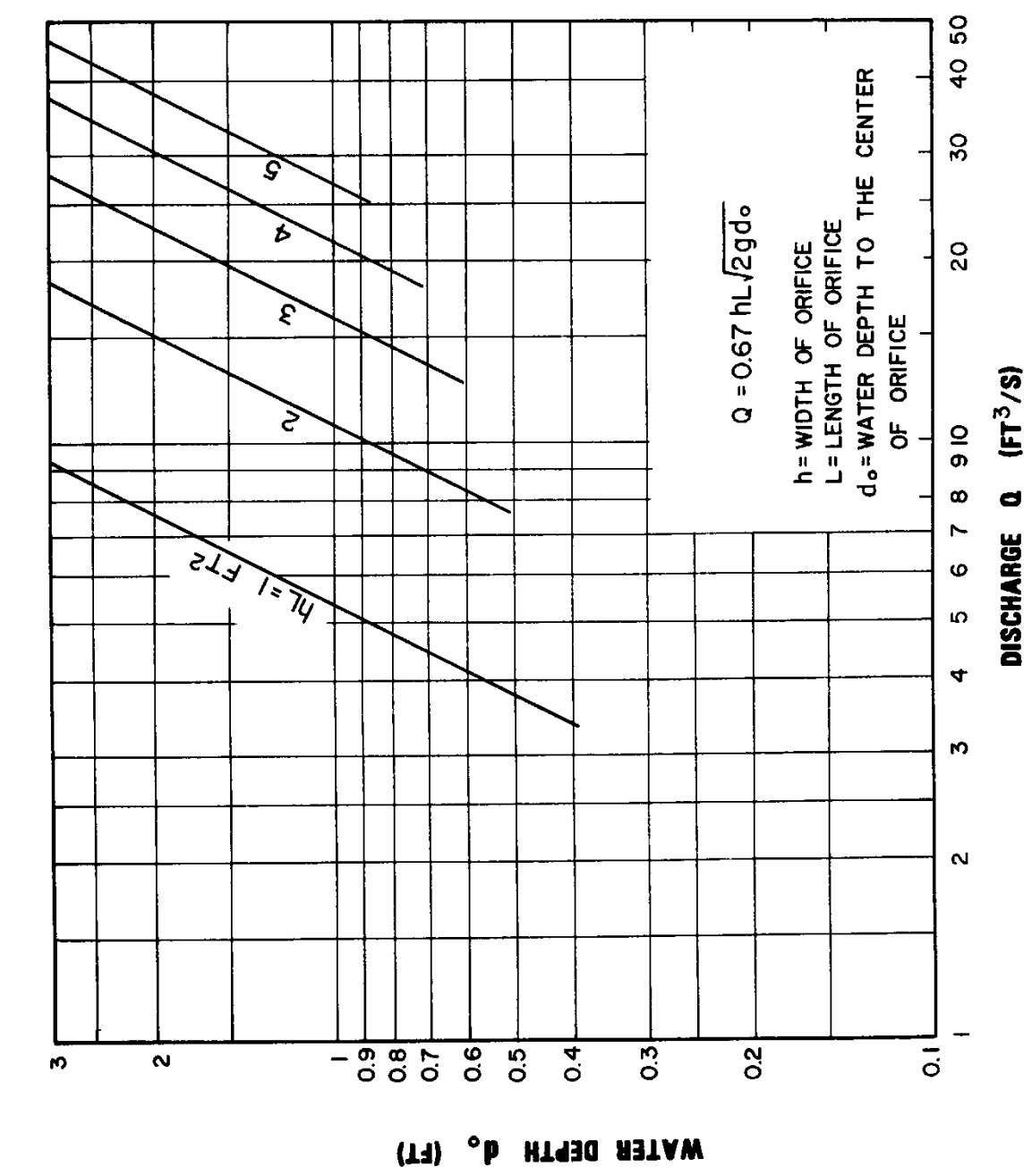


Figure 1.12 Curb-Opening Inlet Orifice Capacity for Inclined and Vertical Orifice Throats
(Source: AASHTO Model Drainage Manual, 1991)

Example:
Given:

Curb-opening inlet in a sump location

L = 5 ft

h = 5 in

1. Undepressed curb opening
   S_x = 0.05 ft/ft
   T = 8 ft
2. Depressed curb opening
   S_x = 0.05 ft/ft
   a = 2 in
   W = 2 ft
   T = 8 ft

Find:

Discharge Q_i

Solution:

1. d = TS_x = 8 x 0.05 = 0.4 ft
   d < h
   From Figure 10, Q_i = 3.8 cfs
2. d = 0.4 ft
   h + a/12 = (5 + 2/12)/12 = 0.43 ft

   since d < 0.43 the weir portion of Figure 1.10 is applicable (lower portion of the figure).

   P = L + 1.8W = 5 + 3.6 = 8.6 ft
   From Figure 1.9, Q_i = 5 cfs

At d = 0.4 ft, the depressed curb-opening inlet has about 30% more capacity than an inlet without depression.

1.2.8 Combination Inlets

Combination Inlets on Grade

On a continuous grade, the capacity of an unclogged combination inlet with the curb opening located adjacent to the grate is approximately equal to the capacity of the grate inlet alone. Thus capacity is computed by neglecting the curb opening inlet and the design procedures should be followed based on the use of Figures 1.5, 1.6, and 1.7.

Combination Inlets in Sump

All debris carried by stormwater runoff that is not intercepted by upstream inlets will be concentrated at the inlet located at the low point, or sump. Because this will increase the probability of clogging for grated inlets, it is generally appropriate to estimate the capacity of a combination inlet at a sump by neglecting the grate inlet capacity. Assuming complete clogging of the grate, Figures 1.10, 1.11, and 1.12 for curb-opening inlets should be used for design.

1.2.9 Closed Conduit Systems (Storm Drains/ Sewers)

Storm drain pipe systems, also known as *storm sewers*, are pipe conveyances used for transporting runoff from roadway and other inlets to outfalls at other structural stormwater controls and receiving waters. Pipe drain systems are suitable mainly for medium to high-density residential and commercial/industrial development where the use of natural drainageways and/or vegetated open channels is not feasible.

Closed conduit system are composed of different lengths and sizes of conduits (system segments) connected by appointment structures (system nodes). Segments are most often circular pipe, but can be a box or other enclosed conduit. Materials used are usually corrugated metal, plastic, and concrete but may be of other materials.

Appurtenant structures serve many functions. Inlets, access holes, and junction chambers are presented in this section.

Inlets

The primary function is to allow surface water to enter the closed conduit system. Inlet structures may also serve as access points for cleaning and inspection. Typical inlets structures are a standard drop inlet, catch basin, curb inlet, combination inlet, and Y inlet. (See Figures 1.13 and 1.14).

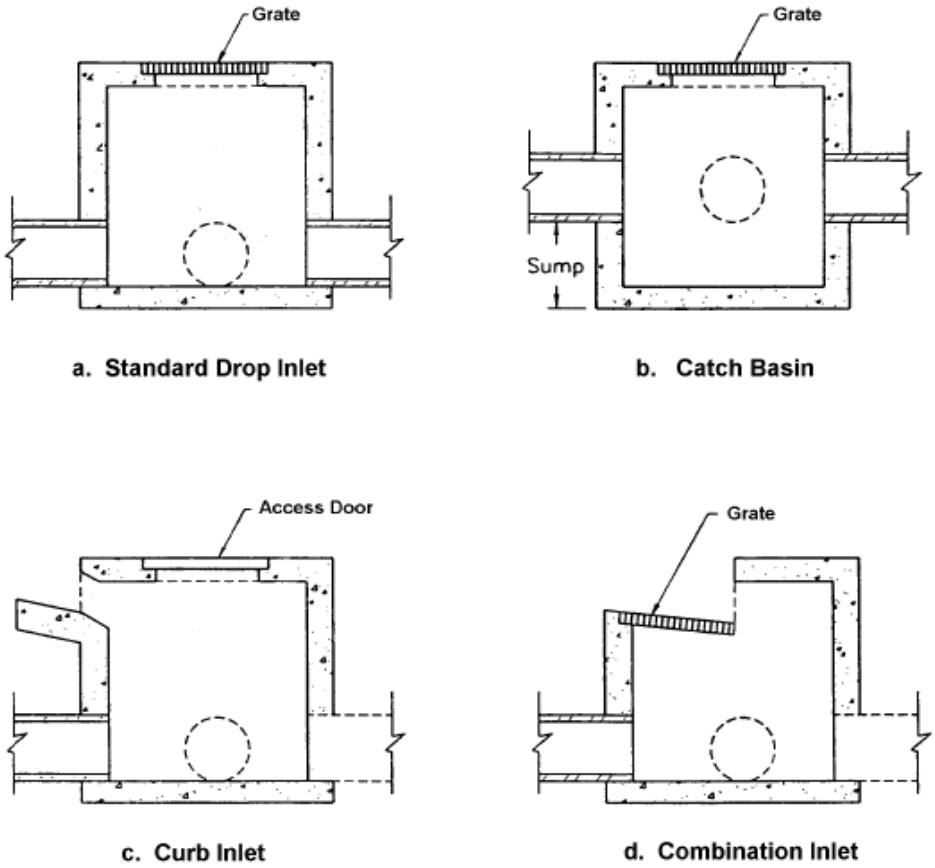


Figure 1.13 Inlet Structures
(HEC 22, 2001)

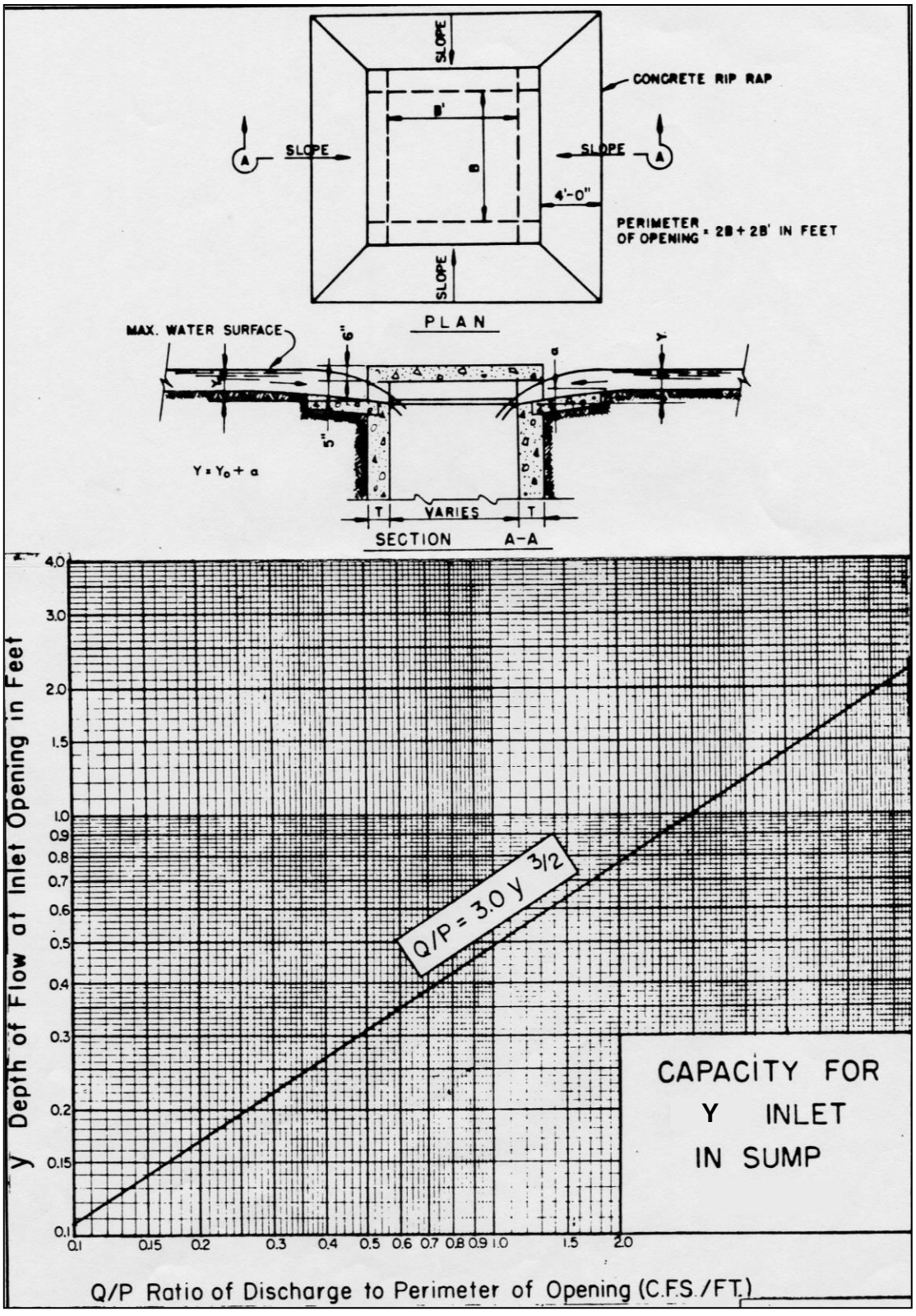


Figure 1.14 Capacity for Y Inlet in Sump
(Fort Worth, 1967)

Inlet structures are located at the upstream end and at intermediate points within the closed conduit system. Inlet placement is generally a trial and error procedure that attempts to produce the most economical and hydraulically effective system (HEC 22, 2001).

Access Holes (Manholes)

The primary function of an access hole is to provide access to the closed conduit system. An access hole can also serve as a flow junction and can provide ventilation and pressure relief. Typical access holes are shown in Figures 1.15 and 1.16 (HEC 22, 2001). The materials commonly used for access hole construction are precast concrete and cast-in-place concrete.

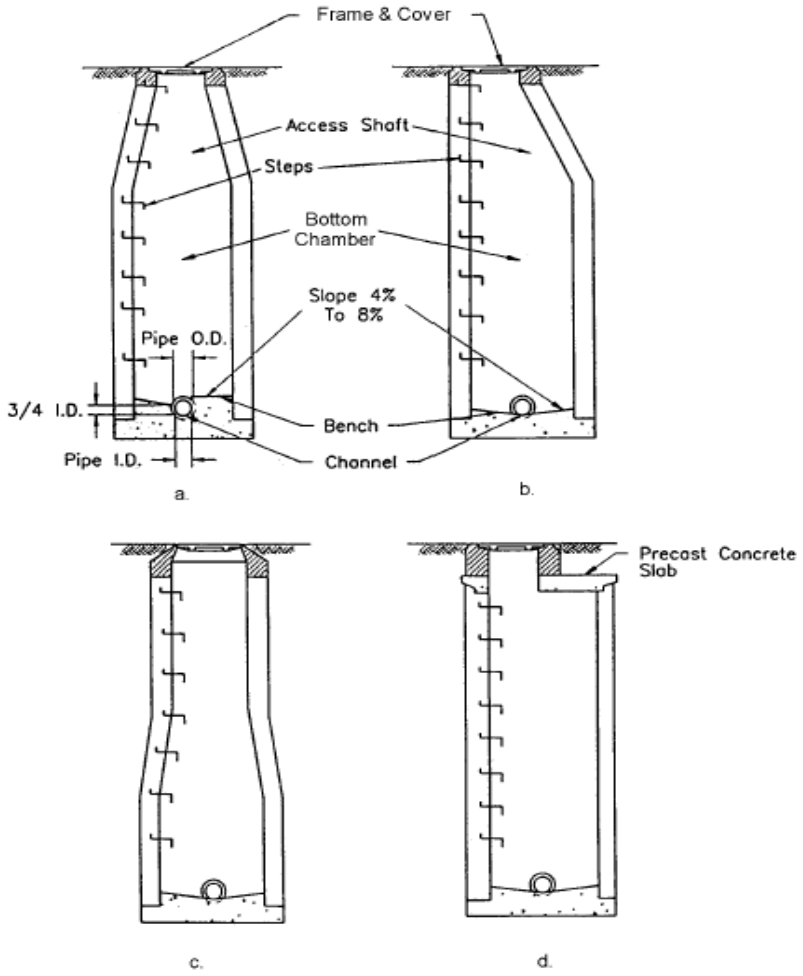


Figure 1.15 Typical Access Hole Configurations.
(HEC22, 2001)

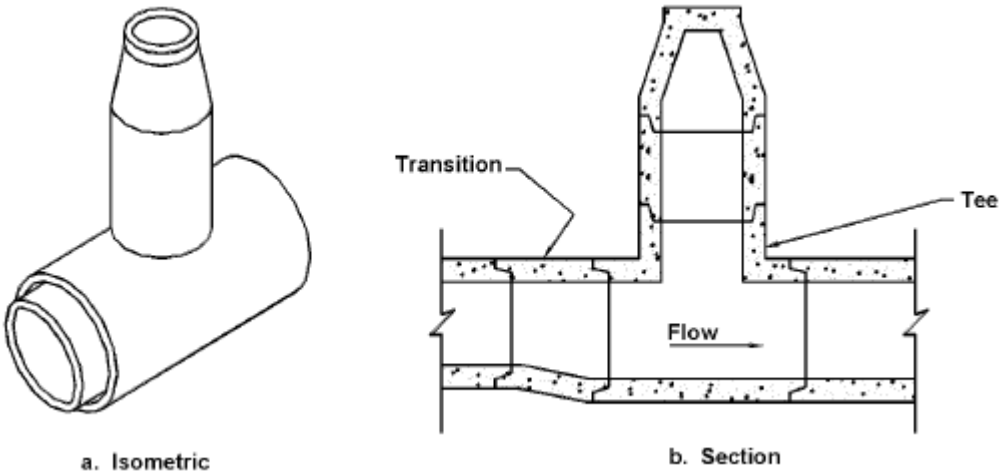


Figure 1.16 "Tee" Access Hole for Large Storm Drains
(HEC 22, 2001)

Junction Chambers

A junction chamber, or junction box, is a special design underground chamber used to join two or more large storm drain conduits. This type of structure is usually required where storm drains are larger than the size that can be accommodated by standard access holes. For smaller diameter storm drains, access holes are typically used instead of junction chambers. Junction chambers by definition do not need to extend to the ground surface and can be completely buried. However, it is recommended that riser structures be used to provide surface access and/or to intercept surface runoff.

Materials commonly used for junction chamber construction include pre-cast concrete and cast-in-place concrete. On storm drains constructed of corrugated steel, the junction chambers are sometimes made of the same material.

To minimize flow turbulence in junction boxes, flow channels and benches are typically built into the bottom of the chambers. Where junction chambers are used as access points for the storm drain system, their location should adhere to the spacing criteria outlined in *Table 3.9 of the Criteria Manua*l.

General Design Procedure

The design of storm drain systems generally follows these steps:

Step 1 Determine inlet location and spacing as outlined earlier in this section.

Step 2 Prepare a tentative plan layout of the storm sewer drainage system including:

a. Location of storm drains
b. Direction of flow
c. Location of manholes
d. Location of existing facilities such as water, gas, or underground cables

Step 3 Determine drainage areas and compute runoff using the Rational Method

Step 4 After the tentative locations of inlets, drain pipes, and outfalls (including tailwaters) have been determined and the inlets sized, compute the rate of discharge to be carried by each storm drain pipe and determine the size and gradient of pipe required to care for this discharge. This is done by proceeding in steps from the upstream end of a line downstream to the point at

which the line connects with other lines or the outfall, whichever is applicable. The discharge for a run is calculated, the pipe serving that discharge is sized, and the process is repeated for the next run downstream. The storm drain system design computation form (Figure 1.31) can be used to summarize hydrologic, hydraulic and design computations.

Step 5 Examine assumptions to determine if any adjustments are needed to the final design.

The rate of discharge at any point in the storm drainage system is not the sum of the inlet flow rates of all inlets above the section of interest. It is generally less that this total. The Rational Method is the most common means of determining design discharges for storm drain design and is explained in *Section 1.2 of the Hydrology Technical Manual*. The designer is usually concerned with two different times of concentration: one for inlet spacing and the other for pipe sizing

The time of concentration for pipe sizing is defined as the time required for water to travel for the most hydraulically distant point in the total contributing watershed to the design point. Typically, this time consists of two components: (1) the time for overland and gutter flow to reach the first inlet, and (2) the time to flow through the storm drainage system to the point of interest. If the total time of concentration to the upstream inlet is less than five minutes, a minimum time of concentration of five minutes is used as the duration of rainfall. The time of concentration for each successive inlet should be determined independently in the same manner as was used for the first inlet.

The flow path having the longest time of concentration to the point of interest in the storm drainage system will usually define the duration used in selecting the intensity value in the Rational Method. Exceptions to the general application of the Rational Equation exist. For example, a small relatively impervious area within a larger drainage area may have an independent discharge higher than that of the total area. This anomaly may occur because of the higher runoff coefficient (C value) and higher intensity resulting from a short time of concentration. If an exception does exist, it can generally be classified as one of two exception scenarios.

The first exception occurs when a highly impervious section exists at the most downstream area of a watershed and the total upstream area flows through the lower impervious area. When this situation occurs, two separate calculations should be made.

- First, calculate the runoff from the total drainage area with its weighed C value and the intensity associated with the longest time of concentration.
- Secondly, calculate the runoff using only the smaller less pervious area. The typical procedure would be followed using the C value for the small less pervious area and the intensity associated with the shorter time of concentration.

The results of these two calculations should be compared and the largest value of discharge should be used for design.

The second exception exists when a smaller less pervious area is tributary to the larger primary watershed. When this scenario occurs, two sets of calculations should also be made.

- First, calculate the runoff form the total drainage area with its weighted C value and the intensity associated with the longest time of concentration.
- Secondly, calculate the runoff to consider how much discharge from the larger primary area is contributing at the same time the peak from the smaller less pervious tributary area is occurring. When the small area is discharging, some discharge from the larger primary area is also contributing to the total discharge. In this calculation, the intensity associated with the time of concentration from the small less pervious area is used. The portion of the larger primary area to be considered is determined by the following: $A_c=A\ (t_{c1}/t_{c2})$.

A_c is the most downstream part of the larger primary area that will contribute to the discharge during the time of concentration associated with the smaller, less pervious area. A is the area of the larger primary area, t_{c1} is the time of concentration of the smaller, less pervious, tributary area, and t_{c2} is the time of concentration associated with the larger primary area as is used in the first calculation. The C value to be used in this computation should be the weighted C value of the smaller less pervious tributary area and the area A_c. The area to be used in the Rational Method would be the area of the less pervious area plus A_c. The second calculation should only be considered when the less pervious area is tributary to the area with the longer time of concentration and is at or near the downstream end of the total drainage area.

Finally, the results of these calculations should be compared and the largest value of discharge should be used for design.

Capacity Calculations

The design procedures presented here assume flow within each storm drain segment is steady and uniform. This means the discharge and flow depth in each segment are assumed to be constant with respect to time and distance. Also, since storm drain conduits are typically prismatic, the average velocity throughout a segment is considered to be constant.

In actual storm drainage systems, the flow at each inlet is variable, and flow conditions are not truly steady or uniform. However, since the usual hydrologic methods employed in storm drain design are based on computed peak discharges at the beginning of each run, it is a conservative practice to design using the steady uniform flow assumption.

Although at times flow in a closed conduit may be under pressure or at other times the conduit may flow partially full, the usual design assumption is that the conduit is flowing fill but not under pressure. Under this assumption the rate of head loss is the same as the slope of the pipe (S_f=S, ft/ft). Designing for full flow is a conservative assumption since the peak flow actually occurs at 93 percent of full flow.

The most widely used formula for determining the hydraulic capacity of storm drain pipes for gravity and pressure flows is the Manning's Formula, expressed by the following equation:

$$\mathbf{V = (1.486/n)\ R^{2/3}\ S^{1/2}} \qquad \mathbf{(1.15)}$$

where:

- V = mean velocity of flow, ft/s
- R = the hydraulic radius, ft - defined as the area of flow divided by the wetted flow surface or wetted perimeter (A/WP)
- S = the slope of hydraulic grade line, ft/ft
- n = Manning's roughness coefficient

In terms of discharge, the above formula becomes:

$$\mathbf{Q = (1.486/n)\ A\ R^{2/3}\ S^{1/2}} \qquad \mathbf{(1.16)}$$

where:

- Q = rate of flow, cfs
- A = cross sectional area of flow, ft^2

For pipes flowing full, the area is $(\pi/4)D^2$ and the hydraulic radius is D/4, so, the above equations become:

$$\mathbf{V = [0.590\ D^{2/3}S^{1/2}]/n} \qquad \mathbf{(1.17)}$$

$$\mathbf{Q = [0.463\ D^{8/3}S^{1/2}]/n} \qquad \mathbf{(1.18)}$$

where:

- D = diameter of pipe, ft
- S = slope of the pipe = S_f hydraulic grade line, ft/ft

The Manning's Equation can be written to determine friction losses for storm drain pipes as:

$$H_f = [0.453\ n^2V^2L]/[R^{4/3}] \quad (1.19)$$

$$H_f = [(2.87\ n^2V^2L]/[D^{4/3}] \quad (1.20)$$

$$H_f = [(185n^2(V^2/2g)L]/[D^{4/3}] \quad (1.21)$$

where:

- H_f = total head loss due to friction, ft (S_f x L)
- n = Manning's roughness coefficient
- D = diameter of pipe, ft
- L = length of pipe, ft
- V = mean velocity, ft/s
- R = hydraulic radius, ft
- g = acceleration of gravity = 32.2 ft/sec^2

A nomograph solution of Manning's Equation for full flow in circular conduits is presented in Figure 1.17. Representative values of the Manning's coefficient for various storm drain materials are provided in Table 1.8. It should be remembered that the values in the table are for new pipe tested in a laboratory. Actual field values for conduits may vary depending on the effect of abrasion, corrosion, deflection, and joint conditions.

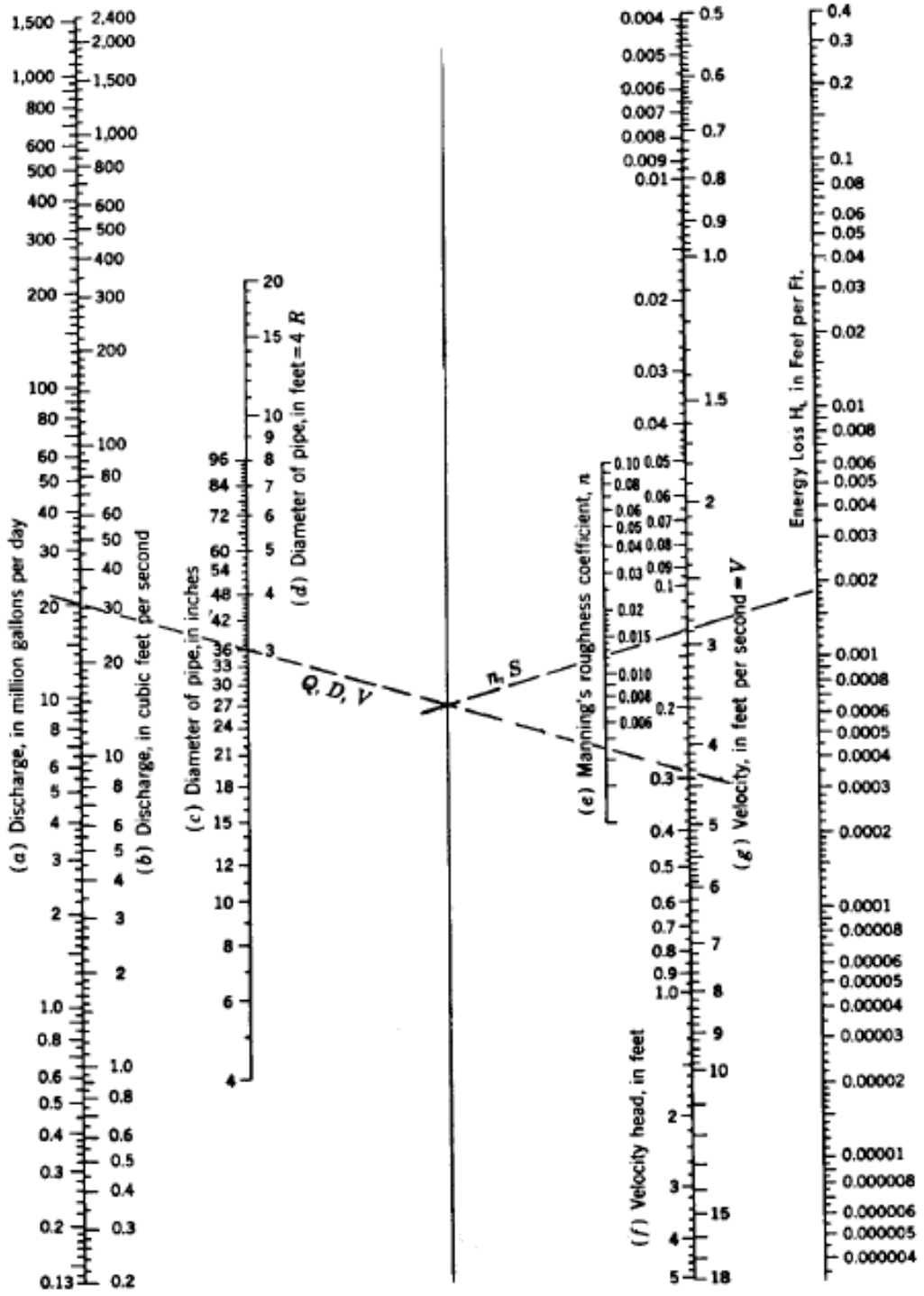
Figure 1.18 illustrates storm drain capacity sensitivity to the parameters in the Manning's Equation. This figure can be used to study the effect changes in individual parameters will have on storm drain capacity. For example, if the diameter of a storm drain is doubled, its capacity will be increased by a factor of 6.0; if the slope is doubled, the capacity is increased by a factor of 1.4; however, if the roughness is doubled, the pipe capacity will be reduced by 50 percent.

The hydraulic elements graph in Figures 1.19a and 1.19b is provided to assist in the solution of the Manning's Equation for part full flow in storm drains. The hydraulic elements chart shows the relative flow conditions at different depths in a circular pipe and makes the following important points:

1. Peak flow occurs at 93 percent of the height of the pipe. This means that if the pipe is designed for full flow, the design will be slightly conservative.
2. The velocity in a pipe flowing half-full is the same as the velocity for full flow.
3. Flow velocities for flow depths greater than half-full are greater than velocities at full flow.
4. As the depth of flow drops below half-full, the flow velocity drops off rapidly. The shape of a storm drain conduit also influences its capacity. Although most storm drain conduits are circular, a significant increase in capacity can be realized by using an alternate shape. Table 1.7 provides a tabular listing of the increase in capacity which can be achieved using alternate conduit shapes that have the same height as the original circular shape, but have a different cross sectional area. Although these alternate shapes are generally more expensive than circular shapes, their use can be justified in some instances based on their increased capacity.

Table 1.7 Increase in Capacity of Alternate Conduit Shapes Based on a Circular Pipe with the Same Height
(HEC-22, 2001)

	Area (Percent Increase)	Conveyance (Percent Increase)
Circular	**--**	**--**
Oval	**63**	**87**
Arch	**57**	**78**
Box (B = D)	**27**	**27**



Alignment chart for energy loss in pipes, for Manning's formula.
Note: Use chart for flow computations, $H_L = S$

Figure 1.17 Solution of Manning's Equation for Flow in Storm Drains-English Units
(Taken from "Modern Sewer Design" by American Iron and Steel Institute)

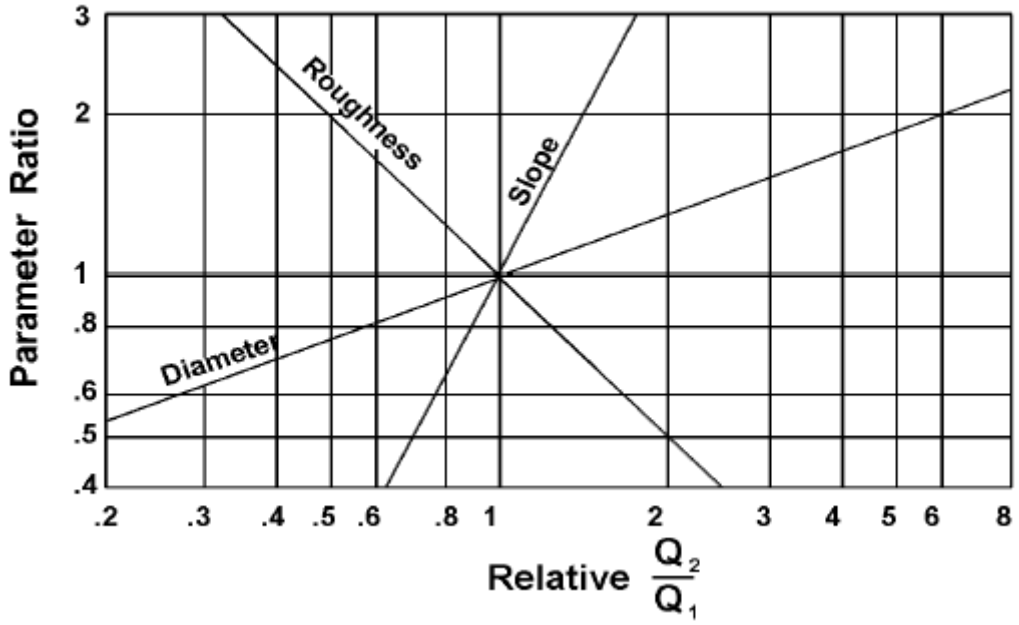


Figure 1.18 Storm Drain Capacity Sensitivity
(HEC 22, 2001)

Table 1.8 Manning's Coefficients for Storm Drain Conduits (HEC 22, 2001)		
Type of Culvert	Roughness or Corrugation	Manning's n
Concrete Pipe	**Smooth**	**0.010-0.011**
Concrete Boxes	**Smooth**	**0.012-0.015**
Spiral Rib Metal Pipe	**Smooth**	**0.012-0.013**
Corrugated Metal Pipe, Pipe-Arch and Box (Annular or Helical Corrugations -- see Figure B-3 in Reference 2, Manning's n varies with barrel size)	**68 by 13 mm 2-2/3 by 1/2 in Annular**	**0.022-0.027**
	68 by 13 mm 2-2/3 by 1/2 in Helical	**0.011-0.023**
	150 by 25 mm 6 by 1 in Helical	**0.022-0.025**
	125 by 25 mm 5 by 1 in	**0.025-0.026**
	75 by 25 mm 3 by 1 in	**0.027-0.028**
	150 by 50 mm 6 by 2 in Structural Plate	**0.033-0.035**
	230 by 64 mm 9 by 2-1/2 in Structural Plate	**0.033-0.037**
Corrugated Polyethylene	**Smooth**	**0.009-0.015**
Corrugated Polyethylene	**Corrugated**	**0.018-0.025**
Polyvinyl chloride (PVC)	**Smooth**	**0.009-0.011**
***NOTE: The Manning's n values indicated in this table were obtained in the laboratory and are supported by the provided reference. Actual field values for culverts may vary depending on the effect of abrasion, corrosion, deflection, and joint conditions.**		

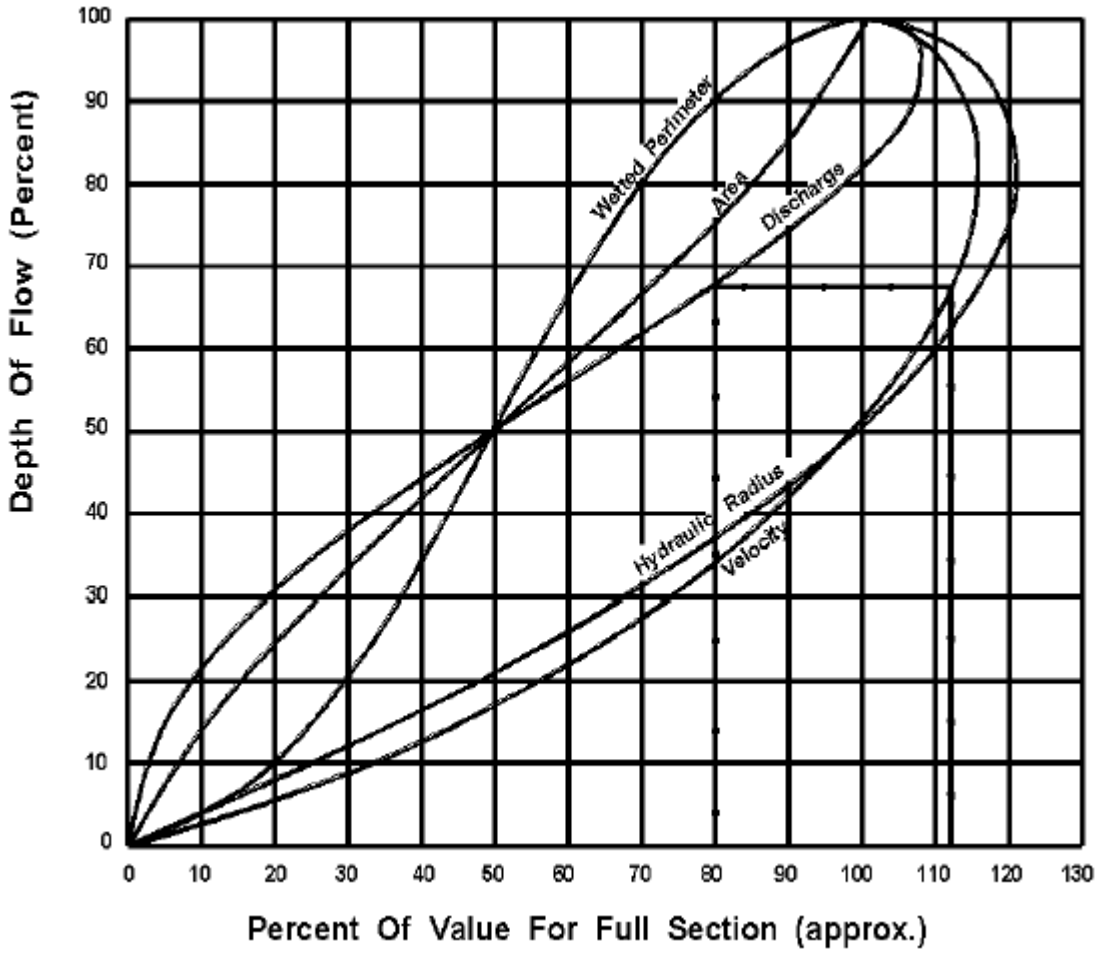


Figure 1.19a Hydraulic Elements of Circular Section
(HEC 22, 2001)

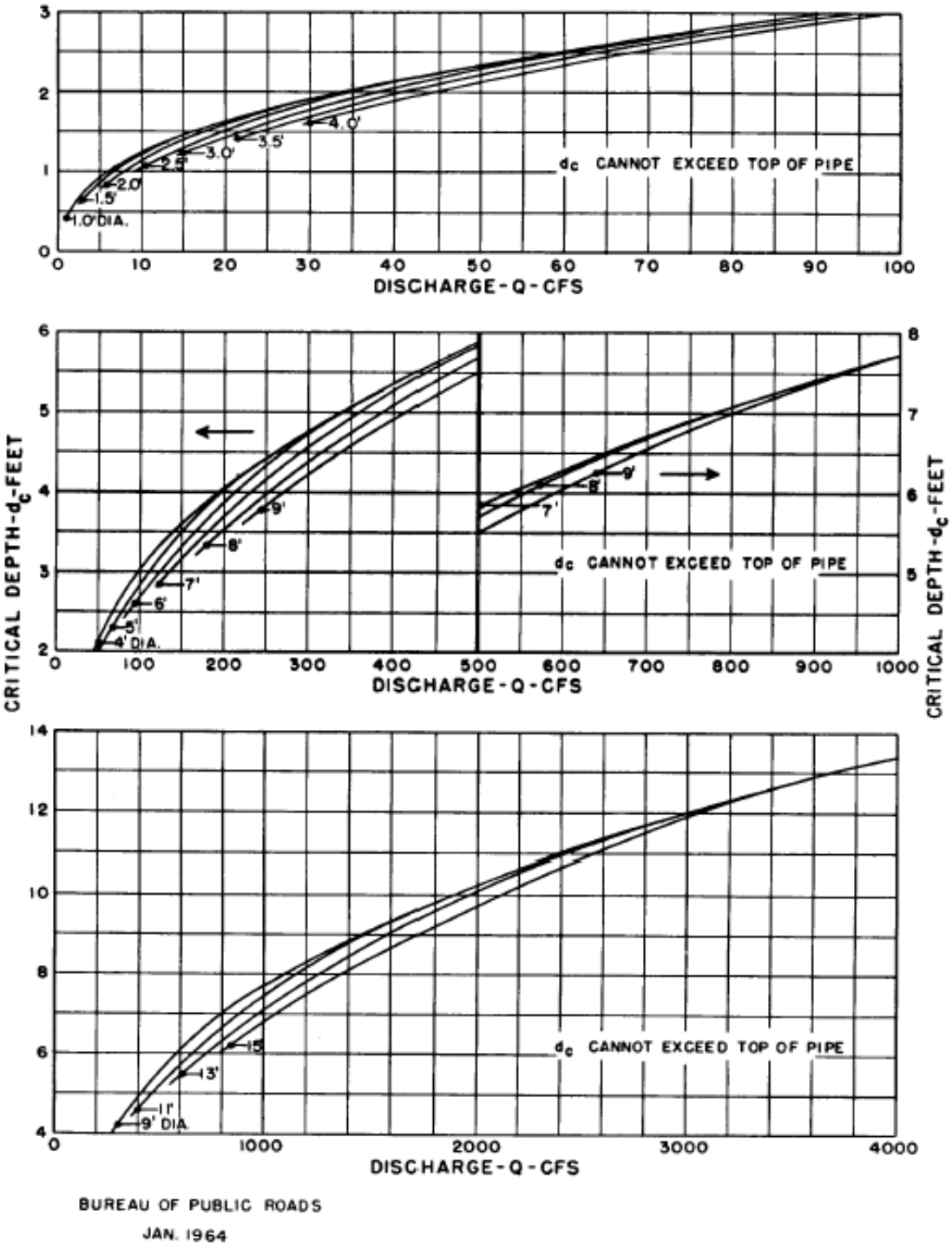


Figure 1.19b Critical Depth in Circular Pipe-English Units
(HEC 22, 2001)

Minimum Grades and Desirable Velocities

The minimum slopes are calculated by the modified Manning's formula:

$$S = [(nV)^2]/[2.208R^{4/3}] \quad (1.22)$$

where:

S = the slope of the hydraulic grade line, ft/ft
n = Manning's roughness coefficient
V = mean velocity of flow, ft/s
R = hydraulic radius, ft (area dived by wetted perimeter)

For circular conduits flowing full but not under pressure, R=D/4, and the hydraulic grade line is equal to the slope of the pipe. For these conditions Equation 1.22 may be expressed as:

$$S = 2.87(nV)^2/D^{4/3} \quad (1.23)$$

For a minimum velocity of 2.5 fps, the minimum slope equation becomes:

$$S = 17.938(n^2/D^{4/3}) \quad (1.24)$$

where:

D = diameter, ft

Table 1.9 gives minimum slopes for two commonly used materials: concrete pipe with an n-value of 0.013 and corrugated metal pipe with an n-value of 0.024.

Minimum Grades

Storm drains should operate with velocities of flow sufficient to prevent excessive deposits of solid materials; otherwise objectionable clogging may result. The controlling velocity is near the bottom of the conduit and considerably less than the mean velocity of the sewer. Storm drains shall be designed to have a minimum mean velocity flowing full of 2.5 fps. Table 1.9 gives minimum slopes for two commonly used materials: concrete pipe (n = 0.013) and corrugated metal pipe (n = 0.024), flowing at 2.5 fps.

Table 1.9 Minimum Grades for Storm Drains for 2.5 fps

Pipe Size (inches)	Concrete Pipe (n = 0.013) Slope ft/ft	Corrugated Metal Pipe (n = 0.024) Slope ft/ft
15	0.0023	0.0077
18	0.0018	0.0060
21	0.0014	0.0049
24	0.0012	0.0041
27	0.0010	0.0035
30	0.0009	0.0030
33	0.0008	0.0027
36	0.0007	0.0024
39	0.0006	0.0021
42	0.0006	0.0020
45	0.0005	0.0018
48	0.0005	0.0016
54	0.0004	0.0014
60	0.0004	0.0012
66	0.0003	0.0011
72	0.0003	0.0010
78	0.0003	0.0009
84	0.0002	0.0008
96	0.0002	0.0006

Storm Drain Storage

If downstream drainage facilities are undersized for the design flow, a structural stormwater control may be needed to reduce the possibility of flooding. The required storage volume can also be provided by using larger than needed storm drain pipe sizes and restrictors to control the release rates at manholes and/or junction boxes in the storm drain system. The same design criteria for sizing structural control storage facilities are used to determine the storage volume required in the system (see *Section 2.0* for more information).

1.2.10 Energy Grade Line/Hydraulic Grade Line

The energy grade line (EGL) is an imaginary line that represents the total energy along a channel or conduit carrying water. Total energy includes elevation head, velocity head and pressure head.

$$E = V^2/2g + p/\gamma + z \qquad (1.25)$$

where:

E = Total energy, ft

$V^2/2g$ = Velocity head, ft (kinetic energy)

p = Pressure, lbs/ft^2

γ = Unit weight of water, 62.4 lbs/ft^3

p/γ = Pressure head, ft (potential energy)

z = Elevation head, ft (potential energy)

Bernoulli's Law expressed between points one (1) and two (2) in a closed conduit accounts for all energy forms and energy losses. The general form of the law may be written as:

$$V_1^2/2g + p_1/\gamma + z_1 = V_2^2/2g + p_2/\gamma + z_2 - H_f - \Sigma H_m \quad (1.26)$$

where:

H_f = Pipe friction loss, ft

ΣH_m = Sum of minor or form losses, ft

The calculation of the EGL for the full length of the system is critical to the evaluation of a storm drain. In order to develop the EGL it is necessary to calculate all of the losses through the system. The energy equation states that the energy head at any cross section must equal that in any other downstream section plus the intervening losses. The intervening losses are typically classified as either friction losses or form losses. The friction losses can be calculated using the Manning's Equation. Form losses are typically calculated by multiplying the velocity head by a loss coefficient, K. Various tables and calculations exist for developing the value of K depending on the structure being evaluated for loss. Knowledge of the location of the EGL is critical to the understanding and estimating the location of the hydraulic grade line (HGL).

The hydraulic grade line (HGL) is a line coinciding with the level of flowing water at any point along an open channel. In closed conduits flowing under pressure, the hydraulic grade line is the level to which water would rise in a vertical tube at any point along the pipe. The hydraulic grade line is used to aid the designer in determining the acceptability of a proposed storm drainage system by establishing the elevation to which water will rise when the system is operating under design conditions.

HGL, a measure of flow energy, is determined by subtracting the velocity head ($V^2/2g$) from the EGL. Energy concepts can be applied to pipe flow as well as open channel flow. Figure 1.20 illustrates the energy and hydraulic grade lines for open channel and pressure flow in pipes.

When water is flowing through the pipe and there is a space of air between the top of the water and the inside of the pipe, the flow is considered as open channel flow and the HGL is at the water surface. When the pipe is flowing full under pressure flow, the HGL will be above the crown of the pipe. When the flow in the pipe just reaches the point where the pipe is flowing full, this condition lies in between open channel flow and pressure flow. At this condition the pipe is under gravity full flow and the flow is influenced by the resistance of the total circumference of the pipe. Under gravity full flow, the HGL coincides with the crown of the pipe.

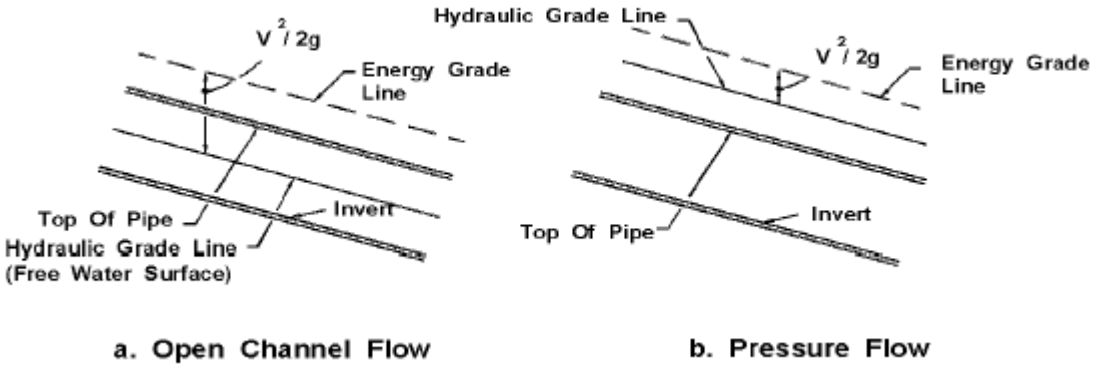


Figure 1.20 Hydraulic and Energy Grade Lines in Pipe Flow
(HEC 22, 2001)

Inlet surcharging and possible access hole lid displacement can occur if the hydraulic grade line rises above the ground surface. A design based on open channel conditions must be carefully planned as well, including evaluation of the potential for excessive and inadvertent flooding created when a storm event larger than the design storm pressurizes the system. As hydraulic calculations are performed, frequent verification of the existence of the desired flow condition should be made. Storm drainage systems can often alternate between pressure and open channel flow conditions from one section to another.

A detailed procedure for evaluating the energy grade line and the hydraulic grade line for storm drainage systems is presented in *Section 1.2.11*.

Storm Drain Outfalls

All storm drains have an outlet where flow from the storm drainage system is discharged. The discharge point can be a natural river or stream, an existing storm drainage system, or a channel which is either existing or proposed for the purpose of conveying the stormwater. The procedure for calculating the energy grade line through a storm drainage system begins at the outfall. Therefore, consideration of outfall conditions is an important part of storm drain design.

Several aspects of outfall design must be given serious consideration. These include the flowline or invert (inside bottom) elevation of the proposed storm drain outlet, tailwater elevations, the need for energy dissipation, and the orientation of the outlet structure.

The flowline or invert elevation of the proposed outlet should be equal to or higher than the flowline of the outfall. If this is not the case, there may be a need to pump or otherwise lift the water to the elevation of the outfall.

The tailwater depth or elevation in the storm drain outfall must be considered carefully. Evaluation of the hydraulic grade line for a storm drainage system begins at the system outfall with the tailwater elevation. For most design applications, the tailwater will either be above the crown of the outlet or can be considered to be between the crown and critical depth of the outlet. The tailwater may also occur between the critical depth and the invert of the outlet. However, the starting point for the hydraulic grade line determination should be either the design tailwater elevation or the average of critical depth and the height of the storm drain conduit, $(d_c + D)/2$, whichever is greater.

An exception to the above rule would be for a very large outfall with low tailwater where a water surface profile calculation would be appropriate to determine the location where the water surface will intersect the top of the barrel and full flow calculations can begin. In this case, the downstream water surface elevation would be based on critical depth or the design tailwater elevation, whichever was highest.

If the outfall channel is a river or stream, it may be necessary to consider the joint or coincidental probability of two hydrologic events occurring at the same time to adequately determine the elevation of the tailwater in the receiving stream. The relative independence of the discharge from the storm drainage system can be qualitatively evaluated by a comparison of the drainage area of the receiving stream to the area of the storm drainage system. For example, if the storm drainage system has a drainage area much smaller than that of the receiving stream, the peak discharge from the storm drainage system may be out of phase with the peak discharge from the receiving watershed. Table 1.10 provides a comparison of discharge frequencies for coincidental occurrence for the 2-, 5-, 10-, 25-, 50-, and flood mitigation design storms. This table can be used to establish an appropriate design tailwater elevation for a storm drainage system based on the expected coincident storm frequency on the outfall channel. For example, if the receiving stream has a drainage area of 200 acres and the storm drainage system has a drainage area of 2 acres, the ratio of receiving area to storm drainage area is 200 to 2 which equals 100 to 1. From Table 1.10 and considering a 10-year design storm occurring over both areas, the flow rate in the main stream will be equal to that of a five year storm when the drainage system flow rate reaches its 10-year peak flow at the outfall. Conversely, when the flow rate in the main channel reaches its 10-year peak flow rate, the flow rate from the storm drainage system will have fallen to the 5- year peak flow rate discharge. This is because the drainage areas are different sizes, and the time to peak for each drainage area is different.

Table 1.10 Frequencies for Coincidental Occurrences
(TxDOT, 2002)

Area ratio	2-year design		5-year design	
	Main Stream	Tributary	Main Stream	Tributary
10,000:1	1	2	1	5
	2	1	5	1
1,000:1	1	2	2	5
	2	1	5	2
100:1	2	2	2	5
	2	2	5	5
10:1	2	2	5	5
	2	2	5	5
1:1	2	2	5	5
	2	2	5	5
Area ratio	**10-year design**		**25-year design**	
	Main Stream	Tributary	Main Stream	Tributary
10,000:1	1	10	2	25
	10	1	25	2
1,000:1	2	10	5	25
	10	2	25	5
100:1	5	10	10	25
	10	5	25	10
10:1	10	10	10	25
	10	10	25	10
1:1	10	10	25	25
	10	10	25	25
Area ratio	**50-year design**		**100-year design**	
	Main Stream	Tributary	Main Stream	Tributary
10,000:1	2	50	2	100
	50	2	100	2
1,000:1	5	50	10	100
	50	5	100	10
100:1	10	50	25	100
	50	10	100	25
10:1	25	50	50	100
	50	25	100	50
1:1	50	50	100	100
	50	50	100	100

There may be instances in which an excessive tailwater causes flow to back up the storm drainage system and out of inlets and access holes, creating unexpected and perhaps hazardous flooding conditions. The potential for this should be considered. Flap gates placed at the outlet can sometimes alleviate this condition; otherwise, it may be necessary to isolate the storm drain from the outfall by use of a pump station.

Energy dissipation may be required to protect the storm drain outlet. Protection is usually required at the outlet to prevent erosion of the outfall bed and banks. Riprap aprons or energy dissipators should be provided if high velocities are expected. See *Section 4.0* for guidance on design of Energy Dissipation Structures.

The **orientation of the outfall** is another important design consideration. Where practical, the outlet of the storm drain should be positioned in the outfall channel so that it is pointed in a downstream direction. This will reduce turbulence and the potential for excessive erosion. If the outfall structure can not be oriented in a downstream direction, the potential for outlet scour must be considered. For example, where a storm drain outfall discharges perpendicular to the direction of flow of the receiving channel, care must be taken to avoid erosion on the opposite channel bank. If erosion potential exists, a channel bank lining of riprap or other suitable material should be installed on the bank. Alternatively, an energy dissipator structure could be used at the storm drain outlet.

Energy Losses

Prior to computing the hydraulic grade line, all energy losses in pipe runs and junctions must be estimated. In addition to the principal energy involved in overcoming the friction in each conduit run, energy (or head) is required to overcome changes in momentum or turbulence at outlets, inlets, bends, transitions, junctions, and access holes. The following presents relationships for estimating typical energy losses in storm drainage systems. The application of some of these relationships is included in the design example in *Section 1.2.12*.

Pipe Friction Losses

The major loss in a storm drainage system is the friction or boundary shear loss. The head loss due to friction in a pipe is computed as follows:

$$\mathbf{H_f = S_f L} \qquad \mathbf{(1.27)}$$

where:

H_f = friction loss, ft

S_f = friction slope, ft/ft

L = length of pipe, ft

The friction slope in Equation 1.27 is also the slope of the hydraulic gradient for a particular pipe run. As indicated by Equation 1.27, the friction loss is simply the hydraulic gradient multiplied by the length of the run. Since this design procedure assumes steady uniform flow in open channel flow, the friction slope will match the pipe slope for part full flow. Pipe friction losses for full flow can be determined by the use of Equation 1.20.

Exit Losses

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

$$\mathbf{H_o = 1.0\ [(V_o^2/2g) - (V_d^2/2g)]} \qquad \mathbf{(1.28)}$$

where:

V_o = average outlet velocity

V_d = channel velocity downstream of outlet

Note that when $V_d = 0$, as in a reservoir, the exit loss is one velocity head. For part full flow where the pipe outlets in a channel with water moving in the same direction as the outlet water, the exit loss may be reduced to virtually zero.

Bend Losses

The bend loss coefficient for storm drain design is minor but can be estimated using the following formula (AASHTO, 1991):

$$h_b = 0.0033\ (\Delta)\ (V^2/2g) \qquad (1.29)$$

where:

Δ = angle of curvature in degrees

Transition Losses

A transition is a location where a conduit or channel changes size. Typically, transitions should be avoided and access holes should be used when pipe size increases. However, sometimes transitions are unavoidable. Transitions include expansions, contractions, or both. In small storm drains, transitions may be confined within access holes. However, in larger storm drains or when a specific need arises, transitions may occur within pipe runs as illustrated in Figures 1.16 and 1.21.

Energy losses in expansions or contractions in non-pressure flow can be expressed in terms of the kinetic energy at the two ends. Contraction and expansion losses can be evaluated with Equations 1.30 and 1.31 respectively.

$$H_c = K_c\ [V_1^2/(2g) - V_2^2/(2g)] \qquad (1.30)$$

$$H_e = K_e\ [V_1^2/(2g) - V_2^2/(2g)] \qquad (1.31)$$

where:

K_e = expansion coefficient

K_c = contraction coefficient (0.5 K_e)

V_1 = velocity upstream of transition

V_2 = velocity downstream of transition

g = acceleration due to gravity (32.2 ft/s_2)

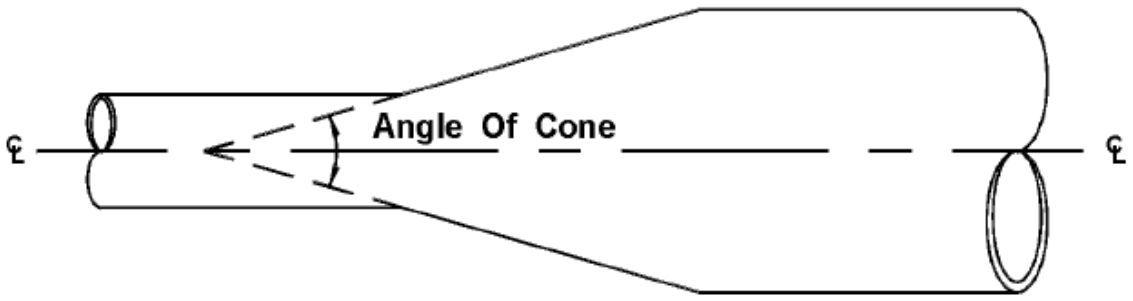


Figure 1.21 Angle of Cone for Pipe Diameter Changes

For gradual contractions, it has been observed that K_c = 0.5 K_e. Typical values of K_e for gradual enlargements are tabulated in Table 1.11a. Typical values of K_c for sudden contractions are tabulated in Table 1.11b. The angle of the cone that forms the transition is defined in Figure 1.21.

Table 1.11a Typical Values for K_e for Gradual Enlargement of Pipes in Non-Pressure Flow							
D_2/D_1	Angle of Cone						
	10°	20°	45°	60°	90°	120°	180°
1.5	0.17	0.40	1.06	1.21	1.14	1.07	1.00
3	0.17	0.40	0.86	1.02	1.06	1.04	1.00
D_2/D_1= Ratio of Diameter of larger pipe to smaller pipe (ASCE, 1992)							

Table 1.11b Typical Values of K_c for Sudden Pipe Contractions	
D_2/D_1	K_c
0.2	0.5
0.4	0.4
0.6	0.3
0.8	0.1
1	0
D_2/D_1= Ratio of Diameter of smaller pipe to larger pipe (ASCE, 1992)	

For storm drain pipes functioning under pressure flow, the loss coefficients listed in Tables 1.12 and 1.13 can be used with Equation 1.32 for sudden and gradual expansions respectively. For sudden contractions in pipes with pressure flow, the loss coefficients listed in Table 1.14 can be used in conjunction with Equation 1.33 (ASCE, 1992).

$$H_e = K_e (V_1^2/2g) \quad (1.32)$$

$$H_c = K_c (V_2^2/2g) \quad (1.33)$$

where:

- K_e = expansion coefficient (Tables 1.13 and 1.14)
- K_c = contraction coefficient (Table 1.15)
- V_1 = velocity upstream of transition
- V_2 = velocity downstream of transition
- g = acceleration due to gravity 32.2 ft/s^2

Table 1.12 Values of K_e for Determining Loss of Head due to Sudden Enlargement in Pipes

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

D_2/D_1 = ratio of diameter of larger pipe to smaller pipe
V_1 = velocity in smaller pipe (upstream of transition)
(ASCE, 1992)

Table 1.13 Values of K_e for Determining Loss of Head due to Gradual Enlargement in Pipes

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

D_2/D_1 = ratio of diameter of larger pipe to smaller pipe
Angle of cone is the angle in degrees between the sides of the tapering section
(ASCE, 1992)

Table 1.14 Values of K_e for Determining Loss of Head due to Sudden Contraction

D_2/D_1	Velocity, V_2, in feet Per Second												
	2.0	3.0	4.0	5.0	6.0	7.0	8.0	10.0	12.0	15.0	20.0	30.0	40.0
1.1	0.03	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.05	0.05	0.06
1.2	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.08	0.08	0.08	0.09	0.11	0.11
1.4	0.17	0.17	0.17	0.17	0.17	0.17	0.17	0.18	0.18	0.18	0.18	0.19	0.20
1.6	0.26	0.26	0.26	0.26	0.26	0.26	0.26	0.26	0.26	0.26	0.25	0.25	0.24
1.8	0.34	0.34	0.34	0.34	0.34	0.34	0.33	0.33	0.32	0.32	0.32	0.29	0.27
2.0	0.38	0.38	0.37	0.37	0.37	0.37	0.36	0.36	0.35	0.34	0.33	0.31	0.29
2.2	0.40	0.40	0.40	0.39	0.39	0.39	0.39	0.38	0.37	0.37	0.35	0.33	0.30
2.5	0.42	0.42	0.42	0.41	0.41	0.41	0.40	0.40	0.39	0.38	0.37	0.34	0.31

Table 1.14 Values of K_e for Determining Loss of Head due to Sudden Contraction

D_2/D_1	Velocity, V_2, in feet Per Second												
	2.0	3.0	4.0	5.0	6.0	7.0	8.0	10.0	12.0	15.0	20.0	30.0	40.0
3.0	0.44	0.44	0.44	0.43	0.43	0.43	0.42	0.42	0.41	0.40	0.39	0.36	0.33
4.0	0.47	0.46	0.46	0.46	0.45	0.45	0.45	0.44	0.43	0.42	0.41	0.37	0.34
5.0	0.48	0.48	0.47	0.47	0.47	0.46	0.46	0.45	0.45	0.44	0.42	0.38	0.35
10.0	0.49	0.48	0.48	0.48	0.48	0.47	0.47	0.46	0.46	0.45	0.43	0.40	0.36
∞	0.49	0.49	0.48	0.48	0.48	0.47	0.47	0.47	0.46	0.45	0.44	0.41	0.38

D_2/D_1 = ratio of diameter of larger pipe to smaller pipe
V_2 = velocity in smaller pipe (downstream of transition)
(ASCE, 1992)

Junction Losses

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

$$H_j=[((Q_o V_o) - (Q_i V_i) - (Q_l V_l \cos\theta)) / (0.5g(A_o+A_i))] + h_i - h_o \quad (1.34)$$

where:

H_j = junction loss (ft)

Q_o, Q_i, Q_l = outlet, inlet, and lateral flows, respectively (ft^3/s)

V_o, V_i, V_l = outlet, inlet, and lateral velocities, respectively (ft/s)

h_o, h_i = outlet and inlet velocity heads (ft)

A_o, A_i = outlet and inlet cross-sectional areas (ft^2)

θ = angle between the inflow and outflow pipes (Figure 1.22)

Inlet and Access Hole Losses - Preliminary Estimate

The initial layout of a storm drain system begins at the upstream end of the system. The designer must estimate sizes and establish preliminary elevations as the design progresses downstream. An approximate method for estimating losses across an access hole is provided in this section. This is a preliminary estimate only and will not be used when the energy grade line calculations are made. Methods defined in later in this section will be used to calculate the losses across an access hole when the energy grade line is being established.

The approximate method for computing losses at access holes or inlet structures involves multiplying the velocity head of the outflow pipe by a coefficient as represented in Equation 1.35. Applicable coefficients (K_{ah}) are tabulated in Table 1.15. This method can be used to estimate the initial pipe crown drop across an access hole or inlet structure to offset energy losses at the structure. The crown drop is then used to establish the appropriate pipe invert elevations. However, this method is used only in the preliminary design process and should not be used in the EGL calculations.

$$H_{ah}=K_{ah}(V_o^2/2g) \quad (1.35)$$

Table 1.15 Head Loss Coefficients (FHA, Revised 1993)	
Structure Configuration	**K_{ah}**
Inlet-straight run	0.5
Inlet-angled through	
90°	1.5
60°	1.25
45°	1.1
22.5°	0.7
Manhole-straight run	0.15
Manhole-angled through	
90°	1
60°	0.85
45°	0.75
22.5°	0.45

Inlet and Access Hole Losses for EGL Calculations - Energy-Loss Methodology

Various methodologies have been advanced for evaluating losses at access holes and other flow junctions. The energy loss method presented in this section is based on laboratory research and does not apply when the inflow pipe invert is above the water level in the access hole.

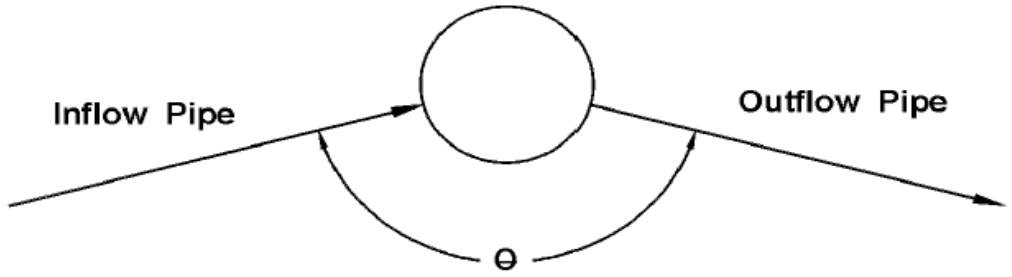


Figure 1.22 Head Loss Coefficients

The energy loss encountered going from one pipe to another through an access hole is commonly represented as being proportional to the velocity head of the outlet pipe. Using K to represent the constant of proportionality, the energy loss, H_{ah}, is approximated by Equation 1.36. Experimental studies have determined that the K value can be approximated by the relationship in Equation 1.37 when the inflow pipe invert is below the water level in the access hole.

$$\mathbf{H_{ah} = K\ (V_o^2/2g)} \qquad \mathbf{(1.36)}$$

$$\mathbf{K = K_O\ C_D\ C_d\ C_Q\ C_p\ C_B} \qquad \mathbf{(1.37)}$$

where:

- K = adjusted loss coefficient
- K_o = initial head loss coefficient based on relative access hole size
- C_D = correction factor for pipe diameter (pressure flow only)
- C_d = correction factor for flow depth
- C_Q = correction factor for relative flow
- C_p = correction factor for plunging flow
- C_B = correction factor for benching
- V_o = velocity of outlet pipe

For cases where the inflow pipe invert is above the access hole water level, the outflow pipe will function as a culvert, and the access hole loss and the access hole HGL can be computed using procedures found in *Hydraulic Design of Highway Culverts* (HDS-5, 1985). If the outflow pipe is flowing full or partially full under outlet control, the access hole loss (due to flow contraction into the outflow pipe) can be computed by setting K in Equation 1.36 to K_e as reported in Table 1.16. If the outflow pipe is flowing under inlet control, the water depth in the access hole should be computed using the inlet control nomographs in HDS- 5 (for example see Figure 3.31a).

The initial head loss coefficient, K_o in Equation 1.38, is estimated as a function of the **relative access hole** size and the angle of deflection between the inflow and outflow pipes as represented in Equation 1.6. This deflection angle is represented in Figure 1.22.

$$\mathbf{K_o = 0.1\ (b/D_o)(1\text{-}\sin\theta) + 1.4(b/D_o)^{0.15}\ \sin\theta} \qquad \mathbf{(1.38)}$$

where:

- θ = angle between the inflow and outflow pipes (Figure 1.22)
- b = access hole or junction diameter
- D_o = outlet pipe diameter

A change in head loss due to differences in **pipe diameter** is only significant in pressure flow situations when the depth in the access hole to outlet pipe diameter ratio, d_{aho}/D_o, is greater than 3.2. In these cases a correction factor for pipe diameter, C_D, is computed using Equation 1.39. Otherwise C_D is set equal to 1.

$$\mathbf{C_D=(D_o/D_i)^3} \qquad \mathbf{(1.39)}$$

where:

- D_o = outgoing pipe diameter
- D_i = inflowing pipe diameter

Table 1.16 Coefficients for Culverts; Outlet Control, Full, or Partly Full

Type of Structure and Design of Entrance	Coefficient K_e
Pipe, Concrete	
Projecting from fill, socket end (groove-end)	0.2
Projecting from fill, sq. cut end	0.5
Headwall or headwall and wingwalls	
Socket end of pipe (groove-end)	0.2
Square-edge	0.5
Rounded (radius = 1/12 D)	0.2
Mitered to conform to fill slope	0.7
*End-section conforming to fill slope	0.5
Beveled edges, 33.7° or 45° levels	0.2
Side-or slope-tapered inlet	0.2
Pipe, or Pipe-Arch, Corrugated Metal	
Project from fill (no headwall)	0.9
Headwall or headwall and wingwalls square-edge	0.5
Mitered to conform to fill slope, paved or unpaved slope	0.7
*End-section conforming to fill slope	0.5
Beveled edges, 33.7° or 45° bevels	0.2
Side-or slope-tapered inlet	0.2
Box, Reinforced Concrete	
Headwall parallel to embankment (no wingwalls)	
Square-edged on 3 edges	0.5
Rounded on 3 edges to radius of 1/12 barrel dimension, or beveled edges on 3 sides	0.2
Wingwalls at 30° to 75° to barrel	
Square-edged at crown	0.4
Crown edge rounded to radius of 1/2 barrel dimension, or beveled top edge	0.2
Wingwall at 10° to 25° to barrel	
Square-edged at crown	0.05
Wingwalls parallel (extension of sides)	
Square-edged at crown	0.7
Side-or slope-tapered inlet	0.2

*Note: "End-section conforming to fill slope," made of either metal or concrete, are the sections commonly available from manufacturers. From limited hydraulic tests they are equivalent in operation to a headwall in both inlet and outlet control. Some end sections, incorporating a closed taper in their design have a superior hydraulic performance.
(Source: Reference HDS No.5, 1985)

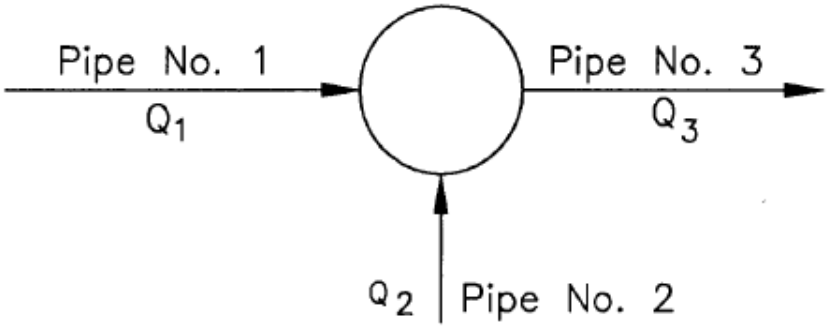


Figure 1.23 Relative flow effect

The correction factor for **flow depth**, C_d, is significant only in cases of free surface flow or low pressures, when the d_{aho}/D_o ratio is less than 3.2. In cases where this ratio is greater than 3.2, C_d is set equal to 1. To determine the applicability of this factor, the water depth in the access hole is approximated as the level of the hydraulic grade line at the upstream end of the outlet pipe. The correction factor is calculated using Equation 1.40.

$$C_D = 0.5(d_{aho}/D_o)^{0.6} \quad (1.40)$$

where:

d_{aho} = water depth in access hole above the outlet pipe invert

D_o = outlet pipe diameter

The correction factor for **relative flow**, C_Q, is a function of the angle of the incoming flow as well as the percentage of flow coming in through the pipe of interest versus other incoming pipes. It is computed using Equation 1.41. The correction factor is only applied to situations where there are 3 or more pipes entering the structure at approximately the same elevation. Otherwise, the value of C_Q is equal to 1.0.

$$C_Q = (1- 2\sin\theta)\,[1-(Q_i / Q_o)]^{0.75} + 1 \quad (1.41)$$

where:

C_Q = correction factor for relative flow

θ = the angle between the inflow and outflow pipes (Figure 1.22)

Q_i = flow in the inflow pipe

Q_o = flow in the outflow pipe

As can be seen from Equation 1.41, C_Q is a function of the angle of the incoming flow as well as the ratio of inflow coming through the pipe of interest and the total flow out of the structure. To illustrate this effect, consider the access hole shown in Figure 1.23 and assume the following two cases to determine the correction factor of pipe number 2 entering the access hole. For each of the two cases, the angle between the inflow pipe number 1 and the outflow pipe, θ, is 180°.

Case 1:
$Q_1 = 3$ ft³/s
$Q_2 = 1$ ft³/s
$Q_3 = 4$ ft³/s

Using Equation 1.41,
$C_Q = (1 - 2 \sin\theta)(1 - Q_i/Q_o)^{0.75} + 1$
$C_Q = (1 - 2 \sin 180^o)(1 - 3/4)^{0.75} + 1$
$C_Q = 1.35$

Case 2:
$Q_1 = 1.0$ ft³/s
$Q_2 = 3.0$ ft³/s
$Q_3 = 4.0$ ft³/s

Using Equation 1.41,
$C_Q = (1 - 2 \sin\theta)(1 - Q_i/Q_o)^{0.75} + 1$
$C_Q = (1 - 2 \sin 180^o)(1 - 1/4)^{0.75} + 1$
$C_Q = 1.81$

The correction factor for **plunging flow**, C_p, is calculated using Equation 1.42. This correction factor corresponds to the effect another inflow pipe, plunging into the access hole, has on the inflow pipe for which the head loss is being calculated. Using the notations in Figure 1.23, C_p is calculated for pipe #1 when pipe #2 discharges plunging flow. The correction factor is only applied when $h > d_{aho}$. Additionally, the correction factor is only applied when a higher elevation flow plunges into an access hole that has both an inflow line and an outflow in the bottom of the access hole. Otherwise, the value of C_p is equal to 1.0. Flows from a grate inlet or a curb opening inlet are considered to be plunging flow and the losses would be computed using Equation 1.42.

$$C_p = 1 + 0.2(h/D_o)\,[(h - d_{aho})/D_o] \qquad (1.42)$$

where:

C_p = correction for plunging flow

h = vertical distance of plunging flow from the flow line of the higher elevation inlet pipe to the center of the outflow pipe

D_o = outlet pipe diameter

d_{aho} = water depth in access hole relative to the outlet pipe invert

The correction for **benching** in the access hole, C_B, is obtained from Table 1.17. Figure 1.24 illustrates benching methods listed in Table 1.17. Benching tends to direct flow through the access hole, resulting in a reduction in head loss. For flow depths between the submerged and unsubmerged conditions, a linear interpolation is performed.

Table 1.17 Correction for Benching
(HEC 22, 2001)

Bench Type	Correction Factors, C_B	
	Submerged*	Unsubmerged**
Flat or Depressed Floor	1.00	1.00
Half Bench	0.95	0.15
Full Bench	0.75	0.07

*pressure flow, $d_{aho}/D_o \geq 3.2$

**free surface flow, $d_{aho}/D_o \leq 1.0$

In summary, to estimate the head loss through an access hole from the outflow pipe to a particular inflow pipe using the energy-loss method, multiply the above correction factors together to get the head loss coefficient, K. This coefficient is then multiplied by the velocity head in the outflow pipe to estimate the minor loss for the connection.

Composite Energy Loss Method

The Energy Loss Method described in earlier in the section resulted from preliminary experimental and analytical techniques that focused on relatively simple access hole layout and a small number of inflow pipes. A more suitable method is available to analyze complex access holes that have, for example, many inflow pipes. This complex method, referred to as the Composite Energy Loss Method, is implemented in the FHWA storm drain analysis and design package HYDRA (GKY, 1994). Details on the method are described in the HYDRA program technical documentation and the associated research report (Chang, et. al., 1994).

This complex minor loss computation approach focuses on the calculation of the energy loss from the inflow pipes to the outflow pipe (Chang, et. al., 1994). The methodology can be applied by determining the estimated energy loss through an access hole given a set of physical and hydraulic parameters. Computation of the energy loss allows determination and analysis of the energy gradeline and hydraulic gradeline in pipes upstream of the access hole. This methodology only applies to subcritical flow in pipes.

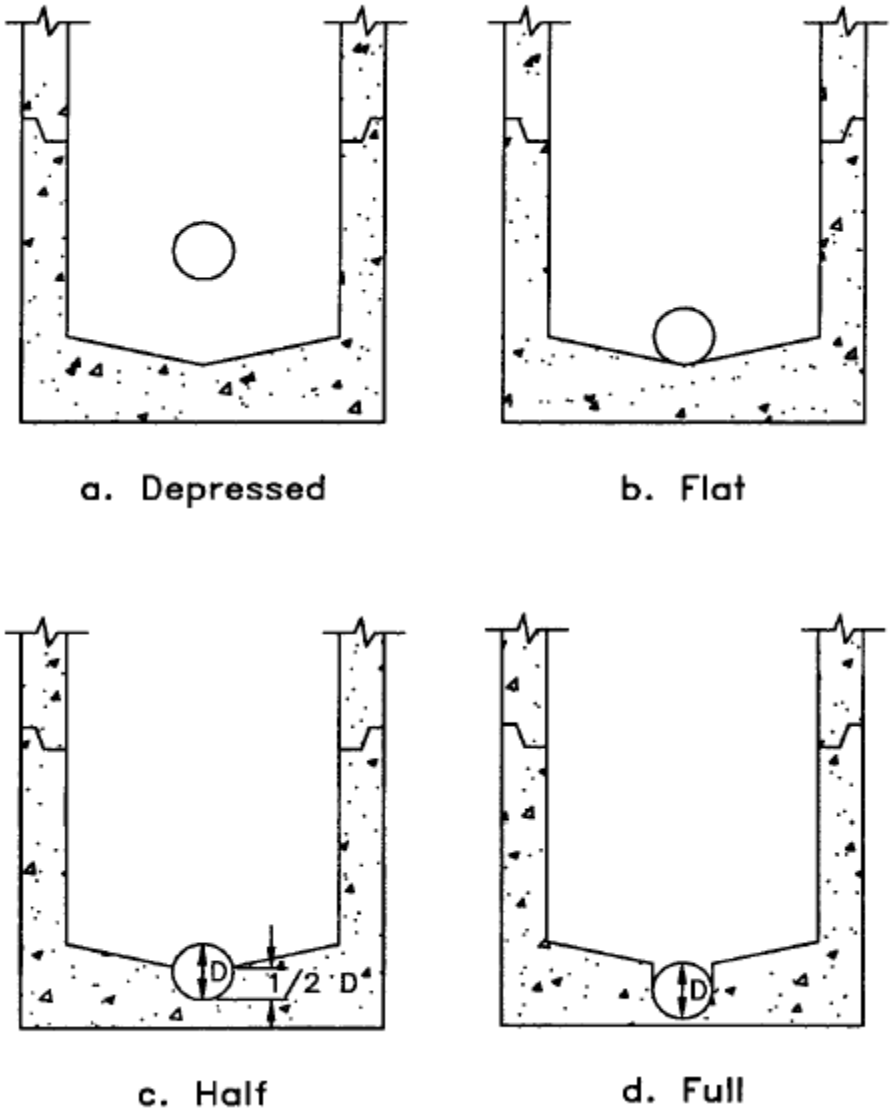


Figure 1.24 Access to Benching Methods

Preliminary Design Procedure

The preliminary design of storm drains can be accomplished by using the following steps and the storm drain computation sheet provided in Figure 1.25. This procedure assumes that each storm drain will be initially designed to flow full under gravity conditions. The designer must recognize that when the steps in this section are complete, the design is only preliminary. Final design is accomplished after the energy grade line and hydraulic grade line computations have been completed.

Step 1 Prepare a working plan layout and profile of the storm drainage system establishing the following design information:

a. Location of storm drains.
b. Direction of flow.
c. Location of access holes and other structures.
d. Number or label assigned to each structure.
e. Location of all existing utilities (water, sewer, gas, underground cables, etc.).

Step 2 Determine the following hydrologic parameters for the drainage areas tributary to each inlet to the storm drainage system:

a. Drainage areas.
b. Runoff coefficients.
c. Travel time

Step 3 Using the information generated in Steps 1 and 2, complete the following information on the design form for each run of pipe starting with the upstream most storm drain run:

a. "From" and "To" stations, Columns 1 and 2b, "Length" of run, Column 3
b. "Length" of run, Column 3
c. "Inc." drainage area, Column 4

   The incremental drainage area tributary to the inlet at the upstream end of the storm drain run under consideration.
d. "C," Column 6

   The runoff coefficient for the drainage area tributary to the inlet at the upstream end of the storm drain run under consideration. In some cases a composite runoff coefficient will need to be computed.
e. "Inlet" time of concentration, Column 9

   The time required for water to travel from the hydraulically most distant point of the drainage area to the inlet at the upstream end of the storm drain run under consideration.
f. "System" time of concentration, Column 10

   The time for water to travel from the most remote point in the storm drainage system to the upstream end of the storm drain run under consideration. For the upstream most storm drain run this value will be the same as the value in Column 9. For all other pipe runs this value is computed by adding the "System" time of concentration (Column 10) and the "Section" time of concentration (Column 17) from the previous run together to get the system time of concentration at the upstream end of the section under consideration (See *Section 1.2.4 of the Hydrology Technical Manual* for a general discussion of times of concentration).

Step 4 Using the information from Step 3, compute the following:

a. "TOTAL" area, Column 5

Add the incremental area in Column 4 to the previous sections total area and place this value in Column 5.

b. "INC." area x "C," Column 7

Multiply the drainage area in Column 4 by the runoff coefficient in Column 6. Put the product, CA, in Column 7.

c. "TOTAL" area x "C," Column 8

Add the value in Column 7 to the value in Column 8 for the previous storm drain run and put this value in Column 8.

d. "I," Column 11

Using the larger of the two times of concentration in Columns 9 and 10, and an Intensity-Duration-Frequency (IDF) curve, determine the rainfall intensity, I, and place this value in Column 11.

e. "TOTAL Q," Column 12

Calculate the discharge as the product of Columns 8 and 11. Place this value in Column 12.

f. "SLOPE," Column 21

Place the pipe slope value in Column 21. The pipe slope will be approximately the slope of the finished roadway. The slope can be modified as needed.

g. "PIPE DIA.," Column 13

Size the pipe using relationships and charts presented in this section to convey the discharge by varying the slope and pipe size as necessary. The storm drain should be sized as close as possible to a full gravity flow. Since most calculated sizes will not be available, a nominal size will be used. The designer will decide whether to go to the next larger size and have part full flow or whether to go to the next smaller size and have pressure flow.

h. "CAPACITY FULL," Column 14

Compute the full flow capacity of the selected pipe using Equation 1.18 and put this information in Column 14.

i. "VELOCITIES," Columns 15 and 16

Compute the full flow and design flow velocities (if different) in the conduit and place these values in Columns 15 and 16. If the pipe is flowing full, the velocities can be determined from V = Q/A, Equations 1.17 and 1.18. If the pipe is not flowing full, the velocity can be determined from Figure 1.19a.

j. "SECTION TIME," Column 17

Calculate the travel time in the pipe section by dividing the pipe length (Column 3) by the design flow velocity (Column 16). Place this value in Column 17.

k. "CROWN DROP," Column 20

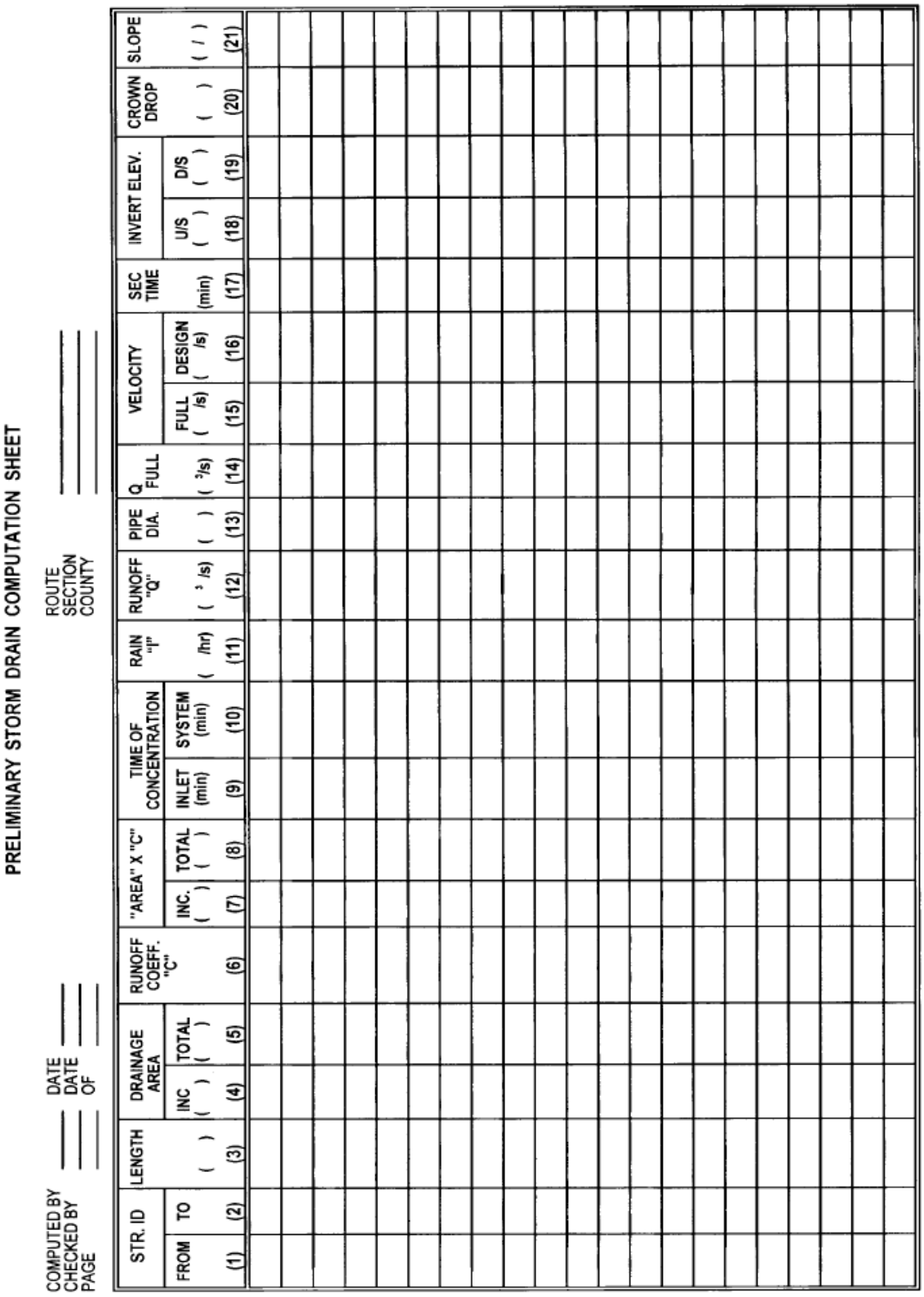
Calculate an approximate crown drop at the structure to off-set potential structure energy losses using Equation 1.33. Place this value in Column 20.

l. "INVERT ELEV.," Columns 18 and 19

Compute the pipe inverts at the upper (U/S) and lower (D/S) ends of this section of pipe, including any pipe size changes that occurred along the section.

Step 5 Repeat steps 3 and 4 for all pipe runs to the storm drain outlet. Use equations and nomographs to accomplish the design effort.

Step 6 Check the design by calculating the energy grade line and hydraulic grade line as described in this section.



PRELIMINARY STORM DRAIN COMPUTATION SHEET

COMPUTED BY _____ DATE _____
CHECKED BY _____ DATE _____
PAGE _____ OF _____

ROUTE _____
SECTION _____
COUNTY _____

STR. ID FROM (1)	STR. ID TO (2)	LENGTH ( ) (3)	DRAINAGE AREA INC ( ) (4)	DRAINAGE AREA TOTAL ( ) (5)	RUNOFF COEFF. "C" (6)	"AREA" X "C" INC. ( ) ( ) (7)	"AREA" X "C" TOTAL ( ) ( ) (8)	TIME OF CONCENTRATION INLET (min) (9)	TIME OF CONCENTRATION SYSTEM (min) (10)	RAIN "I" (/hr) (11)	RUNOFF "Q" (³ /s) (12)	PIPE DIA. ( ) (13)	Q FULL (³/s) (14)	VELOCITY FULL (/s) (15)	VELOCITY DESIGN (/s) (16)	SEC TIME (min) (17)	INVERT ELEV. U/S ( ) (18)	INVERT ELEV. D/S ( ) (19)	CROWN DROP ( ) (20)	SLOPE (/) (21)

1.2.11 Energy Grade Line Evaluation Procedure

This section presents a step-by-step procedure for manual calculation of the energy grade line (EGL) and the hydraulic grade line (HGL) using the energy loss method. For most storm drainage systems, computer methods such as HYDRA (GKY, 1994) are the most efficient means of evaluating the EGL and the HGL. However, it is important that the designer understand the analysis process so that he can better interpret the output from computer generated storm drain designs.

Figure 1.26 provides a sketch illustrating use of the two grade lines in developing a storm drainage system. The following step-by-step procedure can be used to manually compute the EGL and HGL. The computation tables in Figure 1.27 and Figure 1.28 can be used to document the procedure outlined below.

Before outlining the computational steps in the procedure, a comment relative to the organization of data on the form is appropriate. In general, a line will contain the information on a specific structure and the line downstream from the structure. As the table is started, the first two lines may be unique. The first line will contain information about the outlet conditions. This may be a pool elevation or information on a known downstream system. The second line will be used to define the conditions right at the end of the last conduit. Following these first two lines the procedure becomes more general. A single line on the computation sheet is used for each junction or structure and its associated outlet pipe. For example, data for the first structure immediately upstream of the outflow pipe and the outflow pipe would be tabulated in the third full line of the computation sheet (lines may be skipped on the form for clarity). Table A (Figure 1.27) is used to calculate the HGL and EGL elevations while Table B (Figure 1.28) is used to calculate the pipe losses and structure losses. Values obtained in table B are transferred to table A for use during the design procedure. In the description of the computation procedures, a column number will be followed by a letter A or B to indicate the appropriate table to be used.

EGL computations begin at the outfall and are worked upstream taking each junction into consideration. Many storm drain systems are designed to function in a subcritical flow regime. In subcritical flow, pipe and access hole losses are summed to determine the upstream EGL levels. If supercritical flow occurs, pipe and access losses are not carried upstream. When a storm drain section is identified as being supercritical, the designer should advance to the next upstream pipe section to determine its flow regime. This process continues until the storm drain system returns to a subcritical flow regime.

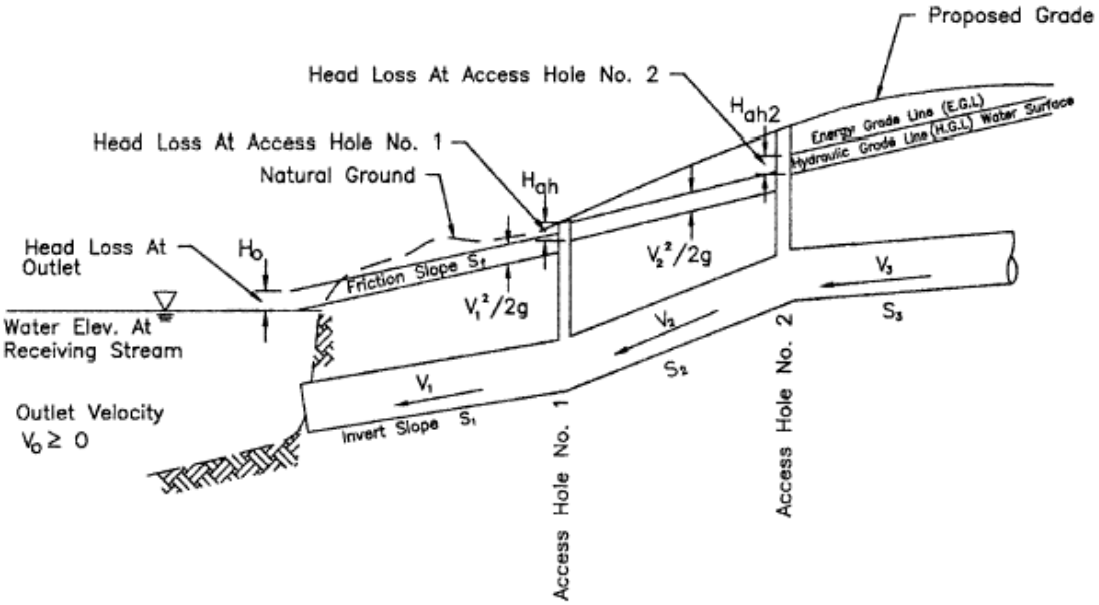


Figure 1.26 Energy and Hydraulic Grade Line Illustration

The EGL computational procedure follows:

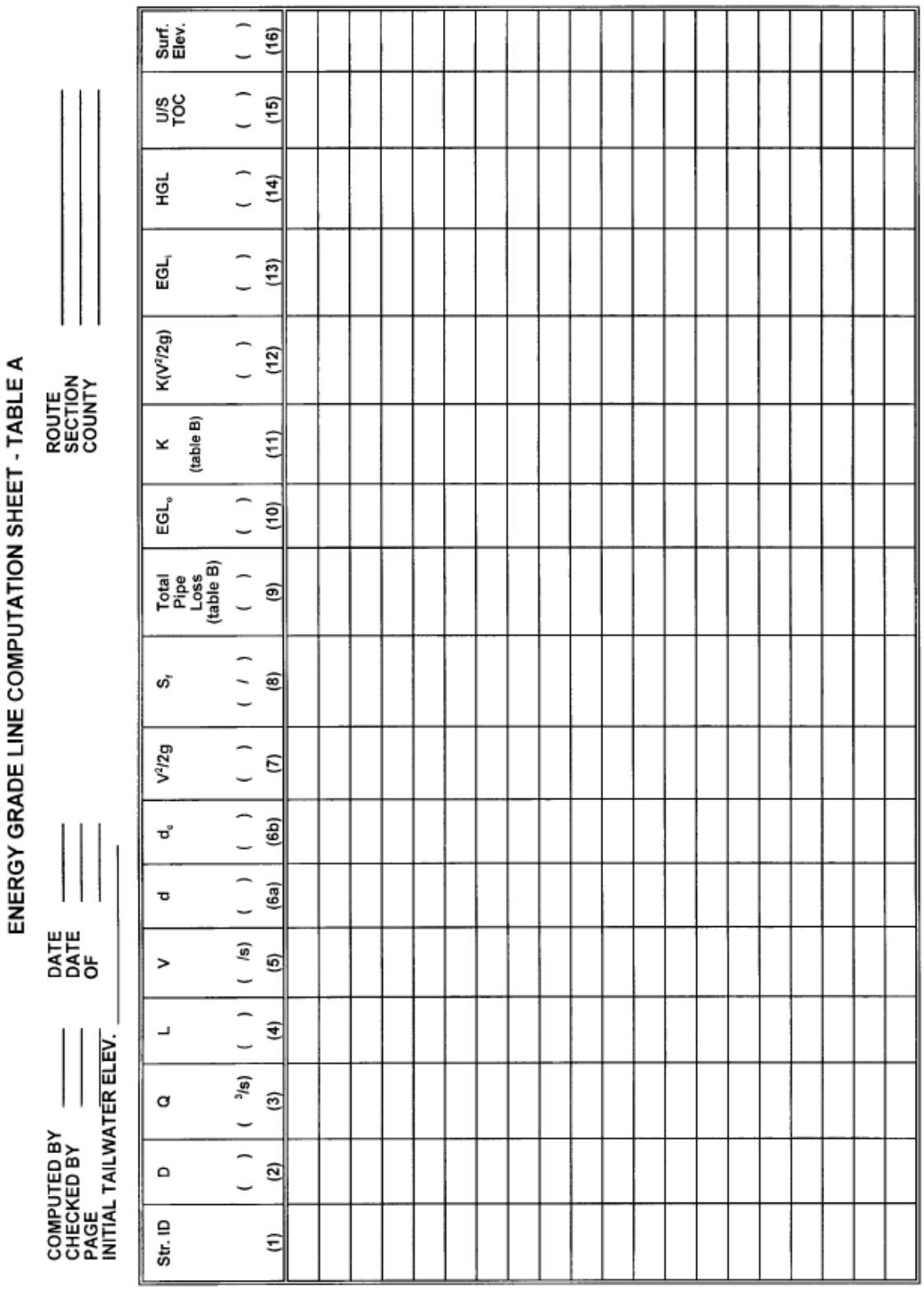
Step 1 The first line of Table A includes information on the system beyond the end of the conduit system. Define this as the stream, pool, existing system, etc. in column 1A. Determine the EGL and HGL for the downstream receiving system. If this is a natural body of water, the HGL will be at the water surface. The EGL will also be at the water surface if no velocity is assumed or will be a velocity head above the HGL if there is a velocity in the water body. If the new system is being connected to an existing storm drain system, the EGL and the HGL will be that of the receiving system. Enter the HGL in Column 14A and the EGL in Column 10A of the first line on the computation sheet.

Step 2 Identify the structure number at the outfall (this may be just the end of the conduit, but it needs a structure number), the top of conduit (TOC) elevation at the outfall end, and the surface elevation at the outfall end of the conduit. Place these values in Columns 1A, 15A, and 16A respectively. Also add the structure number in Col.1B.

Step 3 Determine the EGL just upstream of the structure identified in Step 2. Several different cases exist as defined below when the conduit is flowing full:

Case 1: If the TW at the conduit outlet is greater than $(d_c + D)/2$, the EGL will be the TW elevation plus the velocity head for the conduit flow conditions.

Case 2: If the TW at the conduit outlet is less than $(d_c + D)/2$, the EGL will be the HGL plus the velocity head for the conduit flow conditions. The equivalent hydraulic grade line, EHGL, will be the invert plus $(d_c + D)/2$.



ENERGY GRADE LINE COMPUTATION SHEET - TABLE A

COMPUTED BY _____ DATE _____
CHECKED BY _____ DATE _____
PAGE _____ OF _____
INITIAL TAILWATER ELEV. _____

ROUTE _____
SECTION _____
COUNTY _____

Str. ID	D	Q	L	V	d	d_c	$V^2/2g$	S_f	Total Pipe Loss (table B)	EGL_o	K (table B)	$K(V^2/2g)$	EGL_i	HGL	U/S TOC	Surf. Elev.
	( )	(^{3}/s)	( )	(/s)	( )	( )	( )	(/)	( )	( )		( )	( )	( )	( )	( )
(1)	(2)	(3)	(4)	(5)	(6a)	(6b)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)

Figure 1.27 Energy Grade Line Computation Sheet - Table A

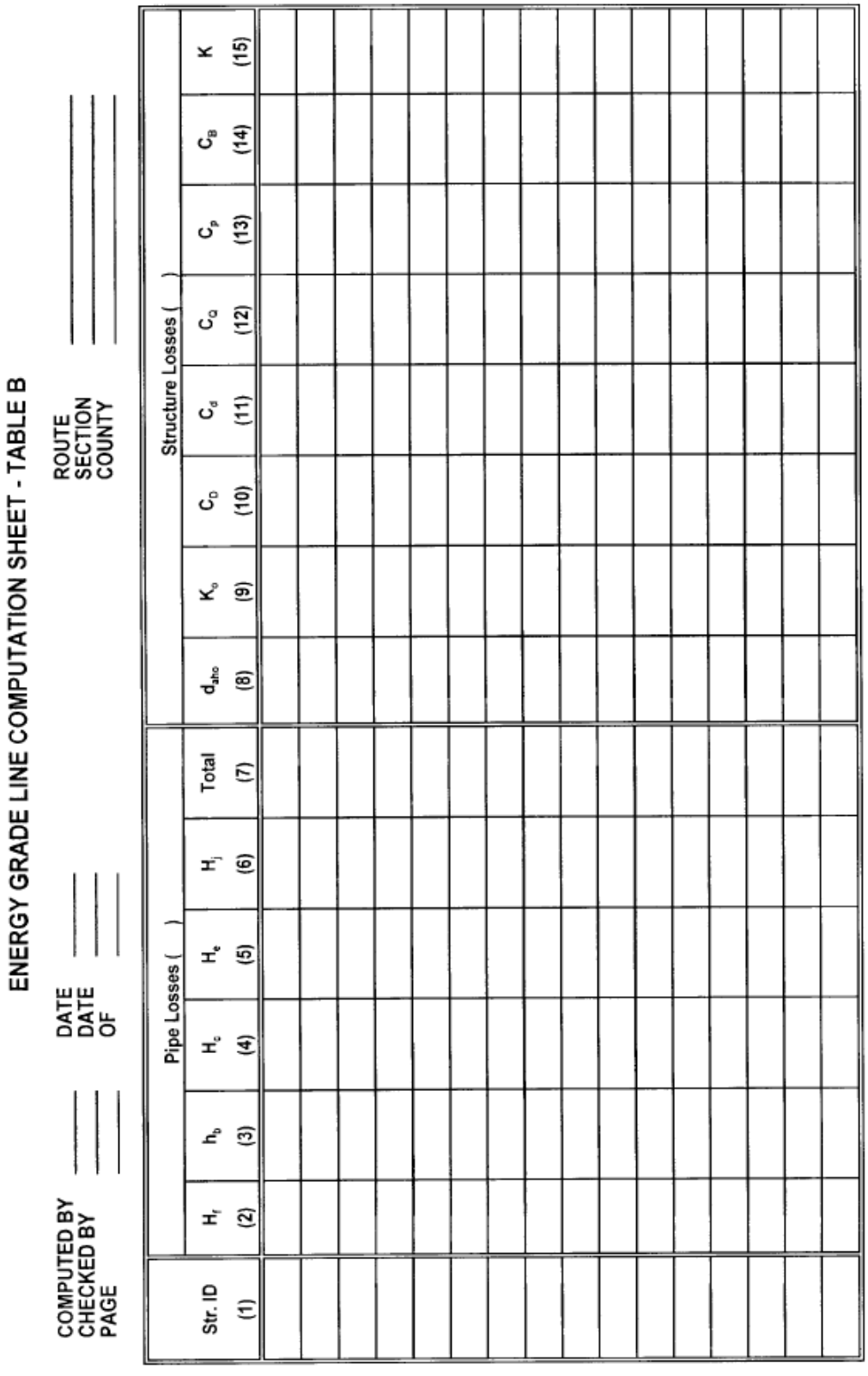


Figure 1.28 Energy Grade Line Computation Sheet - Table B

The velocity head needed in either Case 1 or 2 will be calculated in the next steps, so it may be helpful to complete Step 4 and work Step 5 to the point where velocity head (Col. 7A) is determined and then come back and finish this step. Put the EGL in Column 13A.

Note: The values for d_c for circular pipes can be determined from Figure 1.19b. Charts for other conduits or other geometric shapes can be found in *Hydraulic Design of Highway Culverts*, HDS-5, and cannot be greater than the height of the conduit.

Step 4 Identify the structure ID for the junction immediately upstream of the outflow conduit (for the first conduit) or immediately upstream of the last structure (if working with subsequent lines) and enter this value in Columns 1A and 1B of the next line on the computation sheets. Enter the conduit diameter (D) in column 2A, the design discharge (Q) in Column 3A, and the conduit length (L) in Column 4A.

Step 5 If the barrel flows full, enter the full flow velocity from continuity in Column 5A and the velocity head ($V^2/2g$) in column 7A. Put "full" in Column 6a and not applicable (n/a) in Column 6b of Table A. Continue with Step 6. If the barrel flows only partially full, continue with Step 5A.

Note: If the pipe is flowing full because of high tailwater or because the pipe has reached its capacity for the existing conditions, the velocity will be computed based on continuity using the design flow and the full cross sectional area. Do not use the full flow velocity determined in Column 15 of the Preliminary Storm Drain Computation Form for part-full flow conditions. For part-full conditions discussed in Step 5, the calculations in the preliminary form may be helpful. Actual flow velocities need to be used in the EGL/HGL calculations.

Step 5A Part full flow: Using the hydraulic elements graph in Figure 1.19a with the ratio of part full to full flow (values from the preliminary storm drain computation form), compute the depth and velocity of flow in the conduit. Enter these values in Column 6a and 5 respectively of Table A. Compute the velocity head ($V_2/2g$) and place in Column 7A.

Step 5B Compute critical depth for the conduit using Figure 1.19b. If the conduit is not circular, see HDS-5 for additional charts. Enter this value in Column 6b of Table A.

Step 5C Compare the flow depth in Column 6a (Table A) with the critical depth in Column 6b (Table A) to determine the flow state in the conduit. If the flow depth in Column 6a is greater than the critical depth in Column 6b, the flow is subcritical, continue with Step 6. If the flow depth in Column 6a is less than or equal to the critical depth in Column 6b, the flow is supercritical, continue with Step 5D. In either case, remember that the EGL must be higher upstream for flow to occur. If after checking for super critical flow in the upstream section of pipe, assure that the EGL is higher in the pipe than in the structure.

Step 5D Pipe losses in a supercritical pipe section are not carried upstream. Therefore, enter a zero (0) in Column 7B for this structure.

Step 5E Enter the structure ID for the next upstream structure on the next line in Columns 1A and 1B. Enter the pipe diameter (D), discharge (Q), and conduit length (L) in Columns 2A, 3A, and 4A respectively of the same line.

Note: After a downstream pipe has been determined to flow in supercritical flow, it is necessary to check each succeeding upstream pipe for the type of flow that exists. This is done by calculating normal depth and critical depth for each pipe. If normal depth is less than the diameter of the pipe, the flow will be open channel flow and the critical depth calculation can be used to determine whether the flow is sub or supercritical. If the flow line elevation through an access hole drops enough that the invert of the

upstream pipe is not inundated by the flow in the downstream pipe, the designer goes back to Step1A and begins a new design as if the downstream section did not exist.

Step 5F Compute normal depth for the conduit using Figure 1.19a and critical depth using Figure 1.19b. If the conduit is not circular see HDS-5 for additional charts. Enter these values in Columns 6A and 6b of Table A.

Step 5G If the pipe barrel flows full, enter the full flow velocity from continuity in Column 5A and the velocity head ($V^2/2g$) in Column 7A. Go to Step 3, Case 2 to determine the EGL at the outlet end of the pipe. Put this value in Column 10A and go to Step 6. For part full flow, continue with Step 5H.

Step 5H Part full flow: Compute the velocity of flow in the conduit and enter this value in Column 5A. Compute the velocity head ($V^2/2g$) and place in Column 7A.

Step 5I Compare the flow depth in Column 6a with the critical depth in Column 6b to determine the flow state in the conduit. If the flow depth in Column 6a is greater than the critical depth in Column 6b, the flow is subcritical, continue with Step 5J. If the flow depth in Column 6a is less than or equal to the critical depth in Column 6b, the flow is supercritical, continue with Step 5K.

Step 5J Subcritical flow upstream: Compute EGL_o at the outlet of the previous structure as the outlet invert plus the sum of the outlet pipe flow depth and the velocity head. Place this value in Column 10A of the appropriate structure and go to Step 9.

Step 5K Supercritical flow upstream: Access hole losses do not apply when the flow in two (2) successive pipes is supercritical. Place zeros (0) in Columns 11A, 12A, and 15B of the intermediate structure (previous line). The HGL at the structure is equal to the pipe invert elevation plus the flow depth. Check the invert elevations and the flow depths both upstream and downstream of the structure to determine where the highest HGL exists. The highest value should be placed in Column 14A of the previous structure line. Perform Steps 20 and 21 and then repeat Steps 5E through 5K until the flow regime returns to subcritical. If the next upstream structure is end-of-line, skip to step 10b then perform Steps 20, 21, and 24.

Step 6 Compute the friction slope (S_f) for the pipe using Equation 1.19 divided by $L[S_f = H_f/L = [185\ n^2\ (V^2/2g)]/D^{4/3}]$ for a pipe flowing full. Enter this value in Column 8A of the current line. If full flow does not exist, set the friction slope equal to the pipe slope.

Step 7 Compute the friction loss (H_f) by multiplying the length (L) in Column 4A by the friction slope (S_f) in Column 8A and enter this value in Column 2B. Compute other losses along the pipe run such as bend losses (h_b), transition contraction (H_c) and expansion (H_e) losses, and junction losses (H_j) using Equations 1.29 through 1.34 and place the values in Columns 3B, 4B, 5B, and 6B, respectively. Add the values in 2B, 3B, 4B, 5B, and 6B and place the total in Column 7B and 9A.

Step 8 Compute the energy grade line value at the outlet of the structure (EGL_o) as the EGL_i elevation from the previous structure (Column 13A) plus the total pipe losses (Column 9A). Enter the EGL_o in Column 10A.

Step 9 Estimate the depth of water in the access hole (estimated as the depth from the outlet pipe invert to the hydraulic grade line in the pipe at the outlet). Computed as EGL_o (Column 10A) minus the pipe velocity head in Column 7A minus the pipe invert elevation (from the preliminary storm drain computation form). Enter this value in Column 8B. If supercritical flow exists in this structure, leave this value blank and skip to Step 5E.

Step 10 If the inflow storm drain invert is submerged by the water level in the access hole, compute access hole losses using Equations 1.36 and 1.37. Start by computing the initial structure head loss coefficient, K_o, based on relative access hole size. Enter this value in Column 9B. Continue with Step 11. If the inflow storm drain invert is not submerged by the water level in the access hole, compute the head in the access hole using culvert techniques from HDS-5 as follows:

a. If the structure outflow pipe is flowing full or partially full under outlet control, compute the access hole loss by setting K in 1.35 to K_e as reported in Table 1.15. Enter this value in Column 15B and 11A, continue with Step 17. Add a note on Table A indicating that this is a drop structure.

b. If the outflow pipe functions under inlet control, compute the depth in the access hole (HGL) using Chart 28 or 29. If the storm conduit shape is other than circular, select the appropriate inlet control nomograph from HDS-5. Add these values to the access hole invert to determine the HGL. Since the velocity in the access hole is negligible, the EGL and HGL are the same. Enter HGL in Col.14A and EGL in Col.13A. Add a note on Table A indicating that this is a drop structure. Go to Step 20.

Step 11 Using Equation 1.39 compute the correction factor for pipe diameter, C_D, and enter this value in Column 10B. Note, this factor is only significant in cases where the d_{aho}/D_o ratio is greater than 3.2.

Step 12 Using Equation 1.40 compute the correction factor for flow depth, C_D, and enter this value in Column 11B. Note, this factor is only significant in cases where the d_{aho}/D_o ratio is less than 3.2.

Step 13 Using Equation 1.41, compute the correction factor for relative flow, C_Q, and enter this value in Column 12B. This factor = 1.0 if there are less than 3 pipes at the structure.

Step 14 Using Equation 1.42, compute the correction factor for plunging flow, C_p, and enter this value in Column 13B. This factor = 1.0 if there is no plunging flow. This correction factor is only applied when $h>d_{aho}$.

Step 15 Enter in Column 14B the correction factor for benching, C_B, as determined from Table 1.17. Linear interpolation between the two columns of values will most likely be necessary.

Step 16 Using Equation 1.37, compute the value of K and enter this value in Column 15B and 11A.

Step 17 Compute the total access hole loss, H_{ah}, by multiplying the K value in Column 11A by the velocity head in Column 7A. Enter this value in Column 12A.

Step 18 Compute EGL_i at the structure by adding the structure losses in Column 12A to the EGL_o value in Column 10A. Enter this value in Column 13A.

Step 19 Compute the hydraulic grade line (HGL) at the structure by subtracting the velocity head in Column 7A from the EGL_i value in Column 13A. Enter this value in Column 14A.

Step 20 Determine the top of conduit (TOC) value for the inflow pipe (using information from the storm drain computation sheet) and enter this value in Column 15A.

Step 21 Enter the ground surface, top of grate elevation or other high water limits at the structure in Column 16A. If the HGL value in Column 14A exceeds the limiting elevation, design modifications will be required.

Step 22 Enter the structure ID for the next upstream structure in Column 1A and 1B of the next line. When starting a new branch line, skip to Step 24.

Step 23 Continue to determine the EGL through the system by repeating Steps 4 through 23. (Begin with Step 2 if working with a drop structure. This begins the design process again as if there were no system down stream from the drop structure).

Step 24 When starting a new branch line, enter the structure ID for the branch structure in Column 1A and 1B of a new line. Transfer the values from Columns 2A through 10A and 2B to 7B associated with this structure on the main branch run to the corresponding columns for the branch line. If flow in the main storm drain at the branch point is subcritical, continue with Step 9; if supercritical, continue with Step 5E.

1.2.12 Storm Drain Design Example

The following storm drain design example illustrates the application of the design procedures outlined in *Section 1.2.10*.

Example of Preliminary Storm Drain Design

Given: The roadway plan and section illustrated in Figure 1.29, duration intensity information in Table 1.19 and inlet drainage area information in Table 1.18. All grates are type P 50 x 100, all piping is reinforced concrete pipe (RCP) with a Manning's n value of 0.013, and the minimum design pipe diameter = 18 in for maintenance purposes.

Find:

(1) Using the procedures outlined in *Section 1.2.10* determine appropriate pipe sizes and inverts for the system illustrated in Figure 1.29.

(2) Evaluate the HGL for the system configuration determined in part (1) using the procedure outlined in *Section 1.2.10*.

Solution:

(1) Preliminary Storm Drain Design

Step 1. Figure 1.29 illustrates the proposed system layout including location of storm drains, access holes, and other structures. All structures have been numbered for reference. Figure 1.30 (a) and (b) illustrate the corresponding storm drain profiles.

Step 2. Drainage areas, runoff coefficients, and times of concentration are tabulated in Figure 1.31. Example problems documenting the computation of these values are included in this section

Starting at the upstream end of a conduit run, Steps 3 and 4 from *Section 1.2.10* are completed for each storm drain pipe. A summary tabulation of the computational process is provided in Figure 1.31. The column by column computations for each section of conduit follow:

Table 1.18 Drainage Area Information for Design Example

Inlet No.	Drainage Area (ac)	"C"	Time of Concentration (min)
40	0.64	0.73	3
41	0.35	0.73	2
42	0.32	0.73	2
43	--	--	--
44			

Table 1.19 Intensity/Duration Data Design Example									
Time (min)	5	10	15	20	30	40	50	60	120
Intensity (in/hr)	7.1	5.9	5.1	4.5	3.5	3	2.6	2.4	1.4

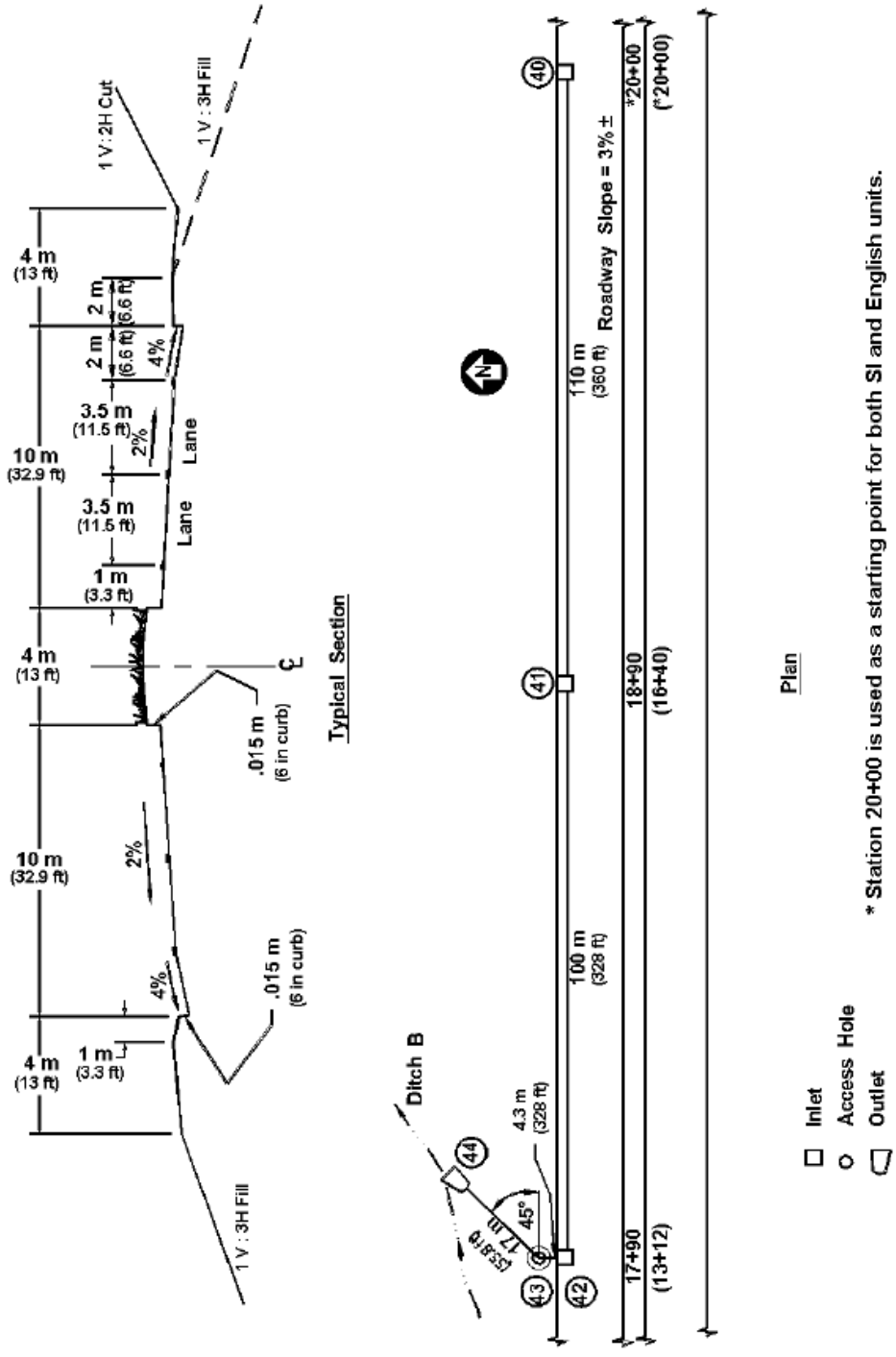


Figure 1.29 Roadway Plan and Sections for Example

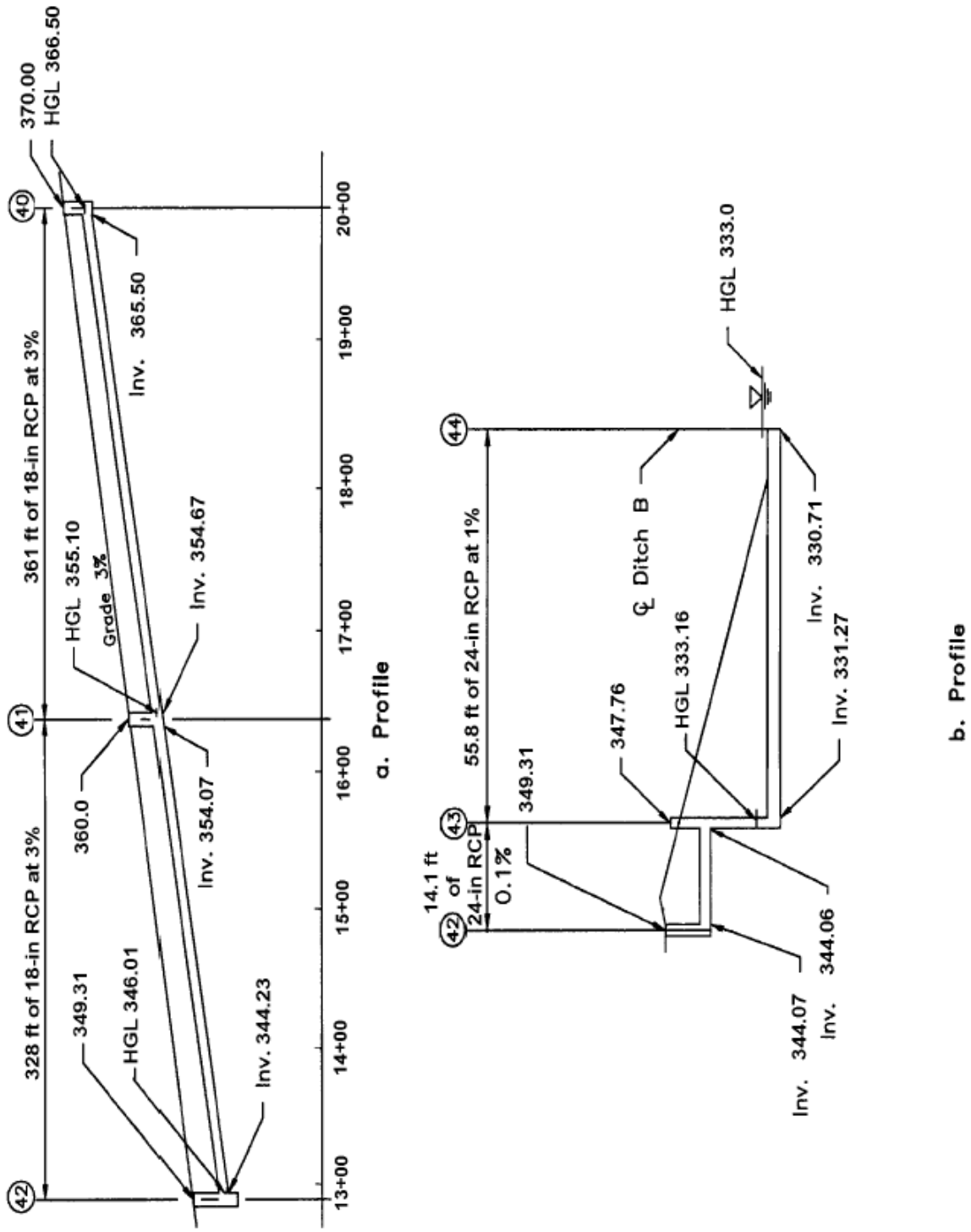


Figure 1.30 Storm Drain Profiles for Example

PRELIMINARY STORM DRAIN COMPUTATION SHEET
(English Solution)

COMPUTED BY _____ DATE _____
CHECKED BY _____ DATE _____
PAGE _____ OF _____

ROUTE _____
SECTION _____
COUNTY _____

STR. ID		LENGTH	DRAINAGE AREA		RUNOFF COEFF. "C"	"AREA" X "C"		TIME OF CONCENTRATION		RAIN "I"	RUNOFF "Q"	PIPE DIA.	Q FULL	VELOCITY		SEC TIME	INVERT ELEV.		CROWN DROP	SLOPE
FROM	TO	(Ft)	INC (ac)	TOTAL (ac)		INC. (ac)	TOTAL (ac)	INLET (min)	SYSTEM (min)	(in/hr)	(ft³ /s)	(ft)	(ft³/s)	FULL (ft /s)	DESIGN (ft /s)	(min)	U/S (ft)	D/S (ft)	(ft)	(ft/ft)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)
40	41	361.0	0.64	0.64	0.73	0.47	0.47	3	5	7.1	3.3	1.50	18.1	10.3	7.52	1	365.50	354.67	----	0.03
41	42	328.0	0.35	0.99	0.73	0.25	0.72	2	5	7.1	5.1	1.5	18.1	10.3	8.7	1	354.07	344.23	0.6	0.03
42	43	14.1	0.32	1.31	0.73	0.23	0.95	2	5	7.1	6.75	2.0	7.12	2.28	2.6	0	344.07	344.06	0.16	0.001
43	44	55.8	0.00	1.31	-	-	0.95	-	5	7.1	6.75	2.0	22.52	7.22	6.10	0	331.27	330.71	12.79	0.01

Figure 1.31 Storm Drain Computation Sheet for Example

Structure 40 to 41

Col. 1 From structure 40

Col. 2 To structure 41

Col. 3 Run Length	L = 2000 ft - 1639 ft L = 361 ft	Figure 1.30
Col. 4 Inlet Area	A_i = 0.64 ac	Table 1.18
Col. 5 Total Area	A_t = 0.64 ac	Total area up to inlet 40
Col. 6 "C"	C = 0.73	Table 1.18
Col. 7 Inlet CA	CA = (0.64)(0.73) CA = 0.47 ac	Col. 4 times Col. 6
Col. 8 Sum CA	ΣCA = 0.47 + 0 ΣCA= 0.47 ac	Col. 7 plus previous Col. 8
Col. 9 Inlet Time	t_i = 3 min	Table 1.18
Col. 10 Sys. Time	t_c = 3 min (use 5 min)	same as Col. 9 for upstream most section
Col. 11 Intensity	I = 7.1 in/hr	Table 1.19; System time less than 5 minutes therefore, use 5 minutes
Col. 12 Runoff	$Q = C_f (CA) (I)$ Q = 1.0(0.47)(7.1) $Q = 3.3 ft^3/sec$	*Equation 1.3 of the Hydrology Technical Manual*; C_f = 1.0 (10yr) Col. 8 times Col. 11 multiplied by 1.0
Col. 13 Pipe Dia.	$D = [(Qn)/(K_Q S_o^{0.5})]^{0.375}$ $D = [(3.3)(0.013)/(0.46)(0.03)^{0.5})]^{0.375}$ D = 0.8 ft D_{min} = 1.5 ft	Equation 1.18 or Figure 1.17 use D_{min}
Col. 14 Full Cap	$Q_f = (K_Q/n) D^{2.67} S_o^{0.5}$ $Q_f = (0.46/0.013) (1.5)^{2.67} (0.03)^{0.5}$ $Q_f = 18.1 ft^3/s$	Equation 1.18 or Figure 1.17
Col. 15 Vel. Full	$V_f = (K_V/n) D^{0.67} S_o^{0.5}$ $V_f = (0.59/0.013) (1.5)^{0.67} (0.03)^{0.5}$ V_f = 10.3 ft/s	Equation 1.17 or Figure 1.17
Col. 16 Vel. Design	$Q/Q_f = 3.3/18.1 = 0.18$ $V/V_f = 0.73$ V = (0.73) (10.3) V = 7.52 ft/s	Figure 1.19a
Col. 17 Sect. Time	$t_s = L/V = 361 / 7.52 / 60$ t_s = 0.8 min; use 1 min	Col. 3 divided by Col. 16

Col. 18 U/S Invert	= Grnd - 3.0 ft - dia = 370.0 - 3.0 - 1.5 = 365.5 ft	3 ft = min cover Ground elevation from Figure 1.30
Col. 19 D/S Invert	= (365.5) - (361.0)(0.03) = 354.67 ft	Col. 18 - (Col. 3)(Col. 21)
Col. 20 Crown Drop	= 0	Upstream most invert
Col. 21 Slope	S = 0.03	select desired pipe slope

At this point, the pipe should be checked to determine if it still has adequate cover.

354.67 + 1.5 + 3.0 = 359.17 Invert elev. + Diam + min cover
Ground elevation of 360.0 ft is greater than 359.17 ft so OK

Structure 41 to 42

Col. 1 From	= 41	
Col. 2 To	= 42	
Col. 3 Run Length	L = 1639 - 1311 \ L = 328 ft	Figure 1.30
Col. 4 Inlet Area	A_i = 0.35 ac	Table 1.18
Col. 5 Total Area	A_t = 0.35 + 0.64 A_t = 0.99 ac	Col. 4 plus structure 42 total area, Table 1.18
Col. 6 "C"	C = 0.73	Table 1.18
Col. 7 Inlet CA	CA = (0.73)(0.35) CA = 0.25 ac	Col. 4 times Col. 6
Col. 8 Sum CA	ΣCA = 0.25 + 0.47 ΣCA = 0.72 ac	Col. 7 plus previous Col. 8
Col. 9 Inlet Time	t_i = 2 min	Table 1.18
Col. 10 Sys. Time	t_c = 4 min (use 5 min)	Col. 9 + Col. 17 for line 40-41
Col. 11 Intensity	I = 7.1 in/hr	Table 1.19; system time equals 5 min
Col. 12 Runoff	$Q = (C_f)(CA)(I)$ Q = 1.0(0.72) (7.1) Q = 5.1 ft^3/sec	*Equation 1.3 of the Hydrology Technical Manual*; C_f =1.0(10-yr) Col. 8 times Col. 11 times C_f
Col. 13 Pipe Dia.	$D = [(Qn)/(K_Q S_o^{0.5})]^{0.375}$ $D = [(5.1) (0.013)/(0.46)(0.03)^{0.5}]^{0.375}$ D = 0.93 ft D_{min} = 1.5 ft use D_{min}	Equation 1.18 or Figure 1.17 use D_{min}

Col. 14 Full Cap.	$Q_f = (K_Q/n)\ D^{2.67}\ S_o^{0.5}$ $Q_f = (0.46/0.013)(1.5)^{2.67}(0.03)^{0.5}$ $Q_f = 18.1$ ft3/s	Equation 1.18 or Figure 1.17
Col. 15 Vel. Full	$V_f = (K_V/n)\ D^{0.67}\ S_o^{0.5}$ $V_f = (0.59/0.013)(1.5)^{0.67}\ (0.03)^{0.5}$ $V_f = 10.3$ ft/s	Equation 1.18 or Figure 1.17
Col. 16 Vel. Design	$Q/Q_f = 5.1/18.1 = 0.28$ $V/V_f = 0.84$ $V = (0.84)\ (10.3)$ $V = 8.7$ ft/s	Figure 1.19a
Col. 17 Sect. Time	$T_s = L/V = 328 / 8.75 / 60$ $T_s = 0.6$ min; use 1 min	Col. 3 divided by Col. 16
Col. 18 U/S Invert	= 354.67 - 0.6 = 354.07 ft	Downstream invert of upstream conduit minus estimated structure loss (drop)
Col. 19 D/S Invert	= (354.07) - (328)(0.03) = 344.23 ft	Col. 18 - (Col. 3)(Col. 21)
Col. 20 Crown Drop	$= H_{ah} = K_{ah}\ (V^2 / 2g)$ $= (0.5)(8.7)^2 / [(2)(32.2)]$ = 0.6 ft	Equation 1.36 with Table 1.15 $K_{ah} = 0.5$ for inlet - straight run
Col. 21 Slope	S = 0.03	select desired pipe slope
Structure 42 to 43		
Col. 1 From structure	= 42	
Col. 2 To structure	= 43	
Col. 3 Run Length	L = 14.1 ft	Figure 1.30
Col. 4 Inlet Area	$A_i = 0.32$ ac	Table 1.18
Col. 5 Total Area	$A_t = 0.32 + 0.99$ $A_t = 1.31$ ac	Col. 4 plus previous Col. 5 total area, Table 1.18
Col. 6 "C"	C = 0.73	Table 1.18
Col. 7 Inlet CA	CA = (0.73)(0.32) CA = 0.23 ac	Col. 4 times Col. 6
Col. 8 Sum CA	ΣCA = 0.23 + 0.72 ΣCA = 0.95 ac	Col. 7 plus structure 43 total CA values
Col. 9 Inlet Time	$t_i = 2$ min	Table 1.18

Col. 10 Sys. Time	t_c = 5 min	Col. 9 + Col. 17 for line 40-41 plus Col.17 for line 41-42
Col. 11 Intensity	I = 7.1 in/hr	Table 1.19
Col. 12 Runoff	$Q = (C_f)(CA)(I)$ Q = 1.0 (0.95) (7.1) Q = 6.75 ft^3/sec	Col. 8 times Col. 11
Col. 13 Pipe Dia.	$D = [(Qn)/(K_Q S_o^{0.5})]^{0.375}$ $D =[(6.75)(0.013)/(0.46)(0.001)^{0.5}]^{0.375}$ D = 1.96 ft D = 2.0 ft	Equation 1.18 or Figure 1.17 Use nominal size
Col. 14 Full Cap .	$Q_f = (K_Q/n)(D^{2.67})(S_o^{0.5})$ $Q_f = (0.46/(0.013)(2.0)^{2.67} (0.001)^{0.5}$ $Q_f = 7.12$ ft^3/s	Equation 1.18 or Figure 1.17
Col. 15 Vel. Full	$V_f = (K_v/n) D^{0.67} S_o^{0.5}$ $V_f = (0.59)/(0.013)(2.0)^{0.67} (0.001)^{0.5}$ V_f = 2.28 ft/s	Equation 1.18 or Figure 1.17
Col. 16 Vel. Design	Q/Q_f = 6.75/7.12 = 0.95 V/V_f = 1.15 V = (1.15) (2.28) V = 2.6 ft/s	Figure 1.19a
Col. 17 Sect. Time	t_s = L/V = 14.1 / 2.6 / 60 t_s = 0.09 min, use 0.0 min	Col. 3 divided by Col. 16
Col. 18 U/S Invert	= 344.23 - 0.16 = 344.07 ft	Downstream invert of upstream conduit minus estimated structure loss (drop)
Col. 19 D/S Invert	= 344.07 - (14.1)(0.001) = 344.06 ft	Col. 18 - (Col. 3)(Col. 21)
Col. 20 Crown Drop	$= H_{ah} = K_{ah} (V^2 / 2g)$ $= (1.5)(2.6)^2 / [(2)(32.2)]$ = 0.16 ft	Equation 1.36 and Table 1.15; K_{ah} =1.5 for inlet - angled through 90 degrees
Col. 21 Slope	S = 0.001	Select desired pipe slope
Structure 43 to 44		
Col. 1 From	= 43	
Col. 2 To	= 44	
Col. 3 Run Length	L = 55.8 ft	Figure 1.30
Col. 4 Inlet Area	A_i = 0.0 ac	Table 1.18

Col. 5 Total Area	A_t = 1.31 ac	Col. 4 plus previous Col. 5
Col. 6 "C"	C = n/a	Table 1.18
Col. 7 Inlet CA	CA = 0.0	Col. 4 times Col. 6
Col. 8 Sum CA	ΣCA = 0.00 + 0.95 ΣCA = 0.95 ac	Col. 7 plus previous Col. 8
Col. 9 Inlet Time	n/a	No inlet
Col. 10 Sys. Time	t_c = 5 min	Col. 10 + Col. 17 for line 42-43
Col. 11 Intensity	I = 7.1 in/hr	Table 1.18
Col. 12 Runoff	$Q = C_f (CA) I$ Q =1.0 (0.95) (7.1) $Q = 6.75 ft^3/sec$	Col. 8 times Col. 11 times C_f
Col. 13 Pipe Dia.	$D = [(Qn)/(K_Q S_o^{0.5})]^{0.375}$ $D = [(6.75)(0.013)/(0.46)(0.01)^{0.5}]^{0.375}$ D = 1.27 ft D = 2.0 ft	Equation 1.18 or Figure 1.17 U/S conduit was 2.0 ft. - Do not reduce size inside the system
Col. 14 Full Cap.	$Q_f = (K_Q/n)(D^{2.67})(S_o^{0.5})$ $Q_f = (0.46)/(0.013)(2.0)^{2.67} (0.01)^{0.5}$ $Q_f = 22.52 ft^3/s$	Equation 1.18 or Figure 1.17
Col. 15 Vel. Full	$V_f = (K_v/n) D^{0.67} S_o^{0.5}$ $V_f = (0.59)/(0.013)(2.0)^{0.67} (0.01)^{0.5}$ $V_f = 7.22$ ft/s	Equation 1.17 or Figure 1.17
Col. 16 Vel. Design	$Q/Q_f = 6.75/22.52 = 0.30$ $V/V_f = 0.84$ V = (0.84) (7.22) V = 6.1 ft/s	Figure 1.19a
Col. 17 Sect. Time	t_s = 55.8 /6.1 / 60 t_s = 0.15 min, use 0.0 min	Col. 3 divided by Col. 16
Col. 19 D/S Invert	= 330.71 ft	Invert at discharge point in ditch
Col. 18 U/S Invert	= 330.71 + (55.8)(0.01) = 331.27 ft	Col. 19 + (Col. 3)(Col. 21)
Col. 20 Crown Drop	= 344.06 - 331.27 = 12.79 ft straight run	Col. 19 previous run - Col. 18
Col. 21 Slope	S = 0.01	Select desired pipe slope

(2) Energy Grade Line Evaluation Computations - English Units

The following computational procedure follows the steps outlined in *Section 1.2.11* above. Starting at structure 44, computations proceed in the upstream direction. A summary tabulation of

the computational process is provided in Figure 1.32 English and Figure 1.33 English. The column by column computations for each section of storm drain follow:

RUN FROM STRUCTURE 44 TO 43

Outlet

Step 1	Col. 1A	Outlet	
	Col. 14A	HGL = 333.0	Downstream pool elevation
	Col. 10A	EGL = 333.0	Assume no velocity in pool

Structure 44

Step 2	Col. 1A, 1B	Str. ID = 44	Outlet
	Col. 15A	Invert = 330.71 ft	Outfall invert
		TOC = 330.71 + 2.0	Top of storm drain at outfall
		TOC = 332.71	
		Surface Elev = 332.71	Match TOC
Step 3		HGL = TW = 333.0	From Step 1
		$EGL_i = HGL + V^2/2g$	Use Case 1 since TW is above the top of conduit
		$EGL_i = 333.0 + 0.07$	
	Col. 13A	$EGL_i = 333.07$	EGL_i for str. 44

Structure 43

Step 4	Col. 1A, 1B	Str. ID = 43	Next Structure
	Col. 2A	D = 2.0 ft	Pipe Diameter
	Col. 3A	Q = 6.75 cfs	Conduit discharge (design value)
	Col. 4A	L = 55.8 ft	Conduit length
Step 5	Col. 5A	$V = Q/A$	Velocity; use full barrel velocity since outlet is submerged.
		$V = 6.75/[(\pi/4)\ (2.0)^2]$	
		$V = 2.15$ ft/s	
	Col. 7A	$V^2/2g = (2.15)^2/(2)(32.2)$	Velocity head in conduit
		$= 0.07$ ft	
Step 6	Col. 8A	$S_f = [(Qn)/(K_Q D^{2.67})]^2$	Equation 1.18
		$S_f = [(6.75)(0.013)/(0.46)(2.0)^{2.67}]^2$	
		$S_f = 0.00090$ ft/ft	
Step 7	Col. 2B	$H_f = S_f L$	Equation 1.27
		$H_f = (0.0009)\ (55.8)$	Col. 8A x Col. 4A
		$H_f = 0.05$	
	Col. 7B &	h_b, H_c, H_e, $H_j = 0$	
	Col. 9A	Total = 0.05 ft	

ENERGY GRADE LINE COMPUTATION SHEET - TABLE A
(English Solution)

COMPUTED BY _____ DATE _____
CHECKED BY _____ DATE _____
PAGE _____ OF _____
INITIAL TAILWATER ELEV. _____

ROUTE _____
SECTION _____
COUNTY _____

Str. ID	D	Q	L	V	d	d_o	$V^2/2g$	S_f	Total Pipe Loss (table B)	EGL_o	K table B	$K(V^2/2g)$	EGL_i	HGL	U/S TOC	Surf. Elev.
	(ft)	(ft^3/s)	(ft)	(fps)	(ft)		(ft)	(ft / ft)	(ft)	(ft)		(ft)	(ft)	(ft)	(ft)	(ft)
(1)	(2)	(3)	(4)	(5)	(6a)	(6b)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16
OUTLET										333.00				333.00		
44													333.07		332.71	332.71
43	2.0	6.75	55.8	2.15	FULL	n/a	0.07	0.0009	0.05	333.12	0.5	0.04	333.16	*333.16	346.06	347.76
											*(Drop Structure)					
43	(New Outlet)			2.6		0.8	0.10						345.56	345.46	346.06	347.76
	(Above calculations are for inlet end of STR 43 for conduit 42-43)															
42	2.0	6.75	14.1	2.6	1.56	0.80	0.10	0.001	0.014	345.57	0.62	0.06	346.11	346.01	345.73	349.31
41	1.5	5.10	328.0	8.65	0.56	0.85	1.16	-	0	355.79	-	-	355.98	355.10	356.17	360.0
40	1.5	3.35	361.0	7.52	0.43	0.70	0.88		0	-	0	0	366.50	366.50	367.0	370.0

Figure 1.32 Energy Grade Line Computation Sheet, Table A, for English Example

ENERGY GRADE LINE COMPUTATION SHEET - TABLE B
(English Solution)

COMPUTED BY _____ DATE _____ ROUTE _____
CHECKED BY _____ DATE _____ SECTION _____
PAGE _____ OF _____ COUNTY _____

Str. ID (1)	Pipe Losses (ft)						Structure Losses (ft)							
	H_f (2)	h_v (3)	H_c (4)	H_e (5)	H_j (6)	Total (7)	d_{sho} (8)	K_o (9)	C_D (10)	C_d (11)	C_Q (12)	C_p (13)	C_B (14)	K (15)
44														
	0.05					0.05								
43							1.89							0.5
	0.014					0.014								
42							1.40	1.55	1.0	0.40	1.0	1.0	1.0	0.62
						0.0								
41														
														0.0
40							1.0							

Figure 1.33 Energy Grade Line Computation Sheet, Table B, for English Example

Step 8	Col. 10A	$EGL_o = EGL_i$ + pipe loss EGL_o = 333.07 + 0.05 EGL_o = 333.12 ft	
		HGL = 333.12 - 0.07 = 333.05	Check for full flow - close
		TOC = 331.27 + 2.0 = 333.27	Assumption OK
Step 9	Col. 8B	Not applicable due to drop structure	
Step 10	Col. 9B and 11A	K_e = 0.5	Inflow pipe invert much higher than d_{aho}. Assume square edge entrance
Step 17	Col. 12A	$K(V^2/2g)$ = (0.50)(0.07) $K(V^2/2g)$ = 0.04 ft	Col. 11A times Col. 7A
Step 18	Col. 13A	$EGL_i = EGL_o$ EGL_i = 333.12 + 0.04 EGL_i = 333.16 ft	Col 10A plus 12A
Step 19	Col. 14A	HGL = EGL_i = 333.16 ft	For drop structures, the HGL is the same as the EGL
		d_{aho} = HGL- invert = 333.16 - 331.27 = 1.89 ft	Col. 8B
Step 20	Col. 15A	U/S TOC = Inv. + Dia. U/S TOC = 344.06 + 2.0 U/S TOC = 346.06 ft	From storm drain comp. sheet (Figure 1.32)
Step 21	Col. 16A	Surf. Elev. = 347.76 ft 347.76 > 333.09	From Figure 1.31. Surface elev. exceeds HGL, OK
Step 2	Col. 1A, 1B	Str. ID = 43	Drop Structure - new start
	Col. 15A	U/S TOC = 344.06 + 2.0 = 346.06	
	Col. 16A	Surface Elev = 347.76	
Step 3		HGL' = inv. + (d_c+D)/2 HGL' = 344.06 + (0.80 + 2.0)/2	Calculate new HGL - Use Case 2 d_c from Figure 1.19b
	Col. 14A	HGL = 345.46 ft	
		EGL = HGL + $V^2/2g$	V = 2.6 fps from Prelim. Comp. Sht.
	Col. 13A	EGL = 345.46 + 0.10 EGL = 345.56 ft	

Structure 42

Step 4	Col.1A	Str. ID = 42	
	Col. 2A	D = 2.0 ft	Pipe Diameter
	Col. 3A	Q = 6.75 cfs	Conduit discharge (design value)
	Col. 4A	L = 14.1 ft	Conduit length
Step 5A	Col. 5A	V = 2.6 ft/s Q/Qf = 6.75 / 7.12 = 0.95	For flow: Actual velocity from storm drain computation sheet.
	Col. 6A	d_n = 1.56 ft Chart 26	Figure 1.32
	Col. 7A	$V^2/2g = (2.6)^2/(2)(32.2)$ $V^2/2g$ = 0.10 ft	Velocity head in conduit
Step 5B	Col.6bA	d_c = 0.80 ft	From HDS-5
Step 5C		$d_n < d_c$	Flow is subcritical
Step 6	Col. 8A	S_f = 0.001	Conduit not full so S_f = pipe slope d_n = 1.56 (Figure 1.19a) d_c = 0.80 (HDS-5) Flow is subcritical
Step 7		$H_f = S_f L$ H_f = (0.001) (14.1)	Equation 1.27 Col. 8A x Col. 5A
	Col. 2B	H_f = 0.014 ft h_b, H_c, H_e, H_j = 0	
	Col. 7B and 9A	Total = 0.014 ft	
Step 8		$EGL_o = EGL_i$ + total pipe loss EGL_o = 345.56 + 0.014	Col. 14A plus Col. 9A
	Col. 10A	EGL_o = 345.57 ft	
Step 9	Col. 8B	d_{aho} = EGL_o - velocity head - pipe invert d_{aho} = 345.57 - 0.10 - 344.07 d_{aho} = 1.40 ft	Col. 10A - Column 7A - pipe invert
Step 10	Col. 9B	$K_o = 0.1(b/D_o)(1-\sin\theta)+1.4(b/D_o)^{0.15}\sin(\theta)$ b = 4.0 ft D_o = 2.0 ft $\theta = 90^o$ $K_o = 0.1(4.0/2.0)(1 - \sin 90)+ 1.4(4.0/2.0)^{0.15}\sin 90$ K_o = 1.55	Equation 1.38 Access hole diameter. Col. 2A - outlet pipe diam Flow deflection angle
Step 11	Col. 10B	$C_D = (D_o/D_i)^3$ d_{aho} = 1.40 d_{aho}/D_o = (1.40/2.0) d_{aho}/D_o = 0.70 < 3.2 C_D = 1.0	Equation 1.39; pipe diameter Column 8B therefore

Step 12	Col. 11B	$C_d = 0.5 (d_{aho}/ D_o)^{0.6}$ $d_{aho}/D_o = 0.70 < 3.2$ $C_d = 0.5 (1.4/2.0)^{0.6}$ $C_d = 0.40$	Equation 1.40; Flow depth correction.
Step 13	Col. 12B	$C_Q = (1\text{-}2 \sin \theta)(1\text{-}Q_i/Q_o)^{0.75}+1$ $C_Q = 1.0$	Equation 1.41; relative flow No additional pipes entering
Step 14	Col. 13B	$C_p = 1 + 0.2(h/D_o)[(h\text{-}d)/D_o]$ $C_p = 1.0$	Equation 1.42; plunging flow No plunging flow
Step 15	Col. 14B	$C_B = 1.0$	Benching Correction, flat floor (Table 1.17)
Step 16	Col. 15B and 11A	$K = K_o C_D C_d C_Q C_p C_B$ $K = (1.55)(1.0)(0.40)(1.0)(1.0)(1.0)$ $K = 0.62$	Equation 1.37
Step 17	Col. 12A	$K(V^2/2g) = (0.62)(0.10)$ $K(V^2/2g) = 0.06$ ft	Col. 11A times Col. 7A
Step 18	Col. 13A	$EGL_i = EGL_o + K(V_2/2g)$ $EGL_i = 346.05 + 0.06$ $EGL_i = 346.11$	Col. 10A plus 12A
Step 19	Col. 14A	$HGL = EGL_i - V^2/2g$ HGL = 346.11 - 0.10 HGL = 346.01 ft	Col. 13A minus Col. 7A
Step 20	Col 15A	U/S TOC = Inv. + Dia. U/S TOC = 344.23 + 1.5 U/S TOC = 345.73 ft	Information from storm drain comp.sheet (Figure 1.31)
Step 21	Col 16A	Surf. Elev. = 349.31 ft 349.31 > 345.96	From Figure 1.30 Surface elev. exceeds HGL, OK

Structure 41

Step 4	Col. 1A, 1B	Str. ID = 41	Next Structure
	Col. 2A	D = 1.50 ft	Pipe Diameter
	Col. 3A	Q = 5.10 cfs	Conduit discharge (design value)
	Col. 4A	L = 328 ft	Conduit length
Step 5	Part full flow from column's 12 and 15 of storm drain computation sheet.		Continue with Step 5A
Step 5A		$Q/Q_f = 5.1/18.1 = 0.28$ $d/d_f = 0.37$ d = (0.37) (1.5)	Figure 1.19a
	Col. 6aA	d = 0.56 ft	
		$V/V_f = 0.84$ V = (0.84)(10.3)	Figure 1.19a
	Col. 5A	V = 8.65 fps	

		$V^2/2g = (8.65)_2/(2)(32.2)$	Velocity head
	Col. 7A	$V^2/2g = 1.16$ ft	
Step 5B	Col. 6bA	$d_c = 0.85$ ft	Figure 1.19b
Step 5C		$0.56 < 0.85$	Supercritical flow since $d_n < d_c$
Step 5D	Col. 7B	Total pipe loss = 0	

Structure 40

Step 5E	Col. 1A,1B	Str. Id. = 40	Next structure
	Col. 2A	D = 1.5 ft	Pipe diameter
	Col. 3A	Q = 3.35 cfs	Conduit discharge (design)
	Col. 4A	L = 361.0 ft	Conduit length
Step 5F		$Q/Q_f = 3.3/18.1 = 0.18$	
		$d/d_c = 0.29$	Figure 1.19a
		d = (0.29)(1.5)	
	Col. 6aA	d = 0.43 ft	
	Col. 6bA	$d_c = 0.7$ ft	Figure 1.19b
Step 5H		$V/V_f = 0.73$	Figure 1.19a
		V = (0.73)(10.3)	
	Col. 5A	V = 7.52 fps	
	Col. 7A	$V^2/2g = (7.52)^2/(2)(32.2)$	Velocity head
		$V^2/2g = 0.88$ ft	
Step 5I		$d_n = 0.43$ ft < 0.70 ft $= d_c$	Supercritical flow since $d_n < d_c$
Step 5K	Col. 11A, and 15B	K = 0.0	Str. 41 line; supercritical flow;
	Col. 12A	$K(V^2/2g) = 0$	no structure losses

Since both conduits 42-41 and 41-40 are supercritical - establish HGL and EGL at each side of access hole 41.

		HGL = Inv. + d	
		HGL = 354.07 + 0.56	D/S Invert + Flow depth
		HGL = 354.63 ft	
		EGL = 354.63 + 1.16	
		HGL + velocity head	
	Col. 10A	EGL = 355.79 ft	EGL_o of Str.41
		HGL = 354.67 + 0.43	U/S invert + Flow depth
	Col. 14A	HGL = 355.10 ft	Highest HGL
		EGL = 355.10 + 0.88	HGL + velocity head
	Col. 13A	EGL = 355.98 ft	EGL_i of Str. 41
Step 20	Col. 15A	U/S TOC = Inv. + Dia.	Information from storm drain comp
		U/S TOC = 354.67 + 1.5	Sheet (Figure 1.31) for Str. 41
		U/S TOC = 356.17 ft	
Step 21	Col. 16A	Surf. Elev. = 360.0 ft	From Figure 1.30.
		360.0 > 355.10	Surface elev. > HGL, OK

Step 10b	Col. 8B	d_{aho} = 0.67 (1.5) = 1.0 ft	Figure 1.31, HW/D = 0.67
		HGL = Str. 40 Inv. + d_{aho}	Structure Inv. from storm
		HGL = 365.50 + 1.0.	drain comp. sheet
	Col. 14A	HGL = 366.50 ft	
	Col.13A	EGL = 366.50 ft	Assume no velocity in str.
Step 20	Col. 15A	U/S TOC = Inv. + Dia.	Information from storm drain
		U/S TOC = 365.5 + 1.5	comp. sheet (Figure 1.31) for Str.
		U/S TOC = 367.0 ft	40
Step 21	Col. 16A	Surf. Elev. = 370.0 ft	From Figure 1.30
		370.0 ft > 366.50 ft	Surface Elev. > HGL, OK

See Figures 1.32 and 1.33 for the tabulation of results. The final HGL values are indicated in Figure 1.30.

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2.0 Storage Design

2.1 General Storage Concepts

2.1.1 Introduction

This section provides general guidance on stormwater runoff storage for meeting stormwater management control objectives (i.e., water quality protection, downstream streambank protection, and flood control).

Storage of stormwater runoff within a stormwater management system is essential to providing the extended detention of flows for water quality protection and downstream streambank protection, as well as for peak flow attenuation of larger flows for flood protection. Runoff storage can be provided within an on-site system through the use of structural stormwater controls and/or nonstructural features and landscaped areas. Figure 2.1 illustrates various storage facilities that can be considered for a development site.

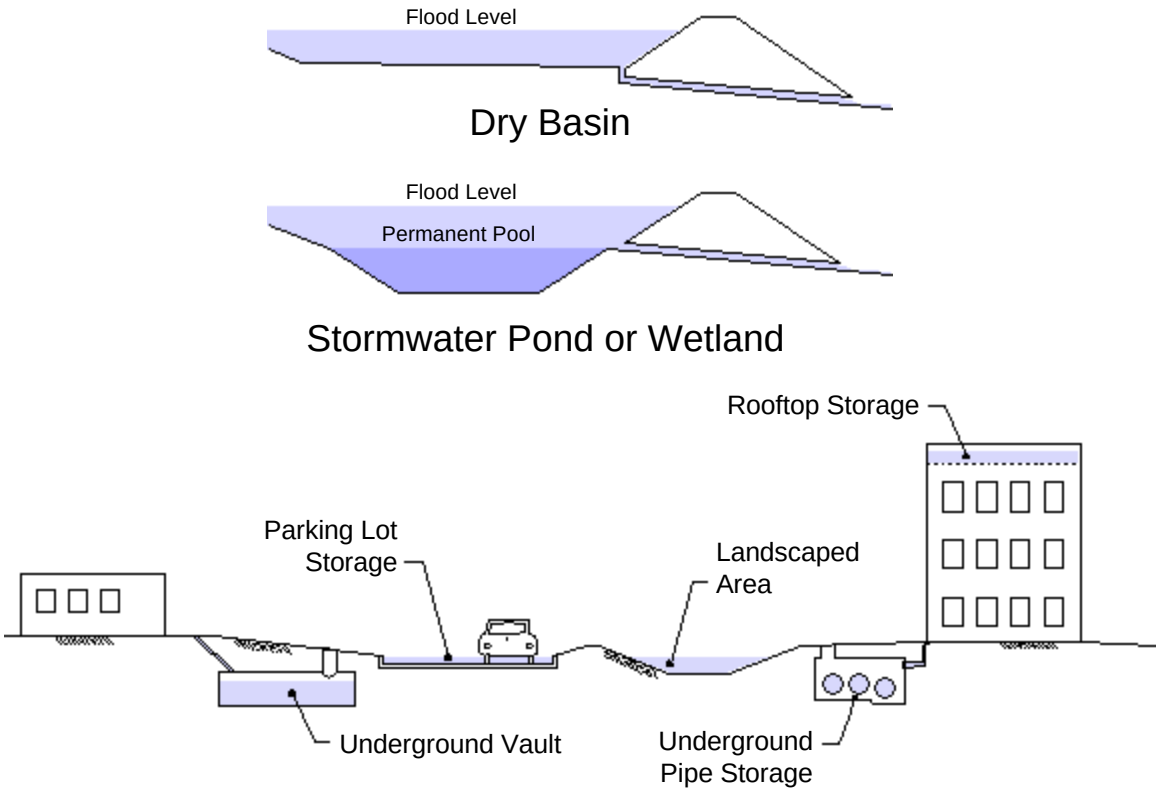


Figure 2.1 Examples of Typical Stormwater Storage Facilities

2.1.2 Storage Classification

Stormwater storage(s) can be classified as either detention, extended detention or retention. Some facilities include one or more types of storage.

Stormwater *detention* is used to reduce the peak discharge and detain runoff for a specified short period of time. Detention volumes are designed to completely drain after the design storm has passed. Detention is used to meet streambank protection criteria, and flood criteria where required.

Extended detention (ED) is used to drain a runoff volume over a specified period of time, typically 24 hours, and is used to meet streambank protection criteria. Some structural control designs (wet ED pond, micropool ED pond, and shallow ED marsh) also include extended detention storage of a portion of the water quality protection volume.

Retention facilities are designed to contain a permanent pool of water, such as stormwater ponds and wetlands, which is used for water quality protection.

Storage facilities are often classified on the basis of their location and size. *On-site* storage is constructed on individual development sites. *Regional* storage facilities are constructed at the lower end of a subwatershed and are designed to manage stormwater runoff from multiple projects and/or properties. A discussion of regional stormwater controls is found in *Section 1.0 of the Site Development Controls Technical Manual*.

Storage can also be categorized as *on-line* or *off-line*. On-line storage uses a structural control facility that intercepts flows directly within a conveyance system or stream. Off-line storage is a separate storage facility to which flow is diverted from the conveyance system. Figure 2.2 illustrates on-line versus off-line storage.

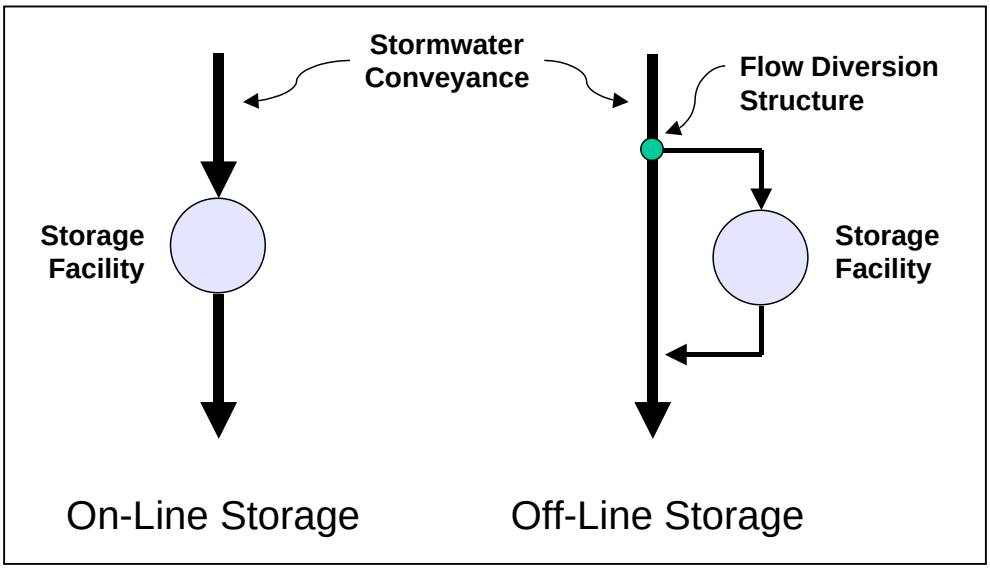


Figure 2.2 On-Line versus Off-Line Storage

2.1.3 Stage-Storage Relationship

A stage-storage curve defines the relationship between the depth of water and storage volume in a storage facility (see Figure 2.3). The volume of storage can be calculated by using simple geometric formulas expressed as a function of depth.

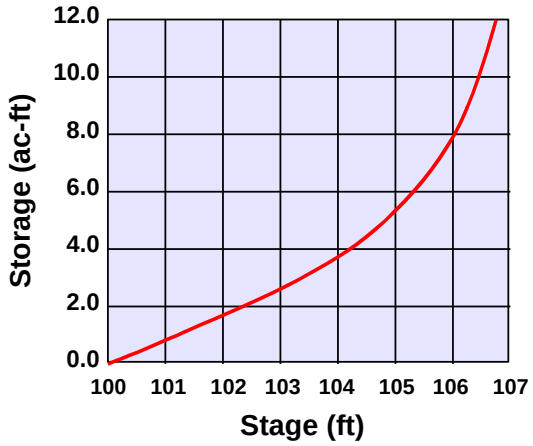


Figure 2.3 Stage-Storage Curve

The storage volume for natural basins may be developed using a topographic map and the double-end area, frustum of a pyramid, prismoidal or circular conic section formulas.

The double-end area formula (see Figure 2.4) is expressed as:

$$V_{1,2} = [(A_1 + A_2)/2]d \qquad (2.1)$$

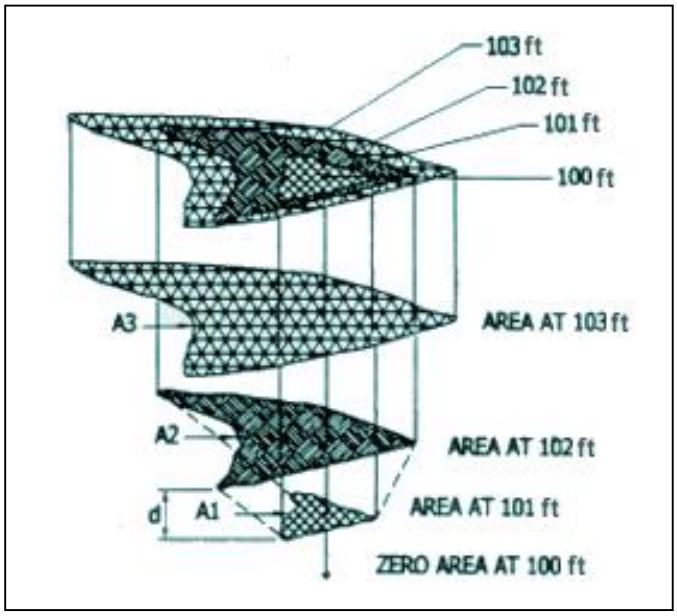


Figure 2.4 Double-End Area Method

where:

$V_{1,2}$ = storage volume (ft^3) between elevations 1 and 2
A_1 = surface area at elevation 1 (ft^2)
A_2 = surface area at elevation 2 (ft^2)
d = change in elevation between points 1 and 2 (ft)

The frustum of a pyramid formula is expressed as:

$$V = d/3 [A_1 + (A_1 \times A_2)^{0.5} + A_2]/3 \quad (2.2)$$

where:

V = volume of frustum of a pyramid (ft^3)
d = change in elevation between points 1 and 2 (ft)
A_1 = surface area at elevation 1 (ft^2)
A_2 = surface area at elevation 2 (ft^2)

The prismoidal formula for trapezoidal basins is expressed as:

$$V = LWD + (L + W) ZD^2 + 4/3 Z^2 D^3 \quad (2.3)$$

where:

V = volume of trapezoidal basin (ft^3)
L = length of basin at base (ft)
W = width of basin at base (ft)
D = depth of basin (ft)
Z = side slope factor, ratio of horizontal to vertical

The circular conic section formula is:

$$V = 1.047 D (R_1^2 + R_2^2 + R_1R_2) \quad (2.4)$$

$$V = 1.047 D (3 R_1^2 + 3ZDR_1 + Z_2D^2) \quad (2.5)$$

where:

R_1, R_2 = bottom and surface radii of the conic section (ft)
D = depth of basin (ft)
Z = side slope factor, ratio of horizontal to vertical

2.1.4 Stage-Discharge Relationship

A stage-discharge curve defines the relationship between the depth of water and the discharge or outflow from a storage facility (see Figure 2.5). A typical storage facility has two outlets or spillways: a principal outlet and a secondary (or emergency) outlet. The principal outlet is usually designed with a capacity sufficient to convey the design flows without allowing flow to enter the emergency spillway. A pipe culvert, weir, or other appropriate outlet can be used for the principal spillway or outlet.

The emergency spillway is sized to provide a bypass for floodwater during a flood that exceeds the design capacity of the principal outlet. This spillway should be designed taking into account the potential threat to downstream areas if the storage facility were to fail. The stage-discharge curve should take into account the discharge characteristics of both the principal spillway and the emergency spillway. For more details, see *Section 2.2*.

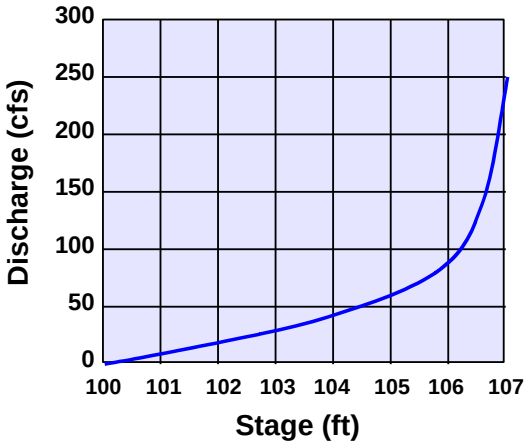


Figure 2.5 Stage-Discharge Curve

2.1.5 Symbols and Definitions

To provide consistency within this section as well as throughout this Manual, the symbols listed in Table 2.1 will be used. These symbols were selected because of their wide use in technical publications. In some cases, the same symbol is used in existing publications for more than one definition. Where this occurs in this section, the symbol will be defined where it occurs in the text or equations.

Table 2.1 Symbols and Definitions

Symbol	Definition	Units
A	Cross sectional or surface area	ft^2
A_m	Drainage area	mi^2
C	Weir coefficient	-
d	Change in elevation	ft
D	Depth of basin or diameter of pipe	ft
t	Routing time period	sec
g	Acceleration due to gravity	ft/s^2
H	Head on structure	ft
H_C	Height of weir crest above channel bottom	ft
K	Coefficient	-
I	Inflow rate	cfs
L	Length	ft
Q, q	Peak inflow or outflow rate	cfs, in
R	Surface Radii	ft
S, V_S	Storage volume	ft^3
t_b	Time base on hydrograph	hrs
T_I	Duration of basin inflow	hrs
t_P	Time to peak	hrs
V_S, S	Storage volume	ft^3, in, acre-ft
V_r	Volume of runoff	ft^3, in, acre-ft
W	Width of basin	ft
Z	Side slope factor	-

2.1.6 General Storage Design Procedures

Introduction

This section discusses the general design procedures for designing storage to provide standard detention of stormwater runoff for flood control (Q_f).

The design procedures for all structural control storage facilities are the same whether or not they include a permanent pool of water. In the latter case, the permanent pool elevation is taken as the "bottom" of storage and is treated as if it were a solid basin bottom for routing purposes.

It should be noted that the location of structural stormwater controls is very important as it relates to the effectiveness of these facilities to control downstream impacts. In addition, multiple storage facilities located in the same drainage basin will affect the timing of the runoff through the conveyance system, which could decrease or increase flood peaks in different downstream locations. Therefore, a downstream peak flow analysis should be performed as part of the storage facility design process (see *Section 2.0 of the Hydrology Technical Manual*).

In multi-purpose multi-stage facilities such as stormwater ponds, the design of storage must be integrated with the overall design for water quality protection objectives. See *Section 1.0 of the Site Development Controls Technical Manual* for further guidance and criteria for the design of structural stormwater controls.

Data Needs

The following data are needed for storage design and routing calculations:

- Inflow hydrograph for all selected design storms (this can be generated using a unit hydrograph method or the Modified Rational Method, see the *Hydrology Technical Manual* for more details)
- Stage-storage curve for proposed storage facility
- Stage-discharge curve for all outlet control structures

Design Procedure

A general procedure for using the above data in the design of storage facilities is presented below.

Step 1 Compute inflow hydrograph for runoff from the "Conveyance" (e.g., Q_{p25}) and flood mitigation (Q_{p100}) design storms using the hydrologic methods outlined in *Section 1.0 of the Hydrology Technical Manual*. Both existing- and post-development hydrographs are required for both the "Conveyance" and flood mitigation design storms.

Step 2 Perform preliminary calculations to evaluate detention storage requirements for the hydrographs from Step 1 (see *Section 2.1.7*).

Step 3 Determine the physical dimensions necessary to hold the estimated volume from Step 2, including freeboard. The maximum storage requirement calculated from Step 2 should be used. From the selected shape determine the maximum depth in the pond.

Step 4 Select the type of outlet and size the outlet structure. The estimated peak stage will occur for the estimated volume from Step 2. The outlet structure should be sized to convey the allowable discharge at this stage.

Step 5 Perform routing calculations using inflow hydrographs from Step 1 to check the preliminary design using a storage routing computer model. If the routed post-development peak discharges from the "Conveyance" design storm exceed the existing-development peak discharges, then revise the available storage volume, outlet device, etc., and return to Step 3.

Step 6 Perform routing calculations using the flood mitigation hydrograph to determine if any increases in downstream flows from this hydrograph will cause damages and/or drainage and flooding problems. If problems will be created (e.g., flooding of habitable dwellings, property damage, or public access and/or utility interruption) then the storage facility must be designed to control the increased flows from the flood mitigation storm. If not then consider emergency overflow from runoff due to the flood mitigation (or larger) design storm and established freeboard requirements.

Step 7 Evaluate the downstream effects of detention outflows for the "Conveyance" and flood mitigation storms to ensure that the routed hydrograph does not cause downstream flooding problems. The exit hydrograph from the storage facility should be routed though the downstream channel system to the location where the discharge from the proposed development no longer has a significant impact upon the receiving stream or storm drainage system (see *Section 2.0 of the Hydrology Technical Manual*).

Step 8 Evaluate the control structure outlet velocity and provide channel and bank stabilization if the velocity will cause erosion problems downstream.

Routing of hydrographs through storage facilities is critical to the proper design of these facilities. Although storage design procedures using inflow/outflow analysis without routing have been developed, their use in designing detention facilities has not produced acceptable results in many areas of the country, including North Central Texas.

Although hand calculation procedures are available for routing hydrographs through storage facilities, they are very time consuming, especially when several different designs are evaluated. Many standard hydrology and hydraulics textbooks give examples of hand-routing techniques. For this Manual, it assumed that designers will be using one of the many computer programs available for storage routing and thus other procedures and example applications will not be given here.

2.1.7 Preliminary Detention Calculations

Introduction

Procedures for preliminary detention calculations are included here to provide a simple method that can be used to estimate storage needs and also provide a quick check on the results of using different computer programs. Standard routing should be used for actual (final) storage facility calculations and design.

Storage Volume

For small drainage areas, a preliminary estimate of the storage volume required for peak flow attenuation may be obtained from a simplified design procedure that replaces the actual inflow and outflow hydrographs with the standard triangular shapes shown in Figure 2.6.

The required storage volume may be estimated from the area above the outflow hydrograph and inside the inflow hydrograph, expressed as:

$$V_S = 0.5T_i (Q_i - Q_O) \qquad (2.6)$$

where:

V_S = storage volume estimate (ft^3)
Q_i = peak inflow rate (cfs)
Q_O = peak outflow rate (cfs)
T_i = duration of basin inflow (s)

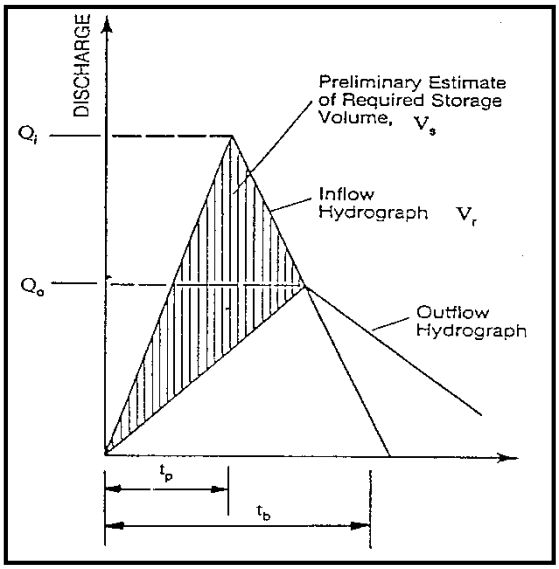


Figure 2.6 Triangular-Shaped Hydrographs
(For Preliminary Estimate of Required Storage Volume)

Alternative Method

An alternative preliminary estimate of the storage volume required for a specified peak flow reduction can be obtained by the following regression equation procedure (Wycoff and Singh, 1976).

Determine input data, including the allowable peak outflow rate, Q_O, the peak flow rate of the inflow hydrograph, Q_i, the time base of the inflow hydrograph, t_b, and the time to peak of the inflow hydrograph, t_p.

Calculate a preliminary estimate of the ratio V_S/V_r using the input data from Step 1 and the following equation:

$$\mathbf{V_s/V_r = \frac{1.291(1 - Q_o/Q_i)^{0.753}}{(t_b/t_p)^{0.411}}} \qquad \mathbf{(2.7)}$$

where:

- V_S = volume of storage (in)
- V_r = volume of runoff (in)
- Q_O = outflow peak flow (cfs)
- Q_i = inflow peak flow (cfs)
- t_b = time base of the inflow hydrograph (hr) [Determined as the time from the beginning of rise to a point on the recession limb where the flow is 5% of the peak]
- t_p = time to peak of the inflow hydrograph (hr)

Multiply the volume of runoff, V_r, times the ratio V_S/V_r, calculated in Step 2 to obtain the estimated storage volume V_S.

Peak Flow Reduction

A preliminary estimate of the potential peak flow reduction for a selected storage volume can be obtained by the following procedure.

Determine volume of runoff, V_r, peak flow rate of the inflow hydrograph, Q_i, time base of the inflow hydrograph, t_b, time to peak of the inflow hydrograph, t_p, and storage volume V_S.

Calculate a preliminary estimate of the potential peak flow reduction for the selected storage volume using the following equation (Wycoff and Singh, 1976):

$$\mathbf{Q_O/Q_i = 1 - 0.712(V_S/V_r)^{1.328}(t_b/t_p)^{0.546}} \qquad \mathbf{(2.8)}$$

where:

- Q_O = outflow peak flow (cfs)
- Q_i = inflow peak flow (cfs)
- V_S = volume of storage (in)
- V_r = volume of runoff (in)
- t_b = time base of the inflow hydrograph (hr) [Determined as the time from the beginning of rise to a point on the recession limb where the flow is 5 percent of the peak]
- t_p = time to peak of the inflow hydrograph (hr)

Multiply the peak flow rate of the inflow hydrograph, Q_i, times the potential peak flow reduction calculated from Step 2 to obtain the estimated peak outflow rate, Q_O, for the selected storage volume.

2.2 Outlet Structures

2.2.1 Symbols and Definitions

To provide consistency within this section as well as throughout this Manual, the symbols listed in Table 2.2 will be used. These symbols were selected because of their wide use in technical publications. In some cases, the same symbol is used in existing publications for more than one definition. Where this occurs in this section, the symbol will be defined where it occurs in the text or equations.

Table 2.2 Symbols and Definitions

Symbol	Definition	Units
A, a	Cross sectional or surface area	ft^2
A_m	Drainage area	mi^2
B	Breadth of weir	ft
C	Weir coefficient	-
d	Change in elevation	ft
D	Depth of basin or diameter of pipe	ft
g	Acceleration due to gravity	ft/s^2
H	Head on structure	ft
H_C	Height of weir crest above channel bottom	ft
K, k	Coefficient	-
L	Length	ft
n	Manning's n	-
Q, q	Peak inflow or outflow rate	cfs, in
V_u	Approach velocity	ft/s
WQ_v	Water quality protection volume	ac ft
w	Maximum cross sectional bar width facing the flow	in
x	Minimum clear spacing between bars	in
θ	Angle of v-notch	degrees
θ_g	Angle of the grate with respect to the horizontal	degrees

2.2.2 Primary Outlets

Introduction

Primary outlets provide the critical function of the regulation of flow for structural stormwater controls. There are several different types of outlets that may consist of a single stage outlet structure, or several outlet structures combined to provide multi-stage outlet control.

For a single stage system, the stormwater facility can be designed as a simple pipe or culvert. For multistage control structures, the inlet is designed considering a range of design flows.

A stage-discharge curve is developed for the full range of flows that the structure would experience. The outlets are housed in a riser structure connected to a single outlet conduit. An alternative approach would be to provide several pipe or culvert outlets at different levels in the basin that are either discharged separately or are combined to discharge at a single location.

This section provides an overview of outlet structure hydraulics and design for stormwater storage facilities. The design engineer is referred to an appropriate hydraulics text for additional information on outlet structures not contained in this section.

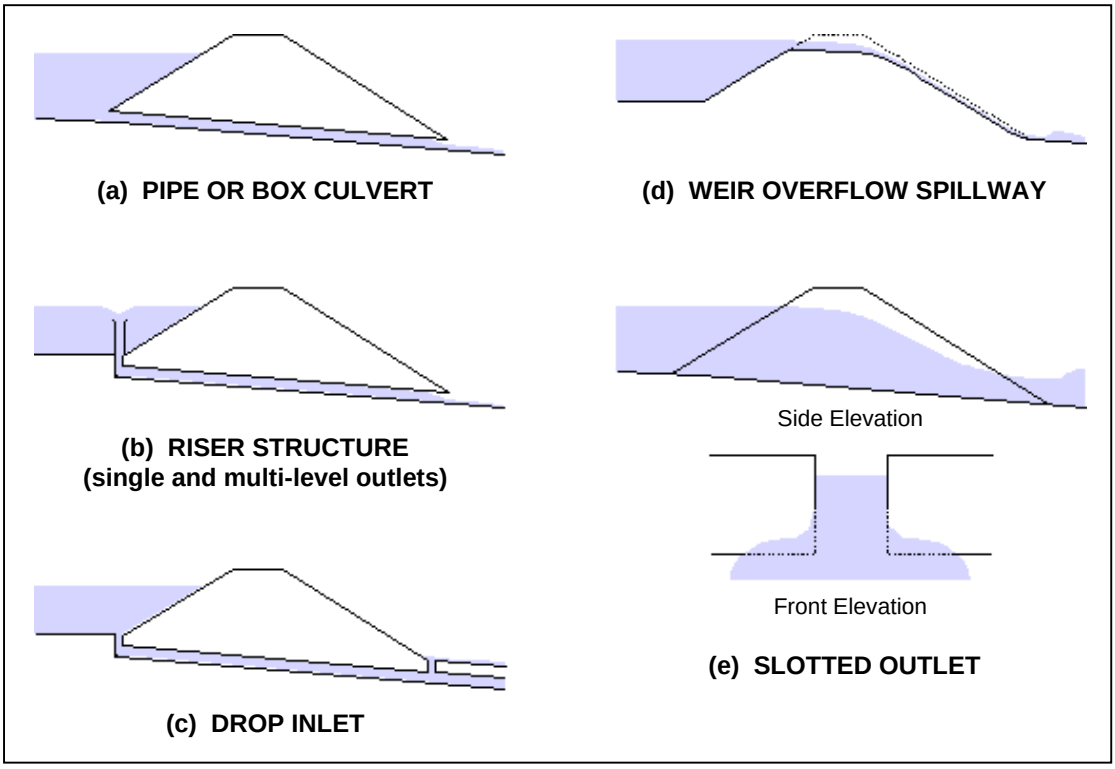


Figure 2.7 Typical Primary Outlets

Outlet Structure Types

There are a wide variety of outlet structure types, the most common of which are covered in this section. Descriptions and equations are provided for the following outlet types for use in stormwater facility design:

- Orifices
- Perforated risers
- Pipes / Culverts
- Sharp-crested weirs
- Broad-crested weirs
- V-notch weirs
- Proportional weirs
- Combination outlets

The design professional must pay attention to material types and construction details when designing an outlet structure or device. Non-corrosive material and mounting hardware are key to device longevity, ease of operation, and low cost maintenance. Special attention must also be paid to not placing dissimilar metal materials together where a cathodic reaction will cause deterioration and destruction of metal parts.

Protective coatings, paints, and sealants must also be chosen carefully to prevent contamination of the stormwater flowing through the structure/device. This is not only important while they are being applied, but also as these coating deteriorate and age over the functional life of the facility.

Each of these outlet types has a different design purpose and application:

- Water quality and streambank protection flows are normally handled with smaller, more protected outlet structures such as reverse slope pipes, hooded orifices, orifices located within screened pipes or risers, perforated plates or risers, and V-notch weirs.

- Larger flows, such as flood flows, are typically handled through a riser with different sized openings, through an overflow at the top of a riser (drop inlet structure), or a flow over a broad crested weir or spillway through the embankment. Overflow weirs can also be of different heights and configurations to handle control of multiple design flows.

Orifices

An orifice is a circular or rectangular opening of a prescribed shape and size. The flow rate depends on the height of the water above the opening and the size and edge treatment of the orifice.

For a single orifice, as illustrated in Figure 2.8(a), the orifice discharge can be determined using the standard orifice equation below.

$$\mathbf{Q = CA\,(2gH)^{0.5}} \qquad \mathbf{(2.9)}$$

where:

Q = the orifice flow discharge (cfs)
C = discharge coefficient
A = cross-sectional area of orifice or pipe (ft^2)
g = acceleration due to gravity (32.2 ft/s^2)
H = effective head on the orifice, from the center of orifice to the water surface (ft)

If the orifice discharges as a free outfall, then the effective head is measured from the center of the orifice to the upstream (headwater) surface elevation. If the orifice discharge is submerged, then the effective head is the difference in elevation of the headwater and tailwater surfaces as shown in Figure 2.8(b).

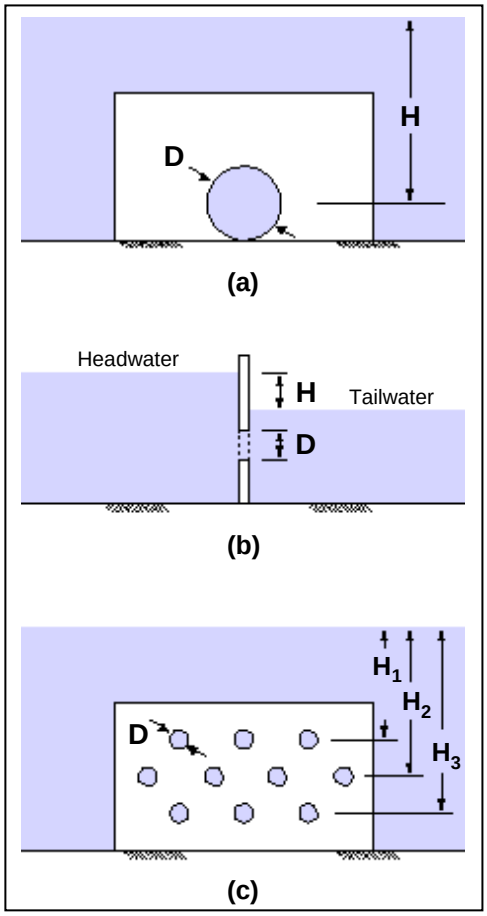


Figure 2.8 Orifice Definitions

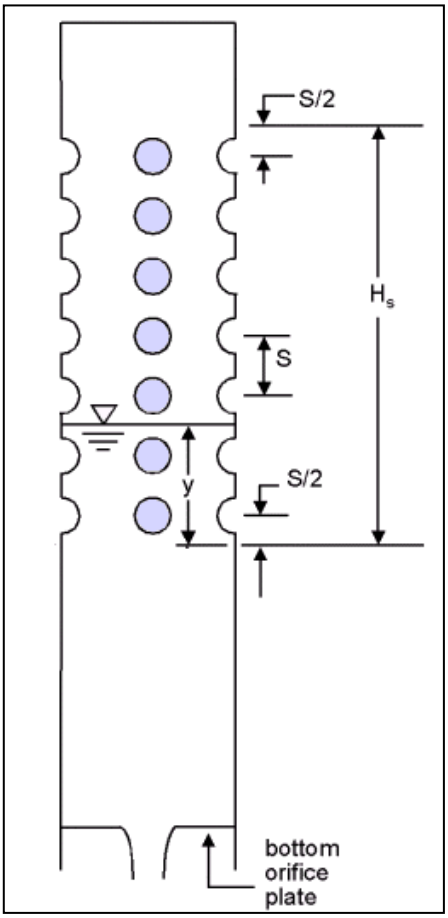


Figure 2.9 Perforated Riser

When the material is thinner than the orifice diameter, with sharp edges, a coefficient of 0.6 should be used. For square-edged entrance conditions the generic orifice equation can be simplified:

$$Q = 0.6A\,(2gH)^{0.5} = 3.78D^2H^{0.5} \qquad (2.10)$$

where:

D = diameter of orifice or pipe (ft)

When the material is thicker than the orifice diameter a coefficient of 0.80 should be used. If the edges are rounded, a coefficient of 0.92 can be used.

Flow through multiple orifices, such as the perforated plate shown in Figure 2.8(c), can be computed by summing the flow through individual orifices. For multiple orifices of the same size and under the influence of the same effective head, the total flow can be determined by multiplying the discharge for a single orifice by the number of openings.

Perforated orifice plates for the control of discharge can be of any size and configuration. However, the Denver Urban Drainage and Flood Control District has developed standardized dimensions that have worked well. Table 2.3 gives appropriate dimensions. The vertical spacing between hole centerlines is always 4 inches.

Table 2.3 Circular Perforation Sizing

Hole Diameter (in)	Minimum Column Hole Centerline Spacing (in)	Flow Area per Row (in^2)		
		1 column	2 columns	3 columns
1/4	1	0.05	0.1	0.15
5/16	2	0.08	0.15	0.23
3/8	2	0.11	0.22	0.33
7/16	2	0.15	0.3	0.45
1/2	2	0.2	0.4	0.6
9/16	3	0.25	0.5	0.75
5/8	3	0.31	0.62	0.93
11/16	3	0.37	0.74	1.11
3/4	3	0.44	0.88	1.32
13/16	3	0.52	1.04	1.56
7/8	3	0.6	1.2	1.8
15/16	3	0.69	1.38	2.07
1	4	0.79	1.58	2.37
1 1/16	4	0.89	1.78	2.67
1 1/8	4	0.99	1.98	2.97
1 3/16	4	1.11	2.22	3.33
1 1/4	4	1.23	2.46	3.69
1 5/16	4	1.35	2.7	4.05
1 3/8	4	1.48	2.96	4.44
1 7/16	4	1.62	3.24	4.86
1 1/2	4	1.77	3.54	5.31
1 9/16	4	1.92	3.84	5.76
1 5/8	4	2.07	4.14	6.21
1 11/16	4	2.24	4.48	6.72
1 3/4	4	2.41	4.82	7.23
1 13/16	4	2.58	5.16	7.74
1 7/8	4	2.76	5.52	8.28
1 15/16	4	2.95	5.9	8.85
2	4	3.14	6.28	9.42
Number of columns refers to parallel columns of holes				
Minimum plate thickness		1/4”	5/16”	3/8”

Source: Urban Drainage and Flood Control District, Denver, CO

For rectangular slots the height is normally 2 inches with variable width. Only one column of rectangular slots is allowed.

Figure 2.10 provides a schematic of an orifice plate outlet structure for a wet extended detention pond showing the design pool elevations and the flow control mechanisms.

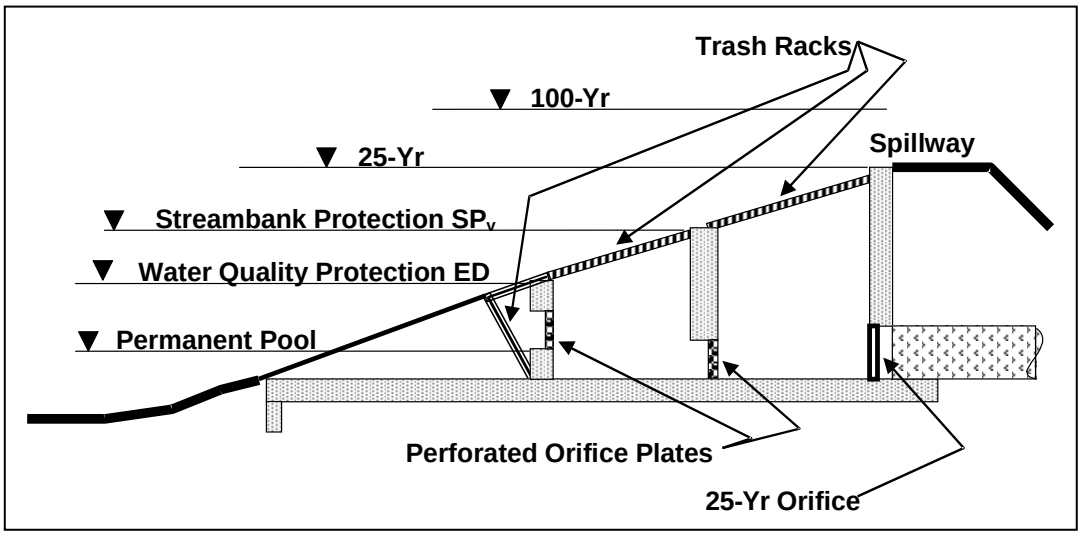


Figure 2.10 Schematic of Orifice Plate Outlet Structure

Perforated Risers

A special kind of orifice flow is a perforated riser as illustrated in Figure 2.9. In the perforated riser, an orifice plate at the bottom of the riser, or in the outlet pipe just downstream from the elbow at the bottom of the riser, controls the flow. It is important that the perforations in the riser convey more flow than the orifice plate so as not to become the control.

Referring to Figure 2.9, a shortcut formula has been developed to estimate the total flow capacity of the perforated section (McEnroe, 1988):

$$Q = C_p[(2A_p)/(3H_s)]*(\sqrt{2g})*H^{3/2} \qquad (2.11)$$

where:

- Q = discharge (cfs)
- C_p = discharge coefficient for perforations (normally 0.61)
- A_p = cross-sectional area of all the holes (ft^2)
- H_s = distance from S/2 below the lowest row of holes to S/2 above the top row (ft)

Pipes and Culverts

Discharge pipes are often used as outlet structures for stormwater control facilities. The design of these pipes can be for either single or multi-stage discharges. A reverse-slope underwater pipe is often used for water quality or streambank protection outlets.

Pipes smaller than 12 inches in diameter may be analyzed as a submerged orifice as long as H/D is greater than 1.5. Note: For low flow conditions when the flow reaches and begins to overflow the pipe, weir flow controls. As the stage increases the flow will transition to orifice flow.

Pipes greater than 12 inches in diameter should be analyzed as a discharge pipe with headwater and tailwater effects taken into account. The outlet hydraulics for pipe flow can be determined from the outlet

control culvert nomographs and procedures given in *Section 3.3*, or by using Equation 2.12 (NRCS, 1984).

The following equation is a general pipe flow equation derived through the use of the Bernoulli and continuity principles.

$$Q = a[(2gH) / (1 + k_m + k_pL)]0.5 \quad (2.12)$$

where:

- Q = discharge (cfs)
- a = pipe cross sectional area (ft^2)
- g = acceleration of gravity (ft/s^2)
- H = elevation head differential (ft)
- k_m = coefficient of minor losses (use 1.0)
- k_p = pipe friction coefficient = $5087n^2/D^{4/3}$
- L = pipe length (ft)

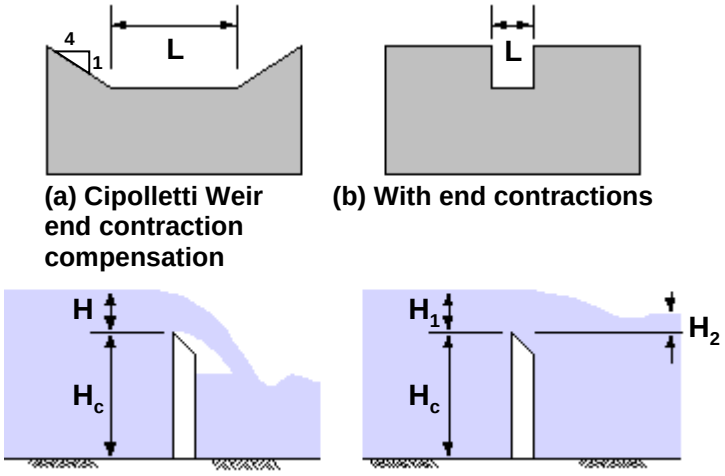
Sharp-Crested Weirs

If the overflow portion of a weir has a sharp, thin leading edge such that the water springs clear as it overflows, the overflow is termed a *sharp-crested* weir. If the sides of the weir also cause the through flow to contract, it is termed an *end-contracted* sharp-crested weir. Sharp-crested weirs have stable stage-discharge relations and are often used as a measurement device. A sharp-crested weir with compensation for end contractions is illustrated in Figure 2.11(a). The discharge equation for this configuration is (Chow, 1959) which can also be used for circular pipe risers:

$$Q = [(3.27 + 0.4(H/H_C)] LH^{1.5} \quad (2.13)$$

were:

- Q = discharge (cfs)
- H = head above weir crest excluding velocity head (ft)
- H_C = height of weir crest above channel bottom (ft)
- L = horizontal weir length (ft)



(a) Cipolletti Weir end contraction compensation **(b) With end contractions**

Figure 2.11 Sharp-Crested Weir

The discharge equation for the Cipolletti Weir is $Q = 3.367\ LH^{1/2}$

A sharp-crested weir with two end contractions is illustrated in Figure 2.11(b). The discharge equation for this configuration is (Chow, 1959) which can also be used for circular pipe risers:

$$\mathbf{Q = [(3.27 + 0.4(H/H_C)]\ (L - 0.2H)\ H^{1.5}} \qquad \mathbf{(2.14)}$$

where:

- Q = discharge (cfs)
- H = head above weir crest excluding velocity head (ft)
- H_C = height of weir crest above channel bottom (ft)
- L = horizontal weir length (ft)

A sharp-crested weir will be affected by submergence when the tailwater rises above the weir crest elevation. The result will be that the discharge over the weir will be reduced. The discharge equation for a sharp-crested submerged weir is (Brater and King, 1976):

$$\mathbf{Q_S = Q_f\ (1 - (H_2/H_1)^{1.5})^{0.385}} \qquad \mathbf{(2.15)}$$

where:

- Q_S = submergence flow (cfs)
- Q_f = free flow (cfs)
- H_1 = upstream head above crest (ft)
- H_2 = downstream head above crest (ft)

Broad-Crested Weirs

A weir in the form of a relatively long raised channel control crest section is a *broad-crested* weir. The flow control section can have different shapes, such as triangular or circular. True broad-crested weir flow occurs when upstream head above the crest is between the limits of about 1/20 and 1/2 the crest length in the direction of flow. For example, a thick wall or a flat stop log can act like a sharp-crested weir when the approach head is large enough that the flow springs from the upstream corner. If upstream head is small enough relative to the top profile length, the stop log can act like a broad-crested weir (USBR, 1997).

The equation for the broad-crested weir is (Brater and King, 1976):

$$\mathbf{Q = CLH^{1.5}} \qquad \mathbf{(2.16)}$$

where:

- Q = discharge (cfs)
- C = broad-crested weir coefficient
- L = broad-crested weir length perpendicular to flow (ft)
- H = head above weir crest (ft)

If the upstream edge of a broad-crested weir is so rounded as to prevent contraction and if the slope of the crest is as great as the loss of head due to friction, flow will pass through critical depth at the weir crest; this gives the maximum C value of 3.087. For sharp corners on the broad-crested weir, a minimum C value of 2.6 should be used. Information on C values as a function of weir crest breadth and head is given in Table 2.4.

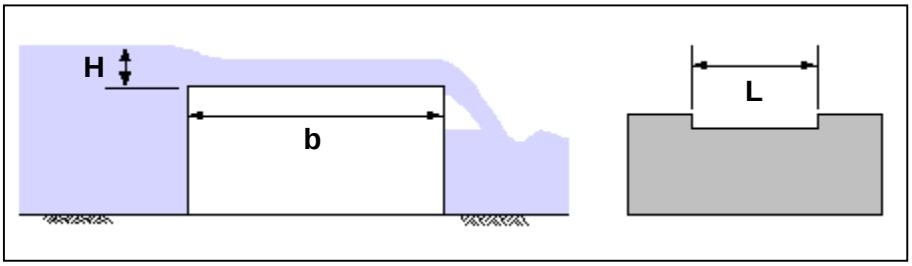


Figure 2.12 Broad-Crested Weir

Table 2.4 Broad-Crested Weir Coefficient (C) Values

Measured Head (H)*	Weir Crest Breadth (b) in feet										
In feet	0.50	0.75	1.00	1.50	2.00	2.50	3.00	4.00	5.00	10.00	15.00
0.2	2.80	2.75	2.69	2.62	2.54	2.48	2.44	2.38	2.34	2.49	2.68
0.4	2.92	2.80	2.72	2.64	2.61	2.60	2.58	2.54	2.50	2.56	2.70
0.6	3.08	2.89	2.75	2.64	2.61	2.60	2.68	2.69	2.70	2.70	2.70
0.8	3.30	3.04	2.85	2.68	2.60	2.60	2.67	2.68	2.68	2.69	2.64
1.0	3.32	3.14	2.98	2.75	2.66	2.64	2.65	2.67	2.68	2.68	2.63
1.2	3.32	3.20	3.08	2.86	2.70	2.65	2.64	2.67	2.66	2.69	2.64
1.4	3.32	3.26	3.20	2.92	2.77	2.68	2.64	2.65	2.65	2.67	2.64
1.6	3.32	3.29	3.28	3.07	2.89	2.75	2.68	2.66	2.65	2.64	2.63
1.8	3.32	3.32	3.31	3.07	2.88	2.74	2.68	2.66	2.65	2.64	2.63
2.0	3.32	3.31	3.30	3.03	2.85	2.76	2.72	2.68	2.65	2.64	2.63
2.5	3.32	3.32	3.31	3.28	3.07	2.89	2.81	2.72	2.67	2.64	2.63
3.0	3.32	3.32	3.32	3.32	3.20	3.05	2.92	2.73	2.66	2.64	2.63
3.5	3.32	3.32	3.32	3.32	3.32	3.19	2.97	2.76	2.68	2.64	2.63
4.0	3.32	3.32	3.32	3.32	3.32	3.32	3.07	2.79	2.70	2.64	2.63
4.5	3.32	3.32	3.32	3.32	3.32	3.32	3.32	2.88	2.74	2.64	2.63
5.0	3.32	3.32	3.32	3.32	3.32	3.32	3.32	3.07	2.79	2.64	2.63
5.5	3.32	3.32	3.32	3.32	3.32	3.32	3.32	3.32	2.88	2.64	2.63

* Measured at least 2.5H upstream of the weir.
Source: Brater and King (1976)

V-Notch Weirs

The discharge through a V-notch weir (Figure 2.13) can be calculated from the following equation (Brater and King, 1976).

$$Q = 2.5 \tan(\theta/2) H^{2.5} \quad (2.17)$$

where:

Q = discharge (cfs)

θ = angle of V-notch (degrees)

H = head on apex of notch (ft)

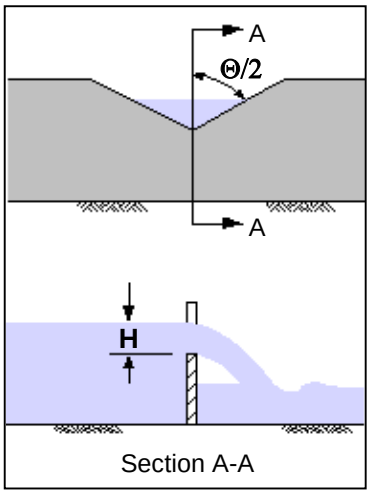


Figure 2.13 V-Notch Weir

Proportional Weirs

Although it may be more complex to design and construct, a proportional weir may significantly reduce the required storage volume for a given site. The proportional weir is distinguished from other control devices by having a linear head-discharge relationship achieved by allowing the discharge area to vary nonlinearly with head. A typical proportional weir is shown in Figure 2.14. Design equations for proportional weirs are (Sandvik, 1985):

$$Q = 4.97\, a^{0.5}\, b\, (H - a/3) \quad (2.18)$$

$$x/b = 1 - (1/3.17)(\arctan(y/a)^{0.5}) \quad (2.19)$$

where:

Q = discharge (cfs)

Dimensions a, b, H, x, and y are shown in Figure 2.14

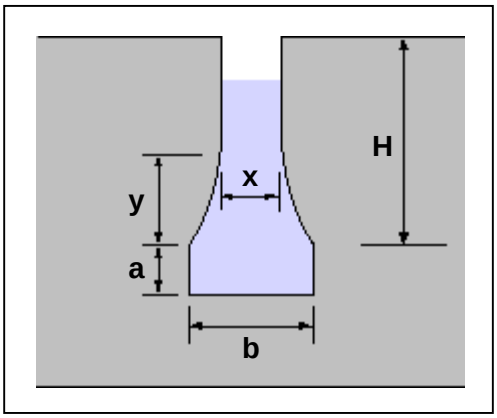


Figure 2.14 Proportional Weir Dimensions

Combination Outlets

Combinations of orifices, weirs, and pipes can be used to provide multi-stage outlet control for different control volumes within a storage facility (i.e., water quality protection volume, streambank protection volume, and flood control volume).

They are generally two types of combination outlets: shared outlet control structures and separate outlet controls. Shared outlet control is typically a number of individual outlet openings (orifices), weirs, or drops at different elevations on a riser pipe or box which all flow to a common larger conduit or pipe. Figure 2.15 shows an example of a riser designed for a wet extended detention pond. The orifice plate outlet structure in Figure 2.9 is another example of a combination outlet.

Separate outlet controls are less common and may consist of several pipe or culvert outlets at different levels in the storage facility that are either discharged separately or are combined to discharge at a single location.

The use of a combination outlet requires the construction of a composite stage-discharge curve (as shown in Figure 2.16) suitable for control of multiple storm flows. The design of multi-stage combination outlets is discussed later in this section.

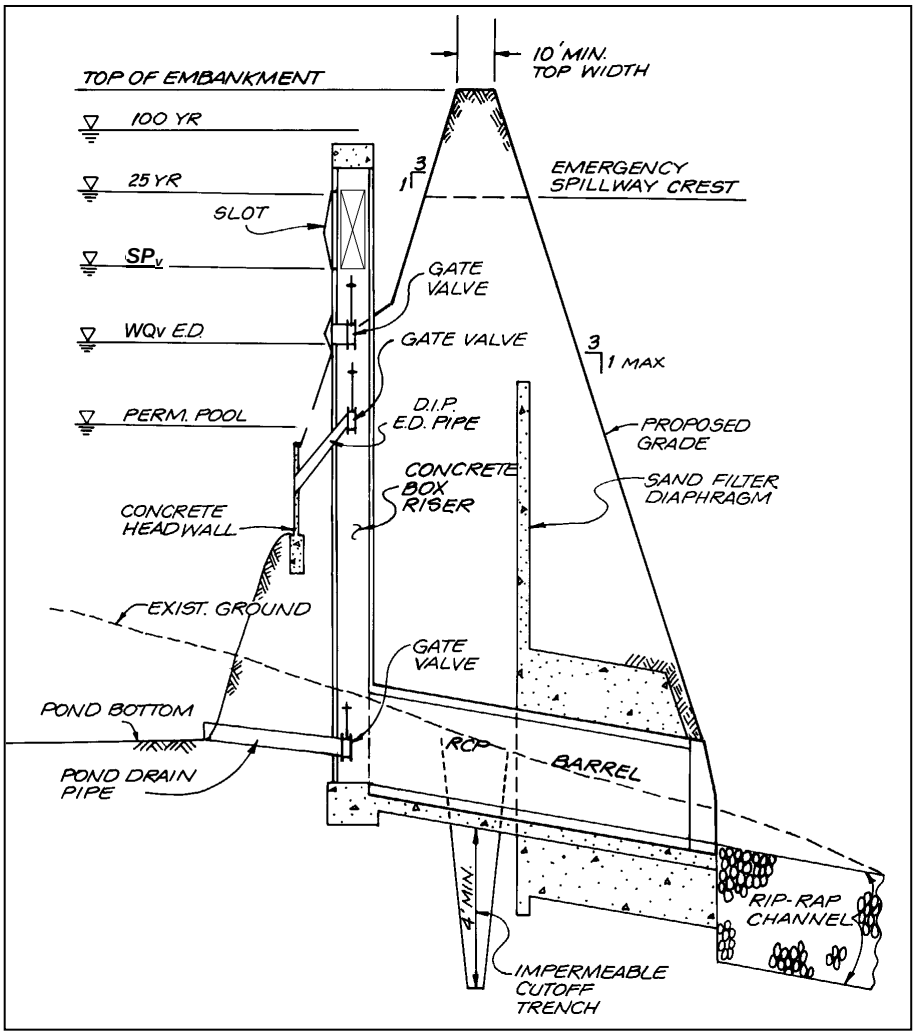


Figure 2.15 Schematic of Combination Outlet Structure

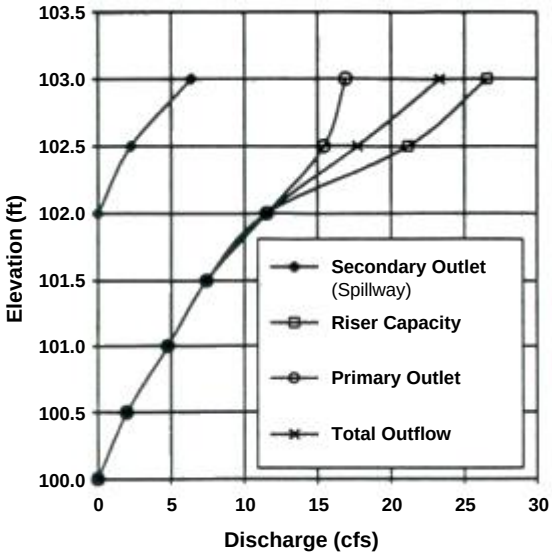


Figure 2.16 Composite Stage-Discharge Curve

2.2.3 Extended Detention (Water Quality and Streambank Protection) Outlet Design

Introduction

Extended detention (ED) orifice sizing is required in design applications that provide extended detention for downstream streambank protection or the ED portion of the water quality protection volume. The release rate for both the WQ_v and SP_v should be one that discharges the ED volume in a period of 24 hours or longer. In both cases an extended detention orifice or reverse slope pipe can be used for the outlet. For a structural control facility providing both WQ_v extended detention and SP_v control (wet ED pond, micropool ED pond, and shallow ED wetland), there will be a need to design two outlet orifices – one for the water quality control outlet and one for the streambank protection drawdown.

(The following procedures are based on the water quality outlet design procedures included in the Virginia Stormwater Management Handbook, 1999)

The outlet hydraulics for peak control design (flood control) is usually straightforward in that an outlet is selected to limit the peak flow to some predetermined maximum. Since volume and the time required for water to exit the storage facility are not usually considered, the outlet design can easily be calculated and routing procedures used to determine if quantity design criteria are met.

In an extended detention facility for water quality protection or downstream streambank protection, however, the storage volume is detained and released for each over a specified amount of time (e.g., 24-hours). The release period is a "brim" drawdown time, beginning at the time of peak storage of the WQ_v or SP_v until the entire calculated volume drains out of the basin. This assumes the brim volume is present in the basin prior to any discharge. In reality, however, water is flowing out of the basin prior to the full or brim volume being reached. Therefore, the extended detention outlet can be sized using either of the following methods:

- Using the maximum hydraulic head associated with the brim storage volume and maximum discharge, calculate the orifice size needed to achieve the required drawdown time. Route the volume through the basin to verify the actual storage volume used and the drawdown time.
- Approximate the orifice size using the average hydraulic head associated with the storage volume and the required drawdown time.

These two procedures are outlined in the examples below and can be used to size an extended detention orifice for water quality and/or streambank protection.

Method 1: Maximum Hydraulic Head with Routing

A wet ED pond sized for the required water quality protection volume will be used here to illustrate the sizing procedure for an extended-detention orifice.

Given the following information, calculate the required orifice size for water quality protection design.

Given: Water Quality Protection Volume (WQ_v) = 0.76 ac ft = 33,106 ft^3
Maximum Hydraulic Head (H_{max}) = 5.0 ft (from stage vs. storage data)

Step 1 Determine the maximum discharge resulting from the 24-hour drawdown requirement. It is calculated by dividing the Water Quality Protection Volume (or Streambank Protection Volume) by the required time to find the average discharge, and then multiplying by two to obtain the maximum discharge.

Q_{avg} = 33,106 ft^3 / (24 hr)(3,600 s/hr) = 0.38 cfs
$Q_{max} = 2 * Q_{avg} = 2 * 0.38 = 0.76$ cfs

Step 2 Determine the required orifice diameter by using the orifice Equation 2.9 and Q_{max} and H_{max}:

$Q = CA(2gH)^{0.5}$, or $A = Q / C(2gH)^{0.5}$

$A = 0.76 / 0.6[(2)(32.2)(5.0)]^{0.5} = 0.071\ ft^2$

Determine pipe diameter from $A = 3.14d^2/4$, then $d = (4A/3.14)^{0.5}$

$D = [4(0.071)/3.14]^{0.5} = 0.30$ ft = 3.61 in

Use a 3.6-inch diameter water quality protection orifice.

Routing the water quality protection volume of 0.76 ac ft through the 3.6-inch water quality protection orifice will allow the designer to verify the drawdown time, as well as the maximum hydraulic head elevation. The routing effect will result in the actual drawdown time being less than the calculated 24 hours. Judgment should be used to determine whether the orifice size should be reduced to achieve the required 24 hours.

Method 2: Average Hydraulic Head and Average Discharge

Using the data from the previous example use Method 2 to calculate the size of the outlet orifice.

Given: Water Quality Protection Volume (WQ_v) = 0.76 ac ft = 33,106 ft^3
Average Hydraulic Head (h_{avg}) = 2.5 ft (from stage vs storage data)

Step 1 Determine the average release rate to release the water quality protection volume over a 24-hour time period.

Q = 33,106 ft^3 / (24 hr)(3,600 s/hr) = 0.38 cfs

Step 2 Determine the required orifice diameter by using the orifice Equation 2.9 and the average head on the orifice:

$Q = CA(2gH)^{0.5}$, or $A = Q / C(2gH)^{0.5}$

$A = 0.38 / 0.6[(2)(32.2)(2.5)]^{0.5} = 0.05\ ft^3$

Determine pipe diameter from $A = 3.14r^2 = 3.14d^2/4$, then $d = (4A/3.14)^{0.5}$

$D = [4(0.05)/3.14]^{0.5} = 0.252$ ft = 3.03 in

Use a 3-inch diameter water quality protection orifice.

Use of Method 1, utilizing the maximum hydraulic head and discharge and routing, results in a 3.6-inch diameter orifice (though actual routing may result in a changed orifice size) and Method 2, utilizing average hydraulic head and average discharge, results in a 3.0-inch diameter orifice.

2.2.4 Multi-Stage Outlet Design

Introduction

A combination outlet such as a multiple orifice plate system or multi-stage riser is often used to provide adequate hydraulic outlet controls for the different design requirements (e.g., water quality protection, streambank protection, and flood control) for stormwater ponds, stormwater wetlands and detention-only facilities. Separate openings or devices at different elevations are used to control the rate of discharge from a facility during multiple design storms. Figures 2.9 and 2.15 are examples of multi-stage combination outlet systems.

A design engineer may be creative to provide the most economical and hydraulically efficient outlet design possible in designing a multi-stage outlet. Many iterative routings are usually required to arrive at a minimum structure size and storage volume that provides proper control. The stage-discharge table or rating curve is a composite of the different outlets that are used for different elevations within the multi-stage riser (see Figure 2.16)

Multi-Stage Outlet Design Procedure

Below are the steps for designing a multi-stage outlet. Note that if a structural control facility will not control one or more of the required storage volumes (WQ_v, SP_v, and Q_f), then that step in the procedure is skipped.

Step 1 Determine Stormwater Control Volumes. Using the procedures from *Section 1.0 of the Hydrology and Water Quality Technical Manuals*, estimate the required storage volumes for water quality protection (WQ_v), streambank protection (SP_v), and flood control (Q_f).

Step 2 Develop Stage-Storage Curve. Using the structure geometry and topography, develop the stage-storage curve for the facility in order to provide sufficient storage for the control volumes involved in the design.

Step 3 Design Water Quality Protection Outlet. Design the water quality protection extended detention (WQ_v-ED) orifice using either Method 1 or Method 2 outlined in *Section 2.2.3*. If a permanent pool is incorporated into the design of the facility, a portion of the storage volume for water quality protection will be above the elevation of the permanent pool. The outlet can be protected using either a reverse slope pipe, a hooded protection device, or another acceptable method (see *Section 2.2.5*).

Step 4 Design Streambank Protection Outlet. Design the streambank protection extended detention outlet (SP_v-ED) using either method from *Section 2.2.3*. For this design, the storage needed for streambank protection will be greater than the water quality protection volume storage elevation determined in Step 3. The total stage-discharge rating curve at this point will include the water quality protection orifice and the outlet used for streambank protection. The outlet should be protected in a manner similar to that for the water quality protection orifice.

Step 5 Design Flood Control Outlet. The storage needed for flood control will be greater than the water quality protection and streambank protection storage. Establish the Q_f maximum water surface elevation using the stage-storage curve and subtract the SP_v elevation to find the maximum head. Select an outlet type and calculate the initial size and geometry based upon maintaining the predevelopment peak discharge rate. Develop a stage-discharge curve for the combined set of outlets (WQ_v, SP_v and Q_f).

Step 6 Check Performance of the Outlet Structure. Perform a hydraulic analysis of the multi-stage outlet structure using reservoir routing to ensure that all outlets will function as designed. Several iterations may be required to calibrate and optimize the hydraulics and outlets that are used. Also, the structure should operate without excessive surging, noise, vibration, or vortex action at any stage. This usually requires that the outlet structure have a larger cross-sectional area than the outlet conduit.

The hydraulic analysis of the design must take into account the hydraulic changes that will occur as depth of storage changes for the different design storms. As shown in Figure 2.17, as the water passes over the rim of a riser, the riser acts as a weir. However, when the water surface reaches a certain height over the rim of a riser, the riser will begin to act as a submerged orifice. The designer must compute the elevation at which this transition from riser weir flow control to riser orifice flow control takes place for an outlet where this transition will occur. Also note in Figure 2.17 that as the elevation of the water increases further, the control can change from barrel inlet flow control to barrel pipe flow control. Figure 2.18 shows another condition where weir flow can change to orifice flow, which must be taken into account in the hydraulics of the rating curve as different design conditions results in changing water surface elevations.

Step 7 Size the Emergency Spillway. It is recommended that all stormwater impoundment structures have a vegetated emergency spillway (see *Section 2.2.7*). An emergency spillway provides a degree of safety to prevent overtopping of an embankment if the primary outlet or principal spillway should become clogged, or otherwise inoperative. The flood mitigation storm should be routed through the outlet devices and emergency spillway to ensure the hydraulics of the system will operate as designed. Also check the dam safety requirements to be sure of an adequate design.

Step 8 Design Outlet Protection. Design necessary outlet protection and energy dissipation facilities to avoid erosion problems downstream from outlet devices and emergency spillway(s). See *Section 4.0*, for more information.

Step 9 Perform Buoyancy Calculations. Perform buoyancy calculations for the outlet structure and footing. Flotation will occur when the weight of the structure is less than or equal to the buoyant force exerted by the water.

Step 10 Provide Seepage Control. Seepage control should be provided for the outflow pipe or culvert through an embankment. The two most common devices for controlling seepage are (1) filter and drainage diaphragms and (2) anti-seep collars.

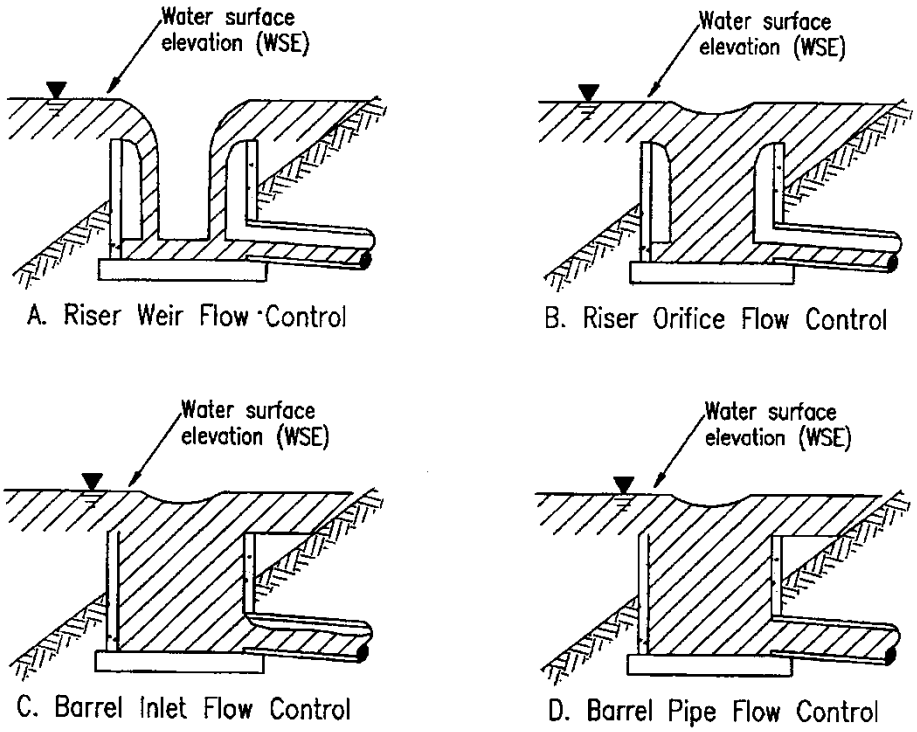


Figure 2.17 Riser Flow Diagrams
(Source: VDCR, 1999)

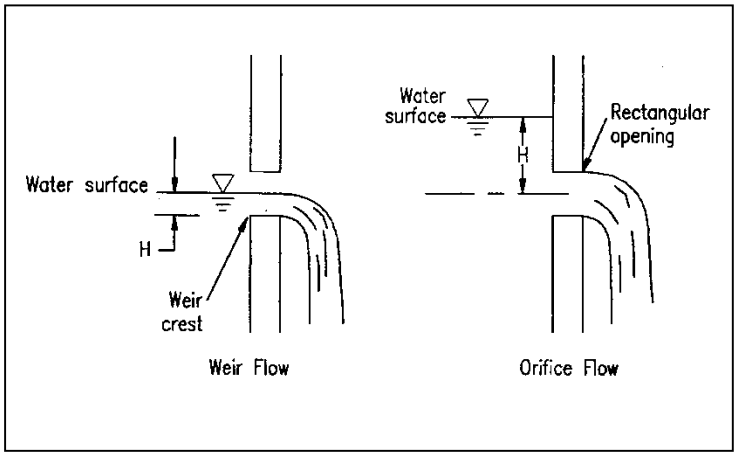


Figure 2.18 Weir and Orifice Flow
(Source: VDCR, 1999)

2.2.5 Extended Detention Outlet Protection

Small low flow orifices such as those used for extended detention applications can easily clog, preventing the structural control from meeting its design purpose(s) and potentially causing adverse impacts. Therefore, extended detention orifices need to be adequately protected from clogging. There are a number of different anti-clogging designs, including:

The use of a reverse slope pipe attached to a riser for a stormwater pond or wetland with a permanent pool (see Figure 2.19). The inlet is submerged a minimum of 1 foot below the elevation of the permanent pool to prevent floatables from clogging the pipe and to avoid discharging warmer water at the surface of the pond.

The use of a hooded outlet for a stormwater pond or wetland with a permanent pool is shown in Figures 2.20 and 2.21.

Internal orifice protection through the use of an over-perforated vertical stand pipe with ½-inch orifices or slots that are protected by wirecloth and a stone filtering jacket (see Figure 2.22).

Internal orifice size requirements may be attained by the use of adjustable gate valves to achieve an equivalent orifice diameter.

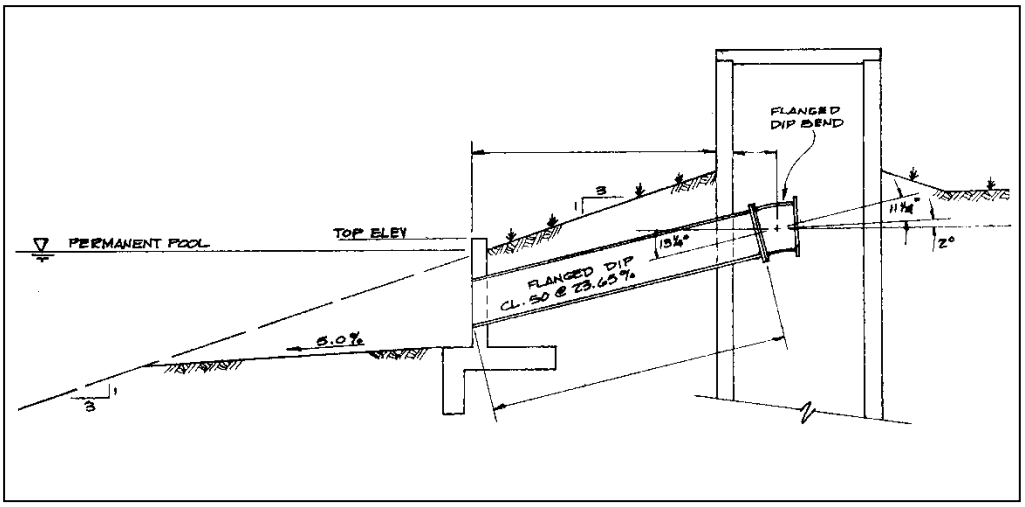


Figure 2.19 Reverse Slope Pipe Outlet

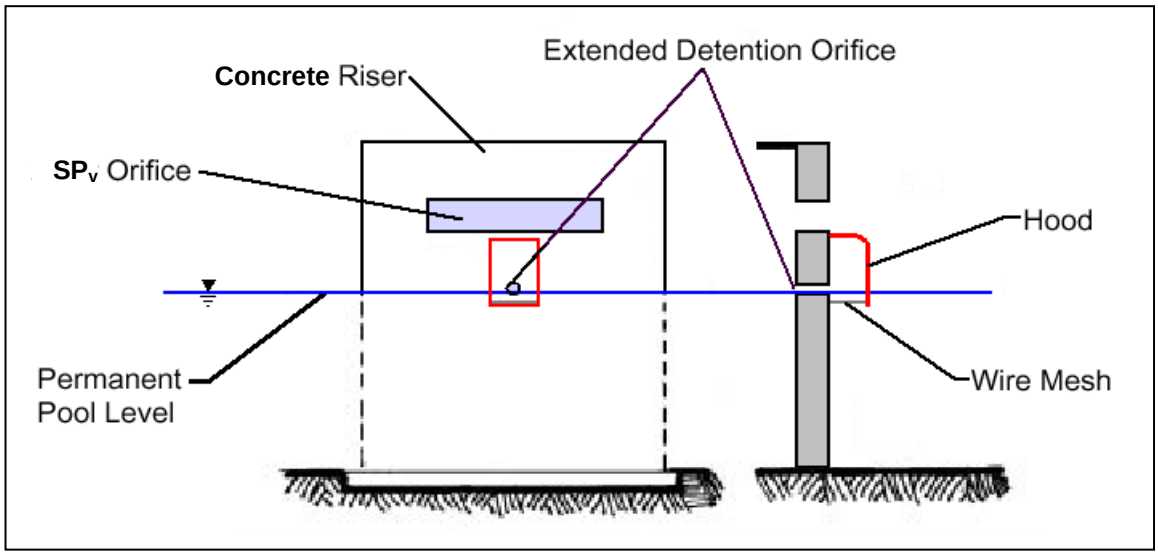


Figure 2.20 Hooded Outlet

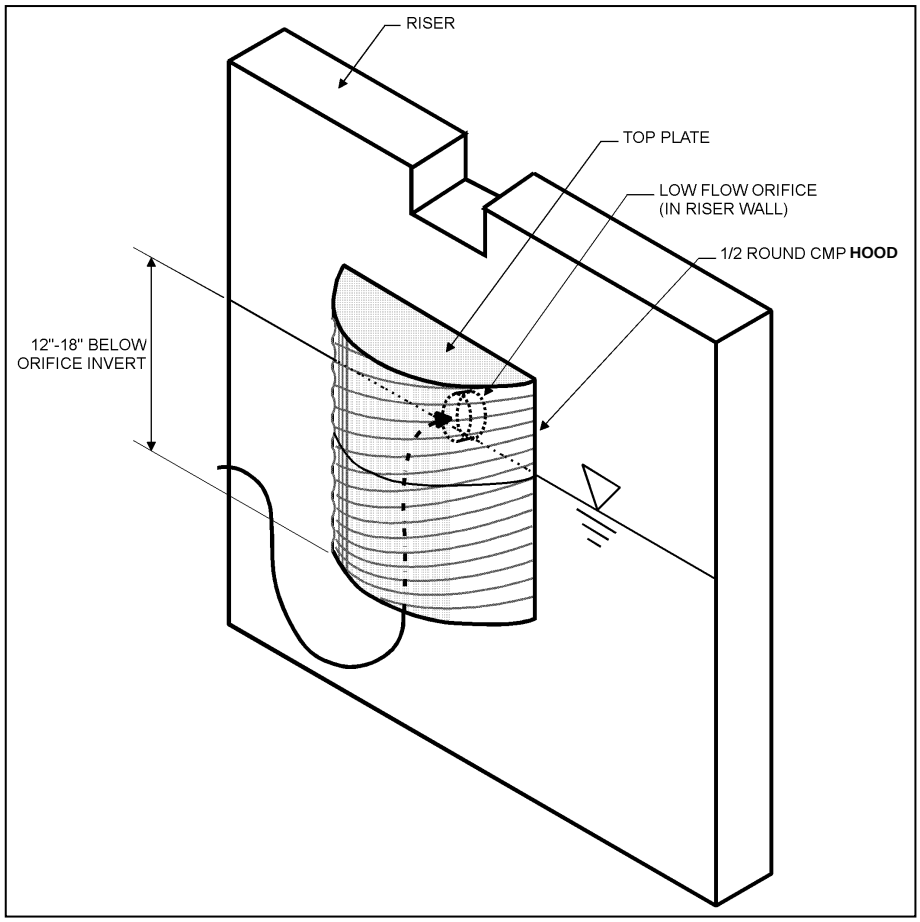


Figure 2.21 Half-Round CMP Orifice Hood

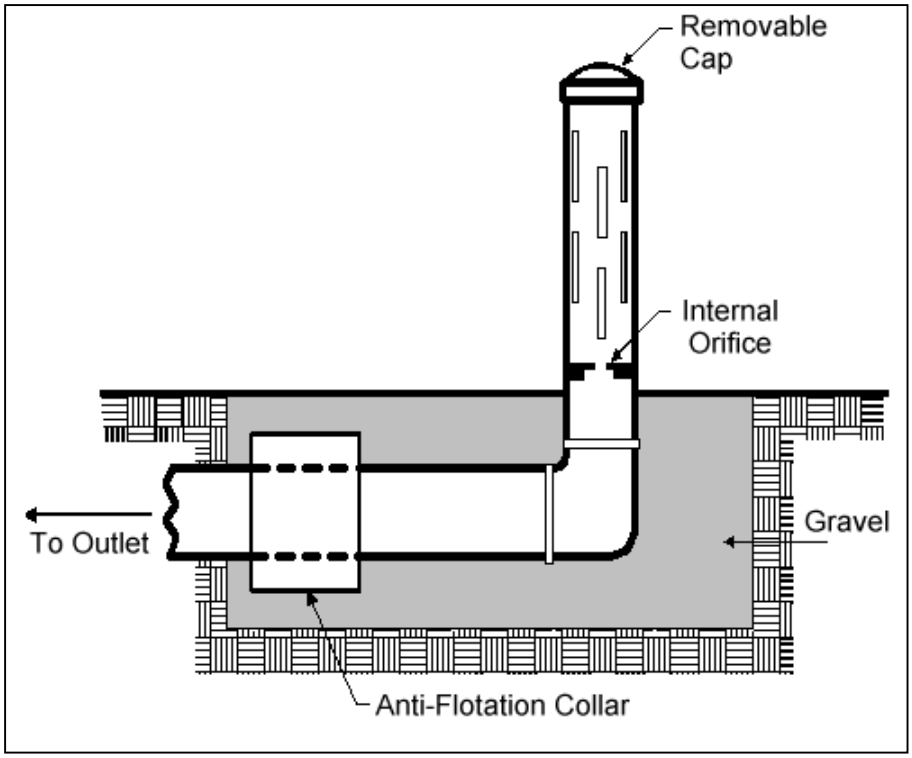


Figure 2.22 Internal Control for Orifice Protection

2.2.6 Trash Racks and Safety Grates

Introduction

The susceptibility of larger inlets to clogging by debris and trash needs to be considered when estimating their hydraulic capacities. In most instances trash racks will be needed. Trash racks and safety grates are a critical element of outlet structure design and serve several important functions:

- Keeping debris away from the entrance to the outlet works where they will not clog the critical portions of the structure
- Capturing debris in such a way that relatively easy removal is possible
- Ensuring that people and large animals are kept out of confined conveyance and outlet areas
- Providing a safety system that prevents anyone from being drawn into the outlet and allows them to climb to safety

When designed properly, trash racks serve these purposes without interfering significantly with the hydraulic capacity of the outlet (or inlet in the case of conveyance structures) (ASCE, 1985; Allred-Coonrod, 1991). The location and size of the trash rack depends on a number of factors, including head losses through the rack, structural convenience, safety and size of outlet. Well-designed trash racks can also have an aesthetically pleasing appearance.

An example of trash racks used on a riser outlet structure is shown in Figure 2.23. The inclined vertical bar rack is most effective for lower stage outlets. Debris will ride up the trash rack as water levels rise. This design also allows for removal of accumulated debris with a rake while standing on top of the structure.

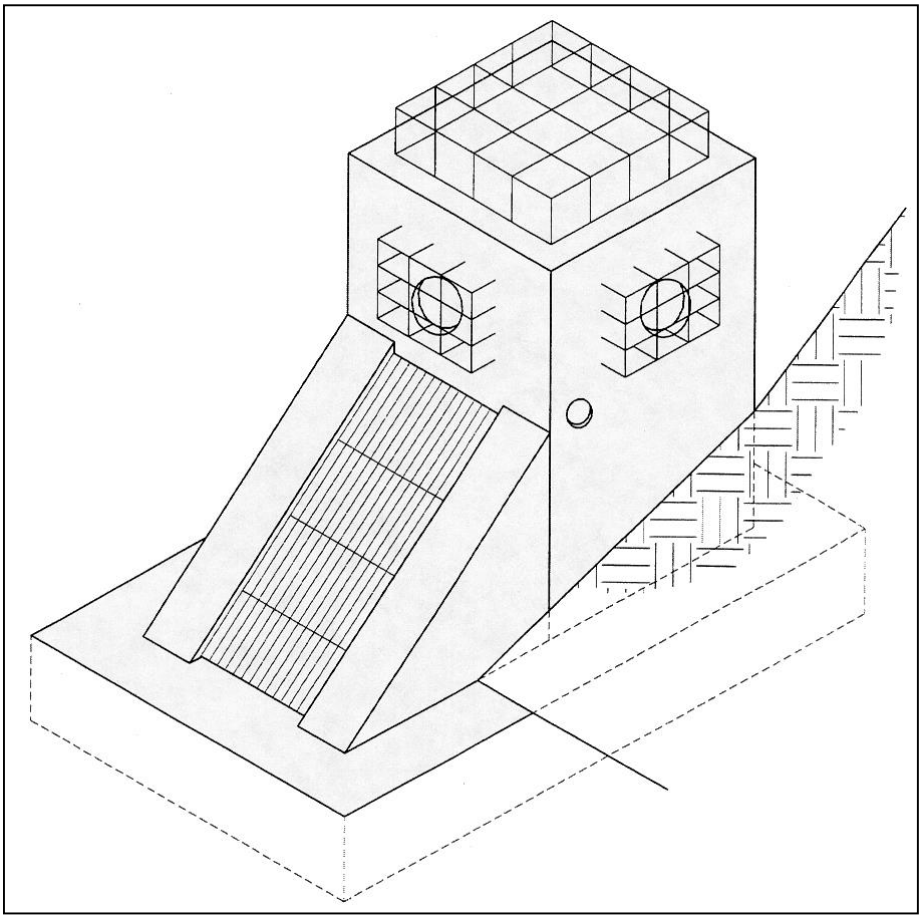


Figure 2.23 Example of Various Trash Racks Used on a Riser Outlet Structure
(Source: VDCR, 1999)

Trash Rack Design

Trash racks must be large enough so that partial plugging will not adversely restrict flows reaching the control outlet. There are no universal guidelines for the design of trash racks to protect detention basin outlets, although a commonly used "rule-of-thumb" is to have the trash rack area at least ten times larger than the control outlet orifice.

The surface area of all trash racks should be maximized and the trash racks should be located a suitable distance from the protected outlet to avoid interference with the hydraulic capacity of the outlet. The spacing of trash rack bars must be proportioned to the size of the smallest outlet protected. However, where a small orifice is provided, a separate trash rack for that outlet should be used, so that a simpler, sturdier trash rack with more widely spaced members can be used for the other outlets. Spacing of the rack bars should be wide enough to avoid interference, but close enough to provide the level of clogging protection required.

To facilitate removal of accumulated debris and sediment from around the outlet structure, the racks should have hinged connections. If the rack is bolted or set in concrete it will preclude removal of accumulated material and will eventually adversely affect the outlet hydraulics.

Since sediment will tend to accumulate around the lowest stage outlet, the inside of the outlet structure for a dry basin should be depressed below the ground level to minimize clogging due to sedimentation.

Depressing the outlet bottom to a depth below the ground surface at least equal to the diameter of the outlet is recommended.

Trash racks at entrances to pipes and conduits should be sloped at about 3H:1V to 5H:1V to allow trash to slide up the rack with flow pressure and rising water level — the slower the approach flow, the flatter the angle. Rack opening rules-of-thumb are found in literature. Figure 2.24 gives opening estimates based on outlet diameter (UDFCD, 1992). Judgment should be used in that an area with higher debris (e.g., a wooded area) may require more opening space.

The bar opening space for small pipes should be less than the pipe diameter. For larger diameter pipes, openings should be 6 inches or less. Collapsible racks have been used in some places if clogging becomes excessive or a person becomes pinned to the rack.

Alternately, debris for culvert openings can be caught upstream from the opening by using pipes placed in the ground or a chain safety net (USBR, 1978; UDFCD, 1999). Racks can be hinged on top to allow for easy opening and cleaning.

The control for the outlet should not shift to the grate, nor should the grate cause the headwater to rise above planned levels. Therefore head losses through the grate should be calculated. A number of empirical loss equations exist though many have difficult to estimate variables. Two will be given to allow for comparison.

Metcalf & Eddy (1972) give the following equation (based on German experiments) for losses. Grate openings should be calculated assuming a certain percentage blockage as a worst case to determine losses and upstream head. Often 40 to 50% is chosen as a working assumption.

$$\mathbf{H_g = K_{g1} (w/x)^{4/3} (V_u^2/2g) \sin \theta_g} \qquad \mathbf{(2.20)}$$

Where:

H_g = head loss through grate (ft)

K_{g1} = bar shape factor:
- 2.42 - sharp edged rectangular
- 1.83 - rectangular bars with semicircular upstream faces
- 1.79 - circular bars
- 1.67 - rectangular bars with semicircular up- and downstream faces

w = maximum cross-sectional bar width facing the flow (in)

x = minimum clear spacing between bars (in)

V_u = approach velocity (ft/s)

g = acceleration due to gravity (32.2 ft/s^2)

θ_g = angle of the grate with respect to the horizontal (degrees)

The Corps of Engineers (HDC, 1988) has developed curves for trash racks based on similar and additional tests. These curves are for vertical racks but presumably they can be adjusted, in a manner similar to the previous equation, through multiplication by the sine of the angle of the grate with respect to the horizontal.

$$\mathbf{H_g = \frac{K_{g2} V_u^2}{2g}} \qquad \mathbf{(2.21)}$$

Where:

K_{g2} is defined from a series of fit curves as:

- sharp edged rectangular (length/thickness = 10)
  $K_{g2} = 0.00158 - 0.03217\ A_r + 7.1786\ A_r^2$

- sharp edged rectangular (length/thickness = 5)
  $K_{g2} = -0.00731 + 0.69453\ A_r + 7.0856\ A_r^2$
- round edged rectangular (length/thickness = 10.9)
  $K_{g2} = -0.00101 + 0.02520\ A_r + 6.0000\ A_r^2$
- circular cross section
  $K_{g2} = 0.00866 + 0.13589\ A_r + 6.0357\ A_r^2$

and A_r is the ratio of the area of the bars to the area of the grate section.

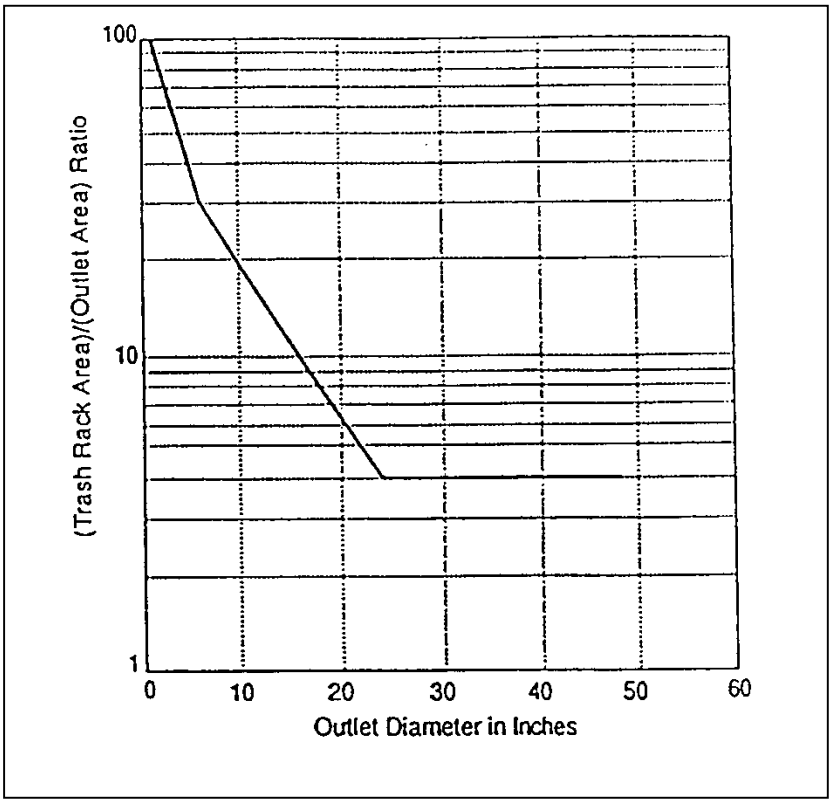


Figure 2.24 Minimum Rack Size vs. Outlet Diameter
(Source: UDCFD, 1992)

2.2.7 Secondary Outlets

Introduction

The purpose of a secondary outlet (emergency spillway) is to provide a controlled overflow for flows in excess of the maximum design storm for a storage facility. Figure 2.25 shows an example of an emergency spillway.

In many cases, on-site stormwater storage facilities do not warrant elaborate studies to determine spillway capacity. While the risk of damage due to failure is a real one, it normally does not approach the catastrophic risk involved in the overtopping or breaching of a major reservoir. By contrast, regional facilities with homes immediately downstream could pose a significant hazard if failure were to occur, in which case emergency spillway considerations are a major design factor.

Emergency Spillway Design

Emergency spillway designs are open channels, usually trapezoidal in cross section, and consist of an inlet channel, a control section, and an exit channel (see Figure 2.25). The emergency spillway is

proportioned to pass flows in excess of the design flood (typically the flood mitigation storm or greater) without allowing excessive velocities and without overtopping of the embankment. Any dam, six feet or higher, must meet appropriate state and federal design standards, especially those regarding spillway design requirements related to passage of the probable maximum flood. In any case, the flood mitigation storm discharge, assuming blockage of outlet works, must be conveyed with some freeboard as specified by local criteria. Flow in the emergency spillway is open channel flow (see *Section 3.2*, for more information). Normally, it is assumed that critical depth occurs at the control section.

NRCS (SCS) manuals provide guidance for the selection of emergency spillway characteristics for different soil conditions and different types of vegetation. The selection of degree of retardance for a given spillway depends on the vegetation. Knowing the retardance factor and the estimated discharge rate, the emergency spillway bottom width can be determined. For erosion protection during the first year, assume minimum retardance. Both the inlet and exit channels should have a straight alignment and grade. Spillway side slopes should be no steeper the 3:1 horizontal to vertical.

The most common type of emergency spillway used is a broad-crested overflow weir cut through original ground next to the embankment. The transverse cross section of the weir cut is typically trapezoidal in shape for ease of construction. Such an excavated emergency spillway is illustrated below.

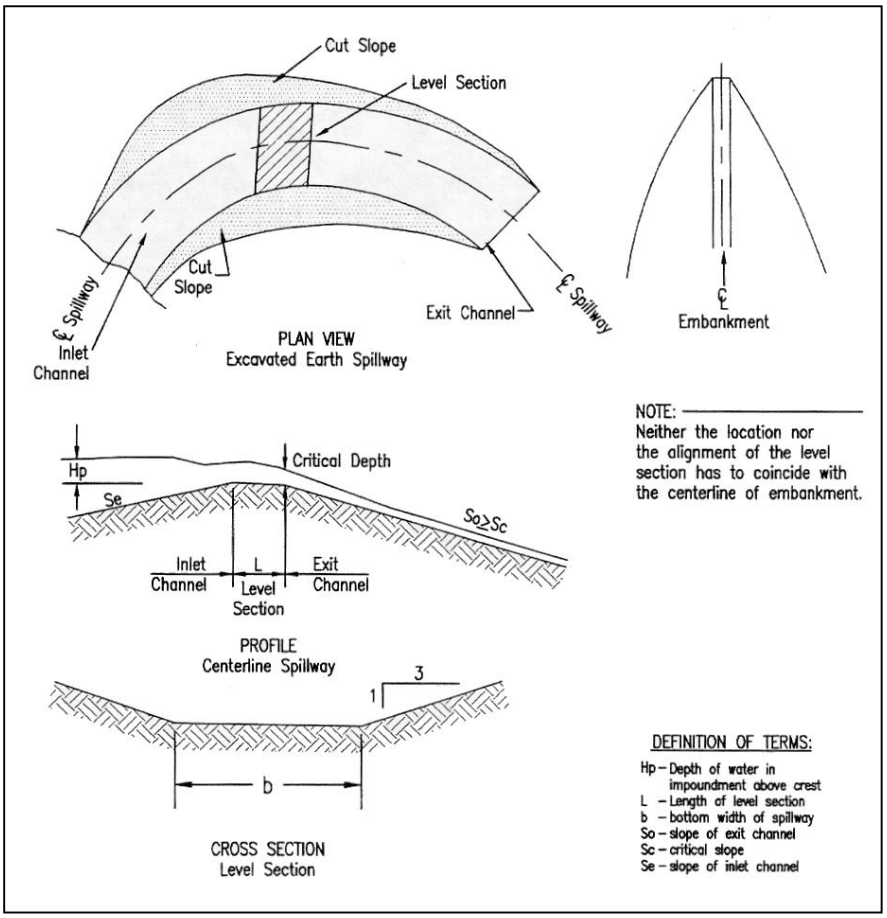


Figure 2.25 Emergency Spillway
(Source: VDCR, 1999)

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3.0 Open Channels, Culverts and Bridges

3.1 Open Channels, Culverts and Bridges Overview

3.1.1 Key Issues in Stormwater System Design

Introduction

The traditional design of stormwater systems has been to collect and convey stormwater runoff as rapidly as possible to a suitable location where it can be discharged. This manual takes a different approach wherein the design methodologies and concepts of drainage design are to be integrated with the objectives for water quantity and quality control. This means:

- Stormwater systems are to remove water efficiently enough to meet flood protection criteria and level of service requirements, and
- These systems are to complement the ability of the site design and structural stormwater controls to mitigate the major stormwater impacts of urban development.

The following are some of the key issues in integrating water quantity and quality control consideration in stormwater system design.

General Design Considerations

- Stormwater systems should be planned and designed so as to generally conform to natural drainage patterns and discharge to natural drainage pathways within a drainage basin. These natural drainage pathways should only be modified as a last resort to contain and safely convey the peak flows generated by the development.
- Runoff must be discharged in a manner that will not cause adverse impacts on downstream properties or stormwater systems. In general, runoff from development sites within a drainage basin should be discharged at the existing natural drainage outlet or outlets. If the developer wishes to change discharge points he or she must demonstrate that the change will not have any adverse impacts on downstream properties or stormwater (minor) systems.
- It is important to ensure that the combined on-site flood control system and major stormwater system can handle blockages and flows in excess of the design capacity to minimize the likelihood of nuisance flooding or damage to private properties. If failure of minor stormwater systems and/or major stormwater structures occurs during these periods, the risk to life and property could be significantly increased.
- In establishing the layout of stormwater systems, it is essential to ensure that flows are not diverted onto private property during flows up to the major stormwater system design capacity.

Open Channels

- Open channels provide opportunities for reduction of flow peaks and pollution loads. They may be designed as wet or dry enhanced swales or grass channels.
- Channels can be designed with natural meanders improving both aesthetics and pollution removal through increased contact time.
- Grass channels generally provide better habitat than hardened channel sections, though studies have shown that riprap interstices provide significant habitat as well. Velocities should be carefully checked at design flows and the outer banks at bends should be specifically designed for increased shear stress and super elevation.

- Compound sections can be developed to carry the annual flow in the lower section and higher flows above them. Figure 3.1 illustrates a compound section that carries the 2-year and 100-year flows within banks. This reduces channel erosion at lower flows, and meandering, self-forming low flow channels that attack banks. The shelf in the compound section should have a minimum 1:12 slope to ensure drainage.

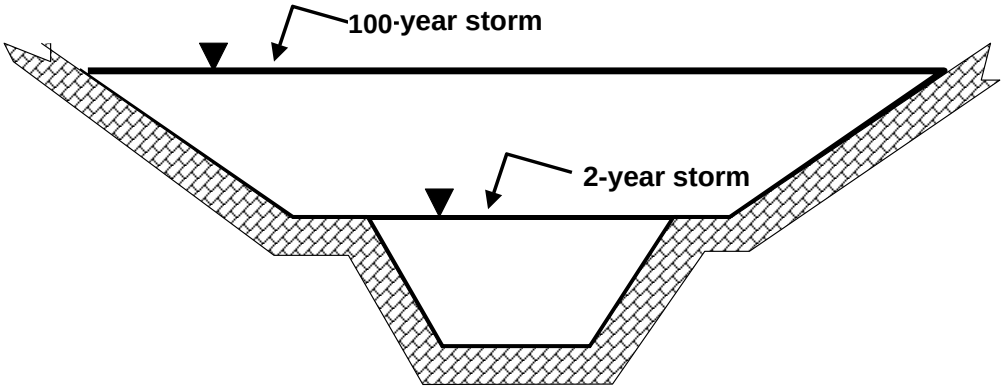


Figure 3.1 Compound Channel

- Flow control structures can be placed in the channels to increase residence time. Higher flows should be calculated using a channel slope from the top of the cross piece to the next one if it is significantly different from the channel bottom for normal depth calculations. Channel slope stability can also be ensured through the use of grade control structures that can serve as pollution reduction enhancements if they are set above the channel bottom. Regular maintenance is necessary to remove sediment and keep the channels from aggrading and losing capacity for larger flows.

Culverts

- Culverts can serve double duty as flow retarding structures in grass channel design. Care should be taken to design them as storage control structures if depths exceed several feet, and to ensure safety during flows.
- Improved entrance designs can absorb considerable slope and energy for steeper sloped designs, thus helping to protect channels.

Bridges

- Bridges enable streams to maintain flow conveyance.
- Bridges are usually designed so that they are not submerged.
- Bridges may be vulnerable to failure from flood-related causes.
- Flow velocities through bridge openings should not cause scour within the bridge opening or in the stream reaches adjacent to the bridge.

Storage Design

- Stormwater storage within a stormwater system is essential to providing the extended detention of flows for water quality treatment and downstream streambank protection, as well as for peak flow attenuation of larger flows for flood protection.
- Runoff storage can be provided within an on-site flood control system through the use of structural stormwater controls and/or nonstructural features.
- Stormwater storage can be provided by detention, extended detention, or retention.

- Storage facilities may be provided on-site, or as regional facilities designed to manage stormwater runoff from multiple projects.

Outlet Structures

- Outlet structures provide the critical function of the regulation of flow for structural stormwater controls.
- Outlet structures may consist of a single stage outlet structure, or several outlet structures combined to provide multi-stage outlet control.
- Smaller, more protected outlet structures should be used for water quality and streambank protection flows.
- Large flows, such as flood flows, are typically handled through a broad crested weir, a riser with different sized openings, a drop inlet structure, or a spillway through an embankment.

Energy Dissipators

- Energy dissipators should be designed to return flows to non-eroding velocities to protect downstream channels.
- Care must be taken during construction that design criteria are followed exactly. The designs presented in this Manual have been carefully developed through model and full-scale tests. Each part of the criteria is important to their proper function.

3.2 Open Channel Design

3.2.1 Overview

Introduction

Open channel systems and their design are an integral part of stormwater drainage design, particularly for development sites utilizing better site design practices and open channel structural controls. Open channels include drainage ditches, grass channels, dry and wet enhanced swales, stone riprap channels and concrete-lined channels.

The purpose of this section is to provide an overview of open channel design criteria and methods, including the use of channel design nomographs.

Open Channel Types

The three main classifications of open channel types according to channel linings are vegetated, flexible, and rigid. Vegetated linings include grass with mulch, sod and lapped sod, and wetland channels. Stone riprap and some forms of flexible man-made linings or gabions are examples of flexible linings, while rigid linings are generally concrete or rigid block.

Vegetative Linings – Vegetation, where practical, is the most desirable lining for an artificial channel. It stabilizes the channel body, consolidates the soil mass of the bed, checks erosion on the channel surface, provides habitat, and provides water quality benefits (see *Section 3.6.3 of the Criteria Manual* and the *Site Development Controls Technical Manual* for more details on using enhanced swales and grass channels for water quality purposes).

Conditions under which vegetation may not be acceptable include but are not limited to:

- High velocities
- Standing or continuously flowing water
- Lack of regular maintenance necessary to prevent growth of taller or woody vegetation
- Lack of nutrients and inadequate topsoil
- Excessive shade

Proper seeding, mulching, and soil preparation are required during construction to assure establishment of healthy vegetation.

If low flows are prevalent, a hard lined pilot channel may be needed, and it should be wide enough to accommodate maintenance equipment.

Flexible Linings – Rock riprap, including rubble and gabion baskets, is the most common type of flexible lining for channels. It presents a rough surface that can dissipate energy and mitigate increases in erosive velocity. These linings are usually less expensive than rigid linings and have self-healing qualities that reduce maintenance. However, they may require the use of a filter fabric depending on the underlying soils, and the growth of grass, weeds, and trees may present maintenance problems.

Rigid Linings – Rigid linings are generally constructed of concrete and used where high flow capacity is required. Higher velocities, however, create the potential for scour at channel lining transitions and channel headcutting.

3.2.2 Symbols and Definitions

To provide consistency within this section as well as throughout this Manual, the symbols listed in Table 3.1 will be used. These symbols were selected because of their wide use. In some cases, the same

symbol is used in existing publications for more than one definition. Where this occurs in this section, the symbol will be defined where it occurs in the text or equations.

Table 3.1 Symbols and Definitions

Symbol	Definition	Units
α	Energy coefficient	-
A	Cross-sectional area	ft^2
b	Bottom width	ft
C_g	Specific weight correction factor	-
D or d	Depth of flow	ft
d	Stone diameter	ft
delta d	Super-elevation of the water surface profile	ft
d_x	Diameter of stone for which x percent, by weight, of the gradation is finer	ft
E	Specific energy	ft
Fr	Froude Number	-
g	Acceleration of gravity	32.2 ft/s^2
h_{loss}	Head loss	ft
K	Channel conveyance	-
k_e	Eddy head loss coefficient	ft
K_T	Trapezoidal open channel conveyance factor	-
L	Length of channel	ft
L_p	Length of downstream protection	ft
n	Manning's roughness coefficient	-
P	Wetted perimeter	ft
Q	Discharge rate	cfs
R	Hydraulic radius of flow	ft
R_c	Mean radius of the bend	ft
S	Slope	ft/ft
SW_s	Specific weight of stone	lbs/ft^3
T	Top width of water surface	ft
V or v	Velocity of flow	ft/s
w	Stone weight	lbs
y_c	Critical depth	ft
y_n	Normal depth	ft
z	Critical flow section factor	-

3.2.3 Manning's n Values

The Manning's n value is an important variable in open channel flow computations. Variation in this variable can significantly affect discharge, depth, and velocity estimates. Since Manning's n values depend on many different physical characteristics of natural and man-made channels, care and good engineering judgment must be exercised in the selection process.

Recommended Manning's n values for artificial channels with rigid, unlined, temporary, and stone riprap linings are given in Table 3.2. Recommended values for vegetative linings should be determined using Figure 3.2, which provides a graphical relationship between Manning's n values and the product of velocity and hydraulic radius for several vegetative retardance classifications (see Table 3.6). Figure 3.2 is used iteratively as described in *Section 3.2.5*. Recommended Manning's values for natural channels that are either excavated or dredged, and natural are given in Table 3.2. For natural channels, Manning's n values should be estimated using experienced judgment and information presented in publications such as the *Guide for Selecting Manning's Roughness Coefficients for Natural Channels and Flood Plains*, FHWA-TS-84-204, 1984, FHWA HEC-15, 1988, or Chow, 1959. When designing open channels, the usual choice of Manning's roughness coefficients may be found in Table 3.5. The local jurisdiction may choose to vary from these values.

Table 3.2 Roughness Coefficients (Manning's n) and Allowable Velocities for Natural Channels

Channel Description	Manning's n	Maximum Permissible Channel Velocity (ft/s)
MINOR NATURAL STREAMS		
Fairly regular section		
1. Some grass and weeds; little or no brush	0.030	3 to 6
2. Dense growth of weeds, depth of flow materially greater than weed height	0.035	3 to 6
3. Some weeds, light brush on banks	0.035	3 to 6
4. Some weeds, heavy brush on banks	0.050	3 to 6
5. Some weeds, dense willows on banks	0.060	3 to 6
For trees within channels with branches submerged at high stage, increase above values by	0.010	
Irregular section with pools, slight channel meander, increase above values by	0.010	
Floodplain – Pasture		
1. Short grass	0.030	3 to 6
2. Tall grass	0.035	3 to 6
Floodplain – Cultivated Areas		
1. No crop	0.030	3 to 6
2. Mature row crops	0.035	3 to 6
3. Mature field crops	0.040	3 to 6
Floodplain – Uncleared		
1. Heavy weeds scattered brush	0.050	3 to 6
2. Wooded	0.120	3 to 6

Table 3.2 Roughness Coefficients (Manning's n) and Allowable Velocities for Natural Channels

Channel Description	Manning's n	Maximum Permissible Channel Velocity (ft/s)
MAJOR NATURAL STREAMS		
Roughness coefficient is usually less than for minor streams of similar description on account of less effective resistance offered by irregular banks or vegetation on banks. Values of "n" for larger streams of mostly regular sections, with no boulders or brush	Range from 0.028 to 0.060	3 to 6
UNLINED VEGETATED CHANNELS		
Clays (Bermuda Grass)	0.035	5 to 6
Sandy and Silty Soils (Bermuda Grass)	0.035	3 to 5
UNLINED NON-VEGETATED CHANNELS		
Sandy Soils	0.030	1.5 to 2.5
Silts	0.030	0.7 to 1.5
Sandy Silts	0.030	2.5 to 3.0
Clays	0.030	3.0 to 5.0
Coarse Gravels	0.030	5.0 to 6.0
Shale	0.030	6.0 to 10.0
Rock	0.025	15

Table 3.3 Maximum Velocities for Vegetative Channel Linings

Vegetation Type	Slope Range (%)[1]	Maximum Velocity[2] (ft/s)
Bermuda grass	0-5	6
Bahia		4
Tall fescue grass mixtures[3]	0-10	4
Kentucky bluegrass	0-5	6
Buffalo grass	5-10	5
	>10	4
Grass mixture	0-5[1]	4
	5-10	3
Sericea lespedeza, Weeping lovegrass, Alfalfa	0-5[4]	3
Annuals[5]	0-5	3
Sod		4
Lapped sod		5

[1] Do not use on slopes steeper than 10% except for side-slope in combination channel.
[2] Use velocities exceeding 5 ft/s only where good stands can be maintained.
[3] Mixtures of Tall Fescue, Bahia, and/or Bermuda
[4] Do not use on slopes steeper than 5% except for side-slope in combination channel.
[5] Annuals - used on mild slopes or as temporary protection until permanent covers are established.

Source: Manual for Erosion and Sediment Control in Georgia, 1996

Table 3.4 Manning's Roughness Coefficients (n) for Artificial Channels

		Depth Ranges		
Category	**Lining Type**	**0-0.5 ft**	**0.5-2.0 ft**	**>2.0 ft**
Rigid	Concrete	0.015	0.013	0.013
	Grouted Riprap	0.040	0.030	0.028
	Stone Masonry	0.042	0.032	0.030
	Soil 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
Temporary*	Woven Paper Net	0.016	0.015	0.015
	Jute Net	0.028	0.022	0.019
	Fiberglass Roving	0.028	0.022	0.019
	Straw with Net	0.065	0.033	0.025
	Curled Wood Mat	0.066	0.035	0.028
	Synthetic Mat	0.036	0.025	0.021
Gravel Riprap	1-inch D_{50}	0.044	0.033	0.030
	2-inch D_{50}	0.066	0.041	0.034
Rock Riprap	6-inch D_{50}	0.104	0.069	0.035
	12-inch D_{50}	–	0.078	0.040

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

*Some "temporary" linings become permanent when buried.

Source: HEC-15, 1988.

Table 3.5 Manning's Roughness Coefficients for Design

Lining Type	Manning's n	Comments
Grass Lined	0.035	Use for velocity check.
	0.050	Use for channel capacity check (freeboard check)
Concrete Lined	0.015	
Gabions	0.030	
Rock Riprap	0.040	$n = 0.0395 d_{50}^{1/6}$ where d_{50} is the stone size of which 50% of the sample is smaller
Grouted Riprap	0.028	FWHA

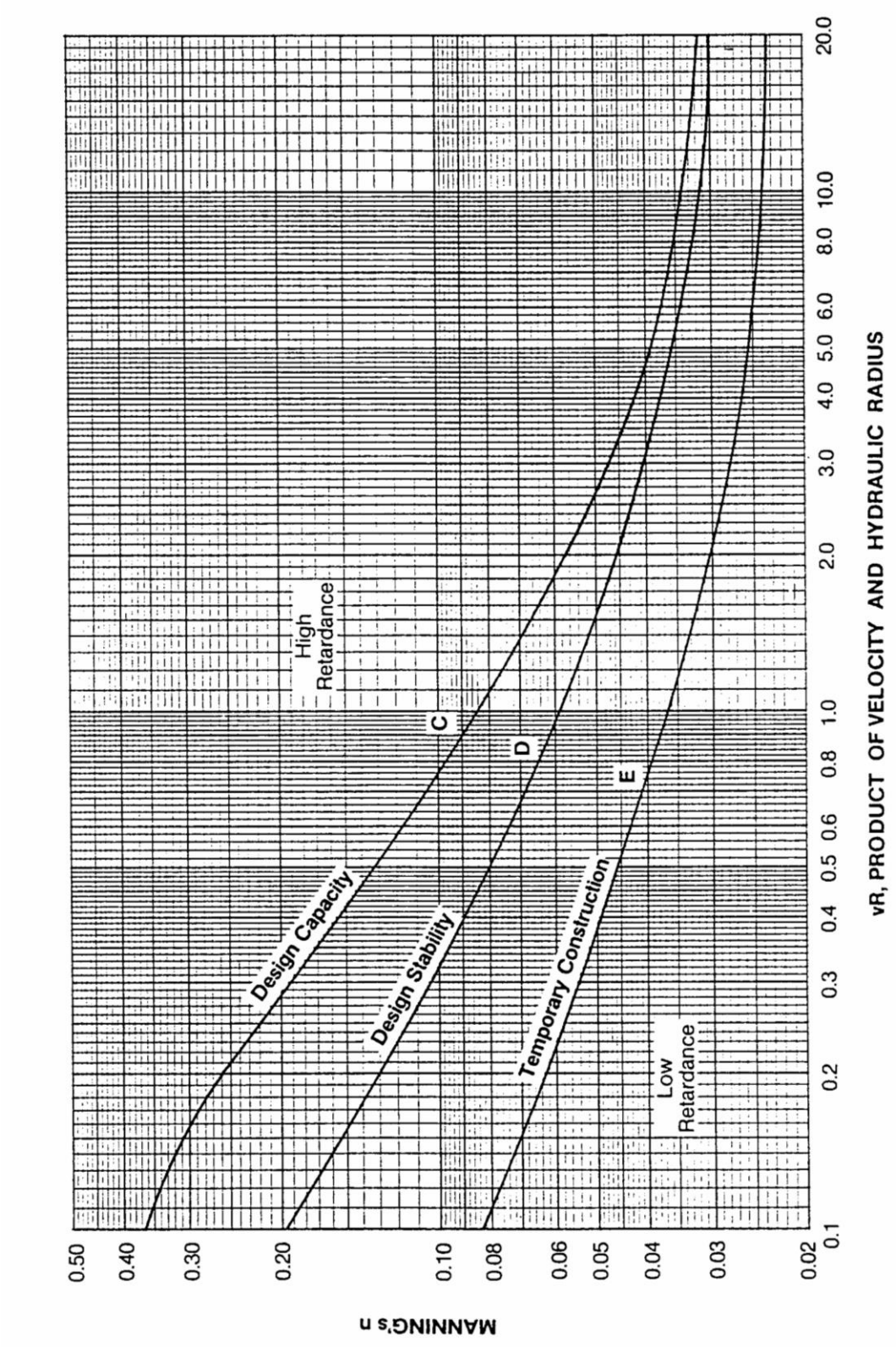


Figure 3.2 Manning's n Values for Vegetated Channels
(Source: USDA, TP-61, 1947)

Table 3.6 Classification of Vegetal Covers as to Degrees of Retardance

Retardance	Cover	Condition
A	Weeping Lovegrass	Excellent stand, tall (average 30")
	Yellow Bluestem Ischaemum	Excellent stand, tall (average 36")
B	Kudzu	Very dense growth, uncut
	Bermuda grass	Good stand, tall (average 12")
	Native grass mixture Little bluestem, bluestem, blue gamma other short and long stem Midwest grasses	Good stand, unmowed
	Weeping lovegrass	Good stand, tall (average 24")
	Laspedeza sericea	Good stand, not woody, tall (average 19")
	Alfalfa	Good stand, uncut (average 11")
	Weeping lovegrass	Good stand, unmowed (average 13")
	Kudzu	Dense growth, uncut
	Blue gamma	Good stand, uncut (average 13")
C	Crabgrass	Fair stand, uncut (10 – 48")
	Bermuda grass	Good stand, mowed (average 6")
	Common lespedeza	Good stand, uncut (average 11")
	Grass-legume mixture: summer (orchard grass redtop, Italian ryegrass, and common lespedeza)	Good stand, uncut (6 – 8 ")
	Centipede grass	Very dense cover (average 6")
	Kentucky bluegrass	Good stand, headed (6 – 12")
D	Bermuda grass	Good stand, cut to 2.5"
	Common lespedeza	Excellent stand, uncut (average 4.5")
	Buffalo grass	Good stand, uncut (3 – 6")
	Grass-legume mixture: fall, spring (orchard grass, redtop, Italian ryegrass, and common lespedeza)	Good stand, uncut (4 – 5")
	Lespedeza serices	After cutting to 2" (very good before cutting)
E	Bermuda grass	Good stand, cut to 1.5"
	Bermuda grass	Burned stubble

Note: Covers classified have been tested in experimental channels. Covers were green and generally uniform.
Source: HEC-15, 1988

3.2.4 Uniform Flow Calculations

Design Charts

Following is a discussion of the equations that can be used for the design and analysis of open channel flow. The Federal Highway Administration has prepared numerous design charts to aid in the design of rectangular, trapezoidal, and triangular open channel cross sections. In addition, design charts for grass-lined channels have been developed. Examples of these charts and instructions for their use are given in *Section 3.2.11*.

Manning's Equation

Manning's Equation, presented in three forms below, is recommended for evaluating uniform flow conditions in open channels:

$$v = (1.49/n)\, R^{2/3}\, S^{1/2} \quad (3.1)$$

$$Q = (1.49/n)\, A\, R^{2/3}\, S^{1/2} \quad (3.2)$$

$$S = [Q_n/(1.49\, A\, R^{2/3})]^2 \quad (3.3)$$

where:

- v = average channel velocity (ft/s)
- Q = discharge rate for design conditions (cfs)
- n = Manning's roughness coefficient
- A = cross-sectional area (ft^2)
- R = hydraulic radius A/P (ft)
- P = wetted perimeter (ft)
- S = slope of the energy grade line (ft/ft)

For prismatic channels, in the absence of backwater conditions, the slope of the energy grade line, water surface and channel bottom are assumed to be equal.

For a more comprehensive discussion of open channel theory and design, see the reference USACE, 1991/1994.

Geometric Relationships

Area, wetted perimeter, hydraulic radius, and channel top width for standard channel cross sections can be calculated from geometric dimensions. Irregular channel cross sections (i.e., those with a narrow deep main channel and a wide shallow overbank channel) must be subdivided into segments so that the flow can be computed separately for the main channel and overbank portions. This same process of subdivision may be used when different parts of the channel cross section have different roughness coefficients. When computing the hydraulic radius of the subsections, the water depth common to the two adjacent subsections is not counted as wetted perimeter.

Direct Solutions

When the hydraulic radius, cross-sectional area, and roughness coefficient and slope are known, discharge can be calculated directly from Equation 3.2. The slope can be calculated using Equation 3.3 when the discharge, roughness coefficient, area, and hydraulic radius are known.

Nomographs for obtaining direct solutions to Manning's Equation are presented in Figures 3.3 and 3.4. Figure 3.3 provides a general solution for the velocity form of Manning's Equation, while Figure 3.4 provides a solution of Manning's Equation for trapezoidal channels.

General Solution Nomograph

The following steps are used for the general solution nomograph in Figure 3.3:

Step 1 Determine open channel data, including slope in ft/ft, hydraulic radius in ft, and Manning's n value.

Step 2 Connect a line between the Manning's n scale and slope scale and note the point of intersection on the turning line.

Step 3 Connect a line from the hydraulic radius to the point of intersection obtained in Step 2.

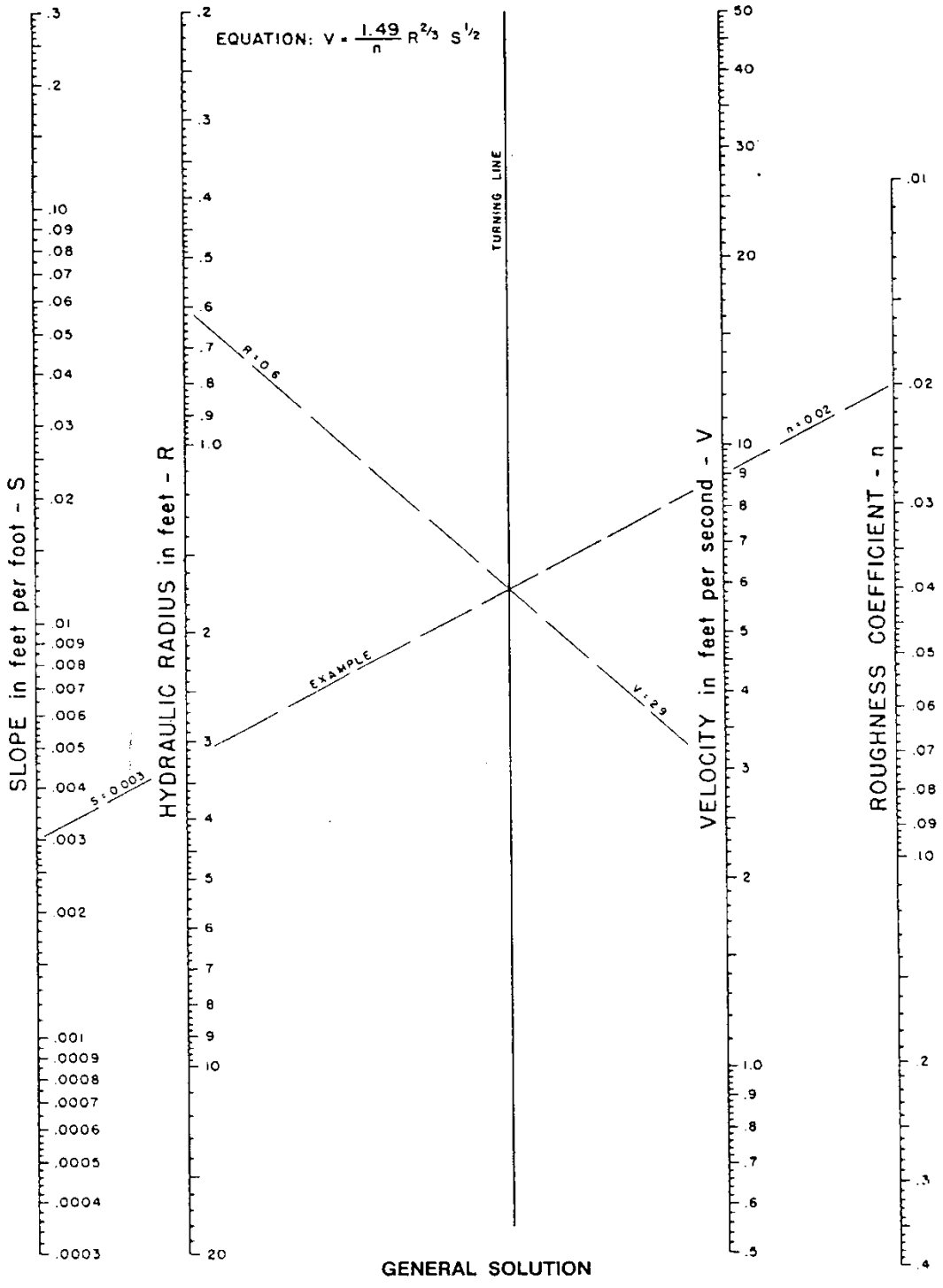
Step 4 Extend the line from Step 3 to the velocity scale to obtain the velocity in ft/s.

Trapezoidal Solution Nomograph

The trapezoidal channel nomograph solution to Manning's Equation in Figure 3.4 can be used to find the depth of flow if the design discharge is known or the design discharge if the depth of flow is known.

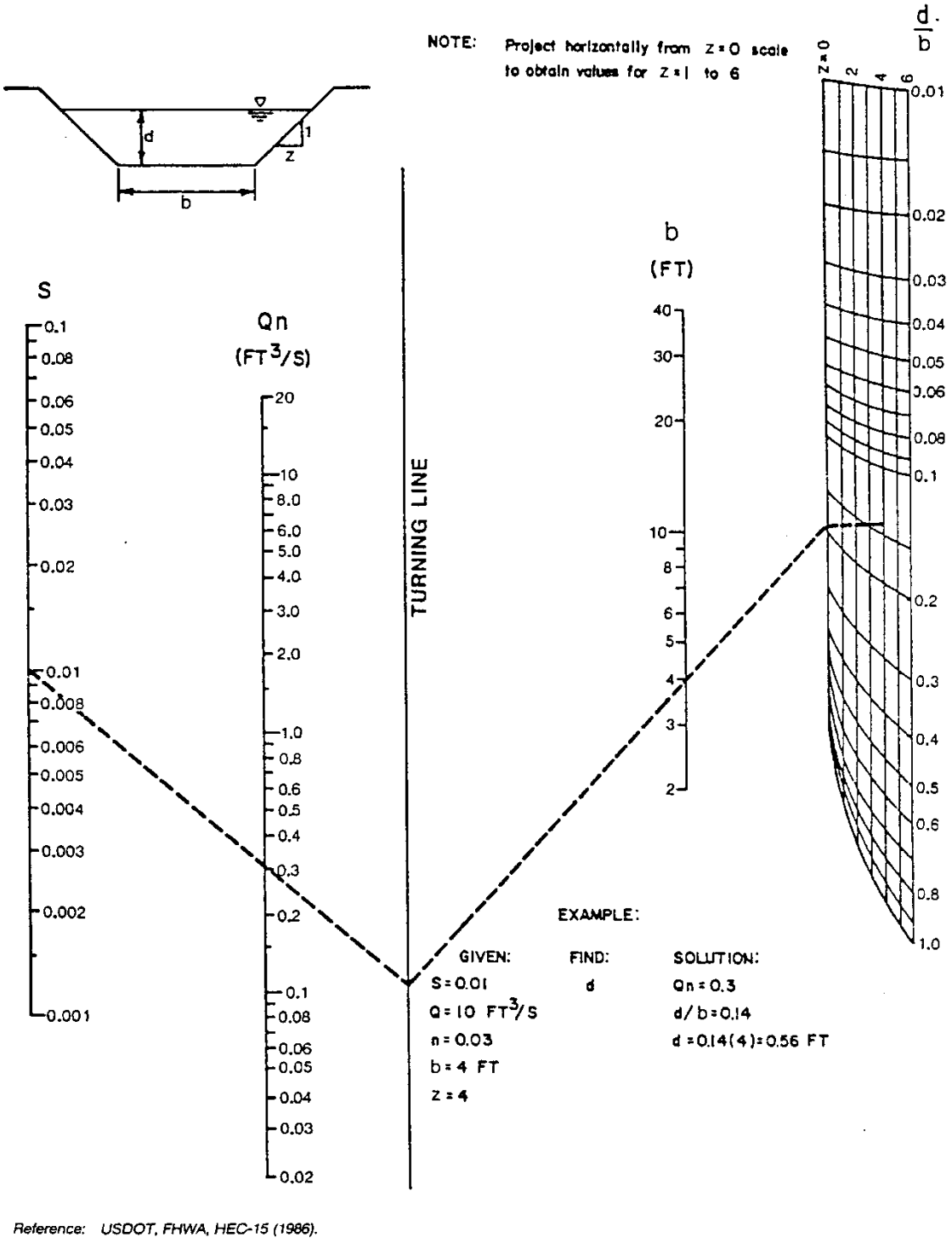
Determine input data, including slope in ft/ft, Manning's n value, bottom width in ft, and side slope in ft/ft.

- Given Q, find d.
  a. Given the design discharge, find the product of Q times n, connect a line from the slope scale to the Qn scale, and find the point of intersection on the turning line.
  b. Connect a line from the turning point from Step 2a to the b scale and find the intersection with the z = 0 scale.
  c. Project horizontally from the point located in Step 2b to the appropriate z value and find the value of d/b.
  d. Multiply the value of d/b obtained in Step 2c by the bottom width b to find the depth of uniform flow, d.
- Given d, find Q
  a. Given the depth of flow, find the ratio d divided by b and project a horizontal line from the d/b ratio at the appropriate side slope, z, to the z = 0 scale.
  b. Connect a line from the point located in Step 3a to the b scale and find the intersection with the turning line.
  c. Connect a line from the point located in Step 3b to the slope scale and find the intersection with the Q_n scale.
  d. Divide the value of Qn obtained in Step 3c by the n value to find the design discharge, Q.



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

Figure 3.3 Nomograph for the Solution of Manning's Equation



Reference: USDOT, FHWA, HEC-15 (1986).

Figure 3.4 Solution of Manning's Equation for Trapezoidal Channels

Trial and Error Solutions

A trial and error procedure for solving Manning's Equation is used to compute the normal depth of flow in a uniform channel when the channel shape, slope, roughness, and design discharge are known. For purposes of the trial and error process, Manning's Equation can be arranged as:

$$AR^{2/3} = (Qn)/(1.49\ S^{1/2}) \qquad (3.4)$$

where:

- A = cross-sectional area (ft)
- R = hydraulic radius (ft)
- Q = discharge rate for design conditions (cfs)
- n = Manning's roughness coefficient
- S = slope of the energy grade line (ft/ft)

To determine the normal depth of flow in a channel by the trial and error process, trial values of depth are used to determine A, P, and R for the given channel cross section. Trial values of $AR^{2/3}$ are computed until the equality of Equation 3.4 is satisfied such that the design flow is conveyed for the slope and selected channel cross section.

Graphical procedures for simplifying trial and error solutions are presented in Figure 3.5 for trapezoidal channels. Computer programs are also available for these calculations.

Step 1 Determine input data, including design discharge, Q, Manning's n value, channel bottom width, b, channel slope, S, and channel side slope, z.

Step 2 Calculate the trapezoidal conveyance factor using the equation:

$$K_T = (Qn)/(b^{8/3}S^{1/2}) \qquad (3.5)$$

where:

- K_T = trapezoidal open channel conveyance factor
- Q = discharge rate for design conditions (cfs)
- n = Manning's roughness coefficient
- b = bottom width (ft)
- S = slope of the energy grade line (ft/ft)

Step 3 Enter the x-axis of Figure 3.5 with the value of K_T calculated in Step 2 and draw a line vertically to the curve corresponding to the appropriate z value from Step 1.

Step 4 From the point of intersection obtained in Step 3, draw a horizontal line to the y-axis and read the value of the normal depth of flow over the bottom width, d/b.

Step 5 Multiply the d/b value from Step 4 by b to obtain the normal depth of flow.

Note: If bends are considered, refer to Equation 3.11

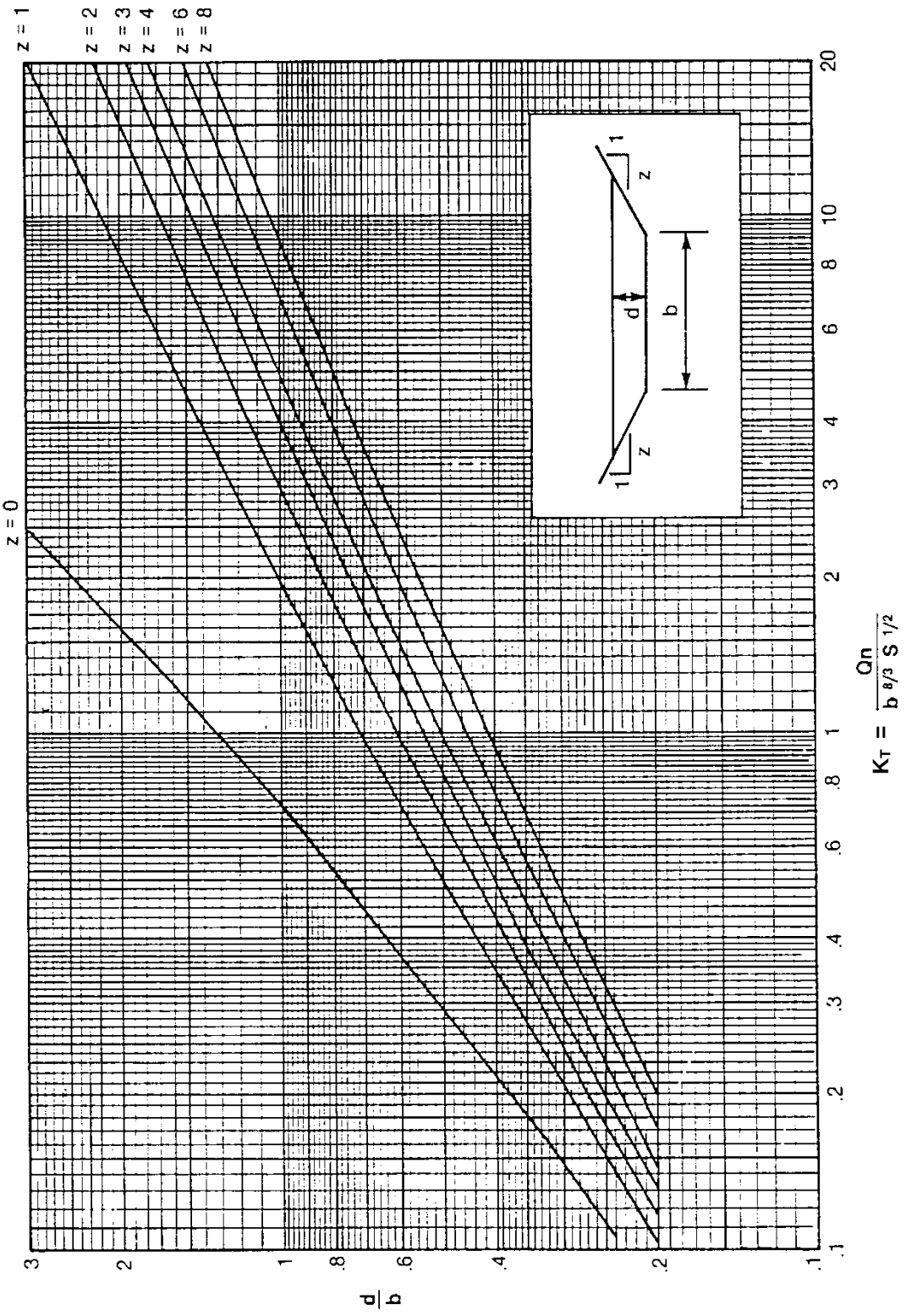


Figure 3.5 Trapezoidal Channel Capacity Chart
(Source: Nashville Stormwater Management Manual, 1988)

3.2.5 Critical Flow Calculations

Background

In the design of open channels, it is important to calculate the critical depth in order to determine if the flow in the channel will be subcritical or supercritical. If the flow is subcritical it is relatively easy to handle the flow through channel transitions because the flows are tranquil and wave action is minimal. In subcritical flow, the depth at any point is influenced by a downstream control, which may be either the critical depth or the water surface elevation in a pond or larger downstream channel. In supercritical flow, the depth of flow at any point is influenced by a control upstream, usually critical depth. In addition, the flows have relatively shallow depths and high velocities. Hydraulic jumps are possible under these conditions and consideration should be given to stabilizing the channel.

Critical depth depends only on the discharge rate and channel geometry. The general equation for determining critical depth is expressed as:

$$\mathbf{Q^2/g = A^3/T} \qquad \mathbf{(3.6)}$$

where:

Q = discharge rate for design conditions (cfs)
g = acceleration due to gravity (32.2 ft/sec^2)
A = cross-sectional area (ft^2)
T = top width of water surface (ft)

Note: A trial and error procedure is needed to solve Equation 3.6.

Semi-Empirical Equations

Semi-empirical equations (as presented in Table 3.7) or section factors (as presented in Figure 3.6) can be used to simplify trial and error critical depth calculations. The following equation is used to determine critical depth with the critical flow section factor, Z:

$$\mathbf{Z = Q/(g^{0.5})} \qquad \mathbf{(3.7)}$$

where:

Z = critical flow section factor
Q = discharge rate for design conditions (cfs)
g = acceleration due to gravity (32.3 ft/sec^2)

The following guidelines are given for evaluating critical flow conditions of open channel flow:

- A normal depth of uniform flow within about 10% of critical depth is unstable and should be avoided in design, if possible.
- If the velocity head is less than one-half the mean depth of flow, the flow is subcritical.
- If the velocity head is equal to one-half the mean depth of flow, the flow is critical.
- If the velocity head is greater than one-half the mean depth of flow, the flow is supercritical.

Note: The head is the height of water above any point, plane, or datum of reference. The velocity head in flowing water is calculated as the velocity squared divided by 2 times the gravitational constant ($V^2/2g$).

The Froude number, Fr, calculated by the following equation, is useful for evaluating the type of flow conditions in an open channel:

$$Fr = v/(gA/T)^{0.5} \quad (3.8)$$

where:

Fr = Froude number (dimensionless)
v = velocity of flow (ft/s)
g = acceleration of gravity (32.2 ft/sec^2)
A = cross-sectional area of flow (ft^2)
T = top width of flow (ft)

If Fr is greater than 1.0, flow is supercritical; if it is under 1.0, flow is subcritical. Fr is 1.0 for critical flow conditions.

Table 3.7 Critical Depth Equations for Uniform Flow in Selected Channel Cross Sections

Channel Type[1]	Semi-Empirical Equations[2] for Estimating Critical Depth	Range of Applicability
1. Rectangular[3]	$d_c = [Q^2/(gb^2)]^{1/3}$	N/A
2. Trapezoidal[3]	$d_c = 0.81[Q^2/(gz^{0.75b1.25})]^{0.27} - b/30z$	$0.1 < 0.5522\ Q/b^{2.5} < 0.4$ For $0.5522\ Q/b^{2.5} < 0.1$, use rectangular channel equation
3. Triangular[3]	$d_c = [(2Q^2)/(gz^2)]^{1/5}$	N/A
4. Circular[4]	$d_c = 0.325(Q/D)^{2/3} + 0.083D$	$0.3 < d_c/D < 0.9$
5. General[5]	$(A^3/T) = (Q^2/g)$	N/A

where:

d_c = critical depth (ft)
Q = design discharge (cfs)
g = acceleration due to gravity (32.3 ft/s^2)
b = bottom width of channel (ft)
z = side slopes of a channel (horizontal to vertical)
D = diameter of circular conduit (ft)
A = cross-sectional area of flow (ft^2)
T = top width of water surface (ft)

[1] See Figure 3.6 for channel sketches
[2] Assumes uniform flow with the kinetic energy coefficient equal to 1.0
[3] Reference: French (1985)
[4] Reference: USDOT, FHWA, HDS-4 (1965)
[5] Reference: Brater and King (1976)

If the water surface profile in a channel transitions from supercritical flow to subcritical flow, a hydraulic jump must occur. The location of the hydraulic jump and its sequent depth are critical to proper design of free flow conveyances. To determine the location of a hydraulic jump, the standard step method is used to compute the water surface profile and specific force (momentum principle) and specific energy relationships are used. For computational methods refer to Chow, 1959, TxDOT, 2002, and Mays, 1999. The HEC-RAS computer program can be used to compute water surface profiles for both subcritical and supercritical flow regimes.

Section	Area A	Wetted Perimeter p	Hydraulic Radius R	Top Width T	Critical Depth Factor, $\frac{Z}{\sqrt{b}}$
Trapezoid	$bd+zd^2$	$b+2d\sqrt{z^2+1}$	$\frac{bd+zd^2}{b+2d\sqrt{z^2+1}}$	$b+2zd$	$\frac{[(b+zd)d]^{1.5}}{\sqrt{b+2zd}}$
Rectangle	bd	$b+2d$	$\frac{bd}{b+2d}$	b	$bd^{1.5}$
Triangle	zd^2	$2d\sqrt{z^2+1}$	$\frac{zd}{2\sqrt{z^2+1}}$	$2zd$	$\frac{\sqrt{2}}{2}zd^{2.5}$
Parabola	$\frac{2}{3}dT$	$T+\frac{8d^2}{3T}$ [1]	$\frac{2dT^2}{3T^2+8d^2}$ [1]	$\frac{3a}{2d}$	$\frac{2}{9}\sqrt{6}\,Td^{1.5}$
Circle - < ½ full [2]	$\frac{D^2}{8}\left(\frac{\pi\theta}{180}-\sin\theta\right)$	$\frac{\pi D\theta}{360}$	$\frac{45D}{\pi\theta}\left(\frac{\pi\theta}{180}-\sin\theta\right)$	$D\sin\frac{\theta}{2}$ or $2\sqrt{d(D-d)}$	$a\sqrt{\frac{a}{D\sin\frac{\theta}{2}}}$
Circle - > ½ full [3]	$\frac{D^2}{8}\left(2\pi-\frac{\pi\theta}{180}+\sin\theta\right)$	$\frac{\pi D(360-\theta)}{360}$	$\frac{45D}{\pi(360-\theta)}\left(2\pi-\frac{\pi\theta}{180}+\sin\theta\right)$	$D\sin\frac{\theta}{2}$ or $2\sqrt{d(D-d)}$	$a\sqrt{\frac{a}{D\sin\frac{\theta}{2}}}$

[1] Satisfactory approximation for the interval $0<\frac{d}{T}\leq 0.25$
When $d/T>0.25$, use $p=\frac{1}{2}\sqrt{16d^2+T^2}+\frac{T^2}{8d}\sinh^{-1}\frac{4d}{T}$

[2] $\theta=4\sin^{-1}\sqrt{d/D}$ } Insert θ in degrees in above equations
[3] $\theta=4\cos^{-1}\sqrt{d/D}$ }

Note: Small z = Side Slope Horizontal Distance
Large Z = Critical Depth Section Factor

Reference: USDA, SCS, NEH-5 (1956).

Figure 3.6 Open Channel Geometric Relationships for Various Cross Sections

3.2.6 Vegetative Design

Introduction

A two-part procedure is recommended for final design of temporary and vegetative channel linings. Part 1, the design stability component, involves determining channel dimensions for low vegetative retardance conditions, using Class D as defined in Table 3.6. Part 2, the design capacity component, involves determining the depth increase necessary to maintain capacity for higher vegetative retardance conditions, using Class C as defined in Table 3.6. If temporary lining is to be used during construction, vegetative retardance Class E should be used for the design stability calculations.

If the channel slope exceeds 10%, or a combination of channel linings will be used, additional procedures not presented below are required. References include HEC-15 (USDOT, FHWA, 1986) and HEC-14 (USDOT, FHWA, 1983).

Design Stability

The following are the steps for design stability calculations:

Step 1 Determine appropriate design variables, including discharge, Q, bottom slope, S, cross section parameters, and vegetation type.

Step 2 Use Table 3.3 to assign a maximum velocity, v_m based on vegetation type and slope range.

Step 3 Assume a value of n and determine the corresponding value of vR from the n versus vR curves in Figure 3.2. Use retardance Class D for permanent vegetation and E for temporary construction.

Step 4 Calculate the hydraulic radius using the equation:

$$\mathbf{R = (vR)/v_m} \qquad \mathbf{(3.9)}$$

where:

R = hydraulic radius of flow (ft)

vR = value obtained from Figure 3.2 in Step 3

v_m = maximum velocity from Step 2 (ft/s)

Step 5 Use the following form of Manning's Equation to calculate the value of vR:

$$\mathbf{vR = (1.49\ R^{5/3}\ S^{1/2})/n} \qquad \mathbf{(3.10)}$$

where:

vR = calculated value of vR product

R = hydraulic radius value from Step 4 (ft)

S = channel bottom slope (ft/ft)

n = Manning's n value assumed in Step 3

Step 6 Compare the vR product value obtained in Step 5 to the value obtained from Figure 3.2 for the assumed n value in Step 3. If the values are not reasonably close, return to Step 3 and repeat the calculations using a new assumed n value.

Step 7 For trapezoidal channels, find the flow depth using Figures 3.4 or 3.5, as described in *Section 3.2.4*. The depth of flow for other channel shapes can be evaluated using the trial and error procedure described in *Section 3.2.4*.

Step 8 If bends are considered, calculate the length of downstream protection, L_p, for the bend, using Figure 3.7. Provide additional protection, such as gravel or riprap in the bend and extending downstream for length, L_p.

Design Capacity

The following are the steps for design capacity calculations:

Step 1 Assume a depth of flow greater than the value from Step 7 above and compute the waterway area and hydraulic radius (see Figure 3.6 for equations).

Step 2 Divide the design flow rate, obtained using appropriate procedures from the *Hydrology Technical Manual*, by the waterway area from Step 1 to find the velocity.

Step 3 Multiply the velocity from Step 2 by the hydraulic radius from Step 1 to find the value of vR.

Step 4 Use Figure 3.2 to find a Manning's n value for retardance Class C based on the vR value from Step 3.

Step 5 Use Manning's Equation (Equation 3.1) or Figure 3.3 to find the velocity using the hydraulic radius from Step 1, Manning's n value from Step 4, and appropriate bottom slope.

Step 6 Compare the velocity values from Steps 2 and 5. If the values are not reasonably close, return to Step 1 and repeat the calculations.

Step 7 Add an appropriate freeboard to the final depth from Step 6. Generally, 20% is adequate.

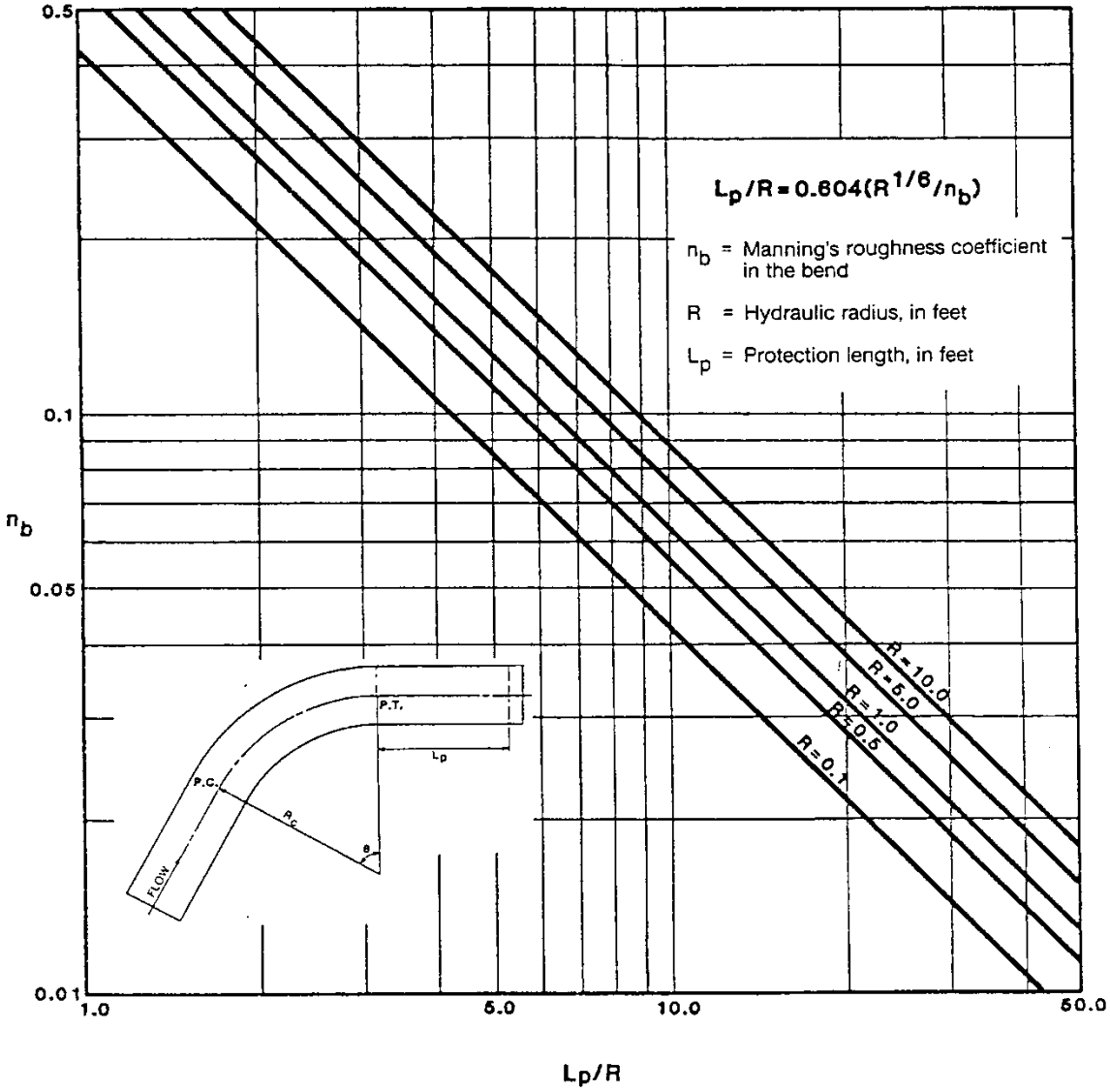
Step 8 If bends are considered, calculate super-elevation of the water surface profile at the bend using the equation:

$$\Delta d = (v^2 T)/(g R_c) \qquad (3.11)$$

where:

Δd = super-elevation of the water surface profile due to the bend (ft)
v = average velocity from Step 6 (ft/s)
T = top width of flow (ft)
g = acceleration of gravity (32.2 ft/sec^2)
R_c = mean radius of the bend (ft)

Note: Add freeboard consistent with the calculated Δd.



Reference: USDOT, FHWA, HEC-15 (1986).

Figure 3.7 Protection Length, Lp, Downstream of Channel Bend

3.2.7 Stone Riprap Design

Introduction

A number of agencies and researchers have studied and developed empirical equations to estimate the required size of rock riprap to resist various hydraulic conditions, including the U.S. Army Corps of Engineers (USACE), Natural Resource Conservation Service (NRCS), and the Federal Highway Administration (FHWA). As with all empirical equations based on the results of laboratory experiments, they must be used with an understanding of the range of data on which they are based.

The following methods give design guidance for designing stone riprap for open channels. Design guidance for designing stone riprap for culvert outfall protection is also provided in this section. *Section 4.0* gives additional guidance on the design of riprap aprons for erosion protection at outfalls, and the design of riprap basins for energy dissipation.

Method #1: Maynord & Reese

The following procedure is based on results and analysis of laboratory and field data (Maynord, 1987; Reese, 1984; Reese, 1988). This procedure applies to riprap placement in both natural and prismatic channels and has the following assumptions and limitations:

- Minimum riprap thickness equal to d_{100}
- The value of d_{85}/d_{15} less than 4.6
- Froude number less than 1.2
- Side slopes up to 2:1
- A safety factor of 1.2
- Maximum velocity less than 18 feet per second
- If significant turbulence is caused by boundary irregularities, such as vertical drops, obstructions, or structures, this procedure is not applicable.

Procedure

Following are the steps in the procedure for riprap design using the method by Maynord & Reese:

Step 1 Determine the average velocity in the main channel for the design condition. Manning's n values for riprap can be calculated from the equation:

$$\mathbf{n = 0.0395 (d_{50})^{1/6}} \qquad \mathbf{(3.12)}$$

where:

n = Manning's roughness coefficient for stone riprap

d_{50} = diameter of stone for which 50%, by weight, of the gradation is finer (ft)

Step 2 If rock is to be placed at the outside of a bend, multiply the velocity determined in Step 1 by the bend correction coefficient, C_b, given in Figure 3.8 for either a natural or prismatic channel. This requires determining the channel top width, T, just upstream from the bend and the centerline bend radius, R_b.

Step 3 If the specific weight of the stone varies significantly from 165 pounds per cubic foot, multiply the velocity from Step 1 or 2 (as appropriate) by the specific weight correction coefficient, C_g, from Figure 3.9.

Step 4 Determine the required minimum d_{30} value from Figure 3.10, or from the equation:

$$d_{30}/D = 0.193\ Fr^{2.5} \qquad (3.13)$$

where:

d_{30} = diameter of stone for which 30%, by weight, of the gradation is finer (ft)
D = depth of flow above stone (ft)
Fr = Froude number (see Equation 3.8), dimensionless
v = mean velocity above the stone (ft/s)
g = acceleration of gravity (32.2 ft/sec)

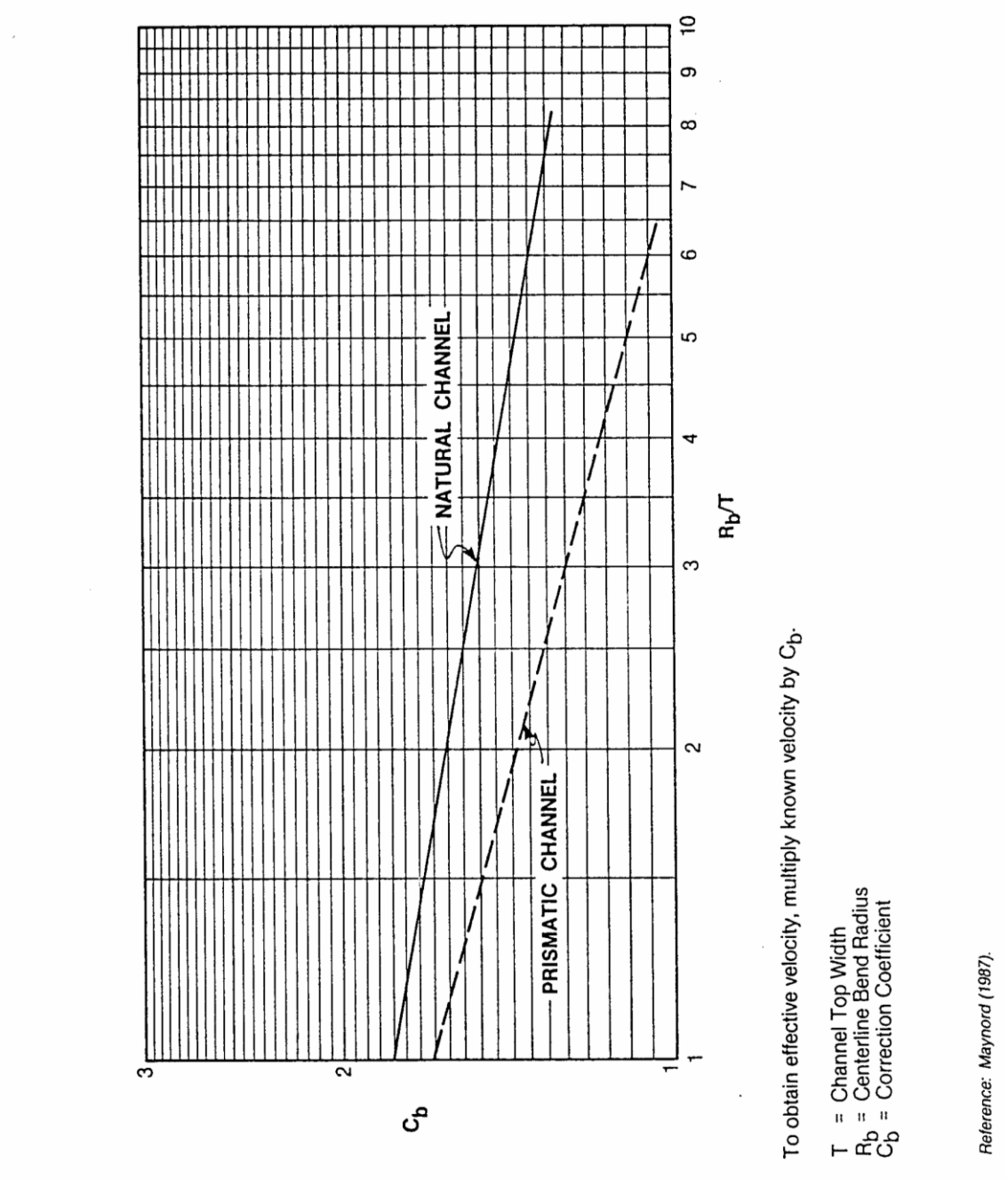


Figure 3.8 Riprap Lining Bend Correction Coefficient

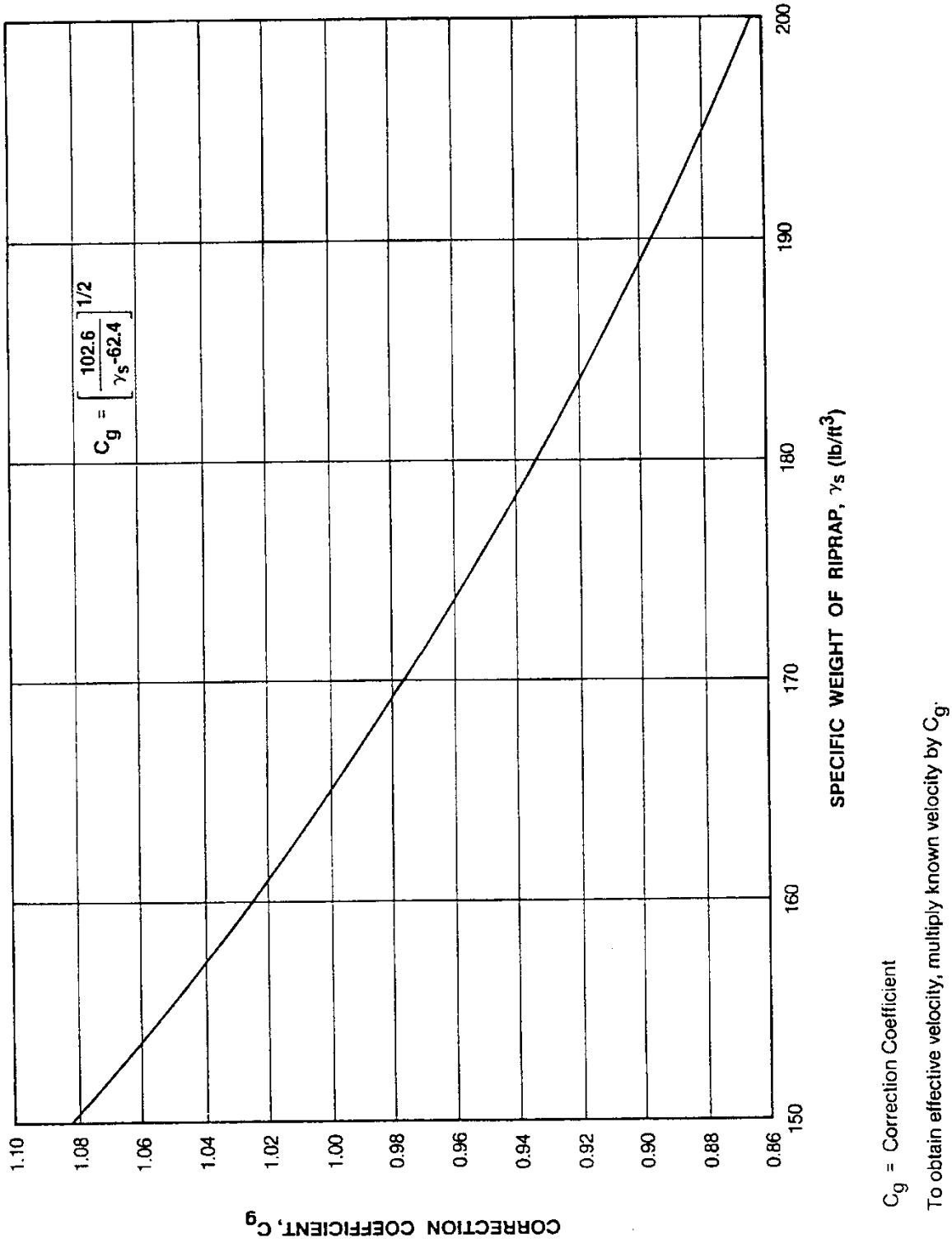
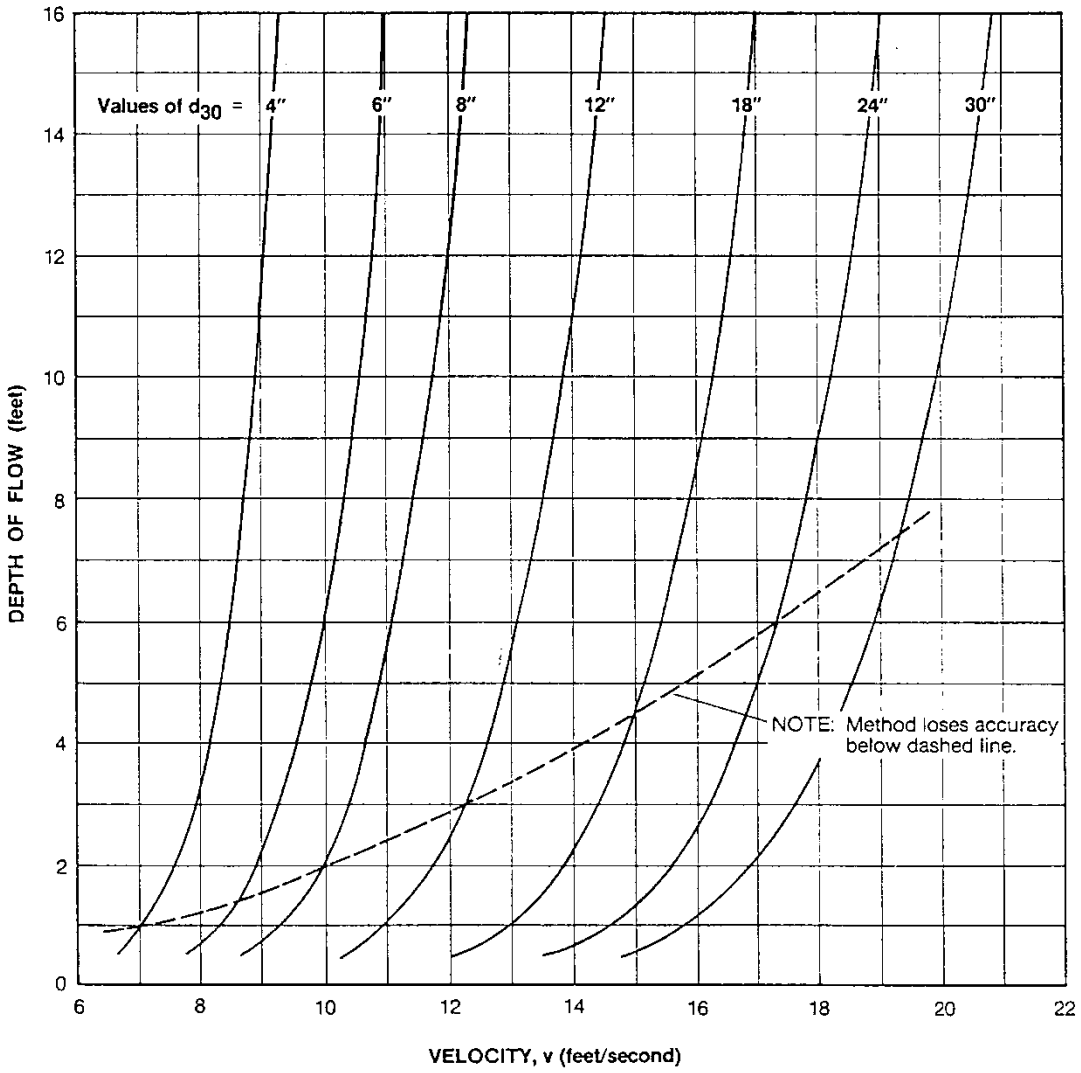


Figure 3.9 Riprap Lining Specific Weight Correction Coefficient
(Source: Nashville Stormwater Management Manual, 1988)



Reference: Reese (1988).

Figure 3.10 Riprap Lining d_{30} Stone Size – Function of Mean Velocity and Depth

Step 5 Determine available riprap gradations. A well graded riprap is required. The diameter of the largest stone, d_{100}, should not be more than 1.5 times the d_{50} size. Blanket thickness should be greater than or equal to d_{100} except as noted below. Sufficient fines (below d_{15}) should be available to fill the voids in the larger rock sizes. The stone weight for a selected stone size can be calculated from the equation:

$$W = 0.5236\ SW_s\ d^3 \qquad (3.14)$$

where:

W = stone weight (lbs)

d = selected stone diameter (ft)

SW_s = specific weight of stone (lbs/ft^3)

Filter fabric or a filter stone layer should be used to prevent turbulence or groundwater seepage from removing bank material through the stone or to serve as a foundation for unconsolidated material. Layer thickness should be increased by 50% for underwater placement.

Step 6 If d_{85}/d_{15} is between 2.0 and 2.3 and a smaller d_{30} size is desired, a thickness greater than d_{100} can be used to offset the smaller d_{30} size. Figure 3.11 can be used to make an approximate adjustment using the ratio of d_{30} sizes. Enter the y-axis with the ratio of the desired d_{30} size to the standard d_{30} size and find the thickness ratio increase on the x-axis. Other minor gradation deficiencies may be compensated for by increasing the stone blanket thickness.

Step 7 Perform preliminary design, ensuring that adequate transition is provided to natural materials both up and downstream to avoid flanking and that toe protection is provided to avoid riprap undermining.

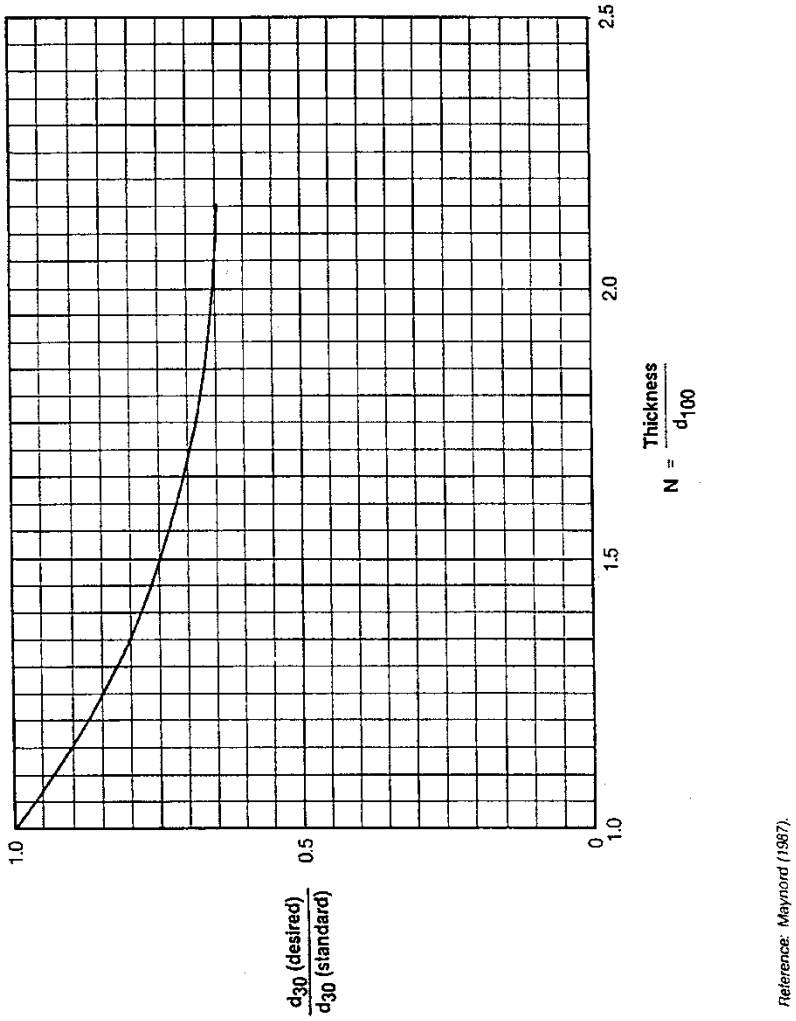


Figure 3.11 Riprap Lining Thickness Adjustment for d_{85}/d_{15} = 1.0 to 2.3
(Source: Maynord, 1987)

Method #2: Gregory

The following procedure is based on excerpts from a paper prepared by Garry Gregory (June, 1987) and has been widely used in the Dallas-Fort Worth area for riprap design.

Procedure

Following are the steps in the procedure for riprap design using the method by Gregory:

Step 1 Calculate the boundary shear (tractive stress or tractive force) by:

$$T_o = \Upsilon_w RS \quad (3.15)$$

where:

T_o = average tractive stress on channel bottom (lb/ft^2)
Υ_w = unit weight of water (62.4 lb/ft^3)
R = hydraulic radius of channel (ft)
S = slope of energy gradient (ft/ft)

$$T_o' = T_o(1-(\sin^2\Phi/\sin^2\Theta))^{0.5} \quad (3.16)$$

where:

T_o' = average tractive stress on channel side slopes (lb/ft^2)
Φ = angle of side slope with the horizontal
Θ = angle of repose of riprap (approximately 40°)

The greater value of T_o or T_o' governs.

Step 2 Determine the tractive stress in a bend in the channel by:

$$T_b = 3.15T(r/w)^{-0.5} \quad (3.17)$$

where:

T_b = local tractive stress in the bend (lb/ft^2)
T = the greater of T_o or T_o' from Equations 3.15 and 3.16
r = center-line radius of the bend (ft)
w = water surface width at upstream end of bend (ft)

Step 3 Determine D_{50} size of riprap stone (size at which 50% of the gradation is finer weight) from:

$$D_{50} = T/0.04(\Upsilon_s-\Upsilon_w) \quad (3.18)$$

where:

D_{50} = required average size of riprap stone (ft)
T = the greater of T_o or T_o' from Equations 3.15 and 3.16
Υ_s = saturated surface dry (SSD) unit weight of stone (lb/ft^3)
Υ_w = unit weight of water (62.4 lb/ft^3)

Step 4 Select minimum riprap thickness required from grain size curves in Figures 3.12 through 3.17. Select from smaller side of band at 50% finer gradation.

Step 5 Select riprap gradations table (Figures 3.18 and 3.19) based on riprap thickness selected in Step 4.

Step 6 Select bedding thickness from grain size curves in Figures 3.12 through 3.17, which was used to select the riprap thickness in Step 4. Note: The bedding thicknesses included in Figures 3.12 through 3.17 are based on using a properly designed geotextile underneath the bedding. If a geotextile is not used, the bedding thickness must be increased to a minimum of 9 inches for 24 inch and 30 inch riprap and a minimum of 12 inches for the 36 inch riprap.

Step 7 To provide stability in the riprap layer the riprap gradations should meet the following criteria for GRADATION INDEX:

$$\textbf{GRADATION INDEX: } [D_{85}/D_{50} + D_{50}/D_{15}] \leq 5.5 \qquad (3.19)$$

where: D_{85}, D_{50}, and D_{15} are the riprap grain sizes (mm) of which 85%, 50%, and 15% respectively are finer by weight.

Step 8 To provide stability of the bedding layer the bedding should meet the following filter criteria with respect to the riprap:

$$D_{15}/d_{85} < 5 < D_{15}/d_{15} < 40 \qquad (3.20)$$

$$D_{50}/d_{50} < 40 \qquad (3.21)$$

where: D refers to riprap sizes, and d refers to bedding sizes, both in mm.

Step 9 The geotextile underneath the bedding should be designed as a filter to the soil.

Step 10 Typical riprap design sections are shown in Figures 3.20 and 3.21, from the USACE publication EM1110-2-160.

Culvert Outfall Protection

The following procedure is used to design riprap for protection at culvert outfalls.

Step 1 Determine D_{50} size of riprap determined from:

$$D_{50} = \sqrt{V/[1.8\sqrt{(2g(\Upsilon_s - \Upsilon_w)/\Upsilon_w)}]} \qquad (3.22)$$

Step 2 Select riprap and bedding from Figures 3.12 through 3.17 using D_{50} from Equation 3.22.

Step 3 Select gradations from tables in Figures 3.18 and 3.19.

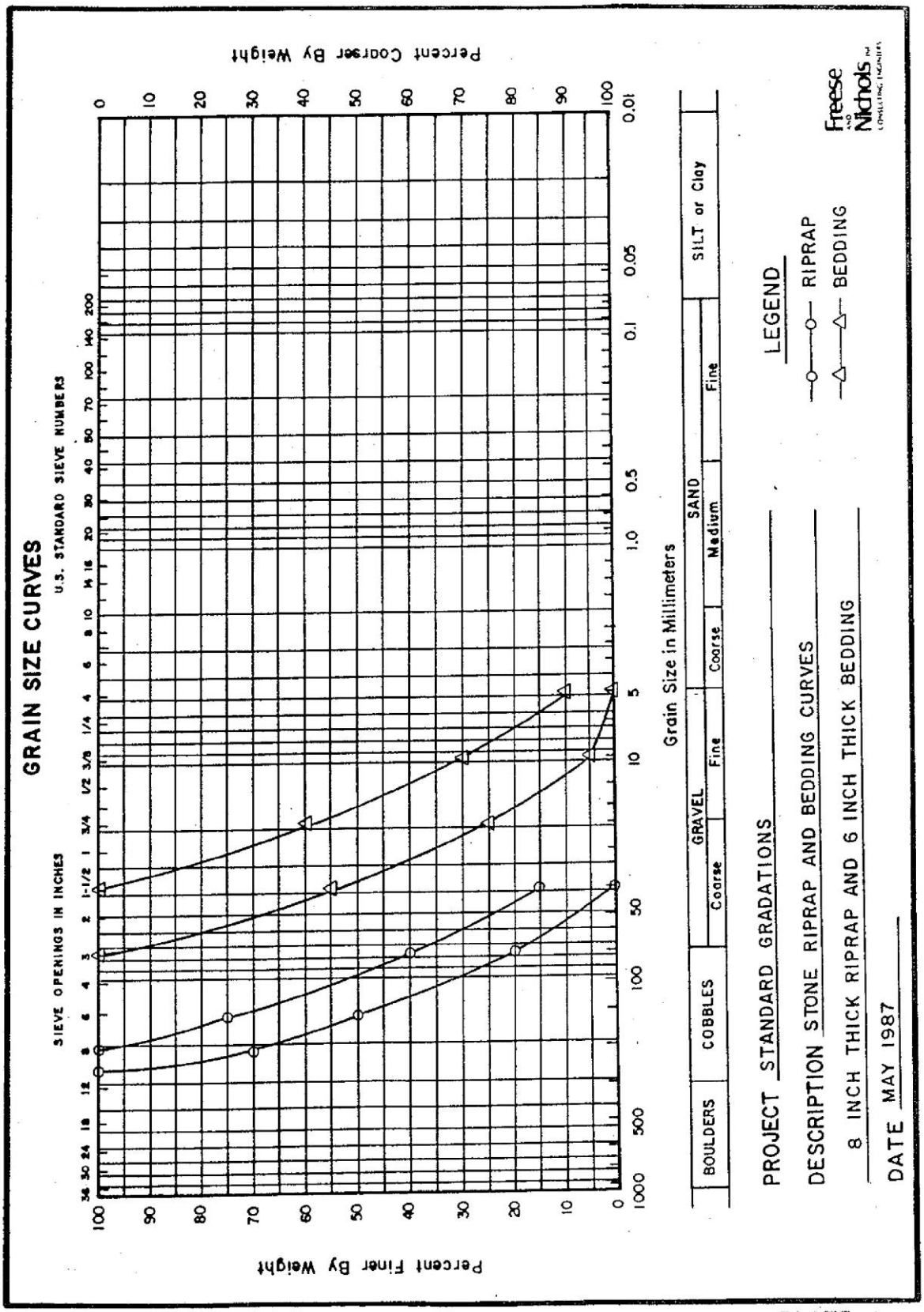


Figure 3.12 Grain Size Curve for 8" Riprap and 6" Bedding

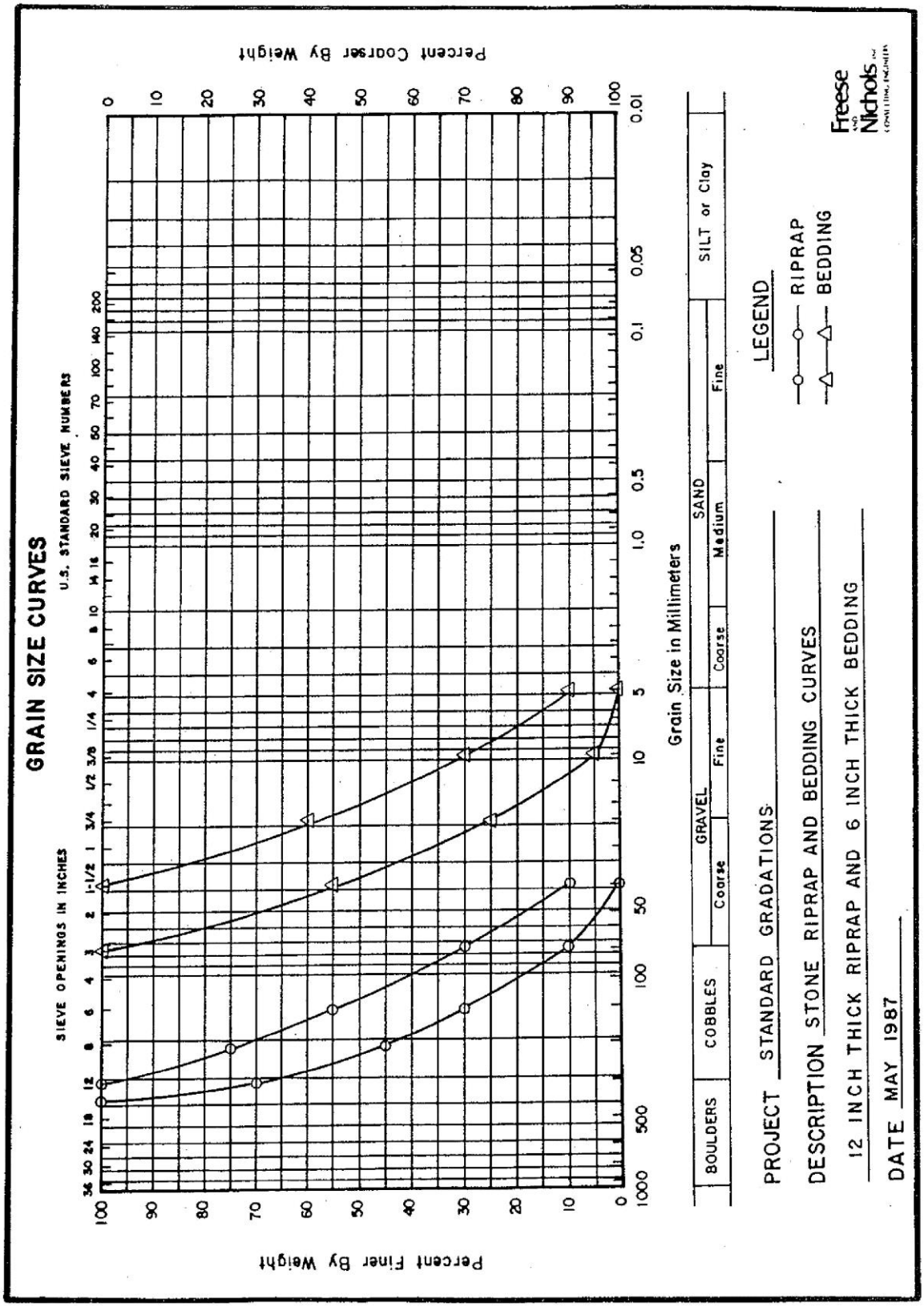


Figure 3.13 Grain Size Curve for 12” Riprap and 6” Bedding

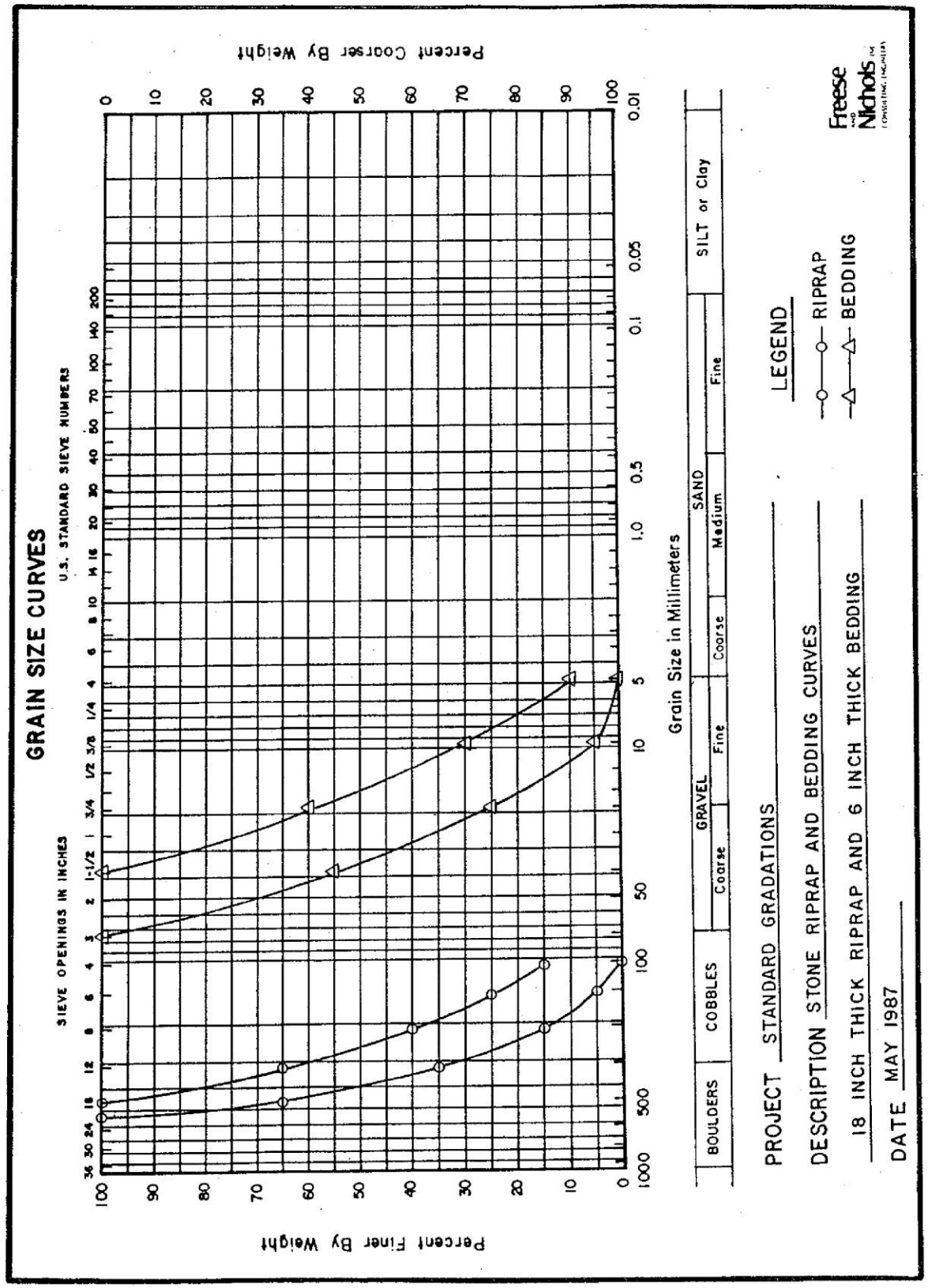


Figure 3.14 Grain Size Curve for 18" Riprap and 6" Bedding

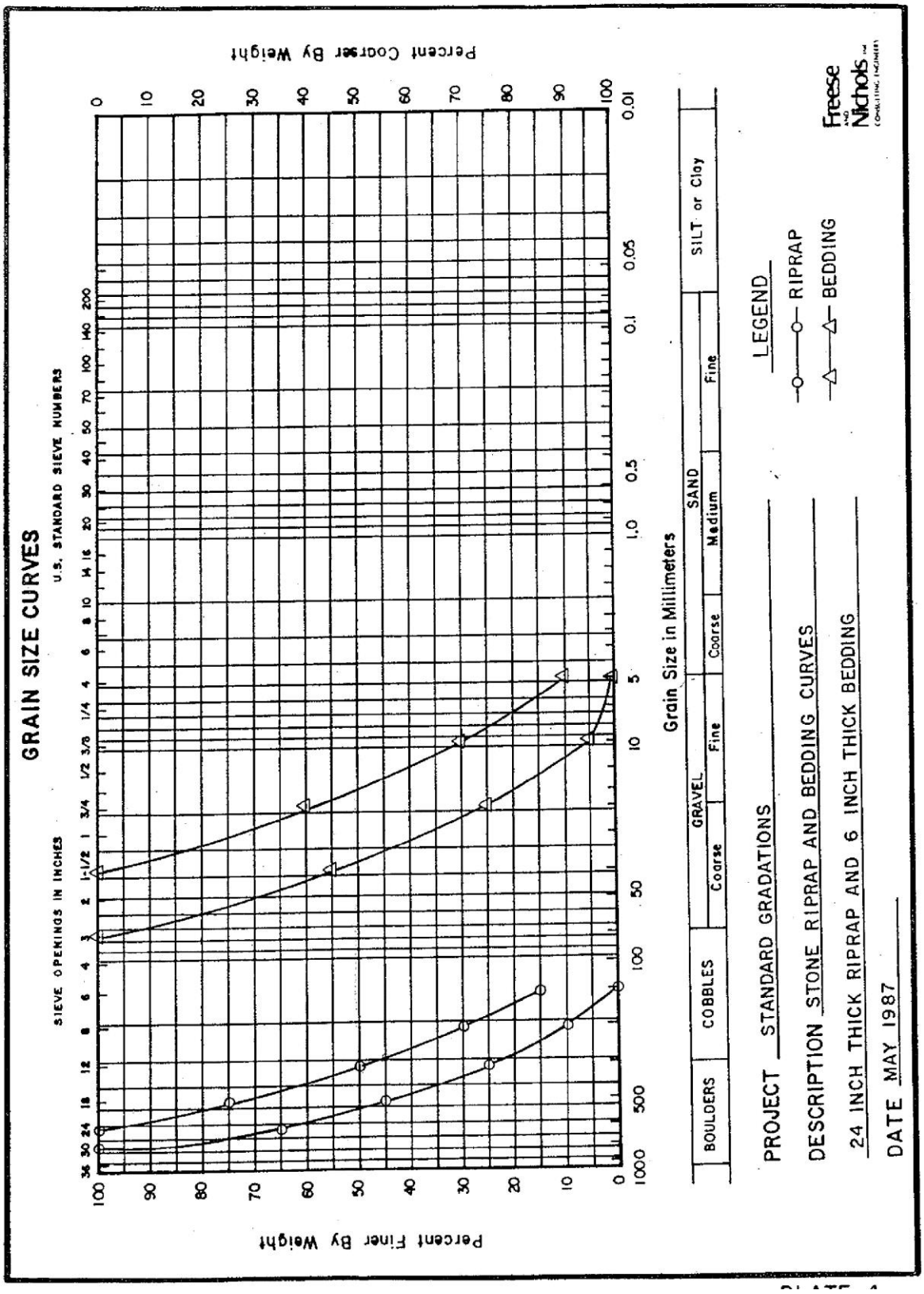


Figure 3.15 Grain Size Curve for 24" Riprap and 6" Bedding

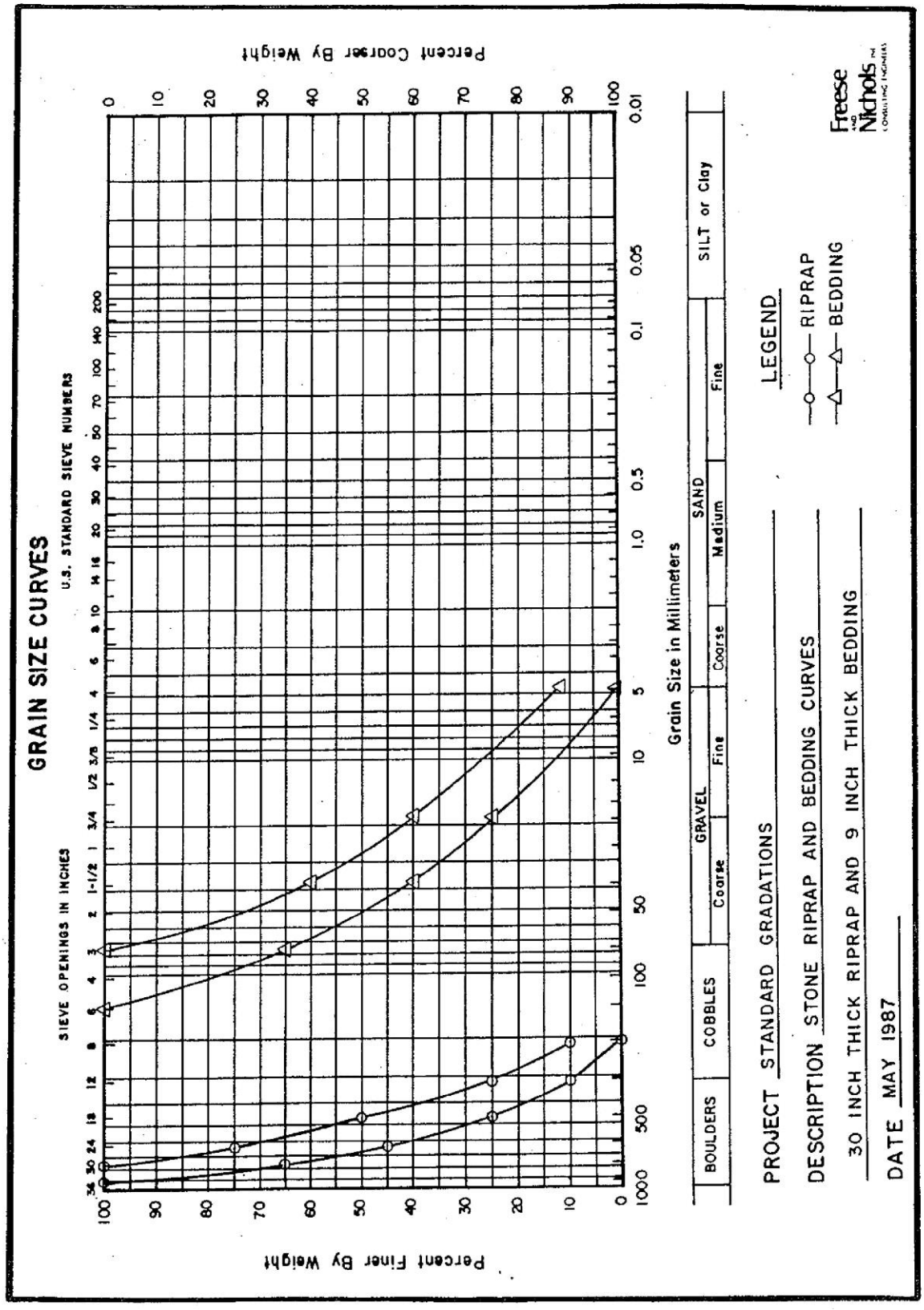


Figure 3.16 Grain Size Curve for 30" Riprap and 9" Bedding

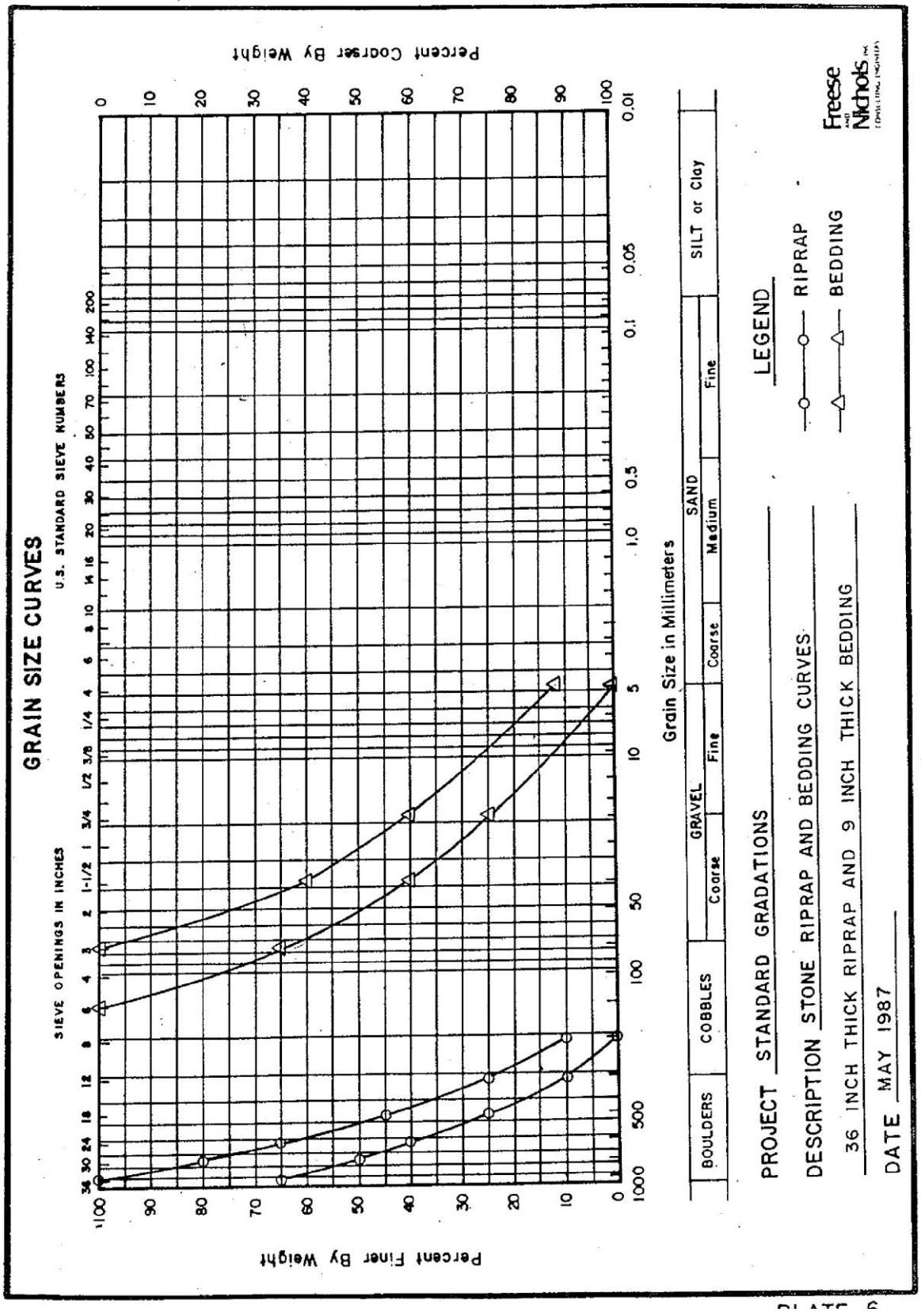
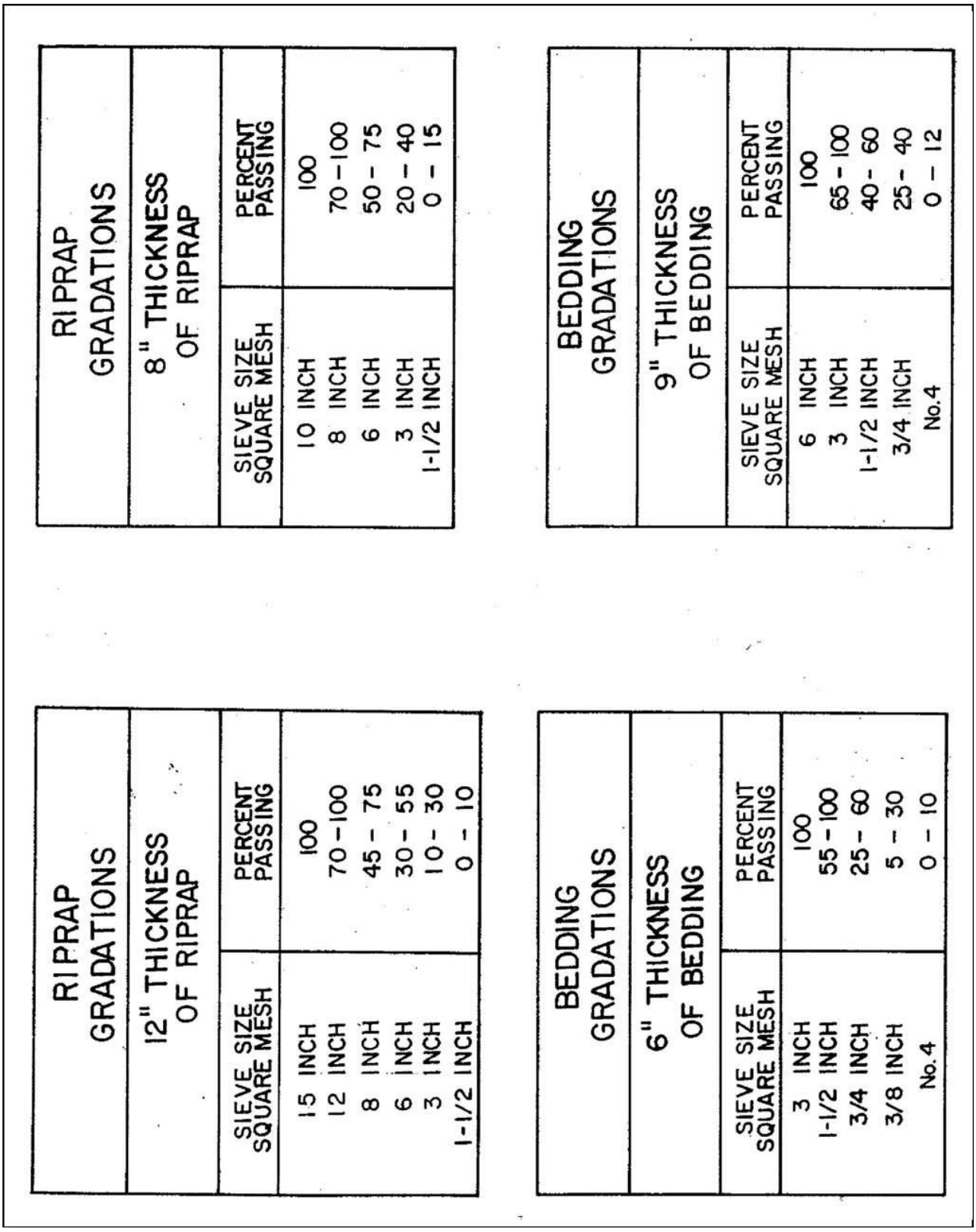


Figure 3.17 Grain Size Curve for 36” Riprap and 9” Bedding



RIPRAP GRADATIONS

12" THICKNESS OF RIPRAP

SIEVE SIZE SQUARE MESH	PERCENT PASSING
15 INCH	100
12 INCH	70 – 100
8 INCH	45 – 75
6 INCH	30 – 55
3 INCH	10 – 30
1-1/2 INCH	0 – 10

BEDDING GRADATIONS

6" THICKNESS OF BEDDING

SIEVE SIZE SQUARE MESH	PERCENT PASSING
3 INCH	100
1-1/2 INCH	55 – 100
3/4 INCH	25 – 60
3/8 INCH	5 – 30
No.4	0 – 10

RIPRAP GRADATIONS

8" THICKNESS OF RIPRAP

SIEVE SIZE SQUARE MESH	PERCENT PASSING
10 INCH	100
8 INCH	70 – 100
6 INCH	50 – 75
3 INCH	20 – 40
1-1/2 INCH	0 – 15

BEDDING GRADATIONS

9" THICKNESS OF BEDDING

SIEVE SIZE SQUARE MESH	PERCENT PASSING
6 INCH	100
3 INCH	65 – 100
1-1/2 INCH	40 – 60
3/4 INCH	25 – 40
No.4	0 – 12

Figure 3.18 Riprap Gradation Tables for 6", 8", 9", and 12" Thickness of Riprap

RIPRAP GRADATIONS	
30" THICKNESS OF RIPRAP	
SIEVE SIZE SQUARE MESH	PERCENT PASSING
36 INCH	100
30 INCH	65 – 100
24 INCH	45 – 75
18 INCH	25 – 50
12 INCH	10 – 25
8 INCH	0 – 10

RIPRAP GRADATIONS	
18" THICKNESS OF RIPRAP	
SIEVE SIZE SQUARE MESH	PERCENT PASSING
21 INCH	100
18 INCH	65 – 100
12 INCH	35 – 65
8 INCH	15 – 40
6 INCH	5 – 25
4 INCH	0 – 15

RIPRAP GRADATIONS	
36" THICKNESS OF RIPRAP	
SIEVE SIZE SQUARE MESH	PERCENT PASSING
44 INCH	100
36 INCH	65 – 100
30 INCH	50 – 80
18 INCH	25 – 45
12 INCH	10 – 25
8 INCH	0 – 10

RIPRAP GRADATIONS	
24" THICKNESS OF RIPRAP	
SIEVE SIZE SQUARE MESH	PERCENT PASSING
30 INCH	100
24 INCH	65 – 100
18 INCH	45 – 75
12 INCH	25 – 50
8 INCH	10 – 30
6 INCH	0 – 15

Figure 3.19 Riprap Gradation Tables for 18”, 24”, 30”, and 36” Thickness of Riprap

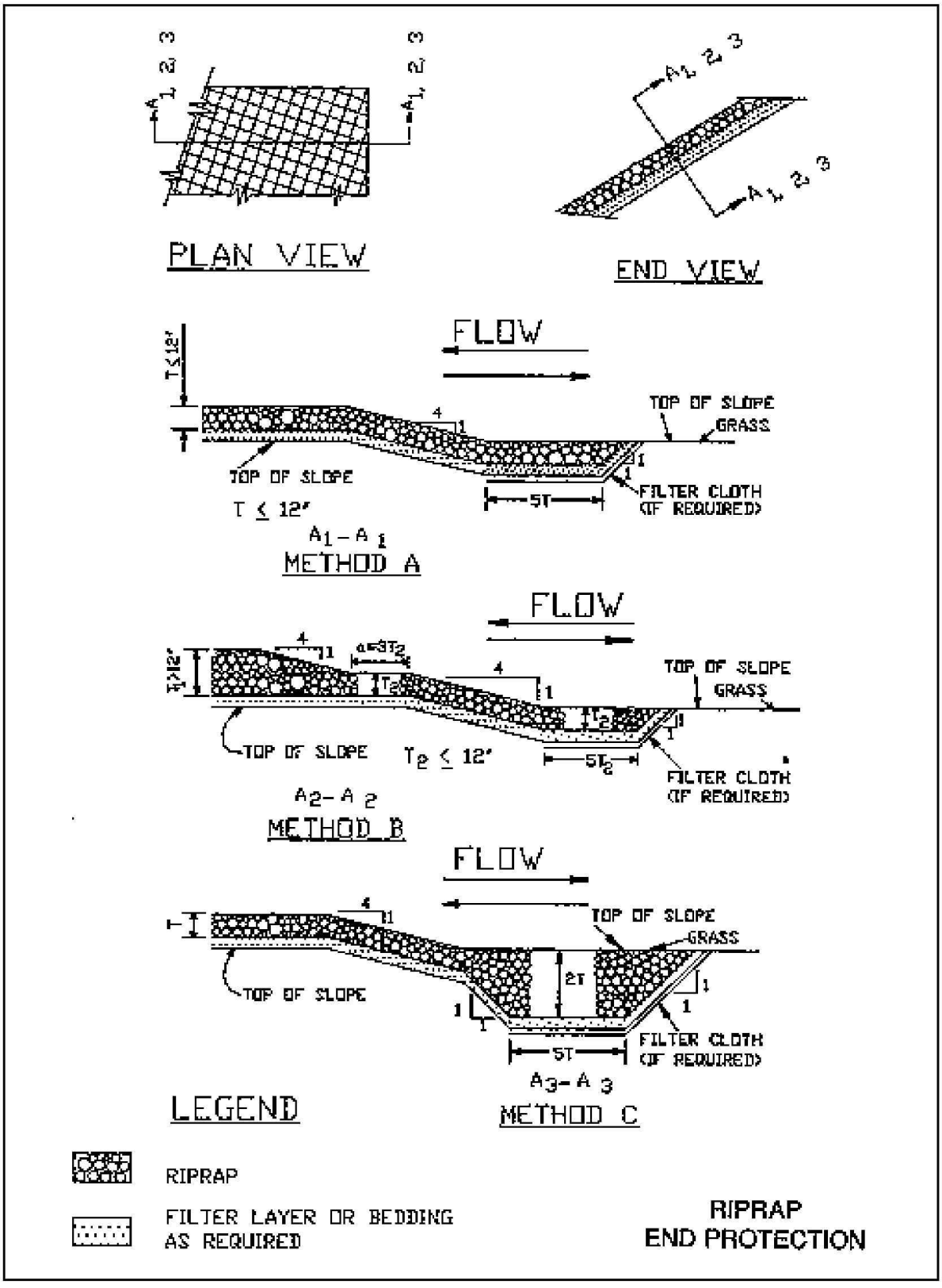


Figure 3.20 Typical Riprap Design Cross Sections

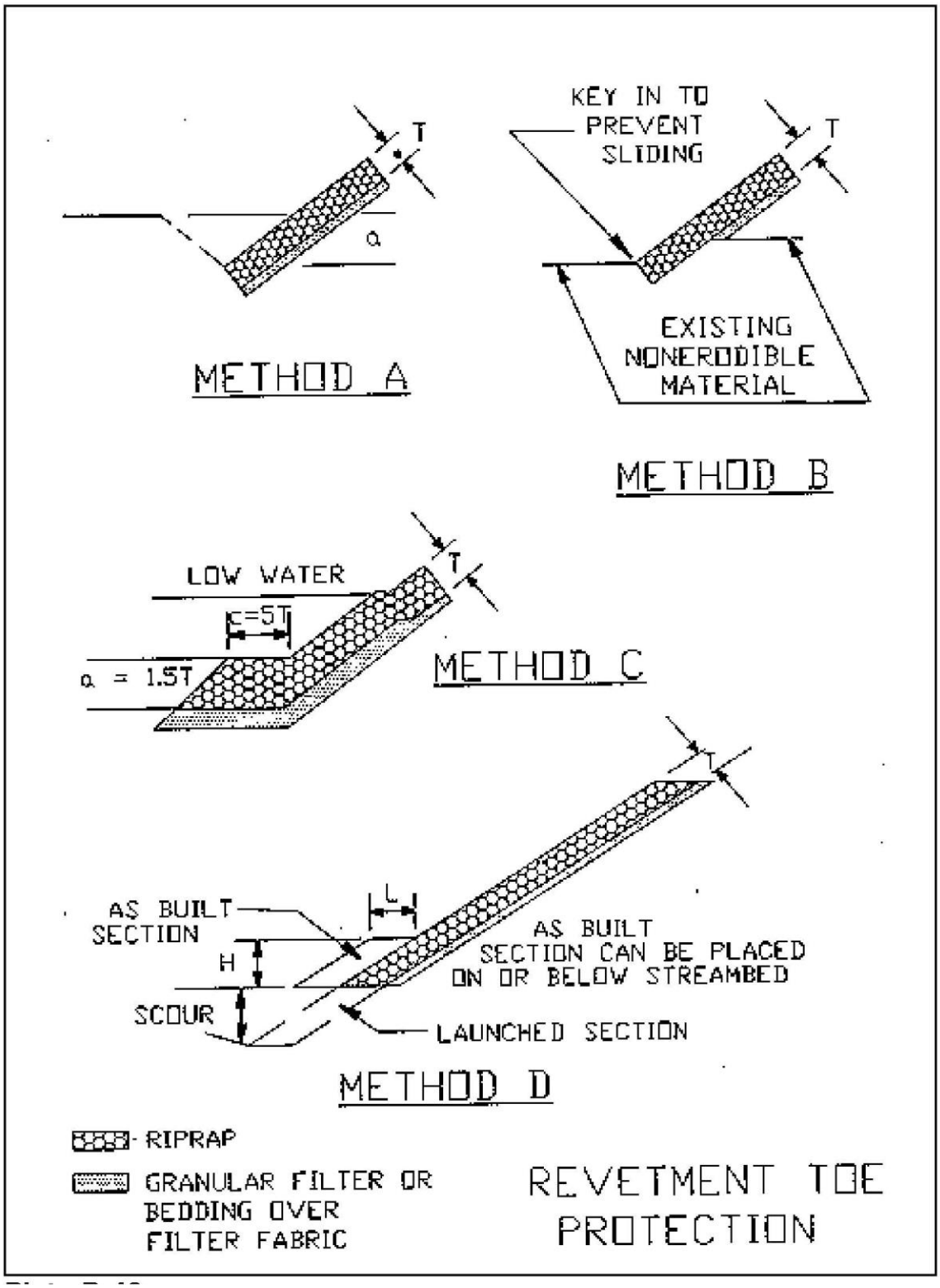


Figure 3.21 Typical Riprap Design for Revetment Toe Protection

3.2.8 Gabion Design

Introduction

Gabions come in three basic forms, the gabion basket, gabion mattress, and sack gabion. All three types consist of wire mesh baskets filled with cobble or small boulder material. The fill normally consists of rock material but other materials such as bricks have been used to fill the baskets. The baskets are used to maintain stability and to protect streambanks and beds.

The difference between a gabion basket and a gabion mattress is the thickness and the aerial extent of the basket. A sack gabion is, as the name implies, a mesh sack that is filled with rock material. The benefit of gabions is that they can be filled with rocks that would individually be too small to withstand the erosive forces of the stream. The gabion mattress is shallower (0.5 to 1.5 ft) than the basket and is designed to protect the bed or banks of a stream against erosion.

Gabion baskets are normally much thicker (about 1.5 to 3 ft) and cover a much smaller area. They are used to protect banks where mattresses are not adequate or are used to stabilize slopes (Figure 3.22), construct drop structures, pipe outlet structures, or nearly any other application where soil must be protected from the erosive forces of water. References to gabions in this manual refer generally to both mattresses and baskets. Sack gabions are rarely used in the United States and are not discussed.



Gabion baskets can be made from either welded or woven wire mesh. The wire is normally galvanized to reduce corrosion but may be coated with plastic or other material to prevent corrosion and/or damage to the wire mesh containing the rock fill. New materials such as Tensar, a heavy-duty polymer plastic material, have been used in some applications in place of the wire mesh. If the wire baskets break, either through corrosion, vandalism, or damage from debris or bed load, the rock fill in the basket can be lost and the protective value of the method endangered. Gabions are often used where available rock size is too small to withstand the erosive and tractive forces present at a project site. The available stone size may be too small due to the cost of transporting larger stone from remote sites, or the desire to have

Figure 3.22 Gabion Baskets Installed for Slope Stabilization along a Stream

a project with a smoother appearance than obtained from riprap or other methods. Gabions also require about one third the thickness of material when compared to riprap designs. Riprap is often preferred, however, due to the low labor requirements for its placement.

The science behind gabions is fairly well established, with numerous manufacturers providing design methodology and guidance for their gabion products. Dr. Stephen T. Maynord of the U.S. Army Corps of Engineers Research and Development Center in Vicksburg, Mississippi, has also conducted research to develop design guidance for the installation of gabions. Two general methods are typically used to determine the stability of gabion baskets in stream channels, the critical shear stress calculation and the critical velocity calculation. A software package known as CHANLPRO has been developed by Dr. Maynord (Maynord et al. 1998).

Manufacturers have generated extensive debate regarding the use and durability of welded wire baskets versus woven wire baskets in project design and construction. Project results seem to indicate that performance is satisfactory for both types of mesh.

The rocks contained within the gabions provide substrates for a wide variety of aquatic organisms. Organisms that have adapted to living on and within the rocks have an excellent home, but vegetation may be difficult to establish unless the voids in the rocks contained within the baskets are filled with soil or a planting bed mixture.

If large woody vegetation is allowed to grow in the gabions, there is a risk that the baskets will break when the large woody vegetation is uprooted or as the root and trunk systems grow. Thus, it is normally not acceptable to allow large woody vegetation to grow in the baskets. The possibility of damage must be weighed against the desirability of vegetation on the area protected by gabions and the stability of the large woody vegetation. If large woody vegetation is kept out of the baskets, grasses and other desirable vegetation types may be established and provide a more aesthetic and ecologically desirable project than gabions alone.

Design

Primary design considerations for gabions and mattresses are: 1) foundation stability; 2) sustained velocity and shear-stress thresholds that the gabions must withstand; and 3) toe and flank protection. The base layer of gabions should be placed below the expected maximum scour depth. Alternatively, the toe can be protected with mattresses that will fall into any scoured areas without compromising the stability of the bank or bed protection portion of the project. If bank protection does not extend above the expected water surface elevation for the design flood, measures such as tiebacks to protect against flanking should be installed.

The use of a filter fabric behind or under the gabion baskets to prevent the movement of soil material through the gabion baskets is an extremely important part of the design process. This migration of soil through the baskets can cause undermining of the supporting soil structure and failure of the gabion baskets and mattresses.

Primary Design Considerations

The major consideration in the design of gabion structures is the expected velocity at the gabion face. The gabion must be designed to withstand the force of the water in the stream.

Since gabion mattresses are much shallower and more subject to movement than gabion baskets, care should be taken to design the mattresses such that they can withstand the forces applied to them by the water. However, mattresses have been used in application where very high velocities are present and have performed well. But, projects using gabion mattresses should be carefully designed.

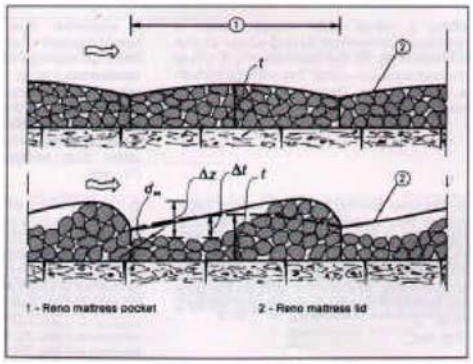
The median stone size for a gabion mattress can be determined from the following equation:

$$d_m = S_f C_s C_v d[(\Upsilon_w/(\Upsilon_s - \Upsilon_w))^{0.5}(V/\sqrt{(gdK_1)})]^{2.5} \qquad (3.23)$$

where:

- d_m = average rock diameter in gabions (ft)
- S_f = safety factor (1.1 minimum)
- C_s = stability coefficient (usually 0.1)
- C_v = velocity distribution coefficient = 1.283-0.2log(r/w) (minimum of 1.0) and equals 1.25 at end of dikes and concrete channels
- r = center-line bend radius of main channel flow (ft)
- w = water surface width of main channel (ft)
- d = local flow depth at V (ft)
- g = acceleration due to gravity (32.2 ft/s^2)
- V = depth-averaged velocity (ft/s)
- K_1 = side slope correction factor (Table 3.8)
- Υ_w = unit weight of water (62.4 lb/ft^3)
- Υ_s = unit weight of stone (lb/ft^3)

Table 3.8 Values of K_1 for various Side Slopes to be used in Equation 3.23	
Side Slope	**K_1**
1V : 1H	0.46
1V : 1.5H	0.71
1V : 2H	0.88
1V : 3H	0.98
<1V : 4H	1.0



Equation 3.23 was developed to design stone size such that the movement of filler stone in the mattresses is prevented. This eliminates deformation that can occur when stone sizes are not large enough to withstand the forces of the water. The result of mattresses deformation is stress on the basket wire and increases in resistance to flow and the likelihood of basket failure. The upper portion of Figure 3.23 shows an undeformed gabion, while the lower portion shows how gabions deform under high-velocity conditions. Maccaferri Gabions gives guidance on sizing stone and allowable velocities for gabion baskets and mattresses, shown in Table 3.9.

Figure 3.23 Gabion Mattress Showing Deformation of Mattress Pockets under High Velocities

Table 3.9 Stone Sizes and Allowable Velocities for Gabions					
Type	**Thickness (ft)**	**Filling Stone Range**	**D_{50}**	**Critical Velocity**	**Limit Velocity**
Mattress	0.5	3 – 4”	3.4”	11.5	13.8
	0.5	3 – 6”	4.3”	13.8	14.8
	0.75	3 – 4”	3.4”	14.8	16
	0.75	3 – 6”	4.7”	14.8	20
	1.0	3 – 5”	4”	13.6	18
	1.0	4 – 6”	5”	16.4	21
Basket	1.5	4 – 8”	6”	19	24.9
	1.5	5 – 10”	7.5”	21	26.2

When the data in Table 3.9 are compared to Equation 3.22, if V = 11.5, C_s = 0.1, C_v = 1.0, K_1 = 0.71, ,Υ_s = 150 lb/ft^3 and S_f = 1.1, the local flow depth must be on the order of 25 ft in order to arrive at the stone diameter of 3.4 in. shown in Table 3.9. Designers should use Equation 3.23 to take the depth of flow into account. Table 3.9 does, however, give some general guidelines for fill sizes and is a quick reference for maximum allowable velocities.

Maccaferri also gives guidance on the stability of gabions in terms of shear stress limits. The following equation gives the shear for the bed of the channel as:

$$\tau_b = \Upsilon_w S d \quad \textbf{(3.24)}$$

where S = bed or water surface slope through the reach (ft/ft)

The bank shear is generally taken as 75 percent of the bed shear, i.e.,

$$\tau_m = 0.75\,\tau_m \quad \textbf{(3.25)}$$

These values are then compared to the critical stress for the bed calculated by the following equation:

$$\tau_c = 0.10(\Upsilon_s - \Upsilon_w) d_m \quad \textbf{(3.26)}$$

with critical shear stress for the banks given as:

$$\tau_s = \tau_c \sqrt{(1-(\sin^2\Theta/0.4304))} \quad \textbf{(3.27)}$$

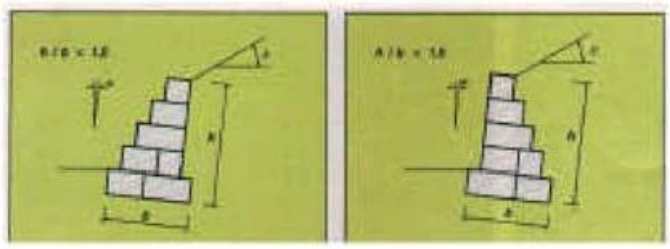
where Θ = angle of the bank rotated up from horizontal.

A design is acceptable if $\tau_b < \tau_c$ and $\tau_m < \tau_s$. If either $\tau_b > \tau_c$ or $\tau_m > \tau_s$ then a check must be made to see if they are less than 120 percent of τ_b and τ_s. If the values are less than 120 percent of τ_b and τ_s the gabions will not be subject to more than what Maccaferri defines as “acceptable” deformation. However it is recommended that the stone size be increased to limit deformation if possible.

Research has indicated that stone in the gabion mattress should be sized such that the largest stone diameter is not more than about two times the diameter of the smallest stone diameter and the mattress should be at least twice the depth of the largest stone size. The size range should, however, vary by about a factor of two to ensure proper packing of the stone material into the gabions. Since the mattresses normally come in discrete sizes, i.e. 0.5, 1.0, and 1.5 ft in depth, normal practice is to size the stone and then select the basket depth that is deep enough to be at least two times the largest stone diameter. The smallest stone should also be sized such that it cannot pass through the wire mesh.

Stability of Underlying Bed and Bank Material

Another critical consideration is the stability of the gabion foundation. This includes both geotechnical stability and the resistance of the soil under the gabions to the erosive forces of the water moving through the gabions. If there is any question regarding the stability of the foundation, i.e. possibility of rotational failures, slip failures, etc., a qualified geotechnical engineer should be consulted prior to and during the design of the bank/channel protection. Several manufacturers give guidance on how to check for geotechnical failure.



One of the critical factors in determining stability is the velocity of the water that passes through the gabions and reaches the soil behind the gabion. The water velocity under the filter fabric, i.e. water that moves through the gabions and filter fabric, is estimated to be one-fourth to one-half of the velocity at the mattress/filter interface.

Figure 3.24 Front-step and Rear-step Gabion Layout

The velocity at the mattress/filter interface, V_b, is estimated to be

$$V_b = 1.486\sqrt{S}(d_m/2)^{2/3}/n_f \quad (3.28)$$

where n_f = 0.02 for filter fabric, 0.022 for gravel filter material

If the underlying soil material is not stable, additional filter material must be installed under the gabions to ensure soil stability.

The limit for velocity on the soil is different for each type of soil. The limit for cohesive soils is obtained from a chart, while maximum allowable velocities for other soil types are obtained by calculating V_e, the maximum velocity allowable at the soil interface, and comparing it to V" the residual velocity on the bed, i.e. under the gabion mattress and under the filter fabric. V_e for loose soils is equal to $16.1d^2$ while V_f is calculated by:

$$V_f = 1.486S\sqrt{V_a}(d_m/2)^{2/3}/n_f \quad (3.29)$$

where V_a = average channel velocity (ft/s)

If V_f is larger than two to four times V_e, a gravel filter is required to further reduce the water velocity at the soil interface under the gabions until V_f is in an acceptable range. To check for the acceptability of the filter use the average gravel size for d_m in Equation 5.28. If the velocity V_f is still too high, the gravel size should be reduced to obtain an acceptable value for V_f.

Other Design Considerations

It may be possible to combine gabions with less harsh methods of bank protection on the upper bank and still achieve the desired result of a stable channel. Provisions for large woody vegetation and a more aesthetically pleasing project may also be used on the upper banks or within the gabions. However, the stability of vegetation or other upper bank protection should be carefully analyzed to ensure stability of the upper bank area. A failure in the upper bank region can adversely affect gabion stability and lead to project failure.

3.2.9 Uniform Flow – Example Problems

Example 1 -- Direct Solution of Manning's Equation

Use Manning's Equation to find the velocity, v, for an open channel with a hydraulic radius value of 0.6 ft, an n value of 0.020, and slope of 0.003 ft/ft. Solve using Figure 3.3:

- Connect a line between the slope scale at 0.003 and the roughness scale at 0.020 and note the intersection point on the turning line.
- Connect a line between that intersection point and the hydraulic radius scale at 0.6 ft and read the velocity of 2.9 ft/s from the velocity scale.

Example 2 -- Grassed Channel Design Stability

A trapezoidal channel is required to carry 50 cfs at a bottom slope of 0.015 ft/ft. Find the channel dimensions required for design stability criteria (retardance Class D) for a grass mixture.

From Table 3.3, the maximum velocity, v_m, for a grass mixture with a bottom slope less than 5% is 4 ft/s.

Assume an n value of 0.035 and find the value of vR from Figure 3.2, vR = 5.4

- Use Equation 3.9 to calculate the value of R: R = 5.4/4 = 1.35 ft
- Use Equation 3.10 to calculate the value of vR:
  $vR = [1.49\ (1.35)^{5/3}\ (0.015)^{1/2}]/0.035 = 8.60$

- Since the vR value calculated in Step 4 is higher than the value obtained from Step 2, a higher n value is required and calculations are repeated. The results from each trial of calculations are presented below:

Assumed n Value	vR (Figure 3.2)	R (Eq. 3.9)	vR (Eq. 3.10)
0.035	5.40	1.35	8.60
0.038	3.8	0.95	4.41
0.039	3.4	0.85	3.57
0.040	3.2	0.80	3.15

Select n = 0.040 for stability criteria.

- Use Figure 3.4 to select channel dimensions for a trapezoidal shape with 3:1 side slopes.

  Qn = (50) (0.040) = 2.0, S = 0.015
  For b = 10 ft, d = (10) (0.098) = 0.98 ft, b = 8 ft, d = (8) (0.14) = 1.12 ft

  Select:

  b = 10 ft, such that R is approximately 0.80 ft
  z = 3
  d = 1 ft
  v = 3.9 ft/s (Equation 3.1)
  Fr = 0.76 (Equation 3.8)
  Flow is subcritical

  Design capacity calculations for this channel are presented in Example 3 below.

Example 3 -- Grassed Channel Design Capacity

Use a 10-ft bottom width and 3:1 side-slopes for the trapezoidal channel sized in Example 2 and find the depth of flow for retardance Class C.

- Assume a depth of 1.0 ft and calculate the following (see Figure 3.6):

  $A = (b + zd)\, d = [10 + (3)\,(1)]\,(1) = 13.0$ square ft

  $R = [(b + zd)\, d]/\{b + [2d(1 + z^2)^{0.5}]\} = \{[10+(3)(1)]1\}/\{10+[(2)(1)(1+3^2)^{0.5}]\}$

  $R = 0.796$ ft

- Find the velocity: $v = Q/A = 50/13.0 = 3.85$ ft/s
- Find the value of vR: $vR = (3.85)\,(0.796) = 3.06$
- Using the vR product from Step 3, find Manning's n from Figure 3.2 for retardance Class C (n = 0.047)
- Use Figure 3.3 or Equation 3.1 to find the velocity for S = 0.015, R = 0.796, and n = 0.047: <u>v = 3.34 ft/s</u>
- Since 3.34 ft/s is less than 3.85 ft/s, a higher depth is required and calculations are repeated. Results from each trial of calculations are presented below:

Assumed Depth (ft)	Area (ft^2)	R (ft)	Velocity Q/A (ft/sec)	vR	Manning's n (Fig. 5.2)	Velocity (Eq. 5.1)
1.0	13.00	0.796	3.85	3.06	0.047	3.34
1.05	13.81	0.830	3.62	3.00	0.047	3.39
1.1	14.63	0.863	3.42	2.95	0.048	3.45
1.2	16.32	0.928	3.06	2.84	0.049	3.54

- Select a depth of 1.1 with an n value of 0.048 for design capacity requirements. Add at least 0.2 ft for freeboard to give a design depth of 1.3 ft. Design data for the trapezoidal channel are summarized as follows:

  Vegetation lining = grass mixture, v_m = 4 ft/s

  Q = 50 cfs

  b = 10 ft, d = 1.3 ft, z = 3, S = 0.015
  Top width = (10) + (2) (3) (1.3) = 17.8 ft

  n (stability) = 0.040, d = 1.0 ft, v = 3.9 ft/s, Froude number = 0.76 (Equation 3.8)
  n (capacity) = 0.048, d = 1.1 ft, v = 3.45 ft/s, Froude number = 0.64 (Equation 3.8)

Example 4 -- Riprap Design

A natural channel has an average bankfull channel velocity of 8 ft per second with a top width of 20 ft and a bend radius of 50 ft. The depth over the toe of the outer bank is 5 ft. Available stone weight is 170 lbs/ft^3. Stone placement is on a side slope of 2:1 (horizontal:vertical). Determine riprap size at the outside of the bend.

- Use 8 ft/s as the design velocity, because the reach is short and the bend is not protected.
- Determine the bend correction coefficient for the ratio of R_b/T = 50/20 = 2.5. From Figure 3.8, C_b = 1.55. The adjusted effective velocity is (8) (1.55) = 12.4 ft/s.
- Determine the correction coefficient for the specific weight of 170 lbs from Figure 3.9 as 0.98. The adjusted effective velocity is (12.4) (0.98) = 12.15 ft/s.
- Determine minimum d_{30} from Figure 3.10 or Equation 3.13 as about 10 inches.
- Use a gradation with a minimum d_{30} size of 10 inches.
- *(Optional)* Another gradation is available with a d_{30} of 8 inches. The ratio of desired to standard stone size is 8/10 or 0.8. From Figure 3.11, this gradation would be acceptable if the blanket thickness was increased from the original d_{100} (diameter of the largest stone) thickness by 35% (a ratio of 1.35 on the horizontal axis).
- Perform preliminary design. Make sure that the stone is carried up and downstream far enough to ensure stability of the channel and that the toe will not be undermined. The downstream length of protection for channel bends can be determined using Figure 3.7.

3.2.10 Gradually Varied Flow

The most common occurrence of gradually varied flow in storm drainage is the backwater created by culverts, storm sewer inlets, or channel constrictions. For these conditions, the flow depth will be greater than normal depth in the channel and the water surface profile should be computed using backwater techniques.

Many computer programs are available for computation of backwater curves. The most general and widely used programs are, HEC-RAS, developed by the U.S. Army Corps of Engineers and Bridge

Waterways Analysis Model (WSPRO) developed for the Federal Highway Administration. These programs can be used to compute water surface profiles for both natural and artificial channels.

For prismatic channels, the backwater calculation can be computed manually using the Direct Step method (Chow, 1959, TxDOT, 2002). In the Direct Step method an increment of water depth is chosen, and the distance over which the depth change occurs is computed. This method is often used in association with culvert hydraulics. It is most accurate when the slope and depth increments are small. It is appropriate for prismatic channel sections which occur in most conduits, and can be useful when estimating both supercritical and subcritical profiles. For supercritical flow, the water surface profile is computed downstream. For subcritical flow, the water surface profile is computed upstream.

For an irregular nonuniform channel, the Standard Step method is recommended, although it is a more tedious and iterative process. The use of HEC-RAS is recommended for Standard Step calculations.

Cross sections for water surface profile calculations should be normal to the direction of flood flow. The number of sections required will depend on the irregularity the designed waterway. Channel cross sections will be required at each location along the waterway where there are changes in channel shape or dimension, changes in the flowline slope, and changes in vegetation or channel lining. These sections are in addition to any sections necessary to define obstructions such as culverts, bridges, damns, energy dissipation features, or aerial crossings (pipelines). Sections should usually be no more than 4 to 5 channel widths apart or 100 feet apart for ditches or streams and 500 feet apart for floodplains, unless the channel is very regular.

3.2.11 Rectangular, Triangular and Trapezoidal Open Channel Design

Introduction

The Federal Highway Administration has prepared numerous design figures to aid in the design of open channels. Copies of these figures, a brief description of their use, and several example design problems are presented. For design conditions not covered by the figures, a trial and error solution of Manning's Equation must be used. However, it is anticipated that available software programs will be the first choice for solving these design computations.

Description of Figures

Figures given in FHWA, HDS No. 3, 1973 and Atlanta Regional Commission, 2001 are for the direct solution of the Manning's Equation for various sized open channels with rectangular, triangular, and trapezoidal cross sections. Each figure (except for the triangular cross section) is prepared for a channel of given bottom width and a particular value of Manning's n.

The figures for rectangular and trapezoidal cross section channels are used the same way. The abscissa scale of discharge in cubic feet per second (cfs), and the ordinate scale is velocity in feet per second (ft/s). Both scales are logarithmic. Superimposed on the logarithmic grid are steeply inclined lines representing depth (ft), and slightly inclined lines representing channel slope (ft/ft). A heavy dashed line on each figure shows critical flow conditions. Auxiliary abscissa and ordinate scales are provided for use with other values of n and are explained in the example problems. In the figures, interpolations may be made not only on the ordinate and abscissa scales but also between the inclined lines representing depth and slope.

The chart for a triangular cross section (see Figure 1.2) is in nomograph form. It may be used for street sections with a vertical (or nearly vertical) curb face. The nomograph also may be used for shallow V-shaped sections by following the instructions on the chart.

Instructions for Rectangular and Trapezoidal Figures

Figures such as Figure 3.25 provide a solution of the Manning equation for flow in open channels of uniform slope, cross section, and roughness, provided the flow is not affected by backwater and the channel has a length sufficient to establish uniform flow.

For a given slope and channel cross section, when n is 0.015 for rectangular channels or 0.03 for trapezoidal channels, the depth and velocity of uniform flow may be read directly from the figure for that size channel. The initial step is to locate the intersection of a vertical line through the discharge (abscissa) and the appropriate slope line. At this intersection, the depth of flow is read from the depth lines, and the mean velocity is read on the ordinate scale.

The procedure is reversed to determine the discharge at a given depth of flow. Critical depth, slope, and velocity for a given discharge can be read on the appropriate scale at the intersection of the critical curve and a vertical line through the discharge.

Auxiliary scales, labeled Qn (abscissa) and Vn (ordinate), are provided so the figures can be used for values of n other than those for which the charts were basically prepared. To use these scales, multiply the discharge by the value of n and use the Qn and Vn scales instead of the Q and V scales, except for computation of critical depth or critical velocity. To obtain normal velocity V from a value on the Vn scale, divide the value by n. The following examples will illustrate these points.

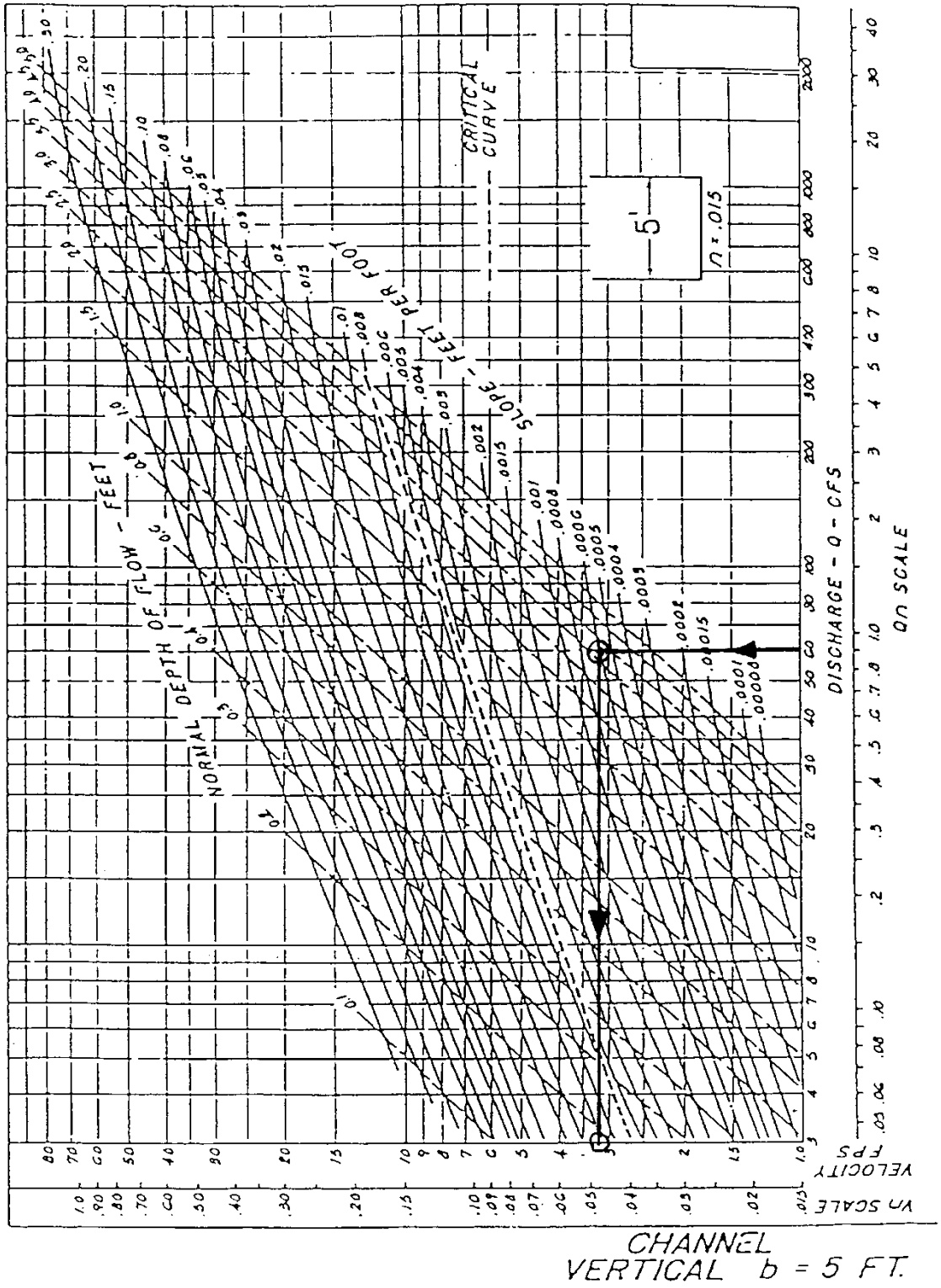
Example Design Problem 1

Given: A rectangular concrete channel 5 ft wide with n = 0.015, .06 percent slope (S = .0006), discharging 60 cfs.

Find: Depth, velocity, and type of flow

Procedure:

1. From *Section 3.2.11*, select the rectangular figure for a 5-ft width (Figure 3.25).
2. From 60 cfs on the Q scale, move vertically to intersect the slope line S = .0006, and from the depth lines read d_n = 3.7 ft.
3. Move horizontally from the same intersection and read the normal velocity, V = 3.2 ft/s, on the ordinate scale.
4. The intersection lies below the critical curve, and the flow is therefore in the subcritical range.



Source: Federal Highway Administration

Figure 3.25 Example Nomograph #1

Example Design Problem 2

Given: A trapezoidal channel with 2:1 side slopes and a 4 ft bottom width, with n = 0.030, 0.2% slope (S = 0.002), discharging 50 cfs.

Find: Depth, velocity, type flow.

Procedure:

1. Select the trapezoidal figure for b = 4 ft (see Figure 3.26).
2. From 50 cfs on the Q scale, move vertically to intersect the slope line S = 0.002 and from the depth lines read d_n = 2.2 ft.
3. Move horizontally from the same intersection and read the normal velocity, V = 2.75 ft/s, on the ordinate scale. The intersection lies below the critical curve, and the flow is therefore subcritical.

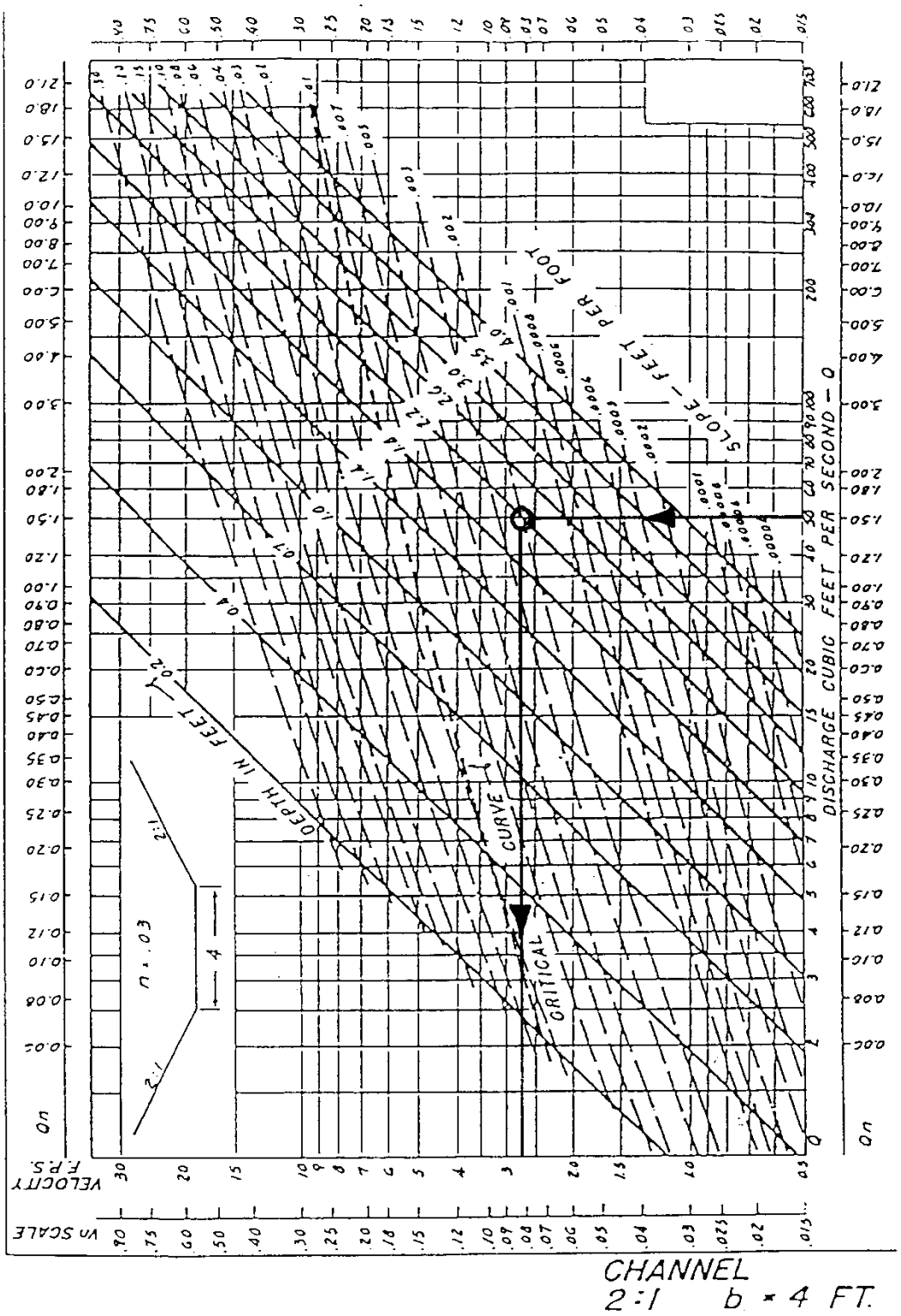
Example Design Problem 3

Given: A rectangular cement rubble masonry channel 5 ft wide, with n = 0.025, 0.5% slope (S = 0.005), discharging 80 cfs.

Find: Depth velocity and type of flow

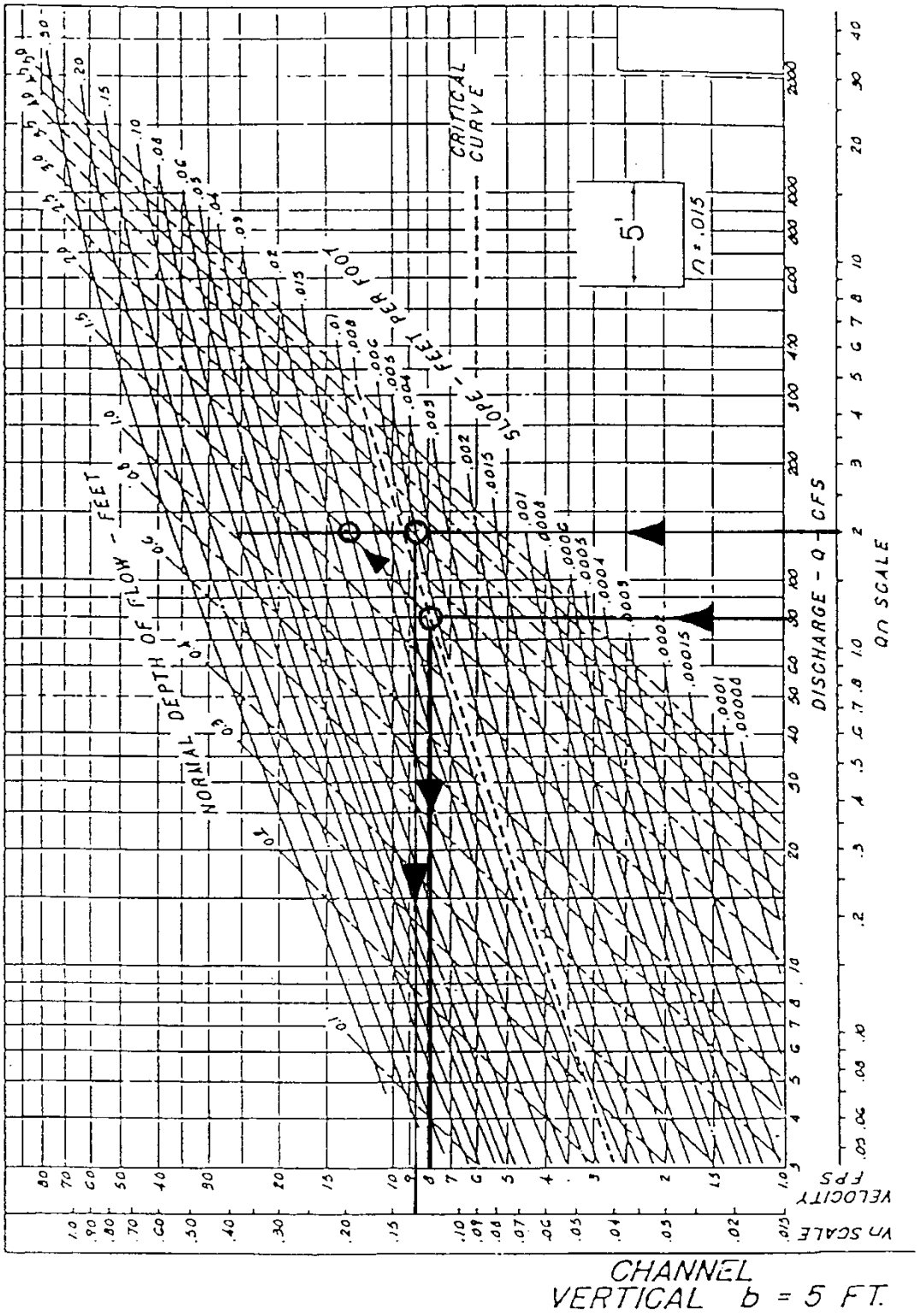
Procedure:

1. Select the rectangular figure for a 5 ft width (Figure 3.27).
2. Multiply Q by n to obtain Qn: 80 x 0.025 = 2.0.
3. From 2.0 on the Qn scale, move vertically to intersect the slope line, S = 0.005, and at the intersection read d_n = 3.1 ft.
4. Move horizontally from the intersection and read Vn = .13, then Vn/n = 0.13/0.025 = 5.2 ft/s.
5. Critical depth and critical velocity are independent of the value of n so their values can be read at the intersection of the critical curve with a vertical line through the discharge. For 80 cfs, on Figure 3.14, d_c = 2.0 ft and V_c = 7.9 ft/s. The normal velocity, 5.2 ft/s (from step 4), is less than the critical velocity, and the flow is therefore subcritical. It will also be noted that the normal depth, 3.0 ft, is greater than the critical depth, 2.0 ft, which also indicates subcritical flow.
6. To determine the critical slope for Q = 80 cfs and n = 0.025, start at the intersection of the critical curve and a vertical line through the discharge, Q = 80 cfs, finding d_c (2.0 ft) at this point. Follow along this d_c line to its intersection with a vertical line through Qn = 2.0 (step 2), at this intersection read the slope value S_c = 0.015.



Source: Federal Highway Administration

Figure 3.26 Example Nomograph #2



Source: Federal Highway Administration

Figure 3.27 Example Nomograph #3

3.2.12 Grassed Channel Figures

The Manning equation can be used to determine the capacity of a grass-lined channel, but the value of n varies with the type of grass, development of the grass cover, depth, and velocity of flow. The variable value of n complicates the solution of the Manning equation. The depth and velocity of flow must be estimated and the Manning equation solved using the n value that corresponds to the estimated depth and velocity. The trial solution provides better estimates of the depth and velocity for a new value of n and the equation is again solved. The procedure is repeated until a depth is found that carries the design discharge.

To prevent excessive erosion, the velocity of flow in a grass-lined channel must be kept below some maximum value (referred to as permissible velocity). The permissible velocity in a grass-lined channel depends upon the type of grass, condition of the grass cover, texture of the soil comprising the channel bed, channel slope, and to some extent the size and shape of the drainage channel. To guard against overtopping, the channel capacity should be computed for taller grass than is expected to be maintained, while the velocity used to check the adequacy of the protection should be computed assuming a lower grass height than will likely be maintained.

To aid in the design of grassed channels, the Federal Highway Administration has prepared numerous design figures. Copies of these figures are in *Section 3.2.11*. Following is a brief description of general design criteria, instructions on how to use the figures, and several example design problems. For design conditions not covered by the figures, a trial-and-error solution of the Manning equation must be used.

Description of Figures

A set of figures in FHWA, NDS No. 3, 1973 and Atlanta Regional Commission, 2001 are designed for use in the direct solution of the Manning equation for various channel sections lined with grass. The figures are similar in appearance and use to those for trapezoidal cross sections described earlier. However, their construction is much more difficult because the roughness coefficient (n) changes as higher velocities and/or greater depths change the condition of the grass. The effect of velocity and depth of flow on n is evaluated by the product of velocity and hydraulic radius V times R. The variation of Manning's n with the retardance (Table 3.6) and the product V times R is shown in Figure 3.2. As indicated in Table 3.6, retardance varies with the height of the grass and the condition of the stand. Both of these factors depend upon the type of grass, planting conditions, and maintenance practices. Table 3.6 is used to determine retardance classification.

The grassed channel figures each have two graphs, the upper graph for retardance Class D and the lower graph for retardance Class C. The figures are plotted with discharge in cubic feet per second on the abscissa and slope in feet per foot on the ordinate. Both scales are logarithmic.

Superimposed on the logarithmic grid are lines for velocity in feet per second and lines for depth in feet. A dashed line shows the position of critical flow.

Instructions for Grassed Channel Figures

The grassed channel figures like those in Figure 3.12 provide a solution of the Manning equation for flow in open grassed channels of uniform slope and cross section. The flow should not be affected by backwater and the channel should have length sufficient to establish uniform flow. The figures are sufficiently accurate for design of drainage channels of fairly uniform cross section and slope, but are not appropriate for irregular natural channels.

The design of grassed channels requires two operations: (1) selecting a section that has the capacity to carry the design discharge on the available slope and (2) checking the velocity in the channel to ensure that the grass lining will not be eroded. Because the retardance of the channel is largely beyond the control of the designer, it is good practice to compute the channel capacity using retardance Class C and the velocity using retardance Class D. The calculated velocity should then be checked against the

permissible velocities listed in Tables 3.2 and 3.3. The use of the figures is explained in the following steps:

Step 1 Select the channel cross section to be used and find the appropriate figure.

Step 2 Enter the lower graph (for retardance Class C) on the figure with the design discharge value on the abscissa and move vertically to the value of the slope on the ordinate scale. As this intersection, read the normal velocity and normal depth and note the position of the critical curve. If the intersection point is below the critical curve, the flow is subcritical; if it is above, the flow is supercritical.

Step 3 To check the velocity developed against the permissible velocities (Tables 3.2 and 3.3), enter the upper graph on the same figure and repeat Step 2. Then compare the computed velocity with the velocity permissible for the type of grass, channel slope, and erosion resistance of the soil. If the computed velocity is less, the design is acceptable. If not, a different channel section must be selected and the process repeated.

Example Design Problem 1

Given: A trapezoidal channel in easily eroded soil, lined with a grass mixture with 4:1 side slopes, and a 4 ft bottom width on slope of 0.02 ft per foot (S=.02), discharging 20 cfs.

Find: Depth, velocity, type of flow, and adequacy of grass to prevent erosion

Procedure:

1. Select figure for 4:1 side slopes (see Figure 3.28).
2. Enter the lower graph with Q = 20 cfs, and move vertically to the line for S=0.02. At this intersection read d_n = 1.0 ft, and normal velocity V_n 2.6 ft/s.
3. The velocity for checking the adequacy of the grass cover should be obtained from the upper graph, for retardance Class D. Using the same procedure as in step 2, the velocity is found to be 3.0 ft/s. This is about three-quarters of that listed as permissible, 4.0 ft/s in Table 3.3.

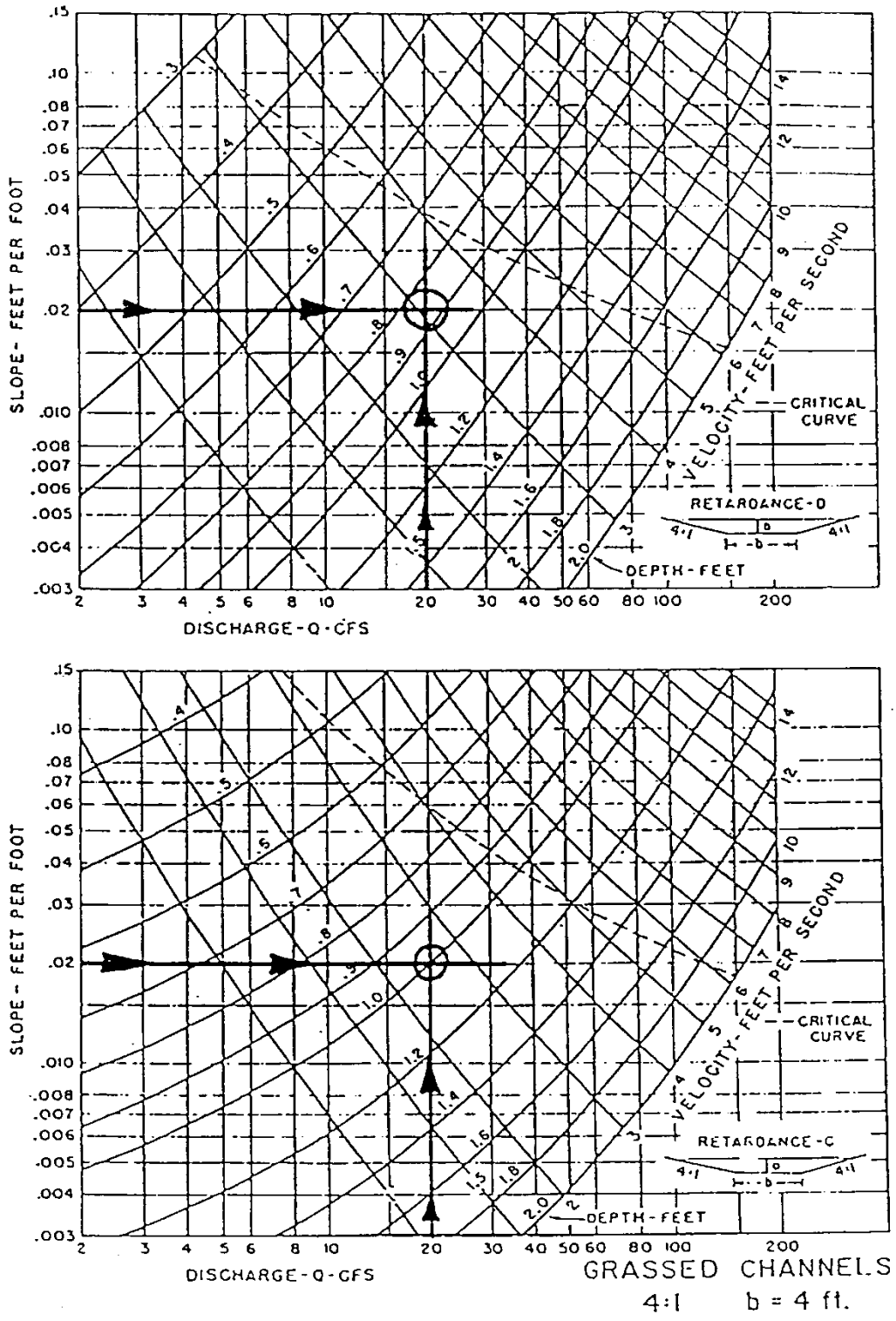
Example Design Problem 2

Given: The channel and discharge of Example 1.

Find: The maximum grade on which the 20 cfs could safely be carried

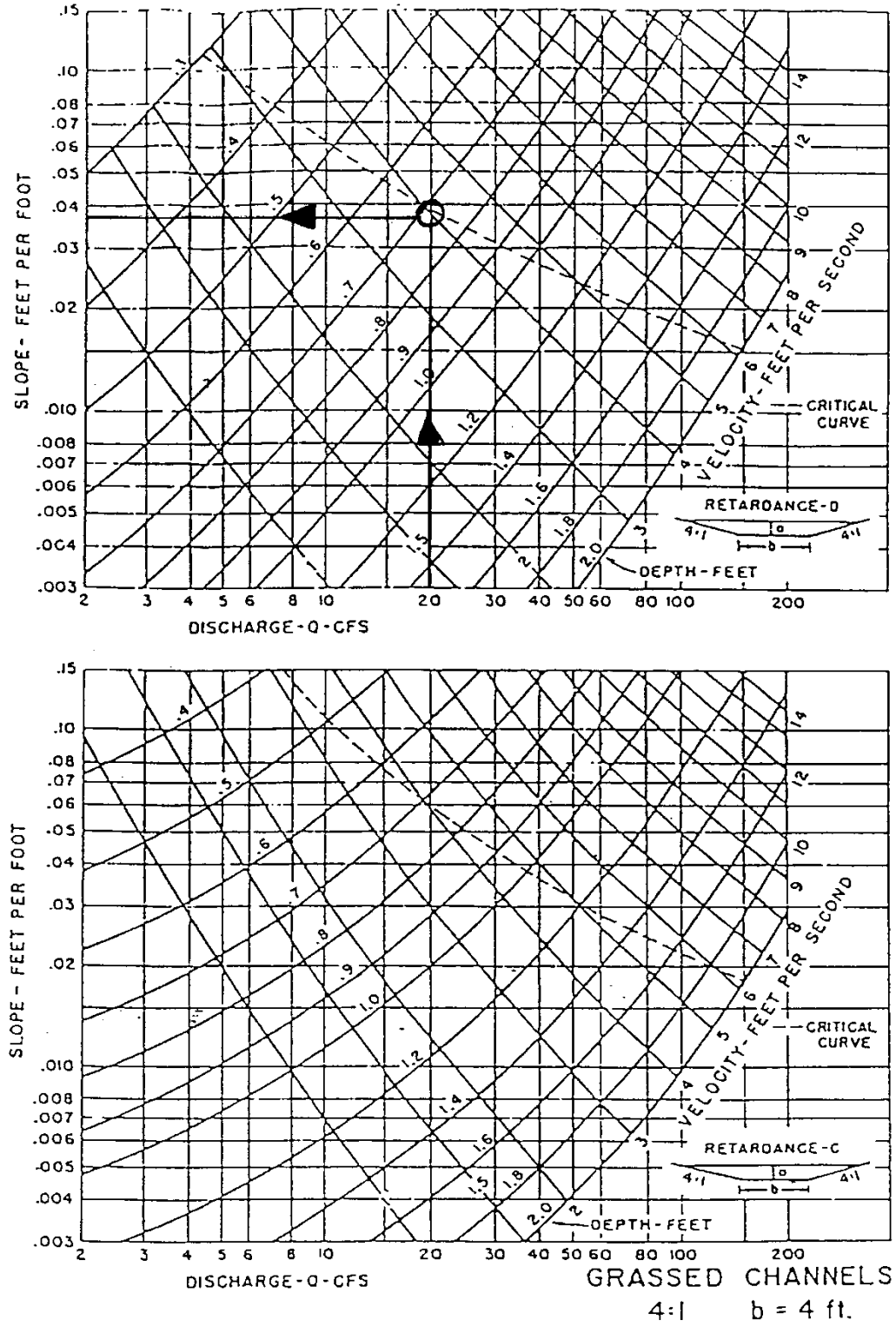
Procedure:

With an increase in slope (but still less than 5%), the allowable velocity is estimated to be 4 ft/s (see Table 3.3). On the upper graph of Figure 3.29 for short grass, the intersection of the 20 cfs line and the 4 ft/s line indicates a slope of 3.7% and a depth of 0.73 ft.



Source: Federal Highway Administration

Figure 3.28 Example Nomograph #4



Source: Federal Highway Administration

Figure 3.29 Example Nomograph #5

3.3 Culvert Design

3.3.1 Overview

A *culvert* is a short, closed (covered) conduit that conveys stormwater runoff under an embankment or away from the street right-of-way. The primary purpose of a culvert is to convey surface water, but properly designed it may also be used to restrict flow and reduce downstream peak flows.

The hydraulic and structural designs of a culvert must be such that minimal risks to traffic, property damage, and failure from floods prove the results of good engineering practice and economics. For economy and hydraulic efficiency, engineers should design culverts to operate with the inlet submerged during flood flows, if conditions permit. Design considerations include site and roadway data, design parameters (including shape, material, and orientation), hydrology (flood magnitude versus frequency relation), and channel analysis (stage versus discharge relation).

3.3.2 Symbols and Definitions

To provide consistency within this section as well as throughout this Manual the symbols listed in Table 3.10 will be used. These symbols were selected because of their wide use.

Table 3.10 Symbols and Definitions

Symbol	Definition	Units
A	Area of cross section of flow	ft^2
B	Barrel width	ft
C_d	Overtopping discharge coefficient	-
D	Culvert diameter or barrel depth	in or ft
d	Depth of flow	ft
d_c	Critical depth of flow	ft
d_u	Uniform depth of flow	ft
g	Acceleration of gravity	ft/s
H_f	Depth of pool or head, above the face section of invert	ft
h_o	Height of hydraulic grade line above outlet invert	ft
HW	Headwater depth above invert of culvert (depth from inlet invert to upstream total energy grade line)	ft
K_e	Inlet loss coefficient	-
L	Length of culvert	ft
N	Number of barrels	-
Q	Rate of discharge	cfs
S	Slope of culvert	ft/f
TW	Tailwater depth above invert of culvert	ft
V	Mean velocity of flow	ft/s
V_c	Critical velocity	ft/s

3.3.3 Design Considerations

The design of a culvert should take into account many different engineering and technical aspects at the culvert site and adjacent areas. The following list of design recommendations should be considered for all culvert designs as applicable. Refer to *Section 3.6.3 of the Criteria Manual* or the local review authority for design criteria details.

- **Frequency Flood**
- **Velocity Limitations**
- **Buoyancy Protection**
    - Headwalls, endwalls, slope paving, or other means of anchoring to provide buoyancy protection should be considered for all flexible culverts.
- **Length and Slope**
- **Debris Control**
    - In designing debris control structures, it is recommended that the Hydraulic Engineering Circular No. 9 entitled Debris Control Structures be consulted.
- **Headwater Limitations**
- **Tailwater Considerations**
- **Storage**
- **Culvert Inlets**
    - Hydraulic efficiency and cost can be significantly affected by inlet conditions. The inlet coefficient Ke, is a measure of the hydraulic efficiency of the inlet, with lower values indicating greater efficiency. Recommended inlet coefficients are given in Table 3.11.
- **Inlets with Headwalls**
    - Headwalls may be used for a variety of reasons, including increasing the efficiency of the inlet, providing embankment stability, providing embankment protection against erosion, providing protection from buoyancy, and shortening the length of the required structure. Headwalls are required for all metal culverts and where buoyancy protection is necessary. If high headwater depths are to be encountered, or the approach velocity in the channel will cause scour, a short channel apron should be provided at the toe of the headwall.
    - This apron should extend at least one pipe diameter upstream from the entrance, and the top of the apron should not protrude above the normal streambed elevation.
- **Wingwalls and Aprons**
    - Wingwalls are used where the side slopes of the channel adjacent to the entrance are unstable or where the culvert is skewed to the normal channel flow.
- **Improved Inlets**
    - Where inlet conditions control the amount of flow that can pass through the culvert, improved inlets can greatly increase the hydraulic performance of the culvert.
- **Material Selection**
    - Reinforced concrete pipe (RCP), pre-cast and cast in place concrete boxes are recommended for use (1) under a roadway, (2) when pipe slopes are less than 1%, or (3) for all flowing streams. RCP and fully coated corrugated metal pipe can be used in all other cases. High-density polyethylene (HDPE) pipe may also be used as specified in

the municipal regulations. Table 3.12 gives recommended Manning's n values for different materials.

- **Culvert Skews**
  - Culvert skews shall not exceed 45 degrees as measured from a line perpendicular to the roadway centerline without approval.
- **Weep Holes**
  - Weep holes are sometimes used to relieve uplift pressure on headwalls and concrete rip-rap. Filter materials should be used in conjunction with the weep holes in order to intercept the flow and prevent the formation of piping channels through the fill embankment. The filter materials should be designed as an underdrain filter so as not to become clogged and so that piping cannot occur through the pervious material and the weep hole.
- **Outlet Protection**
  - See *Section 2.2* for information on the design of outlet protection.
- **Erosion and Sediment Control**
- **Environmental Considerations**
  - Where compatible with good hydraulic engineering, a site should be selected that will permit the culvert to be constructed to cause the least impact on the stream or wetlands. This selection must consider the entire site, including any necessary lead channels.
- **Safety Considerations**
  - Roadside safety should be considered for culverts crossing under roadways. Guardrails or safety end treatments may be needed to enhance safety at culvert crossings. The AASHTO roadside design guide should be consulted for culvert designs under and adjacent to roadways.

Table 3.11 Inlet Coefficients

Type of Structure and Design of Entrance	Coefficient K_e
Pipe, Concrete	
Projecting from fill, socket end (grove-end)	0.2
Projecting from fill, square cut end	0.5
Headwall or headwall and wingwalls	
Socket end of pipe (groove-end)	0.2
Square-edge	0.5
Rounded [radius = 1/12(D)]	0.2
Mitered to conform to fill slope	0.7
*End-Section conforming to fill slope	0.5
Beveled edges, 33.7° or 45° bevels	0.2
Side- or slope-tapered inlet	0.2
Pipe, or Pipe-Arch, Corrugated Metal[1]	
Projecting form fill (no headwall)	0.9
Headwall or headwall and wingwalls square-edge	0.5
Mitered to fill slope, paved or unpaved slope	0.7
*End Section conforming to fill slope	0.5
Beveled edges, 33.7° or 45° bevels	0.2
Slide- or slope-tapered inlet	0.2
Box, Reinforced Concrete	
Headwall parallel to embankment (no wingwalls)	
Square-edged on 3 edges	0.5
Rounded on 3 edges to radius of [1/12(D)] or [1/12(B)] or beveled edges on 3 sides	0.2
Wingwalls at 30° to 75° to barrel	
Square-edged at crown	0.4
Crown edge rounded to radius of [1/12(D)] or beveled top edge	0.2
Wingwalls at 10° or 25° to barrel	
Square-edged at crown	0.5
Wingwalls parallel (extension of sides)	
Square-edged at crown	0.7
Side- or slope-tapered inlet	0.2

1 Although laboratory tests have not been completed on K_e values for High-Density Polyethylene (HDPE) pipes, the K_e values for corrugated metal pipes are recommended for HDPE pipes.

* Note: "End Section conforming to fill slope", made of either metal or concrete, are the sections commonly available from manufacturers. From limited hydraulic tests they are equivalent in operation to a headwall in both inlet and outlet control. Some end sections incorporating a closed taper in their design have a superior hydraulic performance. These latter sections can be designed using the information given for the beveled inlet.

Source: HDS No. 5, 2001

Table 3.12 Manning's n Values

Type of Conduit	Wall & Joint Description	Manning's n
Concrete Pipe	Good joints, smooth walls	0.012
	Good joints, rough walls	0.016
	Poor joints, rough walls	0.017
Concrete Box	Good joints, smooth finished walls	0.012
	Poor joints, rough, unfinished walls	0.018
Corrugated Metal Pipes and Boxes Annular Corrugations	2 2/3- by ½-inch corrugations	0.024
	6- by 1-inch corrugations	0.025
	5- by 1-inch corrugations	0.026
	3- by 1-inch corrugations	0.028
	6-by 2-inch structural plate	0.035
	9-by 2-1/2 inch structural plate	0.035
Corrugated Metal Pipes, Helical Corrugations, Full Circular Flow	2 2/3-by ½-inch corrugated 24-inch plate width	0.012
Spiral Rib Metal Pipe	3/4 by 3/4 in recesses at 12 inch spacing, good joints	0.013
High Density Polyethylene (HDPE)	Corrugated Smooth Liner	0.015
	Corrugated	0.020
Polyvinyl Chloride (PVC)		0.011

Source: HDS No. 5, 2001

Note: For further information concerning Manning n values for selected conduits consult Hydraulic Design of Highway Culverts, Federal Highway Administration, 2001, HDS No. 5, pages 201 - 208.

3.3.4 Design Procedures

Types of Flow Control

There are two types of flow conditions for culverts that are based upon the location of the control section and the critical flow depth:

Inlet Control – Inlet control occurs when the culvert barrel is capable of conveying more flow than the inlet will accept. This typically happens when a culvert is operating on a steep slope. The control section of a culvert is located just inside the entrance. Critical depth occurs at or near this location, and the flow regime immediately downstream is supercritical.

Outlet Control – Outlet control flow occurs when the culvert barrel is not capable of conveying as much flow as the inlet opening will accept. The control section for outlet control flow in a culvert is located at the barrel exit or further downstream. Either subcritical or pressure flow exists in the culvert barrel under these conditions.

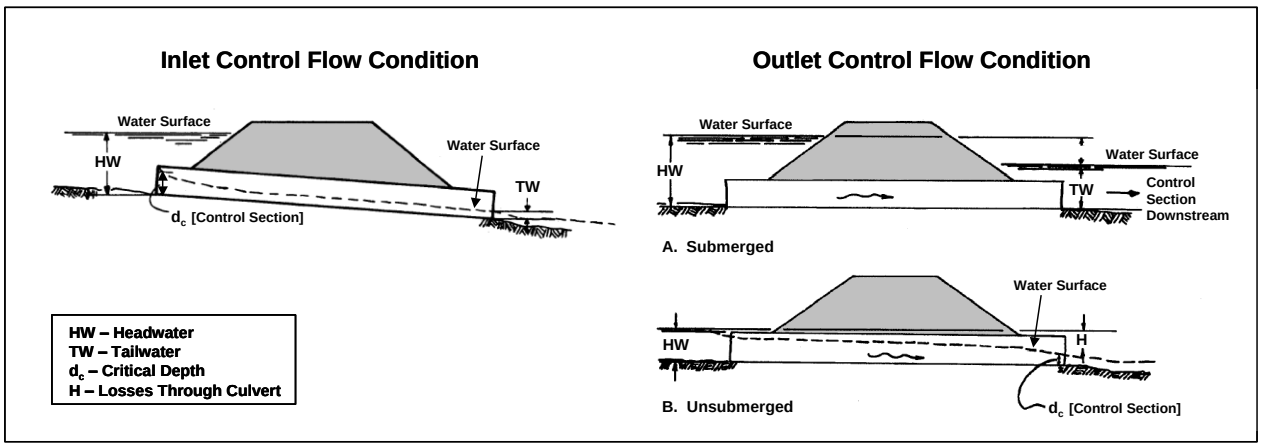


Figure 3.30 Culvert Flow Conditions
(Adapted from: HDS-5, 2001)

Proper culvert design and analysis requires checking for both inlet and outlet control to determine which will govern particular culvert designs. For more information on inlet and outlet control, see the FHWA Hydraulic Design of Highway Culverts, HDS-5, 2001.

Procedures

The culvert design process includes the following basic stages:

1. Define the location, orientation, shape, and material for the culvert to be designed. In many instances, consider more than single shape and material.
2. With consideration of the site data, establish allowable outlet velocity and maximum allowable depth of barrel.
3. Based on upon subject discharges, associated tailwater levels, and allowable headwater level, define an overall culvert configuration to be analyzed (culvert hydraulic length, entrance conditions, and conduit shape and material).
4. Determine the flow type (supercritical or subcritical) to establish the proper path for determination of headwater and outlet velocity.
5. Optimize the culvert configuration.
6. Treat any excessive outlet velocity separately from headwater.

There are three procedures for designing culverts: inlet control design equations, manual use of inlet and outlet control nomographs, and the use computer programs such as HY8. It is recommended that the HY8 computer model or equivalent be used for culvert design. The computer software package HYDRAIN, which includes HY8, uses the theoretical basis from the nomographs to size culverts. In addition, this software can evaluate improved inlets, route hydrographs, consider road overtopping, and evaluate outlet streambed scour. By using water surface profiles, this procedure is more accurate in predicting backwater effects and outlet scour.

Inlet Control Design Equations

This section contains explanations of the equations and methods used to develop the design charts in HDS No. 5, where those equations and methods are not fully described in the main text. The following topics are discussed: the design equations for the unsubmerged and submerged inlet control nomographs, the dimensionless design curves for culvert shapes and sizes without nomographs, and the dimensionless critical depth charts for long span culverts and corrugated metal box culverts.

Inlet Control Nomograph Equations: The design equations used to develop the inlet control nomographs are based on the research conducted by the National Bureau of Standards (NBS) under the sponsorship of the Bureau of Public Roads (now the Federal Highway Administration). Seven progress reports were produced as a result of this research. Of these, the first and fourth through seventh reports dealt with the hydraulics of pipe and box culvert entrances, with and without tapered inlets (4, 7, to 10). These reports were one source of the equation coefficients and exponents, along with other references and unpublished FHWA notes on the development of the nomographs (56 and 57).

The two basic conditions on inlet control depend upon whether the inlet end of the culvert is or is not submerged by the upstream headwater. If the inlet is not submerged, the inlet performs as a weir. If the inlet is submerged, the inlet performs as an orifice. Equations are available for each of the above conditions.

Between the unsubmerged and the submerged conditions, there is a transition zone for which the NBS research provided only limited information. The transition zone is defined empirically by drawing a curve between and tangent to the curves defined by the unsubmerged and submerged equations. In most cases, the transition zone is short and the curve is easily constructed.

Table 3.13 contains the unsubmerged and submerged inlet control design equations. Note that there are two forms of the unsubmerged equation. Form (1) is based on the specific head at critical depth, adjusted with tow correction factors. Form (2) is an exponential equation similar to a weir equation. Form (1) is preferable from a theoretical standpoint, but Form (2) is easier to apply and is the only documented form of equation for some of the inlet control nomographs.

The constants and the corresponding equation form are given in Table 3.14. Table 3.14 is arranged in the same order as the design nomographs shown later in this section, and provides the unsubmerged and submerged equation coefficients for each shape, material, and edge configuration. For the unsubmerged equations, the form of the equation is also noted.

The equations may be used to develop design curves for any conduit shape or size. Careful examination of the equation constants for a given form of equation reveals that there is very little difference between the constants for a given inlet configuration. Therefore, given the necessary conduit geometry for a new shape from the manufacturer, a similar shape is chosen from Table 3.14, and the constants are used to develop new design curves. The curves may be quasi-dimensionless, in terms of $Q/AD^{0.5}$ and HW_i/D, or dimensional, in terms of Q and HW_i for a particular conduit size. To make the curves truly dimensionless, $Q/AD^{0.5}$ must be divided by $g^{0.5}$, but this results in small decimal numbers. Note that coefficients for rectangular (Box) shapes should not be used for nonrectangular (circular, arch, pipe-arch, etc.) shapes and vice-versa. A constant slope value of 2 percent (0.02) is usually selected for the development of design curves. This is because the slope effect is small and the resultant headwater is conservatively high for sites with slopes exceeding 2 percent (except for mitered inlets).

Table 3.13 Inlet Control Design Equations

Unsubmerged*	
Form (1)	$\frac{HW_i}{D} = \frac{H_c}{D} + K\left(\frac{K_u Q}{AD^{0.5}}\right)^M - 0.5S$ *** **(3.30)**
Form (2)	$\frac{HW_i}{D} = K\left(\frac{K_u Q}{AD^{0.5}}\right)^M$ **(3.31)**
Submerged**	
	$\frac{HW_i}{D} = c\left(\frac{K_u Q}{AD^{0.5}}\right)^2 + Y - 0.5S$ *** **(3.32)**
Definitions	
HW_i	Headwater depth above inlet control section invert, m (ft)
D	Interior height of culvert barrel, m (ft)
H_c	Specific head at critical depth ($d_c + V_c^2/2g$), m^2 (ft^2)
Q	Discharge, m^3/s (ft^3/s)
A	Full cross sectional area of culvert barrel, m^2 (ft^2)
S	Culvert barrel slope, m/m (ft/ft)
K, M, c, Y	Constants from Table 3.14
K_u	1.811 SI (1.0 English)
* Equations 3.30 and 3.31 (unsubmerged) apply to about $Q/AD^{0.5} = 1.93$ (3.5 English)	
** Equation 3.32 (submerged) above applies to about $Q/AD^{0.5} = 2.21$ (4.0 English)	
*** For mitered inlets use +0.7 S instead of -0.5 S as the slope correction factor.	

Table 3.14 Constants for Inlet Control Design Equations

Chart No.	Shape and Material	Nomograph Scale	Inlet Edge Description	Equation Form	Unsubmerged		Submerged		References*
					K	M	c	Y	
1	Circular	1	Square edge w/ headwall	1	.0098	2.0	.0398	.67	56/57
	Concrete	2	Groove end w/ headwall		.0018	2.0	.0292	.74	
		3	Groove end projecting		.0045	2.0	.0317	.69	
2	Circular	1	Headwall	1	.0078	2.0	.0379	.69	56/57
	CMP	2	Mitered to slope		.0210	1.33	.0463	.75	
		3	Projecting		.0340	1.50	.0553	.54	
3	Circular	A	Beveled ring, 45º bevels	1	.0018	2.50	.0300	.74	57
		B	Beveled ring, 33.7º bevels		.0018	2.50	.0243	.83	
8	Rectangular	1	30º to 75º wingwall flares	1	.026	1.0	.0347	.81	56
	Box	2	90º and 15º wingwall flares		.061	.75	.0400	.80	56
		3	0º wingwall flares		.061	.75	.0423	.82	8
9	Rectangular	1	45º wingwall flare d = .043D	2	.510	.667	.0309	.80	8
	Box	2	18º to 33.7º wingwall flare d = .083D		.486	.667	.0249	.83	
10	Rectangular	1	90º headwall w/ 3/4" chamfers	2	.515	.667	.0375	.79	8
	Box	2	90º headwall w/ 45º bevels		.495	.667	.0314	.82	
		3	90º headwall w/ 33.7º bevels		.486	.667	.0252	.865	

Table 3.14 Constants for Inlet Control Design Equations

Chart No.	Shape and Material	Nomograph Scale	Inlet Edge Description	Equation Form	Unsubmerged		Submerged		References*
					K	M	c	Y	
11	Rectangular Box	1	3/4" chamfers; 45º skewed headwall	2	.545	.667	.0451	.73	8
		2	3/4" chamfers; 30º skewed headwall		.533	.667	.0425	.705	
		3	3/4" chamfers; 15º skewed headwall		.522	.667	.0402	.68	
		4	45º bevels; 10º-45º skewed headwall		.498	.667	.0327	.75	
12	Rectangular Box 3/4" chamfers	1	45º non-offset wingwall flares	2	.497	.667	.0339	.803	8
		2	18.4º non-offset wingwall flares		.493	.667	.0361	.806	
			18.4º non-offset wingwall flares		.495	.667	.0386	.71	
		3	30º skewed barrel						
13	Rectangular Box Top Bevels	1	45º wingwall flares - offset	2	.497	.667	.0302	.835	8
		2	33.7º wingwall flares - offset		.495	.667	.0252	.881	
		3	18.4º wingwall flares - offset		.493	.667	.0227	.887	
16-19	CM Boxes	2	90º headwall	1	.0083	2.0	.0379	.69	57
		3	Thick wall projecting		.0145	1.75	.0419	.64	
		5	Thin wall projecting		.0340	1.5	.0496	.57	
29	Horizontal Ellipse Concrete	1	Square edge w/ headwall	1	.0100	2.0	.0398	.67	57
		2	Groove end w/ headwall		.0018	2.5	.0292	.74	
		3	Groove end projecting		.0045	2.0	.0317	.69	
30	Vertical Ellipse Concrete	1	Square edge w/ headwall	1	.0100	2.0	.0398	.67	57
		2	Groove end w/ headwall		.0018	2.5	.0292	.74	
		3	Groove end projecting		.0095	2.0	.0317	.69	
34	Pipe Arch 18" Corner Radius CM	1	90º headwall	1	.0083	2.0	.0379	.69	57
		2	Mitered to slope		.0300	1.0	.0463	.75	
		3	Projecting		.0340	1.5	.0496	.57	
35	Pipe Arch 18" Corner Radius CM	1	Projecting	1	.0300	1.5	.0496	.57	56
		2	No Bevels		.0088	2.0	.0368	.68	
		3	33.7º Bevels		.0030	2.0	.0269	.77	
36	Pipe Arch 31" Corner Radius CM	1	Projecting	1	.0300	1.5	.0496	.57	56
		2	No Bevels		.0088	2.0	.0368	.68	
		3	33.7º Bevels		.0030	2.0	.0269	.77	
41-43	Arch CM	1	90º headwall	1	.0083	2.0	.0379	.69	57
		2	Mitered to slope		.0300	1.0	.0463	.75	
		3	Thin wall projecting		.0340	1.5	.0496	.57	
55	Circular	1	Smooth tapered inlet throat	2	.534	.555	.0196	.90	3
		2	Rough tapered inlet throat		.519	.64	.0210	.90	
56	Elliptical Inlet Face	1	Tapered inlet-beveled edges	2	.536	.622	.0368	.83	3
		2	Tapered inlet-square edges		.5035	.719	.0478	.80	
		3	Tapered inlet-thin edge projecting		.547	.80	.0598	.75	
57	Rectangular	1	Tapered inlet throat	2	.475	.667	.0179	.97	3
58	Rectangular Concrete	1	Side tapered-less favorable edges	2	.56	.667	.0446	.85	3
					.56	.667	.0378	.87	
		2	Side tapered-more favorable edges						

Table 3.14 Constants for Inlet Control Design Equations

Chart No.	Shape and Material	Nomograph Scale	Inlet Edge Description	Equation Form	Unsubmerged		Submerged		References*
					K	M	c	Y	
59	Rectangular Concrete	1	Slope tapered-less favorable edges	2	.50	.667	.0446	.65	3
		2	Slope tapered-more favorable edges		.50	.667	.0378	.71	

* These references are cited in FHWA, 2001, HYD-5. They can be accessed at the Federal Highway Administration web site: www.fhwa.dot.gov/bridge/hydpub.htm.

Nomographs

The use of culvert design nomographs requires a trial and error solution. Nomograph solutions provide reliable designs for many applications. It should be remembered that velocity, hydrograph routing, roadway overtopping, and outlet scour require additional, separate computations beyond what can be obtained from the nomographs. Figures 3.31 (a) and (b) show examples of an inlet control and outlet control nomographs for the design of concrete pipe culverts. For other culvert designs, refer to the complete set of nomographs in FHWA Hydraulic Design of Highway Culverts, HDS-5, 2001, Second Edition.

This section presents design guidance for culverts originally published in HEC-12, Drainage of Highway Pavements and AASHTO's Model Drainage Manual.

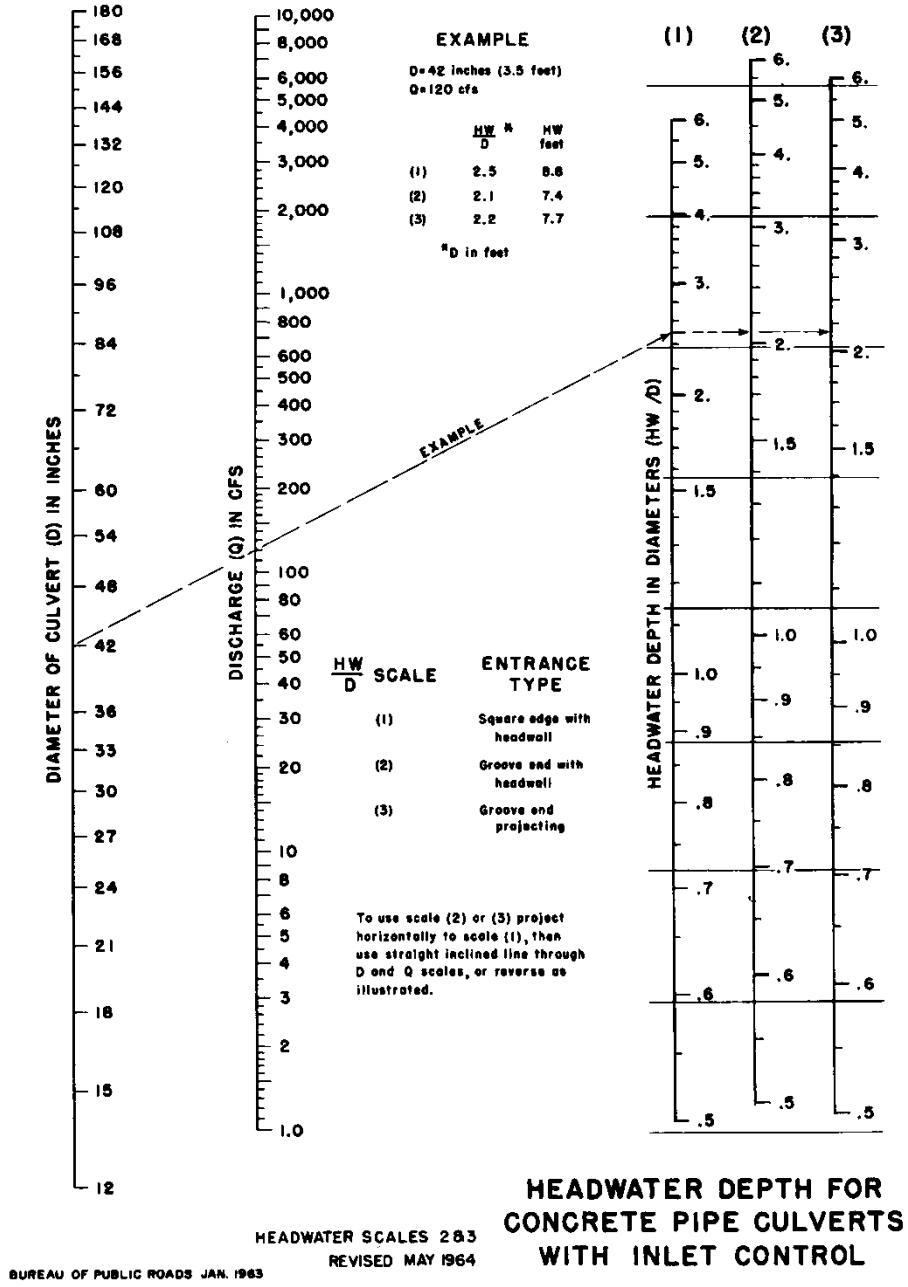


Figure 3.31a Headwater Depth for Concrete Pipe Culvert with Inlet Control

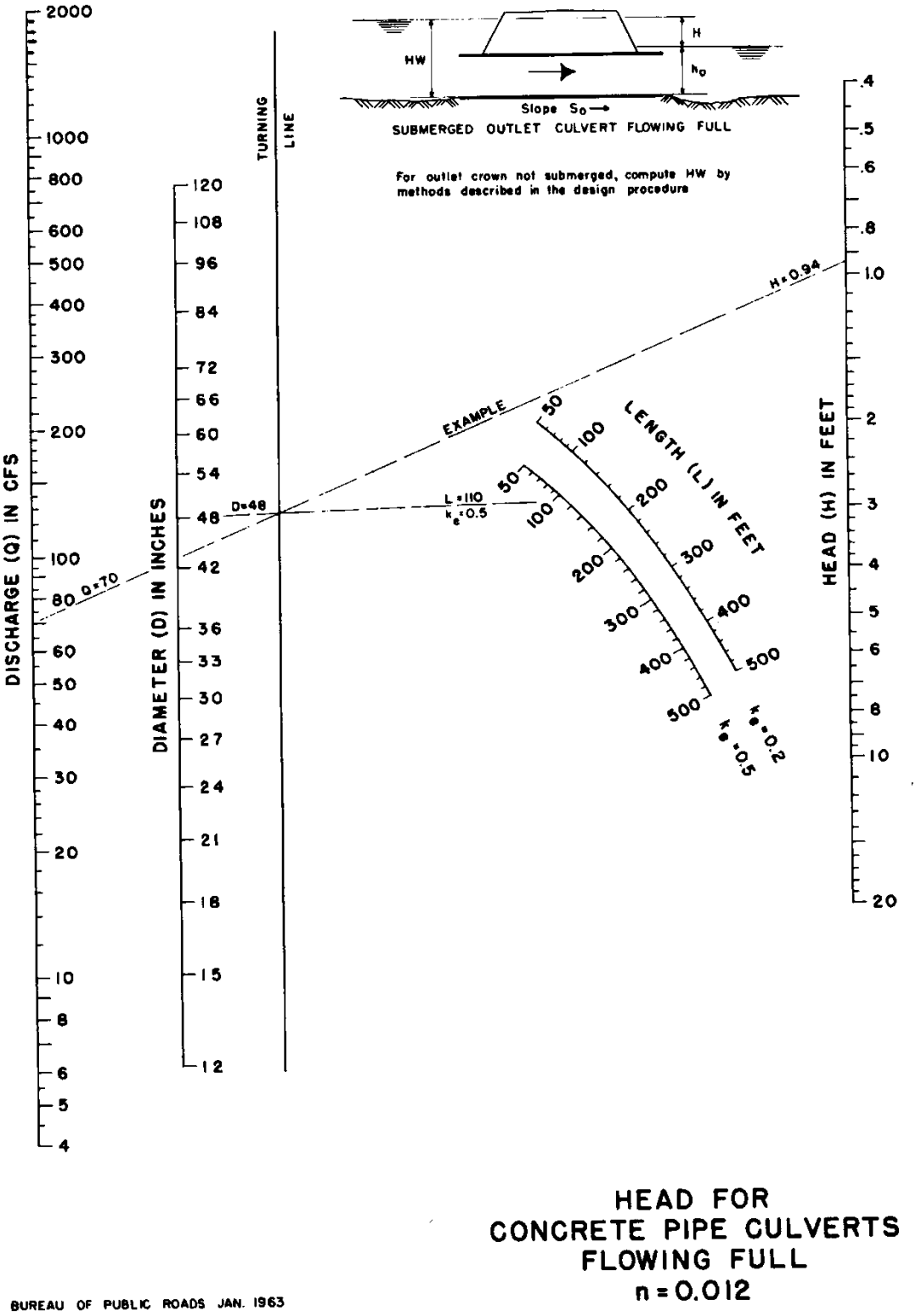


Figure 3.31b Head for Concrete Pipe Culverts Flowing Full

Design Procedure

The following design procedure requires the use of inlet and outlet nomographs.

Step 1 List design data:

- Q = discharge (cfs)
- L = culvert length (ft)
- S = culvert slope (ft/ft)
- TW = tailwater depth (ft)
- V = velocity for trial diameter (ft/s)
- K_e = inlet loss coefficient
- HW = allowable headwater depth for the design storm (ft)

Step 2 Determine trial culvert size by assuming a trial velocity of 3 to 5 ft/s and computing the culvert area, A = Q/V. Determine the culvert diameter (inches).

Step 3 Find the actual HW for the trial size culvert for both inlet and outlet control.

- For inlet control, enter inlet control nomograph with D and Q and find HW/D for the proper entrance type.
- Compute HW and, if too large or too small, try another culvert size before computing HW for outlet control.
- For outlet control enter the outlet control nomograph with the culvert length, entrance loss coefficient, and trial culvert diameter.
- To compute HW, connect the length scale for the type of entrance condition and culvert diameter scale with a straight line, pivot on the turning line, and draw a straight line from the design discharge through the turning point to the head loss scale H. Compute the headwater elevation HW from the equation:

$$\mathbf{HW = H + h_o - LS} \qquad \mathbf{(3.33)}$$

where:

- h_o = ½ (critical depth + D), or tailwater depth, whichever is greater
- L = culvert length
- S = culvert slope

Step 4 Compare the computed headwaters and use the higher HW nomograph to determine if the culvert is under inlet or outlet control.

- If inlet control governs, then the design is complete and no further analysis is required.
- If outlet control governs and the HW is unacceptable, select a larger trial size and find another HW with the outlet control nomographs. Since the smaller size of culvert had been selected for allowable HW by the inlet control nomographs, the inlet control for the larger pipe need not be checked.

Step 5 Calculate exit velocity and if erosion problems might be expected, refer to *Section 4.0* for appropriate energy dissipation designs. Energy dissipation designs may affect the outlet hydraulics of the culvert.

Performance Curves - Roadway Overtopping

A performance curve for any culvert can be obtained from the nomographs by repeating the steps outlined above for a range of discharges that are of interest for that particular culvert design. A graph is then plotted of headwater versus discharge with sufficient points so that a curve can be drawn through the range of interest. These curves are applicable through a range of headwater, velocities, and scour depths versus discharges for a length and type of culvert. Usually charts with length intervals of 25 to 50 feet are satisfactory for design purposes. Such computations are made much easier by the use of computer programs.

To complete the culvert design, roadway overtopping should be analyzed. A performance curve showing the culvert flow as well as the flow across the roadway is a useful analysis tool. Rather than using a trial and error procedure to determine the flow division between the overtopping flow and the culvert flow, an overall performance curve can be developed.

The overall performance curve can be determined as follows:

Step 1 Select a range of flow rates and determine the corresponding headwater elevations for the culvert flow alone. The flow rates should fall above and below the design discharge and cover the entire flow range of interest. Both inlet and outlet control headwaters should be calculated.

Step 2 Combine the inlet and outlet control performance curves to define a single performance curve for the culvert.

Step 3 When the culvert headwater elevations exceed the roadway crest elevation, overtopping will begin. Calculate the equivalent upstream water surface depth above the roadway (crest of weir) for each selected flow rate. Use these water surface depths and Equation 3.34 to calculate flow rates across the roadway.

$$\mathbf{Q = C_d L (HW)^{1.5}} \qquad \mathbf{(3.34)}$$

where:

- Q = overtopping flow rate (ft^3/s)
- C_d = overtopping discharge coefficient
- L = length of roadway (ft)
- HW = upstream depth, measured from the roadway crest to the water surface upstream of the weir drawdown (ft)

Note: See Figure 3.32 on the next page for guidance in determining a value for C_d. For more information on calculating overtopping flow rates see pages 38 - 44 in HDS No. 5, 2001.

Step 4 Add the culvert flow and the roadway overtopping flow at the corresponding headwater elevations to obtain the overall culvert performance curve.

Storage Routing

A significant storage capacity behind a highway embankment attenuates a flood hydrograph. Because of the reduction of the peak discharge associated with this attenuation, the required capacity of the culvert, and its size, may be reduced considerably. If significant storage is anticipated behind a culvert, the design should be checked by routing the design hydrographs through the culvert to determine the discharge and stage behind the culvert. See *Section 3.3.7* for more information on routing. Additional routing procedures are outlined in Hydraulic Design of Highway Culverts, Section V - Storage Routing, HDS No. 5, 2001, Federal Highway Administration, pages 123 - 142.

Note: Storage should be taken into consideration only if the storage area will remain available for the life of the culvert as a result of purchase of ownership or right-of-way or an easement has been acquired.

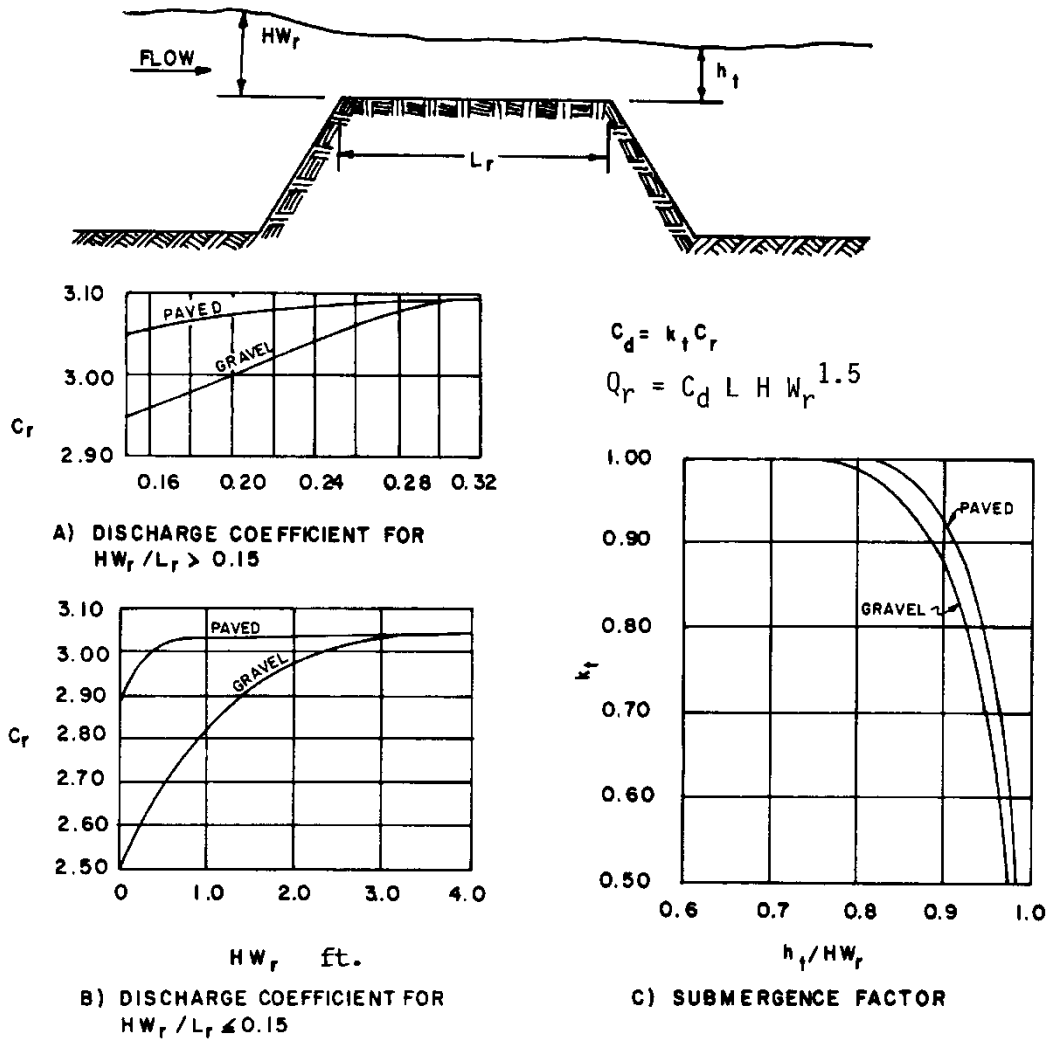


Figure 3.32 Discharge Coefficients for Roadway Overtopping
(Source: HDS No. 5, 2001)

3.3.5 Culvert Design Example

The following example problem illustrates the procedures to be used in designing culverts using the nomographs.

Example

Size a culvert given the following example data, which were determined by physical limitations at the culvert site and hydraulic procedures described elsewhere in this handbook.

Input Data

Discharge for 2-yr flood = 35 cfs

Discharge for 25-yr flood = 70 cfs

Allowable H_w for 25-yr discharge = 5.25 ft

Length of culvert = 100 ft

Natural channel invert elevations - inlet = 15.50 ft, outlet = 14.30 ft

Culvert slope = 0.012 ft/ft

Tailwater depth for 25-yr discharge = 3.5 ft

Tailwater depth is the normal depth in downstream channel

Entrance type = Groove end with headwall

Computations

1. Assume a culvert velocity of 5 ft/s. Required flow area = 70 cfs/5 ft/s = 14 ft^2 (for the 25-yr recurrence flood).
2. The corresponding culvert diameter is about 48 in. This can be calculated by using the formula for area of a circle: Area = $(3.14D^2)/4$ or D = (Area times $4/3.14)^{0.5}$. Therefore: D = ((14 sq ft x 4)/3.14) $^{0.5}$ x 12 in/ft) = 50.7 in
3. A grooved end concrete culvert with a headwall is selected for the design. Using the inlet control nomograph (Figure 3.31a), with a pipe diameter of 48 inches and a discharge of 70 cfs; read a HW/D value of 0.93.
4. The depth of headwater (HW) is (0.93) x (4) = 3.72 ft, which is less than the allowable headwater of 5.25 ft. Since 3.72 ft is considerably less than 5.25 try a small culvert.
5. Using the same procedures outlined in steps 4 and 5 the following results were obtained.

   42-inch culvert – HW = 4.13 ft

   36-inch culvert – HW = 5.04 ft

   Select a 36-inch culvert to check for outlet control.
6. The culvert is checked for outlet control by using Figure 3.31b.

   With an entrance loss coefficient K_e of 0.20, a culvert length of 100 ft, and a pipe diameter of 36 in., an H value of 2.8 ft is determined. The headwater for outlet control is computed by the equation: $HW = H + h_o - LS$

   Compute h_o

   $h_o = T_w$ or ½ (critical depth in culvert + D), whichever is greater.

   h_o = 3.5 ft or h_o = ½ (2.7 + 3.0) = 2.85 ft

   Note: critical depth is obtained from Figure 1.19(b).

   Therefore: h_o = 3.5 ft

   The headwater depth for outlet control is:

   $HW = H + h_o - LS = 2.8 + 3.5 - (100) \times (0.012) = 5.10$ ft
7. Since HW for outlet control (5.10 ft) is greater than the HW for inlet control (5.04 ft), outlet control governs the culvert design. Thus, the maximum headwater expected for a 25-year recurrence flood is 5.10 ft, which is less than the allowable headwater of 5.25 ft.
8. Estimate outlet exit velocity. Since this culvert is an outlet control and discharges into an open channel downstream with tailwater above culvert, the culvert will be flowing full at the flow depth in the channel. Using the design peak discharge of 70 cfs and the area of a 36-inch or 3.0-foot diameter culvert the exit velocity will be:

   $Q = VA$

   Therefore: $V = 70 / (3.14(3.0)^2)/4 = 9.9$ ft/s

With this high velocity, consideration should be given to provide an energy dissipator at the culvert outlet. See *Section 4.0*.

9. Check for minimum velocity using the 2-year flow of 35 cfs.

   Therefore: $V = 35 / (3.14(3.0)^2/4 = 5.0$ ft/s > minimum of 2.5 - OK

10. The flood mitigation storm flow should be routed through the culvert to determine if any flooding problems will be associated with this flood.

Figure 3.33 provides a convenient form to organize culvert design calculations.

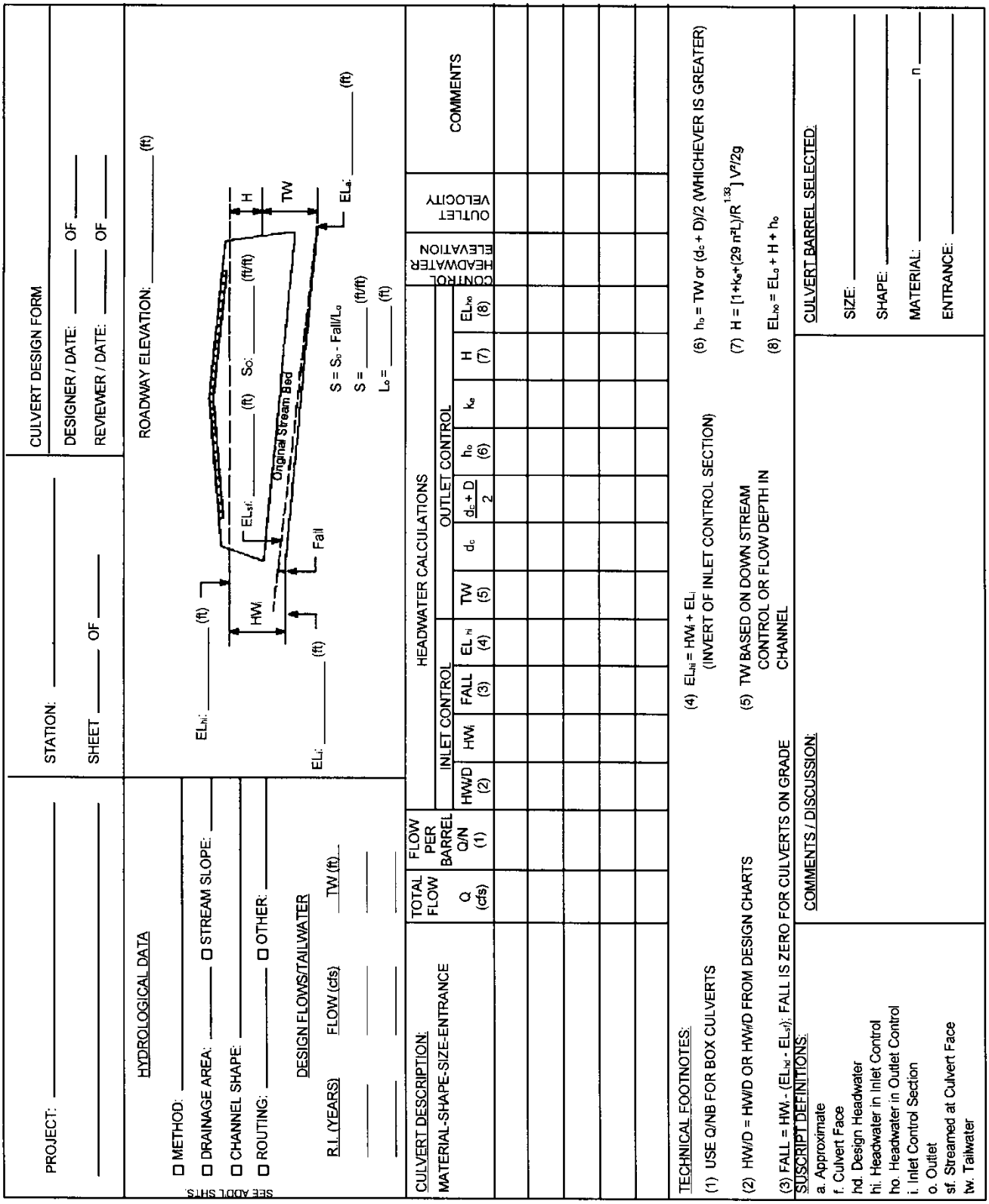


Figure 3.33 Culvert Design Calculation Form
(Source: HDS No. 5, 2001)

3.3.6 Design Procedures for Beveled-Edged Inlets

Introduction

Improved inlets include inlet geometry refinements beyond those normally used in conventional culvert design practice. Several degrees of improvements are possible, including bevel-edged, side-tapered, and slope-tapered inlets. Those designers interested in using side- and slope-tapered inlets should consult the detailed design criteria and example designs outlined in the U. S. Department of Transportation publication Hydraulic Design Series No. 5 entitled, Hydraulic Design of Highway Culverts.

Design Figures

Four inlet control figures for culverts with beveled edges are found in Appendix D of HDS No. 5.

Chart	Page	Use for
3	D-3A & B	circular pipe culverts with beveled rings
10	D-10A & B	90° headwalls (same for 90 ° wingwalls)
11	D-11A & B	skewed headwalls
112	D-12A & B	wingwalls with flare angles of 18 to 45 degrees

The following symbols are used in these figures:

B – Width of culvert barrel or diameter of pipe culvert

D – Height of box culvert or diameter of pipe culvert

H_f – Depth of pool or head, above the face section of invert

N – Number of barrels

Q – Design discharge

Design Procedure

The figures for bevel-edged inlets are used for design in the same manner as the conventional inlet design nomographs discussed earlier. Note that Charts 10, 11, and 12 found in Appendix D of HDS No. 5 apply only to bevels having either a 33° angle (1.5:1) or a 45° angle (1:1).

For box culverts the dimensions of the bevels to be used are based on the culvert dimensions. The top bevel dimension is determined by multiplying the height of the culvert by a factor. The side bevel dimensions are determined by multiplying the width of the culvert by a factor. For a 1:1 bevel, the factor is 0.5 inch/ft. For a 1.5:1 bevel the factor is 1 inch/ft. For example, the minimum bevel dimensions for an 8 ft x 6 ft box culvert with 1:1 bevels would be:

Top Bevel = d = 6 ft x 0.5 inch/ft = 3 inches

Side Bevel = b = 8 ft x 0.5 inch/ft = 4 inches

For a 1.5:1 bevel computations would result in d = 6 and b = 8 inches.

Design Figure Limits

The improved inlet design figures are based on research results from culvert models with barrel width, B, to depth, D, ratios of from 0.5:1 to 2:1. For box culverts with more than one barrel, the figures are used in the same manner as for a single barrel, except that the bevels must be sized on the basis of the total clear opening rather than on individual barrel size.

For example, in a double 8 ft by 8 ft box culvert:

Top Bevel is proportioned based on the height of 8 feet, which results in a bevel of 4 in. for the 1:1 bevel and 8 in. for the 1.5:1 bevel.

Side Bevel is proportioned based on the clear width of 16 feet, which results in a bevel of 8 in. for the 1:1 bevel and 16 in. for the 1.5:1 bevel.

Multi-barrel Installations

For multi-barrel installations exceeding a 3:1 width to depth ratio, the side bevels become excessively large when proportioned on the basis of the total clear width. For these structures, it is recommended that the side bevel be sized in proportion to the total clear width, B, or three times the height, whichever is smaller.

The top bevel dimension should always be based on the culvert height.

The shape of the upstream edge of the intermediate walls of multi-barrel installations is not as important to the hydraulic performance of a culvert as the edge condition of the top and sides. Therefore, the edges of these walls may be square, rounded with a radius of one-half their thickness, chamfered, or beveled. The intermediate walls may also project from the face and slope downward to the channel bottom to help direct debris through the culvert.

Multi-barrel pipe culverts should be designed as a series of single barrel installations since each pipe requires a separate bevel.

Skewed Inlets

It is recommended that Chart 11 found in Appendix D of HDS No. 5 for skewed inlets not be used for multiple barrel installations, as the intermediate wall could cause an extreme contraction in the downstream barrels. This would result in underdesign due to a greatly reduced capacity. Skewed inlets (at an angle with the centerline of the stream) should be avoided whenever possible and should not be used with side- or slope-tapered inlets. It is important to align culverts with streams in order to avoid erosion problems associated with changing the direction of the natural stream flow.

3.3.7 Flood Routing and Culvert Design

Introduction

Flood routing through a culvert is a practice that evaluates the effect of temporary upstream ponding caused by the culvert's backwater. By not considering flood routing it is possible that the findings from culvert analyses will be conservative. If the selected allowable headwater is accepted without flood routing, then costly over-design of both the culvert and outlet protection may result, depending on the amount of temporary storage involved. However, if storage is used in the design of culverts, consideration should be given to:

- The total area of flooding,
- The average time that bankfull stage is exceeded for the design flood up to 48 hours in rural areas or 6 hours in urban areas, and
- Ensuring that the storage area will remain available for the life of the culvert through the purchase of right-of-way or easement.

Design Procedure

The design procedure for flood routing through a culvert is the same as for reservoir routing. The site data and roadway geometry are obtained and the hydrology analysis completed to include estimating a hydrograph. Once this essential information is available, the culvert can be designed. Flood routing through a culvert can be time consuming. It is recommended that a computer program be used to

perform routing calculations; however, an engineer should be familiar with the culvert flood routing design process.

A multiple trial and error procedure is required for culvert flood routing. In general:

Step 1 A trial culvert(s) is selected

Step 2 A trial discharge for a particular hydrograph time increment (selected time increment to estimate discharge from the design hydrograph) is selected

Step 3 Flood routing computations are made with successive trial discharges until the flood routing equation is satisfied

Step 4 The hydraulic findings are compared to the selected site criteria

Step 5 If the selected site criteria are satisfied, then a trial discharge for the next time increment is selected and this procedure is repeated; if not, a new trial culvert is selected and the entire procedure is repeated.

Comprehensive Design Guidance

Comprehensive design discussions and guidance may be found in the Federal Highway Administration, National Design Series No. 5, document entitled Hydraulic Design of Highway Culverts, Second Edition, published in 2001. This document is available from the National Technical Information Service as Item Number PB2003102411*DL. (http://www.ntis.gov/search.htm) Search for this document using the Item Number.

3.4 Bridge Design

3.4.1 Overview

Bridges enable streams to maintain flow conveyance and to sustain aquatic life. They are important and expensive highway hydraulic structures vulnerable to failure from flood related causes. In order to minimize the risk of failure, the hydraulic requirements of a stream crossing must be recognized and considered during the development, construction, and maintenance phases.

This section addresses structures designed hydraulically as bridges, regardless of length. For economy and hydraulic efficiency, engineers should design culverts to operate with the inlet submerged during flood flows, if conditions permit. Bridges, on the other hand, are not covered with embankment or designed to take advantage of submergence to increase hydraulic capacity, even though some are designed to be inundated under flood conditions. This discussion of bridge hydraulics considers the total crossing, including approach embankments and structures on the floodplains.

The following subsections present considerations related to the hydraulics of bridges. It is generally excerpted from Chapter 9 of the Texas Department of Transportation (TxDOT) Hydraulics Design Manual dated March 2004.

Bridge Hydraulics Considerations

When beginning analysis for a cross-drainage facility, the flood frequency and stage-discharge curves should first be established, as well as the type of cross-drain facility. The choice is usually between a bridge and a culvert. Bridges are usually chosen if the discharge is significant or if the stream to be crossed is large in extent. Both types of facilities should be evaluated and a choice made based on performance and economics. If the stream crossing is wide with multiple concentrations of flow, a multiple opening facility may be in order.

Highway-Stream Crossing Analysis

The hydraulic analysis of a highway-stream crossing for a particular flood frequency involves:

- Determining the backwater associated with each alternative profile and waterway opening(s)
- Determining the effects on flow distribution and velocities
- Estimating scour potential

The hydraulic design of a bridge over a waterway involves the following such that the risks associated with backwater and increased velocities are not excessive:

- Establishing a location
- Bridge length
- Orientation
- Roadway and bridge profiles

A hydrologic and hydraulic analysis is recommended for designing all new bridges over waterways, bridge widening, bridge replacement, and roadway profile modifications that may adversely affect the floodplain, even if no structural modifications are necessary. Typically, this should include the following:

- An estimate of peak discharge (sometimes complete runoff hydrographs)
- Existing and proposed condition water surface profiles for design and check flood conditions
- Consideration of the potential for stream stability problems and scour potential.

Freeboard

Navigational clearance and other reasons notwithstanding, the low chord elevation is defined as the sum of the design normal water surface elevation (high water) and a *freeboard*. For on system TxDOT bridges, TxDOT recommends a minimum freeboard of 2 ft to allow for passage of floating debris and to provide a safety factor for design flood flow. Higher freeboards may be appropriate over streams that are prone to heavy debris loads, such as large tree limbs, and to accommodate other clearance needs. Other constraints may make lower freeboards desirable, but the low chord should not impinge on the design high water. Generally, for off-system bridge replacement structures, the low chord should approximate that of the structure to be replaced, unless the results of a risk assessment indicate a different structure is the most beneficial option.

Roadway/Bridge Profile

A bridge is integrated into both the stream and the roadway and must be fully compatible with both. Therefore, the alignment of the roadway and the bridge are the same between the ends of the bridge. Hydraulically, the complete bridge profile can be any part of the structure that stream flow can strike or impact in its movement downstream. If the stream gets high enough to inundate the structure, then all parts of the roadway and the bridge become part of the complete bridge profile.

For TxDOT design, the roadway must not be inundated by the design flood, but inundation by the flood mitigation storm is allowed. Unless the route is an emergency escape route, it is often desirable to allow floods in excess of the design flood to overtop the road. This helps minimize both the backwater and the required length of structure.

Several vertical alignment alternatives are available for consideration, depending on site topography, traffic requirements, and flood damage potential. The alternatives range from crossings that are designed to overtop frequently to crossings that are designed to rarely or never overtop.

Crossing Profile

The horizontal alignment of a highway at a stream crossing should be taken into consideration when selecting the design and location of the waterway opening as well as the crossing profile. Every effort should be made to align the highway so that the crossing will be normal to the stream flow direction (highway centerline perpendicular to the streamline).

Often, this is not possible because of the highway or stream configuration. When a skewed structure is necessary, it should be ensured that substructure fixtures such as foundations, columns, piers, and bent caps offer minimum resistance to the stream flow.

Bent caps should be oriented as near to the skew of the streamlines at flood stage as possible. Headers should be skewed to minimize eddy-causing obstructions. A relief opening may be provided to reduce the likelihood of trapped flow and minimize the amount of flow that would have to travel up against the general direction of flow along the embankment.

Single Versus Multiple Openings

For a single structure, the flow will find its way to an opening until the roadway is overtopped. If two or more structures have flow area available, after accumulating a head, the flow will divide and proceed to the structures offering the least resistance. The point of division is called a stagnation point.

In usual practice, the TxDOT recommends that the flood discharge be forced to flow parallel to the highway embankment for no more than about 800 ft. If flow distances along the embankment are greater than recommended, an additional relief structure or opening should be considered. A possible alternative to the provision of an additional structure is a guide bank (spur dike) to control the turbulence at the header. Also, natural vegetation between the toe of slope and the right-of-way line is useful in controlling

flow along the embankment. Therefore, special efforts should be made to preserve any natural vegetation in such a situation.

Factors Affecting Bridge Length

The discussions of bridge design assume normal cross sections and lengths (perpendicular to flow at flood stage). Usually one-dimensional flow is assumed, and cross sections and lengths are considered 90° to the direction of stream flow at flood stage.

If the crossing is skewed to the stream flow at flood stage, all cross sections and lengths should be normalized before proceeding with the bridge length design. If the skew is severe and the floodplain is wide, the analysis may need to be adjusted to offset the effects of elevation changes within the same cross section.

The following examples illustrate various factors that can cause a bridge opening to be larger than that required by hydraulic design.

- Bank protection may be placed in a certain location due to local soil instability or a high bank.
- Bridge costs may be cheaper than embankment costs.
- A highway profile grade line might dictate an excessive freeboard allowance. For sloping abutments, a higher freeboard will result in a longer bridge.
- High potential for meander to migrate, or other channel instabilities may warrant a longer opening.

3.4.2 Symbols and Definitions

The hydraulics of bridge openings are basically the same as those of open channel flow. Therefore, the symbols and definitions are essentially the same as those of in Table 3.1. There are other definitions unique to bridges which are presented here. They are defined in the TxDOT Hydraulic Design Manual.

Flow Zones and Energy Losses

Figure 3.34 shows a plan of typical cross section locations that establish three flow zones that should be considered when estimating the effects of bridge openings.

Zone 1 represents the area between the downstream face of the bridge and a cross section downstream of the bridge within which expansion of flow from the bridge is expected to occur. The distance over which this expansion occurs can vary depending on the flow rate and the floodplain characteristics. No detailed guidance is available, but a distance equal to about four times the length of the average embankment constriction is reasonable for most situations. Section 1 represents the effective channel flow geometry at the end of the expansion zone, which is also called the “exit” section. Cross sections 2 and 3 are at the toe of roadway embankment and represent the portion of unconstricted channel geometry that approximates the effective flow areas near the bridge opening as shown in Figure 3.35.

Zone 2 represents the area under the bridge opening through which friction, turbulence, and drag losses are considered. Generally, the bridge opening is obtained by superimposing the bridge geometry on cross sections 2 and 3.

Zone 3 represents an area from the upstream face of the bridge to a distance upstream where the contraction of flow must occur. A distance upstream of the bridge equal to the length of the average embankment constriction is a reasonable approximation of the location at which contraction begins. Cross section 4 represents the effective channel flow geometry where contraction begins. This is sometimes referred to as the “approach” cross section.

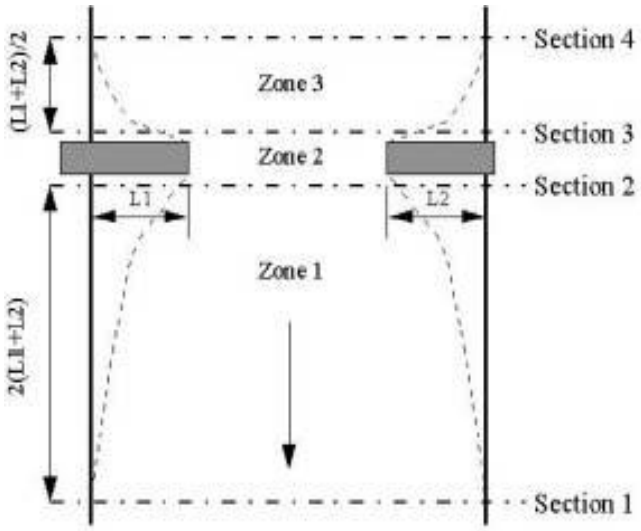


Figure 3.34 Flow Zones at Bridges
(TxDOT Hydraulic Design Manual)

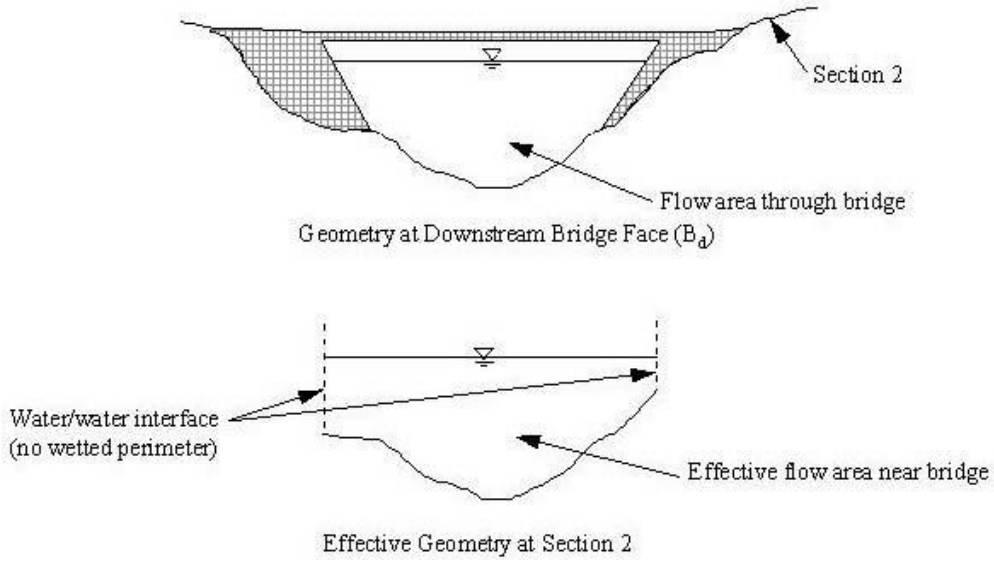


Figure 3.35 Effective Geometry for Bridge (Section 2 shown, Section 3 similar)
(TxDOT Hydraulic Design Manual)

Bridge Flow Class

The losses associated with flow through bridges depend on the hydraulic conditions of low or high flow.

Low flow describes hydraulic conditions in which the water surface between Zones 1, 2, and 3 is open to atmospheric pressure. That means the water surface does not impinge upon the superstructure. (This condition should exist for the design frequency of all new on-system bridges.) Low flow is divided into categories as described in the "Low Flow Classes" table below Type I is the most common in Texas, although severe constrictions compared to the flow conditions could result in Types IIA and IIB. Type III is likely to be limited to steep hills and mountainous regions.

Table 3.15 Low Flow Classes

Low Flow Class	Description
I	Subcritical flow through all Zones
IIA	Subcritical flow through Zones 1 and 3; flow through critical depth in Zone 2
IIB	Subcritical flow through Zone 3; flow through critical depth in Zone 2, hydraulic jump in Zone 1
III	Supercritical flow through all Zones

High flow refers to conditions in which the water surface impinges on the bridge superstructure:

- When the tailwater does not submerge the low chord of the bridge, the flow condition is comparable to a pressure flow sluice gate.
- When the tailwater submerges the low chord but does not exceed the elevation of critical depth over the road, the flow condition is comparable to orifice flow.
- If the tailwater overtops the roadway, neither sluice gate flow nor orifice flow is reasonable, and the flow is either weir flow or open flow.

3.4.3 Design Recommendations

The design of a bridge should take into account many different engineering and technical aspects at the bridge site and adjacent areas. The following design recommendations should be considered for all bridge designs as applicable. See the design criteria of the local jurisdiction for specific requirements.

Frequency Flood

Design discharges chosen by TxDOT for bridges vary with the functional classification and structure type. For major river crossings, a return period of 50 years is recommended. Flow small bridges, the recommended return period is 25 years. In all cases the check flood is for the flood mitigation storm return period.

Freeboard

Typical freeboard, the length between the computed design water surface and the low chord, is two feet. In urban settings, it may be prudent to use the flood mitigation storm fully-developed discharge to check the bridge design. The flood mitigation storm discharge, assuming blockage of outlet works, with 6" of freeboard. Some municipalities may specify different design storms and freeboard requirements.

Loss Coefficients

The contraction and expansion of water through the bridge opening creates hydraulic losses. These losses are accounted for through the use of loss coefficients. Table 3.16 gives recommended values for the Contraction (K_c) and Expansion (K_e) Coefficients.

Table 3.16 Recommended Loss Coefficients for Bridges		
Transition Type	**Contraction (K_c)**	**Expansion (K_e)**
No losses computed	0.0	0.0
Gradual transition	0.1	0.3
Typical bridge	0.3	0.5
Severe transition	0.6	0.8

3.4.4 Design Procedures

The following is a general bridge hydraulic design procedure.

1. Determine the most efficient alignment of proposed roadway, attempting to minimize skew at the proposed stream crossing.
2. Determine design discharge from hydrologic studies or available data (Municipality, Federal Emergency Management Agency (FEMA), US Army Corp of Engineers (USACE), TxDOT, or similar sources).
3. If available, obtain effective FEMA hydraulic backwater model. It is assumed that if a bridge is required instead of a culvert, the drainage area would exceed one square mile and could already be included in a FEMA study. If an effective FEMA model or other model is not available, a basic hydrologic model and backwater analysis for the stream must be prepared. The HEC-RAS computer model is routinely used to compute backwater water surface profiles.
4. Using USACE or FEMA guidelines, compute or duplicate an existing conditions water surface profile for the design storm(s). Compute a profile for the fully-developed watershed to use as a baseline for design of a new bridge/roadway crossing.
5. Use the design discharge to compute an approximate opening that will be needed to pass the design storm (for preliminary sizing, use a normal-depth design procedure, or simply estimate a required trapezoidal opening.
6. Prepare a bridge crossing data set in the hydraulic model to reflect the preliminary design opening, which includes the required freeboard and any channelization upstream or downstream to transition the floodwaters through the proposed structure.
7. Compute the proposed bridge flood profile and design parameters (velocities, flow distribution, energy grade, etc.). Review for criteria on velocities and freeboard, and revise model as needed to accommodate design flows.
8. Review the velocities and determine erosion control requirements downstream, through the structure, and upstream.
9. Finalize the design size and erosion control features, based on comparing the proposed model with the existing conditions profiles, impacts on other properties, FEMA guidelines, and local criteria.
10. Exceptions/Other Issues
    A. Conditional Letter of Map Amendment (CLOMR) may be needed for new crossings of streams studied by FEMA.
    B. If applicable, coordinate with USACE Regulatory Permit requirements.
    C. Evaluate the project with respect to iSWM policy regarding downstream impacts.
    D. Design should be for fully developed watershed conditions. If the available discharges are from FEMA existing conditions hydrology, the following options are available: (1) obtain new hydrology, (2) extrapolate fully-developed from existing data, or (3) variance from the local jurisdiction on design discharges
    E. Freeboard criteria may require an unusually expensive bridge or impracticable roadway elevation. A reasonable variance in criteria from the local jurisdiction may be available.

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4.0 Energy Dissipation

4.1 Overview

4.1.1 Introduction

The outlets of pipes and lined channels are points of critical erosion potential. Stormwater transported through man-made conveyance systems at design capacity generally reaches a velocity that exceeds the capacity of the receiving channel or area to resist erosion. To prevent scour at stormwater outlets, protect the outlet structure and minimize the potential for downstream erosion, a flow transition structure is needed to absorb the initial impact of flow and reduce the speed of the flow to a non-erosive velocity.

Energy dissipators are engineered devices such as rip-rap aprons or concrete baffles placed at the outlet of stormwater conveyances for the purpose of reducing the velocity, energy and turbulence of the discharged flow.

4.1.2 General Criteria

Erosion problems at culvert, pipe and engineered channel outlets are common. Determination of the flow conditions, scour potential, and channel erosion resistance shall be standard procedure for all designs.

Energy dissipators shall be employed whenever the velocity of flows leaving a stormwater management facility exceeds the erosion velocity of the downstream area channel system.

Energy dissipator designs will vary based on discharge specifics and tailwater conditions.

Outlet structures should provide uniform redistribution or spreading of the flow without excessive separation and turbulence.

4.1.3 Recommended Energy Dissipators

For many designs, the following outlet protection devices and energy dissipators provide sufficient protection at a reasonable cost:

- Riprap apron
- Riprap outlet basins
- Baffled outlets
- Grade Control Structures

This section focuses on the design on these measures. The reader is referred to the Federal Highway Administration Hydraulic Engineering Circular No. 14 entitled, Hydraulic Design of Energy Dissipators for Culverts and Channels, for the design procedures of other energy dissipators.

4.2 Symbols and Definitions

To provide consistency within this section as well as throughout this Manual, the symbols listed in Table 4.1 will be used. These symbols were selected because of their wide use. In some cases, the same symbol is used in existing publications for more than one definition. Where this occurs in this section, the symbol will be defined where it occurs in the text or equations.

Table 4.1 Symbols and Definitions

Symbol	Definition	Units
A	Cross-sectional area	ft^2
D	Height of box culvert	ft
d_{50}	Size of riprap	ft
d_w	Culvert width	ft
Fr	Froude Number	-
g	Acceleration of gravity	ft/s^2
h_s	Depth of dissipator pool	ft
L	Length	ft
L_a	Riprap apron length	ft
L_B	Overall length of basin	ft
L_s	Length of dissipator pool	ft
PI	Plasticity index	-
Q	Rate of discharge	cfs
S_v	Saturated shear strength	lbs/in^2
t	Time of scour	min.
t_c	Critical tractive shear stress	lbs/in^2
TW	Tailwater depth	ft
V_L	Velocity L feet from brink	ft/s
V_o	Normal velocity at brink	ft/s
V_o	Outlet mean velocity	ft/s
V_s	Volume of dissipator pool	ft^2
W_o	Diameter or width of culvert	ft
W_s	Width of dissipator pool	ft
y_e	Hydraulic depth at brink	ft
y_o	Normal flow depth at brink	ft

4.3 Design Guidelines

If outlet protection is required, choose an appropriate type. Suggested outlet protection facilities and applicable flow conditions (based on Froude number and dissipation velocity) are described below:

a. Riprap aprons may be used when the outlet Froude number (Fr) is less than or equal to 2.5. In general, riprap aprons prove economical for transitions from culverts to overland sheet flow at terminal outlets, but may also be used for transitions from culvert sections to stable channel sections. Stability of the surface at the termination of the apron should be considered.

b. Riprap outlet basins may also be used when the outlet Fr is less than or equal to 2.5. They are generally used for transitions from culverts to stable channels. Since riprap outlet basins function by creating a hydraulic jump to dissipate energy, performance is impacted by tailwater conditions.

c. Baffled outlets have been used with outlet velocities up to 50 feet per second. Practical application typically requires an outlet Fr between 1 and 9. Baffled outlets may be used at both terminal outlet and channel outlet transitions. They function by dissipating energy through impact and turbulence and are not significantly affected by tailwater conditions.

When outlet protection facilities are selected, appropriate design flow conditions and site-specific factors affecting erosion and scour potential, construction cost, and long-term durability should be considered.

If outlet protection is not provided, energy dissipation will occur through formation of a local scourhole. A cutoff wall will be needed at the discharge outlet to prevent structural undermining. The wall depth should be slightly greater than the computed scourhole depth, h_s. The scourhole should then be stabilized. If the scourhole is of such size that it will present maintenance, safety, or aesthetic problems, other outlet protection will be needed.

Evaluate the downstream channel stability and provide appropriate erosion protection if channel degradation is expected to occur. Figure 4.1 provides the riprap size recommended for use downstream of energy dissipators.

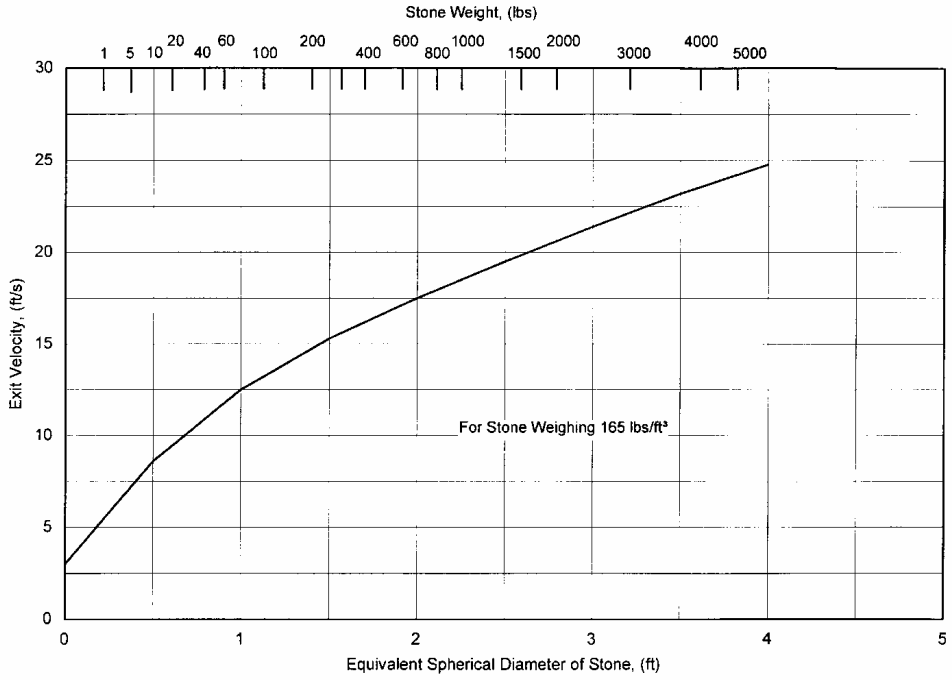


Figure 4.1 Riprap Size for Use Downstream of Energy Dissipator

(Source: Searcy, 1967)

4.4 Riprap Aprons

4.4.1 Description

A riprap-lined apron is a commonly used practice for energy dissipation because of its relatively low cost and ease of installation. A flat riprap apron can be used to prevent erosion at the transition from a pipe or box culvert outlet to a natural channel. Protection is provided primarily by having sufficient length and flare to dissipate energy by expanding the flow. Riprap aprons are appropriate when the culvert outlet Fr is less than or equal to 2.5.

4.4.2 Design Procedure

The procedure presented in this section is taken from USDA, SCS (1975). Two sets of curves, one for minimum and one for maximum tailwater conditions, are used to determine the apron size and the median riprap diameter, d_{50}. If tailwater conditions are unknown, or if both minimum and maximum conditions may occur, the apron should be designed to meet criteria for both. Although the design curves are based on round pipes flowing full, they can be used for partially full pipes and box culverts. The design procedure consists of the following steps:

If possible, determine tailwater conditions for the channel. If tailwater is less than one-half the discharge flow depth (pipe diameter if flowing full), minimum tailwater conditions exist and the curves in Figure 4.2 apply. Otherwise, maximum tailwater conditions exist and the curves in Figure 4.3 should be used.

Determine the correct apron length and median riprap diameter, d_{50}, using the appropriate curves from Figures 4.2 and 4.3. If tailwater conditions are uncertain, find the values for both minimum and maximum conditions and size the apron as shown in Figure 4.4.

a. For pipes flowing full:

   Use the depth of flow, d, which equals the pipe diameter, in feet, and design discharge, in cfs, to obtain the apron length, L_a, and median riprap diameter, d_{50}, from the appropriate curves.

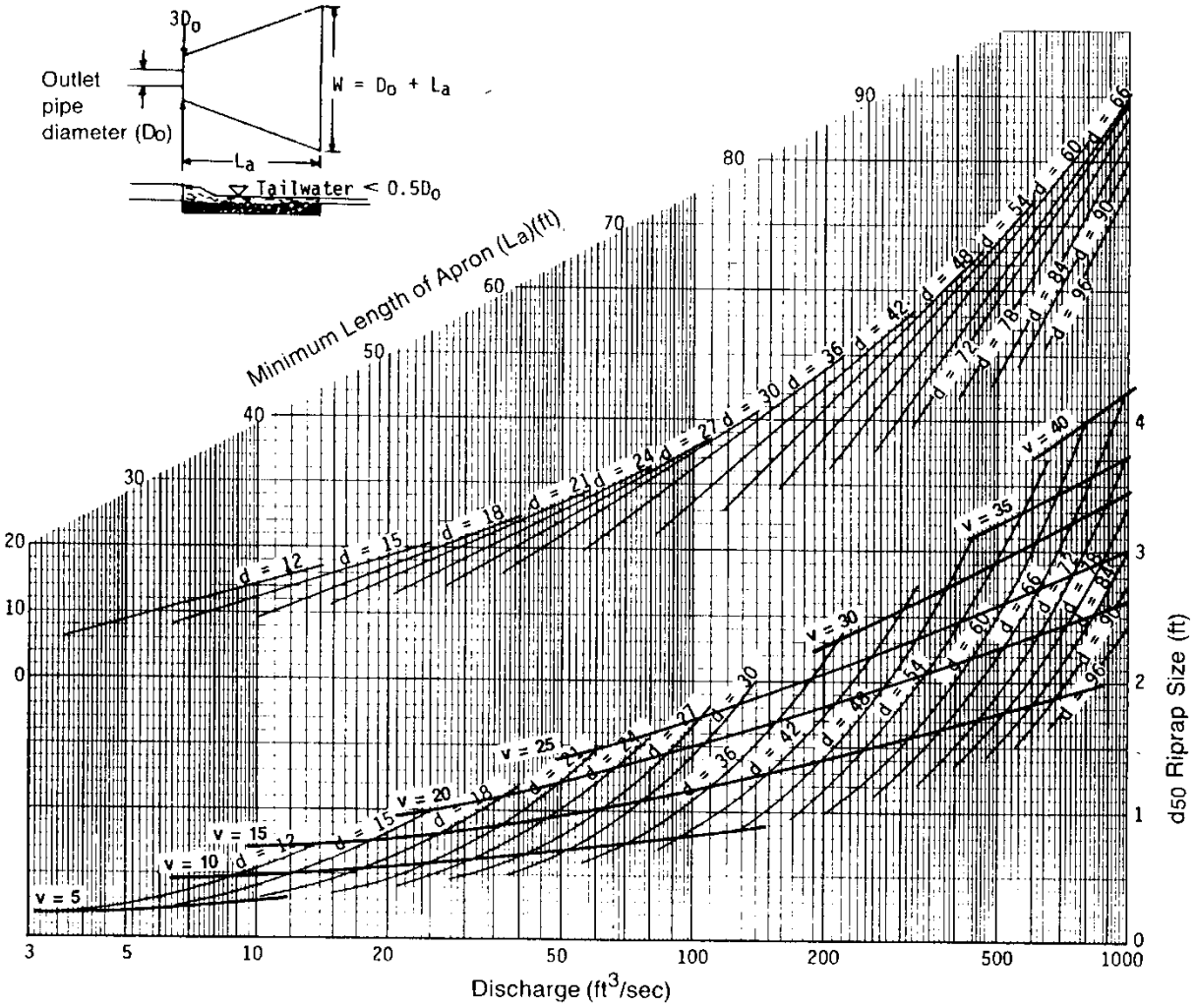
b. For pipes flowing partially full:

   Use the depth of flow, d, in feet, and velocity, v, in ft/s. On the lower portion of the appropriate figure, find the intersection of the d and v curves, and then find the riprap median diameter, d_{50}, from the scale on the right. From the lower d and v intersection point, move vertically to the upper curves until intersecting the curve for the correct flow depth, d. Find the minimum apron length, L_a, from the scale on the left.

c. For box culverts:

   Use the depth of flow, d, in feet, and velocity, v, in feet/second. On the lower portion of the appropriate figure, find the intersection of the d and v curves, and then find the riprap median diameter, d_{50}, from the scale on the right. From the lower d and v intersection point, move vertically to the upper curve until intersecting the curve equal to the flow depth, d. Find the minimum apron length, L_a, using the scale on the left.

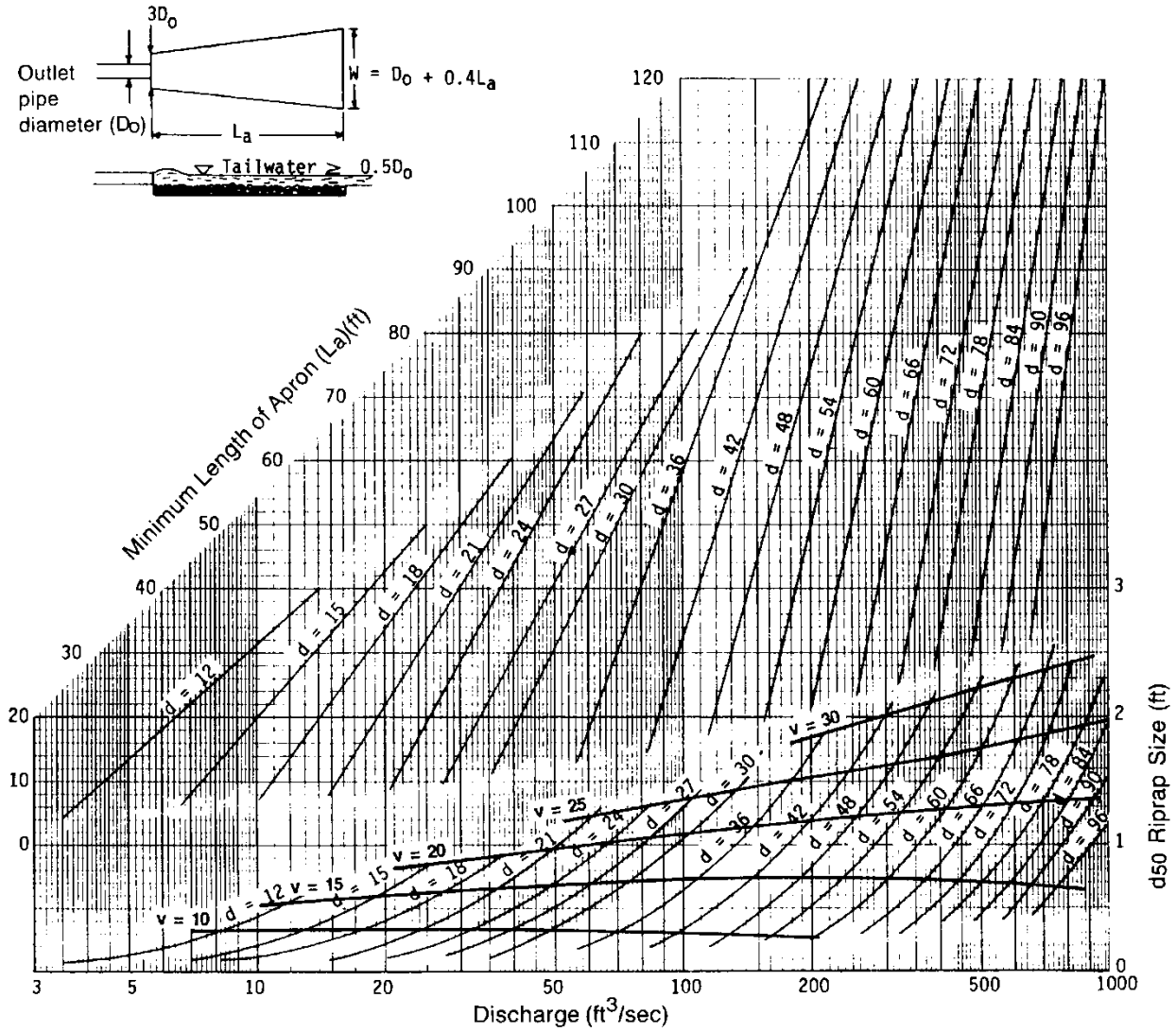
If tailwater conditions are uncertain, the median riprap diameter should be the larger of the values for minimum and maximum conditions. The dimensions of the apron will be as shown in Figure 4.4. This will provide protection under either of the tailwater conditions.



Curves may not be extrapolated.

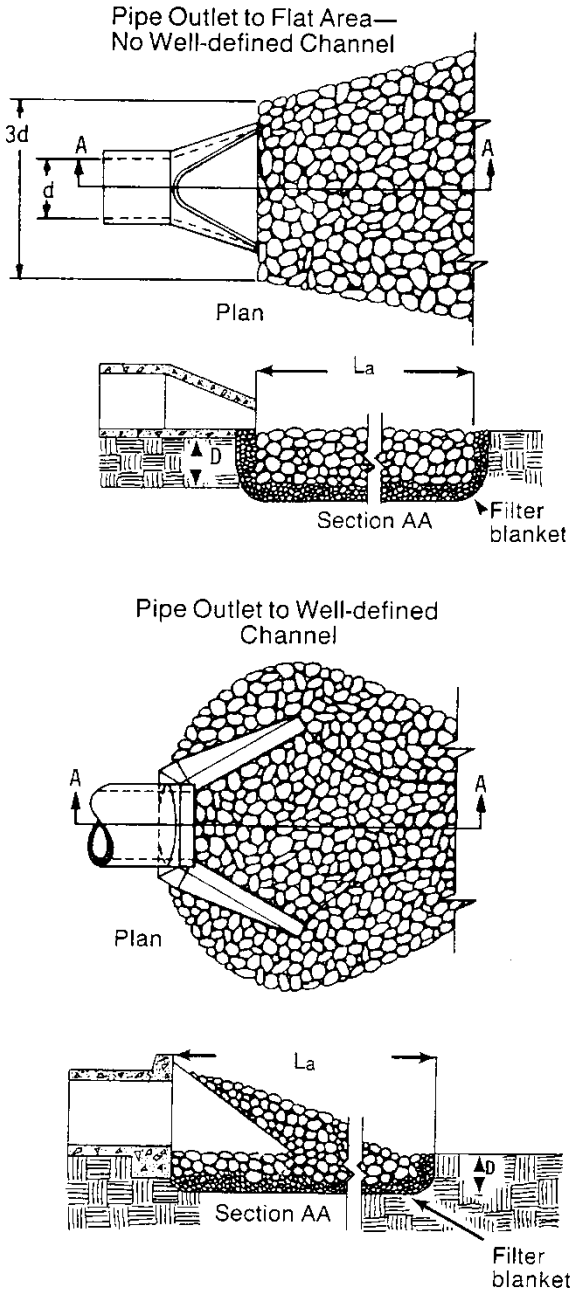
Figure 4.2 Design of Riprap Apron under Minimum Tailwater Conditions

(Source: USDA, SCS, 1975)



Curves may not be extrapolated.

Figure 4.3 Design of Riprap Apron under Maximum Tailwater Conditions
(Source: USDA, SCS, 1975)



Notes

1. L_a is the length of the riprap apron.
2. D = 1.5 times the maximum stone diameter but not less than 6".
3. In a well-defined channel extend the apron up the channel banks to an elevation of 6" above the maximum tailwater depth or to the top of the bank, whichever is less.
4. A filter blanket or filter fabric should be installed between the riprap and soil foundation.

Figure 4.4 Riprap Apron

(Source: Manual for Erosion and Sediment Control in Georgia, 1996)

4.4.3 Design Considerations

The following items should be considered during riprap apron design:

The maximum stone diameter should be 1.5 times the median riprap diameter.
d_{max} = 1.5 x d_{50} , d_{50} = the median stone size in a well-graded riprap apron.

The riprap thickness should be 1.5 times the maximum stone diameter or 6 inches, whichever is greater.
Apron thickness = 1.5 x d_{max}
(Apron thickness may be reduced to 1.5 x d_{50} when an appropriate filter fabric is used under the apron.)

The apron width at the discharge outlet should be at least equal to the pipe diameter or culvert width, d_w. Riprap should extend up both sides of the apron and around the end of the pipe or culvert at the discharge outlet at a maximum slope of 2:1 and a height not less than the pipe diameter or culvert height, and should taper to the flat surface at the end of the apron.

If there is a well-defined channel, the apron length should be extended as necessary so the downstream apron width is equal to the channel width. The sidewalls of the channel should not be steeper than 2:1.

If the ground slope downstream of the apron is steep, channel erosion may occur. The apron should be extended as necessary until the slope is gentle enough to prevent further erosion.

The potential for vandalism should be considered if the rock is easy to carry. If vandalism is a possibility, the rock size must be increased or the rocks held in place using concrete or grout.

4.4.4 Example Designs

Example 1 Riprap Apron Design for Minimum Tailwater Conditions

A flow of 280 cfs discharges from a 66-in pipe with a tailwater of 2 ft above the pipe invert. Find the required design dimensions for a riprap apron.

Minimum tailwater conditions = 0.5 d_o, d_o = 66 in = 5.5 ft; therefore, 0.5 d_o = 2.75 ft.

Since TW = 2 ft is less than 2.75 ft, use Figure 4.2 for minimum tailwater conditions.

By Figure 4.2, the apron length, L_a, and median stone size, d_{50}, are 38 ft and 1.2 ft, respectively.

The downstream apron width equals the apron length plus the pipe diameter:

$W = d + L_a = 5.5 + 38 = 43.5$ ft

Maximum riprap diameter is 1.5 times the median stone size:

1.5 (d_{50}) = 1.5 (1.2) = 1.8 ft

Riprap depth = 1.5 (d_{max}) = 1.5 (1.8) = 2.7 ft.

Example 2 Riprap Apron Design for Maximum Tailwater Conditions

A concrete box culvert 5.5 ft high and 10 ft wide conveys a flow of 600 cfs at a depth of 5.0 ft. Tailwater depth is 5.0 ft above the culvert outlet invert. Find the design dimensions for a riprap apron.

Minimum tailwater conditions = 0.5 d_o, d_o = 0.5 (5.0) = 2.5 ft.

Since TW = 5.0 ft is greater than 2.5 ft, use Figure 4.3 for maximum tailwater conditions.

$v = Q/A = 600/[(5)(10)] = 12$ ft/s

On Figure 4.3, at the intersection of the curve, $d_o = 60$ in and $v = 12$ ft/s, $d_{50} = 0.4$ ft. Reading up to the intersection with $d = 60$ in, find $L_a = 40$ ft.

Apron width downstream = $d_w + 0.4\ L_a = 10 + 0.4\ (40) = 26$ ft.

Maximum stone diameter = $1.5\ d_{50} = 1.5\ (0.4) = 0.6$ ft.

Riprap depth = $1.5\ d_{max} = 1.5\ (0.6) = 0.9$ ft.

4.5 Riprap Basins

4.5.1 Description

Another method to reduce the exit velocities from stormwater outlets is through the use of a riprap basin. A riprap outlet basin is a preshaped scourhole lined with riprap that functions as an energy dissipator by forming a hydraulic jump.

4.5.2 Basin Features

General details of the basin recommended in this section are shown in Figure 4.5. Principal features of the basin are:

The basin is preshaped and lined with riprap of median size (d_{50}).

The floor of the riprap basin is constructed at an elevation of h_s below the culvert invert. The dimension h_s is the approximate depth of scour that would occur in a thick pad of riprap of size d_{50} if subjected to design discharge. The ratio of h_s to d_{50} of the material should be between 2 and 4.

The length of the energy dissipating pool is 10 x h_s or 3 x W_o, whichever is larger. The overall length of the basin is 15 x h_s or 4 x W_o, whichever is larger.

4.5.3 Design Procedure

The following procedure should be used for the design of riprap basins.

Estimate the flow properties at the brink (outlet) of the culvert. Establish the outlet invert elevation such that $TW/y_o \leq 0.75$ for the design discharge.

For subcritical flow conditions (culvert set on mild or horizontal slope) use Figure 4.6 or Figure 4.7 to obtain y_o/D, then obtain V_o by dividing Q by the wetted area associated with y_o. D is the height of a box culvert. If the culvert is on a steep slope, V_o will be the normal velocity obtained by using the Manning equation for appropriate slope, section, and discharge.

For streambank protection, compute the Froude number for brink conditions with $y_e = (A/2)^{1.5}$. Select d_{50}/y_e appropriate for locally available riprap (usually the most satisfactory results will be obtained if $0.25 < d_{50}/y_e < 0.45$). Obtain h_s/y_e from Figure 4.8, and check to see that $2 < h_s/d_{50} < 4$. Recycle computations if h_s/d_{50} falls out of this range.

Size basin as shown in Figure 4.5.

Where allowable dissipator exit velocity is specified:

a. Determine the average normal flow depth in the natural channel for the design discharge.

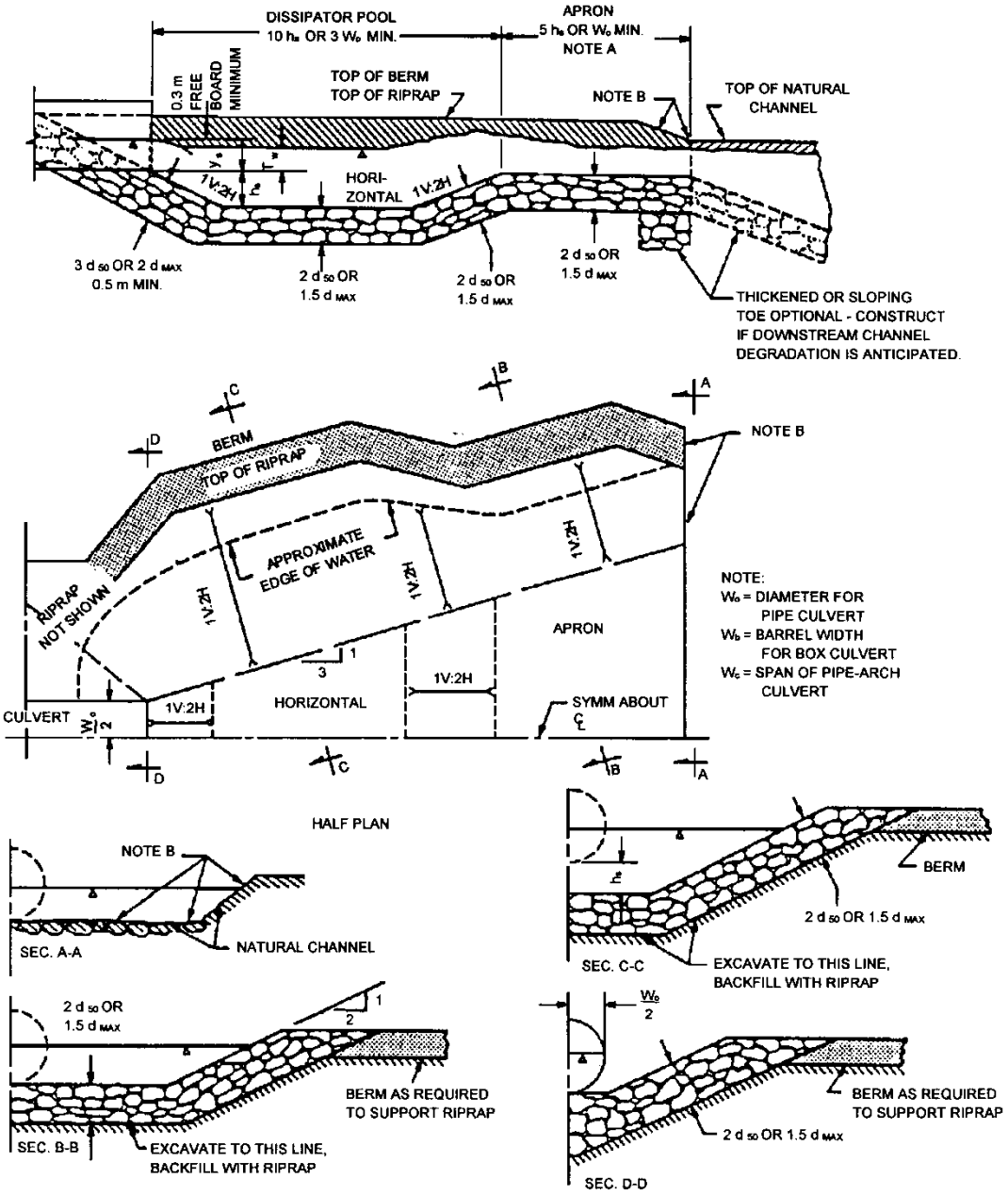
b. Extend the length of the energy basin (if necessary) so the width of the energy basin at section A-A, Figure 4.5, times the average normal flow depth in the natural channel is approximately equal to the design discharge divided by the specified exit velocity.

In the exit region of the basin, the walls and apron of the basin should be warped (or transitioned) so the cross section of the basin at the exit conforms to the cross section of the natural channel. Abrupt transition of surfaces should be avoided to minimize separation zones and resultant eddies.

If high tailwater is a possibility and erosion protection is necessary for the downstream channel, the following design procedure is suggested:

- Design a conventional basin for low tailwater conditions in accordance with the instructions above.
- Estimate centerline velocity at a series of downstream cross sections using the information shown in Figure 4.9.
- Shape downstream channel and size riprap using Figure 4.1 and the stream velocities obtained above.

Material, construction techniques, and design details for riprap should be in accordance with specifications in the Federal Highway publication HEC No. 11 entitled Use of Riprap for Bank Protection.



NOTE A - IF EXIT VELOCITY OF BASIN IS SPECIFIED, EXTEND BASIN AS REQUIRED TO OBTAIN SUFFICIENT CROSS-SECTIONAL AREA AT SECTION A-A SUCH THAT Q/(CROSS SECTION AREA AT SEC. A-A) = SPECIFIED EXIT VELOCITY.

NOTE B - WARP BASIN TO CONFORM TO NATURAL STREAM CHANNEL. TOP OF RIPRAP IN FLOOR OF BASIN SHOULD BE AT THE SAME ELEVATION OR LOWER THAN NATURAL CHANNEL BOTTOM AT SEC. A-A.

Figure 4.5 Details of Riprap Outlet Basin

(Source: HEC-14, 1983)

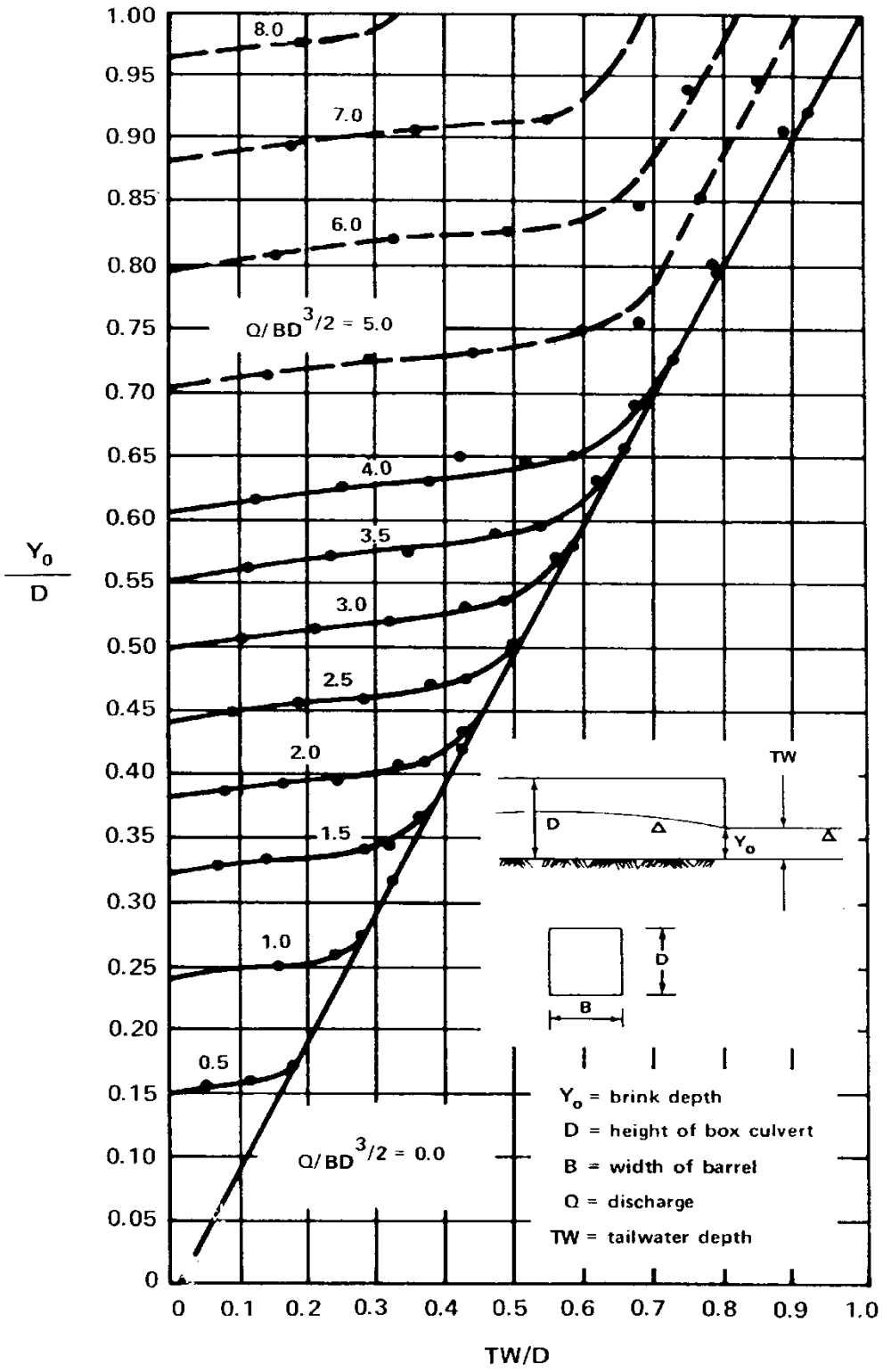


Figure 4.6 Dimensionless Rating Curves for the Outlets of Rectangular Culverts on Horizontal and Mild Slopes

(Source: USDOT, FHWA, HEC-14, 1983)

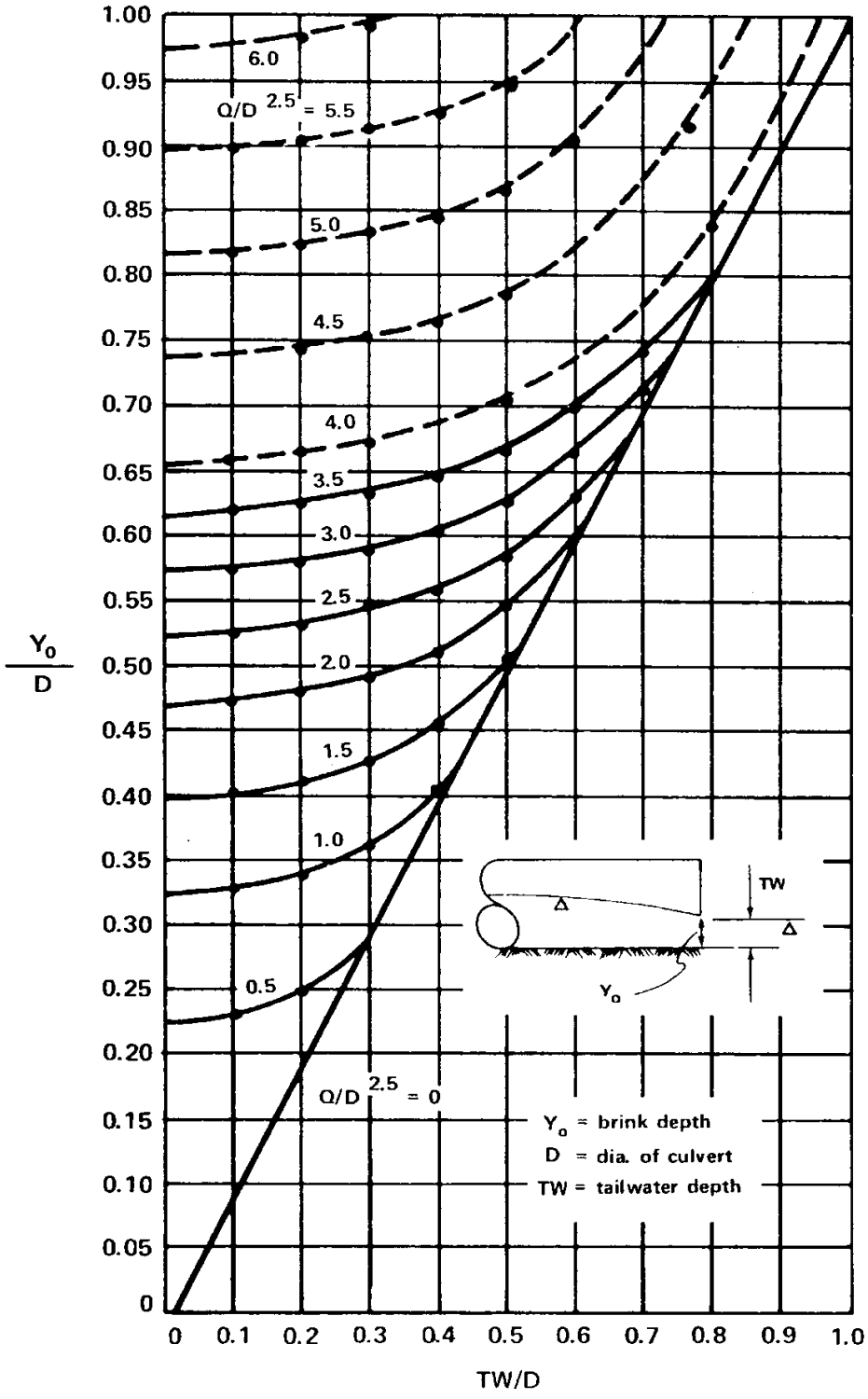


Figure 4.7 Dimensionless Rating Curves for the Outlets of Circular Culverts on Horizontal and Mild Slopes

(Source: USDOT, FHWA, HEC-14, 1983)

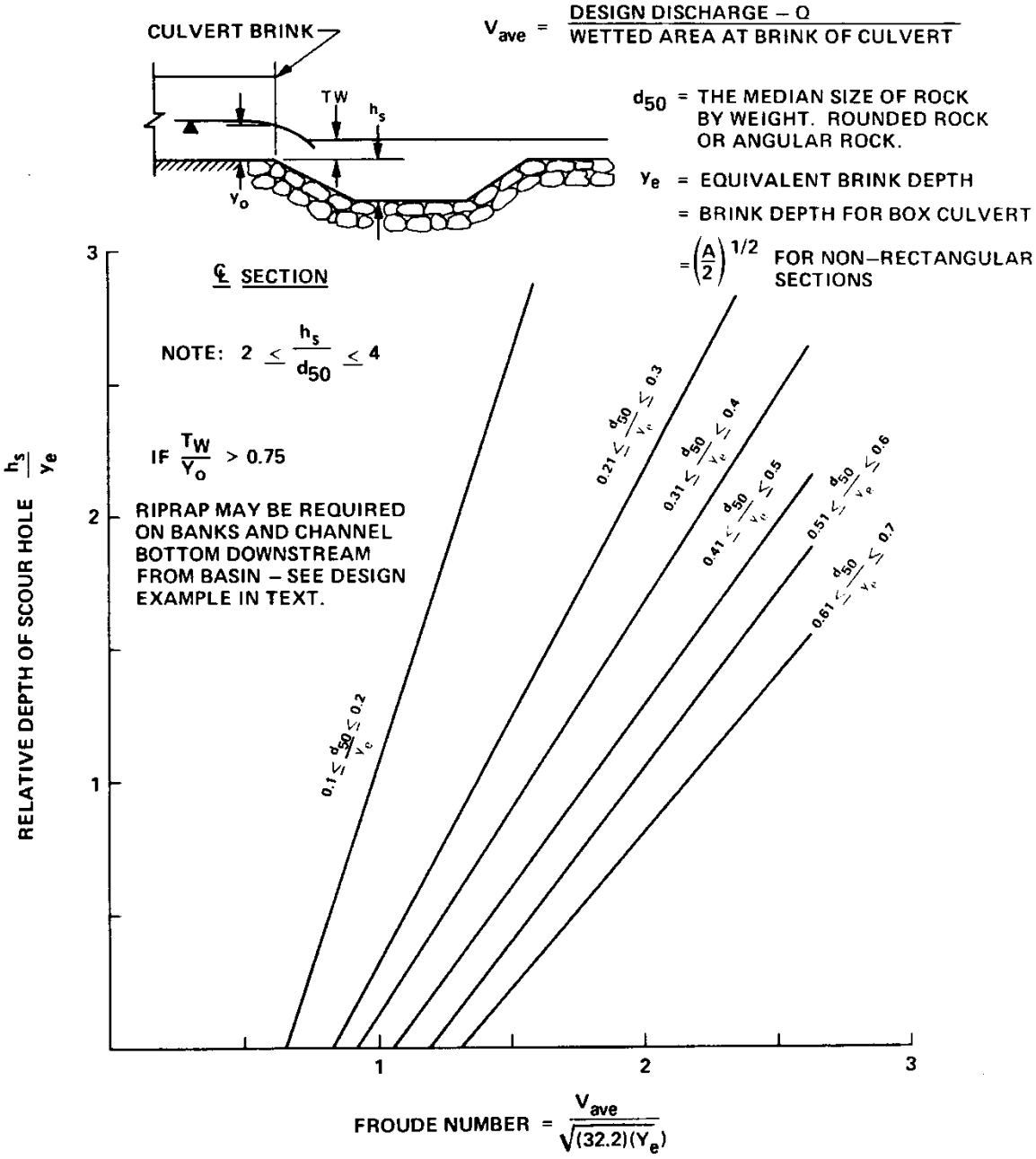


Figure 4.8 Relative Depth of Scour Hole Versus Froude Number at Brink of Culvert with Relative Size of Riprap as a Third Variable

(Source: USDOT, FHWA, HEC-14, 1983)

4.5.4 Design Considerations

Riprap basin design should include consideration of the following:

The dimensions of a scourhole in a basin constructed with angular rock can be approximately the same as the dimensions of a scourhole in a basin constructed of rounded material when rock size and other variables are similar.

When the ratio of tailwater depth to brink depth, TW/y_o, is less than 0.75 and the ratio of scour depth to size of riprap, h_s/d_{50}, is greater than 2.0, the scourhole should function very efficiently as an energy dissipator. The concentrated flow at the culvert brink plunges into the hole, a jump forms against the downstream extremity of the scourhole, and flow is generally well dispersed leaving the basin.

The mound of material formed on the bed downstream of the scourhole contributes to the dissipation of energy and reduces the size of the scourhole; that is, if the mound from a stable scoured basin is removed and the basin is again subjected to design flow, the scourhole will enlarge.

For high tailwater basins (TW/y_o greater than 0.75), the high velocity core of water emerging from the culvert retains its jet-like character as it passes through the basin and diffuses similarly to a concentrated jet diffusing in a large body of water. As a result, the scourhole is much shallower and generally longer. Consequently, riprap may be required for the channel downstream of the rock-lined basin.

It should be recognized that there is a potential for limited degradation to the floor of the dissipator pool for rare event discharges. With the protection afforded by the $2(d_{50})$ thickness of riprap, the heavy layer of riprap adjacent to the roadway prism, and the apron riprap in the downstream portion of the basin, such damage should be superficial.

See Standards in the in FHWA HEC No. 11 for details on riprap materials and use of filter fabric.

Stability of the surface at the outlet of a basin should be considered using the methods for open channel flow as outlined in *Section 3.2.*

4.5.5 Example Designs

Following are some example problems to illustrate the design procedures outlined.

Example 1

Given: Box culvert - 8 ft by 6 ft; Design Discharge Q = 800 cfs
Supercritical flow in culvert; Normal flow depth = brink depth
Y_o = 4 ft; Tailwater depth TW = 2.8 ft

Find: Riprap basin dimensions for these conditions

Solution: Definition of terms in Steps 1 through 5 can be found in Figures 4.5 and 4.8.

$y_o = y_e$ for rectangular section; therefore, with y_o given as 4 ft, y_e = 4 ft.

$V_o = Q/A = 800/(4 \times 8) = 25$ ft/s

Froude Number = $Fr = V/(g \times y_e)^{0.5}$ ($g = 32.2$ ft/s^2)
$Fr = 25/(32.2 \times 4)^{0.5} = 2.20 < 2.5$ O.K.

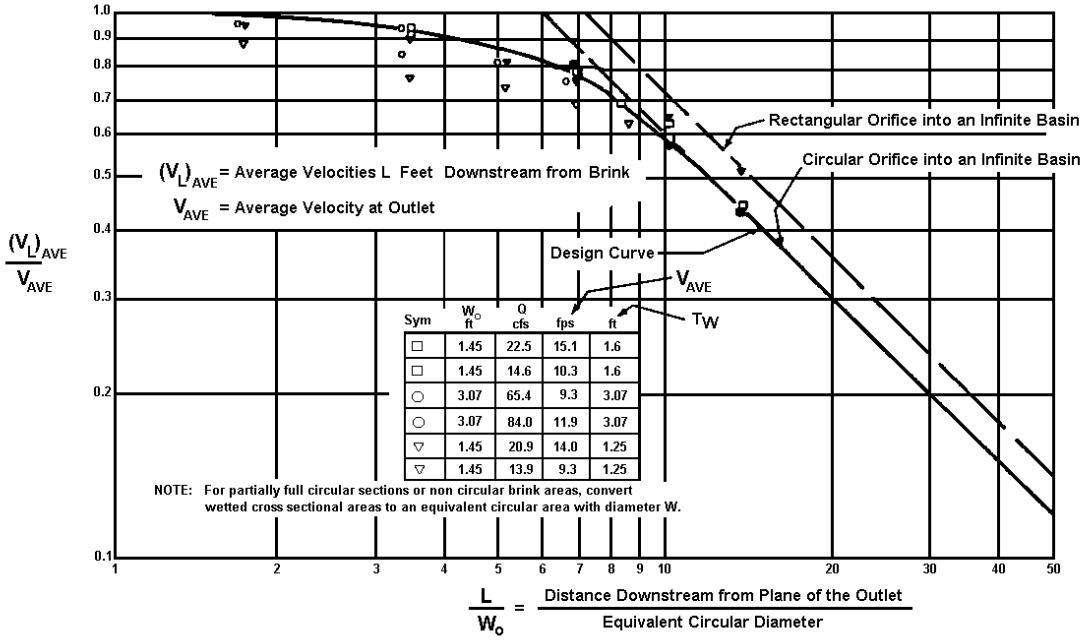


Figure 4.9 Distribution of Centerline Velocity for Flow from Submerged Outlets to Be Used for Predicting Channel Velocities Downstream from Culvert Outlet Where High Tailwater Prevails

(Source: USDOT, FHWA, HEC-14, 1983)

$TW/y_e = 2.8/4.0 = 0.7$ Therefore, $TW/y_e < 0.75$ OK

Try $d_{50}/y_e = 0.45$, $d_{50} = 0.45 \times 4 = 1.80$ ft
From Figure 4.8, $h_s/y_e = 1.6$, $h_s = 4 \times 1.6 = 6.4$ ft
$h_s/d_{50} = 6.4/1.8 = 3.6$ ft, $2 < h_s/d_{50} < 4$ OK

$L_s = 10 \times h_s = 10 \times 6.4 = 64$ ft (L_s = length of energy dissipator pool)
L_s min $= 3 \times W_o = 3 \times 8 = 24$ ft; therefore, use $L_s = 64$ ft

$L_B = 15 \times h_s = 15 \times 6.4 = 96$ ft (L_B = overall length of riprap basin)
L_B min $= 4 \times W_o = 4 \times 8 = 32$ ft; therefore, use $L_B = 96$ ft

Thickness of riprap: On the approach $= 3 \times d_{50} = 3 \times 1.8 = 5.4$ ft

Remainder $= 2 \times d_{50} = 2 \times 1.8 = 3.6$ ft
Other basin dimensions designed according to details shown in Figure 4.5.

Example 2

Given: Same design data as Example 1 except:

Tailwater depth TW = 4.2 ft
Downstream channel can tolerate only 7 ft/s discharge

Find: Riprap basin dimensions for these conditions

Solutions: Note -- High tailwater depth, $TW/y_o = 4.2/4 = 1.05 > 0.75$

From Example 1: d_{50} = 1.8 ft, h_s = 6.4 ft, L_s = 64 ft, L_B = 96 ft.

Design riprap for downstream channel. Use Figure 4.9 for estimating average velocity along the channel. Compute equivalent circular diameter D_e for brink area from:

$A = 3.14D_e^2/4 = y_o \times W_o = 4 \times 8 = 32$ ft^2

$D_e = ((32 \times 4)/3.14)^{0.5} = 6.4$ ft

V_o = 25 ft/s (From Example 1)

Set up the following table:

				Rock Size
L/D_e	**L (ft)**	**V_L/V_o**	**v_1 (ft/s)**	**d_{50} (ft)**
(Assume)	(Compute)	(Fig. 9)	(Fig. 1)	$D_e = W_o$
10	64	0.59	14.7	1.4
15*	96	0.37	9.0	0.6
20	128	0.30	7.5	0.4
21	135	0.28	7.0	0.4

*L/W_o is on a logarithmic scale so interpolations must be done logarithmically.

Riprap should be at least the size shown but can be larger. As a practical consideration, the channel can be lined with the same size rock used for the basin. Protection must extend at least 135 ft downstream from the culvert brink. Channel should be shaped and riprap should be installed in accordance with details shown in the HEC No. 11 publication.

Example 3

Given: 6-ft diameter CMC
Design discharge Q = 135 cfs
Slope channel S_o = 0.004
Manning's n = 0.024
Normal depth in pipe for Q = 135 cfs is 4.5 ft
Normal velocity is 5.9 ft/s
Flow is subcritical
Tailwater depth TW = 2.0 ft

Find: Riprap basin dimensions for these conditions.

Solution:

Determine y_o and V_o

$Q/D^{2.5} = 135/6^{2.5} = 1.53$

$TW/D = 2.0/6 = 0.33$

From Figure 4.7, $y_o/D = 0.45$

$y_o = .45 \times 6 = 2.7$ ft

$TW/y_o = 2.0/2.7 = 0.74$ $TW/y_o < 0.75$ O.K.

Determine Brink Area (A) for $y_o/D = 0.45$

From Uniform Flow in Circular Sections Table (from Table 3.7)

For $y_o/D = d/D = 0.45$

$A/D^2 = 0.3428$; therefore, $A = 0.3428 \times 6^2 = 12.3$ ft^2

$V_o = Q/A = 135/12.3 = 11.0$ ft/s

For Froude number calculations at brink conditions,
$y_e = (A/2)^{1/2} = (12.3/2)^{1/2} = 2.48$ ft

Froude number = $Fr = V_o/(32.2 \times y_e)^{1/2} = 11/(32.2 \times 2.48)^{1/2} = 1.23 < 2.5$ OK

For most satisfactory results, $0.25 < d_{50}/y_e < 0.45$
Try $d_{50}/y_e = 0.25$
$d_{50} = 0.25 \times 2.48 = 0.62$ ft
From Figure 4.8, $h_s/y_e = 0.7$; therefore, $h_s = 0.7 \times 2.48 = 1.74$ ft

Uniform Flow in Circular Sections Flowing Partly Full (From *Section 3.2.4*)
Check: $h_s/d_{50} = 1.74/0.62 = 2.8$, $2 < h_s/d_{50} < 4$ OK

$L_s = 10 \times h_s = 10 \times 1.74 = 17.4$ ft or $L_s = 3 \times W_o = 3 \times 6 = 18$ ft;
therefore, use $L_s = 17.4$ ft

$L_B = 15 \times h_s = 15 \times 1.74 = 26.1$ ft or $L_B = 4 \times W_o = 4 \times 6 = 24$ ft;
therefore, use $L_B = 26.1$ ft

$d_{50} = 0.62$ ft or use $d_{50} = 8$ in

Other basin dimensions should be designed in accordance with details shown on Figure 4.5. Figure 4.10 is provided as a convenient form to organize and present the results of riprap basin designs.

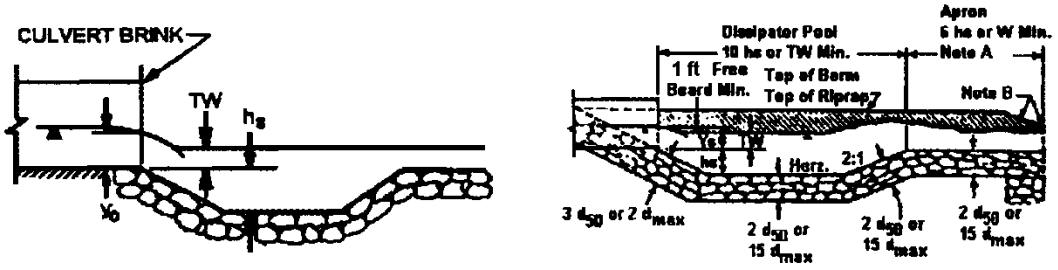
Note: When using the design procedure outlined in this section, it is recognized that there is some chance of limited degradation of the floor of the dissipator pool for rare event discharges. With the protection afforded by the $3 \times d_{50}$ thickness of riprap on the approach and the $2 \times d_{50}$ thickness of riprap on the basin floor and the apron in the downstream portion of the basin, the damage should be superficial.

RIPRAP BASIN

Project No. _____

Designer _____ Date _____

Reviewer _____ Date _____



DESIGN VALUES	TRIAL 1	FINAL TRIAL
Equi. Depth, d_E		
D_{50}/d_E		
D_{50}		
Froude No., Fr		
h_s/d_E		
h_s		
h_s/D_{50}		
$2 < h_s/D_{50} < 4$		

BASIN DIMENSIONS		FEET	
Pool length is the larger of:	$10h_s$		
	$3W_o$		
Basin length is the larger of:	$15h_s$		
	$4W_o$		
Approach Thickness	$3D_{50}$		
Basin Thickness	$2D_{50}$		

TAILWATER CHECK	
Tailwater, TW	
Equivalent depth, d_E	
TW/d_E	
IF $TW/d_E > 0.75$, calculate riprap downstream	
$D_E=(4A_c/\pi)^{0.5}$	

DOWNSTREAM RIPRAP				
L/D_E	L	V_L/V_o	V_L	D_{50}

Figure 4.10 Riprap Basin Design Form

(Source: USDOT, FHWA, HEC-14, 1983)

4.6 Baffled Outlets

4.6.1 Description

The baffled outlet (also known as the Impact Basin - USBR Type VI) is a boxlike structure with a vertical hanging baffle and an end sill, as shown in Figure 4.11. Energy is dissipated primarily through the impact of the water striking the baffle and, to a lesser extent, through the resulting turbulence. This type of outlet protection has been used with outlet velocities up to 50 feet per second and with Froude numbers from 1 to 9. Tailwater depth is not required for adequate energy dissipation, but a tailwater will help smooth the outlet flow.

4.6.2 Design Procedure

The following design procedure is based on physical modeling studies summarized from the U.S. Department of Interior (1978). The dimensions of a baffled outlet as shown in Figure 4.11 should be calculated as follows:

Determine input parameters, including:

- h = Energy head to be dissipated, in ft (can be approximated as the difference between channel invert elevations at the inlet and outlet)
- Q = Design discharge (cfs)
- v = Theoretical velocity (ft/s = 2gh)
- A = Q/v = Flow area (ft^2)
- d = $A^{0.5}$= Representative flow depth entering the basin (ft) *assumes square jet*
- Fr = $v/(gd)^{0.5}$ = Froude number, dimensionless
- g = Acceleration of gravity (32.2 ft/s)

Calculate the minimum basin width, W, in ft, using the following equation.

$$\mathbf{W/d = 2.88Fr^{0.566} \text{ or } W = 2.88dFr^{0.566}} \qquad \mathbf{(4.1)}$$

Where:

- W = minimum basin width (ft)
- d = depth of incoming flow (ft)
- Fr = $v/(gd)^{0.5}$ = Froude number, dimensionless

The limits of the W/d ratio are from 3 to 10, which corresponds to Froude numbers 1 and 9. If the basin is much wider than W, flow will pass under the baffle and energy dissipation will not be effective.

Calculate the other basin dimensions as shown in Figure 4.11, as a function of W. Construction drawings for selected widths are available from the U.S. Department of the Interior (1978).

Calculate required protection for the transition from the baffled outlet to the natural channel based on the outlet width. A riprap apron should be added of width W, length W (or a 5-foot minimum), and depth f (W/6). The side slopes should be 1.5:1, and median rock diameter should be at least W/20.

Calculate the baffled outlet invert elevation based on expected tailwater. The maximum distance between expected tailwater elevation and the invert should be b + f or some flow will go over the baffle with no energy dissipation. If the tailwater is known and fairly controlled, the baffled outlet invert should be a distance, b/2 + f, below the calculated tailwater elevation. If tailwater is uncontrolled, the baffled outlet invert should be a distance, f, below the downstream channel invert.

Calculate the outlet pipe diameter entering the basin assuming a velocity of 12 ft/s flowing full.

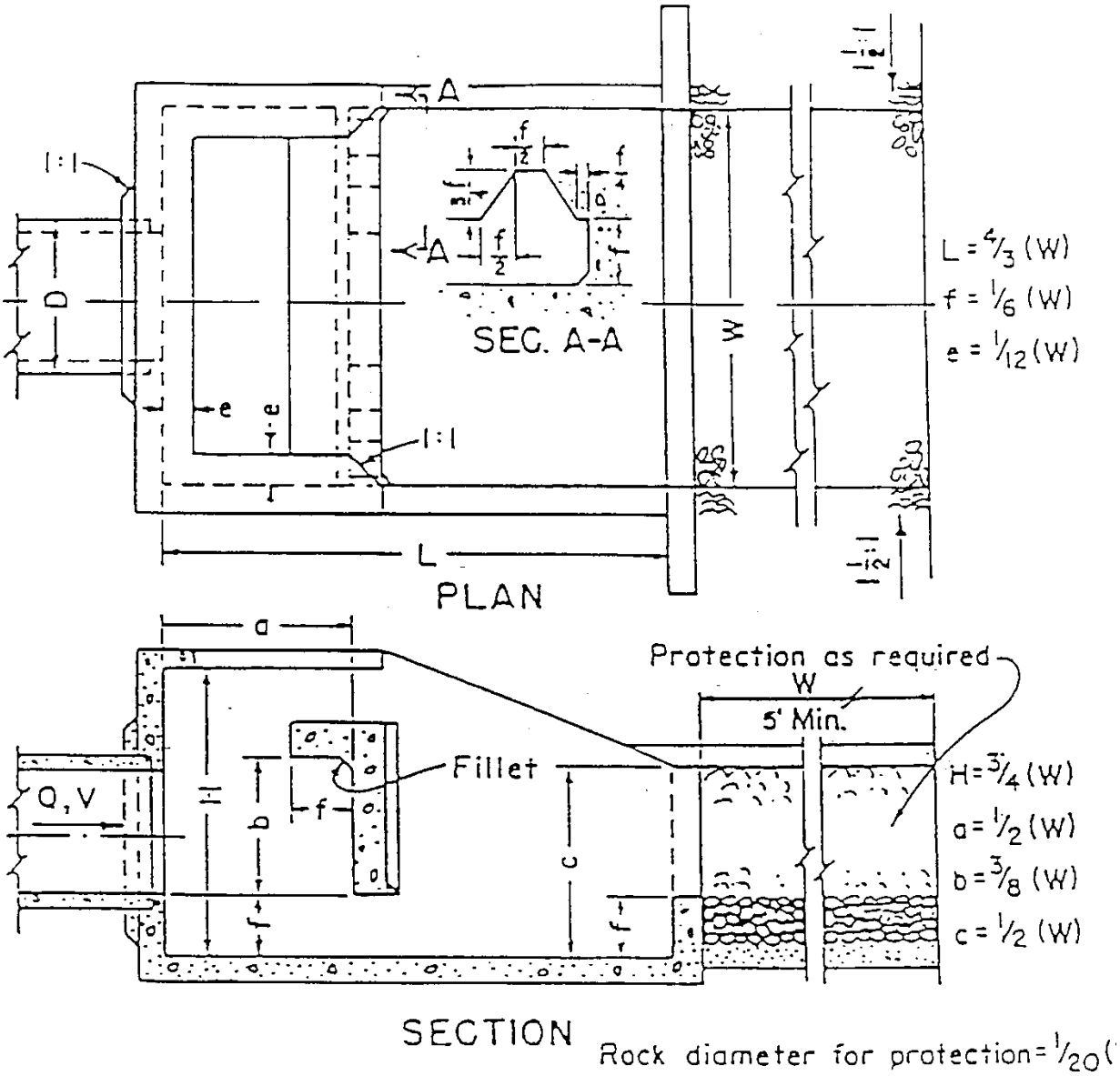


Figure 4.11 Schematic of Baffled Outlet

(Source: U.S. Dept. of the Interior, 1978)

If the entrance pipe slopes steeply downward, the outlet pipe should be turned horizontal for at least 3 ft before entering the baffled outlet.

If it is possible that both the upstream and downstream ends of the pipe will be submerged, provide an air vent approximately 1/6 the pipe diameter near the upstream end to prevent pressure fluctuations and possible surging flow conditions.

4.6.3 Example Design

A cross-drainage pipe structure has a design flow rate of 150 cfs, a head, h, of 15 ft from invert of pipe, and a tailwater depth, TW, of 3 ft above ground surface. Find the baffled outlet basin dimensions and inlet pipe requirements.

1. Compute the theoretical velocity from
   $v = (2gh)^{0.5} = [2(32.2 \text{ ft/sec}^2)(15 \text{ ft})]^{0.5} = 31.1$ ft/s
   This is less than 50 ft/s, so a baffled outlet is suitable.

2. Determine the flow area using the theoretical velocity as follows:
   $A = Q/v = 150 \text{ cfs}/31.1 \text{ ft/sec} = 4.8 \text{ ft}^2$

3. Compute the flow depth using the area from Step 2.
   $d = (A)^{0.5} = (4.8 \text{ ft}^2)^{0.5} = 2.12$ ft

4. Compute the Froude number using the results from Steps 1 and 3.
   $Fr = v/(gd)^{0.5} = 31.1 \text{ ft/sec}/[(32.2 \text{ ft/sec}^2)(2.12 \text{ ft})]^{0.5} = 3.8$

5. Determine the basin width using Equation 4.1 with the Froude number from Step 4.
   $W = 2.88 \, dFr^{0.566} = 2.88 \, (2.12) \, (3.8)^{0.566} = 13.0$ ft (minimum)
   Use 13 ft as the design width.

6. Compute the remaining basin dimensions (as shown in Figure 4.11):
   L = 4/3 (W) = 17.3 ft, use L = 17 ft, 4 in
   f = 1/6 (W) = 2.17 ft, use f = 2 ft, 2 in
   e = 1/12 (W) = 1.08 ft, use e = 1 ft, 1 in
   H = 3/4 (W) = 9.75 ft, use H = 9 ft, 9 in
   a = 1/2 (W) = 6.5 ft, use a = 6 ft, 6 in
   b = 3/8 (W) = 4.88 ft, use b = 4 ft, 11 in
   c = 1/2 (W) = 6.5 ft, use c = 6 ft, 6 in

   Baffle opening dimensions would be calculated as shown in Figure 4.11.

7. Basin invert should be at b/2 + f below tailwater, or
   (4 ft, 11 in)/2 + 2 ft, 2 in = 4.73 ft
   Use 4 ft 8 in; therefore, invert should be 2 ft, 8 in below ground surface.

8. The riprap transition from the baffled outlet to the natural channel should be 13 ft long by 13 ft wide by 2 ft, 2 in deep (W x W x f). Median rock diameter should be of diameter W/20, or about 8 in.

9. Inlet pipe diameter should be sized for an inlet velocity of about 12 ft/s.
   $(3.14d)^2/4 = Q/v$; $d = [(4Q)/3.14v)]^{0.5} = [(4(150 \text{ cfs})/3.14(12 \text{ ft/sec})]^{0.5} = 3.99$ ft
   Use 48-in pipe. If a vent is required, it should be about 1/6 of the pipe diameter or 8 in.

4.7 Grade Control Structures

When channels are relocated through non-stable soils and stream gradients are increased, the stream bottom may degrade or dig itself deeper. This can cause bank instability, increased upstream scouring, and sloughing of natural slopes. The U.S. Soil Conservation Services (SCS) requires that streambed stability be maintained in any of its stream projects. This can be accomplished by grade stabilization structures; in essence a series of low-head weirs.

If designed and constructed with ecological values in mind, these structures can double as habitat enhancement devices. If improperly planned however, they can atually degrade habitat values. The most productive method of installing these structures is to use low weirs that pool water just a short distance (approximately 100 feet) upstream. A plunge pool will form just below the structures, and a riffle area should develop below this pool. The next structure should be located downstream a sufficient distance to avoid impounding the riffle area below the pool at the base of the upstream weir.

Specific construction requirements and techniques can be obtained from the SCS or other agencies upon request. The intent of this general discussion of grade stabilization structures is to promote consideration of such measures early in the planning process.

Source: US Army Corp of Engineers, Nashville District, *"Mitigating the Impacts of Stream Alterations"*, unkn.

City of Denton

City Hall
215 E. McKinney Street
Denton, Texas
www.cityofdenton.com

AGENDA INFORMATION SHEET

DEPARTMENT: Department of Development Services

DCM: Cassey Ogden

DATE: January 12, 2026

SUBJECT

Receive a report, hold a discussion, and give staff direction regarding proposed Code amendments related to the building design of single-family homes and other residential use types.

BACKGROUND

The Denton Development Code (DDC) Subchapter 7.10: Site and Building Design provides architectural and site design regulations for a variety of uses for the purpose of providing variety and visual interest in exterior building design, promoting high-quality development, and enhancing streetscapes within the City of Denton. Specifically, Subsection 7.10.3 addresses building and site design for single-family detached, townhome, duplex, triplex, and fourplex buildings and requires certain architectural features be provided to ensure architectural variety within a subdivision. The DDC Subsection 7.10.2.D also allows the Director to approve alternative building design standards provided the alternative designs achieve the intent of the standards set forth in the DDC, advance goals and policies of the City, benefit the community, and do not impose greater impacts on adjacent properties.

In 2021, the Director memorialized appropriate alternative design features in a memo (see Exhibit 3) for the purpose of assisting in assessing architectural variety as required by DDC Subsection 7.10.3.D. This memo added ten (10) additional design elements as required by DDC 7.10.3E that were determined to be appropriate alternative design standards which met the overall intent of the DDC. Over time, this memo has been updated to reflect additional features and clarifications requested by developers and builders within the City as different architectural styles gain and decline in popularity for new homes. The most recent iterations of the memo are from the Summer of 2025 (see Exhibits 4 & 5). These memos are circulated among Planning and Building Safety staff and are used to guide reviews of new single-family homes. Given the manner in which the memos and determinations have evolved over time, and to ensure the same standards are applied across all developments consistently, the appropriate mechanism to codify these interpretations is to incorporate them into the Denton Development Code Subsection 7.10.3: Single-Family Detached, Duplex, Townhome, Triplex, and Fourplex Site and Building Design.

At the December 8, 2025 DCRC meeting, staff introduced the building design of single-family homes and other residential use types topic. Staff discussed additional design features to be added to the Building Mass and Form subsection of the DDC (see Exhibit 2). There was discussion over how many design features to require and whether to prioritize certain design features which emphasized community building, such as covered front porches. DCRC reached a consensus to require four design features.

At the January 12, 2026 DCRC meeting, staff will continue the discussion on the building design of single-family homes and other residential uses topic. The discussion will include an overview of the current

regulations related to single-family home building design within the DDC (particularly garage design and architectural variety), discuss standards to be included in the DDC, and seek direction regarding proposed amendments.

EXHIBITS

1. Exhibit 1 - Agenda Information Sheet
2. Exhibit 2 – DDC 7.10.3D-F Residential Building Design Standards
3. Exhibit 3 – 2021 Residential Design Determination
4. Exhibit 4 – Memo on Building Design Interpretation (Landmark)
5. Exhibit 5 – Memo on Building Design Interpretation (Stella Hills)
6. Exhibit 6 – Additional Design Features Guide
7. Exhibit 7 – Presentation

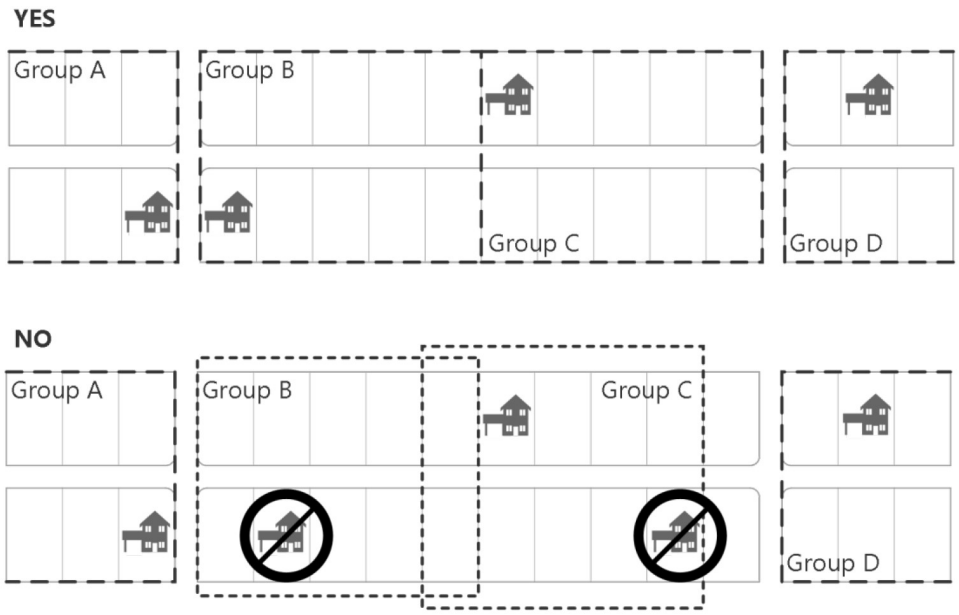
Respectfully submitted:
Hayley Zagurski, AICP
Planning Director

Prepared by:
Matt Bodine
Assistant Planner

7.10.3 Single-Family Detached, Duplex, Townhome, Triplex, and Fourplex Dwelling Site and

D. **Architectural Variety.** The same elevation shall not be used within any 10 lot grouping as defined by the two adjacent lots on either side of the subject property and the five lots immediately across the street from those same lots. This requirement shall not apply across a right-of-way dividing two adjacent blocks:

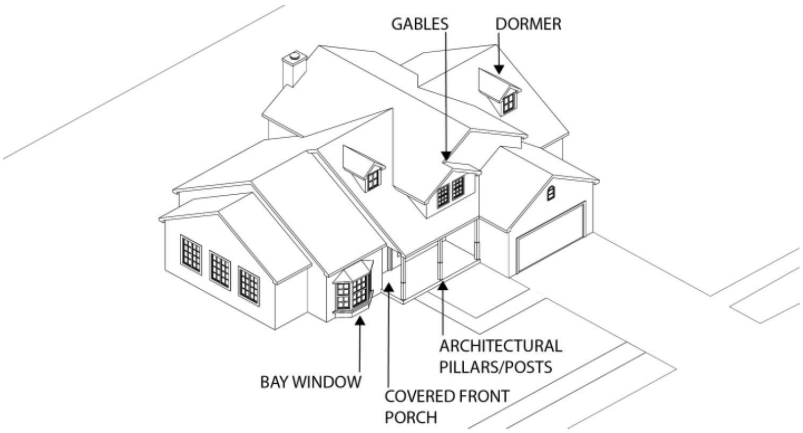
Figure 7.10-A: Architectural Variety



E. **Building Mass and Form.** Buildings shall incorporate at least three of the following design features to provide visual relief along the front of the residence:

1. Dormers;
2. Gables;
3. Recessed entries, a minimum of three feet deep;
4. Covered front porches;
5. Cupolas;
6. Architectural pillars or posts; and/or
7. Bay window with a minimum 24 inches projection.

Figure 7.10-B: Building Mass and Form



F. **Garage Design.**

1. Where alleys are present, garages shall be accessed from the alley to the maximum extent practicable.
2. For front-entry garages:
    a. The garage shall be offset a minimum of three feet from the building; and
    b. The total width of the garage door(s) shall not occupy more than 40 percent of the ground-floor building frontage.
3. Side- and rear-entry garages are encouraged and may encroach into setbacks pursuant to Table 3.7-A: Authorized Exceptions to Setbacks.

Department of Development Services / Planning Division
401 N. Elm St., Denton, TX 76201 • (940) 349-8541

Planning Director Determination

DATE: April 27, 2021

SUBJECT: Architectural Variety and Building Mass and Form Interpretation

For the purposes of determining architectural variety as required in DDC Section 7.10.3.D, the addition or removal of design features (not a change in building materials alone) shall be used to distinguish building elevations. DDC Section 7.10.3E states "Buildings shall incorporate at least three of the following design features to provide visual relief along the front of the residence."

As permitted under Section 7.10.2D, the Director can approve alternative standards that meet or exceed the DDC standards in Section 7.10. The list below in *black text* is the list included in Section 7.10.3E. The *red text* below are additional design elements that can be used to meet the requirements of Section 7.10.3D-E.

1. Dormers;
2. Gables;
3. Recessed entries, a minimum of three feet deep;
4. Covered front porches;
5. Cupolas;
6. Architectural pillars or posts; ~~and/or~~
7. Projection (Bay, bump-out, or bow) window with a minimum 24 inches projection;
8. Balconies;
9. Stoops;
10. Shed roofs or awnings over garage doors;
11. Split garage doors where a column or post divides single-car garage doors;
12. Side-entry or J-swing garage;
13. Changes in wall plane where an entire section of the garage or home at least 8-feet in width extends at least 24 inches out from the primary wall plane;
14. Minimum 10% increase in the percentage of windows and doors (excluding garage doors) compared to a similar elevation;
15. More than one of the following decorative masonry details incorporated in the gables or across the front of the home: rowlock or soldier course rows around windows and garage doors, basket weave, herringbone, or other distinctive brick patterns in gables; cedar or cement shake or stucco board accents in gables; and/or
16. Other distinguishing structural features as approved by the Director.

The following pages provide supporting reference materials to help define these terms.

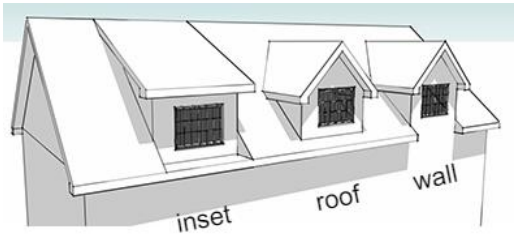
Per planning, window not required for dormer

Dormers

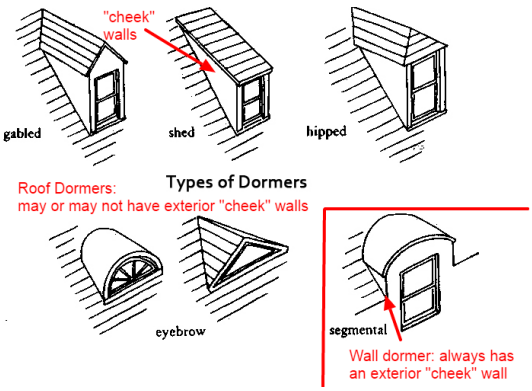
Definition: Dormer means a projecting structure with a ~~window~~ which is set vertically on a sloping building roof, having its own roof, which may be flat, arched, gabled or hip in shape.

Dormer Categories

Dormers can be placed into three structural categories: roof dormer, wall dormer, and inset dormer.



Cheek Wall - The vertical sides of the dormer.



Wall dormers always has a “cheek” wall. This is what distinguishes it from a cross gable, which never has a “cheek” wall.



Covered Front Porches vs Recessed Entries

A porch is a covered shelter projecting in front of the entrance of a house or building. The structure is external to the walls of the building but may be enclosed by walls, columns, rails, or screens extending from the main structure.

A recessed entry is a created when the front door is inset behind the front wall of the house by at least three feet, typically covered by the primary roof line and fully enclosed by walls on both sides. A recessed entry can be paired with a front porch if the width of the recessed space is enlarged by 24 inches or more on a single side or 12 inches or more on both sides of the front door.

Note, this distinction between porches and recessed entries should be applied to new subdivisions that begin the platting process after Section 7.10.3 is updated to include the broader list of design elements proposed above.



(1) Covered front porch



(2) Recessed entry



(3) Wider recessed entry/covered porch

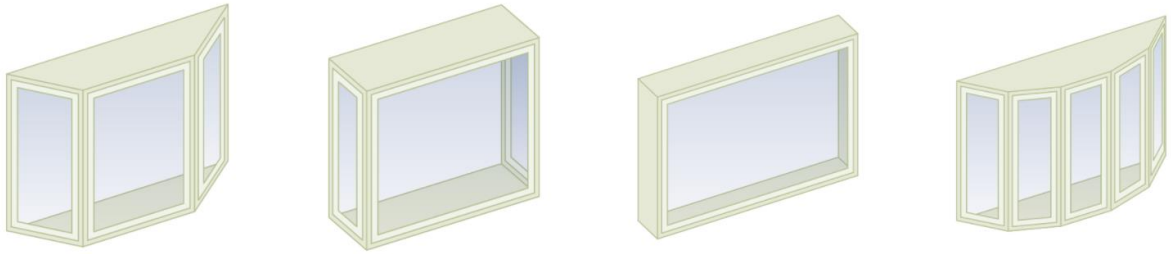
Stoops

Definition: A stoop is a small staircase ending with a platform (covered or uncovered) that leads to the entrance of the residence. Depending on the style and number of steps, the stoop may include railing on one or both sides.



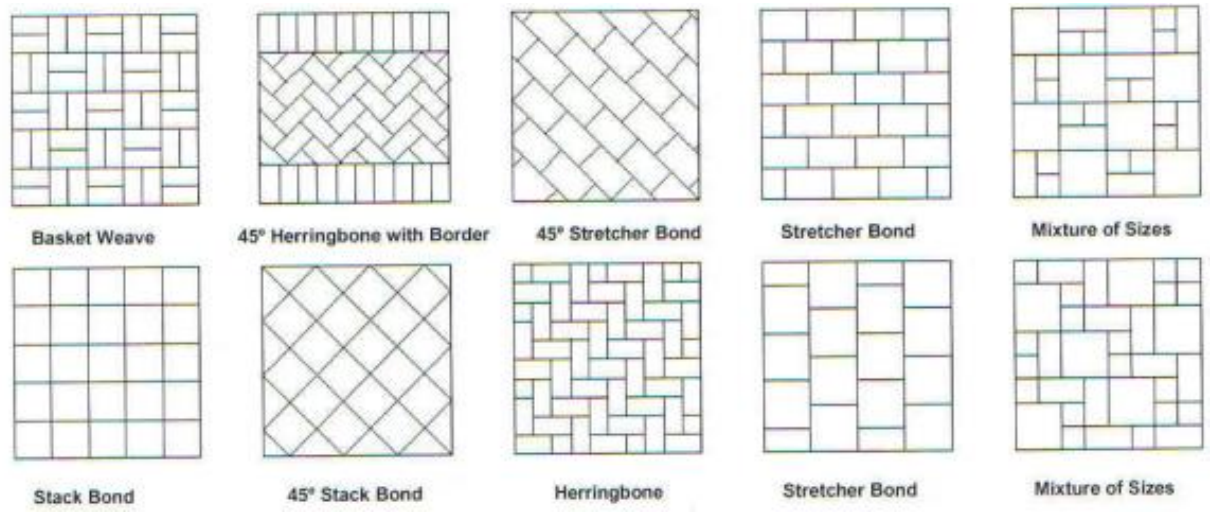


Projection Windows – Styles

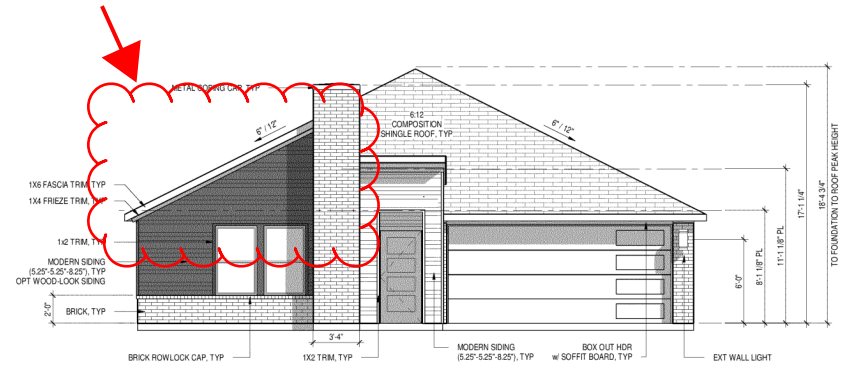


BAY WINDOW | BOX BAY WINDOW | BUMP-OUT BAY WINDOW | BOW WINDOW

Decorative Masonry Patterns

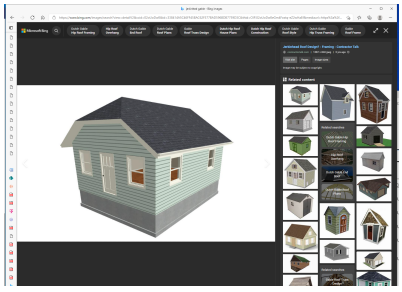


Per Planning below is an example of a split or lean-to gable style. This would be acceptable as a gable feature.



1 ELEVATION M - FRONT
1/8" = 1'-0"

example of jerkinhead gable



Development Services Department
401 N. Elm Street, Denton, TX 76201 • (940) 349-8600

Date: July 3, 2025

To: Karin Sumrall, Senior Planning + Design Architect, Hillwood Communities
Andrew Piper, Vice President, Hillwood Communities

From: Tina M. Firgens, AICP, Deputy Director of Development Services/Planning Director

RE: Planning Director Determination for Hunter Ranch (Landmark) Architectural Variety and Building Mass and Form Interpretation – Updated to include Design Feature #18

As permitted under the Hunter Ranch MPC Section 7.10.2D, the Director may approve alternative standards that meet or exceed the MPC standards in MPC Section 7.10. (Site and Building Design); in other words, the alternatives being granted shall be the same or better degree achieved than that of the current standard. In addition, the Director took into consideration and applied the following provisions listed below from MPC Section 7.10.1 (Purpose) in a manner appropriate for residential development, as well as other prior Director Determinations issued for single-family housing design in the City of Denton:

A. Promote high-quality development and construction;
D. Provide variety and visual interest in the exterior design of buildings; and
E. Enhance the streetscape and diminish the prominence of garages and parking areas.

Architectural Variety

For the purposes of determining architectural variety as required in MPC Section 7.10.3D (Architectural Variety), the addition or removal of design features (not a change in building materials alone) shall be used to distinguish building elevations. MPC Section 7.10.3E (Building Mass and Form) states: "Buildings shall incorporate at least three of the following design features to provide visual relief along the front of the residence."

For the list below, the text shown in "black text" is the list included in MPC Section 7.10.3E. The "***bold italicized text***" are additional design elements that can be used to meet the requirements of MPC Section 7.10.3D and 7.10.3E. It should be noted that per the property owner/developer's request, the design features in 7.10.3E. has been increased to four design features (instead of three) which the Director has no objection.

MPC Section 7.10.3E. Building Mass and Form

Buildings shall incorporate at least ***four*** of the following design features to provide visual relief along the front of the residence:

1. Dormers ***(while a window is preferred, it is not required);***

2. Gables;
3. Recessed entries, a minimum of three feet deep;
4. Covered front porches;
5. Cupolas;
6. Architectural pillars or posts;
7. Bay window with a minimum 24 inches projection;

8. Variable roof pitch and nested roofs;

3. Shed Roof;

10. Balconies;

11. Stoops;

12. Awnings over garage doors;

13. Split garage doors where a column or post divides single-car garage doors;

14. Side-entry, J-swing, or alley garages;

15. Changes in wall plane where an entire section of the garage or home at least 8-feet in width extends at least 24-inches out from the primary wall plane;

1C. Minimum 10% increase in the percentage of windows and doors (excluding garage doors) above the minimum required (15%) for a total 25%; and/or

17. More than one of the following decorative masonry details incorporated:

- ***Masonry in the gables or across the front of the home: rowlock or soldier course rows around windows and garage doors, basket weave herringbone, or other distinctive brick patterns in gables; cedar shake or stucco board accents in gables.***
- ***Horizontal stucco banding extending across the majority of the home's width for the uppermost 30-50% of the façade for homes that otherwise are clad in a single material.***

18. Integrated roof extension window awning with supportive brackets

Refer to Attachment A for supporting reference materials to help define these terms, and that are applied to residences constructed elsewhere in Denton.

Front-Entry Garages

For the purposes of providing alternate design considerations for front-entry garages, the Director is approving design relief in two areas: 1) reduction of the garage off-set from the building to accommodate a variety of plan types where architectural relief is achieved; and 2) for smaller lot widths (45 feet or less) allowing the garage entry to exceed the 40% of the ground floor building-frontage to accommodate a typical two-car garage entrance.

The 24-inches off-set is preferred to be achieved measuring the off-set from a primary building wall associated with enclosed, occupiable, climate controlled living area to the garage face as this still provides an appropriate off-set and achieves the original intent for variable building plane off-sets and diminishing the prominence of the garage entrances, coupled with the property owner/developer's

commitment to providing four architectural design elements in 7.10.3E above.

Alternatively, staff recognizes that covered porches (whether climate controlled or not) can be designed in a manner that also achieves the desired architectural relief, provided that the covered porch is of a minimum size, even if the aforementioned 24-inch offset referenced above is not provided. As such, alternate design relief is being provided to accommodate porches for structures that do not achieve the 24-inch off-set between the garage face and primary building wall.

Allowing the front-entry garage to exceed the 40% limitation is because mathematically for smaller lots, a two-car garage entrance would otherwise not be able to comply thereby prohibiting smaller front-entry housing options. Garage prominence shall be diminished with the 24-inches required off-set, coupled with additional design features provided in accordance with 7.10.3F.2.b.i. below.

For the list below, the text shown in "black text" is the list included in MPC Section 7.10.3F (Garage Design). The "***bold italicized text***" are additional design elements that shall be used to meet the requirements of MPC Section 7.10.3F.

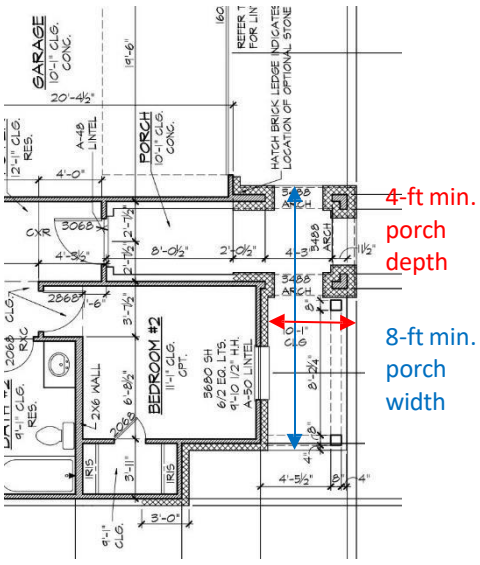
MPC Section 7.10.3F Garage Design

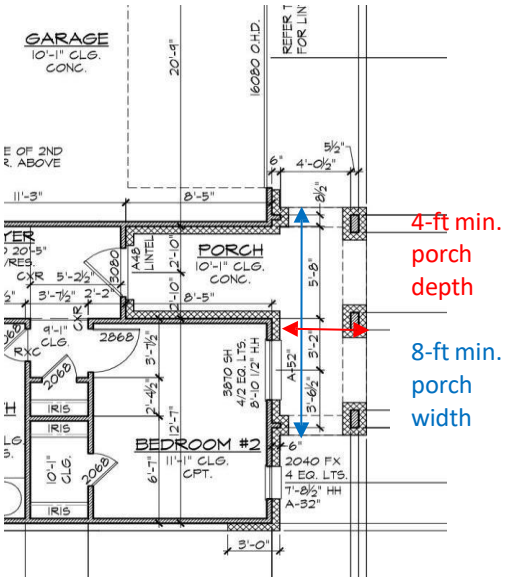
1. Where alleys are present, garages shall be accessed from the alley to the maximum extent practicable.

2. For front-entry garages:

    a. The garage shall be offset a minimum of ***24-inches*** from the building;

Exception: the 24-inches off-set referenced above may be reduced to 0-inches provided that a covered porch, minimum 4-feet in depth (measured from the exterior building wall face to the edge of the column exterior) and minimum 8-feet in width, with supporting columns is provided (does not have to be climate controlled) (see image below)





Arrows above denote where measurements are taken from for covered porch minimum width and depth. Note that a recessed entry area may count towards the minimum 8-foot width dimension, but <u>not</u> the minimum 4-foot porch depth dimension.

and

b. The total width of the garage door(s) shall not occupy more than 40 percent of the ground-floor building frontage, ***except as provided in b.i. below.***

   i. For lot widths 45 feet or less, the garage door(s) may exceed 40% the width of the ground-floor building frontage, provided that the structure shall have a minimum of four design elements from the list provided in 7.10.3E. Building Mass and Form above, <u>and</u> of which one of the design elements shall be from the list below:

      a) Covered front porch;

      b) Split garage doors where a column or post divides single-car garage doors;

      c) Stoops;

      d) Balcony;

      e) Variable roof pitch and nested roofs; or

      f) Dormers.

<u>Building Materials</u>

For the purposes of determining building materials as required in MPC Section 7.10.3H (Building Materials), the Director approves these material allowances due to state law limitations (adopted in 2019) associated with cities regulating exterior building materials.

For the list below, the text shown in "black text" is the list included in MPC Section 7.10.3H (Building Materials). The "***bold italicized text***" are additional materials that can be used to meet the requirements of MPC Section 7.10.3H.

MPC Section 7.10.3H. Building Materials

1. Exterior finishes shall be of wood, masonry, or cementitious siding.
2. Masonry **and cementitious siding** shall comprise a minimum of 50 percent of the exterior finishes of the total building elevation.
3. Other building materials may be approved by the Director provided they meet the intent of this section.

Sincerely,

Tina M. Firgens, AICP
Deputy Director of Development Services/Planning Director

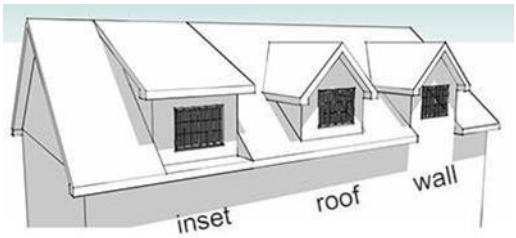
Enclosures:
Attachment A – Architectural Design Elements Defined

Attachment A – Architectural Design Elements Define

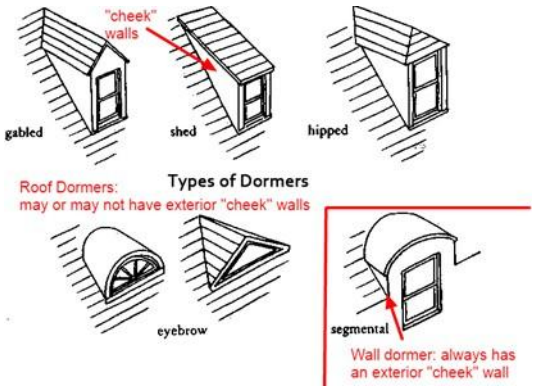
Dormers:

Definition: Dormer means projecting structure which is set vertically on a sloping building roof, having its own roof, which may be flat, arched, gabled or hip in shape. While a window within the dormer is preferred, it is not required.

Dormer Categories
Dormers can be placed into three structural categories: roof dormer, wall dormer, and inset dormer.



Cheek Wall - The vertical sides of the dormer.





Wall dormers always have a “cheek” wall. This is what distinguishes it from a cross gable, which never has a check wall.

Covered Front Porches vs Recessed Entries

A porch is a covered shelter projecting in front of the entrance of a house or building. The structure is external to the walls of the building but may be enclosed by walls, columns, rails, or screens extending from the main structure.

A recessed entry is a created when the front door is inset behind the front wall of the house by at least three feet, typically covered by the primary roof line and fully enclosed by walls on both sides. A recessed entry can be paired with a front porch if the width of the recessed space is enlarged by 24 inches or more on a single side or 12 inches or more on both sides of the front door.



Covered front porch



Recessed entry



Wider recessed entry/covered porch

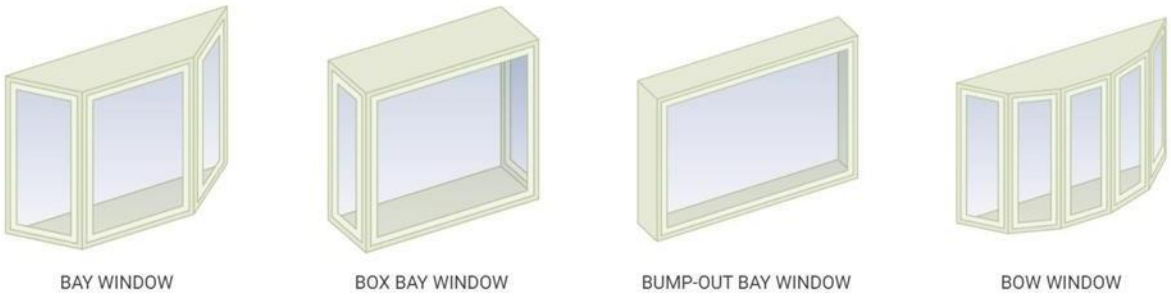
Stoops

Definition: A stoop is a small staircase ending with a platform (covered or uncovered) that leads to the entrance of the residence. Depending on the style and number of steps, the stoop may include railing on one or both sides.

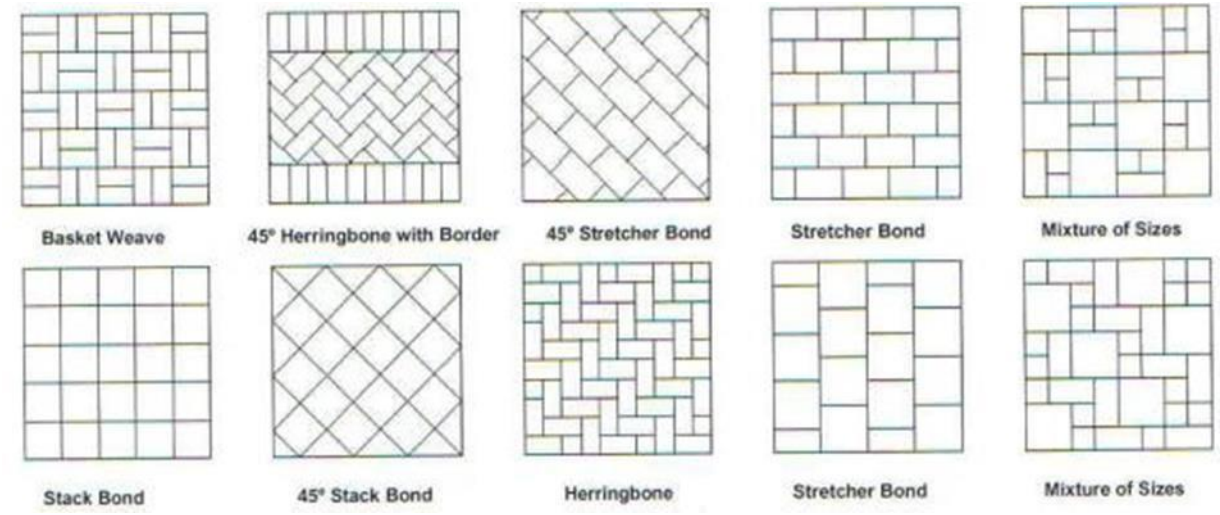




Projection Windows - Styles



Decorative Masonry Patterns



Roof Designs

Below is an example of a split or lean-to gable style. This would be acceptable as a gable feature.



Example of a jerkinhead gable.



Integrated Roof Extension Window Awning with Supportive Brackets

Below are examples that illustrate the integrated roof extension window awning with supportive brackets architectural design feature. Three key elements of this design feature include: 1) the roof extension projects (or extends) lower than the primary roof eave; 2) the roof extension is proportional to the width of the window; and 3) supportive brackets connect underneath the roof extension and attach to the exterior building wall.









Development Services Department
401 N. Elm Street, Denton, TX 76201 • (940) 349-8600

Date: June 26, 2025

To: Mr. Frank Su, Vice President, Land Acquisition – Meritage Homes

From: Tina M. Firgens, AICP, Deputy Director of Development Services/Planning Director

RE: Planning Director Determination for Stella Hills Single-Family Residential Development, Architectural Variety and Building Mass and Form Interpretation

As permitted under the Denton Development Code (DDC) Section 7.10.2D, the Director may approve alternative standards that meet or exceed the DDC standards in DDC Section 7.10. (Site and Building Design); in other words, the alternatives being granted shall be the same or better degree achieved than that of the current standard. In addition, the Director took into consideration and applied the following provisions listed below from DDC Section 7.10.1 (Purpose) in a manner appropriate for residential development, as well as other prior Director Determinations issued for single-family housing design in the City of Denton:

A. Promote high-quality development and construction;
D. Provide variety and visual interest in the exterior design of buildings; and
E. Enhance the streetscape and diminish the prominence of garages and parking areas.

Architectural Variety

For the purposes of determining architectural variety as required in DDC Section 7.10.3D (Architectural Variety), the addition or removal of design features (not a change in building materials alone) shall be used to distinguish building elevations. DDC Section 7.10.3E (Building Mass and Form) states: "Buildings shall incorporate at least three of the following design features to provide visual relief along the front of the residence."

For the list below, the text shown in "black text" is the list included in DDC Section 7.10.3E. The "***bold italicized text***" are additional design elements that can be used to meet the requirements of DDC Section 7.10.3D and 7.10.3E

DDC Section 7.10.3E. Building Mass and Form

Buildings shall incorporate at least three of the following design features to provide visual relief along the front of the residence: ***(Note: the design features shall be increased to four if reducing the required 3-ft garage off-set from the building; see Front Entry Garages section of letter below)***

1. Dormers ***(while a window is preferred, it is not required);***
2. Gables;

3. Recessed entries, a minimum of three feet deep;
4. Covered front porches;
5. Cupolas;
6. Architectural pillars or posts;
7. Bay window with a minimum 24 inches projection;
8. ***Variable roof pitch and nested roofs;***
9. ***Shed Roof;***
10. ***Balconies;***
11. ***Stoops;***
12. ***Awnings over garage doors;***
13. ***Split garage doors where a column or post divides single-car garage doors;***
14. ***Side-entry or J-swing garage;***
15. ***Changes in wall plane where an entire section of the garage or home at least 8-feet in width extends at least 24-inches out from the primary wall plane;***
16. ***Minimum 10% increase in the percentage of windows and doors (excluding garage doors) above the minimum required (15%) for a total 25%; and/or***
17. ***More than one of the following decorative masonry details incorporated in the gables or across the front of the home: rowlock or soldier course rows around windows and garage doors, basket weave herringbone, or other distinctive brick patterns in gables; cedar shake or stucco board accents in gables.***

Refer to Attachment A for supporting reference materials to help define these terms, and that are applied to residences constructed elsewhere in Denton.

Front-Entry Garages

For the purposes of providing alternate design considerations for front-entry garages, the Director is approving design relief related to the reduction of the garage off-set from the building to accommodate a variety of plan types where architectural relief is achieved.

The 24-inches off-set is preferred to be achieved measuring the off-set from a primary building wall associated with enclosed, occupiable, climate controlled living area to the garage face as this still provides an appropriate off-set and achieves the original intent for variable building plane off-sets and diminishing the prominence of the garage entrances, coupled with the builder's commitment to providing four architectural design elements in 7.10.3E above.

Alternatively, staff recognizes that covered porches (whether climate controlled or not) can be designed in a manner that also achieves the desired architectural relief, provided that the covered porch is of a minimum size, even if the aforementioned 24-inch offset referenced above is not provided. As such, alternate design relief is being provided to accommodate porches for

structures that do not achieve the 24-inch off-set between the garage face and primary building wall.

For the list below, the text shown in "black text" is the list included in DDC Section 7.10.3F (Garage Design). The "***bold italicized text***" are additional design elements that shall be used to meet the requirements of DDC Section 7.10.3F.

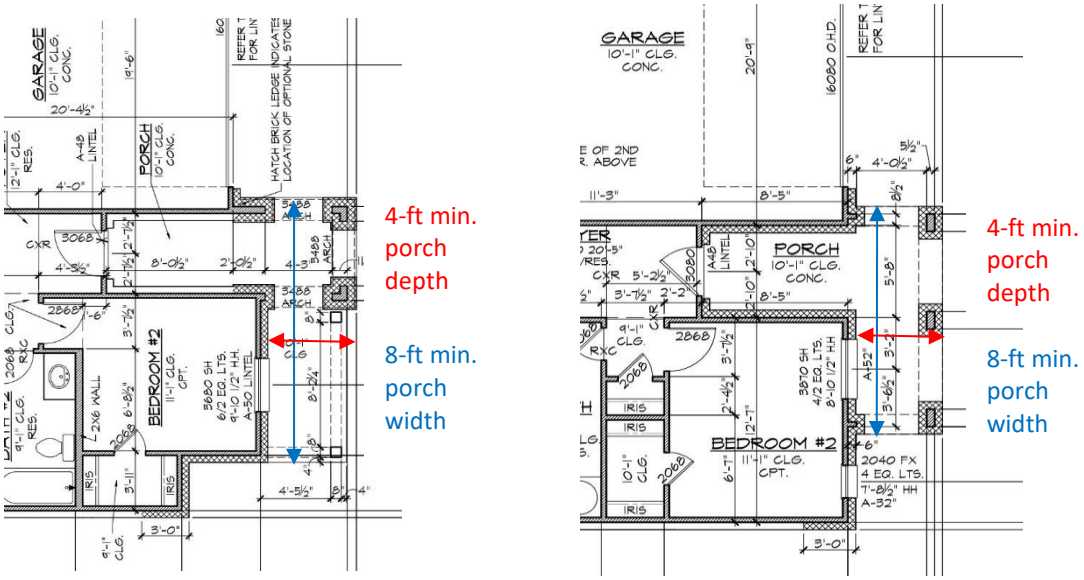
DDC Section 7.10.3F Garage Design

1. Where alleys are present, garages shall be accessed from the alley to the maximum extent practicable.

2. For front-entry garages:

   a. The garage shall be offset a minimum of ***24-inches*** from the building;

      Exception: the 24-inches off-set referenced above may be reduced to 0-inches provided that a covered porch, minimum 4-feet in depth (measured from the exterior building wall face to the edge of the column exterior) and minimum 8-feet in width, with supporting columns is provided (does not have to be climate controlled) (see image below)



      Arrows above denote where measurements are taken from for covered porch minimum width and depth. Note that a recessed entry area may count towards the minimum 8-foot width dimension, but <u>not</u> the minimum 4-foot porch depth dimension.

      and

   b. The total width of the garage door(s) shall not occupy more than 40 percent of the ground-floor building frontage.

3. Side- and rear-entry garages are encouraged and may encroach into setbacks pursuant to Table 3.7-A: Authorized Exceptions to Setbacks.

Additionally, please note that this design relief shall apply to all builders within the Stella Hills single-family residential development, including but not limited to Meritage Homes.

Sincerely,

Tina M. Firgens, AICP
Deputy Director of Development Services/Planning Director

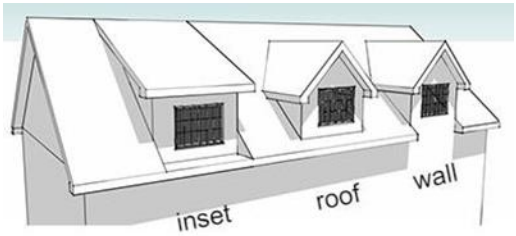
Enclosures:
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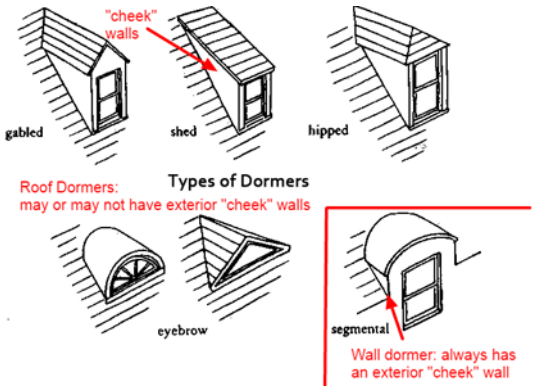
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Covered front porch



Recessed entry



Wider recessed entry/covered porch

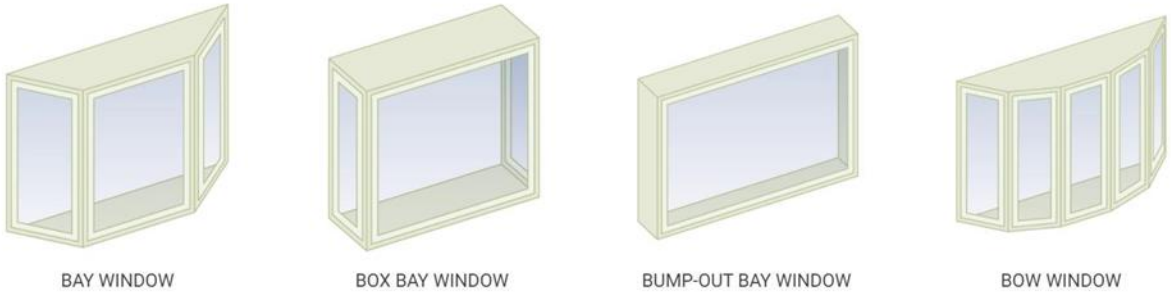
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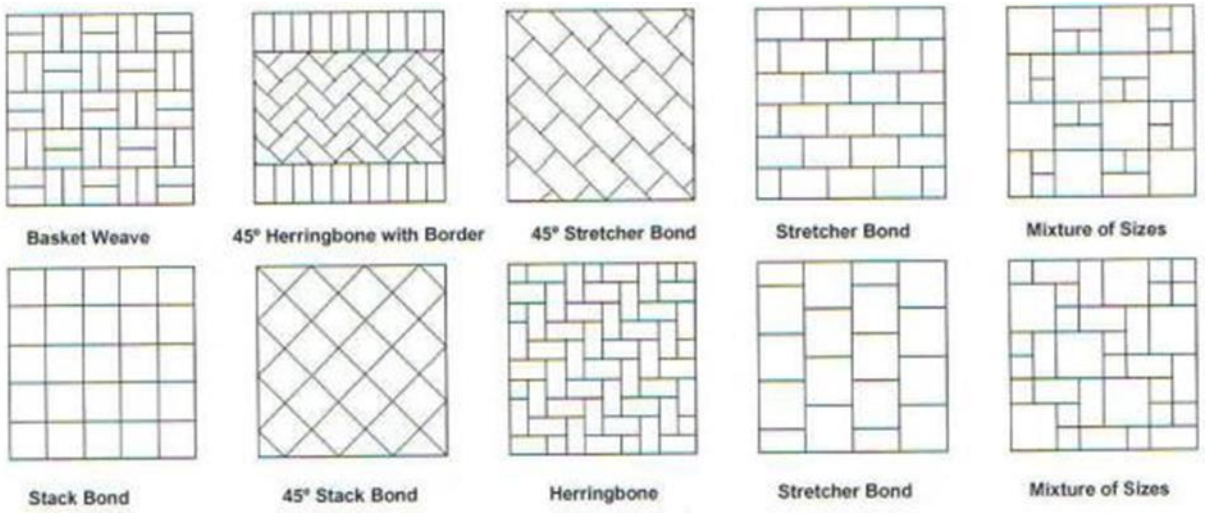




Projection Windows - Styles

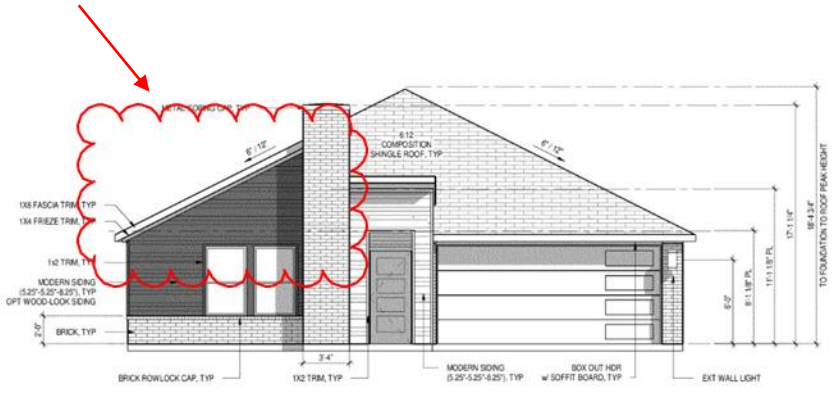


Decorative Masonry Patterns



Roof Designs

Below is an example of a split or lean-to gable style. This would be acceptable as a gable feature.



Example of a jerkinhead gable.



Additional Design Features

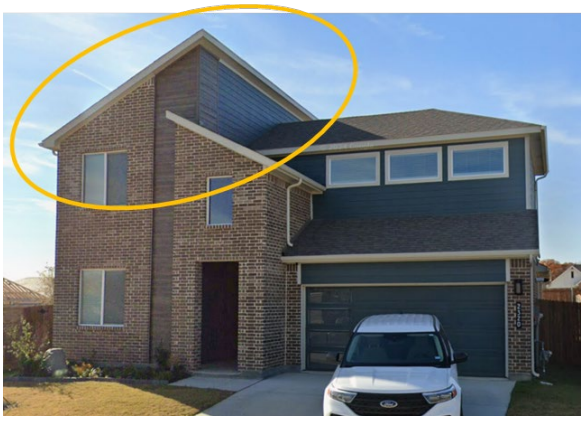
1. Variable roof pitch with nested roofs





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2. Shed roof



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3. Side-entry, alley-entry, or J-swing garage



4. Split garage doors where a column or post divides single-car garage doors



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5. Stoop





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6. Balcony



7. Integrated awning with supportive brackets over a window and/or a garage door





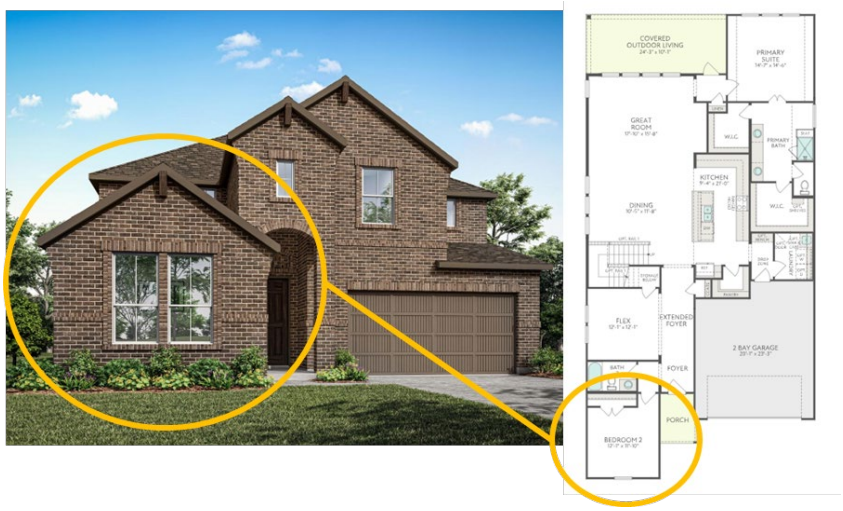
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8. Windows and doors, excluding garage doors, comprise 25% of the front façade.

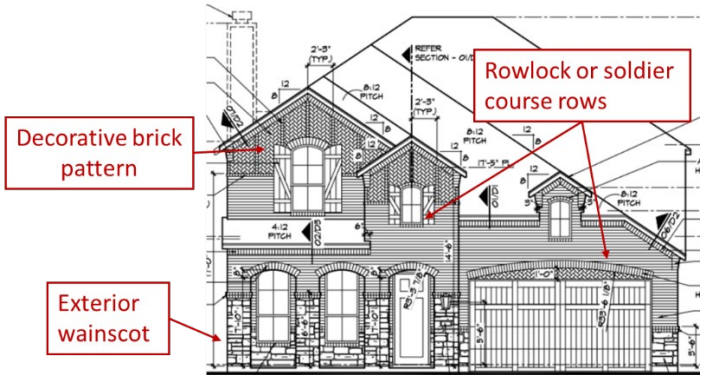


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9. Changes in wall plane where an entire section of the home at least 8-feet in width, excluding garages, extends at least 24-inches out from the primary wall plane.



10. Multiple design details in the façade such as:
    a. Rowlock or soldier course rows around windows and garage doors
    b. Distinctive brick patterns such as basket weave or herringbone
    c. Exterior wainscot that is a different material from the rest of façade
    d. Use of multiple materials or colors in the façade





Single-Family Home – Fourplex Building Design Standards

Matt Bodine

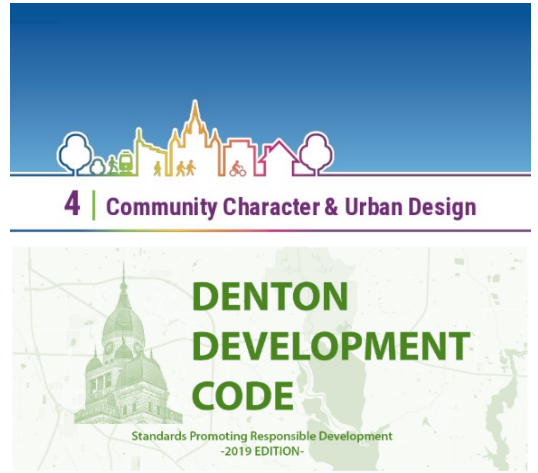
Assistant Planner

1/12/2026

CITY OF DENTON

Policy Direction & Background

- Denton 2040 Comprehensive Plan calls for exemplary urban design.
- DDC supports goals with building design standards.
    - Applicable at the individual house and neighborhood level.



- Homebuilders have wanted to use alternative designs.
- Director can approve alternative design standards.
- **Discuss alternative design standards that have been repeatedly used and incorporate them into the DDC.**

Building Mass and Form – Design Features

Require 4 out of 17 design features

1. Dormers
2. Gables
3. Recessed entries, a minimum of three feet deep
4. Covered front porches
5. Cupulas
6. Architectural pillars or posts
7. Projected window (such as bay, bump-out, or bow) with a minimum 24 inches projection
8. Variable roof pitch and nested roofs
9. Shed roof
10. Side-entry, alley-entry, or J-swing garage
11. Split garage doors where a column or post divides single-car garage doors
12. Stoop
13. Balcony
14. Integrated awning with supportive brackets over a window and/or a garage door
15. Windows and doors, excluding garage doors, comprise 25% of the front façade
16. Changes in wall plane where an entire section of the home at least 8-feet in width, excluding garages, extends at least 24-inches out from the primary wall plane
17. Multiple design details in the façade such as:
    - Rowlock or soldier course rows around windows and garage doors
    - Distinctive brick patterns such as basket weave or herringbone
    - Exterior wainscot that is a different material from the rest of façade
    - Use of multiple materials and/or colors in the facade

Garage Design

Purpose: Enhance the streetscape and diminish the prominence of garages.

Two main ways to regulate attached garages relative to the primary structure:

- Placement of garage
- Width of garage





<u>Questions for DCRC to consider</u>

- Which garage design offset changes are appropriate?
- Is the garage door width option for narrower lots appropriate?

Garage Design – J-Swing

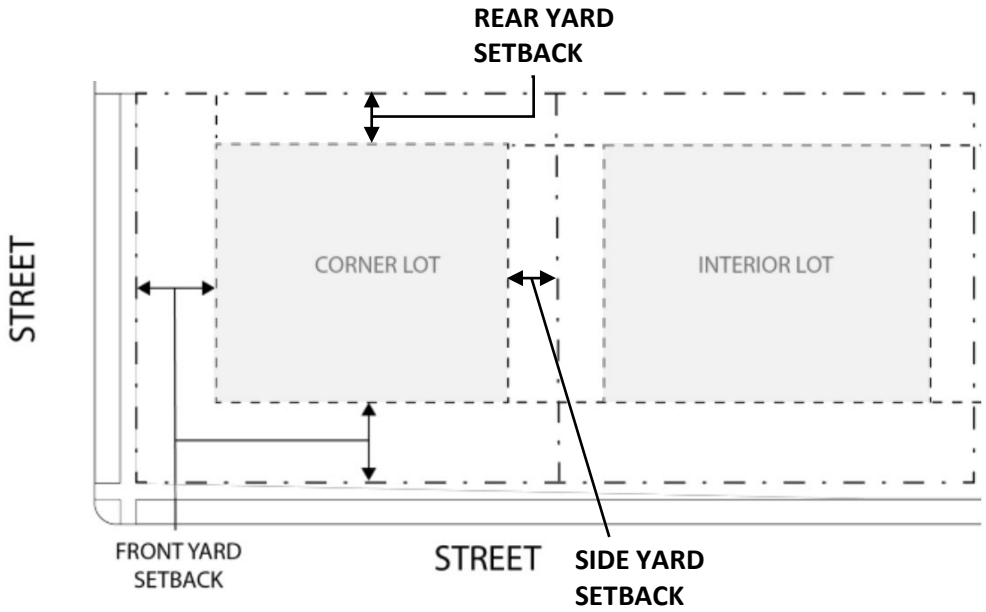
Current Standards:

- Side-entry and rear-entry garages may encroach into setbacks.
  - Must maintain a 10-ft front yard setback and 5-ft rear yard setback.

Proposed Changes

- Allowing J-swing garages to encroach into setbacks.
  - Must maintain 10-ft front yard setback.





Garage Design – Front-Entry Garage Offset

Current Standards: Garage required to be offset a minimum of 3 ft.

Proposed Changes: Option 1 (follows memo)

- Garage required to be offset a minimum of 2 ft.
- Off-set may be reduced to zero but must provide a covered porch.
    - Porch must be a minimum 4-feet in depth and 8-feet in width with supporting columns.



Garage Design – Front-Entry Garage Offset

Proposed Changes: Option 2

- No required garage offset.
- A maximum front offset of 5 ft from the building or porch.
- Required to have a front porch if:
  - No garage offset
  - Offset less than 2 ft
  - Offset in front of the house



Offset <2ft behind



No offset w/ porch + recessed entry



Offset 3½ ft in front of porch

Garage Design – Front-Entry Garage Width

Current Standards:

- Maximum garage door width is 40% of the building's ground-floor width.

Proposed Change:

- For lots 45 ft or less, max garage door width is 60%. Must have at least one design feature from the following:
  - Covered front porch
  - Variable roof pitch and nested roofs
  - Split garage doors where a column or post divides single-car garage doors
  - Stoops
  - Balcony
  - Dormers

Garage Design – Front-Entry Garage Width

Examples of single-family homes with <45’ wide lots



29ft wide house – 56% garage door



33ft wide house – 49% garage door

Considerations

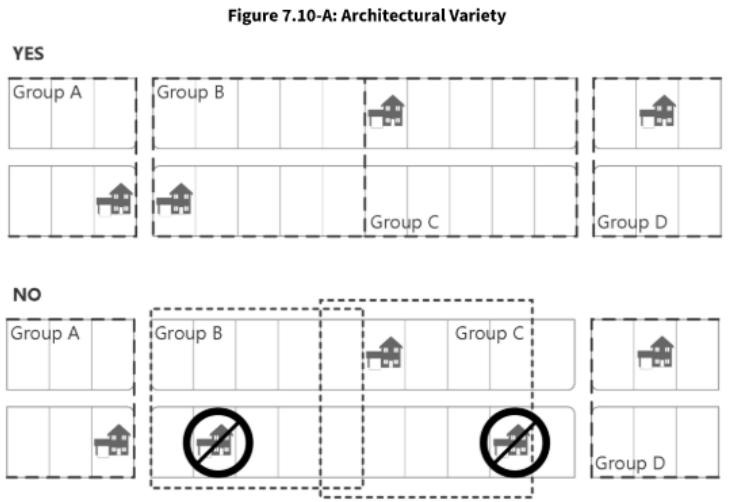
- Allows for affordable house plans on smaller lots.
- Must meet required parking: 4 spaces for single-family house, 2 spaces for townhome.
- Would be applicable in MN, MD, and MR zoning districts and for townhome lots.

Architectural Variety

Current Standard

The same elevation shall not be used within any 10 lot grouping as defined by the two adjacent lots on either side of the subject property and the five lots immediately across the street from those same lots. This requirement shall not apply across a right-of-way dividing two adjacent blocks.

- There's been confusion over "10-lot grouping" language.



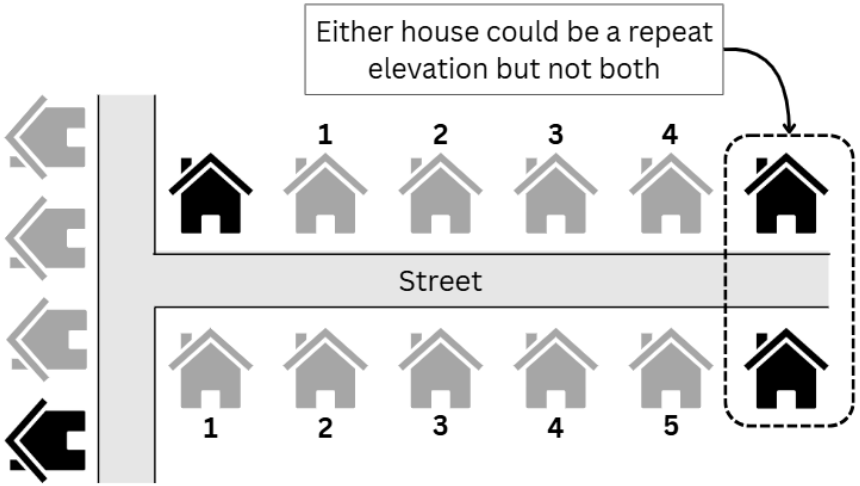
Proposed Change Option 1:

- Remove "10 lot grouping" language.
- Clarify that elevations must have different design features.

Architectural Variety

Proposed Change Option 2: New language, same intent

The *same elevation shall not be repeated* along any blockface *without at least four, intervening lots with different elevations* on the same side of the street and at least five, intervening lots with different elevations on the opposite side of the street. Elevations across the street are not considered if the lots do not face each other.



- Clarify that elevations must have different design features.

Next Steps

Prepare for presentation to Planning & Zoning Commission

City of Denton

City Hall
215 E. McKinney Street
Denton, Texas
www.cityofdenton.com

AGENDA INFORMATION SHEET

DEPARTMENT: Department of Development Services

DCM: Cassey Ogden

DATE: January 12, 2026

SUBJECT

Receive a report, hold a discussion, and give staff direction regarding proposed Code amendments related to Tree Preservation.

BACKGROUND

In late 2024, City Council prioritized a review of the Tree Preservation Code, and on December 9, 2024, the Development Code Review Committee (DCRC) held a discussion regarding prioritizing future potential Denton Development Code (DDC) amendment topics and similarly provided direction to staff regarding the prioritization of amendments to the DDC related to Tree Code. Staff has been tasked with assessing the current DDC Tree Code standards and presenting amendments to the DDC where appropriate.

The proposed amendments Staff has identified relate to industry best practices, improving clarity, and simplification of implementation of regulations for applicable development. Examples of sections that Staff is proposing to amend for clarifications include Tree Protection Requirements During Construction, Minimum Preservation Requirements, and Tree Replacement.

At the May 12, 2025, DCRC meeting, Staff introduced the current Tree Preservation Code regulations in DDC Subsection 7.7.4 and provided an overview of current standards. The discussion included an array of topics including but not limited to tree removal requirements, mitigation and replacement requirements, and City canopy goals.

At the June 23, 2025, DCRC meeting, Staff presented background on the City of Denton's Tree Code ordinance and how it relates to various provisions in the Texas Local Government Code, including discussions on vesting and state law relating to a city's authority to regulate tree preservation. A portion of the discussion regarding state law was held in a Closed Session meeting. The discussion on TLGC 212.905 was continued in closed session along with an overview of takings law at the July 14, 2025 DCRC Meeting.

Also at the July 14, 2025 DCRC meeting, Staff began the discussion of proposed Tree Code amendments; specifically, amendments to Subsection 9.2: Definitions. Following an overview of the proposed amendments, Staff requested consensus and opened the floor for additional feedback from the Committee regarding the proposed edits to the DDC.

At the September 22, 2025 DCRC meeting, Staff continued discussion of proposed amendments to the Tree Code, and revisited Subsection 9.2: Definitions, taking into consideration feedback gathered from the Committee at the July 14, 2025 meeting. Following the presentation of the proposed amendments, Staff

presented proposed amendments to Subsection 7.7.4.A and B: Purpose and Intent and Applicability and Exemptions.

At the October 13, 2025 DCRC meeting, Staff presented proposed amendments to Subsections 7.7.4.C through E. Specifically, these amendments addressed tree removal permits, tree protection during construction, and tree preservation plans.

At the October 27, 2025 DCRC meeting, Staff provided an overview of the Tree Preservation Plan (TPP) review process. This demonstration included a brief overview of a TPP and the associated Tree Inventory Excel file, as well as a brief overview of common issues that are found on TPP's. This presentation demonstrated how a TPP evolves over the course of a project.

At the November 17. 2025 DCRC meeting, staff presented amendments to Subsections 7.7.4.F and 7.7.4.G, Alternative TPP's and Minimum Preservation Requirements. Following this discussion, Staff requested feedback from the Committee regarding proposed replacement rates, reaching a consensus in agreement with Staff's recommendation. In addition, the Committee asked staff to research methods which could be codified to provide flexibility for infill and encumbered sites without requiring Director or City Council approval.

At the December 8, 2025 DCRC meeting, Staff introduced results from testing related to the guidance previously provided by the Committee regarding replacement rates. Following this discussion, Staff requested feedback from the Committee regarding higher mitigation replacement rates and tree preservation credits.

At the January 12, 2026 DCRC meeting, Staff will present Tree Code amendments and incentives related to infill development. Following this discussion, Staff will present results from the testing of higher mitigation rates and replacement rates, requesting confirmation on these items from the committee.

A draft presentation is provided as Exhibit 3. Please note that the material may be difficult to follow without Staff's explanation. The presentation is still being refined, and a finalized version will be shared at the meeting.

EXHIBITS

Exhibit 1 – Agenda Information Sheet
Exhibit 2- Redline markup (DDC 7.7.4.G and M)
Exhibit 3 – Draft Presentation

Respectfully submitted:
Hayley Zagurski, AICP
Planning Director

Bryce VanArsdale
Assistant Planner

7.7.4 Tree Preservation.

G. **Minimum Preservation Requirements.**

1. **Tree Types and Required Preservation.**

b. **Heritage and Quality Trees.** A minimum of 30 percent of the total ~~dbh~~ DBH shall be preserved within the development impact area. Removal of trees shall be replaced in accordance with Subsection 7.7.4H or 7.7.4I.

iii. Properties without a preserved habitat or conservation easement may reduce the 30 percent minimum to 10 percent provided:

a. Removed trees shall be mitigated at the following rates: [INSERT];

b. Subject lot meets the definition of Infill Development;

c. Fewer than 10 protected trees exist on site; and

d. Multifamily and non-residential uses outside of the MD zoning district as identified on the Official Zoning Map of City, as amended, shall provide an additional 10 landscape points from Table 7.E: Landscape Area Points System

M. **Incentives.**

5. **Parking Space Reduction.** Upon application and verification by the Director, an individual shall be entitled to a reduction in the minimum parking to help meet the minimum tree preservation requirements. ~~For the purpose of providing an incentive, t~~The minimum parking requirements in Subsection 7.9.4~~,~~ may be reduced by one parking space for every 12 inches ~~dbh~~ DBH of trees that have been protected ~~or mitigated~~ on a site. Up to 15 percent of the required spaces may be reduced in addition to the total reduction allowed in Section 7.9.5: *Parking Alternatives*.~~; or~~

7. ~~The enforcement and penalty provisions of Section 1.6 shall apply to this subsection.~~ **Height Increase**. An additional 10 feet of maximum building height allowed in the applicable zoning district in Section 3.2– 3.5 may be granted to accommodate for the preservation of one or more protected trees located on the subject property or an adjacent developed property with Driplines within 30 feet but no closer than 10 feet of a proposed building.

a. The amount of floor area on the additional story is limited to:
   i. 25% of the first floor gross floor area for the preservation of each Quality Tree; and
   ii. 35% of the first floor gross floor area for the preservation of each Heritage Tree.
b. In no instance shall the total gross floor area added on an additional story exceed 10,000 square feet unless otherwise permitted elsewhere in this DDC.
c. If this incentive is taken in combination with Section 2.12: *Affordability Incentives Procedures*, the additional building height granted herein shall be counted towards any height increase permitted as an Affordability Incentive.
d. Height increases must comply with Section 7.10.6: *Building Height in Transition Areas*.

8. **Reduced Setbacks**. Where the critical root zone of a protected tree whose trunk lies within a required side or rear yard setback or an adjacent developed property encroaches into the buildable area, the opposite side yard setback may be reduced by the amount that the critical root zone of the protected tree encroaches into the buildable area, provided the setback is reduced no more than 10 feet and that a required setback is no less than 5 feet.

11. **Off-Site Trees**. Where a canopy of a protected tree on an adjacent developed lot or parcel overlaps with the subject property and no development is proposed under within the Dripline, the adjacent tree may receive 0.5 times the amount of the preservation credit prescribed in Subsection 7.7.4I. The off-site tree must be included on the Tree Survey and Tree Preservation Plan and protected in accordance with Subsection 7.7.4D in order to receive credit. Off-site trees shall not account towards the minimum preservation required in Subsection 7.7.4G.
12. **Adjacent Residential Property**. Where protected trees lie within 10 feet of an adjacent property with an occupied residential use, preservation of said trees may receive 1.15 times the amount of preservation credit prescribed in Subsection 7.7.4I so long as no development activity occurs within the Dripline of said trees or 1.25 times the amount of preservation credit prescribed in Subsection 7.7.4I if a common area lot or conservation easement are designated on the plat encompassing the entire Dripline of the trees and providing for permanent preservation.

DCRC25-0095h
Tree Code Definitions

Bryce Van Arsdale
Assistant Planner

January 12, 2026

CITY OF DENTON

Infill Development

7.7.4.G Minimum Preservation Requirements

iii. Properties without a preserved habitat or conservation easement may reduce the 30 percent minimum to 10 percent provided:

a. Trees proposed for removal shall be mitigated in accordance with 7.7.4.G.1.b.ii, above:

b. Subject lot meets the definition of Infill Development;

c. Fewer than 10 protected trees exist on site; and

d. Multifamily and non-residential uses outside of the MD zoning district as identified on the Official Zoning Map of City, as amended, shall provide an additional 10 landscape points from Table 7.E: Landscape Area Points System

7.7.4.M Incentives

7. **Height Increase.** An additional 10 feet of maximum building height allowed in the applicable zoning district in Section 3.2– 3.5 may be granted to accommodate for the preservation of one or more protected trees located on the subject property or an adjacent developed property with Driplines within 30 feet but no closer than 10 feet of a proposed building.

a. The amount of floor area on the additional story is limited to:

i. 25% of the first floor gross floor area for the preservation of each Quality Tree; and

ii. 35% of the first floor gross floor area for the preservation of each Heritage Tree.

b. In no instance shall the total gross floor area added on an additional story exceed 10,000 square feet unless otherwise permitted elsewhere in this DDC.

c. If this incentive is taken in combination with Section 2.12 : Affordability Incentives Procedures, the additional building height granted herein shall be counted towards any height increase permitted as an Affordability Incentive.

d. Height increases must comply with Section 7.10.6: Building Height in Transition Areas.

7.7.4.M Incentives

8. **Reduced Setbacks**. Where the critical root zone of a protected tree whose trunk lies within a required side or rear yard setback or an adjacent developed property encroaches into the buildable area, the opposite side yard setback may be reduced by the amount that the critical root zone of the protected tree encroaches into the buildable area, provided the setback is reduced no more than 10 feet and that a required setback is no less than 5 feet.

Replacement Rates and Preservation credits

- With this round of updates 50/40% reduction has been removed
- Staff is seeking **confirmation** on the following item:
    - Replacement rates for 20% preservation

20% Preservation

- Sub-categories are not used within classification
- "Fair" health classification is not permitted
- Percent increase is compared to Standard Preservation scenario

Higher Mitigation			
Type	**dbh Removed**	**Replacement Ratio**	**Calculated DBH**
Heritage	0.00	4:1	0.00
Quality	0.00	3:1	0.00
Subtotal	0		0.00
Protected Trees Removed			
Secondary	0	4":1 tree	0.00
Total			0.00

Replacement Rates- 20% Preservation

Tree Mitigation Worksheet (20% Preservation)	
Reduction due to Higher Mitigation	
	dbh
Total (Healthy) dbh	0
Dead Tree dbh	0
Total (Healthy) Heritage/Quality dbh	0
Required Preservation (20%)	0
Provided Heritage/Quality Preservation dbh:	0
Preservation dbh Above Required	0
Requires Higher Mitigation (Max 10%)	0
Required Preservation dbh Achieved?	Yes

Higher Mitigation			
Type	dbh Removed	Replacement Ratio	Calculated DBH
Heritage	0.00	2.75:1	0.00
Quality	0.00	2.25:1	0.00
Subtotal	0		0.00
Higher Mitigation Total dbh achieved?			Yes
Protected Trees Removed			
Type	dbh Removed	Replacement Ratio	Calculated DBH
Heritage	0.00	2.5:1	0.00
Quality	0.00	2:1	0.00
Subtotal			0.00
	Trees Removed	Replacement Ratio	Calculated DBH
Secondary	0	4":1 tree	0.00
Total			0.00
Preliminary Mitigation dbh		40% Reduction	0.00

Type	dbh Preserved		Preservation Credit
Landmark	0.00	4:1	0.00
Heritage	0.00	3:1	0.00
Quality	0.00	2:1	0.00
Addt'l Cluster Credit	0.00	1.15:1	0.00
Total	0		0.00
Required Mitigation dbh			0

Higher Mitigation			
Type	dbh Removed	Replacement Ratio	Calculated DBH
Heritage	0.00	4:1	0.00
Quality	0.00	3:1	0.00
Subtotal	0		0.00
Protected Trees Removed			
Secondary	0	4":1 tree	0.00
Total			0.00

DCRC Preferred Scenario Analysis

Sample Sites

- 8 sites
  - All 2025 higher mitigation projects + 4 from past years
- Percentage difference compared to standard mitigation scenario
  - Previously approved amendment

Findings

- Average % increase in mitigation: ~59%
  - Median: ~64%
  - Range: ~34% to ~72%
- Generally: sites with more heritage removal saw the largest change

Discussion

Replacement Rates

- With this round of updates 50/40% reduction has been removed
- Staff is seeking **confirmation** on the following items:
  - Preservation credits for all Tree Preservation Plans

Preservation Credits

- Credits which are applied against required mitigation
- Apply to all TPP's regardless of preservation percentage
- DCRC Direction: Scenario 3
  - Covers ~55% of owed mitigation with 30% tree preservation
  - No mitigation owed at ~44% preservation

Trees Preserved			
Type	dbh Preserved		Preservation Credit
Landmark	0	5:1	**0**
Heritage	90	3.5:1	315
Quality	90	2:1	180
Secondary*	0	0.5:1*	0
Addt'l Cluster Cred	0	1.15:1	0
Total	180		495
Mitigation dbh			**397.5**

DCRC Preferred Scenario Analysis

Sample Sites

- 18 sites
  - All 2025 projects + several from past years
- Percentage difference compared to current preservation credit rates

Findings*

- Average % increase in mitigation: ~8%
  - Median: ~6%
  - Range: ~0% to ~29%
- Sites with more heritage preservation saw the largest change

* One outlier removed: 169%

Questions?