CITY OF BISMARCK
STORMWATER DESIGN
STANDARDS MANUAL

Stormwater Management Plan
Stormwater Facility Design Policy Manual
Stormwater Plan and Permit Requirements

City of Bismarck, North Dakota

March 2000

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in association with Houston Engineering, Inc.
and BlueStem, Inc.

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1.1.1 Preface

The City of Bismarck has prepared this Stormwater Design Standards Manual (Manual) to guide and assist persons needing to comply with its Stormwater Management Ordinance. The development of the Stormwater Management Ordinance and the Stormwater Design Standards Manual for the City of Bismarck and its extraterritorial jurisdiction was a major step in solving existing and future drainage problems in the Bismarck area. The City's Stormwater Management Ordinance requires the development of Stormwater Management Plans for most types of land disturbing activities within the City and its extraterritorial jurisdiction. The Stormwater Management Ordinance outlines approval procedures, plan review procedures, approval standards, as well as the application process, enforcement, and penalties. This Stormwater Design Standards Manual provides guidelines to implement the Stormwater Management Ordinance. As such, this Stormwater Design Standards Manual is intended to be flexible and is subject to revision as conditions or criteria change in the future. The goals set forth for the preparation of the Manual were:

1. To unify standards and improve the quality of drainage evaluations and design by providing specific and detailed criteria and standards for the City of Bismarck.

2. To reduce the effort required to prepare and review Stormwater Management Plans by simplifying methodologies and procedures.

3. To provide the information necessary to prepare and to review the Stormwater Management Plan and design within a single document that is complete, organized, and easily referenced.

4. To provide stormwater evaluation and design standards that will prevent future stormwater problems and enhance the urban environment within the goals and objectives of the City of Bismarck.

The philosophy behind the selection of material included in the Stormwater Design Standards Manual was based upon the following assumptions:

1. The design engineer and reviewer are assumed to have a basic knowledge and understanding of open channel hydraulics, storm sewer hydraulics, and urban hydrological techniques.

2. A need exists for a self-contained document to minimize time expended by design engineers and review engineers, and to minimize conflicting constraints that develop when the data to be used are not clearly specified.

3. A need exists for strict criteria in certain areas, but allowance must be made for design flexibility in other areas, within acceptable limitations.

4. In certain areas of the City, more detailed or strict criteria may be required due to the nature of particular stormwater problems in those areas.

5. Throughout the Manual, emphasis must be placed on simplifying procedures to minimize the time required to prepare and to review Stormwater Management Plans.

In summary, the major emphasis for this Stormwater Design Standards Manual was directed toward organizing the Manual to gain widespread use and acceptance, simplifying procedures to minimize the efforts of the design and the review engineer, incorporating stringent and specific local requirements, and consistency with local standards and procedures.

1.1.2 Glossary

The following terms, phrases, and words, or their derivatives, have been reproduced herein from the Stormwater Management Ordinances adopted by the City of Bismarck. They shall have the meanings as stated in this section. When inconsistent with the context, words used in the present tense include the future tense. Words in plural number include the singular number, and words in the singular number include the plural number. The word "shall" is always mandatory never as permissive. Users of this Manual are encouraged to use these terms, phrases, and words in all reports, stormwater management plans, or other correspondence with the City of Bismarck.

1. "Agricultural Land Use" - The use of land for planting, growing, cultivating and harvesting crops for human or livestock consumption and pasturing or yarding of livestock.

2. "Applicant" - Any person wishing to obtain a building permit, special use permit, zoning, or subdivision approval.

3. "Base Flood" - The flood having a one percent chance of being equaled or exceeded in any given year (i.e., 100-year flood). It is also referred to as the regional flood.

4. "Board" - The Board of City Commissioners of the City of Bismarck.

5. "City" - The City of Bismarck or the Board of City Commissioners of the City of Bismarck.

6. "Control Measure" - A practice or combination of practices to control erosion and attendant pollution.

7. "Conveyance Structure" - A pipe, open channel, or other facility that transports runoff from one location to another.

8. "Detention Facility" - A natural or manmade structure, including wetlands, for the temporary storage of runoff which may contain a pool of water, or may be dry during times of no runoff.

9. "Development Properties" - Lands and properties located within an approved stormwater management permit boundary.

10. "Developer" - A person, firm, corporation, sole proprietorship, partnership, federal or state agency, or political subdivision thereof engaged in a land disturbance and/or land development activity.

11. "E.P.A." - United States Environmental Protection Agency.

12. "Engineer" - The City Engineer of the City of Bismarck or authorized agent.

13. "Erosion" - Any process that wears away at the surface of the land by the action of water, wind, ice, or gravity. Erosion can be accelerated by the activities of man and nature.

14. "Erosion and Sediment Control Plan" - A written description of the number, locations, sizes, and other pertinent information about best management practices designed to meet the requirements of this ordinance.

15. "Extraterritorial Jurisdiction" - Includes all land lying within two (2) miles of the corporate limits of the City as determined by the North Dakota Century Code.

16. "Flood Fringe" - That portion of the flood plain outside of the floodway.

17. "Floodplain" - The areas adjoining a water course or water basin that has been or may be covered by a regional or base flood.

18. "Floodplain Management" - The regulation of the nature and location of construction on (or other occupancy of) lands subject to inundation, so that foreseeable (probable) flooding damages will have an average annual risk smaller than some preselected amount. Floodplain management consists of technical and nontechnical studies, policies, management strategies, statutes and ordinances that collectively manage floodplains along rivers, streams, major drainage ways, outfalls, or other conveyances. The federal government normally plays a major role in floodplain planning and management, whereas in urban stormwater management and design, local governments dominate the decision-making process.

19. "Floodway" - The channel of the water course, the bed of water basins, and those portions of the adjoining flood plains that are reasonably required to carry and discharge floodwater and provide water storage during a regional or base flood.

20. "Hydric Soils" - Soils that are saturated, flooded, or ponded long enough during the growing season to develop anaerobic conditions in the upper part of the soil profile.

21. "Hydrophytic Vegetation" - Macrophytic plant life growing in water, soil, or on a substrate that is at least periodically deficient in oxygen as a result of excessive water content.

22. "Impervious Area" - Impermeable surfaces, such as pavement or rooftops, which prevent the infiltration of water into the soil.

23. "Land Development Activity" - The construction or demolition of buildings, roads, parking lots, paved storage areas, and similar facilities.

24. "Land Disturbing Activity" - Any manmade change of the land surface including removing vegetative cover, excavating, filling and grading, but not including agricultural land uses such as planting, growing, cultivating and harvesting of crops; growing and tending of gardens; and harvesting trees.

25. "Landowner" - Any person holding title to or having an interest in land.

26. "Land User" - Any person operating, leasing, renting, or having made other arrangements with a landowner by which the landowner authorizes use of their land.

27. "Local Detention" - Detention provided to serve only the developing area in question and no areas outside of the development boundaries. This is also known as on-site detention.

28. "Local Drainage System" - The storm drainage system which transports the minor and major stormwater runoff to the major stormwater system and serving only the property within the development boundaries in question. It is also known as the on-site drainage system.

29. "Major Stormwater System" - The portion of the total stormwater system that collects, stores, and conveys runoff that exceeds the capacity of the minor system. The major drainage ways are readily recognizable natural or improved channels that convey runoff that exceeds the capacity of the Minor Drainage System, including emergency overflow facilities. It transports the minor and major stormwater runoff and serves more than the area within the development boundaries in question. The major system is usually less controlled than the minor system, and will function regardless of whether or not it has been deliberately designed and/or protected from encroachment, including when the minor system is blocked or otherwise inoperable. The major stormwater system is usually evaluated for the 100-year runoff event.

30. "Management Practice" - A practice or combination of practices to control erosion and water quality degradation.

31. "May" - May is permissive.

32. "Minor Stormwater System" - The portion of the total drainage system that collects, stores and conveys frequently occurring runoff, and provides a relief from nuisance and inconvenience. This system has traditionally been carefully planned and constructed, and normally represents the major portion of the urban drainage infrastructure investment. The degree of inconvenience the public is willing to accept, balanced against the price it is willing to pay, typically establishes the drainage capacity or design recurrence frequency of a minor system. Minor systems include roof gutters and on-site drainage swales, curbed or side swaled streets, stormwater inlets, underground storm sewers, open channels and street culverts. Generally, the minor stormwater system corresponds to the minor (or ordinary) storm recurring at regular intervals, generally from two to ten years.

33. "Multiple-Purpose Facility" - An urban stormwater facility that fulfills multiple functions, such as enhancement of runoff quality, erosion control, wildlife habitat, or public recreation, in addition to its primary purpose of conveying or controlling runoff.

34. "National Pollution Discharge Elimination System (NPDES) Permit" - Any permit or requirement enforced by the North Dakota State Department of Health pursuant to the Clean Water Act as amended for the purposes of regulating stormwater discharge.

35. "Natural Outlet" - Any outlet including storm sewers and combined sewer overflows, into a watercourse, pond, ditch, lake or other body of surface or groundwater.

36. "On-Site Detention" - Detention provided to serve only the developing area in question and no areas outside of the development boundaries. It is also referred to as local detention.

37. "Outfall Facility" - Any channel, storm sewer, or other conveyance receiving water into which a storm drain or storm drainage system discharges.

38. "Owner or Occupant" - Any person owning or using a lot, parcel of land, or premises connected to and discharging stormwater into the stormwater system of the City, and who pays for and is legally responsible for the payment of stormwater rates or charges made against the lot, parcel of land, building or premises, if connected to the stormwater system or who would pay or be legally responsible for such payment.

39. "Permanent Development" - Any buildings, structures, landscaping and related features constructed as part of a development project approved under a stormwater permit.

40. "Permanent Facilities" - Those features of a stormwater management plan which are part of any natural or constructed stormwater system that requires periodic or minimal maintenance to retain their operational capabilities. This includes but is not limited to storm sewers, infiltration areas, detention areas, channels, streets, etc.

41. "Permittee" - Any person who applies for and receives a stormwater permit from the City.

42. "Person" - Any developer, individual, firm, corporation, partnership, franchise, association, or agency - public or private.

43. "Private Drainage Channel" - A drainage channel on privately-owned land or easements which eventually discharge into a public drainage channel or public storm sewer.

44. "Private Storm Sewer" - A storm sewer on privately-owned land or easements which eventually discharge into a public drainage channel or public storm sewer.

45. "Public Drainage Channel" - A drainage channel located entirely within a naturally occurring or constructed watercourse.

46. "Public Storm Sewer" - A storm sewer located entirely within publicly owned land or easements.

47. "Regional Detention" - Detention facilities provided to serve an area outside the development of boundaries in question. A regional detention site generally receives runoff from multiple stormwater sources.

48. "Regional Flood" - A flood that is representative of large floods known to have occurred generally in the state and recently characteristic of what can be expected to occur on an average frequency in the magnitude of a 100-year recurrence interval. It is also referred to as the base flood.

49. "Retention Facility" - A natural or manmade structure that provides for the storage of stormwater runoff by means of a pool of water.

50. "Runoff" - The rainfall, snowmelt, dewatering, or irrigation water flowing over the ground surface and into open channels, underground storm sewers, and detention or retention ponds.

51. "Sediment" - Solid material or organic material that, in suspension, is being transported or has been moved from its site by air, water, gravity, or ice, and has been deposited at another location.

52. "Shall" - Shall is mandatory.

53. "Site" - The entire area included in the legal description of the parcel or other land division on which the land development or land disturbing activity is proposed in the permit application.

54. "Stabilize" - To make the site steadfast or firm, minimizing soil movement by mulching and seeding, sodding, landscaping, concrete, gravel, or other measures.

55. "State" - The State of North Dakota.

56. "Storm Sewer" - A pipe or conduit for carrying storm waters, surface runoff, street and wash waters, and drainage, excluding sewage and industrial wastes.

57. "Stormwater Detention" - Temporary storage of stormwater runoff in ponds, parking lots, depressed grassy areas, roof tops, buried underground tanks, etc., for future or controlled release. Used to delay and attenuate flow.

58. "Stormwater Management" - The planned set of public policies and activities undertaken to regulate runoff under various specified conditions within various portions of the urban drainage system. It may establish criteria for controlling peak flows or runoff volumes, for runoff detention and retention, or for pollution control, and may specify criteria for the relative elevations among various elements of the drainage system. Stormwater management is primarily concerned with limiting future flood damages and environmental impacts due to development; whereas flood control aims at reducing the extent of flooding that occurs under current conditions.

59. "Stormwater Management Criteria" - Specific guidance provided to the engineer/designer to carry out drainage and stormwater management policies. An example might be the specification of local design hydrology - the design storm.

60. "Stormwater Management System" - Physical facilities that collect, store, convey, and treat stormwater runoff in urban areas. These facilities normally include detention and retention facilities, streets, storm sewers, inlets, open channels, and special structures, such as inlets, manholes, and energy dissipaters.

61. "Stormwater Retention" - Storage designed to eliminate subsequent surface discharge. Wet ponds are the most common type of retention storage (though wet ponds may also be used for detention storage).

62. "Structure" - Anything manufactured, constructed, or erected which is normally attached to or positioned on land, including portable structures, earthen structures, roads, parking lots, and paved storage areas.

63. "Unpolluted Water" - Any water of quality equal to or better than the effluent criteria in effect or water that would not cause violation of receiving water quality standards and would not be benefited by discharge to sanitary sewers and waste water treatment facilities provided.

64. "Urban Area" - Land associated with, or part of, a defined city or town. This title of the Code of Ordinances applies to urban or urbanizing, rather than rural, areas.

65. "User" - Any person, who discharges, causes or permits the discharge of stormwater into the City's stormwater management system.

66. "User Fee" - A fee levied on users of a stormwater management system for the user's proportionate share of the cost of operation and maintenance (including replacement) of such works.

67. "Watershed Master Plan" - The plan that an engineer/designer formulates to manage urban stormwater runoff for a particular project or drainage area. It typically addresses such subjects as characterization of the site development and grading plan; peak rates of runoff, flow duration, runoff volumes for various return frequencies; locations, criteria and sizes of detention or retention ponds and conveyances; runoff control features; land parcels, easement locations, opinions of probable costs, measures to enhance runoff quality, salient regulations, and how the plan addresses them, and consistency with secondary objectives such as public recreation, aesthetics, protection of public safety, and groundwater recharge. It is usually submitted to regulatory officials for their review for adoption.

68. "Wetlands" - Lands transitional between terrestrial and aquatic systems where the water table is usually at or near the surface or the land is covered by shallow water. For purposes of this definition, wetlands must have the following three attributes:

    a. A predominance of hydric soils;
    b. Are inundated or saturated by the surface or groundwater at a frequency and duration sufficient to support a prevalence of hydrophytic vegetation typically adapted for life in saturated soil conditions; and
        c. Under normal circumstances support the prevalence of such vegetation.

1.1.3 General Requirements

Projects to control or collect stormwater runoff within the City of Bismarck shall be designed within the guidelines and information set forth within this Manual. Failure to do so may result in the denial of a stormwater management permit applied for in accordance with the Stormwater Management Ordinance. The concepts provided herein represent the current standards within the industry and the field of professional design practice. Since this field is dynamically changing, this Manual is not all-inclusive and other methods and design concepts can be submitted. However, approvals of other methods or design concepts are subject to the discretion and approval of the City Engineer. Such methods will require justification for their use and verification of the intended purpose and results. The design engineer is encouraged to use cost-effective designs, the most appropriate design methods, and good engineering judgment. The design engineer shall be guided by the following requirements:

1. Design data provided by the design engineer should demonstrate that stormwater management investigations include:

    a. The function of the streets as part of the stormwater conveyance system.
    b. Assurance that gutters and intakes are adequate to prevent excessive flooding of streets and parking areas.
    c. Assurance that culverts and storm sewer pipes are designed to sufficient size and hydraulic capacity.
    d. Assurance that adequate overland relief is present for storm events larger than the design storm, or that an adequate storage volume and a release system are provided to accommodate a 100-year flood with a minimum of 3 feet of freeboard.
    e. Assurance that street grades are coordinated with lot drainage and that lot drainage slopes will not be less than 0.5%.

2. An evaluation should be made of stormwater management alternatives to manage the runoff resulting in the selection of the optimum design which strikes a balance between initial capital costs, operation and maintenance costs, public convenience and safety, and environmental protection.

3. Runoff analysis should be based upon the proposed or projected land use, and should take into consideration all contributing runoff from areas outside of the immediate project area.

4. All undeveloped land lying outside of the study area should be considered as fully developed based upon the Comprehensive Drainage Master Plan for the area, if one exists. The probable future flow pattern in undeveloped areas should be based on existing natural topographic features (existing slopes, drainage ways, etc.). Average land slopes in both developed and undeveloped areas may be used in computing runoff, however, for areas in which drainage patterns and slopes are established, these should be utilized.

5. Flows and velocities which may occur at a design point when the upstream area is fully developed should be considered. Drainage facilities should be so designed that such increased flows, velocities and durations will not cause erosion and/or sedimentation damage.

6. Streets should not be used as floodways for the initial storm runoff. The primary use of streets should be for the conveyance of traffic. The computed amount of runoff in streets should not exceed the requirements set forth herein.

7. The use of on-site detention and natural drainage ways is recommended and encouraged whenever possible. The relocation of a natural drainage way or a substantive change in a

watershed boundary will not be approved unless such change is shown to be without unreasonable hazard and liability, substantiated by thorough analysis and investigation.

8. Restrictive covenants (agreements or contracts), surface flow easements and impoundment easements may be required to be executed and recorded to provide for the protection and maintenance of drainage swales and drainage detention areas within built-up areas. In the use of natural channels, the design engineer may be required to show that the project will have minimum disruption of the existing environment, and covenants may be required by the City Engineer to be executed and recorded to provide this protection.

9. In the design of stormwater management systems, consideration should be given to both surface and subsurface water sources. Subsurface drainage systems should be designed where required and should not allow flow over sidewalks or onto streets after completion of the project.

10. Land grading of the project site should be performed to take advantage of existing contours and minimize soil disturbance. Steep slopes should be avoided. If steep slopes are necessary, an attempt should be made to save natural grasses, shrubs, and trees on these slopes and reestablish ground cover and permanent erosion control measures as soon as possible.

11. During construction grading phases, temporary diversions, contour furrows, terraces and other remedial conservation practices should be used to reduce erosion and excessive water drainage to downstream adjacent properties. Sediment traps and basins should be used at the lower end or along drainage ways and provisions should be made for their maintenance.

12. The planning and design of drainage systems should be such that problems are not transferred from one location to another. Under no circumstance shall significant drainage be diverted and/or released overland to a downstream property at points not receiving such drainage prior to the proposed development. Also, flows shall not be concentrated onto downstream properties where sheet flow previously existed; unless it is determined these properties can accommodate these flows without adverse impacts.

13. Where a proposed development or land disturbing activity discharges into an existing storm drain, the developer shall provide to the City the hydraulic capacity of that drain and computations for that capacity.

14. In the event that stormwater runoff from a development drains into an existing flood drainage area and/or erosion problem area, as determined by the City Engineer, the peak discharge from this site shall not be greater than the peak leaving this site prior to development.

15. In the event that runoff from a proposed land disturbing activity has in the past discharged directly into a relatively large body of water, such as a river, or has or could discharge to such bodies of water via a ditch or pipeline sized to accommodate anticipated increased runoff in the proposed land development, then it shall be the sole decision of the City Engineer to permit or not permit such increased runoff to said bodies of water from the proposed land disturbing activity.

16. Floodplain information (floodway and floodway fringe) will be required on all preliminary and final drainage plans when known, and should include the area inundated by the major storm runoff. Lands that lie within "flood hazard zones" as shown on the appropriate maps prepared by the Federal Emergency Management Agency, shall comply with the regulations of the National Flood Insurance Program. The City of Bismarck may designate floodplains in areas not shown on the Flood Insurance mapping, using approximate or detailed methods of study.

17. Where a comprehensive urban stormwater master plan is available, the flow routing for both the minor storm and the major storm runoff events should conform to said plan. Drainage easements conforming to the master plan will be required and shall be designated on all drainage drawings and subdivision plats.

18. Any proposed building or construction of any type of structure including retaining walls, fences, etc., or the placement of any type of fill material which will encroach on any utility or drainage easement, requires written approval of the City Engineer. The function of the drainage easement (conveyance, ponding, etc.) shall not be impaired, nor shall drainage from adjacent areas be impeded.

19. All stormwater management systems, including collection, conveyance and restrictions (i.e., detention/retention), not located on city property, shall be so located in dedicated public rights-of-way or easements approved by the City. All drainage rights-of-way or easements shall be shown on construction plans and on the final subdivision plat.

20. The design for stormwater management facilities should be in conformance with the following requirements:

    a. Requirements and standards of the North Dakota State Water Commission.
    b. Requirements and standards of the North Dakota Department of Health.
    c. Design standards and construction standards of the City of Bismarck.
    d. The most current plumbing code.
    e. The requirements and standards of the North Dakota State Department of Transportation, where applicable.
    f. In case of conflict between the above design standards, the most restrictive requirement should apply.

21. Construction standards should be in accordance with the most recent revision of the "Construction Specification for Municipal Public Works Improvements - Bismarck, North Dakota", together with the latest addenda. All details, materials, and storm sewer appurtenances should conform to these standards.

22. No polluted water or water from a polluted source shall be discharged into any stormwater management system. This includes wash water and interior building drainage water. Snow disposal sites or run off from irrigations systems may be included under special circumstances.

23. National Pollutant Discharge Elimination System (NPDES) permit. The North Dakota Department of Health administers the NPDES program and permit requirements for stormwater discharges within the State of North Dakota. The design of stormwater management facilities shall comply with the requirements and standards of the North Dakota Department of Health for stormwater discharge permits. See Chapter 11 of this Manual for specific details regarding the permit requirements for erosion and sedimentation control.

1.2 STORMWATER MANAGEMENT POLICY

1.2.1 General Policy

The City of Bismarck's adopted Stormwater Management Ordinance and Stormwater Design Standards Manual are based upon a set of policies outlined as follows:

1. The City of Bismarck considers stormwater to be an integral part of the overall urban system, and shall require stormwater management planning for all developments or land disturbing activities to include the allocation of space for stormwater management facility construction and maintenance, which may entail the dedication of right-of-way and/or easements.

2. The City promotes unified stormwater drainage efforts through an integrated Stormwater Management Plan.

3. The City of Bismarck encourages the development of local stormwater master plans for local flood control facilities which are compatible with regional master plans. These local stormwater master plans shall also set forth site requirements for new development and identify required public improvements.

4. The City recognizes that runoff volume will increase with further development of contributing watersheds. However, the City will require that peak runoff rates not exceed existing conditions and not exceed the capacity of downstream conveyance facilities. Runoff rates shall be controlled by the use of a regional or on-site detention facility.

5. Subwatershed discharge rates and flood storage volumes shall be consistent with an approved stormwater management plan, or a regional stormwater management plan adopted by the City of Bismarck.

6. Stormwater management improvements shall be designed based on the critical storm event for the drainage area.

7. Stormwater conveyance systems shall be designed to insure flood protection for the improved area and for all receiving water resources downstream.

8. The City requires the optimal use of lakes, ponds, and wetlands throughout the City for storing storm water runoff and improving water quality and other amenities.

9. Stormwater discharges may be routed to water treatment ponds or other acceptable facilities before discharging to lakes, streams, or wetlands.

10. The City requires the submission and implementation of an erosion and sediment control plan to prevent erosion or sedimentation from land disturbance activities.

11. The City encourages the design of stormwater management facilities and other measures which enhance the quality of stormwater runoff.

12. The City requires that property owners or developers be responsible for the maintenance of all privately-owned on-site drainage facilities including, but not limited to, inlets, pipes, channels, and detention basins, unless modified by separate agreement. Should the property owner or developer fail to adequately maintain said facilities, the City shall have the right to enter said property, upon notice, for the purposes of maintenance. All such maintenance costs shall be assessed against the owner.

13. The City will provide for the revision of its stormwater ordinance and encourage changes to the Stormwater Design Standards Manual as new technology is developed and experience gained in the use of these standards.

14. The City shall require all new development or land disturbing activities to provide for the planning, design, and construction of stormwater management facilities for both the minor and major storm events, and include emergency flow paths for flows exceeding the major storm event. The minor storm event shall have a recurrence interval of 2 to 10 years, depending on the degree of protection required. The major storm event shall have a recurrence interval of 100 years.

15. The City shall require all Stormwater Management Plans, studies, and construction plans/specifications to be reviewed and approved by the City Engineer.

16. The City encourages the flood proofing of existing structures located within a designed floodplain area which are not built in conformance to the adopted floodplain regulations.

1.2.2 Conformance with Other Ordinances and Regulations

It is the policy of the City of Bismarck to be in conformance with other ordinances and regulations in the vicinity of Bismarck. These existing ordinances and regulations of local, state, and federal jurisdictions are as follows:

1. City of Bismarck. The City of Bismarck has a number of ordinances which are organized as titles, with each title having a number of chapters. The Stormwater Management Ordinance falls under Title 14, Zoning. Title 1 of the City Ordinances, General Provisions, contains general principles and definitions that are applicable to other titles in the Code of Ordinances. Title 11, Utilities, discusses the definitions of various utilities such as water, sanitary sewer, and storm sewers. Reference is made to all other titles for several concepts of establishing a stormwater utility, and for establishing utility rates and charges.

   In Chapter 14-04, special functional Districts within the City, are defined and regulations for each district are outlined. Of particular interest, is Section 14-04-19, Floodplain Districts. Chapter 14-09 contains all regulations governing the subdivision of land for development purposes. The provisions within this chapter are directly related to the stormwater management ordinances contained herein.

   Appendix B of the Code of City Ordinances contains the City of Bismarck Home Rule Charter. The Home Rule Charter of the City provides the authority under which the City operates. In addition, the Home Rule Charter provides for the adoption and amendment of ordinances.

2. Burleigh County. Burleigh County has jurisdiction for all county road systems, and aids townships in the administration of township road systems. Because county and township roads cross natural and manmade drainage systems, it is necessary to provide for bridges or culverts to pass the required drainage flows through the road embankment. The Burleigh County Engineer has the responsibility for determining the required waterway area and hydraulic capacity of drainage structures placed in the county road system.

3. Burleigh County Water Resource District. The Burleigh County Water Resource District is responsible for general water management and drainage activities within the County. The City of Bismarck has agreed to provide the Burleigh County Water Resource District with copies of stormwater management plans and permit applications for those lands located outside the city limits. The District, upon review of said applications, will present written comments and recommendations to the City Engineer for consideration and possible inclusion in any approved permit. The District will also be provided information on any plan or application within the city limits which may result in measurable increases in stormwater runoff, outside of the City limits, as determined by the City Engineer.

4. State of North Dakota. The State of North Dakota has several agencies and commissions which directly or indirectly affect stormwater management through regulation and permit authority. These agencies include:

   a. State Water Commission: The State Water Commission has broad and general powers with respect to controlling the appropriation of groundwater and surface water within the State, coordination of large-scale drainage activities, development of flood control projects, and serves as a point of contact with federal agencies in water resource development projects. The State Engineer has the authority to grant permits for appropriation of water, drainage projects, the design and construction of dams, and other water control projects.

   b. North Dakota Department of Health: The North Dakota Department of Health has overall responsibility for community water supply, waste water disposal, solid waste management, and river and lake water quality standards within the State. In many instances, the State Health Department administers federal pollution control and water quality management programs. These include the National Pollution Discharge Elimination System (NPDES) permits, and stormwater management permits mandated by the Clean Water Act.

   c. North Dakota Department of Transportation: The North Dakota DOT has responsibility for the development of major highway systems and airport development within the State. The NDDOT designs and constructs many facilities which impact the drainage of water within any given locale. As highways cut across drainage paths, bridges, culverts and other water handling facilities must be developed in such a fashion that they do not hinder existing or planned drainage patterns.

   d. State Historical Society: The State Historical Society has responsibility for administering federal regulations pertaining to the protection of cultural and archaeological sites of significance. Any construction project which may impact designated significant archaeological or cultural sites must receive a clearance from the State Historical Preservation Officer.

5. Federal. There are several federal agencies which have rules, regulations, or administer laws impacting stormwater management within the City. These agencies include:

   a. Environmental Protection Agency: The Environmental Protection Agency has responsibility for administering federal laws pertaining to control of water pollution, solid waste management, the quality of water supplies, and other areas including noise and air pollution. In the State of North Dakota, the State Health Department has been designated to administer several federal programs which fall under the EPA. These include stormwater management criteria contained within the Federal Clean Water Act.

b. U.S. Army Corps of Engineers: The U.S. Army Corps of Engineers has responsibility for regulating and permitting activity within the navigable waters of the United States. In the City of Bismarck, the Missouri River is a navigable waterway falling under the Corps' jurisdiction for regulation and permit authority. The Corps of Engineers is also responsible for administering Section 404 of the Clean Water Act which prohibits the placement of fill material in a wetland without a permit issued by the Corps of Engineers. Therefore, any work involving stormwater management which affects existing wetlands within the City or its extraterritorial limits may fall under the Corps' jurisdiction and could require a permit from the Omaha District Corps of Engineers.

1.3 STORMWATER DESIGN OBJECTIVES

1.3.1 Introduction

Stormwater management policy is implemented through the establishment of goals, which in turn are supported by specific objectives. The goals and the objectives of Bismarck's Stormwater Management Ordinance and the Stormwater Design Standards Manual are intended to provide a reasonable, manageable, and enforceable approach to controlling undesirable impacts from urban stormwater runoff in an environmentally responsible manner. These impacts include, but are not limited to, increased runoff volumes, flood peaks, flow duration, increased soil erosion, sedimentation, and water quality degradation. The Stormwater Design Standards Manual outlines methods by which persons applying for a stormwater management permit can comply with the goals and objectives outlined in the following paragraphs.

1.3.2 Water Quantity

Goal: Stormwater design considerations should focus on the prevention of damage to the development site, streams, drainage ways, streets, and public and private property.

1. Emphasis should be placed on the detention and storage of runoff with the goal of not increasing the discharge rate downstream from that which is existing prior to development.

2. Minimize the increase in the rate of flow from developing properties unless downstream facilities are designed to accommodate the increased flow rates.

3. Require local detention storage for new developments when the development increases flow and downstream conveyance capacities of the local drainage system are demonstrated not to be capable of handling undetained flows.

4. The capacity of downstream conveyance systems shall be analyzed and their adequacy determined based on runoff from the development as fully improved.

5. Local detention will be required when designated in a Regional Stormwater Master Plan to reduce the peak runoff rate in regional facilities.

6. The use of streets for stormwater drainage is allowable within specific limitations, as called for in the Stormwater Design Standards Manual.
7. Retain the 100-year flood flows within dedicated streets, storm sewer easements, streams, and within public properties, with the exception of a natural river and streams with designated floodplains and floodways.

1.3.3 Water Quality

Goal: Stormwater management planning and design shall protect and/or enhance the quality of stormwater runoff.

1. Design stormwater management facilities which enhance the quality of stormwater runoff.

2. Promote the utilization of infiltration systems for the management of both water quantity and water quality.

3. Encourage the utilization of grassed or other buffer areas for runoff quality management.

4. Promote the utilization of "wet pond" detention, where deemed acceptable and appropriate, for the enhancement of water quality.

1.3.4 Erosion and Sedimentation

Goal: To reduce or eliminate the potential for soil erosion and sedimentation through proper stormwater management planning and design.

1. Require flows to be discharged to downstream properties at non-erosive velocities and depths of flow.

2. Maintain the flow of stormwater runoff within its natural drainage path unless reasonable use is demonstrated otherwise.

3. Require construction operations to utilize techniques such as silt fences, on-site sediment detention ponds, or other "best management practices" (BMPs) to prevent erosion of recently disturbed areas.

4. Insure the establishment of vegetation and/or other land covers as quickly as possible after completion of any land disturbing activity.

5. Reduce or eliminate sedimentation in stormwater discharge outlets during and after development.

1.3.5 Operation and Maintenance

Goal: Stormwater management facilities should be designed for operational reliability, minimum maintenance, and the ability to function as intended.

1. Require all stormwater management facilities to be designed to minimize facility maintenance as well as to provide ease of maintenance.

2. Require that all stormwater management facilities be provided with reasonable access for maintenance.

3. A minimum 30-foot wide stormwater easement shall be provided for all publicly-maintained drainage facilities.

1.4 MINOR AND MAJOR DRAINAGE SYSTEMS

Every urban area has two separate and distinct drainage systems, whether or not they are actually planned or designed. One is the Minor Drainage system, and the other is the Major Drainage System.

The Major Drainage System is designed to convey runoff from the 100-year recurrence interval flood to minimize health and life hazards, damage to structures, and interruption to traffic and services. In the City of Bismarck and its extraterritorial jurisdiction, a major drainage system is defined as any watershed with a named river or stream, natural or manmade, as its primary conveyance feature. For application purposes, only the following names shall be used in this Stormwater Design Standards Manual:

1. Apple Creek
2. Hay Creek
3. Jackman Coulee
4. Missouri River
5. South Bismarck
6. Tyler Coulee

The Minor Drainage System is designed to transport the runoff from a 2 to a 10-year recurrence interval flood, depending upon land use, with a minimum disruption to the urban environment. The design of the Minor Drainage System shall also account for the conveyance of the 100-year recurrence interval flood event. Minor stormwater drainage can be conveyed in the curb and gutter area of the street or roadside ditch, by storm sewer piping, channel, or other conveyance facilities. A Minor Drainage System within the City of Bismarck and its extraterritorial jurisdiction is considered to be any subwatershed located within a Major Drainage System having as its primary conveyance feature a tributary to the named river or stream and/or a constructed storm sewer system and associated surface flow corridor.

A typical drainage system within a subdivision would consist of flow in the storm sewer and allowable flow in the gutter, which in combination would carry the flows from the minor storm without effects of detention. These flows would be discharged to a larger sewer system or an open channel, having capacity adequate for larger floods. As the storm intensity increases, stormwater detention should be used to reduce the developed flood peaks to undeveloped levels. During the calculation of the major storm runoff, the benefits of upstream on-site detention can be accounted for during the routing of flood peaks through the development.

1.5 WATER QUALITY

Stormwater runoff in urban and non-urban areas has generally been recognized as having inferior quality. Pollutants in the stormwater runoff from urban areas consist of street, household, and commercial litter; soil erosion; lawn and garden litter; chemicals and salts from winter ice control; pesticides, hydrocarbons, heavy metals, herbicides, fertilizers, and bacteriological pollutants. The rural runoff is characterized by sand and silt erosion, fertilizers, and bacteriological pollutants. Industrial area pollutants consist of silts, hydrocarbons, heavy metals, salts, and other chemicals.

The water quality goals and objectives for the City of Bismarck are not intended to compare or analyze detailed stormwater runoff with specific testing requirements in order to set criteria. General criteria are established by the North Dakota Department of Health and should be referenced and used accordingly, based on project size and complexity. The City of Bismarck encourages measures in the design of stormwater management systems which will not only prevent introduction of additional pollutants, but will improve overall water quality, and to maintain water quality downstream from any land disturbing activities.

CHAPTER 2
STORMWATER MANAGEMENT PLAN REPORT

2.1 PURPOSE AND CERTIFICATION

A Stormwater Management Plan Report is a document prepared by the design engineer to comply with the City of Bismarck's Stormwater Management Ordinance. The purpose of the Stormwater Management Plan Report is to document methodology, assumptions, and analyses used to arrive at specific solutions to stormwater management problems that may occur as a result of the proposed development or land disturbing activity. The Report must include an adequate description of the topography to verify all conclusions regarding off-site drainage. Unless known, the capacity of downstream structures must be thoroughly analyzed to determine their ability to convey the developed discharge.

The Stormwater Management Plan Report shall be reviewed and approved by the City Engineer prior to the preparation of final construction drawings and the commencement of any land disturbing activities. Approval of the Stormwater Management Plan Report shall constitute only a conceptual approval and shall not be construed as approval of specific design details. The design engineer may be required to submit the Stormwater Management Plan Report to other Federal, State and Local agencies. Other agencies which may require a stormwater management plan report as part of their permit application process include the North Dakota State Water Commission, the North Dakota Department of Health, the U.S. Army Corps of Engineers, and the Burleigh County Water Resource District.

Certification of the Stormwater Management Plan Report by a Professional Engineer Registered in the State of North Dakota is required to document that the plan has been developed using acceptable design criteria, following generally accepted engineering practice as provided for in this Manual or as approved by the City Engineer.

2.2 CONTENTS

2.2.1 Text

The following information, presented in outline form, should be considered in the preparation of Stormwater Management Plan Reports. This is provided as a guidance, and is not to be construed as all inclusive, or as the specific information to be required on every Stormwater Management Plan Report. Existing and proposed conditions for each development or land disturbing activity will require analysis unique to that area and that specific project. Smaller projects requiring less area may require less information than larger projects. Some areas will have unique natural features that must be addressed in more detail, relative to other projects. This information does not preclude the utilization of methods or topics other than those referred to in the outline, nor does it relieve the designer of responsibility for analysis of problems not specifically mentioned.

Depending upon the complexity of the project or land disturbing activity, the Stormwater Management Plan Report may range in size from a simple letter with attached drawings to a formal Engineer's Report of much greater magnitude and detail. The extent of a report required for a specific project must be coordinated with and approved by the City Engineer. Upon completion of the project, the City Engineer may require that the Stormwater Management Plan Report initially submitted be revised to reflect any changes or modifications to the project carried out during its construction.

A suggested outline of topics to be included in the Stormwater Management Plan Report is as follows:

I. General Location and Description
   A. Location
   B. Description of Property
   C. General Project Description
   D. Construction Schedule

II. Drainage Basins and Sub-Basins
   A. Major Basin Description
   B. Sub-Basin Description

III. Stormwater Design Criteria
   A. Regulations
   B. Development Criteria
   C. Hydrological Criteria
   D. Hydraulic Criteria
      1. Storm Sewers
      2. Storm Sewer Inlets
      3. Streets, Curb and Gutter
      4. Open Channel Design
         a. Natural
         b. Artificial
      5. Culverts and Bridges
      6. Detention/Retention Ponds
      7. Stormwater System Outlet Capacity
      8. Floodplain and Floodway
      9. Erosion and Sedimentation

IV. Stormwater Facility Design
   A. General Concepts
      1. Existing Drainage Patterns
         a. Minor System
         b. Major System
      2. Site Specific Stormwater Problems
         a. Minor System
         b. Major System
         c. Water Quality

3. Proposed Stormwater Improvements
    a. Minor System
    b. Major System
    c. Water Quality
B. Design of Stormwater Facilities
    1. Storm Sewers
    2. Storm Sewer Inlets
    3. Streets, Curb and Gutter
    4. Channels
        a. Natural
        b. Artificial
    5. Culverts and Bridges
    6. Detention/Retention Ponds
    7. Water Quality Protection
C. Design of Erosion and Sedimentation Control Facilities
    1. Temporary
    2. Permanent
D. Analysis of Outlet Adequacy

V. Conclusions
    A. Compliance with Standards
    B. Recommendations

VI. References

VII. Tables

VIII. Charts and Graphs

IX. Appendices
    A. Hydrological Calculations
    B. Hydraulic Calculations
    C. Erosion and Sedimentation Calculations

X. Drawings

2.2.2 Discussion of Report Contents

The first two sections of the Report (I and II) contain information describing the location and existing condition of the property or land(s) to be disturbed. A general description of the project then follows including the purpose, function, and potential impact of the project to existing drainage patterns in the area. The major basin description illustrates the overall hydrological setting of the project in relation to downstream property and receiving waters for the proposed drainage improvements. The subbasin description provides information on the contributing drainage area to the project outlet for both pre- and post-project conditions. This is where an assessment of upstream development activity must be addressed from the standpoint of changes to land use and the hydrologic condition for the contributing drainage area.

The third section of the report (III) provides an opportunity to cite the regulations and design criteria utilized for the proposed project. Pertinent regulations are cited, along with specific development criteria used to design the project. Hydrological criteria would include drainage areas, watershed soils, pre- and post-project land use, runoff coefficients, rainfall intensity, and design frequencies. The hydraulic criteria are then described for the design of the stormwater conveyance facilities. These criteria include maximum allowable headwater, maximum allowable outlet velocities, detention pond storage and release requirements, maximum capacity and freeboard of open channels, maximum capacity of street-curb and gutter systems, conformance with floodplain and floodway criteria, and hydraulic design for erosion and sedimentation control.

The section on Stormwater Facility Design (IV) provides for the documentation of specific design assumptions, requirements, and methodologies. Generally, the existing drainage patterns, site specific stormwater problems, and proposed stormwater improvements are addressed with reference to the minor drainage system, the major drainage system, and water quality concerns. Documentation of the detailed design for each specific stormwater facility can then be discussed. Documentation for erosion and sediment control designs are generally done for two conditions. Temporary facilities are required during project construction, when the site is in its most vulnerable condition. Permanent erosion and sediment control facilities are designed to accommodate the proposed development after it is completed and in operation. An important part of the stormwater facility design is the analysis of outlet adequacy. In order to comply with stormwater management policy, this analysis must be made to determine the ability of the downstream outlets to accommodate any changes to hydrologic loadings due to post-development watershed conditions.

The Stormwater Management Plan Report then concludes with a statement of compliance with the standards outlined in the Stormwater Design Standards Manual and Stormwater Management Ordinances. Recommendations are also provided as to the most optimal set of actions which constitute the proposed Stormwater Management Plan for the project.

The Stormwater Management Report should contain tables of data, charts, and graphs supporting the information and conclusions presented in the report. Detailed calculations for the hydrology, hydraulics, erosion and sediment design may be included as appendices to the report.

2.2.3 Report Drawings and Maps

Drawings and maps are critical elements of the Stormwater Management Plan Report. Drawings and maps are required for all levels of Stormwater Management Plan Reports, including those as brief as a letter. At a minimum, the Stormwater Management Plan Report shall contain the following information:

1. Existing site map: A map of existing site conditions showing the site itself and immediately adjacent areas, including:

    a. The name and address of the applicant, the section, township and range, and the north point, date and scale of drawing, and number of sheets;

b. The location of the tract by an insert map at a scale sufficient to clearly identify the location of the property and giving such information as the names and numbers of adjoining waterways, roads, railroads, utilities, subdivisions, towns, and districts or other defining landmarks;

c. Existing topography with a contour interval appropriate to the topography of the land, but in no case having a contour interval greater than two feet;

d. A watershed boundary map illustrating the project site location as a subwatershed within the watershed of the larger or major drainage basin.

e. A delineation of streams, rivers, public waters and the presence or absence of wetlands located on and immediately adjacent to the site, including depth of water, a general description of vegetative cover found within the site, a statement of general water quality, and any classification given to the water body by state or federal agencies;

f. Location and dimensions of existing stormwater drain systems and natural drainage patterns on and immediately adjacent to the site delineating in which direction and at what rate stormwater is conveyed from the site, identifying the receiving stream, river, public drainage channel, public storm sewer, or wetland, and setting forth those areas of the unaltered site where stormwater collects or passes;

g. A description of the soils on the site, including a map indicating soil types of the areas to be disturbed, containing information on the suitability of the soils for the type of development proposed, potential for erosion, the type of stormwater management system proposed, and any remedial steps to be taken by the developer to render the soils suitable.

h. Current extent of vegetative cover and a clear delineation of any vegetation proposed for removal;

i. The current land use of the area in which the site is located; and

j. The 100-year floodplains, flood fringes, and floodways.

2. Construction Site Plan: A Construction Site Plan shall be provided, including:

a. Locations and dimensions of all proposed land disturbing activities and any phasing or scheduling of those activities;

b. Approximate locations of all temporary soil or dirt stockpile areas;

c. Location and description of all construction site erosion control measures necessary to meet the requirements of this ordinance;

d. A schedule of anticipated starting and completion dates for each land disturbing activity, including the installation of construction site erosion control measures needed to meet the requirements of this ordinance; and

e. Provisions for maintaining the construction site erosion control measures prior to, during, and after construction.

3. Plan of Final Site Conditions: A Plan of Final Site Conditions on the same scale as the existing site map showing the proposed site changes shall be provided including:

a. The proposed final grading plan shown at contours at the same interval as provided above or as required to clearly indicate the relationship of the proposed changes to existing topography and remaining features;

b. A landscape plan, drawn to an appropriate scale, including dimensions and distances and the location, type, size and description of proposed landscape materials which will be added to the site as part of the development;

c. A drainage plan of the developed site delineating the direction and at what rate stormwater runoff will be conveyed from the site and setting forth the areas of the site where stormwater will be collected;

d. The proposed size, alignment, and intended use of any structures to be erected on the site;

e. A clear delineation and tabulation of all existing and proposed impervious areas, including a description of the surfacing material to be used; and

f. Any other information pertinent to the particular project which, in the opinion of the applicant, is necessary for the review of the project.

2.2.4 Construction Drawings and Specifications

Where stormwater management improvements are to be constructed in accordance with the approved Stormwater Management Plan Report, the construction plans and specifications shall be submitted to the City Engineer for review and approval prior to construction. The information required for the construction drawings and specifications shall be in accordance with sound engineering principles, the Stormwater Design Standards Manual, the City's Construction Specification for Municipal Public Works Improvements, including the latest addenda and City requirements for subdivision designs. Construction documents shall include geometric, dimensional, structural, foundation, bedding, hydraulic, landscaping, and other details as needed to construct the stormwater management facility. The approved final Stormwater Management Plan Report shall be included as part of the construction documents for all of the facilities affected by the Stormwater Management Plan.

2.2.5 Record Drawings

Record drawings shall be prepared upon completion of construction of all stormwater management facilities. The record drawings shall be clearly noted as such, dated, and reflect the true after-construction condition of the development.

2.2.6 Drawing Certification

In addition to the Stormwater Management Plan Report, all drawings supporting the plan, construction drawings, construction specifications, and record drawings shall contain a certification by a Professional Engineer Registered in the State of North Dakota. This certification shall be affixed to all reports and drawings in accordance with the requirements of Title 28 of the North Dakota Century Code, which governs the registration and practice of professional engineers and land surveyors in North Dakota.

CHAPTER 3
STORMWATER HYDROLOGY

3.1 INTRODUCTION

This chapter provides the criteria and methodology for determining the stormwater runoff design peak flow rates, durations and volumes to be used in the preparation of the Stormwater Management Plan Report, drainage studies, plans, and stormwater management facility design. Since rainfall and resultant runoff are generally considered randomly occurring events, hydrologic design generally incorporates a risk-based approach. That is, facilities are designed to pass flow rates with a specified frequency or probability of occurrence. Therefore, the first section of this chapter discusses design frequencies for minor and major stormwater management facilities. Rainfall characteristics in the vicinity of the City of Bismarck are discussed next. Since the determination of peak discharges is the most common hydrological requirement for most stormwater management facilities, a special section has been devoted to its determination in this chapter. A number of approaches are suggested, including the rational method, NRCS methods, and USGS methods. For more sophisticated structures, the development of stormwater hydrographs are required. Several methods are presented as documented by the NRCS and the Corps of Engineers. These methods provide for the development of hydrographs, channel routing, and reservoir or pond routing. The chapter concludes with a brief discussion of groundwater hydrology and available computer software.

The storm runoff peak, flow duration, volume, and timing provide the basis for planning, design, and construction of stormwater management facilities. The best method for determining stormwater runoff is to measure the runoff from a flood with a known intensity and recurrence interval. Since this approach is seldom practical, various analytical methods have been developed which predict the storm runoff from preselected hydrological conditions (independent of chance). These methods are referred to as deterministic models. Other methods evaluate measured past trends to predict future trends, which are referred to as probabilistic methods (dependent on chance such as a statistical analysis).

The methodologies presented in this chapter are not intended to preclude the use of other methods. However, the designer is advised to secure approval of the City Engineer before utilizing hydrological methods unique to the City of Bismarck and the State of North Dakota.

3.2 DESIGN FREQUENCIES

The selection of the design frequency for a specific stormwater management system is dependent upon location and use of that system. For example, an isolated residential area with limited traffic can tolerate minor street flooding for short to long durations, while a heavily utilized commercial area cannot. In fact, even short duration flooding on smaller events can result in major traffic delays, congestion, and possibly a public safety and health concern. The following section outlines the City's general criteria which shall be used to develop the Stormwater Management Plan for any given area or project.

3.2.1 Minor System Facilities

Generally, minor stormwater management system facilities are designed to manage storm events occurring at regular intervals, generally between 2- and 10-year events. The minor stormwater management system should be designed to provide protection against regularly recurring damage, to reduce street maintenance costs, to provide an orderly urban drainage system, and to provide convenience to urban residents. Storm sewer systems consisting of underground piping, natural drainage ways, and other required appurtenances should be considered as part of the minor stormwater drainage system.

Storm sewers should have a capacity to convey a 2-year storm event for residential areas, and a 5-year storm event for commercial or industrial areas and under developed conditions, with all flows being contained within the pipe. Culverts should have a capacity to convey the 10-year storm event without the headwater depth exceeding the diameter of the culvert and a 50-year storm without the headwater depth exceeding 1 foot over the top of the culvert. These culvert criteria include consideration and use of upstream storage.

Streets, curb and gutter systems should have a capacity for a design storm equal to a 2-year to a 10-year return period, depending upon street classification and land use. Generally, local and collector street systems should be designed for a 2-year to a 5-year event with no curb overtopping and the flow being allowed to spread to the crown of the street. Collector streets should be designed for a 2- to a 10-year design storm with no curb overtopping and the flow spread leaving at least one lane free from water. Minor arterials should be designed for a 10-year event with no curb overtopping and the flow spread leaving at least one lane free of water in each direction. Major arterials or freeways (controlled access) should be designed for a 10-year storm event with no encroachment allowed on any of the traffic lanes.

Detention basins should have the capacity to regulate at least a 10-year storm event at critical duration. The critical duration of a storm event corresponds to the storm duration producing the maximum water surface elevation in the detention pond after flood routing the inflow hydrograph through the pond. In some cases, the design of detention basins may be designed to regulate a variety of storm events up to and including the 100-year event. However, this design capability is heavily dependent upon contributing watershed and topography of the detention site. In all cases, the top of the detention basin should be a minimum of one foot above the 100-year water surface elevation as it is routed through the detention basin.

Ditches and open channels may be designed for a variety of capacities. Ditches which serve as primary surface water collectors in the upper portions of a drainage system will be considered minor drainage facilities. In such a case, they should have the capacity to convey a 50-year storm event within the banks of the channel. Surface water flowage easements, sufficient to contain a 100-year runoff event, should be provided for all designed drainage ways.

3.2.2 Major Stormwater Management System Facilities

Major stormwater management systems generally include natural or artificial channels, and regional detention facilities. In addition, the major storm event may have to be conveyed through some minor system facilities without hindrance. In such cases, provisions should be made for the 100-year storm event to flow overland through the area served by a minor system without damaging private property.

Streets, and curb and gutter systems are minor stormwater management system components which are required to pass a major storm event within certain limitations. In all cases, the major storm event used for this analysis is the 100-year event. For local and collector streets, 100-year events shall be passed such that residential dwelling, public, commercial and industrial buildings should not be inundated at the ground line unless adequately flood proofed. The depth of water over the gutter flow line shall not exceed the public right-of-way or a maximum depth of 18 inches whichever is less.

For arterial streets and freeways, a 100-year event shall be managed in such a way that residential dwellings, public, commercial and industrial buildings shall not be inundated at the ground line unless adequately flood proofed. In addition, the depth of water at the street crown shall not exceed six inches in order to allow the operation of emergency vehicles. The depth of water over the gutter flow line shall not exceed 18 inches.

Culverts and bridges should be analyzed for the major storm event (100-year event) such that the discharge is conveyed through the structure without the headwater exceeding a depth one foot below the elevation of the roadway. In addition, floodplain regulation standards require that any structure placed in the path of a water course should not cause a stage increase to exceed one foot. In some cases, this one foot stage increase may not be acceptable and shall be reduced because of the potential for upstream property damage. The City Engineer should be consulted as to the required floodplain requirements for a given site.

Stormwater detention basins and detention dams should have the capacity to pass a 100-year storm event at critical duration through its principle and/or emergency spillway system, without causing damage to the embankment or downstream property. In some cases, depending upon the hazard classification of the dam or embankment, dam safety standards may require the structure to have a larger discharge capacity sufficient to carry a 500-year flood, a probable maximum flood, or some fraction thereof. The required emergency spillway capacity shall be designed in accordance with the North Dakota Dam Design Handbook, published by the North Dakota State Engineer, June 1985.

Open channel facilities serving as a regional outlet for upstream stormwater drainage systems are considered major system facilities and shall be designed for the 100-year event with one foot of freeboard below the top of channel.

3.3. RAINFALL

3.3.1 Introduction

Precipitation in the Bismarck area can vary greatly from year to year, averaging 16 inches per year. The depth associated with a given rainfall frequency is dependent on the duration of the storm. As will be discussed later, the selection of an appropriate storm duration is dependent upon the hydrological characteristics of the watershed, including its size, soils, developmental conditions, and time of concentration. Standard duration rainfalls, such as the 6-hour or 24-hour events, are usually used for the design of regional facilities. In hydrologic models, the methodologies developed by the National Resource Conservation Service (NRCS) or the Corps of Engineers (COE) are commonly used. In most applications of urban stormwater hydrology, the design storm and its frequency are assumed to correspond to the design runoff event with the same frequency.

3.3.2 Design Storms

Table No. 3.1 presents the magnitude, duration and frequency of precipitation events which shall be used in the design of major stormwater management facilities within the City of Bismarck and its extraterritorial jurisdiction. These values were derived from the National Weather Service Technical Paper No. 40 and the Hydrology Manual for North Dakota as developed by the National Resource Conservation Service. These values of precipitation can also be used as design input to hydrologic models such as TR55 and TR20 (NRCS), or HEC-1 (COE) to design stormwater detention facilities.

3.3.3 Intensity-Duration-Frequency Curves

The design of minor stormwater management systems utilizing the rational method or similar methodologies requires input of a design storm intensity in units of inches per hour. Intensity-duration-frequency curves have been developed by the North Dakota Department of Transportation for the Bismarck area. These curves are shown on Figure No. 3.1. There is a separate IDF curve for each storm frequency. Generally, as duration of the storm increases for a given frequency event, the corresponding intensity decreases.

TABLE NO. 3.1
Precipitation Events and Frequency - City of Bismarck

Frequency	1-Hour Rainfall (inches)	6-Hour Rainfall (inches)	24-Hour Rainfall (inches)	10-Day Snowmelt (inches of moisture)
2-year	1.1	1.6	2.0	0.9
5-year	1.5	2.2	2.8	1.5
10-year	1.8	2.5	3.4	2.0
25-year	2.2	3.0	3.8	2.7
50-year	2.6	3.4	4.3	3.3
100-year	2.9	3.8	4.8	3.9
July 15, 1993[1]	N/A	N/A	4.49	N/A

[1] As reported by the National Weather Service at the Bismarck Airport.

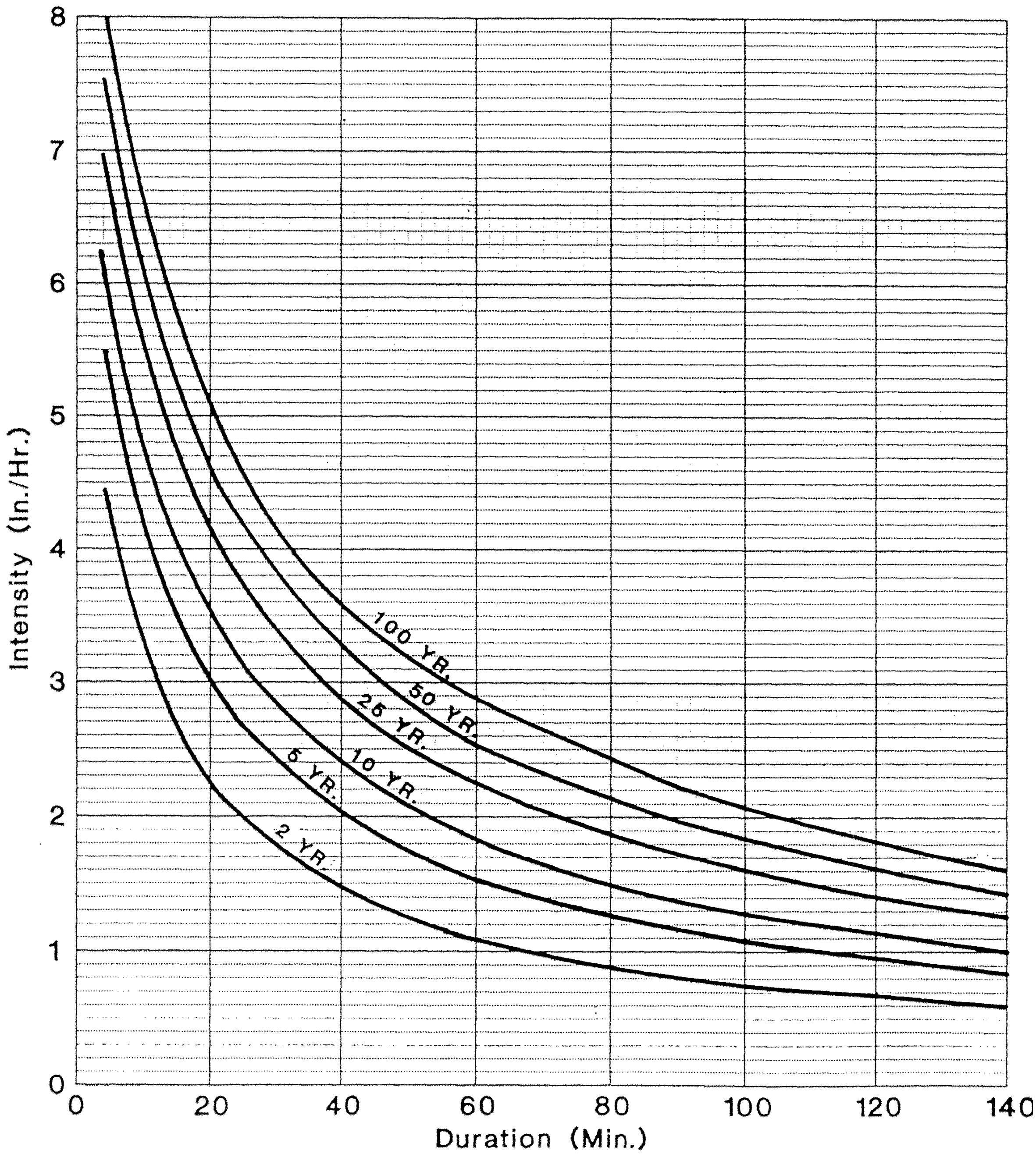
Duration

Figure No. 3.1

Intensity-Duration-Frequency (IDF): Bismarck, North Dakota

Source: HYDRAIN – FHWA–RD–92–061
Integrated Drainage Design – Computer System

DATA Base Source:
(1) National Weather Service (NWS) technical memorandum HYDOR–35
(2) National Oceanic and Atmospheric Administration (NOAA) Atlas 2 doc.



3.4 PEAK DISCHARGE HYDROLOGY

3.4.1 Selection of Methodology

Many types of stormwater management facilities are designed for peak discharge loadings only. Typical facilities designed in this manner include storm sewers, curb and gutter systems, culverts, bridges, and open channels. There are several methodologies available to the designer in the City of Bismarck. These include the rational method, NRCS Hydrology Manual method, the NRCS TR55 procedure, and the USGS runoff-frequency regression equations contained in WRI Report 92-4020. The selection of an appropriate peak discharge method is dependent upon the amount of data available, the size of the watershed, and the personal preferences of the designer. In most cases, it is recommended that peak discharge hydrology be determined by two or more methods and then compared. The final determination is made based upon the experience and professional judgment of the designer.

3.4.2 Rational Method

3.4.2.1 General

For drainage basins whose area is less than 160 acres, the design storm runoff may be analyzed using the Rational Method. This method was introduced in 1889 and is still being used in many engineering offices in the United States. Even though this method has frequently come under academic criticism for its simplicity, no other practical drainage design method has evolved to a level of general acceptance by the practicing engineer. The Rational Method, when properly understood and applied, can produce satisfactory results for urban storm sewer design.

The Rational Method is based on the formula:

$$Q = CIA \qquad \text{(Equation 3.1)}$$

Q is defined as the maximum rate of runoff in cubic feet per second (actually Q has units of acre inches per hour, which is approximately equal to the units of cubic feet per second). C is a runoff coefficient which is the ratio between the maximum rate of runoff from the area and the average rate of rainfall intensity (in inches per hour) for the period of maximum rainfall of a given frequency of occurrence having a duration equal to the time of concentration. I is the average intensity of rainfall in inches per hour for a duration equal to the time of concentration. This time of concentration usually is the time required for water to flow from the most remote point of the basin to the point being investigated. A is the contributing basin area in acres.

The basic assumptions made when applying the Rational Formula are:

1. The computed maximum rate of runoff to the design point is a function of the average rainfall rate during the time of concentration to that point.

2. The maximum rate of rainfall occurs during the time of concentration, and the design rainfall depth during the time of concentration is converted to the average rainfall

intensity for the time of concentration.

3. The maximum runoff rate occurs when the entire area is contributing flow. However, this assumption has been modified from time to time when local rainfall/runoff data was used to improve calculated results.

When using the Rational formula, an assumption is made that the maximum rate of flow is produced by a constant rainfall which is maintained for a time equal to the period of concentration of flow at the point under consideration. Theoretically, this is the time of concentration, which is the time required for the surface runoff from the most remote part of the drainage basin to reach the point being considered. However, in practice, the concentration time is an empirical value resulting in acceptable peak flow estimates.

3.4.2.2 Time of Concentration (Tc)

In applying the Rational Method, the time of concentration must be estimated so that the average rainfall rate of a corresponding duration can be determined from the time-intensity-frequency curves.

Travel time (T_t) is the time it takes water to travel from one location to another in a watershed. T_t is a component of time of concentration (T_c), which is time for runoff to travel from the hydraulically most distant point of the watershed to a point of interest within the watershed. T_c is computed by summing all the travel times for consecutive components of the drainage conveyance system.

T_c influences the shape and peak of the runoff hydrograph. Urbanization usually decreases T_c, thereby increasing the peak discharge. But T_c can be increased as a result of (a) ponding or storage behind small or undersized drainage systems, including storm drain inlets and road culverts, or (b) reduction of land slope through grading, or (c) maintaining vegetated conveyance systems within the watershed.

1. Factors affecting time of concentration and travel time:

    A. Surface Roughness. One of the most significant effects of urban development on flow velocity is less retardance to flow. That is, undeveloped areas with very slow and shallow overland flow through vegetation become modified or are removed by urban development: the flow is then delivered to streets, gutters, and storm sewers that transport runoff downstream more rapidly. Travel time through the watershed is generally decreased.

    B. Channel Shape and Flow Patterns. In small non-urban watersheds, much of the travel time results from overland flow in upstream areas. Typically, urbanization reduces overland flow lengths by conveying storm runoff into a channel as soon as possible. Since channel designs have efficient hydraulic characteristics, runoff flow velocity increases and travel time decreases.

C. Slope. Slopes may be increased or decreased by urbanization, depending on the extent of site grading or the extent to which storm sewers and street ditches are used in the design of the water management system. Slope will tend to increase when channels are straightened and decrease when overland flow is directed through storm sewers, street gutters, and diversions.

2. Computation of Time of Concentration:

Water moves through a watershed as sheet flow, shallow concentrated flow, open channel flow, or some combination of these. The type that occurs is a function of the conveyance system and is best determined by field inspection. Travel time (T_t) is the ratio of flow length to flow velocity:

$$T_t = \frac{L}{60\ V} \qquad \text{(Equation 3.2)}$$

where:

T_t	=	travel time (min.)
L	=	flow length (ft.)
V	=	average velocity (ft./s)
60	=	conversion factor from seconds to minutes

Time of concentration (T_c) is the sum of T_t values for the various consecutive flow segments:

$$T_c = T_{t\,1} + T_{t\,2} + \ldots T_{t\,m} \qquad \text{(Equation 3.3)}$$

where:

T_c	=	time of concentration (min.) and
m	=	number of flow segments.

A. Sheet Flow. Sheet flow is flow over plane surfaces (parking lots, farm fields, lawns). It usually occurs in the headwater of streams. With sheet flow, the friction value (Manning's n) is an effective roughness coefficient that includes the effect of rain drop impact; drag over the plane surface; obstacles such as litter, crop ridges, and rocks; and erosion and transportation of sediment. The n values for very shallow flow depths of about 0.1 foot or less for various surface conditions are provided in Table No. 3.2. For sheet flow of less than 300 feet, use Manning's kinematic solution (Overton and Meadows 1976) to compute T_t:

$$T_t = \frac{0.00398[(n)\ (L)]^{0.8}}{S^{0.4}} \qquad \text{(Equation 3.4)}$$

where:	T_t	=	travel time (hr.)
	n	=	Manning's roughness coefficient (Table No. 3.2)
	L	=	flow length (ft.)
	S	=	slope of hydraulic grade line (land slope, ft. /ft.).

This simplified form of the Manning's kinematic solution is based on the following:

1) shallow steady uniform flow,
2) constant intensity of rainfall excess (rain available as runoff),
3) rainfall duration of 24 hours, and
4) minor effect of infiltration on travel time.

B. Shallow Concentrated Flow.

1) Urban/Suburban Areas. After a maximum of 300 feet, sheet flow (gutter, swales) usually becomes shallow concentrated flow. The average velocity (V) for this flow can be determined from Figure No. 3.2, in which average velocity is a function of watercourse slope and type of channel. For slopes less than 0.005 ft. /ft., use:

Unpaved $V = 16.1345(s)^{.5}$ (Equation 3.5)

Paved $V = 20.3282(s)^{.5}$ (Equation 3.6)

Tillage can affect the direction and velocity of shallow concentrated flow. Flow may not always be directly down the watershed slope if tillage runs across the slope.

After determining average velocity (V) in Figure No. 3.2, use Equation 3.2 to estimate travel time for the shallow concentration flow segment.

2) Rural/Suburban Areas. In drainage basins where a large segment of the area is rural in character and has long hydraulic length, the potential for retention of rainfall on the watershed increases along with travel time. Under these conditions the NRCS watershed lag equation is used to estimate travel time since it accounts for more of the factors influencing the total time of concentration.

$t_c = L/0.6$ (Equation 3.7)

$L = l^{0.8}(S+1)^{0.7}/1900\ Y^{0.5}$ (Equation 3.8)

where:	t_c	=	time of concentration, hours
	L	=	basin lag, hours
	l	=	hydraulic length of watershed, feet
	S	=	$(1000/CN)^{-10}$
	CN	=	SCS curve number (Table Nos. 3.3, 3.4 or 3.5)
	Y	=	average watershed land slope, percent

TABLE NO. 3.2

ROUGHNESS COEFFICIENCTS FOR SHEET FLOW

Surface description	n[1]
Smooth surfaces (concrete asphalt, gravel or bare soil)	0.011
Fallow (no residue)	0.05
Cultivated soils:	
Residue cover ≤ 20%	0.06
Residue cover > 20%	0.17
Grass:	
Short grass prairie	0.15
Dense grasses[2]	0.24
Bermudagrass	0.41
Range (natural)	0.13
Woods:[3]	
Light underbrush	0.40
Dense underbrush	0.80

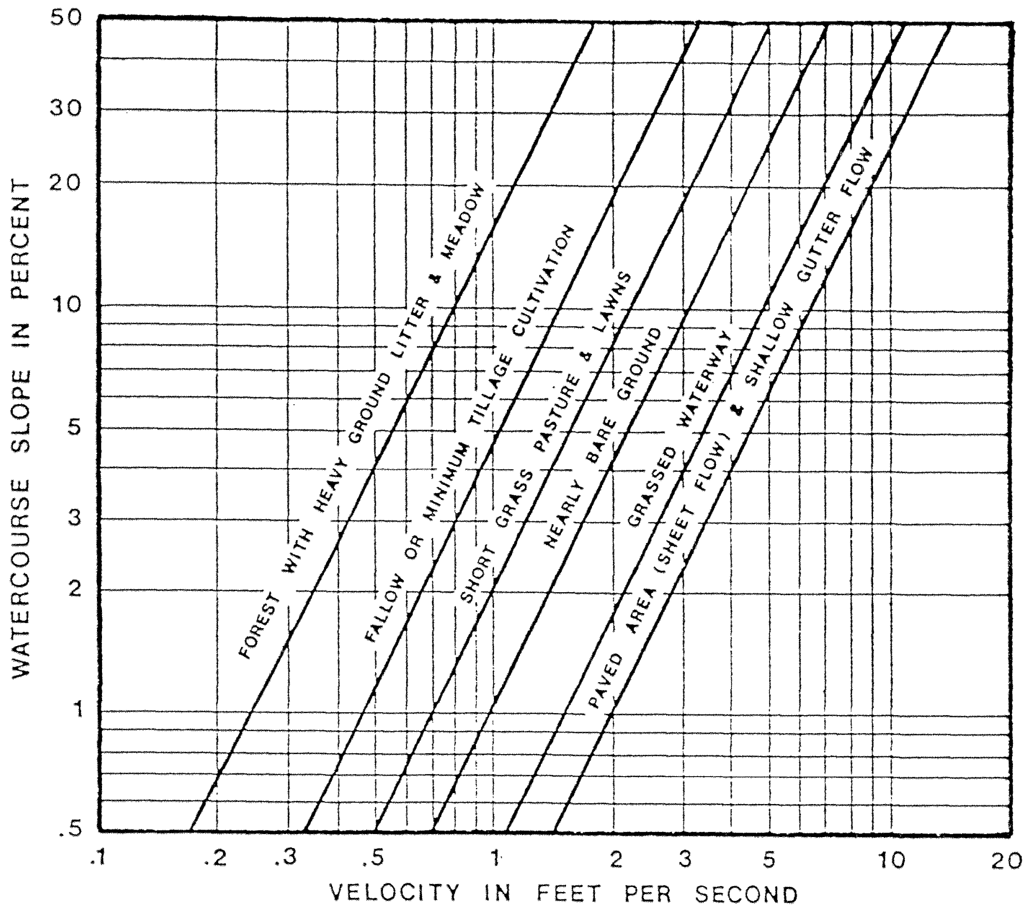
[1]The n values are a composite of fraction compiled for Engman (1986).

[2]Includes species such as weeping lovegrass, bluegrass, buffalo grass, blue grama grass, and native mixtures.

[3]When selecting n, consider cover to a height of about 0.1 ft. This is the only part of the plant cover that will obstruct sheet flow.

Figure No. 3.2

TRAVEL TIME VELOCITY FOR RATIONAL METHOD



The lag equation was developed for essentially rural areas and thus over estimates lag and t_c in urban areas for two reasons. First, the increased amount of impervious area permits water from overland flow sources and side channels to reach the main channel at a much faster rate than under natural conditions. The second is the extent to which a stream (usually the major watercourse in the watershed) has been changed over natural conditions to allow higher flow velocities. Figure No. 3.3 and Figure No. 3.4 may be used to adjust the lag (computed by Equation 3.8 and used in Equation 3.7) for urbanized conditions.

C. Open Channels. Open channels (ditches, storm sewers and tiles not flowing full) are assumed to begin where surveyed cross section information has been obtained, where channels are visible on aerial photographs, or where blue lines (indicating streams) appear on United States Geological Survey (USGS) quadrangle sheets. Manning's equation or water surface profile information can then be used in such cases to estimate average flow velocity. Average flow velocity is usually determined for bank-full elevation.

Manning's equation is:

$$V = \frac{1.49}{n} r^{2/3} S^{1/2} \qquad \text{(Equation 3.9)}$$

where:

V	=	average velocity (ft./s)
r	=	hydraulic radius (ft.) and is equal to a/P_w
a	=	cross sectional flow area (sq. ft.)
P_w	=	wetted perimeter (ft.)
S	=	slope of the hydraulic grade line (channel slope, ft./ft.)
n	=	Manning's roughness coefficient for open channel flow (see Table No. 3.6).

Manning's n values for open channel flow can be obtained from standard textbooks such as Chow (1959) or Linsley et al. (1982). After average velocity is computed using Equation 3.9, the channel segment T_t can be estimated using Equation 3.2.

D. Closed Channels (storm sewers and tiles flowing full). Storm sewers generally handle only a small portion of the large event. The estimated V for a storm sewer is computed using Manning's equation:

$$V = \frac{1.49}{n} r^{2/3} S^{1/2} \qquad \text{(Equation 3.10)}$$

where:

V	=	average velocity (ft./s)
D	=	diameter of pipe
S	=	slope of the pipe grade line (ft./ft.)
n	=	Manning's roughness coefficient for open channel flow (see

Runoff Curve Numbers for Urban Areas
TABLE 3.3

Cover description		Curve Numbers for hydrologic soil group			
Cover type and hydrologic condition	Average percent impervious area[2]	A	B	C	D
Fully developed urban areas (vegetation established)					
Open space (lawns, parks, golf courses, cemeteries, etc.)[3]					
Poor condition (grass cover< 50%)		68	79	86	89
Fair condition (grass cover < 50%)		49	69	79	84
Good condition (grass cover < 50%)		39	61	74	80
Impervious area:					
Paved parking lots, roofs, driveways, etc. (excluding right-of-way)		98	98	98	98
Streets and roads:					
Paved; curbs and storm sewers (excluding right- of-way)		98	98	98	98
Paved; open ditches (including right-of- way)		83	89	92	93
Gravel (including right-of-way)		76	85	89	91
Dirt (including right-of-way)		72	82	87	89
Urban districts:					
Commercial and business	85	89	92	94	95
Industrial	72	81	88	91	93
Residential districts by average lot size:					
1/8 acre or less (town houses)	65	77	85	90	92
1/4 acre	38	61	75	83	87
1/3 acre	30	57	72	81	86
1/2 acre	25	54	70	80	85
1 acre	20	51	68	79	84
2 acre	12	46	65	77	82
Developing urban areas					
Newly graded area (pervious areas only, no vegetation).		77	86	91	94
Idle lands (CN's are determined using cover types similar to those in table 2-2c)					

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

[2]The average percent impervious area shown was used to develop the composite CN’s. Other assumptions are as follows: impervious areas are directly connected to the drainage system, impervious areas have a CN of 98, and pervious areas are considered equiv

[3]CN’s shown are equivalent to those of pasture. Composite CN’s may be computed for other combinations of open space cover type.

[4]Composite CN’s for natural desert landscaping should be computed using figures 2-3 or 2-4 based on the impervious area percentage (CN = 98) and the pervious area CN. The pervious area CN’s are assumed equivalent to desert shrub in poor hydrologic condit

[5]Composite CN’s to use for the design of temporary measures during grading and construction should be computed using figure 2-3 or 2-4, based on the degree of development (impervious area percentage) and the CN’s for the newly graded pervious areas.

Runoff Curve Numbers for Cultivated Agricultural Land
TABLE NO. 3.4

Cover description			Curve Numbers for hydrologic soil group			
Cover type	Treatment[2]	Hydrologic condition[3]	A	B	C	D
Fallow	Bare soil	-	77	86	91	94
	Crop residue cover (CR)	Poor	76	85	90	93
		Good	74	83	88	90
Row crops	Straight row (SR)	Poor	72	81	88	91
		Good	67	78	85	89
	SR + CR	Poor	71	80	87	90
		Good	64	75	82	85
	Contoured (C)	Poor	70	79	84	88
		Good	65	75	82	86
	C + CR	Poor	69	78	83	87
		Good	64	74	81	85
	Contoured & terraced (C&T)	Poor	66	74	80	82
		Good	62	71	78	81
	C&T + CR	Poor	65	73	79	81
		Good	61	70	77	80
Small grain	SR	Poor	65	76	84	88
		Good	63	75	83	87
	SR + CR	Poor	64	75	83	86
		Good	60	72	80	84
	C	Poor	63	74	82	85
		Good	61	73	81	84
	C + CR	Poor	62	73	81	84
		Good	60	72	80	83
	C&T	Poor	61	72	79	82
		Good	59	70	78	81
	C&T + CR	Poor	60	71	78	81
		Good	58	69	77	80
Close-seeded or broadcast legumes or rotation meadow	SR	Poor	66	77	85	89
		Good	58	72	81	85
	C	Poor	64	75	83	85
		Good	55	69	78	83
	C&T	Poor	63	73	80	83
		Good	51	67	76	80

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

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

[3]Hydrologic condition is based on combination of factors that affect infiltration and runoff, including (a) density and canopy of vegetative area, (b) amount of year-round cover, (c) amount of grass or close-seeded legumes in rotations, (d) percent of res

Poor: Factors impair infiltration and tend to increase runoff.

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

Runoff Curve Numbers for other Agricultural Land
TABLE NO. 3.5

Cover description		Curve Numbers for hydrologic soil group			
Cover type	Hydrologic condition	A	B	C	D
Pasture, grassland, or range - continuous forage for grazing.	Poor	68	79	86	89
	Fair	49	69	79	84
	Good	39	61	74	80
Meadow - continuous grass, protected form grazing and generally mowed for hay.	-	30	58	71	78
Brush - brush-weed-grass mixture with brush the major element.[3]	Poor	48	67	77	83
	Fair	35	56	70	77
	Good	[4] 30	48	65	73
Woods - grass combination (orchard or tree farm)[5]	Poor	57	73	82	86
	Fair	43	65	76	82
	Good	32	58	72	79
Woods.[6]	Poor	45	66	77	83
	Fair	36	60	73	79
	Good	[4] 30	55	70	77
Farmsteads - buildings, lanes, driveways, and surrounding lots.	-	59	74	82	86

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

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

[3]*Poor* : < 50% ground cover.
Fair : 50 to 75% ground cover.
Good : > 75% ground cover.

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

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

[6] *Poor* : Forest litter, small trees, and brush are destroyed by heavy grazing or regular burning.
Fair : Woods are grazed but no burned, and some forest litter covers the soil.
Good : Woods are protected from grazing, and litter and brush adequately cover the soil.

FACTORS FOR ADJUSTING LAG FROM EQUATION 3.7 WHEN IMPERVIOUS AREAS OCCUR IN THE WATERSHED

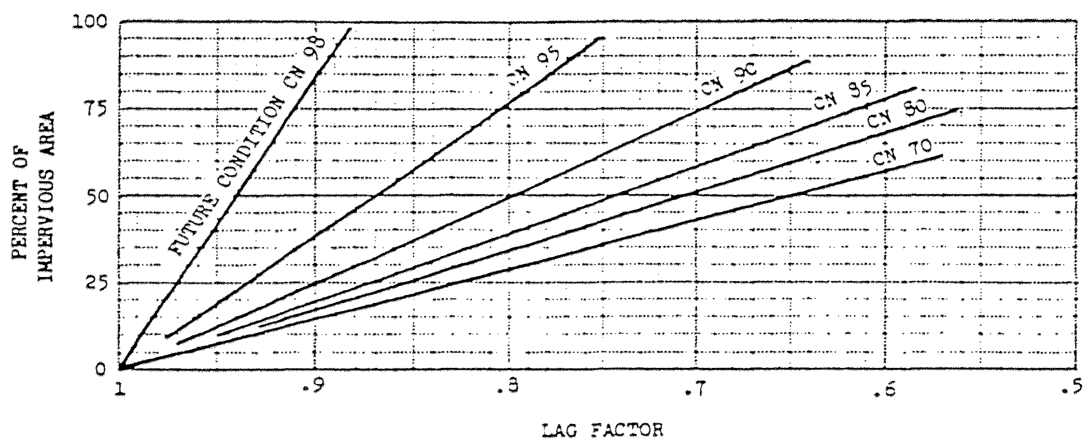


Figure No. 3.3

FACTORS FOR ADJUSTING LAG FROM EQUATION 3.7 WHEN THE MAIN CHANNEL HAS BEEN HYDRAULICALLY IMPROVED

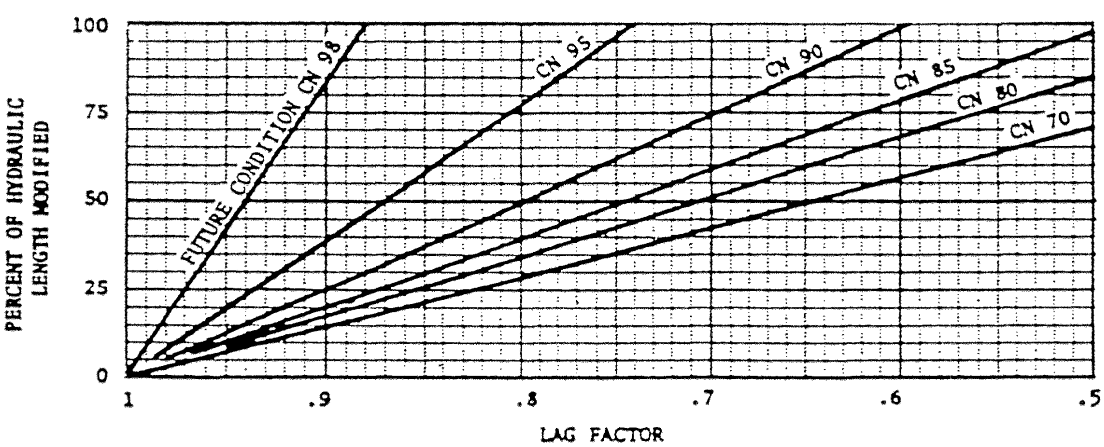


Figure No. 3.4

Manning's Roughness Coefficient for Open Channel Flow
TABLE NO. 3.6 (a)

I. Closed Conduits	Manning's n Range[2]
A. Concrete pipes----------	0.011n0.013
B. Corrugated-metal pipe or pipe-arch.	
1. 2 2/3 by 1/2-in. corrugation (riveted pipe):[3]	
a. Plain or fully coated----------	0.024
b. Paved invert (range values are for 25 and 50 percent of circumference paved):	
(1) Flow full depth----------	0.021n0.018
(2) Flow 0.8 depth----------	0.021n0.016
(3) Flow 0.6 depth----------	0.019n0.013
2. 6 by 2-in. corrugation (field bolted)----------	0.03
C.Vitrified clay pipe----------	0.012n0.014
D. Cast-iron pipe, uncoated----------	0.013
E. Steel pipe----------	0.009n0.011
F. Brick----------	0.014n0.017
G. Monolithic concrete:	
1. Wood forms, rough----------	0.015n0.017
2. Wood forms, smooth----------	0.012n0.014
3. Steel forms----------	0.012n0.013
H. Cemented rubble masonry walls:	
1. Concrete floor and top----------	0.017n0.022
2. Natural floor----------	0.019n0.025
I. Laminated treated wood----------	0.015n0.017
J.Vitrified clay liner plates----------	0.015
II. Open Channels, Lined[4] (straight lignment):[5]	
A. Concrete with surfaces as indicated:	
1. Formed, no finish----------	0.013n0.017
2. Trowel finish----------	0.012n0.014
3. Float finish----------	0.013n0.015
4. Float finish, some gravel on bottom----------	0.015n0.017
5. Gunite, good section----------	0.016n0.019
6. Gunite, wavy section----------	0.018n0.022
B. Concrete, bottom float finished, sides as indicated:	
1. Dressed stone in mortar----------	0.015n0.017
2. Random stone in mortar----------	0.017n0.020
3. Cement rubble masonry----------	0.020n0.025
4. Cement rubble masonry, plastered----------	0.016n0.020
5. Dry rubble (riprap)----------	0.020n0.030
C. Gravel bottom, sides as indicated:	

IV. Highway Channels and Swales with Maintained Vegetation[6] (values shown here are for velocities of 2 and 6 f.p.s.):	Manning's n Range[2]
A. Depth of Flow up to 0.7 foot:	
1. Bermudagrass, Kentucky bluegrass, Buffalograss:	
a. Mowed to 2-inches----------	0.07n0.045
b. Length 4n6 inches----------	0.09n0.05
2. Good stand, any grass:	
a. Length about 12-inches----------	0.18n0.09
b. Length about 24-inches----------	0.30n0.15
3. Fair stand, any grass:	
a. Length about 12-inches----------	0.14n0.08
b. Length about 24-inches----------	0.25n0.13
B. Depth of flow 0.7-1.5 feet:	
1. Bermudagrass, Kentucky bluegrass, Buffalograss:	
a. Mowed to 2-inches----------	0.05n0.035
b. Length 4n6 inches----------	0.06n0.04
2. Good stand, any grass:	
a. Length about 12-inches----------	0.12n0.07
b. Length about 24-inches----------	0.20n0.10
3. Fair stand, any grass:	
a. Length about 12-inches----------	0.10n0.06
b. Length about 24-inches----------	0.17n0.09
V. Street and Expressway Gutters:	
A. Concrete gutter, troweled finish----------	0.012
B. Asphalt pavement:	
1. Smooth texture----------	0.013
2. Rough texture----------	0.016
C. Concrete Gutter with asphalt pavement:	
1. Smooth----------	0.013
2. Rough----------	0.015
D. Concrete pavement:	
1. Float finish----------	0.014
2. Broom finish----------	0.016
E. For gutters with small slope, where sediment may accumulate, increase above values of n by----------	*0.002*
VI. Natural stream channels:[8]	
A. Minor streams[9] (surface width at flood stage less than 100 ft):	
1. Fairly regular section:	

TABLE NO. 3.6 (b)

1. Formed concrete	0.017n0.020
2. Random stone in mortar	0.020n0.023
3. Dry rubble (riprap)	0.023n0.033
D. Brick	0.014n0.017
E. Asphalt:	
1. Smooth	0.013
2. Rough	0.016
F. Wood, planed, clean	0.011n0.013
G. Concrete-lined excavated rock:	
1. Good section	0.017n0.020
2. Irregular section	0.022n0.027
III. Open Channels, excavated[4] (straight alignment,[5] natural lining):	
A. Earth, uniform section:	
1. Clean, recently completed	0.016n0.018
2. Clean, after weathering	0.018n0.020
3. With short grass, few weeds	0.022n0.027
4. In gravelly soil, uniform section, clean	0.022n0.025
B. Earth, fairly uniform section:	
1. No vegetation	0.022n0.025
2. Grass, some weeds	0.025n0.030
3. Dense weeds or aquatic plants in deep channels	0.030n0.035
4. Sides clean, gravel bottom	0.025n0.030
5. Sides clean, cobble bottom	0.030n0.040
C. Dragline excavated or dredged:	
1. No vegetation	0.028n0.033
2. Light brush on banks	0.035n0.050
D. Rock:	
1. Based on design section	0.035
2. Based on actual mean section:	
a. Smooth and uniform	0.035n0.050
b. Jagged and irregular	0.040n0.045
E. Channels not maintained, weeds and brush uncut:	
1. Dense weeds, high as flow depth	0.08n0.12
2. Clean bottom, brush on sides	0.05n0.08
3. Clean bottom, brush on sides, highest stage of flow	0.07n0.11
4. Dense brush, high stage	0.10n0.14

a. Some grass and weeds, little or no brush	0.030n0.035
b. Dense growth of weeds, depth of flow materially greater than weed height	0.035n0.05
c. Some weeds, light brush on banks	0.035n0.05
d. Some weeds, heavy brush on banks	0.05n0.07
e. Some weeds, dense willows on banks	0.06n0.08
f. For trees within channel, with branches submerged at high stage, increase all above values by	*0.01n0.02*
2. Irregular sections, with pools, slight channel meander; increase values given in 1ane about	*0.01n0.02*
3. Mountain streams, no vegetation in channel, banks usually steep, trees and brush along banks submerged at high stage:	
a. Bottom of gravel. cobbles, and few boulders	0.04n0.05
b. Bottom of cobbles, with large boulders	0.05n0.07
B. Flood plains (adjacent to natural streams)	
1. Pasture, no brush:	
a. Short grass	0.030n0.035
b. High grass	0.035n0.05
2. Cultivated areas:	
a. No crop	0.03n0.04
b. Mature row crops	0.035n0.045
c. Mature field crops	0.04n0.05
3. Heavy weeds, scattered brush	0.05n0.07
4. Light brush and trees:[10]	
a. Winter	0.05n0.06
b. Summer	0.06n0.08
5. Medium to dense brush:[10]	
a. Winter	0.07n0.11
b. Summer	0.10n0.16
6. Dense willows, summer, not bent over by current	0.15n0.20
7. Cleared land with tree stumps, 100n150 per acre:	
a. No sprouts	0.04n0.05
b. With heavy growth of sprouts	0.06n0.08
8. Heavy stand of timber, a few down trees, little undergrowth:	
a. Flood depth below branches	0.10n0.12
b. Flood depth reaches branches	0.12n0.16
C. Major streams (surface width at flood stage more that 100 ft.): Roughness coefficient 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 may be somewhat reduced. Follow recommendation in publication cited[8] if possible. The value of n for larger streams of most regular section, with no boulders or brush, may be in the range of	0.028n0.033

E. Reservoirs or Lakes. Sometimes it is necessary to estimate the velocity of flow through a reservoir or lake at the outlet of a watershed. Theoretically, this travel time is very small and is often assumed to be zero.

3.4.2.3 Rainfall Intensity (I)

The intensity (I), is the average rainfall rate in inches per hour for the period of maximum rainfall of a given frequency having a duration equal to the time of concentration. After the design storm frequency has been selected, the rainfall intensity should be obtained from Figure No. 3.1 using the time of concentration calculated above. For the Rational Method, the design intensity should be that which occurs for the design year storm whose duration equals the time of concentration.

3.4.2.4 Runoff Coefficient (C)

The runoff coefficient (C) represents the integrated effects of infiltrations, evaporation, retention, flow routing, and interception, all which affect the time distribution and peak rate of runoff. Table No. 3.7 presents the recommended values of C for the various recurrence frequency storms. The values are presented for different surface characteristics as well as for different aggregate land uses. The coefficient for various surface areas can be used to develop a composite value for a different land use.

The design engineer should realize that the "C" values shown on Table 3.7 are typical values and may have to be adjusted if the site deviates from typical conditions such as an increase or decrease in percent of impervious surfaces.

The hydrologic soil groups, as defined by NRCS soil scientists and used on Table No. 3.7 are:

1. Hydrologic Soil Group 'A' - Low Runoff Potential. Soils having a high infiltration rate even when thoroughly wetted consisting chiefly of deep, well to excessively drained sands or gravels.

2. Hydrologic Soil Group 'B' - Average Runoff Potential. Soils having a moderate infiltration rate when thoroughly wetted and consisting chiefly of moderately deep to deep, moderately well to well drained soils with moderately fine to moderately coarse texture.

3. Hydrologic Soil Group 'C' - Moderate Runoff Potential. Soils having a slow infiltration rate when thoroughly wetted and consisting chiefly of soils with a layer that impedes downward movement of water or soils with moderately fine to fine texture.

4. Hydrologic soil group 'D' - High Runoff Potential. Soils having a very slow infiltration rate when thoroughly wetted and consisting chiefly of clay soils with a high swelling potential, soils with a permanent high water table, soils with a clay pan or clay layer at or near the surface, and shallow soils over nearly impervious material. Soil types for the Bismarck-Burleigh County area can be obtained using the NRCS's Burleigh County Soil

Survey Report and the North Dakota Hydrology Manual.

3.4.2.5 Area (A)

A is the basin drainage area in acres. A map showing the limits of the drainage basin used in design should be provided with design data superimposed on the grading plan showing all subbasins.

Hydrologic Soil Group		A			B			C			D		
Recurrence Interval		5	10	100	5	10	100	5	10	100	5	10	100
Land Use or Surface Characteristics	Percent Impervious												
Business:													
A. Commercial Area	95	.75	.80	.95	.80	.85	.95	.80	.85	.95	.85	.90	.95
B. Neighborhood Area	70	.50	.55	.65	.55	.60	.70	.60	.65	.75	.65	.70	.80
Residential:													
A. Single Family		.25	.25	.30	.30	.35	.40	.40	.45	.50	.45	.50	.55
B. Multi-Unit (Detached)													
C. Multi-Unit (Attached)	50	.35	.40	.45	.40	.45	.50	.45	.50	.55	.50	.55	.65
D. 1/2 A. Lot or Larger	70	.45	.50	.55	.50	.55	.65	.55	.60	.70	.60	.65	.75
E. Apartments		.20	.20	.25	.25	.25	.30	.35	.40	.45	.40	.45	.50
	70	.50	.55	.60	.55	.60	.70	.60	.65	.75	.65	.70	.80
Industrial													
A. Light Areas	80	.55	.60	.70	.60	.65	.75	.65	.70	.80	.70	.75	.90
B. Heavy Areas	90	.75	.80	.95	.80	.85	.95	.80	.85	.95	.80	.85	.95
Parks, Cemeteries, Playgrounds	7	.10	.10	.15	.20	.20	.25	.30	.35	.40	.35	.40	.45
Schools	50	.30	.35	.40	.40	.45	.50	.45	.50	.55	.50	.55	.65
Railroad Yard Areas	40	.20	.20	.25	.30	.35	.40	.40	.45	.45	.45	.50	.55
Undeveloped Areas off Site Flow Analysis	45	.50	.50	.50	.50	.50	.50	.50	.50	.50	.50	.50	.50
Streets													
A. Paved	100	.85	.90	.95	.85	.90	.95	.85	.90	.95	.85	.90	.95
B. Gravel	13	.25	.25	.30	.35	.40	.45	.40	.45	.50	.40	.45	.50
Drives, Walks and Roofs	96	.85	.90	.95	.85	.90	.95	.85	.90	.95	.85	.90	.95
Lawns													
A. 50%-75% Grass (Fair Condition)		.10	.10	.15	.20	.20	.25	.30	.35	.40	.30	.35	.40
B. 75% or More Grass (Good Condition)		.05	.05	.10	.15	.15	.20	.25	.25	.30	.30	.35	.40

RUNOFF COEFFICIENTS FOR THE RATIONAL METHOD
TABLE NO 3.7

3.4.2.6 Application of the Rational Method

The first step in applying the Rational Method is to obtain topographic information and define the boundaries of all the relevant drainage basins. Basins to be defined include all basins tributary to the study area and subbasins within the study area. A field check and possibly field surveys should be made for each basin. At this stage of planning, the possibility for the diversion of transbasin waters should be identified.

The major storm drainage basin does not always coincide with the minor storm drainage basin. This is often the case in urban areas where a low flow stays next to a curb and follows the lowest grade, but when a large flow occurs the water will be deep enough so that part of the water will overflow street crowns and flow into a new sub-basin.

When analyzing the major runoff occurring on an area that has a storm sewer system sized for the initial storm, care must be used when applying the Rational Method. Normal application of the Rational Method assumes that all runoff is collected by the storm sewer. For the initial storm design, the time of concentration is dependent upon the flow time in the sewer. However, during the major storm runoff, the sewers will probably be at capacity and could not carry the additional water flowing to the inlets. This additional water then flows overland past the inlets, generally at a lower velocity than the flow in the storm sewers.

If a separate time of concentration analysis is made for the pipe flow and surface flow, a time lag between the surface flow peak and the pipe flow peak will occur. This lag, in effect, will allow the pipe to carry a larger portion of the major storm runoff then would be predicted using the initial storm time of concentration. The basis for this increased benefit is that the excess water from one inlet will flow to the next inlet downhill, using the overland route. If that inlet is also at capacity, the water will often continue on until capacity is available in the storm sewer. The analysis of this aspect of the interaction between the storm sewer system and the major storm runoff is complex. The simplified approach of using the initial storm time of concentration for frequency analysis is generally acceptable for the City of Bismarck.

3.4.3 NRCS Hydrology Manual for North Dakota

The NRCS, formally known as the Soil and Conservation Service, published a hydrology manual for North Dakota in 1981. The methodology in the hydrology manual follows standard NRCS curve number procedures for converting rainfall to runoff. Chapter 5 of the manual contains a procedure for estimating peak discharges. A discharge used in the NRCS design work, is the highest rate of runoff, expressed in cubic feet per second, that may be expected from a particular watershed for a specific recurrence interval. The four methods of peak discharge determination are described in the hydrology manual. Each method for obtaining a peak discharge focuses on the peak of the hydrograph without plotting the hydrograph. Methods presented in Chapter 5 of the Hydrology Manual include:

1. Time of Concentration Method - derived from Section 4 of the NRCS National Engineering Handbook.

2. Peak Rate of Discharge for Small Watersheds - derived from NRCS-TR-149.

3. Annual Peak Curve - derived from stream flow data collected from US Geological Survey.

4. Drainage Curves - derived from USDA Drainage Handbooks.

Chapter 5 of the Hydrology Manual for North Dakota also contains a very good discussion of the effects of urbanization of runoff peak rates of discharge. The emphasis in this discussion is placed on utilized of NRCS techniques in assessing the effects of urbanization.

No further discussion will be conducted in this Manual relative to the above-mentioned NRCS methodologies. The design engineer is referred to as the publication "Hydrology Manual for North Dakota" published by the NRCS.

3.4.4 USGS WRI Report 92-4020

This report provides techniques for estimating peak-flow frequency relations for North Dakota streams. It is based upon a multiple linear analysis of gauging station data for gauged streams throughout the state of North Dakota. A separate set of regression equations has been prepared for three separate regions of the state. The City of Bismarck is located in Region B, the Missouri River region. Regression equations are published for the estimation for peak discharges for the two, 10, 15, 25, 50, 100 and 500-year peak discharges. The design engineer is referred to publication "Techniques for Estimating Peak-Flow Frequency Relations for North Dakota Streams" published in 1992 by the US Geological Survey as Water Resources Investigation Report 92-4020. Caution is advised with the use of this methodology as it does not account adequately for impacts from urbanized areas.

3.4.5 NRCS TR-55

The National Resource Conservation Service developed TR-55 "Urban Hydrology for Small Watersheds". It was created to address the problem of evaluating runoff from changing areas (i.e., the conversion of rural farm land to residential and industrial uses). As a result, it is capable of evaluating both "pre-development" and "post-development" conditions.

The peak discharges that are computed are approximations of the results received from TR-20, also developed by the NRCS. TR-55 is limited to watersheds of 2,000 acres or smaller. A personal computer version of TR-55 is available. TR-55 has six chapters:

1. Introduction. This chapter discusses the choosing of a computational method.

2. Estimating Runoff. This chapter discusses the utilization of curve numbers for converting rainfall to runoff. It also introduces the concept of connected and unconnected pervious areas within a watershed.

3. Time of Concentration and Travel Time. TR-55 presents new tables and charts for the determination of T_c and T_t.

4. Graphical Peak Discharge Method. This method as presented, is limited only to homogenous watersheds.

5. Tabular Hydrograph Method. This chapter presents the best approximation of TR-20 for small watersheds.

6. Storage Volume for Detention Basins. This chapter describes a procedure on how to determine storage volume needed for specific outflow control structures. This procedure is very useful for sediment basins, detention reservoirs, and small storage facilities. However, the design engineer is cautioned to become acquainted with the limitations of this procedure.

No further comment or discussion will be presented in this Manual concerning TR-55. The design engineer is referred to the TR-55 Manual and documentation supporting the computer software.

3.5 HYDROGRAPH HYDROLOGY

3.5.1 Selection of Methodology

Hydrograph analysis is the most widely used method of analyzing surface runoff. A hydrograph provides the rate of flow at all points in time during and after a storm or snowmelt event. Because a hydrograph plots volumetric flow rates against time, integration of the area beneath a hydrograph between any two points in time gives the total volume of water passing the point of interest during the time interval. Thus, in addition to peak flows, hydrographs allow analysis of sizes of reservoirs, storage tanks, detention ponds, and other facilities that deal with volumes of runoff. This is in contrast to peak discharge hydrology which is only concerned with a determination of a peak rate of flow and the sizing of facilities to pass this peak rate of flow. The peak rate of flow is only one component of a full discharge hydrograph.

Hydrograph analysis techniques involve transforming the rainfall excess that occurs spatially over a basin from a specific storm event into a corresponding hydrograph at the basin outlet. A hydrograph is the time distribution of runoff at the outlet of the basin. The volume under the hydrograph is equal to the amount of rainfall excess that occurs over the basin. The technique used by most hydrologists and hydraulic engineers for transforming rainfall excess to direct surface runoff is the unit hydrograph technique. Direct runoff is that water which travels over the land surface or laterally in the surface soil to a stream channel. The stream channel allows this flow to travel rapidly in comparison to groundwater flow. Therefore, the direct runoff is ordinarily the most important element in the formation of flood peaks. Subsurface flow or "base flow" into a stream, results when the groundwater table is higher than the stream surface. The groundwater distribution to stream flow does not ordinarily fluctuate rapidly due to the medium through which groundwater travels. Up to several weeks, or a few months, and even years in some cases, may be required for a given accretion of groundwater to be discharged into the streams. Figure No. 3.5 illustrates the basic procedure of developing a runoff hydrograph.

The hydrograph is usually considered as having two parts, direct runoff and base flow. The components of a hydrograph are illustrated in Figure No. 3.6. Hydrologic studies usually require an analysis to determine one or more components of a hydrograph. Hydrograph characteristics such as peak discharge, time to peak, and runoff volume are based on the shape of a hydrograph, which in turn is dependent upon precipitation patterns and basin characteristics of the watershed.

The procedure to determine a runoff hydrograph requires:

- determination of rainfall over a basin
- determination of rainfall excess (loss rate analysis)
- determination of the response at the outlet of a basin (unit hydrograph)
- determination of base flow

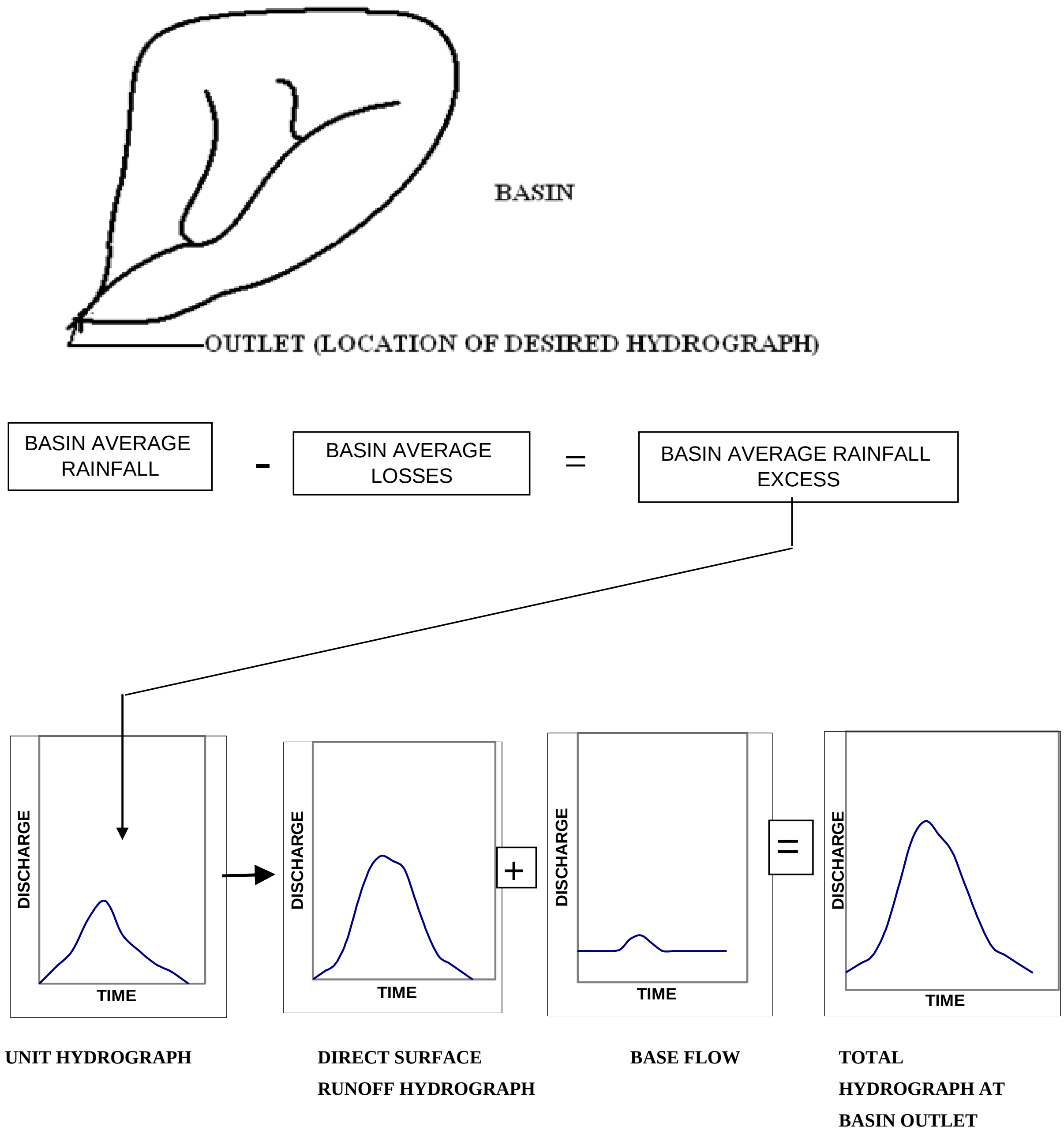
Hydrograph analysis of complex watershed systems requires subdividing the watershed into subbasins, computing the runoff hydrograph for each subbasin and then combining and translating, routing, the hydrographs throughout the watershed. The reasons for subdividing a watershed are:

- The watershed is large and the precipitation does not occur uniformly over the basin, and/or basin characteristics change significantly in the system
- It is desired to place less emphasis on the linear concepts of the unit hydrograph and more on routing throughout the watershed
- The flood hydrographs are to be determined at locations of interest throughout the watershed

Figure No. 3.7 illustrates the basic hydrograph analysis procedure for a complex watershed involving more than two subbasins. A detailed discussion of hydrograph development is beyond the scope of this Manual. The design engineer is referred to several hydrology texts and manuals which describe the theory and development of runoff hydrographs.

As with peak discharge hydrology, there are many methodologies available to the design engineer for the development of runoff hydrographs for a project involving storage. Practically all of these methods involve a unit hydrograph or dimensionless hydrograph approach. These methods include the NRCS Hydrology Manual Method, the NRCS TR-55 Procedure, the NRCS TR-20 Procedure, the U.S. Army Corps of Engineers HEC-1 Procedure, and many other methodologies. The selection of an appropriate hydrograph development method is dependent upon the amount of data available, the size of the watershed and the personal preferences of the design engineer. Practically all hydrograph development procedures are fully automated and available through many commercial software vendors. Final determination of a hydrograph development methodology is made based upon the experience and professional judgment of the design engineer and the availability of various software packages.

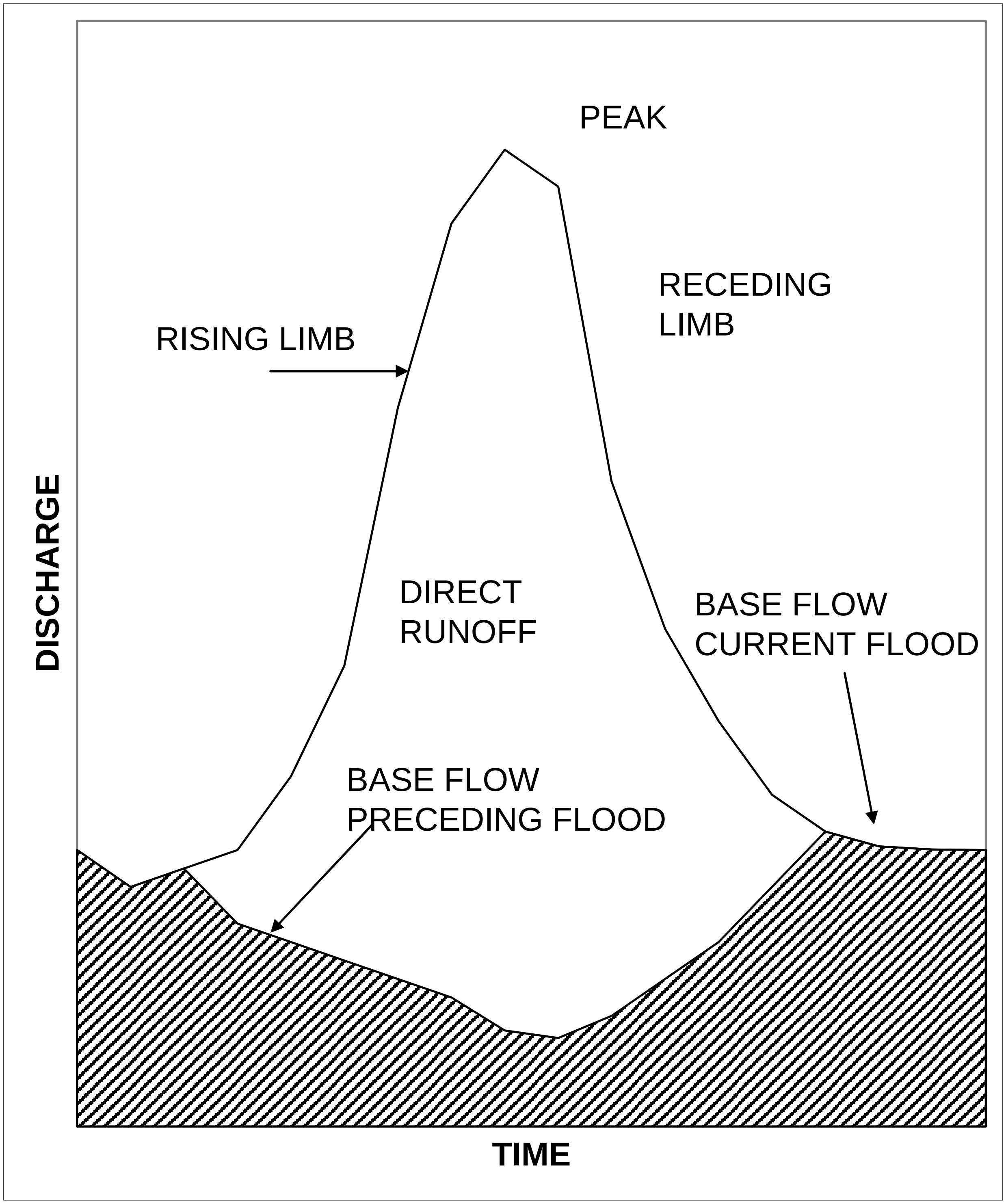
DEVELOPMENT OF HYDROGRAPH AT LOCATION OF INTEREST
FOR A SINGLE STORM EVENT



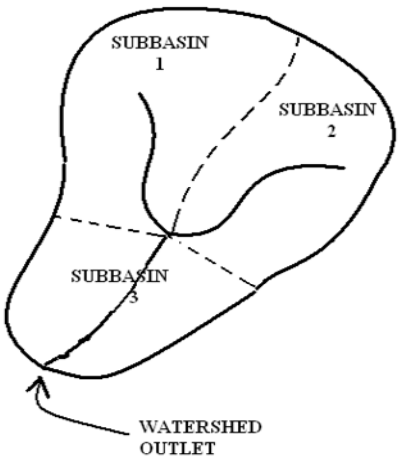
Hydrograph Analysis Concepts

Figure No. 3.5

Runoff Hydrograph Components
Figure No. 3.6

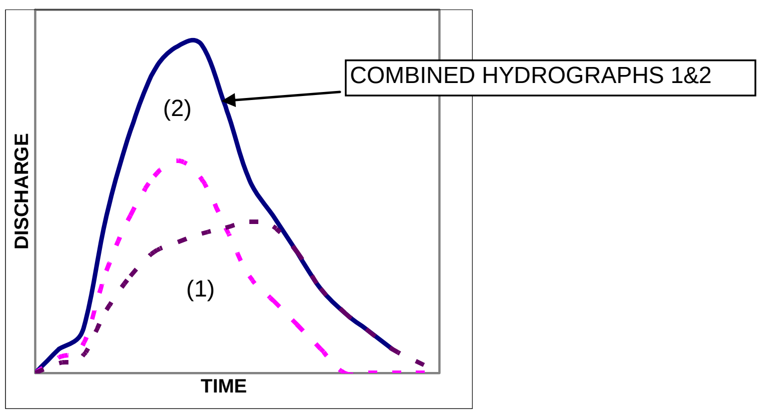


Basic Hydrograph Analysis Procedure
Figure No. 3.7

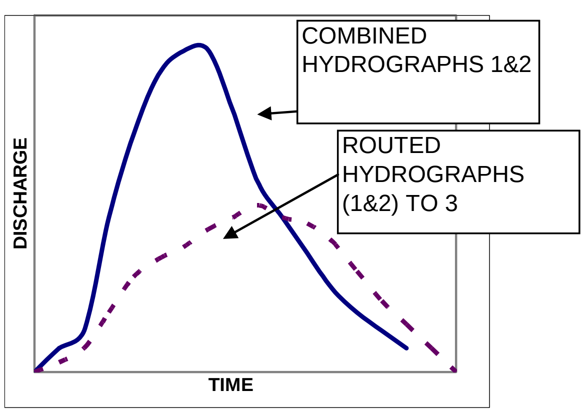


<u>BASIN HYDROGRAPH</u>
<u>ANALYSIS PROCEDURE</u>

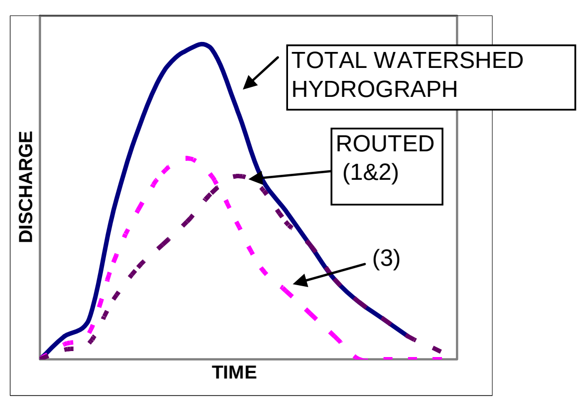
1) Determine hydrologic subbasin data
   a. Average rainfall
   b. Loss rate criteria
   c. Base flow
2) Determine unit hydrographs for each subbasin
3) Construct runoff hydrographs at each subbasin outlet
4) Combine hydrographs from subbasins 1 & 2
5) Route combined hydrograph (1&2) to outlet of subbasin 3
6) Compute runoff hydrograph at outlet of subbasin 3
7) Combine the routed hydrograph with subbasin 3's hydrograph to get the total watershed's runoff hydrograph.



(STEP 4)



(STEP 5)



(STEP 7)

3.5.2 NRCS Hydrology Manual for North Dakota

The Hydrology Manual for North Dakota, published by the NRCS in 1981, contains a chapter on hydrograph development. The procedures described are based on procedures outlined in the National Engineering Handbook, Section 4 - Hydrology, also published by the NRCS. The NRCS hydrograph procedure is based upon the conversion of rainfall to excess rainfall (runoff) through the well known Curve Number (CN) Procedure. This excess rainfall is then converted to a hydrograph based upon NRCS's dimensionless hydrograph. The Hydrology Manual for North Dakota contains tables of hydrograph coordinates for a 24-hour duration-Type I storm for various times of concentration. This tabular development was based upon an analysis by TR-20. TR-20 is a computer software program available for the development and routing of hydrographs, to be described later.

3.5.3 NRCS TR No. 55

The tabular method of TR No. 55, which is discussed in Chapter 5 of this technical release, was designed for use in the following circumstances:

1. For developing composite flood hydrographs at any point within any watershed.
2. For measuring the effects of changes in land use in subwatersheds.
3. Assessing the effects of structures or combinations of structures.

In general, the procedure was intended for measuring the effect on the composite flood hydrograph of changes within subwatersheds of a larger drainage area. The input requirements for the tabular method are minimal. The 24-hour rainfall depth (inches) for a selected exceedence probability is required. For each subwatershed, the drainage area, the runoff curve number, and the time of concentration must be determined. In addition, the travel time for each channel reach is necessary.

Before the method is used, the design engineer should be familiar with several limitations. First, the constraints that were used in developing the method are important in its application. The tabular method was developed by making numerous computer runs with the TR-20 program. In each case, a runoff CN=75 was used, and the rainfall volumes were sufficient to yield 3" of runoff. When the tabular method is applied to cases having characteristics that are significantly different from the conditions used in developing the method, then the resulting hydrograph may not provide close agreement with the hydrograph that would result from a TR-20 analysis. These assumptions are not considered to be critical when the sole purpose of using the method is to assess the effect of changes in the watershed, such as land use or structure changes. The difference in the before/after hydrographs is relatively insensitive to the assumption of CN=75.

In order to make accurate assessments of watershed changes, there are certain limitations that should be adhered to in applying the method. First, within any subwatershed there should be little variation in CN; this does not mean that subwatersheds should have similar CN's but that each subwatershed should have little variation in soil and land use characteristics. Second, the area of each subwatershed should be less than 20 square miles. Third, the precipitation should be sufficient to yield runoff volumes greater than 1.5 inches, especially when CN's are less than 60. The TR-55 tabular methodology is available in an automated version from several commercially available software sources.

3.5.4 NRCS TR NO. 20

The TR No. 20 is a computer program containing the methodologies used by the NRCS (formerly Soil Conservation Service) as presented in the National Engineering Handbook, Section 4 - Hydrology. The program is recognized as engineer oriented rather than a computer oriented package, having been developed for ease of use. Input data sheets and output data are designed for ease in use and interpretation by design engineers, and the program contains a liberal number of operations that are user friendly, even at the expense of machine time.

TR No. 20 was designed to use soil and land use information to determine hydrographs for known storms, and to perform reservoir and channel routing of the generated hydrographs. It is a single-event model, with no provision for additional losses or infiltration between discrete storm events. The program has been used in all 50 states by NRCS engineers and consulting engineers for flood insurance and flood hazard studies, for the design of reservoir and channel projects, and for urban and rural watershed planning.

Surface runoff is computed from any historical or synthetic storm using the NRCS Curve Number (CN) approach. The standard dimensionless hydrograph is used to develop unit hydrographs for each subarea in the watershed. The excess rainfall hyetograph is constructed using the effective rain and the given rainfall distribution. It is then applied incrementally to the unit hydrograph to obtain the subarea runoff hydrograph for the storm.

TR No. 20 uses the storage-indication method to route hydrographs through reservoirs. The base flow is added to the direct runoff hydrographs to produce the total flow rates. The program logic computes the total flow hydrographs from each subarea, routing the flows through stream channels and reservoirs, combining the routed hydrographs with those of other tributaries, and routing the combined hydrographs to the watershed outlet.

Subdivision of the watershed is facilitated by determining the locations of control points. Control points are defined as stream locations corresponding to cross-sectional data, reservoir sites, damage centers, diversion points, gauging stations, or tributary confluences where hydrograph data may be desired. Subarea data requirements include the drainage area, the time of concentration, the reach lengths, structure data, and either routing coefficients for each reach or cross-sectional data along the channels. Whenever cross-section data is provided, the model calculates the water surface elevations in addition to the peak flow rates and time of concentration at each station.

Minimal input data requirements to TR No. 20 include the watershed characteristics; at least one actual or synthetic storm including the depth, duration, and distribution; the discharge, capacity, and elevation data for each structure; and the routing coefficient or cross-sectional data for each reach.

3.5.5 U.S. Army Corps of Engineers HEC-1

The HEC-1 computer model consists of a calling program and six subroutines. Two of these subroutines determine the optimal unit hydrograph, loss rate, or stream flow routing perimeters by matching recorded and simulated hydrograph values. The other subroutines perform snow melt computations, unit hydrograph computations, hydrograph routing and combining computations, and hydrograph balancing computations. In addition to being capable of simulating the usual rainfall-runoff event processes, HEC-1 will also simulate multiple floods for multiple basin development plans and perform the economic analysis of flood damages.

HEC-1 underwent revisions in the early 1970's and again in the 1980's. Several features were added including the NRCS Curve Number method for loss rate, hydraulic routing techniques, kinematic hydrograph routing, simulation of urban runoff, hydrograph analysis for flow over a dam or spillway, analysis of downstream impacts, dam break analysis, multi-stage pumping plants for interior drainage, and flood control system economics.

The HEC-1 software allows the design engineer a choice of several hydrologic or storage-routing techniques for routing floods through stream reaches and ponds or reservoirs. All these use the continuity equation and some form of the storage-outflow relation.

3.5.6 Storm Water Management Model (SWMM)

The Environmental Protection Agency model, SWMM, is primarily an urban runoff simulation model. Like other computer models developed by federal agencies, SWMM, has undergone numerous modifications and improvements since its first release. SWMM's hydrograph and routing routines are hydraulic rather than hydrologic. A distributed parameter approach is used for subcatchments consisting of single parking lots, city lots, and so on. Accumulated rainfall on these plots is first routed as overland flow to gutter or storm drain inlets, where it is then routed as open or closed channel flow to the receiving waters or to some type of treatment facility.

3.5.7 Other Methods

There are several other commercially available software packages which may be used for hydrograph development, routing, and general urban watershed analysis. Generally, these computer packages are based upon sound hydrologic principles and provide adequate information for the design of storm water storage facilities. Because of the great variety of hydrograph software commercially available, and because new software is continuously under development, no attempts will be made to describe these other methodologies.

3.6 HYDROGRAPH ROUTING

3.6.1 General

Routing refers to the engineering procedure of tracing the movement of water throughout a hydrologic system, which may be composed of natural streams and lakes, man made conveyance channels, and reservoirs in addition to the natural channels. The basic scientific principles associated with routing involve the conservation of mass and energy. In the case of water flow in natural systems, the conservation of mass (which includes, volume conservation and time continuity) is normally the dominant factor. Channel routing is concerned with timing and energy conservation, where as reservoir or pond routing is most extensively concerned with volume.

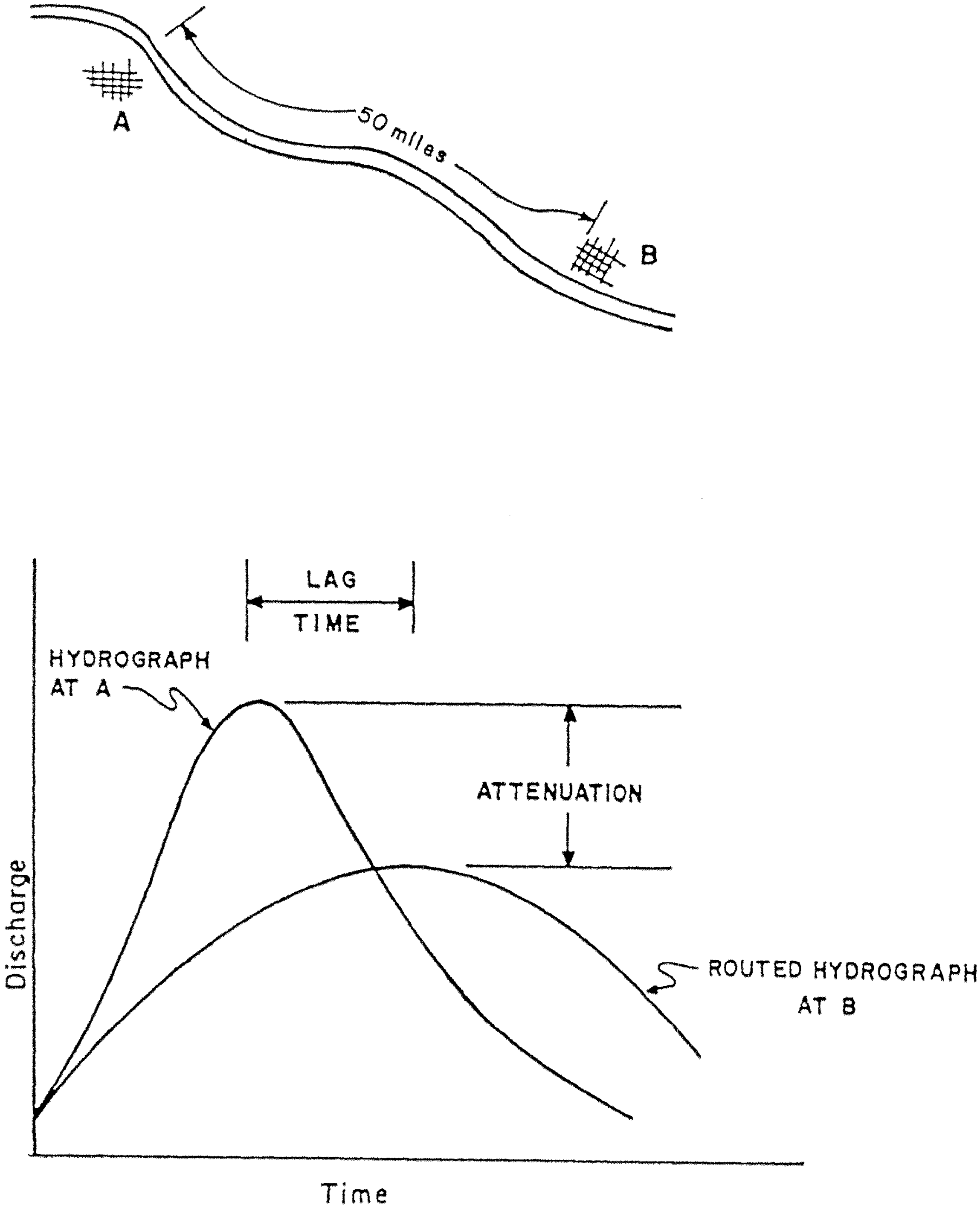
The routing of flood hydrographs is required to ascertain the effect of changes within the watershed on downstream flows and to compute flow at desired or gauged locations throughout the system, using known flows. Figure No. 3.8 illustrates the concept of flood routing of an inflow hydrograph (at point A) to a downstream point (point B). Routing studies are required to predict the effects of man made works on downstream flows. For example, detention reservoirs or ponds are designed to modify the time distribution of the system's storage and consequently also modify stream flows. Other man made works which may be studied using routing techniques include channel modifications, levees, and flood walls.

Routing techniques may be classified into two categories - hydrologic routing and hydraulic routing. Hydrologic routing employs the equation of continuity with either an analytic or an assumed relationship between storage and discharge within the system. Hydraulic routing, on the other hand, uses both the equation of continuity and an equation of motion, customarily the momentum equation. Hydraulic routing more adequately describes the dynamics of flow then does the hydrologic routing technique.

3.6.2 Channel Routing

Hydrologic channel routing techniques are all founded on the equation of continuity. That is, inflow volume minus outflow volume must equal the change in the storage volume within a channel reach for a specified time interval. Several common hydrologic channel routing methods include the Muskingum Method, the Tatum Method, Straddle-Stagger Method, and Modified-Puls Method. All of these hydrologic routing methods are contained as optional subroutines within HEC-1. Hydraulic channel routing methodologies include the kinematic wave approximation, and the Att-Kin method. The HEC-1 model contains the kinematic hydraulic routing option while the NRCS TR-20 model contains the Att-Kin method.

Hydrograph Routing
Figure No. 3.8



3.6.3 Pond and Reservoir Routing

A flood wave passing through a storage pond or reservoir is both delayed and attenuated as it enters and spreads over the pool surface. Water stored in the pond is gradually released as pipe flow through the outlet works, called principal spillways, or in extreme floods, over an emergency spillway. Routing of floods through ponds or reservoirs is usually accomplished by hydrologic methods such as the Storage-Indication Method, also called the Modified-Puls Method. Required information on the pond or reservoir includes:

1. The storage-elevation relationship.
2. A discharge-pond elevation relationship.
3. An inflow discharge hydrograph.

Routing calculations can be performed manually or with computer software. Most existing hydrograph development software (i.e., HEC-1, TR-20, TR-55, etc.) contains subroutines to perform the necessary routing calculations. Design engineers are referred to several hydrology texts or manuals which adequately define the procedures.

3.7 COMPUTER PROGRAMS

There are many computer programs and software applications which automate storm water hydrology calculations. Most of these programs have already been described in previous sections of this chapter. Improvements and enhancements to these computer software packages are continually being made. The more common software packages have been described throughout this chapter. Design engineers are cautioned to utilize software which has been tried and tested, and has found common acceptance throughout the engineering community. However, this does not preclude the development of new and more enhanced software packages in the future. Prior approval should be obtained from the City Engineer before using any hydrologic software package that has not found wide acceptance throughout the engineering community.

CHAPTER 4
STORM SEWER DESIGN

4.1 INTRODUCTION

The criteria presented herein shall be used in the design and evaluation of the storm sewer systems for the City of Bismarck and its extraterritorial jurisdiction. The review of all stormwater management planning submittals will be based on the criteria presented in this section.

The phrase "storm sewer system" refers to the system of inlets, pipes, manholes or junctions, outlets and other appurtenant structures designed to collect and convey the minor storm runoff event and discharge the runoff into a Major Drainage System. The storm sewer system is a part of the Minor Drainage System, which may also include curb and gutters, roadside ditches, swales, and channels.

The storm sewer system generally receives the most attention from the engineer and the public, since the primary function is to collect and convey the regularly recurring storm runoff with minimal inconvenience to the public. If the system does not function accordingly, unnecessary inconvenience and damage can occur. The storm sewer systems must therefore be designed to minimize the nuisances of frequently occurring runoff events.

A storm sewer system is normally required when the other parts of the Minor Drainage System (especially the streets) no longer have the capacity for additional runoff. Because of this requirement, the relationship with the Major Drainage System will affect the need for a storm sewer system. The more extensive the Major Drainage System (i.e., channels), the less extensive is the need for the storm sewer system, which is the largest cost component of the Total Drainage System.

Presented in this section, along with the technical criteria, is the general procedure for design and evaluation of a storm sewer. Storm sewer inlet design is discussed in Chapter 5.

4.2 LOCATION OF STORM SEWERS

1. Storm Sewers in Street Right-of-Way

    A. Parallel storm sewers to the street and in the right-of-way should be placed beneath the traveled lanes, to fit specific manhole or intake connections.

    B. Storm sewers should be located in streets with adequate separation from sanitary sewers.

2. Storm Sewers in Easements

    A. All easements should be identified as described in Chapter 12.

B. All easements for installation of storm sewer pipe should be 30 feet wide. The pipe should be placed only on one side of a joint property line. For a stockpile area, storm sewers placed in easements should normally be located 7.5 feet from the southern edge of the east/west easements and 7.5 feet from the eastern edge of north/south easements. In situations where the Project Engineer can clearly demonstrate that an easement less than 30 feet is adequate (considering depth of storm sewer, stockpile area and maintenance access) the City Engineer may consider such a request.

4.3 PIPE MATERIAL

Pipe material and pipe strength should be in conformance with the City of Bismarck's Construction specifications Section 802. The general criteria are listed below.

1. Storm sewer RCP - Min. strength - (Class 2) under parking lots and shoulders and for 27 inch or larger diameters. (Class 3) for diameters less than 24 inches under all streets and entrance pavement. (Class 5) under railroad tracks for jacked pipe.

2. Storm sewer CSP - Shall not be used under streets in public right-of-way. The use of CSP for other situations and location is subject to approval by the City Engineer.

3. Footing drain sewers - VCP, ABS, PVC, CIP, or DIP.

4. Building storm sewer stub - ABS, PVC, VCP, CIP or concrete.

4.4 PHYSICAL REQUIREMENTS

1. Min. cover (to top of pipe) on storm sewers outside of pavement is 3'-6". Min. cover under pavement for cross runs is 1'-6" and 3'-6" for parallel runs.

2. Min. cover on footing drain sewers - 3'-6".

3. Max. cover should follow the manufacturer's recommendations.

4. Min. pipe size:

    A. Storm sewers - 15" in diameter for inlet connections and 18" for main lines. Storm sewers 12" in diameter may be used for special conditions with special approval of the City Engineer.

    B. Footing drain sewers in public right-of-way - 8" in diameter.

    C. Building storm sewer stubs - 4" in diameter.

5. Crossings:

   Storm sewers near water mains should have a minimum outside of pipe vertical and horizontal separation of 1.5 feet where possible.

   Storm sewer crossings of sanitary sewers should have no less than 6 inches of outside of pipe clearance. Special structural support will be required if there is less than 18 inch clearance. The minimum horizontal clearance should be 2 feet. Clearance refers to the distance from the outside wall of the sewer pipe to the outside wall of the storm sewer pipe.

6. Minimum grade - Storm sewers and footing drain sewers.

   Cross runs - grades (See Chapter 5). A minimum velocity of 3 fps is desired for the design storm.

   All other sewers (See Discharge of Circular Pipe-CFS, Fig. 4.1).

   Building storm sewer stubs - 1/8 inch per foot.

7. Inlet Spacing - (See Storm Sewer Inlet & Design, Chapter 5).

4.5 HORIZONTAL ALIGNMENT AND SIZE CRITERIA

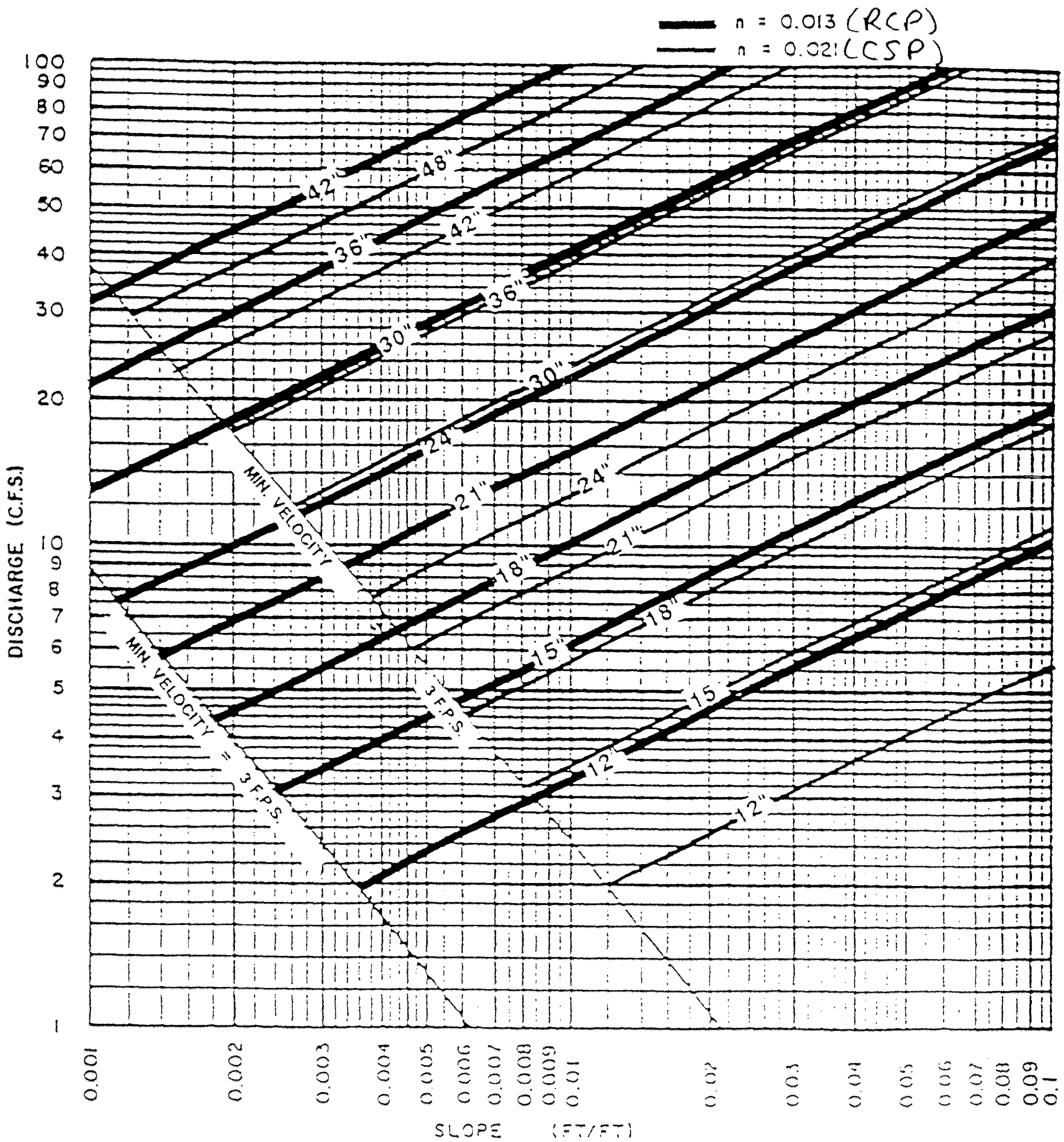
Storm sewer alignment between manholes shall be straight, except when approved in writing by the City Engineer. Storm sewers may be constructed with curvilinear alignment by the deflected straight pipe method (as defined in Concrete Pipe Handbook published by the American Concrete Pipe Association), pipe bends, or by radius pipe. The limitation on the radius for pulled joint pipe is dependent on the pipe length and diameter, and amount of opening permitted in the joint. The maximum allowable joint pull shall be 3/4-inches and is subject to prior approval of the City Engineer. The minimum parameters for radius pipe are shown in Table 4.1. The radius requirement for pipe bends is dependent upon the manufacturer's specifications.

TABLE 4.1
Minimum Radius for Radius Pipe

Pipe Diameter	Radius of Curvature
24" to 54"	28.50 ft.
57" to 72"	32.00 ft.
78" to 108"	38.00 ft.

Short radius bends shall not be used on sewers 21 inches or less in diameter.

Discharge of Circular Pipe
Figure No. 4.1



4.6 MANHOLES

1. Location: Manholes or other maintenance access ports (inlets) will be required whenever there is a change in size, direction, elevation, grade or a junction of two or more storm sewers. A manhole will be required at the beginning and/or end of a curved section of a storm sewer. When feasible, manholes should be installed at street intersections. Manholes must be located in areas which allow direct access by maintenance vehicles.

   Inlets with combined manholes will be used when the size of the connecting pipes so indicate or when horizontal clearance is necessary behind the back of curb. The design engineer is encouraged to combine inlets with manholes for storm sewers that are parallel to the street. Lamp holes or clean-out structures are required at the beginning of footing drain sewers in street right-of-way. Maintenance access ports (manholes, inlets or combinations) will be spaced at distances not greater than shown in Table 4.2.

TABLE 4.2
Manhole and Inlet Spacing

Pipe Diameter	Max. Spacing
15" or less	400'
18" to 21"	450'
24" to 30"	500'
Over 30"	660'

2. Minimum Manhole Drop: When there is an increase in sewer size of a smaller sewer connected with a larger one, the invert of the smaller sewer must be raised to maintain the same energy gradient. An approximate method of doing this is to place the 0.8 depth point of both sewers at the same elevation. When there is a change in alignment between storm sewer of 45° or greater, the suggested minimum manhole drop is 0.10'.

3. Manhole Standards: Manhole materials and construction standards shall conform to the City's Construction Specifications Section 1205, and Standard Details, Sections 1318 and 1318A.

4.7 OUTLETS

1. Where a storm sewer discharges into a natural channel or irrigation ditch, an outlet structure should be provided that will blend the storm sewer discharge into the natural channel flow in such a way as to prevent erosion of the bed or banks of the channel. As a minimum, all storm sewer pipe outlets with drainage ways will require flared end sections with apron guards. Storm sewers 30" in diameter or greater discharging into channels or ditches will require a poured footing at the outlet. Storm sewers smaller than 30" in diameter may require a poured footing depending on the channel slope and outfall conditions.

2. In an instance where the discharge velocity is high (higher than those outlined in Tables 4.3 and 4.4) or supercritical, prevention of erosion of the natural channel bed or banks in the vicinity of the outlet might require an energy dissipating structure, such as:

   A. Riprap.
   B. Concrete Slab.
   C. Gabions.
   D. Headwalls and wing wall with stilling basins.
   E. Soil Reinforcement

4.8 HYDRAULIC DESIGN

Storm sewers should be designed to convey the minor storm flood peaks without surcharging the sewer. In situations where surcharging is a concern, the hydraulic grade line may be calculated by accounting for pipe friction losses and pipe form losses. Total hydraulic losses will include friction, expansion, contraction, bend, and junction losses. The methods for estimating these losses are presented herein.

1. Pipe Friction Losses

   The Manning's "n" values to be used in the calculation of storm sewer capacity and velocity are shown as follows:

Type of Pipe	Manning's "n"
Vitrified clay pipe	0.013
Plastic pipe (smooth wall)	0.013
Concrete pipe	0.013
Corrugated plastic pipe	0.020
CSP (2-2/3" x 1/2" corrugations)	0.024
	0.015 (spun asphalt lined)
CSP (3" x 1" corrugations)	0.027
Structural Plate	0.032

2. Velocity within Pipe

   A. Min. under 5-year design storm flow = 3 fps
   B. Max. under 5-year design storm flow = 20 fps

PERMISSIBLE VELOCITIES FOR CHANNELS WITH ERODIBLE LININGS, BASED ON UNIFORM FLOW IN CONTINUOUSLY WET, AGED CHANNELS[1]

TABLE 4.3

Soil type or lining (earth; no vegetation)	Maximum Permissible Velocities for---		
	Clear Water (fps)	Water carrying fine silts (fps)	Water carrying sand and gravel (fps)
Fine sand (noncolloidal)	1.5	2.5	1.5
Sandy loam (noncolloidal)	1.7	2.5	2.0
Silt loam (noncolloidal)	2.0	3.0	2.0
Ordinary firm loam	2.5	3.5	2.2
Volcanic ash	2.5	3.5	2.0
Fine gravel	2.5	5.0	3.7
Stiff clay	3.7	5.0	3.0
Graded, loam to cobbles (noncolloidal)	3.7	5.0	5.0
Graded, silt to cobbles (colloidal)	4.0	5.5	5.0
Alluvial silts (noncolloidal)	2.0	3.5	2.0
Alluvial silts (colloidal)	3.7	5.0	3.0
Coarse gravel (noncolloidal)	4.0	6.0	6.5
Cobbles and shingles	5.0	5.5	6.5
Shales and hard pans	6.0	6.0	5.0
Fabric and excelsior mat	7.0	7.0	7.0
Dry riprap/gabions	10.0	10.0	10.0
Concrete pilot channel	Use grass permissible velocity - Table 4.4		

[1] As recommended by Special Committee on Irrigation Research, American Society of Civil Engineers, 1926.

PERMISSIBLE VELOCITIES FOR CHANNELS LINED WITH UNIFORM STANDS OF VARIOUS GRASS COVERS, WELL MAINTAINED[1,2]

TABLE 4.4

Cover	Slope Range (Percent)	Permissible Velocity on--	
		Erosion Resistant Soils (fps)	Easily Eroded Soils (fps)
Bermuda grass	0-5 5-10 Over 10	8 7 6	6 5 4
Buffalo grass Kentucky bluegrass Smooth brome Blue grama	0-5 5-10 Over 10	7 6 5	5 4 3
Grass mixture	0-5 5-10	5 4	4 3
Lespedeza sericea Weeping lovegrass Yellow bluestem Kudzu Alfalfa Crabgrass	0-5	3.5	2.5
Common lespedeza[3] Sudangrass[3]	[4]0-5	3.5	2.5

[1] From *Handbook of Channel Design for Soil and Water Conservation*

[2] Use velocities over 5 f.p.s. only where good covers and proper maintenance can be obtained.

[3] Annuals, used on mild slopes or as temporary protection until permanent covers are established.

[4] Use on slopes steeper than 5 percent is not recommended.

3. Velocity at Outlet of Pipe

Energy dissipation should be required when discharge velocities exceed those allowed for the downstream channel. (See Tables 4.3 and 4.4). In certain developed areas gabions may be required by the City Engineer.

A.	Max. with flared end section	=	5 fps
B.	Max. with flared end section, footing, and riprap	=	10 fps
C.	Max. with energy dissipation device	=	20 fps

4. Partially Full Pipe Flow

For convenience, charts for various pipe shapes have been developed for calculating the hydraulic properties (Figures 4.2 and 4.3). The data presented assumes that the friction coefficient, Manning's "n" value, does not vary throughout the depth.

5. Pipe Form Losses

Generally, between the inlet and outlet the flow encounters a variety of configurations in the flow passageway such as changes in pipe size, branches, bends, junctions, expansions, and contractions. These shape variations impose losses in addition to those resulting from pipe friction. Form losses are the result of fully developed turbulence and can be expressed as follows:

$$H_L = \frac{K V^2}{2g} \qquad \text{(Equation 4.1)}$$

where:

H_L = head loss (feet)

K = loss coefficient

$\frac{V^2}{2g}$ = velocity head (feet)

g = gravitational acceleration (32.2 ft/sec)

The following is a discussion of a few of the common types of form losses encountered in sewer system design.

a. Expansion Losses

Expansion in a storm sewer conduit will result in a shearing action between the incoming high velocity jet and the surrounding sewer boundary. As a result, much of the kinetic energy is dissipated by eddy currents and turbulence. The loss of head can be expressed as:

(Equation 4.2)

in which A is the cross section area, V is the average flow velocity, and K_e is the loss coefficient. Subscripts 1 and 2 denote the upstream and downstream sections, respectively. The value of K_e is about 1.0 for a sudden expansion, and about 0.2 for a well designed expansion transition. Table 4.5 presents the expansion loss coefficients for various flow conditions.

b. Contraction Losses

The form loss due to contraction is:

(Equation 4.3)

where K_c is the contraction coefficient. K_c is equal to 0.5 for a sudden contracting and about 0.1 for a well designed transition. Subscripts 1 and 2 denote the upstream and downstream sections, respectively. Table 4.5 presents the contraction loss coefficient for various flow conditions.

c. Bend Losses

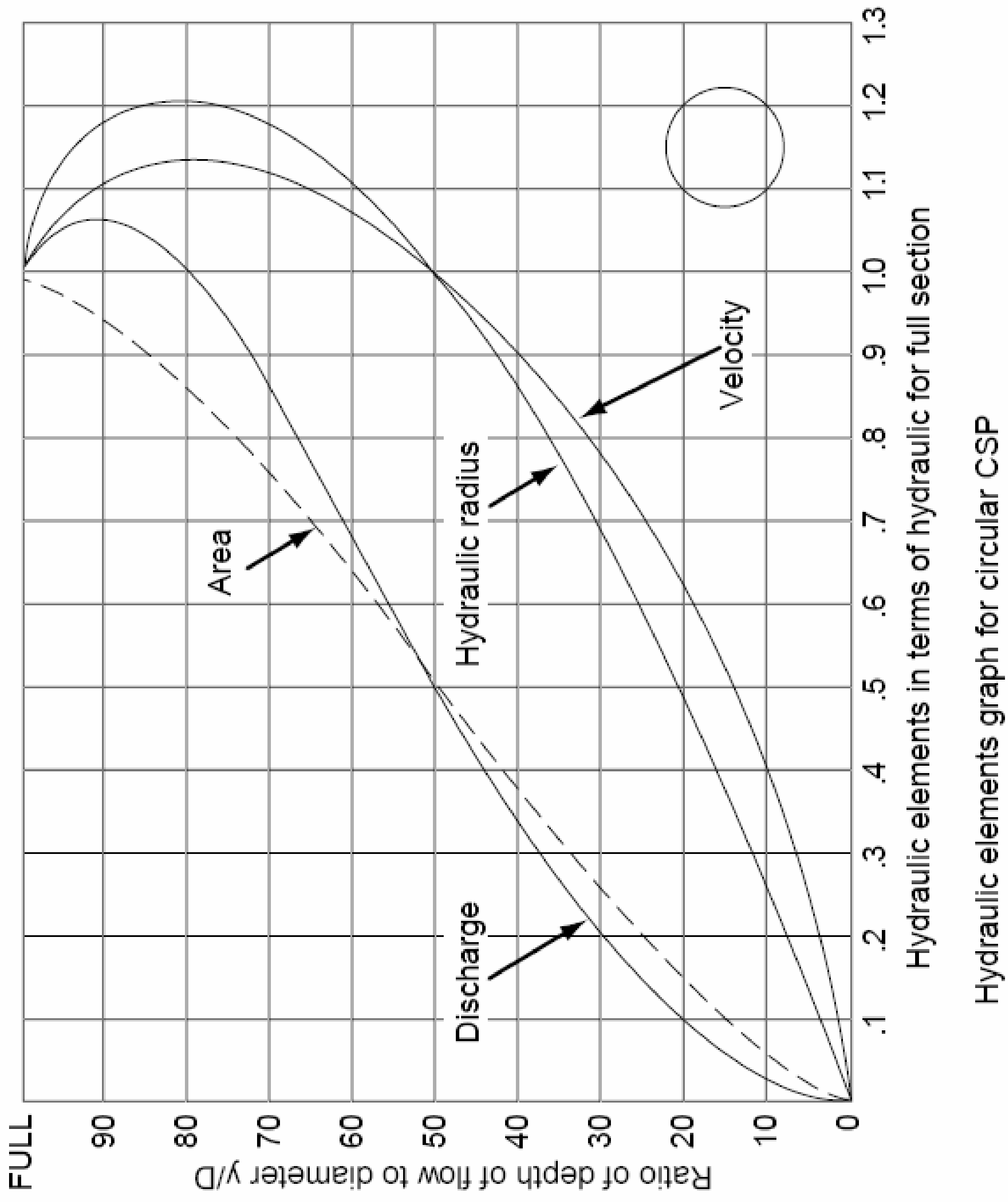
$$H_L = K_b \frac{V^2}{2g} \qquad \text{(Equation 4.4)}$$

The head losses for bends, in excess of that caused by an equivalent length of straight pipe, may be expressed by the relation:
in which K_b is the bend coefficient. The bend coefficient has been found to be a function of, (1) the ratio of the radius of curvature of the bend to the width of the conduit, (2) the deflection angle of the conduit, (3) the geometry of the cross section of flow, and (4) the Reynold's number and relative roughness.

Table 4.6 shows the recommended bend loss coefficients.

Hydraulic Properties Circular Pipe
Figure No. 4.2

HYDRAULIC PROPERTIES CIRCULAR PIPE



Hydraulic Properties ARCH Pipe
Figure No. 4.3

HYDRAULIC PROPERTIES ARCH PIPE

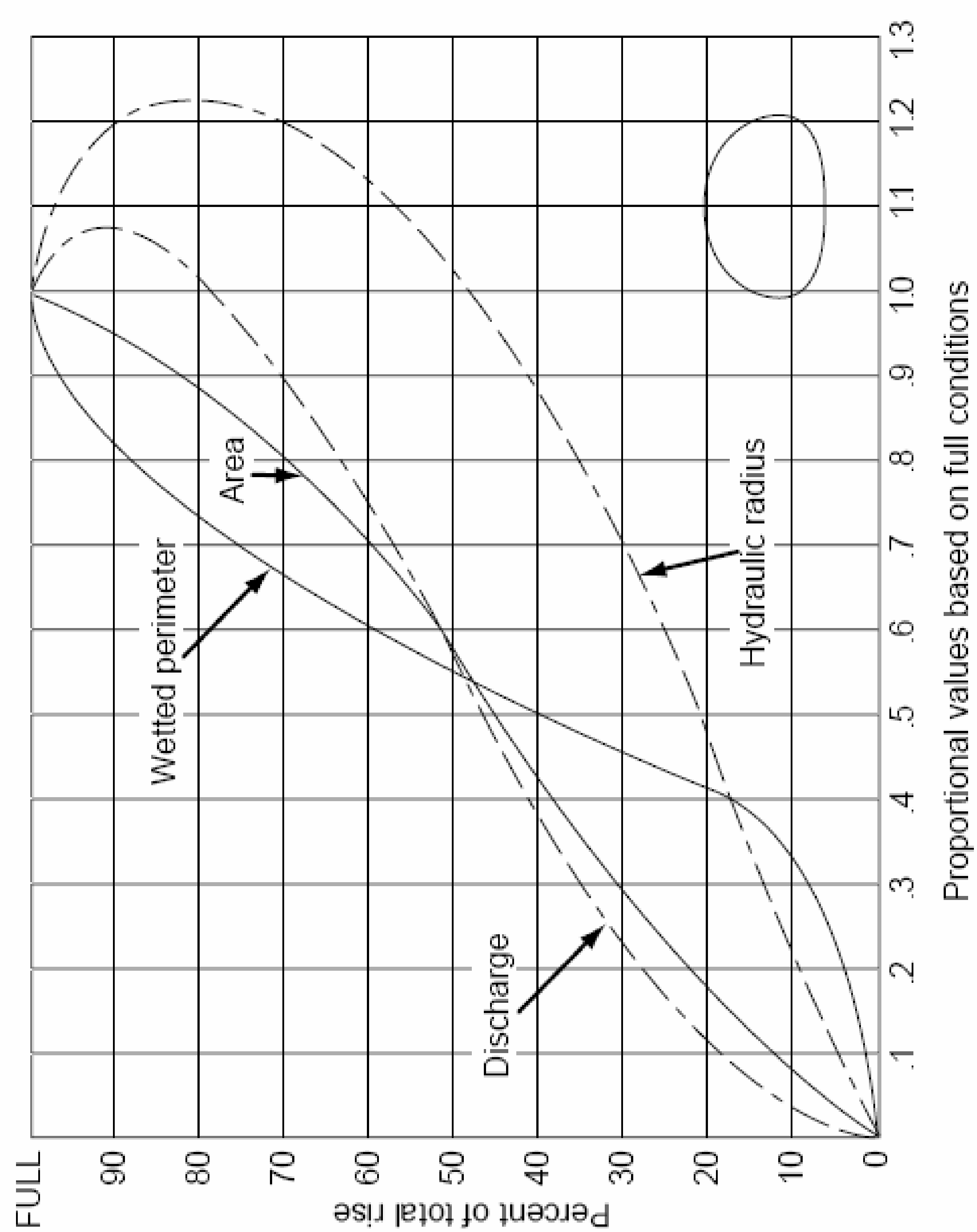


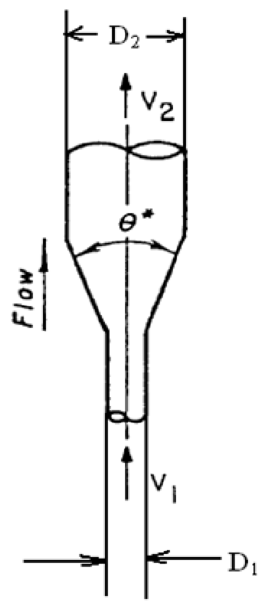
Table No. 4.5

STORM SEWER ENERGY LOSS COEFFICIENT

(a) Expansion (K_e)

θ*	$\frac{D2}{D1}=3$	$\frac{D2}{D1}=1.5$
10	0.17	0.17
20	0.40	0.40
45	0.86	1.06
60	1.02	1.21
90	1.06	1.14
120	1.04	1.07
180	1.00	1.00

* The angle θ is the angle in degrees between the sides of the tapering section.

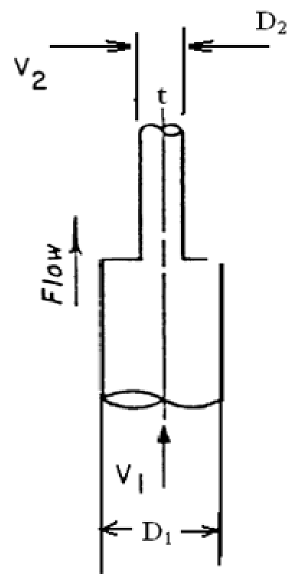


(b) Pipe Entrance from Reservoir

Bell mouth $H_L = 0.04 * \frac{V2}{2g}$

Square -- Edge $H_L = 0.5 * \frac{V2}{2g}$

Groove and U/S for Concrete Pipe $H_L = 0.2 * \frac{V2}{2g}$



(C) Contractions (Kc)

$\frac{D2}{D1}$	(Kc)
0.1	0.5
0.4	0.4
0.6	0.3
0.8	0.1
1.0	0

EQUATIONS:

$$H_L = K_e \left(\frac{V1^2}{2g} \right) \left[1 - \left(\frac{A_1}{A_2} \right) \right]^2$$

$$H_L = K_e \left(\frac{V2^2}{2g} \right) \left[1 - \left(\frac{A_1}{A_2} \right)^2 \right]^2$$

d. Junction and Manhole Losses

A junction occurs where one or more branch sewers enter a main sewer, usually at manholes. The hydraulic design of a junction is in effect the design of two or more transitions, one for each flow path. Allowances should be made for head loss due to the impact of junctions. The head loss for a straight through manhole or at an inlet entering the sewer is calculated from Equation 4.1. The head loss at a junction can be calculated from:

$$H = \frac{V_2^{\ 2}}{2g} - K_j \frac{V_1^{\ 2}}{2g} \qquad \text{(Equation 4.5)}$$

where V_2 is the outfall flow velocity and V_1 is the inlet velocity. The loss coefficient, K_j, for various junctions are presented in Table 4.7.

e. Storm Sewer Outlet Losses

When the storm sewer system discharges into the Major Drainage System (usually an open channel), additional losses occur at the outlet in the form of expansion losses. For a headwall and no wingwalls, the loss coefficient K_e = 1.0 (refer to Table 4.5), and for a flared-end section the loss coefficient is approximately 0.5 or less.

6. Design Example of Loss Calculations

The following calculation example, including the calculation Table 4.8, and Figures 4.4 and 4.5, were obtained from Modern Sewer Design, AISI Wash., D.C., 1980 and edited for the calculation of manhole and junction losses in accordance with this Section.

EXAMPLE 1: HYDRAULIC DESIGN OF STORM SEWERS

Given: a. Plan and Profile of storm sewer (Figure 4.4. and 4.5).

b. Station 0+00 (outfall) data as follows:

				Col. #
Design discharge	Q	=	145 cfs	[9]
Invert of pipe		=	94.50'	[2]
Diameter	D	=	66" RCP	[3]
Starting water surface	W.S.	=	100'	[4]
Area of pipe	A	=	23.76 sq. ft.	[6]
Velocity = Q/A	V	=	6.1 f/s	[8]

Note: (1) Number in brackets refers to the columns of Table 4.8.

(2) Sizes of the storm sewer were determined during the preliminary design phases.

Table No. 4.6

STORM SEWER ENERGY LOSS COEFFICIENT

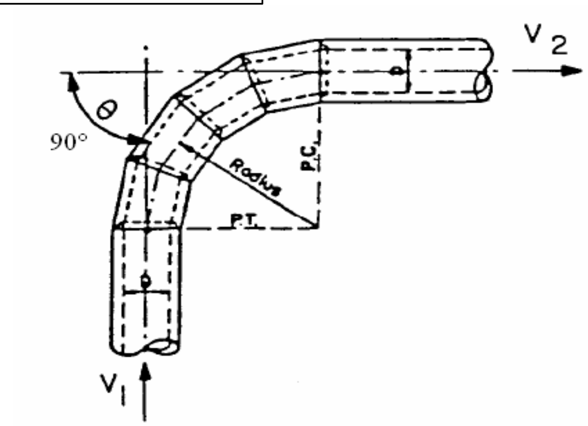
BENDS

$$\boxed{H_L = K_j(V^2/2g)}$$

CASE I
CONDUIT ON 90° CURVES*

NOTE: Head lss applied at P.C. for length

RADIUS	K_b
1 X D	0.5
(2 to 8) X D	0.25
(8 to 20) X D	0.04
>20 X D	0

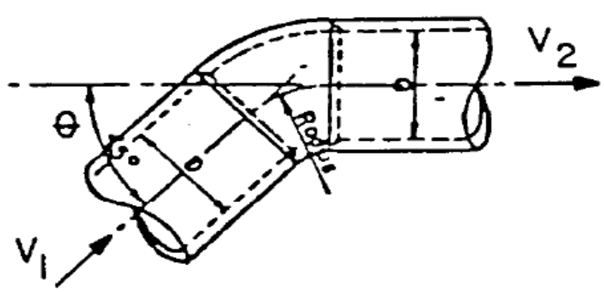


* When curves other than 90° are used, apply the following factors to 90° curves
60° curve 85%
45° curve 70%
22-1/2° curve 40%

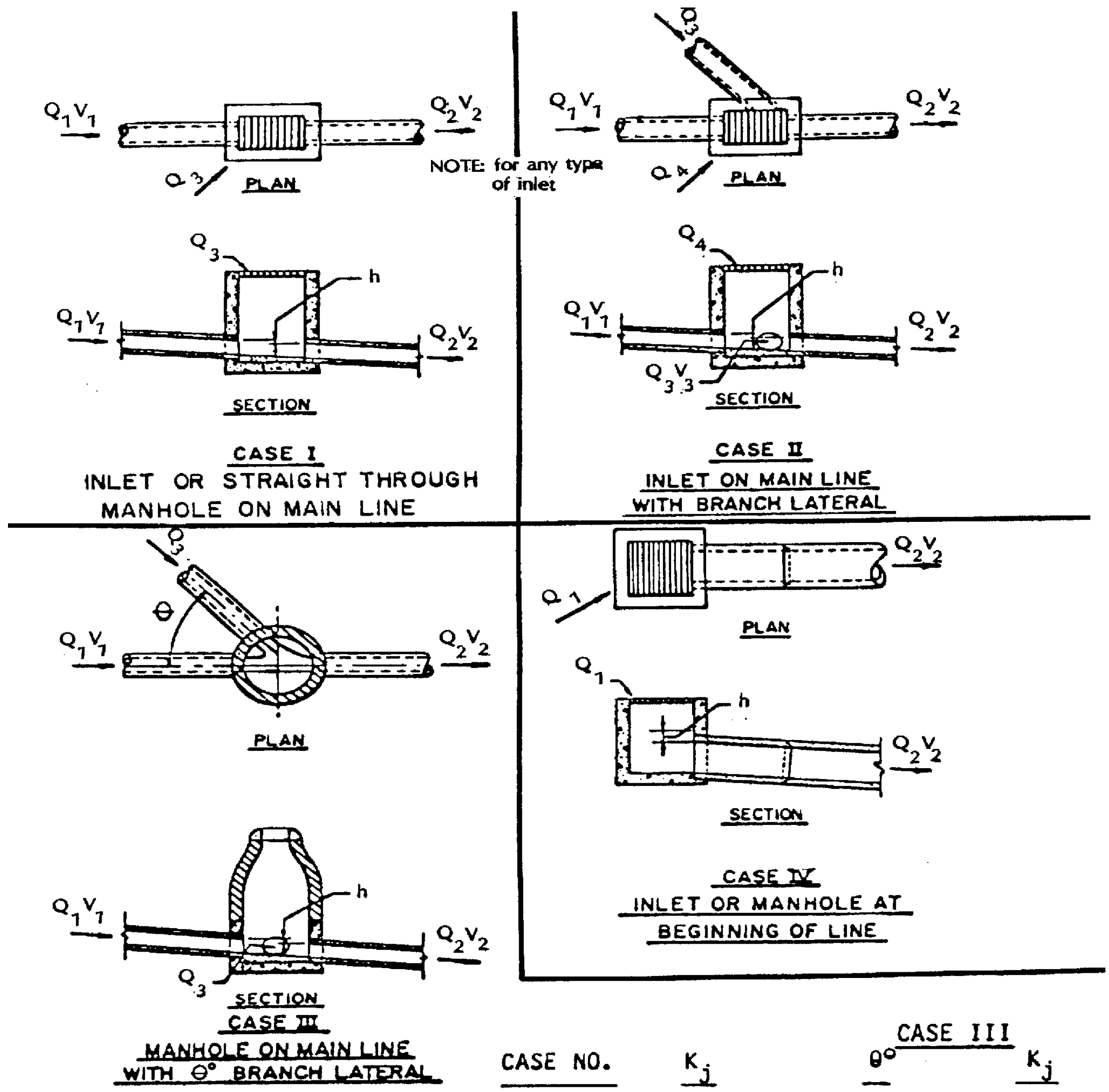
CASE II
BENDS WHERE RADIUS IS
EQUAL TO DIAMETER OF PIPE

NOTE: Head loss applied at beginnin of bend

θ° BEND	K_b
90	0.5
60	0.43
45	0.35
22-1/2	0.2



Manhole and Junction Losses
Table No. 4.7



CASE NO.	K_j
I	0.05
II	0.25
IV	1.25

CASE III

$\theta°$	K_j
0	0.95
22 1/2	0.75
45	0.50
60	0.35
90	0.25

NO LATERAL - SEE CASE I

EQUATION :

$$H_L = \left(\frac{V_2^{\,2}}{2g}\right) - K_j\left(\frac{V_1^{\,2}}{2g}\right)^2$$

FIND: Hydraulic Grade Line and Energy Grade Line for storm sewer.

DISCUSSION: The following procedure is based on full-flow pipe conditions. If the pipe is flowing substantially full (i.e., greater than 80%), the following procedures can be used with minimal loss of assumptions. However, the designer is responsible for checking the assumptions (i.e., check for full flow) to assure that the calculations are correct.

STEP 1: The normal depth is greater than critical depth, $d_n > d_c$; therefore, calculations to begin at outfall, working upstream. Compute the following parameters:

Constant value [7]: $\iota = \dfrac{2gn^2}{2.21} = \dfrac{(2)\,(32.2)\,(0.013)^2}{2.21}$

This equation is derived from the Manning's equation by solving for velocity and converting to velocity head.

$$\varnothing = 0.00492$$

This value remains constant since the n-value does not change.

STEP 2: Velocity head [10]: $H_v = V^2/2g = (6.1)^2/(2)(32.2)$

$H_v = 0.58$

STEP 3: Energy Grade Point, E.G. [11]:

E.G. = W.S. + H_v = 100 + 0.58

E.G. = 100.58

For the initial calculation, the Energy Grade Line is computed as described above. For subsequent calculations, the equation is reversed, and the water surface is calculated as follows (see Step 12):

W.S. = E.G. - H_v

This equation is used since the losses computed in Step 8 are energy losses which are added to the downstream energy grade elevation as the new starting point from which the velocity head is subtracted as shown above.

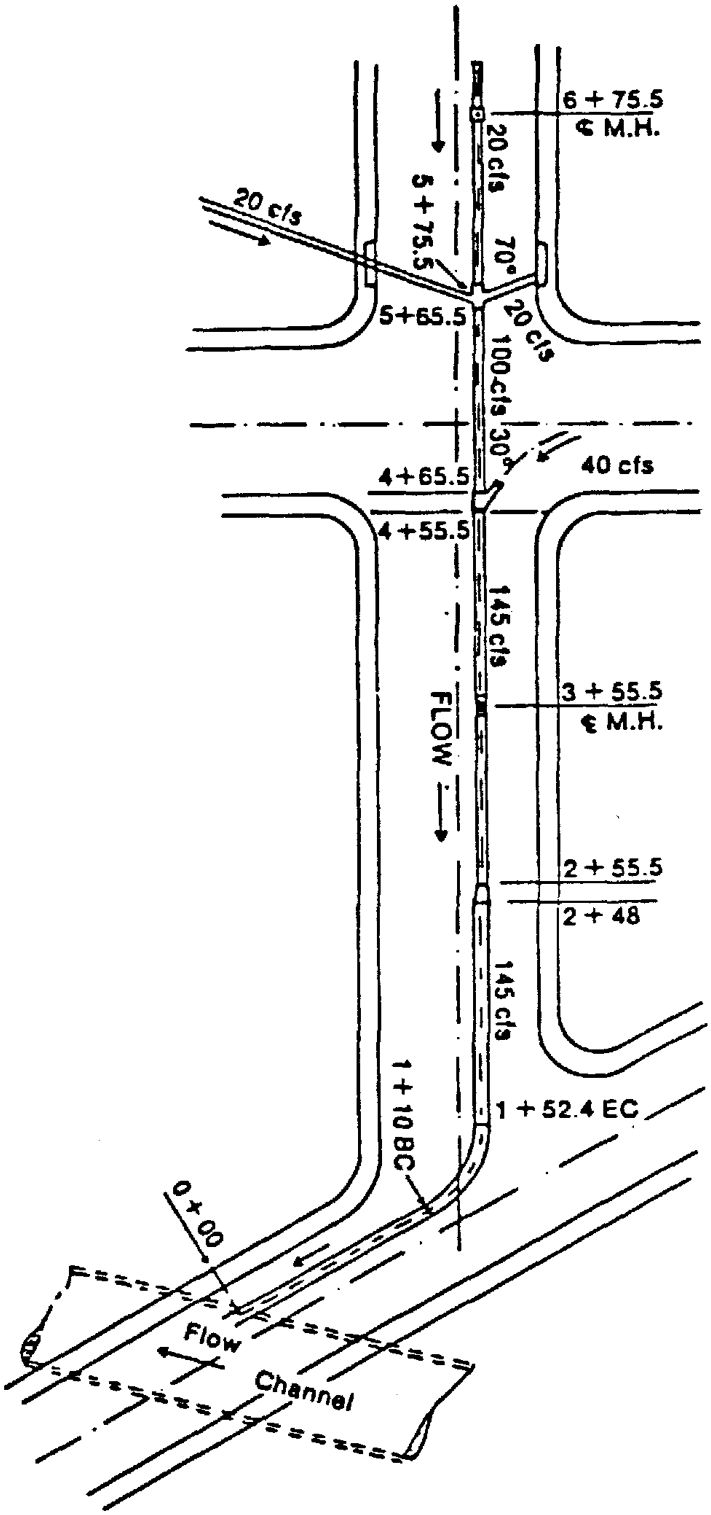
STEP 4: Skin Friction:

S_f value [12]: $S_f = \varnothing\, H_v/R^{4/3} = (0.00492)\,(0.58)/\,(1.375)^{4/3}$

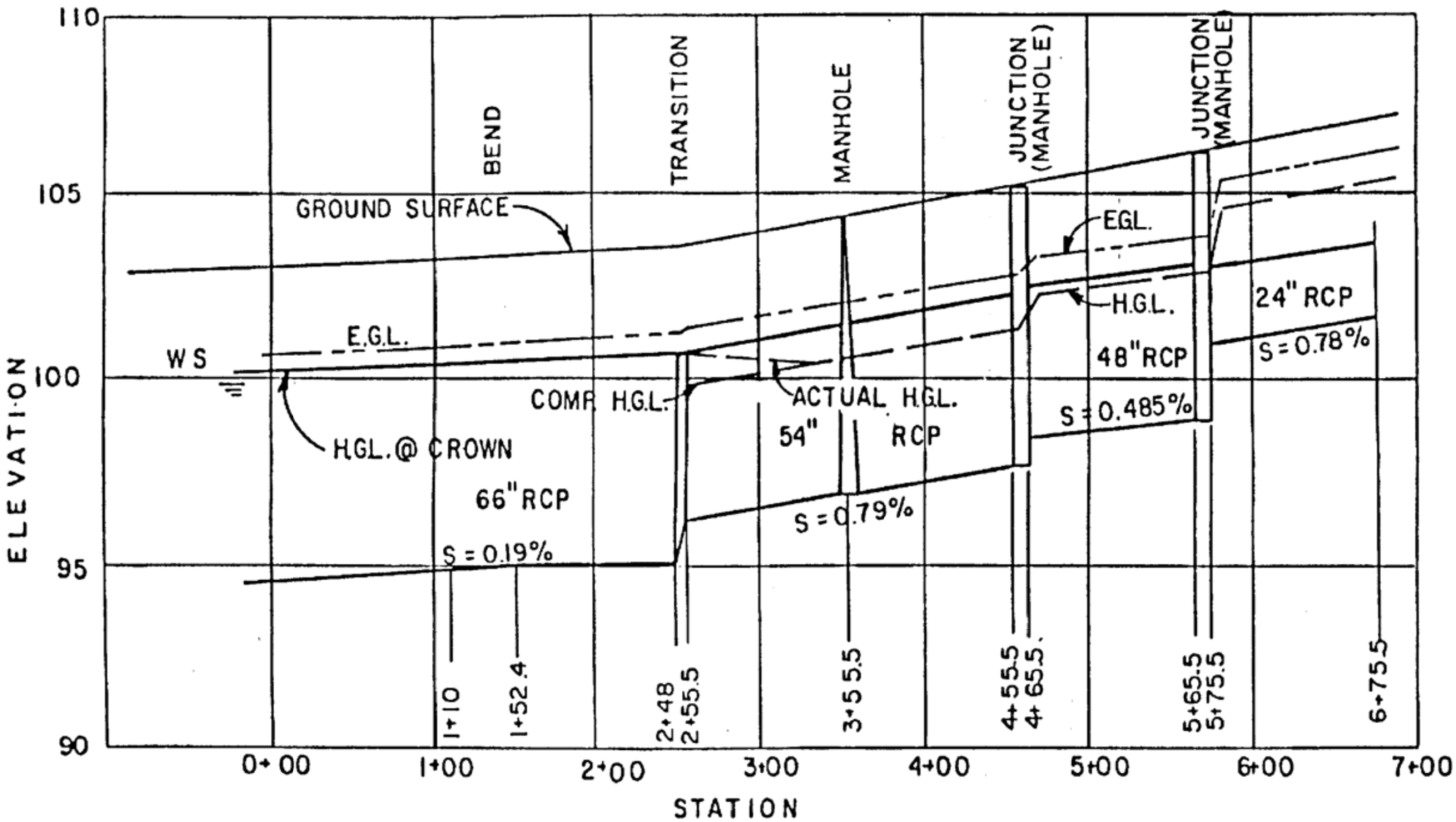
NOTE: R = the hydraulic radius of the pipe.

$S_f = 0.0019$

Figure No. 4.4
DESIGN EXAMPLE - STORM SEWERS
PLAN



Design Example – Storm Sewers Profile
Figure No. 4.5



Design Example – Storm Sewers
Table No. 4.8

Hf = Pipe Friction Loss
Hb = Pipe Bend Loss
Hj = Pipe Junction Loss
Hm = Pipe Manhole Loss
Ht = Pipe Transition Loss

Pipe Material _RCP_
Manning's *n* _0.013_

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)
station	Invert	Pipe Dia.	W.S. Elev	Pipe Shape	Area	¢	Velocity	Flow Rate	H_v	Energy Grade Line	S_f	Avg S_f	Length	H_f	H_b	H_j	H_m	H_t	Total Loss
	(ft)	(in)	(ft)		(ft^2)		(fps)	(cfs)	(ft)	(ft)			(ft)	(ft)	(ft)	(ft)	(ft)	(ft)	(ft)
0+00	94.50	66	100	RND	23.75	0.00492	6.1	145	0.58	100.58	0.0019	0.0019	110	0.21					0.21
1+10	94.71	66	100.21	RND	23.75	0.00492	6.1	145	0.58	100.79	0.0019	0.0019	42.4	0.08	0.12				0.20
1+52.4	94.91	66	100.41	RND	23.75	0.00492	6.1	145	0.58	100.99	0.0019	0.0019	95.6	0.18					0.18
2+48	95.08	66	100.59	RND	23.75	0.00492	6.1	145	0.58	101.17	0.0019	0.0048	7.5	0.04				0.15	0.19
2+55.5	96.08	54	100.07	RND	15.90	0.00492	9.1	145	1.29	101.36	0.0076	0.0076	100	0.76			0.06		0.82
3+55.5	96.90	54	100.89	RND	15.90	0.00492	9.1	145	1.29	102.18	0.0076	0.0076	100	0.76					0.76
4+55.5	97.66	54	101.65	RND	15.90	0.00492	9.1	145	1.29	102.94	0.0076	0.0063	10	0.06		0.68			0.74
4+65.5	98.40	48	102.69	RND	12.56	0.00492	8.0	100	0.99	103.68	0.0049	0.0049	100	0.49					0.49
5+65.5	98.89	48	103.18	RND	12.56	0.00492	8.0	100	0.99	104.17	0.0049	0.0064	10	0.06		1.56			1.62
5+75.5	100.89	24	105.15	RND	3.14	0.00492	6.4	20	0.64	105.79	0.0079	0.0079	100	0.79			0.03		0.82
6+75.5	101.61	24	105.79	RND	3.14	0.00492	6.4	20	0.64	106.61	0.0079								

Total Friction Loss = 3.43

¢ = $(2g(n^2))/2.21$

Total Form Loss = 2.60

Total Loss= 6.03

$S_f = (¢H_v)/R^{1.33}$

STEP 5: Avg. S_f [13]: Average skin friction: This is the average value between S_f of the station being calculated and the previous station. For the first Station, Avg. $S_f = S_f$.

Beginning with Column [13], the entries are placed in the next row since they represent the calculated losses between two stations.

STEP 6: Enter sewer length, L, in column [14].

STEP 7: Friction loss H_f [15]:

H_f = (Avg. S_f) (L)
H_f = (0.0019) (110)
H_f = 0.21

STEP 8: Calculate the form losses for bends, junctions, manholes, and transition losses (expansion or contraction) using equations 4.1, 4.2, 4.3, 4.4, and 4.5. The calculation of these losses is presented below for the various sewer segments, since all the losses occur for one sewer segment in different sewer segments and vary in type.

(a) Station 1 + 10 to 1 + 52.4 (bend)

H_b = K_b H_v, where the degree of bend is 60°
K_b = 0.20 (Table 4.6 - Case I, 85% factor for 60° curve)
H_b = (0.20) (0.58) = 0.12, enter in column 16.

(b) Station 2 + 48 to 2 + 55.5 (transition: expansion)

H_L = $K_e\ H_{v}\text{-}1\ [1 - (A_1/A_2)]^2$
K_e = 1.06 (Table 4.5) for $D_2/D_1 = 1.5$, and = 45°
H_L = $(1.06)1.29\ [1 - (15.9/23.76)]^2 = 0.15$, enter in column 19

(c) Station 3 + 55.5 (manholes, straight through)

H_m = K_m H_v
K_m = 0.05 (Table 4.7, Case I)
H_m = (0.05)(1.29) = 0.06, enter in column 18

(d) Station 4 + 55.5 to 4 + 65.5 (junction)

H_j = $H_v - 2 - K_j\ H_v - 1$
K_j = 0.62 (Table 4.7, Case III), ^{o} = 30°
H_j = 1.29 - (.62) (0.99) = 0.68, enter in column 17

(e) Station 5 + 65.5 to 5 + 75.5 (junction) - since there are two laterals, the loss is estimated as twice the loss for one lateral

K_j = 0.33 (Table 4.7, Case III), ^{o} = 70°
H = 0.99 - (0.33) (0.64) = 0.78 for one lateral

STEP 9: Sum all the form losses from columns [15] through [19] and enter in column [20]. For the reach between Stations 0+00 to 1+10, the total loss is 0.21.

STEP 10: Add the total loss in column [20] to the energy grade at the downstream end (Sta. 0+00) to compute the energy grade at the upstream end (Sta. 1+10) for this example).

E.G. (Upstream)	=	E.G. (Downstream) + TOTAL LOSS
	=	100.58 + 0.21
	=	100.79 (Column 11)

STEP 11: Enter the new invert [2], pipe diameter D [3], pipe shape [5], pipe area A, [6], the compute constant from Step 1 in column [7], the computer velocity V in column [8], the new Q [9], and the computed velocity head H_v [10].

STEP 12: Compute the new water surface, W.S., for the upstream station (1+10 for this example).

W.S.	=	E.G. - H
	=	100.79 - 0.58
	=	100.21 (column 4)

STEP 13: Repeat Steps 1 through 12 until the design is complete. The hydraulic grade line and the energy grade line are plotted on the profile (Fig. 4.5).

<u>DISCUSSION OF RESULTS</u>:

The Hydraulic Grade Line (HGL) is at the crown of the pipe from Station 0+00 to 2+48. Upstream of the transition (Station 2+55.5) the 54" RCP has a greater capacity (approximately 175 cfs) at the slope than the design flow (145 cfs). The pipe is therefore not flowing full but is substantially full (i.e., 145/175 = 0.84 greater than 0.80). The computed HGL is below the crown of the pipe. However, at the outlet, the actual HGL is higher, since the outlet of the 54" RCP is submerged by the headwater for the 66" RCP. To compute the actual profile, a backwater calculation would be required; however, this accuracy is not necessary for storm sewer design in most cases.

At the junction (Station 4+55.5), the HGL is above the top of the pipe due to the losses in the junction. In this case, however, the full flow capacity (100 cfs) is the same as the design capacity, and the HGL remains above and parallel to the top of the pipe. A similar situation occurs at the junction at Station 5+65.5.

If the pipe entering a manhole or junction is at an elevation significantly above the manhole invert, a discontinuity in the Energy Grade Line (EGL) may occur. If the EGL of the incoming pipe for the design flow condition is higher than the EGL in the manhole, then a discontinuity exists, and the higher EGL is used for the incoming pipe.

CHAPTER 5
STORM SEWER INLET DESIGN

5.1 INTRODUCTION

1. A storm inlet is an opening into a storm sewer system for the entrance of surface storm runoff. There are four basic types of inlets:

   A. Curb-gate opening
   B. Curb opening
   C. Combination inlets (Type A, B, or C Street inlet with manhole)
   D. Grate inlets (Parking lot or ditch inlet)

In addition, inlets may be further classified as being on a continuous grade or in a low point. The term "continuous grade" refers to an inlet so located that the grade of the street has a continuous slope past the inlet and therefore ponding does not occur at the inlet. The "sump" or "low point" condition exists whenever water is restricted to the inlet area because the inlet is located at a low point. A low point condition can occur at a change in grade of the street from positive to negative or due to the crown slope of a cross street when the inlet is located at an intersection.

5.2 DEFINITIONS

1. Design flow shall be defined as that quantity of water at a given point calculated from the design storm runoff. For gutter applications, the design flow shall include bypass flow from upstream inlets.

2. Bypass flow is defined as the flow in the gutter that is not intercepted by a given inlet. Bypass flow is calculated by subtracting the allowable or intercepted capacity of the given inlet from the design flow assigned to that inlet. Bypass flow shall be added to the design storm runoff for the next downstream inlet. At a minimum, inlets at a low point shall have a design capacity equal to the assigned storm discharge plus upstream bypass flows.

5.3 ABBREVIATIONS

a = Inlet Depression in Inches
d = Depth of Flow in Gutter at Face of Curb
h = Height of the Curb Opening
H = Total Head in Feet = d + a in Feet
K = Value Used in Equation $Q = K_d$ (See Figures 5.5 and 5.6)
L = Length of Curb Opening in Feet
n = Manning's Roughness Coefficient
Q = Discharge in cfs
Q_{CO}= Flow Intercepted by the Curb Opening
Q_G = Flow Intercepted by the Grate
Q_I = Allowable Flow Intercepted by the Inlet

R_F = Reduction Factor
S_L = Longitudinal Street Slope
S_T = Transverse Street Slope or Crown
T = Spread of Water in Gutter From Face of Curb in Feet
Z = $1/S_T$ = Reciprocal of Transverse Slope

5.4 INLET LOCATION AND SPACING

Street inlets for stormwater shall be located upgrade from intersections, past sidewalks, and outside of intersection radii. At least one inlet shall be located and installed at the low point of street grades.

The optimum spacing of stormwater inlets is dependent upon several factors, including traffic requirements, contributing land use, street slope, and distance to the nearest outfall. The suggested sizing and spacing of inlets is based upon the interception rate of 50 to 100% of the design flow. The most downstream inlet in a development shall be designed to intercept 100% of the flow. Considerable improvements and overall inlet system efficiency can be achieved if inlets are located in the sumps created by street intersections. Maximum spacing of inlets is 600 feet.

5.5 INLET STANDARDS

Stormwater inlets shall be designed and constructed in accordance with the City of Bismarck's Construction Specifications; Sections 1205 and 1206; and Standard Details; Sections 1319, 1320, 1320A, 1321, and 1322.

Precast concrete shall be used on shallow inlet structures when the depth from the gutter flow line to the pipe invert does not exceed 40". Concrete poured walls are allowed for all other inlet structures. Cure time is required for poured wall inlets unless high early strength concrete is used, or concrete beams are taken. Restrictions on the number of pipe connections and angle of entry may be imposed on precast concrete inlets without combination manholes.

Manhole/inlet combinations may be required if utility locations and/or pipe size show a need for the manhole. Inlets with combined manholes will be used when the size of connecting pipes so indicate or when horizontal clearance is necessary behind the back of curb.

5.6 INLET HYDRAULICS

5.6.1 Flow Interception

There are three types of inlets: curb opening, grated, and combination grate and curb opening. Inlets are further classified as being on a "continuous grade" or in a "sump".

The term "continuous grade" refers to an inlet so located that the grade of the street has a continuous slope past the inlet, and therefore ponding does not occur at the inlet. The capacity of the inlet is dependent upon many factors, including gutter slope, depth of flow in the gutter, height and length of curb opening, street cross slope, and the amount of depression at the inlet. In addition, all of the gutter flow will not be intercepted and some flow will continue past the inlet area (i.e., inlet bypass). The amount of bypass must be included in a drainage facility evaluation, as well as the design of the inlet.

The "sump" condition exists whenever water is restricted to the inlet area because the inlet is located at a low point. A sump condition can occur at a change in grade of the street from positive to negative, or at an intersection due to the crown of the slope of a cross street. The capacity of the inlet in a sump condition is dependent on the depth of ponding above the inlet. Typically, the problem consists of estimating depth of flow required to intercept a given flow amount.

Storm sewer inlets shall be designed to intercept design flow with the following allowable bypass from the system:

1. Streets on Continuous Grade - The final downstream inlet or inlets shall be designed to intercept no less than 50% of the design flow.

2. Temporary Dead-End Streets on Down Grade - Unless otherwise approved by the City Engineer, all storm sewer inlets shall be designed to intercept no less than 100% of the design flow.

3. Permanent Dead-End Streets on Down Grade and Low Points - Inlets shall be designed to intercept no less than 100% of the design flow. Depending on downstream conditions, the City Engineer may require oversizing inlets at low points.

5.6.2 Reduction Factors

The capacity of an inlet is decreased by such factors as debris plugging, pavement overlaying, etc. Therefore, the allowable capacity of an inlet shall be determined by applying the applicable reduction factor from the following table to the theoretical capacity calculated from the design procedures outlined in this section. These reduction factors are based on vane grates which are required on all curb grate inlets within the street. Other inlet grates may be approved by the City Engineer outside of the street right-of-way.

TABLE NO. 5.1
Inlet Capacity Reduction Factors

Inlet Type	Location	Min. Reduction Factor
A, B, C	Continuous Grade	90% VANE GRATES
	Low Point	80% VANE GRATES
Curb Opening	Continuous Grade	80% CURB ONLY
	Low Point	INLETS
Grate Only	Continuous Grade	75%
	Low Point	65%

5.6.3 Inlet Capacity

1. Capacity of Gutter for Straight Crown

   Figure 5.1 is the nomograph used to determine the gutter capacity for a straight crown or segmented straight crown. Figure 5.1 can also be used to approximate the capacity of curved crowns.

EXAMPLE 5.1 Give n: S_L = 4.0%

S_T	=	2.5%
n	=	0.016
Q	=	2.5 cfs
Find: d	=	depth of flow
T	=	spread of water from face of curb

Calculate the value of "Z" which is the reciprocal of the transverse slope (S_T). Z = 1/S_T = 1/.025 = 40. Calculated the ratio Z/n = 40/0.016 = 2500. Connect the Z/n ratio (2500) and the channel slope (S_L = 4.0%) with a straight line. This will give a point of intersection on the turning line. Connect this point and the discharge (2.5 cfs) with a straight line and read the depth at the face of curb d = 0.171 feet. Calculate the value of the spread using the equation:

$$T = Zd$$
$$T = 40\ (0.171) = 6.84 \text{ feet}$$

2. Capacity of Type B Inlet on a Continuous Grade:

The allowable capacity of a Type B inlet on a continuous grade shall be determined by the following equation:

$$Q_I = Kd^{5/3}R_F \qquad \text{(Equation 5.1)}$$

Figure 5.3 is used to determine "K" for a vane grate and includes the curb hood.
Figure 5.4 gives "K" for a driveway condition where no curb hood can be used.

The appropriate reduction factor from Table 5.1 must then be applied to obtain the actual flow intercepted by the inlet.

EXAMPLE 5.2 Given: Street conditions as per
Example 5.1 i.e.
26' B/B Street
d = 0.171 feet
S = 4.0%
Find: Flow intercepted by Standard
Type "B" inlet (Q_I)

Enter Figure 5.3 with the transverse slope (S_T approximated for the B/B width of the street on the figure); extend horizontally to the longitudinal slope (S = 4.0%); extend vertically downward and read the value of "K" equal to 26.5. The reduction factor for a standard inlet on a continuous grade is 90% (from Table 5.1).

$Q_I = Kd^{5/3}\ R_F$
$Q_I = 26.5\ (0.171^{5/3})\ 0.90 = 1.26$ cfs

3. Capacity of Standard Curb-Opening Inlet on a Continuous Grade

Figure 5.2 is used to determine the interception ratio of the inlet. This theoretical interception ratio (Q_I/Q) multiplied by the design flow in the gutter and the reduction factor equals the flow intercepted by the inlet.

EXAMPLE 5.3 Given: 26' B/B Street
$Q = 2.5$ cfs
$S_L = 4\%$

Find: Q_I = Flow Intercepted by Standard Inlet

From Example 5.1: d = 0.171 feet and T = 6.84 feet. Enter Figure 5.2 with T=6.84; extend vertically to the curve for $S_L = 4.0\%$; horizontally to the interpolated curve for $S_T = 2.50\%$; extend vertically and read the intercept ratio $Q_I/Q = 0.24$.

$Q_I = Q_I/Q\ (Q)\ R_F$
$Q_I = 0.24\ (2.5)\ 0.9 = 0.54$ cfs

4. Capacity of Type A, B, and C Inlets at a Low Point

Figure 5.5 is used to determine the capacity Q of a Standard Type A or F inlet at a low point. The appropriate reduction factor must be applied to the results.

EXAMPLE 5.4 Given: 26' B/B Street - Residential
S = 2% from East
S = 1% from West
60' Vertical Curve
Q = 3 cfs from East in each gutter
Q = 5 cfs from West in each gutter

Find: Q_I for Type A inlet w/vane grate

Enter Figure 5.5 width d-max. allowable for 26' B/B street, (6"). Extend vertically downward from the Standard Type A vane grate curve and read the value of 7.2 cfs.

$Q_I = 7.2\ (.80) = 5.76$ cfs $= Q_I$ maximum allowable.

Since Q_I is less than Q = 8 cfs additional inlets must be constructed to intercept flow so that flooding beyond the allowable limit on the design event does not occur.

EXAMPLE 5.5 Given: Same as previous except Q_{total} = 10 cfs

Find: d, if one Standard Type A inlet is built.

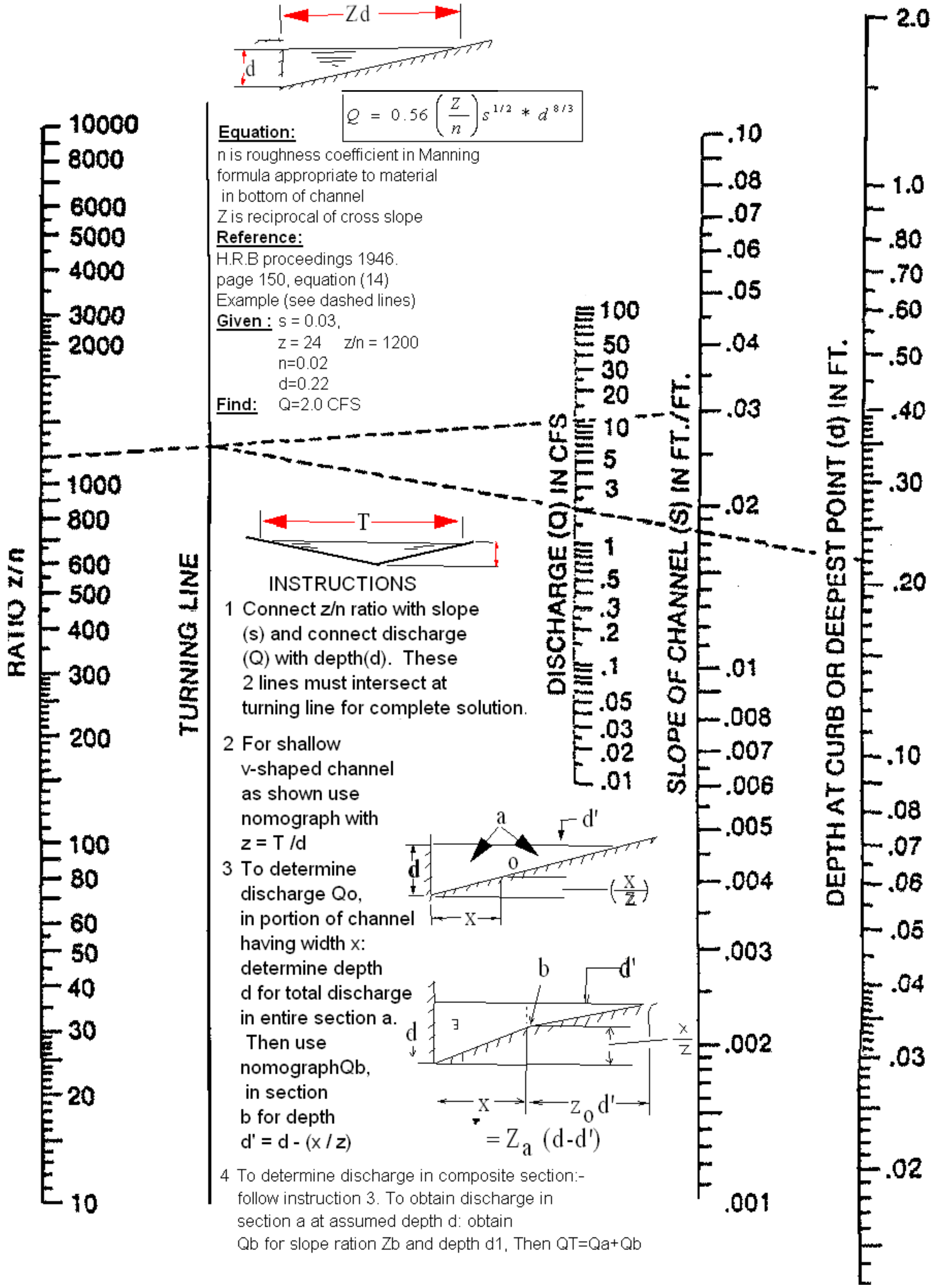
Calculate the actual flow in the gutter (if the inlet is partially clogged) so that the inlet will intercept 8 cfs.

$$Q = \frac{QI}{Rf} = \frac{10.0}{0.80} = 12.5 \text{ cfs}$$

The Q required is off the graph on Figure 5.5. Therefore, the equations shown must be used.

$12.5 = 10.59H^{1/2} + 8.25H^{3/2}$ where H = d + a
H = 0.63 ft.
d = H - a = 0.63 - 0.17 = 0.46 ft.

Nomograph for Capacity of Gutter for Straight Crown
Figure No. 5.1



Interception Ration for Standard Curb-Opening Inlet on Continuous Grade
Figure No. 5.2

STANDARD CURB - OPENING INLET CHART

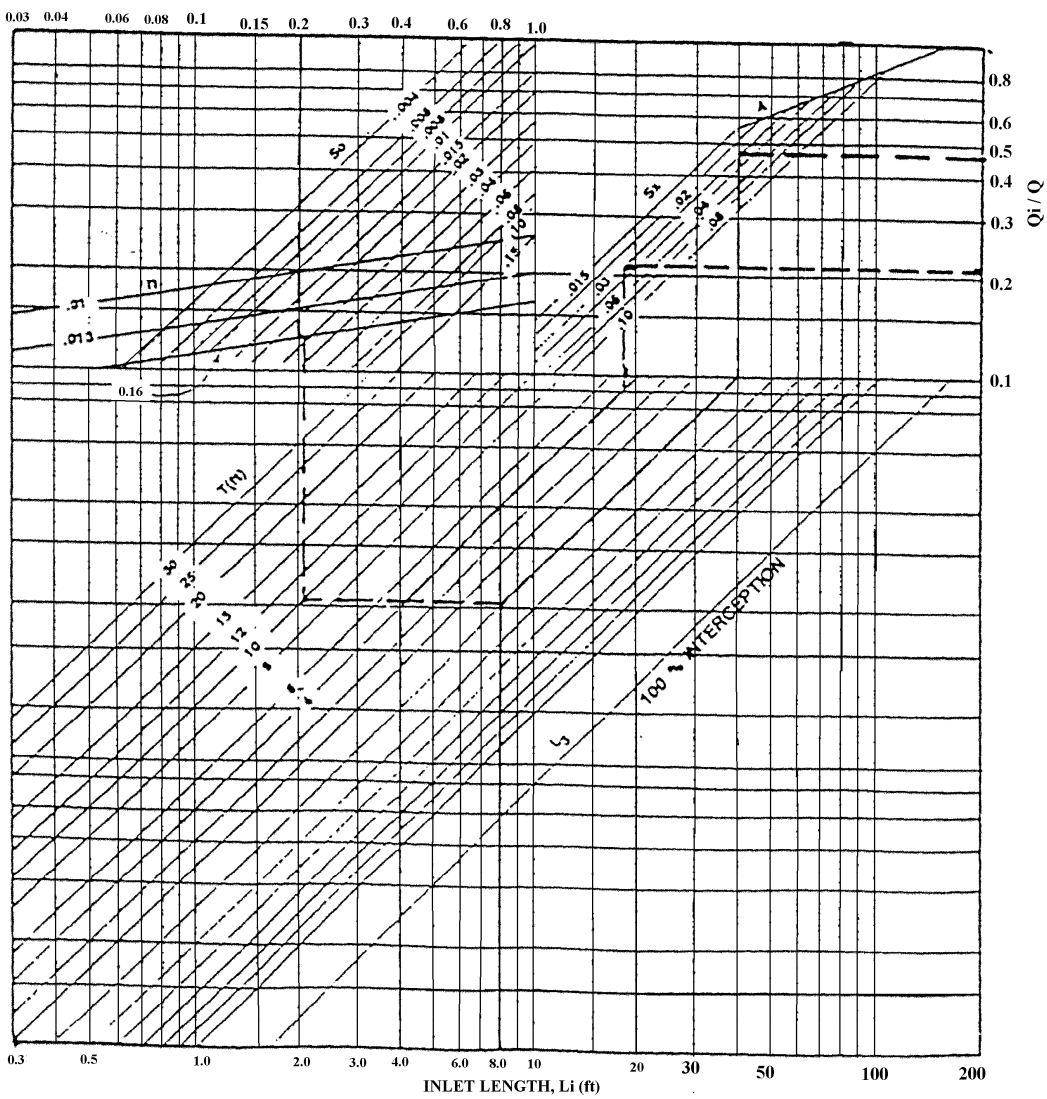
W = 2 ft
a = 2 in.
= 6in.

EXAMPLE

Given: Sx = 0.02 ft/ft.
T = 10 ft
So = 0.03 ft/ft.

Find: Li = 4 ft   Li = 8 ft.
Qi/q = 0.22   Qi / q = 0.48

Sx(T-2) = dw

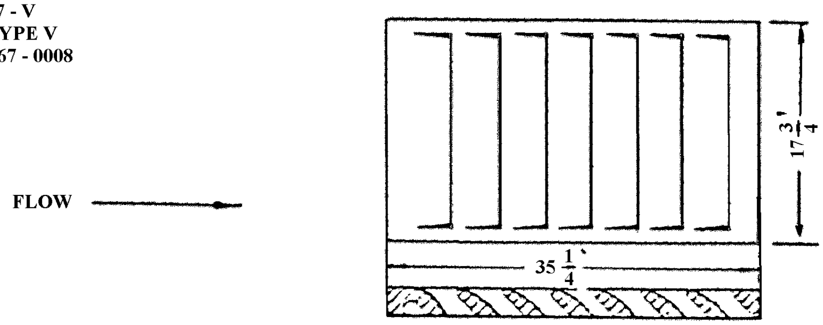


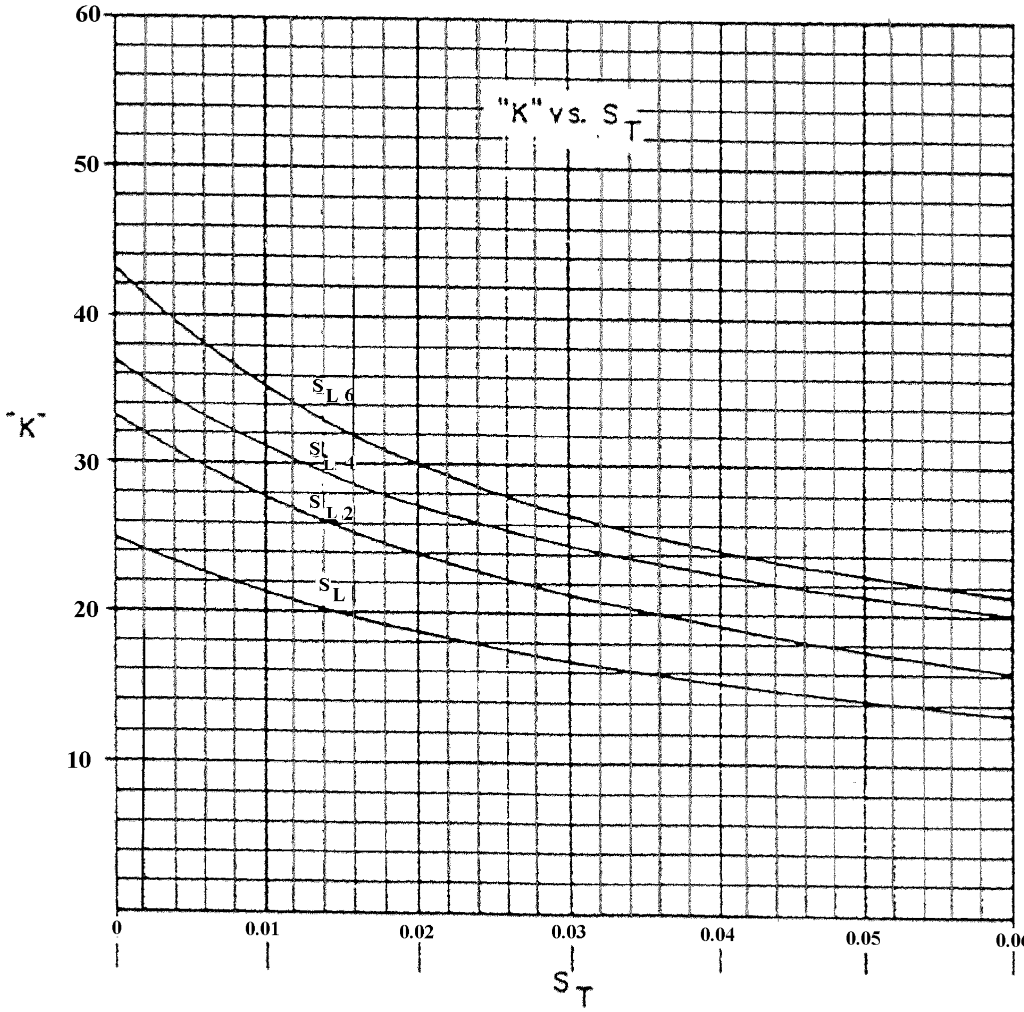
INTERCEPTION RATIO FOR STANDARD CURB-OPENING INLET ON CONTINUOUS GRADE

Inlet Capacities Information may also be found for Neenah Foundry Company at **http://www.nfco.com/literature/brochures/gratecapacities.html**

“K” Values for Type B Inlet on Continuous Grade
Figure No. 5.3

CAT. NO. - R - 3067 - V
DESCRIPTION - TYPE V
COMP. CODE - 3067 - 0008



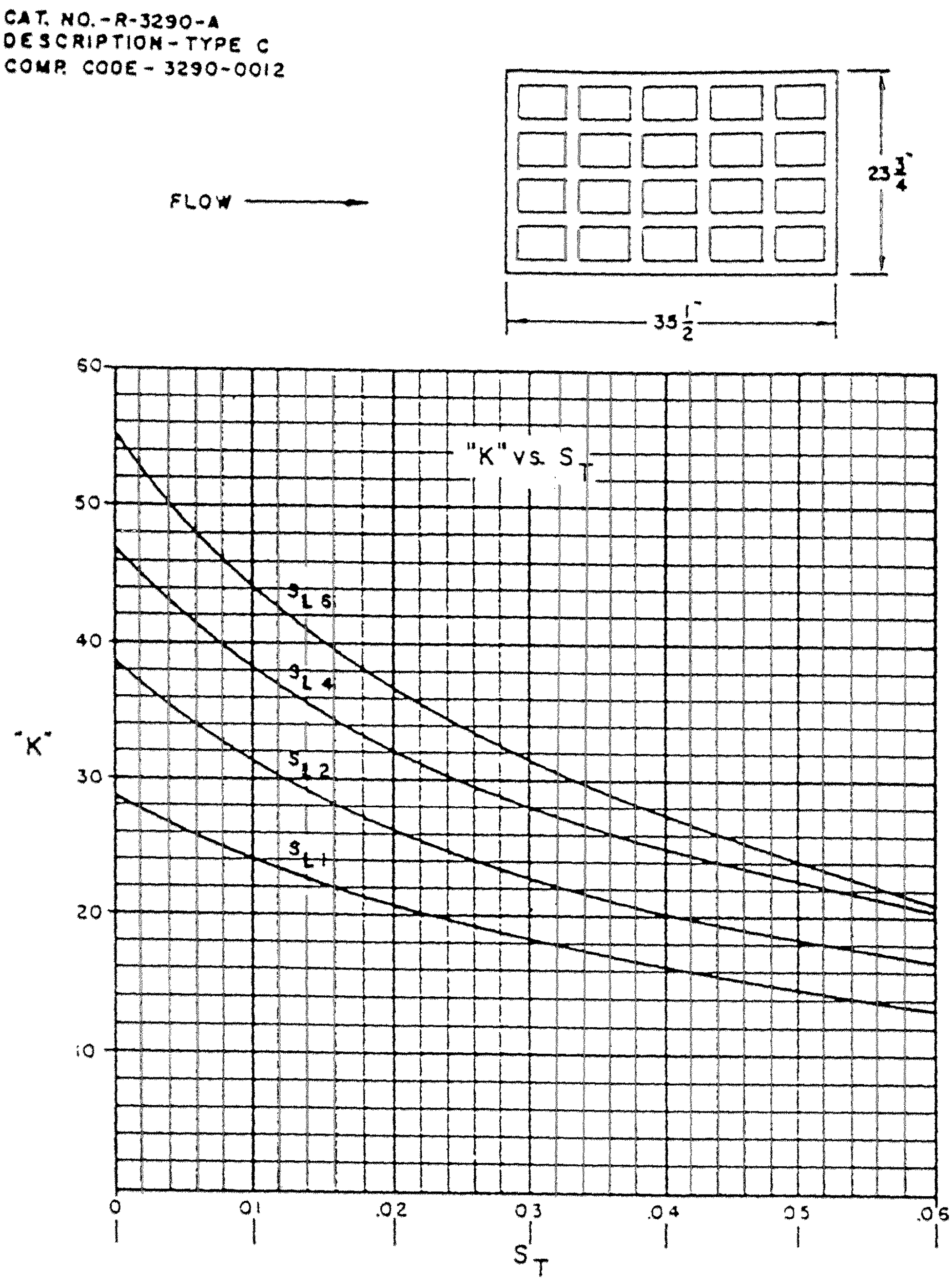


S_T = TRANSVERSE GUTTER SLOPE
S_L = LONGITUDINAL GUTTER SLOPE
K = GRATE INLET COEFFICIENT

Inlet Capacities Information may also be found for Neenah Foundry Company at
http://www.nfco.com/literature/brochures/gratecapacities.html

“K” Values for Driveway Grate Inlet

Figure No. 5.4



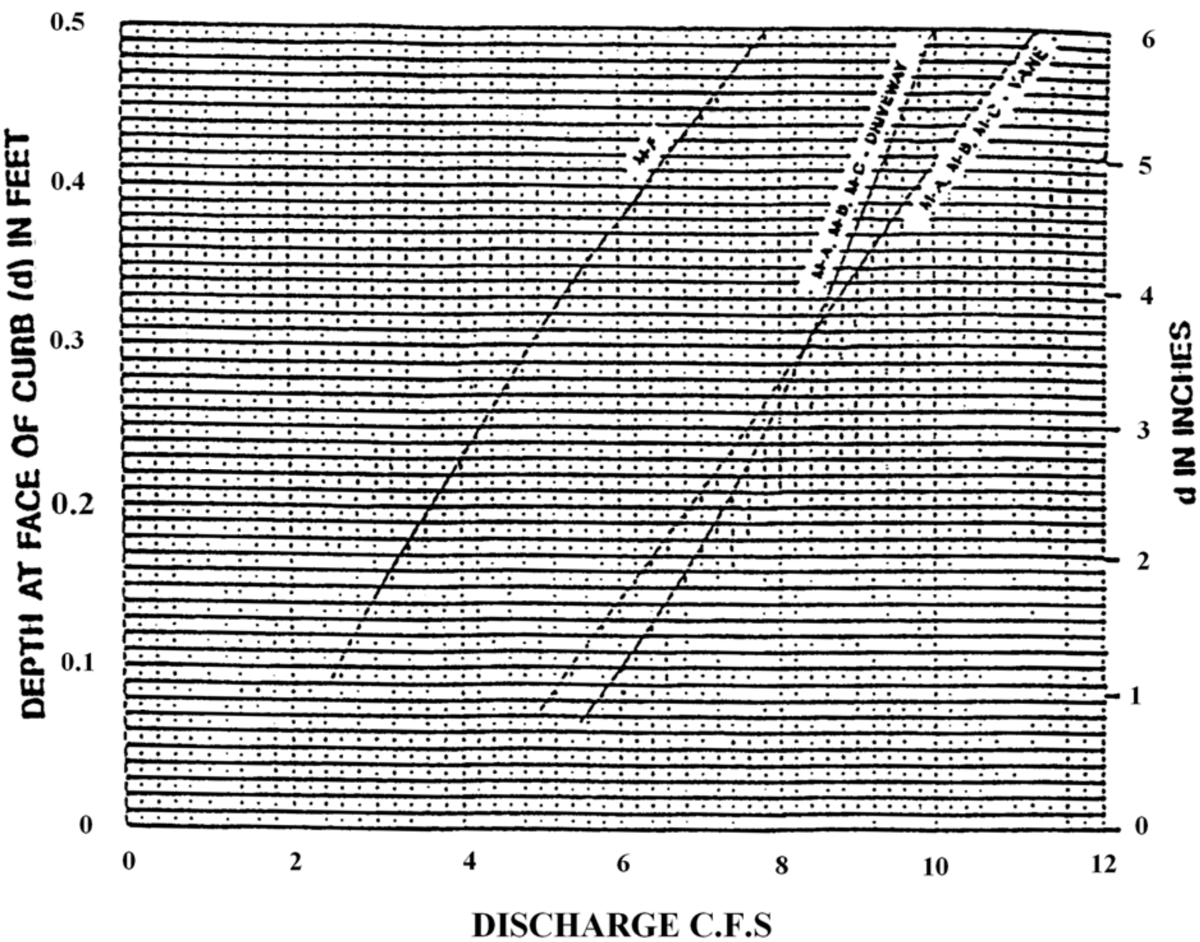
S_T = TRANSVERSE GUTTER SLOPE

S_L = LONGITUDINAL GUTTER SLOPE

K = GRATE INLET COEFFICIENT

Inlet Capacities Information may also be found for Neenah Foundry Company at
http://www.nfco.com/literature/brochures/gratecapacities.html

Capacity of Standard Single Type (A,B,C) Inlets
At a Low Point
Figure No. 5.5



EQUATIONS FOR ABOVE CURVES

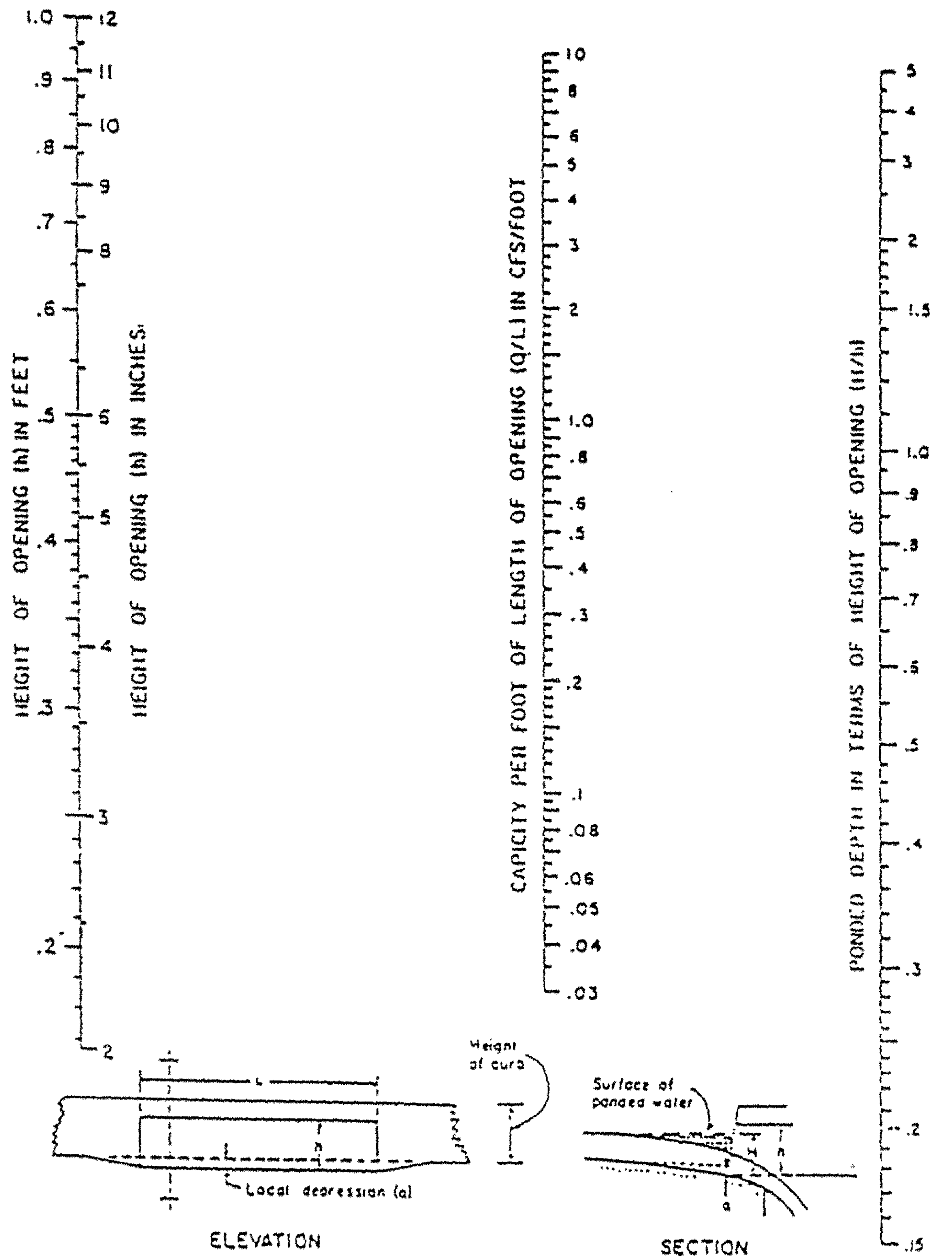
STO. M-F	**$Q = 12 H^{3/2}$**
STO. M-A, M-B, M-C VANE	**$Q = 3.44H^{1/2}-8.25H^{3/2}$**
STO. M-A, M-B, M-C DRIVEWAY	**$Q = 12.52H^{1/2}$**

Where H = d - a in feet

M-F	**a=3”**
VANE	**a=2”**
DRIVEWAY	**a=1.5”**

NOTE: FOR DOUBLE INTAKES TAKE THE VALUES CALCULATED FOR SINGLE INTAKES TIMES TWO FOR “H” GREATER THAN 3 FOOT

Capacity of Curb-Opening Inlet at a Low Point
Figure No. 5.6



CAPACITY OF GRATES FOR PONDING CONDITIONS

Discharge vs Depth On Grate

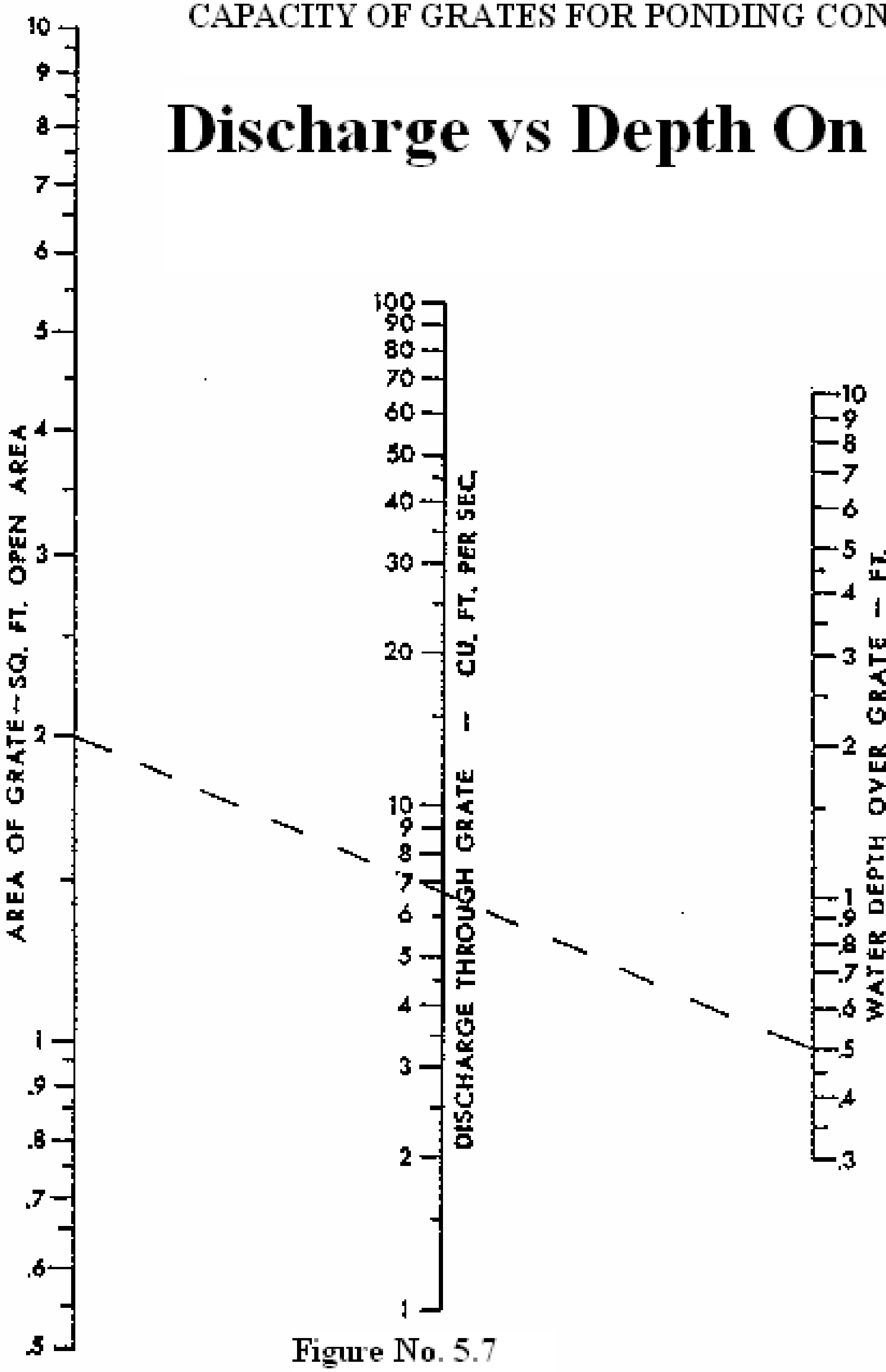
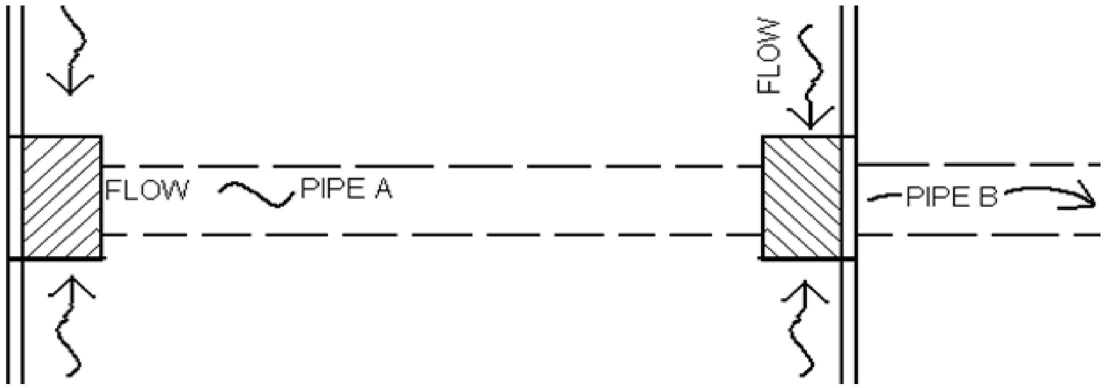


Figure No. 5.7

Pipe Standards at a Low Point for Standard Type A Inlet
Figure No. 5.8



MINIMUM FOR SINGLE INTAKES

Pipe A	Pipe B
Q = 7.6 CFS	Q = 15.2 CFS
18" Pipe @ 1.0% (Minimum)	24" Pipe @ 0.46%
15" Pipe @ 1.5%	18" Pipe @ 2.23%
	15" Pipe @ 6.0%

MINIMUM DESIRABLE FOR SINGLE INTAKES

Pipe A	Pipe B
Q = 11.8 CFS	Q = 23.6 CFS
18" Pipe @ 1.4%	24" Pipe @ 1.1%
15" Pipe @ 3.6%	18" Pipe @ 5.4%

MINIMUM FOR DOUBLE INTAKES

Pipe A	Pipe B
Q = 15.2 CFS	Q = 30.4 CFS
24" Pipe @ 0.46%	30" Pipe @ 0.55%
18" Pipe @ 2.3%	24" Pipe @ 1.8%
15" Pipe @ 6.0%	18" Pipe @ 9.0%

CHAPTER 6
HYDRAULIC DESIGN OF STREETS, CURB AND GUTTER

6.1 INTRODUCTION

The criteria presented in this section shall be used in the evaluation of the allowable drainage encroachment within public streets. The review of all planning submittals will be based on the criteria contained herein.

6.2 FUNCTION OF STREETS IN THE STORMWATER SYSTEM

Urban and rural streets, specifically the curb and gutter or the roadside ditches, are part of the Minor Drainage System. When the drainage in the street exceeds allowable limits, a storm sewer system or an open channel is required to convey the excess flows. The streets are also part of the Major Drainageway System when they carry floods in excess of the minor storm, subject to certain limitations. However, the primary function of urban streets is for traffic movement and therefore the drainage functions are subservient and must not interfere with the traffic function of the street.

Design criteria for the collection and moving of runoff water on public streets are based on a reasonable frequency and magnitude of traffic interference. That is, depending on the character of the street, certain traffic lanes can be fully inundated once during the minor design storm return period. However, during less intense storms, runoff may also inundate traffic lanes but to a lesser degree. The primary function of the streets for the Minor Drainage System is therefore to convey the nuisance flows quickly and efficiently as practical to the storm sewer or open channel drainage without interference with traffic movement. For the Major Drainageway System, the function of the streets is to provide an emergency passageway for the flood flows with minimal damage to the urban environment.

6.3 STORMWATER IMPACTS ON STREETS

6.3.1 Introduction

Storm runoff can influence the traffic function of a street in the following ways:

1. Sheet flow across the pavement resulting from precipitation runoff.
2. Runoff in the gutter.
3. Duration of the storm.
4. Ponded water.
5. Flow across traffic lanes.
6. Deterioration of the street.

To minimize the drainage impact on the streets, each of the above factors must be understood and controlled to within acceptable limits. The effects of the above factors are discussed in the following sections.

6.3.2 Sheet Flow

Rainfall on the paved surface of a street or road must flow overland in what is referred to as sheet flow until it reaches a channel. In streets which have curbs and gutters, the curb and gutter become the channel, while on roads which have a drainage ditch, the ditch becomes the channel. The depth of sheet flow will be essentially zero at the crown of the street and will increase in the direction of the curb and gutter or channel.

Traffic interference due to sheet flow is by hydroplaning or by splash. Hydroplaning occurs when vehicle tires become supported by a film of water which acts as a lubricant between the pavement and the vehicle. This phenomenon generally occurs at higher speeds associated with arterials and freeways and can result in loss of vehicle control. The potential for hydroplaning is decreased by increasing the street cross slope. This drains runoff more quickly.

Splashing of the sheet flows interferes with traffic movement by reducing visibility. Increasing the cross slope of the street crown will also reduce the splash potential. The crown slope must be kept within acceptable limits to prevent the sideways slipping of traffic during icy conditions. In general, a 2 percent cross slope is the minimum desirable slope.

6.3.3 Gutter Flow

Water which enters a street as sheet flow from the pavement surface or as overland flow from adjacent land area will flow in the gutter of the street until reaching some outlet, such as a storm sewer inlet or a channel. As the flow progresses downhill and additional areas contribute to the runoff, the width of flow will increase and progressively infringe upon the traffic lane. If vehicles are parked adjacent to the curb, the flow width will have little influence on traffic capacity until it exceeds the width of the vehicle by several feet. However, on streets where parking is not permitted, the flow width can significantly effect traffic movement after exceeding only a few feet into the street, because the flow encroaches on a moving lane rather than a parking lane. Field observations indicate that drivers will tend to crowd adjacent lanes to avoid curb flow. This creates a traffic hazard when it contributes to the potential for an increase in small accidents during rainstorms.

As the flow width increases, the traffic must eventually move through the inundated lanes progressively reducing traffic speeds and movement as the depth of flow increases. Whereas some drainage effects on traffic movement are acceptable, emergency vehicles (i.e., fire equipment, ambulances, and police vehicles) must be able to travel the streets, such as by moving along the street crown. Therefore, certain limitations on the allowable depth of flow in the street are required.

6.3.4 Storm Duration

Storm duration also plays a role in the drainage impact on the streets. The high intensity, short duration thunderstorms typical in west-central North Dakota generally do not influence traffic for more than 30 minutes per storm, and often do not occur during high traffic volume periods. Therefore, increased flow depths are tolerable for the shorter flood periods.

6.3.5 Ponding

Storm runoff ponded on the street, due to grade changes or intersecting street crowns, affects traffic movement by increasing flow depths and duration at the greater depths. Ponding is also localized and vehicles may enter the ponded area at high speeds unaware of the problem until too late. Ponding will often bring traffic to a complete halt to negotiate the ponded area without stalling the vehicle, resulting in severely reduced traffic movement. Therefore, depths of ponding must be controlled similar to gutter flow and in some cases eliminated on high traffic volume streets.

6.3.6 Cross Flow

Whenever storm runoff, other than sheet flow, moves across a traffic lane, traffic flow is affected. The cross flow may be caused by super-elevation of a curve, by the intersection of two streets, by exceeding the capacity of the higher gutter on a street with cross fall, or simply poor street design. The problem associated with this type of flow is the same as for ponding in that it is localized in nature and vehicles may be traveling at high speed when they reach the location. If the speed limits are slow and the traffic volume is light, then the influence of cross street flow may be within acceptable limits.

Cross street flow at critical designated locations and cross section (called cross pan) can occur by two separate means. One is runoff which has been flowing in a gutter and then flows across the street to the opposite gutter or inlet. The second case is flow from some external source, such as a drainageway or conduit, which will flow across the crown of the street when the conduit capacity beneath the street is exceeded. If the inundated area exceeds the street right-of-way, flow easements must be obtained. The maximum allowable cross street flow depth based on the worst condition shall not exceed the limitation stipulated in the following table.

Table No. 6.1

Allowable Cross Street Flow

Classification	Initial Design Storm Runoff	100-Year Design Storm Runoff
Local	6" depth at crown or in cross pan	9" depth at crown or in cross pan
Collector	Where cross pans allowed, depth of flow shall not	6" depth at crown or in cross pan

	exceed 3"	
Arterial	None	3" or less over crown

6.4 STREET CLASSIFICATION

The streets in Bismarck are generally classified for drainage use as local, collector, arterial, or freeway, sometimes according to the average daily traffic (ADT) for which the street is designed. The larger the ADT, the more restrictive is the allowable drainage encroachment into the driving lanes.

Presented in Table No. 6.2 are the street classification (i.e., arterial, collector, etc.) and the maximum allowable pavement encroachment and depth of flow for the minor storm runoff. Table No. 6.3 presents the allowable depth of flow and inundated area for the major storm runoff (100-year).

Table No. 6.2

Allowable Pavement and Encroachment
And
Depth of Flow for Minor Storm Runoff

Street Classification	Maximum Encroachment*
Local	No curb overtopping. Flow may spread to crown of street.
Collector	No curb overtopping. Flow spread must not encroach to within 8 feet of the centerline of a two-lane street. The flow spread for four-lane streets must leave the equivalent of two 12 feet driving lanes clear of water; one lane in each direction.
Arterials (2, 3, or 4-lane)	No curb overtopping. Flow spread must not exceed 10 feet from the face of the curb of the outside lane. For special conditions, when an intake is necessary in a raised median, the flow spread should not exceed 4 feet from the face of the median curb for an inside lane.
Freeways	No encroachment on driving lanes is allowed on any traffic lane.

*Where no curbing exists, encroachment shall not extend past property lines.

Table No. 6.3

Allowable Depth of Flow
And
Inundated Area for 100-Year Storm Runoff

Street Classification	Allowable Depth and Inundated Area
Local and Collector	The inundated area should not exceed the street right-of-way and the depth of water above the street crown should not exceed 6 inches, whichever is less.
Arterial	The inundated area should not exceed the street right-of-way and the depth of water above the street crown should not exceed 3 inches, whichever is less.

6.5 HYDRAULIC EVALUATION

6.5.1 Allowable Capacity for Minor Storm

The storm sewer system shall commence upstream from the point where the maximum allowable encroachment occurs. All storm sewers shall have the capacity to carry two-year (residential area) or (commercial and industrial areas) a five-year storm within the pipe.

The allowable minor storm capacity of each street section is calculated using the modified Manning's formula.

$$Q = (0.56)\ (Z/n) S^{1/2}\ Y^{(8/3)} \qquad \text{(Equation 6.1)}$$

Where
Q = discharge in cfs
$Q = 1/S_x$, where S_x is the cross slope of the pavement (ft./ft.)
Y = depth of water at face of curb (feet)
S = longitudinal grade of street (ft./ft.)
n = Manning's roughness coefficient

The solution to the above equation can also be obtained through the use of the nomograph of Figure No. 5.1, reproduced in this chapter as Figure No. 6.1.

When the allowable pavement encroachment has been determined, the theoretical gutter carrying capacity for a particular encroachment shall be computed using the modified Manning's formula for flow in a small triangular channel as shown in Figure No. 6.1. An "n" value of 0.016 shall be used unless special considerations exist.

The allowable capacity for each street cross-section has been calculated and is presented in

Figure No. 6.2. The calculations were performed for various allowable flow depths and street slopes. A Manning's n-value of 0.016 was used for the gutter and street flow areas. The theoretical capacity was calculated and then the reduction factor was applied to reduce the potential for damage from regular recurring storm runoff and to prolong the life of the pavement.

6.5.2 Allowable Capacity for Major Storm

The allowable street capacity for the major storm is calculated using the Manning's formula (Chapter 7) by first dividing the street cross section into the pavement area and sidewalk/grass area and then computing the individual flow contributions. The capacity calculations were performed for each street cross section and for various allowable flow depths and street slopes and plotted in Figure No. 6.3. A Manning's n-value of 0.016 for the pavement area and 0.025 for the sidewalk/grass area was used to determine the capacity. The reduction factor was also applied to reduce the allowable storm runoff in the street. The backslope from the curb was assumed to be 2 percent until reaching the street ROW limits. The maximum allowable depth at the gutter is 12 inches, but the street capacity is generally limited by the ROW.

6.5.3 Rural Streets

Rural streets are characterized by roadside ditches rather than curb and gutters for urban streets. The capacity is limited by the depth in the ditch and the maximum flow velocity. For the initial storm, the 5-year flow depth in the ditch shall not exceed 18 inches due to the geometry limitations (Figure No. 7.8). The capacity of the ditch is presented in Table No. 7.5 and is limited also by the maximum allowable Froude Number.

For the major storm, the capacity limitation is also controlled by the ditch geometry and the ROW of the street. The allowable capacity for the major storm is the same as for the initial storm unless additional ROW is provided and the backslope from the ditch will contain all the flows within the ROW.

NOMOGRAPH FOR FLOW IN TRIANGULAR GUTTERS

Figure No. 6.1

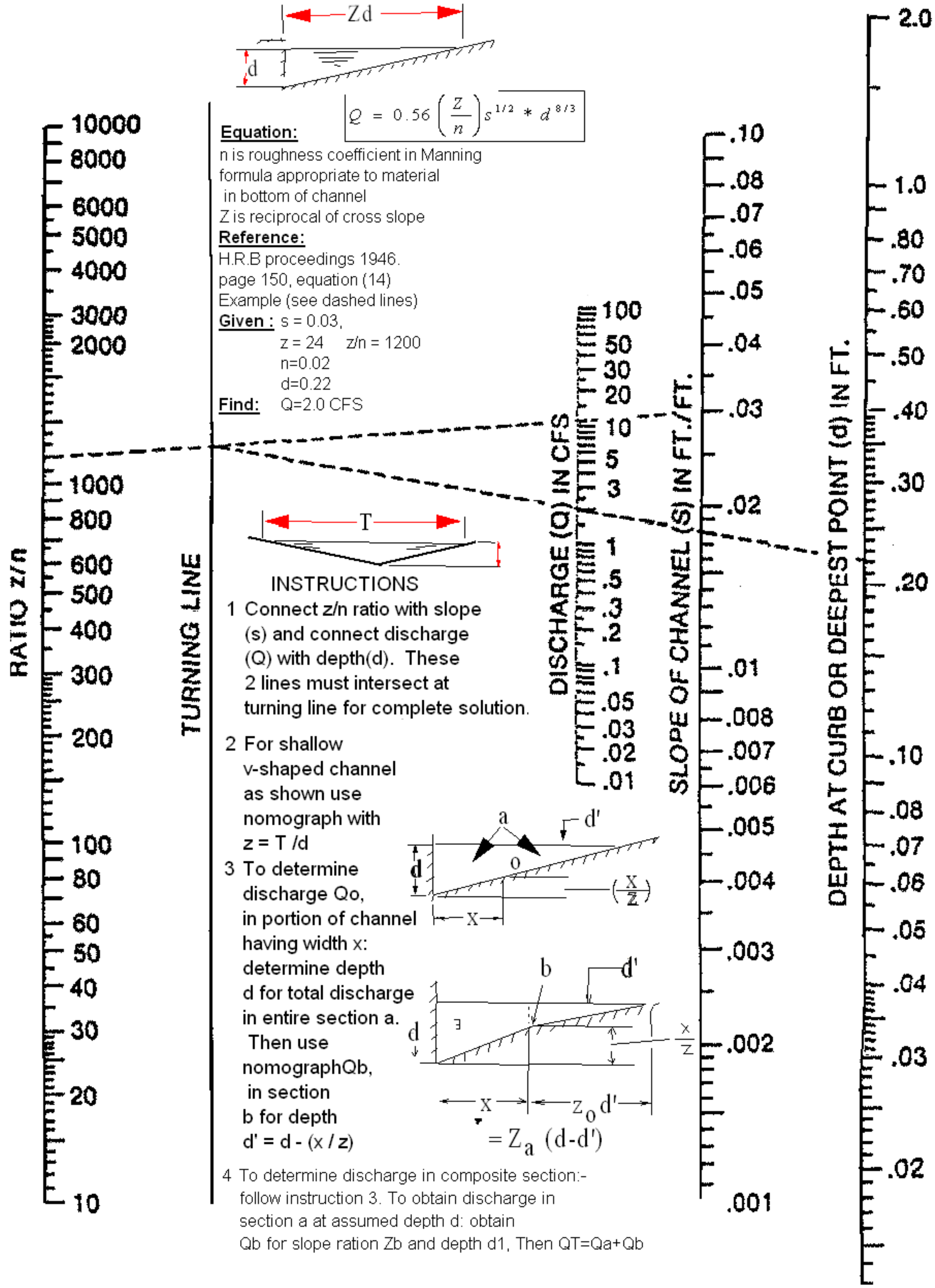
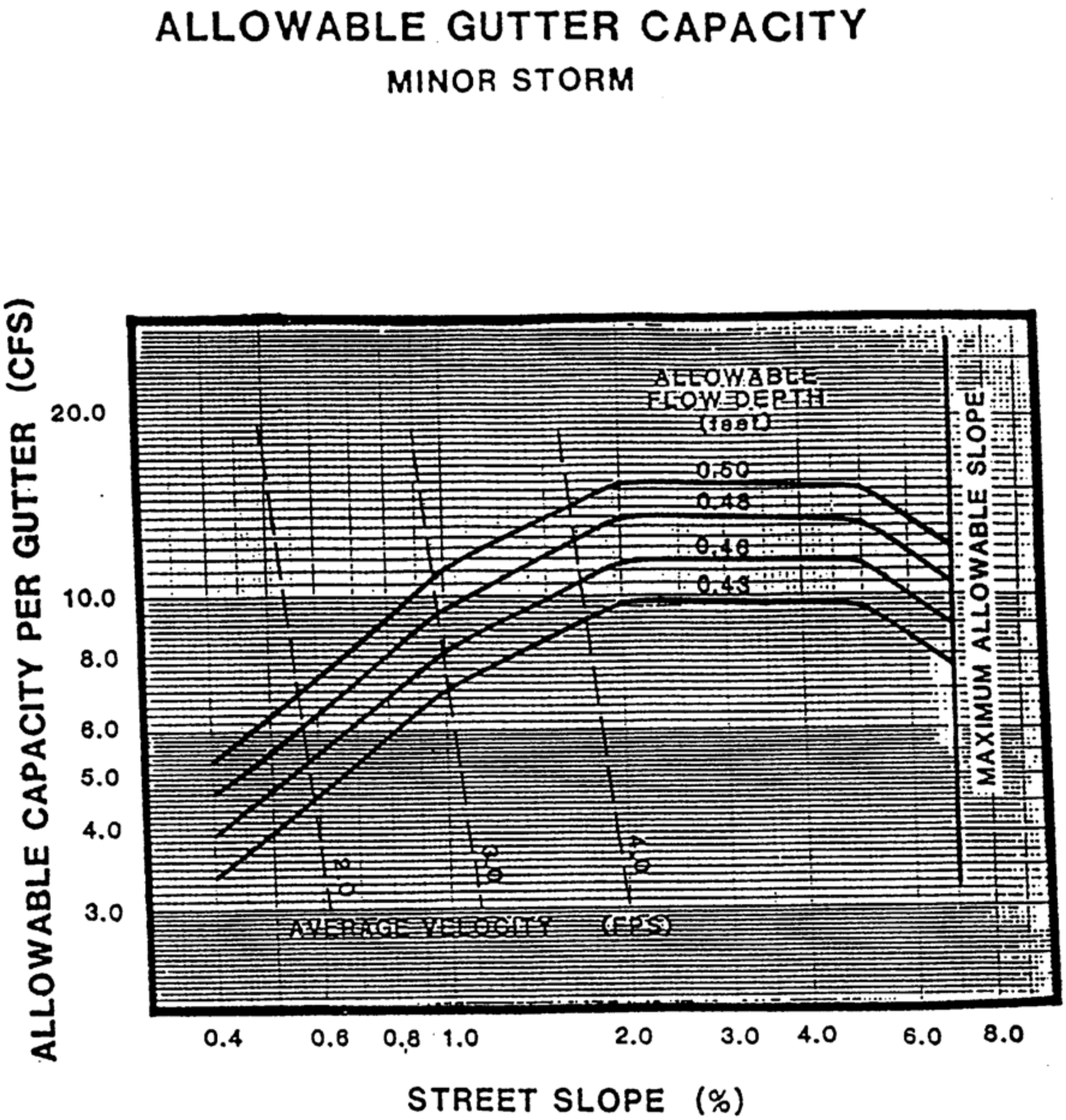


Figure No. 6.2



DESIGN CONDITIONS

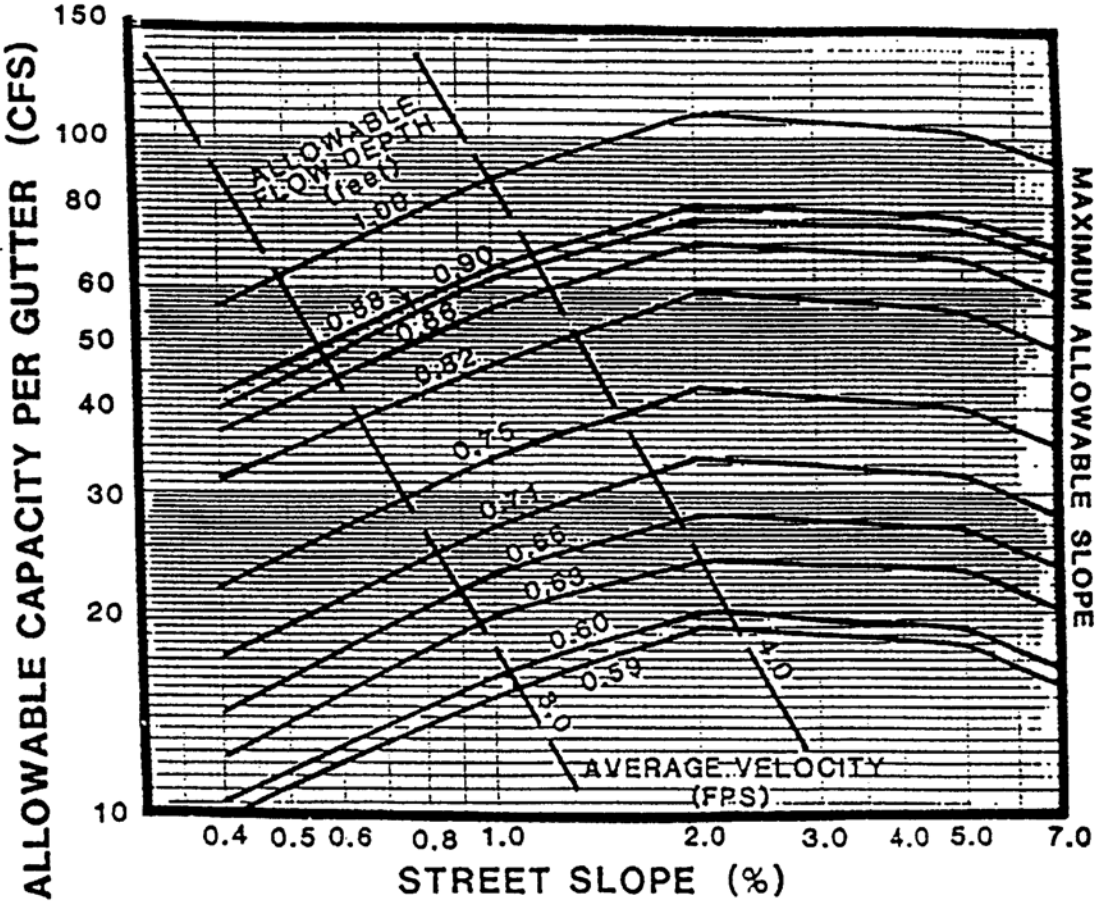
$$Q = F(0.56\frac{z}{n}S^{1/2}d^{8/3})$$

n = 0.016 for STREETS

Figure No. 6.3

ALLOWABLE GUTTER CAPACITY

MAJOR STORM



DESIGN CONDITIONS

$$Q = F(0.56\frac{z}{n}S^{1/2}d^{8/3})$$

n = 0.016 for STREETS

n = 0.025 for GRASS

CHAPTER 7
OPEN CHANNEL DESIGN

7.1 INTRODUCTION

This section presents the technical criteria for the hydraulic evaluation and hydraulic design of open channels including natural channels, grass, concrete or rock lined channels, and roadside ditches. A separate section is provided for the design of gradient control and energy dissipation structures, rock riprap, and vegetation for erosion protection. The individuals using this Manual are assumed to possess a working knowledge of hydraulics and have stormwater drainage evaluation experience. The user is encouraged to review the many referenced materials for additional information.

The open channel is generally the primary conveyance facility for the major drainage system (except for the roadside ditch). The selection of the type, capacity, and location for a major drainage channel will have a significant effect on the requirements for the minor system. If the historic or natural drainage path is selected for the open channel route, the construction costs for the major system can be minimized. Using the historic drainage route with minimum alteration of the existing natural channel is recommended for all drainage ways. In some instances, however, the land use of the property can be improved by relocating or straightening the natural drainage path, thereby improving the economic and development aspects of the project. Altering the channel alignment and shape may require the addition of grade control sections to control the resulting changes in flow velocities.

7.2 OPEN CHANNEL TYPES

7.2.1 General

The channels in the Bismarck area are defined as natural or artificial. Natural channels include all water courses that have developed by the erosion process such as Apple Creek, Hay Creek, Jackman Coulee, Tyler Coulee and their tributaries. Artificial channels are those constructed or significantly altered by human efforts including drainage ditches, roadside ditches, and grassed channels through the many subdivisions.

7.2.2 Natural Channels

The hydraulic properties of natural channels vary along channel reach and can be either controlled to the extent desired or altered to meet given requirements. The initial decision to be made regarding a natural channel is whether or not it can be protected from erosion due to high velocity flows, or protected from excessive silt deposition due to low velocities.

Many natural channels in urbanizing or urbanized areas have mild slopes, are reasonably stable, and are not in a state of serious degradation or aggregation. However, if a natural channel is to be used for carrying the additional storm runoff from an urbanized area, the altered nature of the runoff peaks and volumes from urban development can and will cause erosion. Therefore, a hydraulic analysis will be required for all natural channels in order to identify the potential for erosion. Some on-site modifications to the natural channel may also be required to assure a stabilized condition.

The investigations necessary to assure that a natural channel is adequate are different for every waterway. The design engineer must prepare cross sections of the channel, define the water surface profile for the minor and major design flood, investigate the bed and bank material to determine erosion tendencies, and study the bank slope stability of the channel under various flow conditions. Supercritical flow does not normally occur in natural channels, but calculations must be made to insure such flows do not occur.

7.2.3 ARTIFICIAL CHANNELS

7.2.3.1 Grass Lined Channels

Grass lined channels are the most desirable of the artificial channels. The grass will stabilize the bed and slopes of the channel, consolidate the soil mass of the bed, check the erosion on the channel surface, and control the movement of soil particles along the channel bottom. The channel storage, lower velocities, and the greenbelt multiple-use-benefits obtained create significant advantages over other artificial channels.

The presence of grass in channels creates turbulence which results in loss of energy and increased flow retardance. Therefore, the design engineer must give full consideration to sediment deposition and scour, as well as hydraulics. Unless existing development restricts the availability of right-of-way, only channels lined with grass will be considered acceptable.

7.2.3.2 Concrete Lined Channels

Concrete lined channels will be permitted only where ROW restrictions within an existing development prohibit grass lined channels. The lining must be designed to withstand the various forces and actions which tend to overtop the bank, deteriorate the lining, erode the soil beneath the lining, and erode unlined areas.

If the project constraints dictate the use of a concrete channel, such use shall be allowed only upon approval by the City Engineer.

7.2.3.3 Rock Lined Channels

Rock lined channels are constructed from ordinary riprap or wire enclosed riprap (i.e., gabions). The rock lining increases the turbulence resulting in a loss of energy and increased flow retardance. The rock lining also permits higher design velocities and therefore, a steeper design slope than for grass lined channels. Rock linings are also used for erosion control at culvert/storm sewer outlets, at sharp channel bends, at channel confluences, and at locally steepened channel sections.

If the project constraints dictate the use of a riprap or wire enclosed riprap lining, such use shall be allowed only upon approval of the City Engineer. Riprap for the purposes of local erosion control is permitted, subject to the criteria in this Manual.

7.2.3.4 Other Lining Types

A slope revetment mattress may consist of double layers of woven fabric forms placed on the slope to be protected and filled with concrete or grout. This type of forming system is simple, fast, and economical technique for the placement of concrete for slope protection both above and below the water without the need for dewatering. The performance characteristics and cost advantages make the process adaptable to stabilizing and protecting shorelines, levees, dikes, canals, holding basins, and similar erosion control projects.

These systems make use of the pressure injection of fluid fine-aggregate concrete into flexible fabric forms. Controlled bleeding of mixing water through the porous fabric produces all the desirable features of low water/cement ratios mortar - rapid stiffening, high strength, and exceptional durability.

For normal installations, the fabric forms, prefabricated to job specifications and dimensions, are simply spread over the terrain, which has received minimal grading. The fabric form is then pumped full of mortar. This same concept can be used where slide problems are caused by eroding of the toe of the slopes, and where access is difficult for placement of riprap. See Figure No. 7.1 for typical sections of mattresses.

There are several manufacturers of synthetic fabrics for erosion protection. Included in this category of channel lining are the products which consist of discrete blocks on a continuous fabric backing. The use of synthetic fabric for lining of channels within the City of Bismarck is restricted to areas where the ROW constraints prohibit the use of a grass lined section. Such use shall be allowed only upon approval of the City Engineer.

7.3 HYDRAULICS OF OPEN CHANNELS

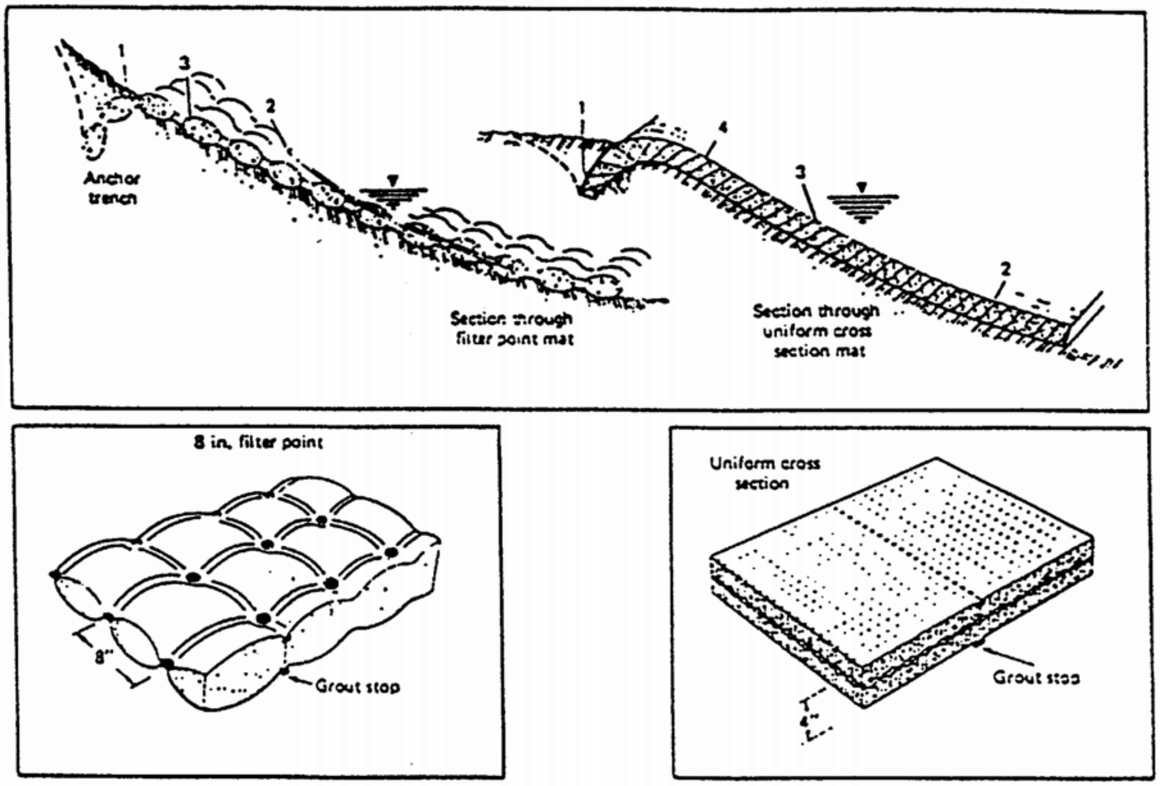
7.3.1 Introduction

An open channel is a conduit in which water flows with a free surface. The hydraulics of an open channel can be very complex, encompassing many different flow conditions from steady state uniform flow to unsteady, rapidly varied flow. Most of the problems in storm water drainage involve uniform, gradually varied (GVF) or rapidly varied (RVF) flow states. An example of these flow conditions is illustrated in Figure No. 7.2. The calculations for uniform and gradually varied flow are relatively straightforward and are based upon similar assumptions (i.e., parallel streamlines). Rapidly varied flow computation, (i.e., hydraulic jumps and flow over spillways) however, can be very complex and the solutions are generally empirical in nature.

Presented in this section are the basic equations and computational procedures for uniform, gradually varied, and rapidly varied flow. The user is encouraged to review the numerous hydraulics textbooks available for a more detailed discussion.

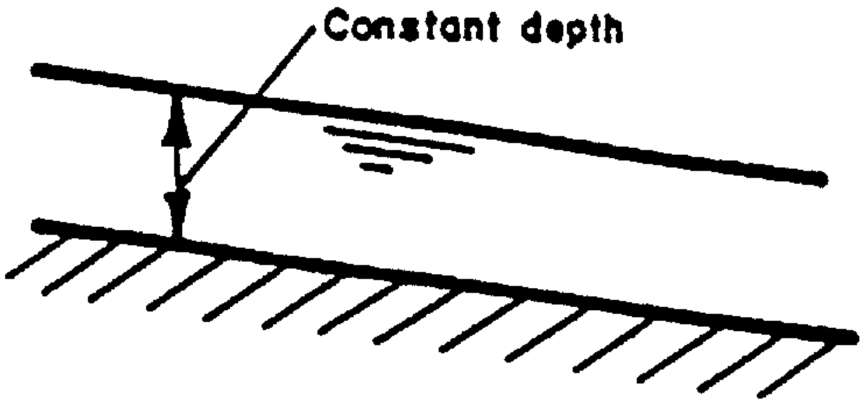
Typical Erosion Control Mattresses

Figure No. 7.1



NOTE: Numbers refer to sequence of mortar injection.

Flow Conditions



UNIFORM FLOW
Flow in a laboratory channel

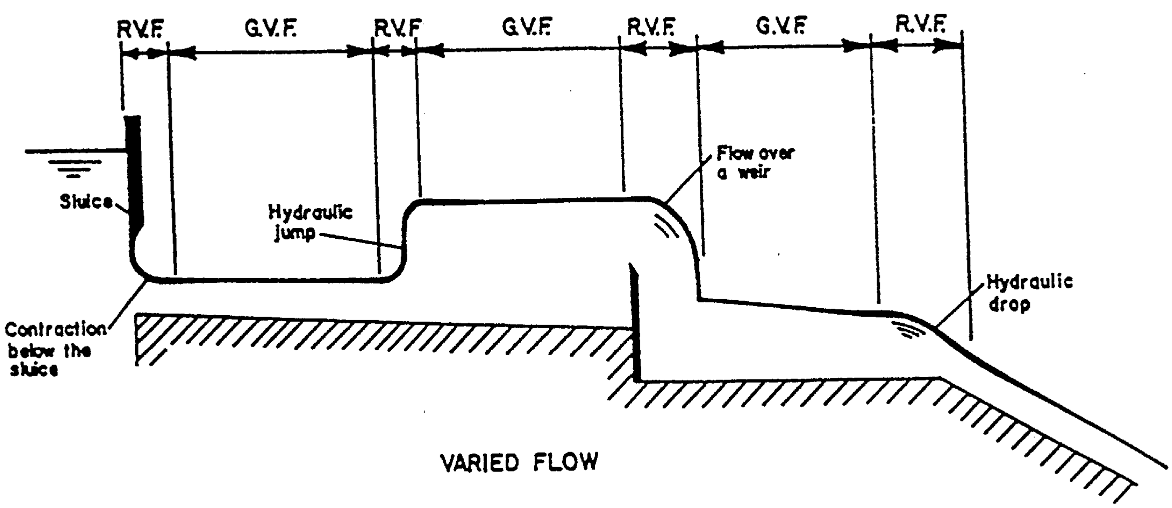


Figure No. 7.2

7.3.2 Uniform Flow

Open channel flow is said to be uniform if the depth of flow is the same at every section of the channel. For a given channel geometry, roughness, discharge and slope, there is only one possible depth for maintaining uniform flow; the normal depth. For a prismatic channel (i.e., uniform cross section) the water surface will be parallel to the channel bottom for uniform flow.

Uniform flow rarely occurs in nature and is difficult to achieve in a laboratory, because not all of the parameters remain exactly the same. However, channels are designed assuming uniform flow as an approximation, which is adequate for planning purposes.

The computation of uniform depth for the City of Bismarck shall be based upon the Manning's formula as follows:

$$Q = (1.49/n)\, AR^{2/3}\, S^{1/2} \qquad \text{(Equation 7.1)}$$

where: Q = Discharge in cfs
n = Roughness coefficient
A = Area in square feet
R = Hydraulic radius, A/P in feet
P = Wetted perimeter, feet
S = Slope of the energy grade line (EGL) in feet/feet

For prismatic channels, the EGL slope and the bottom slope are assumed to be the same.

For convenience, tables have been prepared using the Manning's formula for various uniform cross sections to compute uniform flow conditions given any four of the following parameters: D, b, Q, n, S. For trapezoidal sections, Table No. 7.1 can be used and for circular sections, Table No. 7.2 can be used. The use of the tables is illustrated by the following example:

Example No. 7.1: Uniform Flow for Trapezoidal Channel

Given: Channel bottom width b = 10-feet
Side slopes (Z) = 4:1
Slope (S) = 0.005
Discharge Q = 400 cfs

Find: Normal depth D, compute the following parameters:

Step 1: Estimate roughness coefficient, n = 0.032 for this example as a typical starting value.

Step 2: $Qn/b^{8/3}S^{1/2} = (400)\ (0.032)/\ (10)^{8/3} \times (0.005)^{1/2} = 0.39$

Step 3: Enter Table 7.1 with the value of 0.39 under the Column for Z = 4 and read the corresponding value of D/b, interpolate if necessary D/b = 0.328

Step 4: Compute normal depth D

$$D = (0.328)\ (b) = (0.328)\ (10)$$
$$D = 3.28 \text{ feet}$$

Step 5: Compute the velocity, velocity head and specific energy

Flow Area at D = 3.28, A = 75.8 ft^2

$V = Q/A = 400 \text{ cfs}/75.8 \text{ ft}^2$
$V = 5.28$ fps
Velocity head (h_v) = $V^2/2g = (5.28)^2/2(32.2) = 0.43$
Specific energy (e) = D + h_v = 3.28 + 0.43
e = 3.71 feet

Step 6: Compute hydraulic radius (R) and check assumed n-value using Figure No. 7.5.

R = 2.05 for D = 3.28
RV = (2.05) (5.28) = 10.8

From Figure No. 7.5 for Retardance C, read the value of 0.032, which is equal to the assumed value. If the computed n-value was different (i.e., $\pm$ 0.001), another n-value would be selected and steps 1 to 6 repeated. Also note that since the problem was to determine the normal depth, Retardance curve "C" was used. If the maximum velocity is to be found, curve "D" would be used (see Section 7.4).

The above parameters are important in the hydraulic evaluation of open channels as will be seen in subsequent sections. The design engineer should be aware that roughness conditions greater than those assumed will cause the same discharge to flow at a greater depth, or conversely that flow at the computed depth will result in less discharge. In addition, obstructions in the channels cause an increase in depth above normal and must be taken into account by providing for freeboard (see Section 7.4).

Uniform Flow for Trapezoidal Channels

Table No. 7.1 (a)

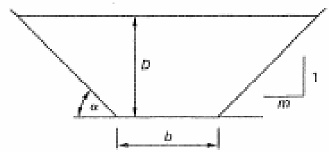
Conveyance Factor, K'

Symmetrical Rectangular,[a] Trapezoidal Open Channels

(use for determining Q or D when b is known)

[Customary U.S. Units[b,c]]

$$K' \text{ in } Q = K'(n)b^{\frac{8}{3}}\sqrt{S}$$



$z = 1/m$: slope of sides, horizontal to vertical

(α: side angle with respect to horizontal)

	m									
	0.0	0.25	0.5	0.75	1.0	1.5	2.0	2.5	3.0	4.0
$x = D/b$	90°	76.0°	63.4°	53.1°	45.0°	33.7°	26.6°	21.8°	18.4°	14.0°
0.01	0.00068	0.00068	0.00069	0.00069	0.00069	0.00069	0.00069	0.00069	0.00070	0.00070
0.02	0.00213	0.00215	0.00216	0.00217	0.00218	0.00220	0.00221	0.00222	0.00223	0.00225
0.03	0.00414	0.00419	0.00423	0.00426	0.00428	0.00433	0.00436	0.00439	0.00443	0.00449
0.04	0.00660	0.00670	0.00679	0.00685	0.00691	0.00700	0.00708	0.00716	0.00723	0.00736
0.05	0.00946	0.00964	0.00979	0.00991	0.01002	0.01019	0.01033	0.01047	0.01060	0.01086
0.06	0.0127	0.0130	0.0132	0.0134	0.0136	0.0138	0.0141	0.0148	0.0145	0.0150
0.07	0.0162	0.0166	0.0170	0.0173	0.0175	0.0180	0.0183	0.0187	0.0190	0.0197
0.08	0.0200	0.0206	0.0211	0.0215	0.0219	0.0225	0.0231	0.0236	0.0240	0.0250
0.09	0.0241	0.0249	0.0256	0.0262	0.0267	0.0275	0.0282	0.0289	0.0296	0.0310
0.10	0.0284	0.0294	0.0304	0.0311	0.0318	0.0329	0.0339	0.0348	0.0358	0.0376
0.11	0.0329	0.0343	0.0354	0.0364	0.0373	0.0387	0.0400	0.0413	0.0424	0.0448
0.12	0.0376	0.0393	0.0408	0.0420	0.0431	0.0450	0.0466	0.0482	0.0497	0.0527
0.13	0.0425	0.0446	0.0464	0.0480	0.0493	0.0516	0.0537	0.0556	0.0575	0.0613
0.14	0.0476	0.0502	0.0524	0.0542	0.0559	0.0587	0.0612	0.0636	0.0659	0.0706
0.15	0.0528	0.0559	0.0585	0.0608	0.0627	0.0662	0.0692	0.0721	0.0749	0.0805
0.16	0.0582	0.0619	0.0650	0.0676	0.0700	0.0740	0.0777	0.0811	0.0845	0.0912
0.17	0.0638	0.0680	0.0716	0.0748	0.0775	0.0823	0.0866	0.0907	0.0947	0.1026
0.18	0.0695	0.0744	0.0786	0.0822	0.0854	0.0910	0.0960	0.1008	0.1055	0.1148
0.19	0.0753	0.0809	0.0857	0.0899	0.0936	0.1001	0.1059	0.1115	0.1169	0.1277
0.20	0.0812	0.0876	0.0931	0.0979	0.1021	0.1096	0.1163	0.1227	0.1290	0.1414
0.22	0.0934	0.1015	0.109	0.115	0.120	0.130	0.139	0.147	0.155	0.171
0.24	0.1061	0.1161	0.125	0.133	0.140	0.152	0.163	0.173	0.184	0.204
0.26	0.119	0.131	0.142	0.162	0.160	0.175	0.189	0.202	0.215	0.241
0.28	0.132	0.147	0.160	0.172	0.182	0.201	0.217	0.234	0.249	0.281
0.30	0.146	0.163	0.179	0.193	0.205	0.228	0.248	0.267	0.287	0.324
0.32	0.160	0.180	0.199	0.215	0.230	0.256	0.281	0.304	0.327	0.371
0.34	0.174	0.198	0.219	0.238	0.256	0.287	0.316	0.343	0.370	0.423
0.36	0.189	0.216	0.241	0.263	0.283	0.319	0.353	0.385	0.416	0.478
0.38	0.203	0.234	0.263	0.288	0.312	0.353	0.392	0.429	0.465	0.537
0.40	0.218	0.253	0.286	0.315	0.341	0.389	0.434	0.476	0.518	0.600
0.42	0.233	0.273	0.309	0.342	0.373	0.427	0.478	0.526	0.574	0.668
0.44	0.248	0.293	0.334	0.371	0.405	0.467	0.525	0.580	0.633	0.740
0.46	0.264	0.313	0.359	0.401	0.439	0.509	0.574	0.636	0.696	0.816
0.48	0.279	0.334	0.385	0.432	0.474	0.553	0.625	0.695	0.763	0.897
0.50	0.295	0.355	0.412	0.463	0.511	0.598	0.679	0.757	0.833	0.983
0.55	0.335	0.410	0.482	0.548	0.609	0.722	0.826	0.926	1.025	1.22

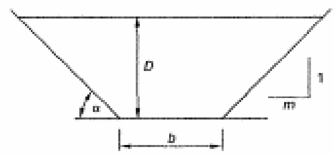
(continued)

Table No. 7.1 (b)

Conveyance Factor, K'
Symmetrical Rectangular,[a] Trapezoidal Open Channels
(use for determining Q or D when b is known)

[Customary U.S. Units[b,c]]

$$K' \text{ in } Q = K'(n) b^{\frac{8}{3}} \sqrt{S}$$



$z = 1/m$: slope of sides, horizontal to vertical
(α: side angle with respect to horizontal)

	m									
	0.0	0.25	0.5	0.75	1.0	1.5	2.0	2.5	3.0	4.0
$x = D/b$	90°	76.0°	63.4°	53.1°	45.0°	33.7°	26.6°	21.8°	18.4°	14.0°
0.60	0.375	0.468	0.557	0.640	0.717	0.858	0.990	1.117	1.24	1.49
0.70	0.457	0.592	0.722	0.844	0.959	1.17	1.37	1.56	1.75	2.12
0.80	0.542	0.725	0.906	1.078	1.24	1.54	1.83	2.10	2.37	2.90
0.90	0.628	0.869	1.11	1.34	1.56	1.98	2.36	2.74	3.11	3.83
1.00	0.714	1.022	1.33	1.64	1.93	2.47	2.99	3.48	3.97	4.93
1.20	0.891	1.36	1.85	2.33	2.79	3.67	4.51	5.32	6.11	7.67
1.40	1.07	1.74	2.45	3.16	3.85	6.17	6.42	7.64	8.84	11.2
1.60	1.25	2.16	3.14	4.14	5.12	6.99	8.78	10.52	12.2	15.6
1.80	1.43	2.62	3.93	5.28	6.60	9.15	11.6	14.0	16.3	20.9
2.00	1.61	3.12	4.82	6.58	8.32	11.7	14.9	18.1	21.2	27.3
2.25	1.84	3.81	6.09	8.46	10.8	15.4	19.8	24.1	28.4	36.7

[a] For rectangular channels, use the 0.0 (90°, vertical sides) column.
[b] Q = flow rate, ft^3/sec; D = depth of flow, ft; b = bottom width of channel, ft; S = geometric slope, ft/ft; n = Manning's roughness constant.
[c] For SI units (e.g., Q in m^3/s, and D and b in m), divide each table value by 1.486.

TABLE NO. 7.2

UNIFORM FLOW FOR CIRCULAR SECTIONS

d = depth of flow
D = diameter of pipe
A = area of flow
R = hydraulic radius
Q = discharge in cubic feet per second by Manning's formula
n = Manning's coefficient
S = slope of the channel bottom and of the water surface

$\frac{d}{D}$	$\frac{A}{D^2}$	$\frac{R}{D}$	$\frac{Qn}{D^{8/3}S^{1/2}}$	$\frac{Qn}{d^{8/3}S^{1/2}}$	$\frac{d}{D}$	$\frac{A}{D^2}$	$\frac{R}{D}$	$\frac{Qn}{D^{8/3}S^{1/2}}$	$\frac{Qn}{d^{8/3}S^{1/2}}$
0.01	0.0013	0.0066	0.00007	15.04	0.51	0.4027	0.2531	0.239	1.442
0.02	0.0037	0.0132	0.00031	10.57	0.52	0.4127	0.2562	0.247	1.415
0.03	0.0069	0.0197	0.00074	8.56	0.53	0.4227	0.2592	0.255	1.388
0.04	0.0105	0.0262	0.00138	7.38	0.54	0.4327	0.2621	0.263	1.362
0.05	0.0147	0.0325	0.00222	6.55	0.55	0.4426	0.2649	0.271	1.336
0.06	0.0192	0.0389	0.00328	5.95	0.56	0.4526	0.2676	0.279	1.311
0.07	0.0242	0.0451	0.00455	5.47	0.57	0.4625	0.2703	0.287	1.286
0.08	0.0294	0.0513	0.00604	5.09	0.58	0.4724	0.2728	0.295	1.262
0.09	0.0350	0.0575	0.00775	4.76	0.59	0.4822	0.2753	0.303	1.238
0.10	0.0409	0.0635	0.00967	4.49	0.60	0.4920	0.2776	0.311	1.215
0.11	0.0470	0.0695	0.01181	4.25	0.61	0.5018	0.2799	0.319	1.192
0.12	0.0534	0.0755	0.01417	4.04	0.62	0.5115	0.2821	0.327	1.170
0.13	0.0600	0.0813	0.01674	3.86	0.63	0.5212	0.2842	0.335	1.148
0.14	0.0668	0.0871	0.01952	3.69	0.64	0.5308	0.2862	0.343	1.126
0.15	0.0739	0.0929	0.0225	3.54	0.65	0.5404	0.2882	0.350	1.105
0.16	0.0811	0.0985	0.0257	3.41	0.66	0.5499	0.2900	0.358	1.084
0.17	0.0885	0.1042	0.0291	3.28	0.67	0.5594	0.2917	0.366	1.064
0.18	0.0961	0.1097	0.0327	3.17	0.68	0.5687	0.2933	0.373	1.044
0.19	0.1039	0.1152	0.0365	3.06	0.69	0.5780	0.2948	0.380	1.024
0.20	0.1118	0.1206	0.0406	2.96	0.70	0.5872	0.2962	0.388	1.004
0.21	0.1199	0.1259	0.0448	2.87	0.71	0.5964	0.2975	0.395	0.985
0.22	0.1281	0.1312	0.0492	2.79	0.72	0.6054	0.2987	0.402	0.965
0.23	0.1365	0.1364	0.0537	2.71	0.73	0.6143	0.2998	0.409	0.947
0.24	0.1449	0.1416	0.0585	2.63	0.74	0.6231	0.3008	0.416	0.928
0.25	0.1535	0.1466	0.0634	2.56	0.75	0.6319	0.3017	0.422	0.910
0.26	0.1623	0.1516	0.0686	2.49	0.76	0.6405	0.3024	0.429	0.891
0.27	0.1711	0.1566	0.0739	2.42	0.77	0.6489	0.3031	0.435	0.873
0.28	0.1800	0.1614	0.0793	2.36	0.78	0.6573	0.3036	0.441	0.856
0.29	0.1890	0.1662	0.0849	2.30	0.79	0.6655	0.3039	0.447	0.838
0.30	0.1982	0.1709	0.0907	2.25	0.80	0.6736	0.3042	0.453	0.821
0.31	0.2074	0.1756	0.0966	2.20	0.81	0.6815	0.3043	0.458	0.804
0.32	0.2167	0.1802	0.1027	2.14	0.82	0.6893	0.3043	0.463	0.787
0.33	0.2260	0.1847	0.1089	2.09	0.83	0.6969	0.3041	0.468	0.770
0.34	0.2355	0.1891	0.1153	2.05	0.84	0.7043	0.3038	0.473	0.753
0.35	0.2450	0.1935	0.1218	2.00	0.85	0.7115	0.3033	0.477	0.736
0.36	0.2546	0.1978	0.1284	1.958	0.86	0.7186	0.3026	0.481	0.720
0.37	0.2642	0.2020	0.1351	1.915	0.87	0.7254	0.3018	0.485	0.703
0.38	0.2739	0.2062	0.1420	1.875	0.88	0.7320	0.3007	0.488	0.687
0.39	0.2836	0.2102	0.1490	1.835	0.89	0.7384	0.2995	0.491	0.670
0.40	0.2934	0.2142	0.1561	1.797	0.90	0.7445	0.2980	0.494	0.654
0.41	0.3032	0.2182	0.1633	1.760	0.91	0.7504	0.2963	0.496	0.637
0.42	0.3130	0.2220	0.1705	1.724	0.92	0.7560	0.2944	0.497	0.621
0.43	0.3229	0.2258	0.1779	1.689	0.93	0.7612	0.2921	0.498	0.604
0.44	0.3328	0.2295	0.1854	1.665	0.94	0.7662	0.2895	0.498	0.588
0.45	0.3428	0.2331	0.1929	1.622	0.95	0.7707	0.2865	0.498	0.571
0.46	0.3527	0.2366	0.201	1.590	0.96	0.7749	0.2829	0.496	0.553
0.47	0.3627	0.2401	0.208	1.559	0.97	0.7785	0.2787	0.494	0.535
0.48	0.3727	0.2435	0.216	1.530	0.98	0.7817	0.2735	0.489	0.517
0.49	0.3827	0.2468	0.224	1.500	0.99	0.7841	0.2666	0.483	0.496
0.50	0.3927	0.2500	0.232	1.471	1.00	0.7854	0.2500	0.463	0.463

7.3.3 Uniform/Critical Flow

The critical state of flow through a channel is characterized by several important conditions:

1. Specific energy; E, is a minimum for a given discharge
2. The discharge is a maximum for a given specific energy
3. The specific force is a minimum for a given discharge
4. The velocity head is equal to half the hydraulic depth in a channel of small slope
5. The Froude Number is equal to 1.0

If the critical state of flow exists throughout an entire reach, the channel flow is critical and the channel slope is at critical slope S_c. A slope less than S_c will cause subcritical flow. A slope greater than S_c will cause supercritical flow. A flow at or near the critical state is unstable because minor changes in specific energy, such as from channel debris, will cause a major change in depth.

The criteria of minimum specific energy for critical flow results in the definition of the Froude Number (F) as follows:

$$F = V/\sqrt{gD} \qquad \text{(Equation 7.2)}$$

Where:

F	=	Froude Number
V	=	Velocity (fps)
g	=	Acceleration of gravity (ft/sec^2)
D	=	Hydraulic Depth (ft) = A/T
A	=	Channel flow area (ft^2)
T	=	Top width of flow area (ft)

When F is equal to 1.0, the flow is critical. The Froude Number should be calculated for the design of all open channels to check the flow state.

The computation of the critical flow state for trapezoidal and circular section can be performed with the use of Figure No. 7.3. The use of Figure No. 7.3 is illustrated by the following example:

Example No. 7.2: Critical Flow Computation for Trapezoidal Channel

Given: Channel conditions for Example No. 7.1

Find: Froude Number and critical flow state

Step 1: Calculate Froude Number

D = A/T = 75.8/36.2 = 2.09 feet

$F = V/\sqrt{gD} = 5.28/\sqrt{(32.2)(2.09)} = 0.64$

Since F is less than 1.0, the flow is subcritical

Step 2: Calculate section factor Z

$$Z = Q/\sqrt{g} = 400\,\text{cfs}/\sqrt{32.2} = 70.5$$

Step 3: $Z/b^{5/2} = 70.5/10^{5/2} = 0.22$

Step 4: Enter Figure No. 7.3 with $Z/b^{5/2} = 0.22$ for a channel side slope (z = 4) and read the value for y/b

y/b = 0.26

Step 5: Calculate critical depth

y = (0.26) (b) = (0.26) (10) = 2.6 feet

The remaining channel parameters (i.e., S, A, V, h_v, E) can be computed from the channel geometry and using Table No. 7.1

S	=	0.0166 ft/ft
A	=	53.0 ft^2
V	=	7.54 fps
h_v	=	0.88 ft
E	=	3.48

Note that the specific energy, E, for critical flow is less than the specific energy at normal depth.

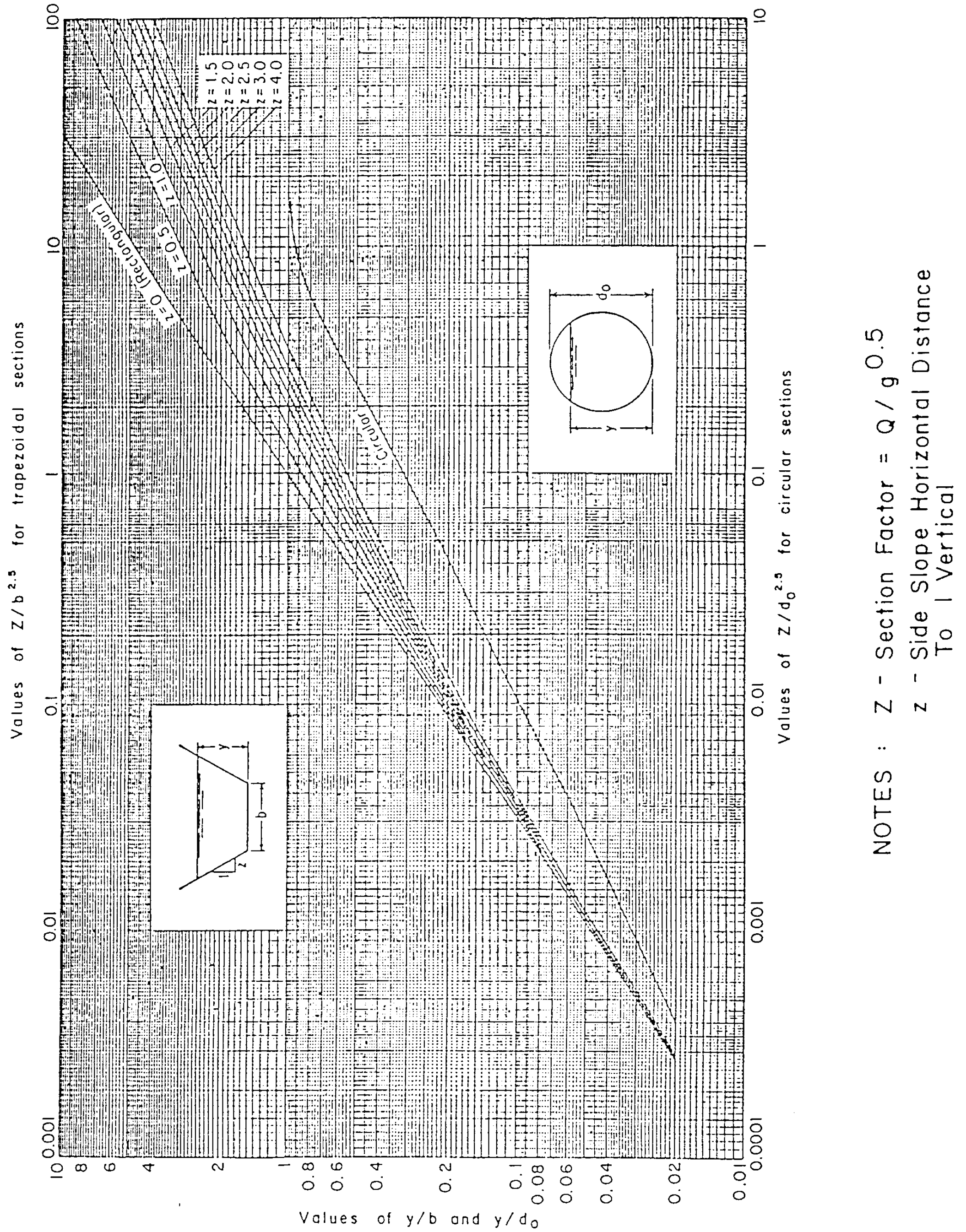
7.3.4 Gradually Varied Flow

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 must be computed using backwater techniques

Backwater computations can be made using the method presented in Chow. Many computer programs are available for computation of backwater curves. The most widely used program is HEC-2, Water Surface Profiles, developed by the U.S. Army Corps of Engineers. It is the program recommended for floodwater profile computations in the City of Bismarck. This program will compute water-surface profiles for natural and manmade channels using the Standard Step Method

Critical Depth for Trapezoidal & Circular Sections
Figure No. 7.3

CRITICAL DEPTH FOR TRAPEZOIDAL AND CIRCULAR SECTIONS



REFERENCE: Chow, V.T., Open-Channel Hydraulics
McGraw Hill Book Company 1959

For prismatic channels, the backwater calculation can be computed manually using the Direct Step Method. For an irregular non-uniform channel, the Standard Step Method is used, which is a more tedious iterative process. The use of HEC-2 is recommended for non-uniform channel analysis. The Direct Step Method can be used for prismatic channels in the City of Bismarck and is illustrated by the following example:

<u>Example No. 7.3</u>: <u>Direct Step Backwater Calculation</u>

<u>Given</u>: Channel condition in Examples No. 7.1 and 7.2, a culvert with a headwater depth of 5.0-feet at the entrance.

<u>Find</u>: Water the surface profile from the culvert upstream to where the flow depth reaches normal depth.

Step 1: Set up Table No. 7.3. Fill in the parameters at the top of the table.

Step 2: Starting with y = 5.00 feet (culvert headwater) compute Columns 2, 3, and 4 from the channel geometry.

TABLE NO. 7.3
FLOW PROFILE COMPUTATION
(Direct Step Method)
<u>Direct Step Backwater Calculation Example 10</u>

Q = 400, n = .032, S_o = 0.005, % = 1.0, y_c= 2.60, y_n= 3.28											
y (1)	A (2)	R (3)	V (4)	%V^2/2g (5)	E (6)	)E (7)	S_f (8)	S_f (9)	S_o-S_f (10)	)X (11)	X (12)
5.00	150.0	2.93	2.67	0.111	5.111	--	0.0007871	--	--	--	--
4.80	140.2	2.83	2.85	0.126	4.426	0.185	0.0007392	0.0008632	0.0041368	45	45
4.60	130.6	2.73	3.06	0.146	4.746	0.180	0.00011358	0.0010375	0.00329625	45	90
4.40	121.4	2.62	3.29	0.168	4.568	0.178	0.0013867	0.0012613	0.0037388	48	178
4.20	112.6	2.52	3.55	0.196	4.396	0.172	0.0017004	0.0015436	0.0034565	50	188
4.00	104.0	2.42	3.85	0.230	4.230	0.166	0.0021105	0.0019055	0.0030946	54	242
3.80	95.8	2.32	4.18	0.271	4.071	0.159	0.0026315	0.0023710	0.0026290	60	302
3.60	87.8	2.21	4.56	0.323	3.923	0.148	0.0033407	0.0029861	0.0020139	73	375
3.40	80.2	2.11	4.99	0.387	3.787	0.136	0.0042545	0.0037976	0.0012024	113	488
3.30	76.6	2.06	5.22	0.423	3.723	0.064	0.0048067	0.0045306	0.0004694	136	624

$$S_f = \frac{n^2 V^2}{2.22 R^{4/3}} \qquad \Delta X = \frac{\Delta E}{S_o - S_f}$$

Step 3: Compute Columns 5 through 12 as follows:

Col. 5: Velocity head, (alpha assumed equal to 1.0).

Col. 6: Specific energy in feet obtained by adding Column 1 and 5.

Col. 7: Change in specific energy in feet equal to the difference between the E value in Column 6 and that of the previous step (not applicable for first row).

Col. 8: Friction slope computed using the equation at the bottom of the table.

Col. 9: Average friction slope between this row and previous row (not applicable for the first row).

Col. 10: Difference between the bottom slope and average friction slope of Column 9 (not applicable for first row).

Col. 11: Length of the reach in feet between consecutive rows, computed by the equation at the bottom of the table (not applicable for the first row).

Col. 12: Accumulated distance from the starting point.

Step 4: Decrease Column 1 depth (Y) by small amount (i.e., 2 to 4%) and enter in Column 1 row 2. Repeat steps 2 and 3 for the new depth.

Step 5: Continue steps 2, 3, and 4 until the depth in Column 1 equals the normal depth ($\pm$ 0.05 feet). The plot of the depth (Column 1) versus distance (Column 23) is the water surface profile.

7.3.5 Rapidly Varied Flow

Rapidly varied flow (RVF) is characterized by very pronounced curvature of the streamlines. The change in curvature may become so abrupt that the flow profile is virtually broken, resulting in a state of high turbulence. Whereas there are several mathematical solutions to some cases of RVF, design engineers have generally relied on empirical solutions of specific problems. The two cases of RVF (weir flow and hydraulic jump) occurring commonly in storm drainage will be discussed in the next section.

7.3.6 Gradient Control and Energy Dissipation

7.3.6.1 Weir Flow

The common use of weirs in storm drainage analysis is for spillway outlets in detention ponds (Chapter 12 Detention). The general form of the equation for horizontal crested weirs is:

$$Q=CLH^{3/2} \qquad \text{(Equation 7.3)}$$

where:

Q	=	discharge (cfs)
C	=	weir coefficient
L	=	horizontal length of weir crest (feet)
H	=	total energy head (feet)

Another common weir is the v-notch, whose equation is as follows:

$$Q=2.5 \tan(\varnothing/2)\, H^{5/2} \qquad \text{(Equation 7.4)}$$

Where $\varnothing$ = angle of the notch at the apex (degrees)

When designing or evaluating weir flow conditions, the effects of submergence must be considered. A single check on submergence can be made by comparing the tailwater to the headwater depth as illustrated in the figure on Table No. 7.4. The coefficients to be used in Equation 7.3 are also listed on Table No. 7.4.

7.3.6.2 Hydraulic Jump

In urban hydraulics, the hydraulic jump may occur at grade control structures (i.e., check drops), inside the storm sewers or concrete box culverts, or at the outlet of an emergency spillway for detention ponds. The evaluation of hydraulic jumps is important since there is a loss of energy and the occurrence of erosive forces associated with a jump. For hard-lined facilities, such as pipes or concrete channels, the forces and the change in energy can affect the structural stability or the hydraulic capacity of the conveyance facility. For grass lined channels, the erosive forces must be controlled or serious damages will result. The control is usually obtained by check drops or grade control structures which confine the erosive forces to a riprap protected area.

The analysis of the jump inside of storm sewers is approximate due to the lack of data for circular, elliptical or arch sections. The jump can be approximately located by intersecting the energy grade line of the supercritical and subcritical flow reaches. The primary concerns are: (1) can the pipe withstand the forces which may separate the joint or damage the pipe wall, and (2) will the jump affect the hydraulic characteristics. Since the mainline storm sewers in Bismarck are currently restricted to concrete pipe and the velocities are limited to 15 fps (Chapter 8), then the hydraulic jump need only to be located and the impact on the pipe capacity determined for storm sewer analysis as discussed above.

The effect on pipe capacity can be determined by evaluating the energy grade line taking into account the energy lost by the jump. In general, for Froude Numbers less than 2.0, the loss of energy is less than 10 percent. For long box culverts, with a concrete bottom, the concerns of the jump are the same as for storm sewers. However, the jump can be adequately defined for box culverts/sewers and for spillways using the jump characteristics of rectangular sections. A detailed evaluation of the hydraulic jump is beyond the scope of this Manual and the user is referred to many available references for computational procedures. The calculations are to be included when necessary with the required Storm Water Management Plan submittals. (see Chapter 2).

The hydraulic jump conditions at vertical check drops have been defined using experimental data. The aerated free-falling nappe in a vertical check drop will reverse the curvature and turn smoothly into supercritical flow on the apron (see Figure No. 7.4) which may form a hydraulic jump downstream. Using the experimental data, the flow geometry can be described by functions of the drop number (D_n) defined as:

$$D = (q^2 / gh^3) \qquad \text{(Equation 7.5)}$$

Where q is the discharge per unit width of the crest of overfall (cfs/ft), g is the acceleration of gravity, and h is the height of drop (feet). The functions are:

$$L_d/h = 4.3\ D_n^{\ 0.27} \qquad \text{(Equation 7.6)}$$
$$d_p/h = 1.00\ D_n^{\ 0.22} \qquad \text{(Equation 7.7)}$$
$$d_1/h = 0.54\ D_n^{\ 0.425} \qquad \text{(Equation 7.8)}$$
$$d_2/h = 1.66\ D_n^{\ 0.27} \qquad \text{(Equation 7.9)}$$

Where L_d is the drop length (the distance from the drop wall to the position of the depth d_1, y_p is the pool depth under the nappe, d_1 is the depth at the toe of the nappe or the beginning of the hydraulic jump, and d_2 is the tailwater depth sequent to d_1. L is the length of the hydraulic jump and may be determined as outlined for stilling basins. From these equations, the drop length and design tailwater depth may be determined. The above equations are contingent upon the length of the spillway crest being approximately the same width as the approach channel.

7.4 DESIGN STANDARDS

7.4.1 Introduction

The design standards for open channels cannot be presented in a step by step fashion because of the wide range of options available to the engineer. Certain planning and conceptual criteria are particularly useful in the preliminary design of a channel. Those criteria, which have the greatest effect on the performance and cost of the channel, are discussed below.

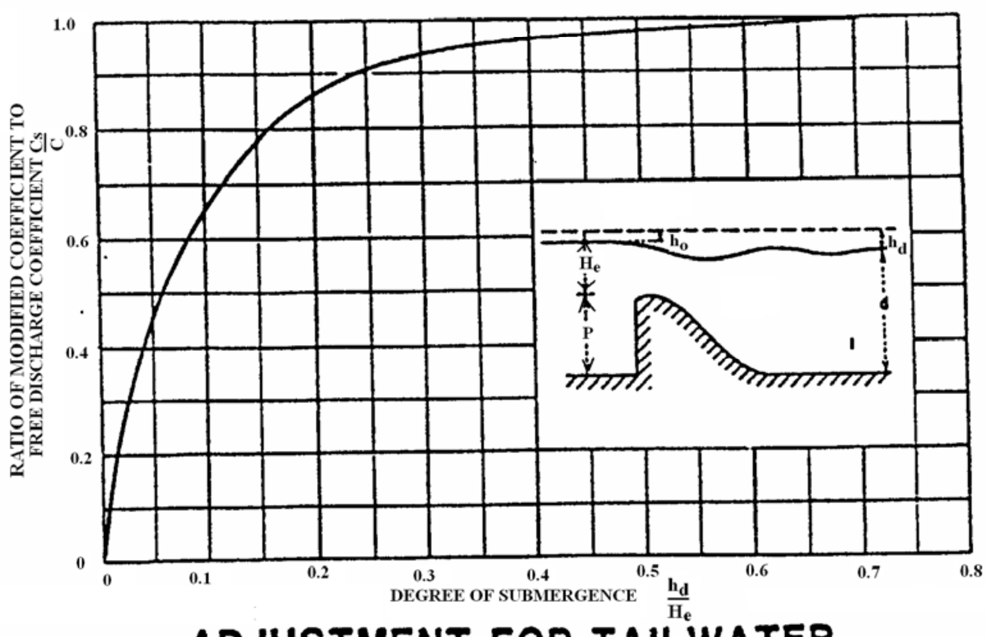
7.4.2 Natural Channels

The design criteria and evaluation techniques for natural channels are:

1. The channel and overbank areas shall have adequate capacity for the 100-year storm runoff.

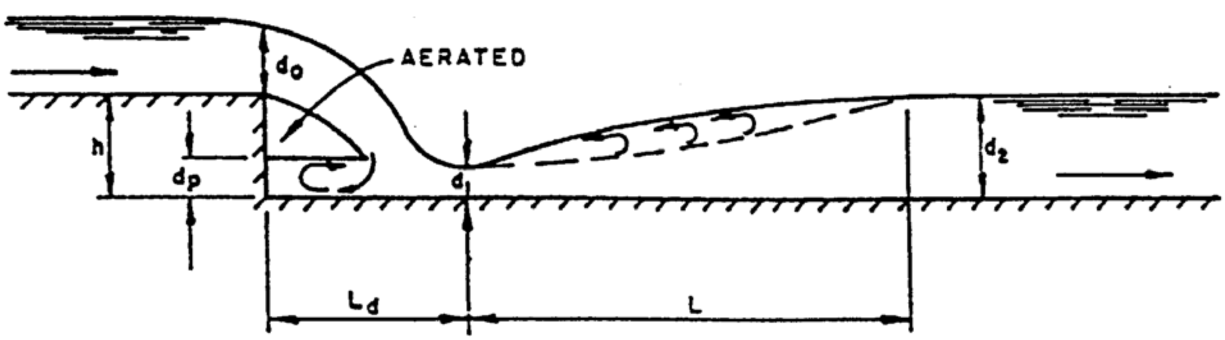
Weir Flow Coefficients
Table No. 7.4

SHAPE	COEFFICIENT	COMMENTS
Sharp Crested	-	
Projection Ratio (H/P = 0.4)	3.4	H ≥1.0
Projection Ratio (H/P = 2.0)	4.0	H ≥1.0
Broad Crested	-	
w/Sharp U/S Corner	2.6	Minimum Value
w/Rounded U/S Corner	3.1	Critical Depth
Triangular Section	-	
A) Vertical U/S Slope	-	
1:1 D/S Slope	3.8	H ≥0.7
4:1 D/S Slope	3.2	H ≥0.7
10:1 D/S Slope	2.9	H ≥0.7
B) 1:1 U/S Slope	-	
1:1 D/S Slope	3.8	H ≥1.0
3:1 D/S Slope	3.5	
Trapezoidal Section		
1:1 U/S Slope, 2:1 D/S Slope	3.4	H ≥1.0
2:1 U/S Slope, 2:1 D/S Slope	3.4	H ≥1.0
Road Crossings		
Gravel	3.0	H ≥1.0
Paved	3.1	H ≥1.0



ADJUSTMENT FOR TAILWATER

Flow Geometry of a Straight Drop Spillway
Figure No. 7.4



2. Natural channel segments which have a Froude Number greater than 0.95 for the 100-year flood peak shall be protected from erosion

3. The water surface profiles shall be defined so that the floodplain can be zoned and protected.

4. Filling of the flood fringe reduces valuable channel storage capacity and tends to increase downstream runoff peaks. Filling of the flood fringe is subject to the restriction of floodplain regulations.

5. Roughness factors (n), which are representative of unmaintained channel conditions, shall be used for the analysis of water surface profiles.

6. Roughness factors (n), which are representative of maintained channel conditions, shall be used to determine velocity limitations.

7. Erosion control structures, such as riprap check drops or check dams, may be required to control flow velocities, including the initial storm runoff.

8. Plan and profile drawings of the floodplain shall be prepared. Appropriate allowances for future bridges or culverts, which can raise the water surface profile and cause the floodplain to be extended, shall be included in the analysis.

With most natural waterways, grade control structures should be constructed at regular intervals to decrease the thalweg slope and to control erosion. However, these channels should be left in as near a natural condition as possible. For that reason extensive modifications should not be undertaken unless they are found to be necessary to avoid excessive erosion with subsequent deposition downstream. Also, modification of the channel within the normal high water line will require a US Army Corps of Engineers Section 404 permit for many of the streams in the City of Bismarck.

The usual rules of freeboard depth, curvature, and other guidelines which are applicable to artificial channels do not necessarily apply to natural channels. All structures constructed along the channel shall be elevated to a minimum of 1-foot above the 100-year water surface. Consideration should be given to the potential for water surface variability due to ice and ice jams. There are significant advantages which may occur if the design engineer incorporates into his planning the overtopping of the channel and localized flooding of adjacent areas which are laid out and developed for the purpose of being inundated during the major storm runoff. The freeboard criteria can be used to advantage in gaging the adequacy of a natural channel for future changes in runoff.

If a natural channel is to be utilized as a drainage way for a development (i.e., historic 100-year flood peaks in excess of 100 cfs), then the applicant shall meet with the City Engineer to discuss the concept and to obtain the requirements for planning and design documentation. Approval of the concept and design will be made in accordance with the requirements of Chapter 2.

7.4.3 Artificial Channels

7.4.3.1 Grass Lined Channels

Key parameters in grass lined channel design include velocity, slopes, roughness coefficients, depth, freeboard, curvature, cross section shape, and lining materials. Other factors such as water surface profile computation, erosion control, drop structures, and transitions also play an important role. A discussion of these parameters is presented below.

1. Flow Velocity and Capacity

   The maximum normal depth velocity for the 100-year flood peak shall not exceed 7.0 feet per second for grass lined channels, except in sandy soil where the maximum velocity shall not exceed 5.0 feet per second. The Froude Number (turbulence factor) shall be less than 0.8 for all grass lined channels. Grass lined channels having a Froude Number greater than 0.8 shall not be permitted. The minimum velocity, wherever possible, shall be greater than 2.0-feet per second for the initial storm runoff. All grass lined channels shall be designed to convey the 100-year flood.

2. Longitudinal Channel Slopes

   Grass lined channels normally will have slopes of 0.2 percent to 0.6 percent. Where the natural topography is steeper than desirable, drop structures shall be utilized to maintain design velocities (see Section 7.4.4)

3. Freeboard

   Except where localized overflow in certain areas is desirable for additional ponding benefits or other reasons, the freeboard shall be:

   $$H_{FB} = 1.0 + V^2/2g \qquad \text{(Equation 7.10)}$$

   where:
   H_{FB} = freeboard height (feet)
   V = average channel velocity (fps)
   g = acceleration of gravity = 32.2 ft/sec^2

   The minimum freeboard shall be 1.5-feet above the computed water surface elevation. Freeboard shall not be obtained by the construction of levees.

   An approximation of the superelevation, h (ft), at a channel bend with velocity V (fps), a centerline radius of curvature r_c (ft), and top width of channel, T_w (ft), can be obtained from the following equation:

   $$h = V^2\,T_w / g r_c \qquad \text{(Equation 7.11)}$$

   The freeboard shall be measured above the superelevation water surface.

4. Curvature

The center line curvature shall have a radius twice the top width of the design flow, but not less than 100-feet.

5. Roughness Coefficient

The variation of Manning's "n" with retardance and the product of mean velocity and hydraulic radius as presented in Figure No. 7.5 shall be used in the capacity computation. Refer to Example No. 7.1 for illustration of the use of Figure No. 7.5.

Retardance curve C shall be used to determine the channel capacity, since a mature channel (i.e., substantial vegetation with minimal pervious maintenance) will have a higher Manning's "n" value. However, a recently constructed channel will have minimal vegetation and the retardance will be less than the mature channel. Therefore, retardance curve D shall be used to determine the limiting velocity in a channel.

6. Cross Sections

The channel shape may be almost any type suitable to the location and to the environmental conditions. Often the shape can be chosen to suit open space and recreational needs (refer to Figures No. 7.6 and 7.7). However, limitations within which the design must fall for the major storm design flow include:

a. Trickle Channel

The base flow shall be carried in a trickle channel. The minimum capacity shall be 1.0 percent to 3.0 percent of the 100-year flow, but not less than 1 cfs. Trickle channel shall be constructed of concrete or other approved materials to minimize erosion, to facilitate maintenance and to aesthetically blend with the adjacent vegetation and soils.

b. Bottom Width

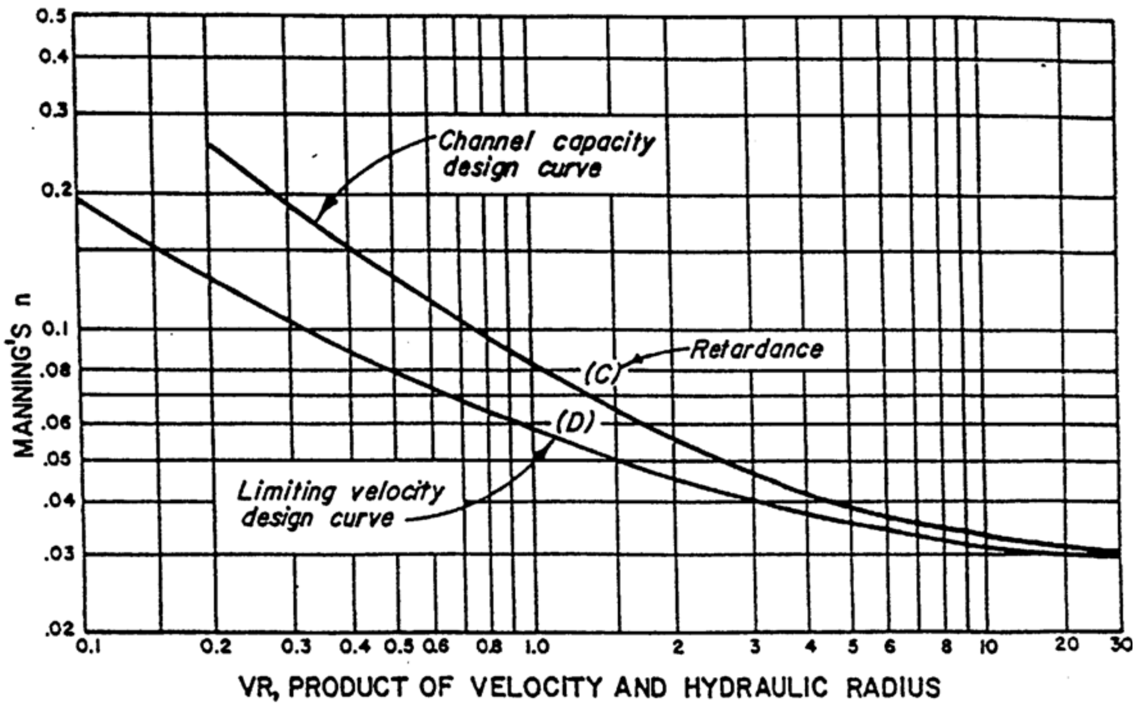
The minimum bottom width shall be consistent with the maximum depth, and velocity criteria. The minimum width shall be 4-feet to accommodate the trickle channel.

c. Right-of-Way Width

The minimum right-of-way width for major drainage ways shall include freeboard and a twelve foot (12') wide maintenance access.

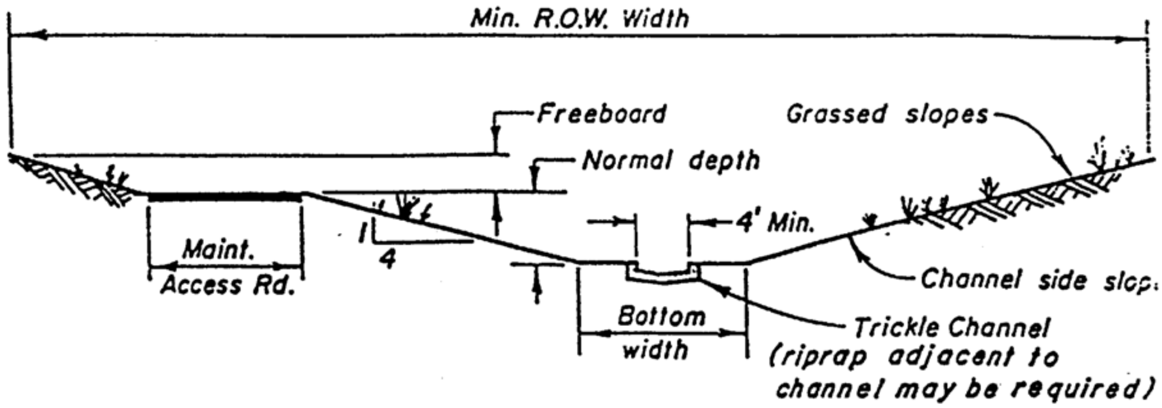
Roughness Coefficient for Grassed Channels

Figure No. 7.5



From "Handbook of Channel Design For Soil and Water Conservation," U.S. Department of Agriculture, Soils Conservation Service, No. SCS-TP-61 March, 1947, Rev. June, 1954

Typical Grassed Lined Channel Section
Type A
Figure No. 7.6



NOTES:

1. Bottom Width: Consistent with maximum allowable depth and velocity requirements, shall not be less trickle channel width.

2. Trickle Channel: Minimum capacity to be 1% to 3% of 100-year flow but not less than 1 cfs. Channel to be constructed of concrete or other approved materials.

3. Normal Depth: Normal depth at 100-year flow shall be such that the product of hydraulic depth (A/T) and the 100-year flow velocity (V_{max} = 7 fps) be less than 35 cfs/ft.

4. Freeboard: Freeboard to be a minimum of 1-foot

5. Maintenance/Access Road: Minimum width to be 12-feet.

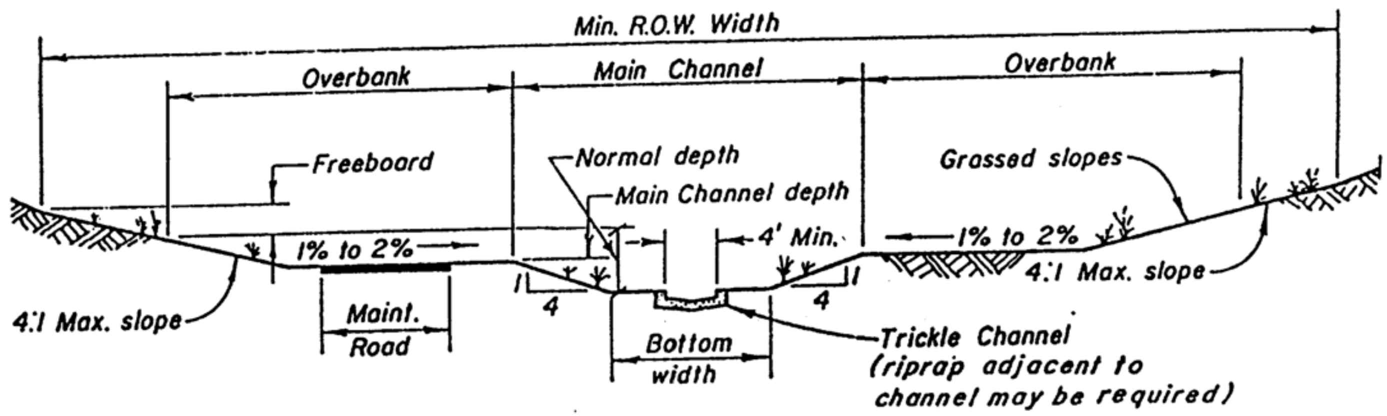
6. ROW Width: Minimum width to include freeboard and maintenance access road.

7. Channel Side Slope: Maximum side slope for grassed channels to be 4:1.

8. Froude Number: Maximum value shall not exceed 0.8 for initial and major floods.

9. The maximum flow velocity to be 7 fps for erosion resistant soils or 5 fps for sandy soils.

TYPICAL GRASSED LINED CHANNEL SECTION TYPE B



NOTES:

1. Main Channel: Capacity to be less than 20% of 100-year at Main Channel depth. Maximum 100-year flow velocity is 7 fps.
2. Trickle Channel: Minimum capacity to be 1% to 3% of 100-year flow but not less than 1 cfs. Channel to be constructed of concrete or other approved materials.
3. Normal Depth: Flow depth for 100-year flow shall be such that the product of hydraulic depth (A/T) and the 100-year flow velocity ($V_{max} = 7$ fps) be less than 35 cfs/ft.
4. Freeboard: Freeboard to be a minimum of 1-foot.
5. Maintenance/Access Road: Minimum width to be 12-feet.
6. ROW Width: Minimum width to include freeboard and maintenance access road.
7. Overbank: Flow in excess of main channel to be carried in this area. Area may be used for recreation purposes.
8. Froude Number: Maximum value shall not exceed 0.8 for initial and major floods.

Figure No. 7.7

d. Flow Depth

The maximum design depth of flow (outside the trickle channel area) for the 100-year flood shall be limited to 5.0-feet in grass lined channels.

e. Maintenance/Access Road

Maintenance access shall be provided for all major drainage ways with a minimum width of 12-feet.

f. Side Slopes

Side slopes shall be 4 (horizontal) to 1 (vertical) or flatter. Slopes as steep as 3:1 may be used in existing developed areas subject to additional erosion protection and approval from the City Engineer.

7. Grass

The grass species chosen must be sturdy, drought resistant, easy to establish and capable of spreading. A thick root structure is necessary to control weed growth and erosion. A mixture of native and introduced grass species has been found to be satisfactory in establishing an erosion-resistant channel lining. The Natural Resource Conservation Service and local landscape architects can provide assistance in selecting grass mixtures which have been successful.

Some of the grasses generally suitable for waterway use include White Dutch Clover, several Wheatgrass species, several Grama species, two Brome Species and Perennial Ryegrass. If a landscaped park or a maintained yard setting is desirable and an irrigation system is provided, Kentucky Bluegrass provides an excellent cover, but requires extensive water and maintenance while sodding is acceptable for some applications it is generally not recommended for use in drainage ways.

Newly constructed channels need a protective cover consisting of mulch and grass seeding immediately after completion. If possible, seed the distributed areas with permanent grass seed mix. To provide quick ground cover the seed mix should include a perennial ryegrass. The perennial ryegrass germinates quickly and will not compete with the sod-forming grasses later on. When immediate seeding of permanent grass is not practical, an annual crop may be planted with the perennial grass seeded later in the stubble or residue. Rye, oats, or ryegrass will provide temporary protection for waterways, through the crop should be clipped before it matures to seed.

Seed quality is important. Grass seed may range from 20 to 100 percent purity. Seeding rates are aimed at planting a given number of pure live seed per square foot. For additional information on seeding rates, the reader is referred to the Natural Resource Conservation Service.

Disturbed areas should be seeded by drilling. The drill must be a specially adapted power drawn grass drill. In areas where access is a problem a hydraulic seeder may be used. Seeding rates should be doubled if hydraulic equipment is used. All areas seeded should be mulched to prevent erosion and hold moisture in the soil surface. Cutting of the grass should be planned so that it will regrow to attain a height of at least 6-inches in November and June. Cutting practices should encourage the establishment and spread of the grass.

General rules for an effective grass lining for a waterway area are as follows:

a. Strip and stockpile topsoil during construction. Use stockpiled materials in the surface preparation prior to seeding operations.
b. Prepare a good, firm seedbed and use a crop residue or a mulch to protect the waterway and grass seedlings during establishment.
c. Allow one year for grass to show an adequate stand.
d. Select a simple grass mixture that best fits the conditions of the waterway.
e. Use good quality seed.
f. Use grass origins and strains known to be adaptable.
g. Plant at the best date for the particular grass.
h. Use planting equipment and methods that give uniform distribution and proper placement of seed.
i. Water grass as required to supplement rainfall until established.
j. Fertilize according to the needs of the grass and soil as shown by soil tests.
k. Overseed or repair bare spots with sod chunks or mulch as necessary.
l. Avoid using the waterway as a road or damaging the sod with tillage implements.
m. Mow when grass can make good regrowth and restore food reserves in the roots.

The following recommendations are made for establishing native grass cover on waterways in the Bismarck area:

a. Site and Soil Preparation

Grading and shaping should be done on the site to eliminate depressions and other rough areas that would interfere with the seeding equipment's ability to place the seed at proper depths and the proper row spacing.

Topsoil should be spread so that at least two inches (or more) covers all areas to be seeded. Hard pan or compacted clay soil areas should be scarified before spreading top soil to eliminate slippage surfaces and to promote root penetration. All areas that will not have two or more inches of topsoil may need to be fertilized. NRCS recommends up to 20 tons of manure to be worked into the soil or 50 lbs of nitrogen and 50 lbs of phosphate applied per acre prior to seeding.

A firm seed bed is essential for grass establishment and assures a shallow precise seeding depth and seed placement in contact with moist soil. The site should be packed firm enough so that tracks made by a person walking are not over one-half inch deep.

b. <u>Protective Cover Crops</u>

Native grass seeding can be made on sites without first planting a cover crop, but only on sites not subject to wind or water erosion. Cover crops add a great deal of protection to new seedlings. Mulching also provides excellent protection and helps retain soil moisture.

Small grains (spring wheat, oats, barley) planted in the spring (before July) should be seeded at a rate of 1-2 bushels per acre. Seeding should be crosswise to the slope of the site. During July and August the best cover crop choice is sudan grass, planted at a rate of around 30 lbs per acre. Areas seeded after August can be planted to winter wheat at the same rate as mentioned for spring wheat. Neither cover crops nor grass seedings should be seeded so late in the growing season that plants cannot establish adequate root systems to survive the winter.

Cover crops will need to be clipped (mowed) if they become too rank and interfere with grass seedling growth.

c. <u>Grass Seed Planting</u>

Sites should be seeded to the selected grass mixture immediately following the planting of the cover crop unless the site is highly erodible. If the soils are subject to wind and water erosion, the cover crop shall be allowed a few weeks to reach two to three inches height before seeding the grass mixture. The "double drilling" method, in which the site is crisscrossed or seeded twice, is recommended. If double drilled, then set the grass drill at one-half the specified seeding rate. The seeding depth should be set at least one-half to three-fourths inch, and the seeding rate for recommended North Dakota native grass species should be around 15 lbs. per acre of pure live seed.

The best times of the year for planting native grass species is early spring whenever surrounding grassland areas are beginning to green up and grow. Early fall is also an excellent time of year for establishing native grasses. Usually good moisture is available in late August and early September and seedlings can become well rooted before freeze up. Worst times are probably so late in the fall that seedlings cannot establish adequate root systems that will carry the seedlings through the winter, or mid summer when soil moisture levels are low and daytime temperatures and evaporation rates are high.

On sites where grass drilling equipment cannot be used, broadcast the grass seed at double the recommended rate and then rake, drag, or very lightly disc the site to help cover the seed.

d. Selected North Dakota Native Grass Species

A mixture of two or more grass species should always be used unless there is a specific reason for only one species. At least half of the species should be the sod-forming grass varieties or species. Selected North Dakota native species are listed below:

Sod Forming Natives	Bunchgrass Natives
Western wheatgrass	Slender wheatgrass
Reed canarygrass	Green needlegrass
Switchgrass	Little bluestem
Big bluestem	Sideoats grama
Prairie sandreed	
Sand bluestem	
Thickspike wheatgrass	

The best choices and recommended mixture rates, from the above lists, for planting in the Bismarck area are: (All weights are Pure Live Seed - PLS)

Western wheat - 5 lbs.	Green needlegrass - 1 lb.
Reed canarygrass (especially wet areas) - 1/2 lb.	Slender wheatgrass - 2 lbs.
Big bluestem (in dryer areas) - 2 lbs.	

A total of 10 1/2 lbs. (PLS) per acre.

There are times and places where mixing native and tame grasses is desirable. Adapted North Dakota tame grasses are listed as follows:

Sod Forming Tame Grasses	Bunchgrass Tame Grasses
Smooth bromegrass	Canada or Mandan wildrye
Garrison creeping foxtail	Tall fescue
Kentucky bluegrass	Hard fescue
Intermediate wheatgrass	Tall wheatgrass
Pubescent wheatgrass	Crested wheatgrass
Streambank wheatgrass	

The best choices and recommended mixture rates, from the above list, for planting in the Bismarck area are:

Smooth bromegrass - 3 lbs.
Crested wheatgrass (dry areas) - 2lbs
Garrison creeping foxtail - 1/2 lb
Canada or Mandan wildrye - 1 lb
Pubescent or Intermediate wheatgrass - 4lbs.

A total of 10 1/2 lbs. (PLS) per acre.

As a general rule a mixture of 8-12 lbs. (PLS) of seed per acre for most of the native and tame grasses listed above will provide a seeding rate of around 30 seeds per square foot, which is the desired rate for this area of North Dakota.

Legumes such as alfalfa or sweet clover are occasionally added to seed mixtures, but should never exceed 20% of the mixture by weight.

e. Maintenance of Seeded Sites

Weeds can be a problem which will cause loss of the new seedlings, especially where cover crops are not used. If weeds are competing severely with the new grass seedlings, they should be mowed short. As mentioned above, cover crops can sometimes become so rank that they compete with the native grass seedlings and should be mowed or can be hayed if the operation will not cause undue disturbance to the site. Proper use of herbicides will also control weed growth, but will not harm the grass seedlings.

Once established, native grass or mixed grass seedings will provide a wind and water-resistant cover that will require little maintenance. An occasional burning every four or five years or a late season mowing or haying can be applied after the second year of growth.

8. Water Surface Profiles

Computation of the water surface profile shall be presented for all open channels utilizing standard backwater methods, taking into consideration losses due to changes in velocity of channel cross section, drops, waterway openings, or obstructions. The energy and hydraulic gradients shall be shown on all preliminary drawings.

7.4.3.2 Concrete Lined Channels

The criteria for the design and construction of concrete lined channels are presented below:

1. Hydraulics

a. Freeboard

Adequate channel freeboard above the designed water surface shall be provided and shall be not less than that determined by the following:

$$H_{FB} = 2.0 + 0.025V\,(d)^{1/3} \qquad \text{(Equation 7.12)}$$

where:

H_{FB}	=	freeboard height (feet)
V	=	velocity (fps)
d	=	depth (feet)

Freeboard shall be in addition to superelevation, standing waves, and/or other water surface disturbances. These special situations should be addressed in the Stormwater Management Plan report submitted with the construction drawings and specifications (Chapter 2).

Concrete side slopes shall be extended to provide freeboard. Freeboard shall not be obtained by the construction of levees, and shall be contained within the natural or excavated section.

b. Superelevation

Superelevation of the water surface shall be determined at all horizontal curves and design of the channel section adjusted accordingly (see Section 7.4.3.1-3).

c. Velocities

Flow velocities shall not exceed 7 fps or result in a Froude Number greater than 0.8 (see Section 7.3.3) during the 100-year flood for non-reinforced linings. Flow velocities shall not exceed 18 fps for reinforced linings during the 100-year flood.

d. n-Values

Refer to Table No. 7.5 for range of values, with the high value used for capacity determination and the low value used for velocity consideration.

2. Concrete Materials

a. Cement Type: II, IIA, or III
b. Minimum cement content: 550 lbs/C.Y.
c. Maximum water-cement ratio: 0.50 (6 gals. per sack)
d. Maximum aggregate size: 1-1/2 inches
e. Air content range: 4-7 percent
f. Slump: 2-4 inches
g. Minimum compressive strength ($f^1{}_c$): 3250 psi at 28 days

3. Concrete Lining Section

a. Reinforced linings shall have a minimum thickness of 7-inches.
b. The side slopes shall be a maximum of 1-1/2 horizontal to 1 vertical or be a structurally reinforced retaining wall if steeper.

4. Concrete Joints

a. Channels shall be continuously reinforced without transverse joints. Expansion joints shall be installed where new concrete lining is connected to a rigid structure or to existing concrete lining which is not continuously reinforced.
b. Longitudinal joints, where required, shall be constructed on the side walls at least 1-foot vertically above channel invert.
c. All joints shall be designed to prevent differential movement.
d. Construction joints are required for all cold joints and where the lining thickness changes.

5. Concrete Finish

The surface of the concrete lining shall be provided with a wood float finish. Excessive working or wetting of the finish shall be avoided.

6. Concrete Curing

All concrete shall be cured by the immediate application of a liquid membrane-forming curing compound (white pigmented) upon completion of the concrete finish.

TABLE NO. 7.5

MANNING'S n-VALUES FOR OPEN CHANNELS

CHANNEL TYPE	N-VALUE RANGE	RECOMMENDED VALUE
A. Earth Lined (ditches/canals)		
1. Clean, Weathered	.018 to .025	.022
2. Clean, Gravel	.022 to .030	.025
3. Some Weeds	.022 to .033	.027
4. Non-Maintained	.030 to .040	.035
B. Grass Lined (man-made)		
1. RV >10 (see Fig.-705)	.029 to .034	.032 (1)
2. RV < 10	.032 to .100	See Fig.-705
C. Natural Streams	.025 to 0.100	Note (2)
D. Riprap Lined		
1. Ordinary Riprap	$n = .0395\ d_{50}^{0.17}$	(Sect. 705.4)
2. Gabions		0.035
3. Grouted Riprap	.023 to .030	0.027
4. Slope Mattress	.025 to .033	.028
E. Concrete Lined		
1. Float Finished/Wood Forms	.013 to .016	Note (3)
2. Slip Formed	.013 to .016	Note (3)
3. Gunite	.016 to .023	Note (3)

NOTES:
1. Use as starting value to estimate channel capacity.
2. Refer to Chow, V.T., Open Channel Hydraulics, McGraw-Hill Book Co., 1959, Table-5-6.
3. High value used for capacity determination and low value used for velocity consideration.

7. Reinforcement Steel

   a. Steel reinforcement shall be grade 60 deformed bars. Wire mesh shall not be used.

   b. Ratio of longitudinal steel area to concrete cross sectional area shall be greater than 0.05.

   c. Ratio of transverse steel area to concrete cross sectional area shall be greater than .0025.

   d. Reinforcing steel shall be placed at the center of the section with a minimum clear cover of 3-inches adjacent to the earth.

8. Earthwork

   The following areas shall be compacted to at least 90 percent of maximum density as determined by ASTM D-1557 (Modified Proctor):

   a. The 12-inches of subgrade immediately beneath concrete lining (both channel bottom and side slopes).

   b. Top 12-inches of a maintenance road.

   c. Top 12-inches of earth surface within 10-feet of concrete channel lip.

   d. All fill material.

9. Bedding

   Provide 6-inches of granular bedding in accordance with design procedures in Section 7.4.1.

10. Underdrain

   Longitudinal underdrains as required shall be provided on 10-foot centers and shall daylight at the check drops. A check valve shall be provided at the outlet to prevent backflow into the drain.

11. Safety Requirements

a. A 4-foot high galvanized chain link fence shall be installed to prevent access wherever the 100-year channel lining depths exceed 3-feet. Gates, with top latch, shall be placed at 250-feet intervals and staggered where fence is required on both sides of the channel. Higher fences may be required near energy dissipation structures.

b. Ladder-type steps shall be installed not more than 400-feet apart on alternating sides of the channel. Bottom rung shall be placed approximately 12-inches vertically above channel invert.

7.4.3.3 Rock Lined Channels

Channel linings constructed from ordinary riprap, grouted riprap, or wire encased rock to control channel erosion have been found to be cost effective where channel reaches are relatively short (less than 1/4 mile). Situations for which riprap linings might be appropriate area: 1) where major flows, such as the 100-year flood are found to produce channel velocities in excess of allowable non-eroding values (5-feet per second for sandy soil conditions and 7-feet per second in erosion resistant soils); 2) where channel side slopes must be steeper than 3:1; 3) for low flow channels, and 4) where rapid changes in channel geometry occur such as channel bends and transitions. Design criteria applicable to these situations are presented in Section 7.4.6.1. Section 7.4.6.1 is valid only for subcritical flow conditions where the Froude Number is 0.8 or less.

7.4.3.4 Roadside Ditches

The criteria for the design of roadside ditches are similar to the criteria for grass lined channels with modifications for the special purpose of minor storm drainage. The criteria are as follows (refer to Figure No. 7.8):

1. Capacity

Roadside ditches shall have adequate capacity for the 5-year storm runoff peaks. Capacity shall be as defined in Table No. 7.6. Where the storm runoff exceeds the capacity of the ditch, a stormwater management system shall be required.

2. Flow Velocity

The maximum velocity for the 5-year flood peak shall not exceed 5.0-feet per second for a Type I ditch and 7.0-feet per second for Type II or III ditch. The capacity limitations for Table No. 7.6 are based on a maximum Froude Number of 0.8 for Types I, II, and 0.9 for Type III.

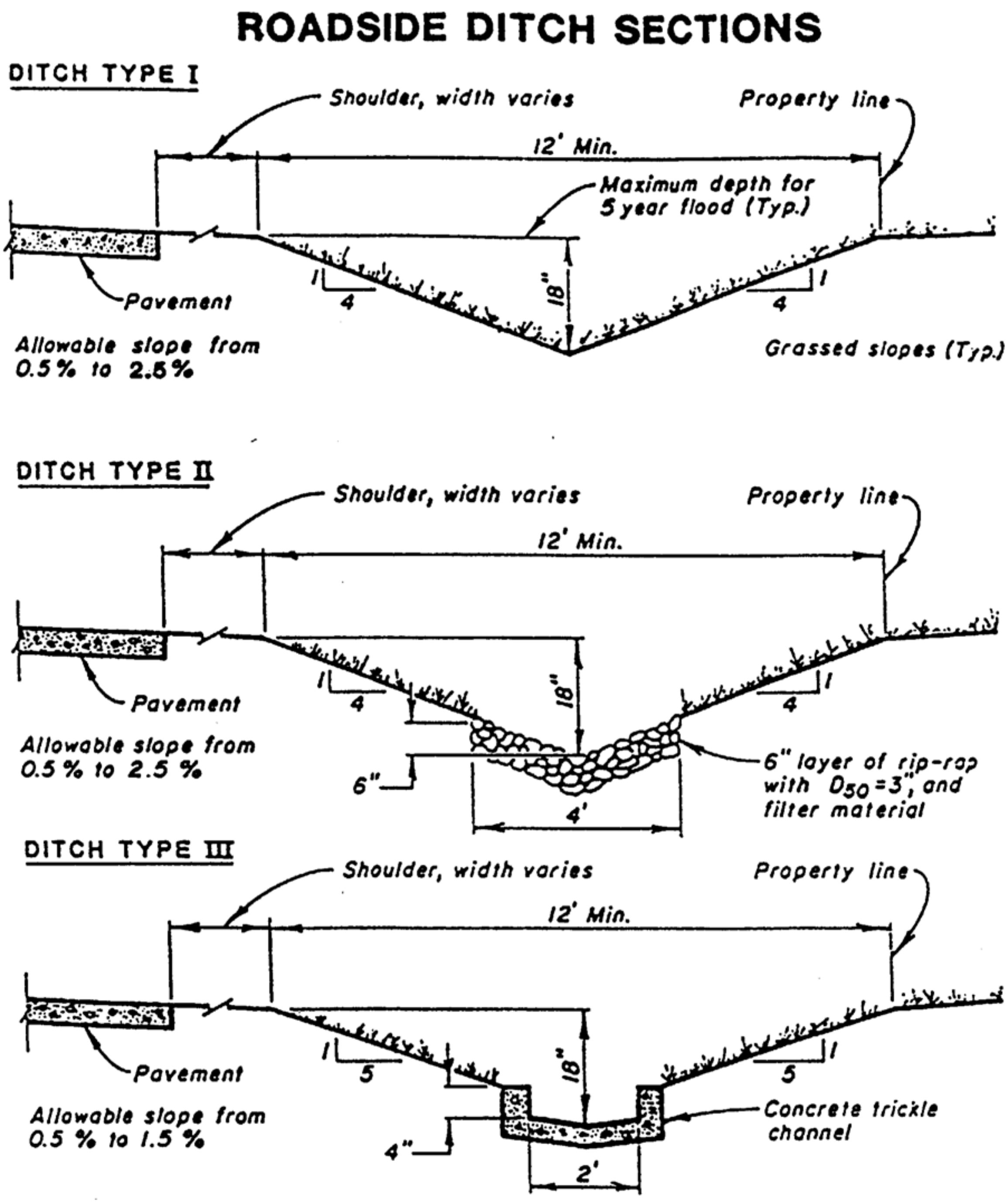
TABLE NO 7.6

ROADSIDE DITCH CAPACITIES

SLOPE (%)	DITCH TYPE I		DITCH TYPE II		DITCH TYPE III	
	VEL. (fps)	Q (cfs)	VEL. (fps)	Q (cfs)	VEL. (fps)	Q (cfs)
0.5	0.7	6	0.7	6	2.5	22
1.0	1.6	14	1.6	14	3.5	31
1.5	2.4	22	2.4	22	4.4	39
2.0	3.2	29	3.2	29	Not Permitted	
2.5 (3)	4.0	36	4.0	36	Not Permitted	

NOTES:
1. See Figure No. 7.8 for geometry of roadside ditch.
2. Velocity and capacity are based upon the SCS Retardance Curve "C".
3. Maximum permissible slope for roadside ditch is 2.5%. Slope limitation is based on a maximum Froude number of 0.8 for Type I & II and 0.9 for Type III ditch.
4. Linearly interpolate for intermediate slopes.

Figure No. 7.8



NOTE: 1. See Table 7.6 for capacity of roadside ditch.
2. For street slopes greater than maximum allowable, check drops (2' maximum height) will be required.

3. Longitudinal Slope

The slope shall be limited by the average velocity of the 5-year flood peaks. Gradient control structures such as check drops may be required where street slopes are in excess of 2.5 percent.

4. Freeboard

No freeboard is required.

5. Curvature

The minimum radius of curvature shall be 25-feet.

6. Roughness Coefficient

Manning's "n" values presented in Figure No. 7.5 have been used in the capacity computation for roadside ditches.

7. Grass Lining

The grass lining shall be in accordance with Section 7.4.3.1.

8. Driveway Culverts

Driveway culverts shall be sized to pass the 5-year ditch flow capacity without overtopping the driveway. The minimum size culvert shall be a 22" x 13" CMPA (18" equivalent round pipe) with flared end sections. More than one culvert may be required.

7.4.3.5 Other Channel Linings

The criteria for the design of channels with linings other than grass, rock, or concrete will be dependent on the manufacturer’s recommendations for the specific product. The applicant will be required to submit the technical data in support of the proposed material. Additional information or calculations may be requested by the City Engineer to verify assumptions or design criteria. The following minimum criteria will also apply.

1. Flow Velocity

The maximum normal depth velocity will be dependent on the construction material utilized. The Froude Number shall be less than 0.8.

2. Freeboard

Defined by Equation 7.10.

3. Curvature

The center line curvature shall have a minimum radius twice the top width of the design flow but not less than 100-feet.

4. Roughness Coefficient

A Manning's "n" value range shall be established by the manufacturers data with the high value used to determine depth/capacity requirements and the low value used to determine Froude Number and velocity restrictions.

5. Cross Sections

Same as for grassed lined channels, Section 7.4.3.1.

7.4.4 Gradient Control Structures

7.4.4.1 General

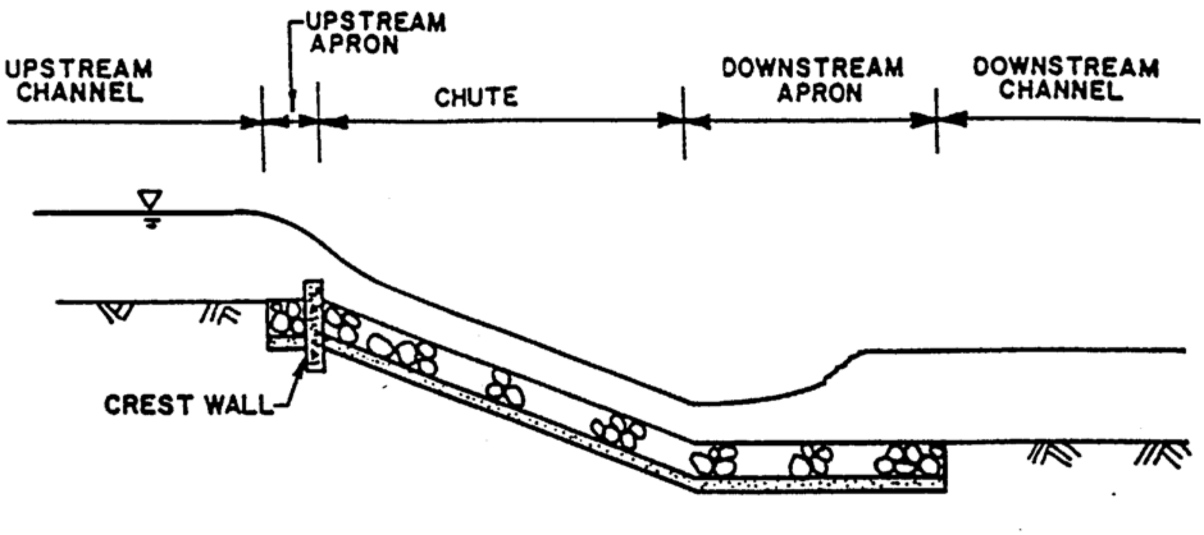
Hydraulic structures for the purpose of this Manual are defined as those structures which control the storm runoff during a condition of rapid directional change, or rapid acceleration or deceleration of velocity. Included in this category are channel drops (sometimes referred to as check drops or erosion control devices) and energy dissipaters (i.e., stilling basins).

The criteria presented in this section in many cases are generalized since each structure is unique, with the possible exception of channel drops. The user is encouraged to coordinate with the City Engineer when planning and designing hydraulic structures.

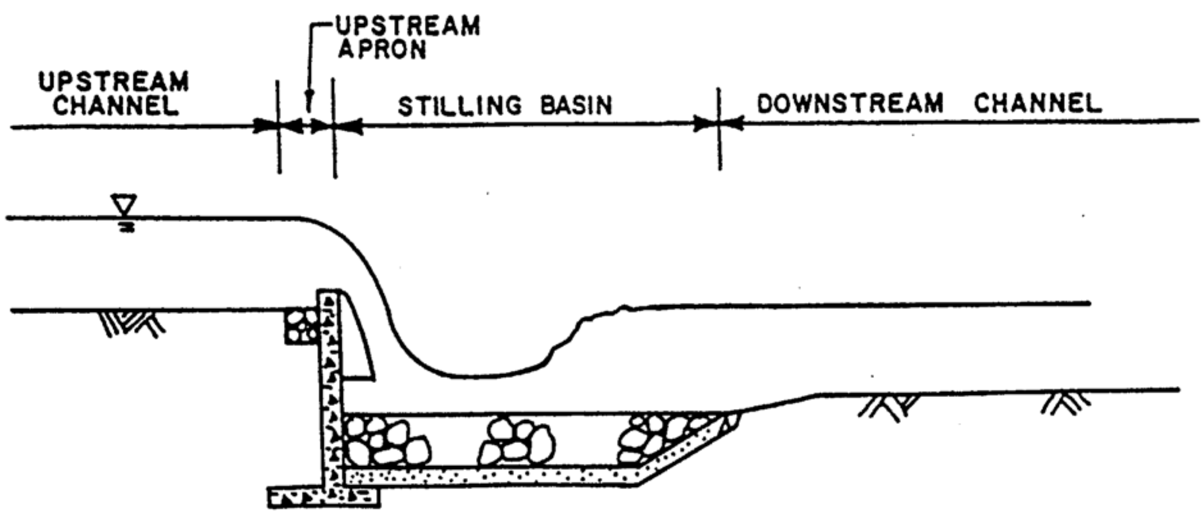
The most common use of channel drops is to control the longitudinal slope of grass lined channels to keep design velocities within acceptable limits. Sloping and vertical riprap drop structures, Figure No. 7.9, are two possible types of drop structures. The following criteria will dictate the use of drop structures for open channel flow:

1. The use of channel drops is required when the channel would otherwise be too steep for design conditions. All drops shall be designed to protect the upstream and downstream channel from erosion. Drop structure analysis may be required to determine the length of hydraulic jump and adequate erosion control measures.

Figure No. 7.9



A. SLOPING CHANNEL DROP



B. VERTICAL CHANNEL DROP

GENERALIZED PROFILES OF RIPRAP DROP STRUCTURES

2. Vertical drops shall be constructed of concrete or gabions.

3. Sloped drops shall be constructed of concrete, gabions or riprap. Riprap drops shall have a minimum of 6" thick gravel base which may require grouting. Engineering fabric under riprap is often preferable depending on soil conditions.

4. At drop structures both the channel bottom and banks shall be protected from erosion.

5. Safety considerations along road sides or where recreational (off-road) vehicle use may be anticipated.

7.4.4.2 Sloping Channel Drop Structures

The design chart for the sloping drop structure, Table No. 7.7, is based upon the unit discharge (q) of the approach channel, the riprap classification and the slope of the drop structure, and is valid only for subcritical flow in the approach channel (i.e., $Fr \leq 0.8$). The unit discharge is found by taking the average or normal channel velocity (V_n) for the 100-year discharge times the normal depth of the channel (Y_n). Since the maximum for a grass lined channel in erosion resistant soils are V_n = 7 ft./sec. and Y_n = 5 ft. (Section 7.4.3.1), the maximum q allowed is $q = V_nY_n$ = (7 ft./sec.)(5 ft.) = 35 cfs/ft. This q is also the practical limit for the largest riprap specified in the Section 7.4.6.1.
The design chart is also based upon a prismatic channel section throughout, from the upstream channel through the drop to the downstream channel. The maximum (steepest) allowable side slope for the riprap lined channel within the drop structure is 4:1. Flatter side slopes are allowable and encouraged when available right-of-way permits.

The classification of riprap chosen for the sloping portion of the structure should be used throughout the structure, including the upstream and downstream aprons, the channel bottom and side slopes. The riprap should extend up the side slopes to a depth equal to 1-foot above the normal depth projected upstream from the downstream channel, or 1-foot above the critical depth, whichever is greater. The maximum fall allowed at any one drop structure is 4-feet from the upper channel bottom to the lower channel bottom, excluding the trickle channel.

Due to recent experience, the requirement for the riprap of sloping check drops has been revised. Where Class "H" or "VH" is specified (Table No. 7.7), Class "M" grouted riprap shall be used. See Section 7.4.6.1 for riprap requirements.

A detailed description of the drop structure and the design procedure, proceeding from upstream to downstream, is given below and shown on Figure No. 7.10.

Sloping Riprap Channel Drop Design Chart

Table No. 7.7

MAXIMUM UNIT DISCHARGE q (cfs / ft)	ALLOWABLE CHUTE SLOPE - So FOR EACH RIP RAP CLASSIFICATION			LENGTH OF DOWNSTREAM APRON l3 (FT)
	M	H	VH	
15	0 to 7:1	7:1 to 4:1	N/A	15
20	0 to 8:1	8:1 to 5:1	5:1 to 4:1	20
25	0 to 10:1	10:1 to 6:1	6:1 to 4:1	20
30	0 to 12:1	12:1 to 7:1	7:1 to 4.5:1	25
35	0 to 13:1	13:1 to 8:1	8:1 to 6:1	25
DR*	1.75'	2.6'	3.5'	
DR**	2.0'	3.0'	4.0'	
DRW	1.5 x DR	1.25 x DR	1.0 x DR	

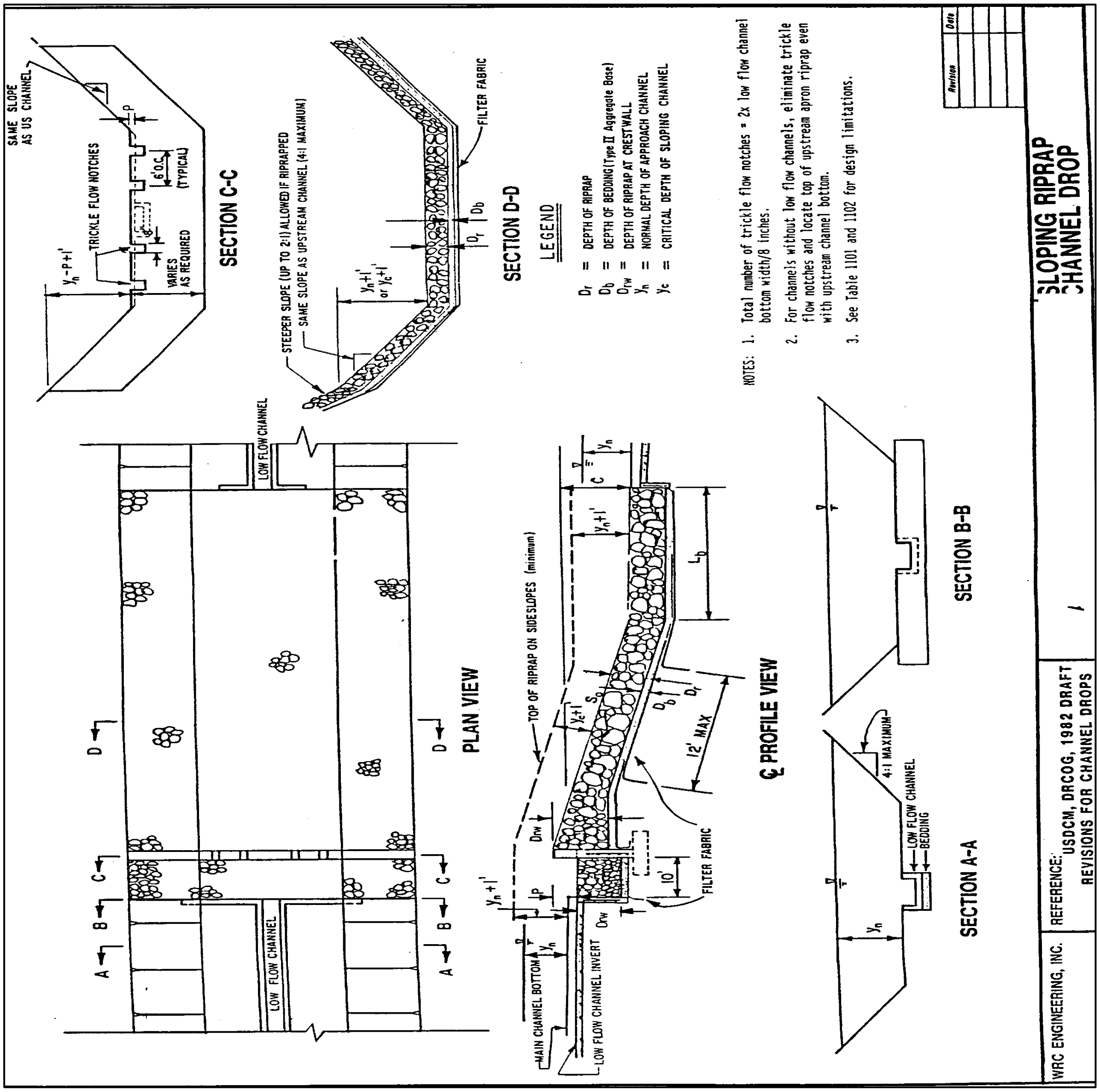
* For Erosion Resistant, Cohesive Soils
** For Sandy or Highly Erosive Soils

NOTES:

1 q = Unit discharge = VnYn, where Vn = average channel velocity and Yn = normal depth of the upstream channel
2 So = Longitudianl channel slope expressed in feet horizontal per foot vertical.
3 Dr = Depth of riprap blanket in feet.
4 Drw = Depth of riprap blanket at the downstream face of the crest wall.
5 Rock size, Dr and Drw shall be the same throughout the drop structure
6 Chute and channel side slopes shall not be steeper than 4:1.
7 Maximum allowable drop = 4.0 '
8 This chart is for ordinary riprap structures only. Other types of drop sturctures require their own hydraulic analysis. (See section 1102.4).
9 For "H" and "VH" riprap substitute grouted "M) riprap

CREST WALL ELEBATION CHART

BOTTOM WIDTH* (FT)	P@Vn = 5 fps (ft)	P@Vn = 7 fps (ft)
5	0.2'	0.2'
40	0.4'	0.2'
100	0.5'	0.3'



Sloping Riprap Channel Drop
Figure No. 7.10

1. Criteria

a. Approach Depth: The upstream and downstream channels will normally be grass lined trapezoidal sections with trickle channels to convey normal low water flows. The maximum normal depth Y_n is 5 feet and the maximum normal velocity V_n is 7-feet/sec. for erosion resistant soils and 5-feet/sec. for easily eroded soils.

b. Trickle Channel: The trickle channel shown in this case is a rectangular concrete channel. The concrete channel ends at the upstream end of the upstream riprap apron. A combination cut-off wall and foundation wall is provided to give the end of the trickle channel additional support. The water is allowed to "trickle" through the upstream apron and through the crest wall (discussed below). Riprap trickle channels would simply feather into the upstream apron.

c. Approach Apron: A 10-foot long riprap apron is provided upstream of the cut-off wall to protect against the increasing velocities and turbulence which result as the water approaches the sloping portion of the drop structure. The same riprap and bedding design should be used as specified for the portion of the drop structure downstream of the cut-off wall.

d. Crest Wall: The crest wall is a very important part of the drop structure, and has several purposes, one of which is to provide a level rigid boundary section and distribute the flow evenly over the entire width of the structure. This is extremely important since the selection of the riprap is based upon the unit discharge, and without the wall, flow concentrations could result which would greatly exceed the design discharge.

The trickle channel is ended at the upstream end of the upstream apron to prevent the trickle channel from concentrating additional water at a point during high flows, thus exceeding the design unit discharge. The apron and the crest wall combine to disperse the concentrated flow. The trickle flows must be allowed through the crest wall to prevent ponding. A series of notches in the wall will allow the trickle flows to do this. The size and number of notches will depend on the design discharge of the trickle channel. Note that they are offset from the trickle channel to permit flow of water through the upstream apron. The voids in the riprap below the notch inverts are expected to silt in rapidly or they can be filled at the time of construction. The crest wall can also be used to reduce or eliminate seepage and piping along with the failures which can result from these problems.

The two most common types of walls used will probably be reinforced concrete or sheet pile. The design of the wall is a structural problem which will not be addressed here. The depth of the wall should be at least to the bottom of the bedding material and could be deeper if necessary for the control of piping.

The top of the crest wall should be placed a distance P above the upstream channel bottom. This is done to create a higher water surface elevation upstream, thus reducing the drawdown effects normally caused by a drop structure. P can be determined from Table No. 7.7 and is not considered in the total allowable vertical drop.

e. Chute Apron: The riprap chute portion of the drop structure and the downstream apron can be sized using Table No. 7.7. The way to use the table is to compute $q = V_nY_n$, enter the table at the next highest value of q in the left-hand column, determine the allowable slopes for the three riprap classifications in the row for that q and select the best combination of riprap classification and slope using site and cost considerations. The length of the downstream apron L_B and the depth of the riprap D_r can also be obtained from Table No. 7.7. The riprap must be placed on bedding and filter fabric as shown in Figure No. 7.10. The 2-foot long filter fabric cut-offs help prevent piping failures. The riprap should extend up the side slopes a distance of Y_n + 1' as projected from the downstream channel or the critical depth plus 1-foot, whichever is greater. The side slopes for the chute and the downstream apron should be the same as the crest wall and upstream channel with the exception that a riprap slope as steep as 2:1 can be used starting above the riprap lining at the height required above. The thickness of the riprap immediately downstream of the crest wall should be increased to D_{RW} as shown in Table No. 7.7. This extra thickness is necessary to protect the most critical area of the structure. The voids in the apron can be filled during construction to reduce ponding of low flows in the apron area.

f. Exit Depth: The downstream channel should be the same as the upstream channel, including a trickle channel. The trickle channel invert must be below the top of the adjacent riprap section to insure that trickle flows will drain into the trickle channel. For concrete trickle channels, a foundation wall similar to the one used for the upstream trickle channel should be used. In some instances, the wall may also be used to control seepage and piping.

Example No. 7.4: Sloping Riprap Channel Drop

Given:

Q = 1600 cfs

Upstream and downstream channel dimensions

bottom width = 50 ft. S = .0043 ft./ft.

side slopes = 4:1 y_c = 2.9 ft.

Y_n = 4.0 ft. V_n = 6.0 fps

Erosion resistant soils

Concrete trickle channel

Drop required = 3.0 ft.

Procedure:

Step 1: Determine maximum unit discharge.

$q = V_n Y_n$ = (6.0 fps)(4 ft.) = 24 cfs/ft.

Step 2: Select the best combination of riprap classification and chute slope from Table No. 7.7 for q = 25 cfs/ft.

The following options are available:

a. Type M at 10:1 or flatter; D_r = 1.75'; D_{rw} = 2.6'
b. Type H at 6:1 or flatter; D_r = 2.6'; D_{rw} = 3.25'
c. Type VH at 4:1 or flatter; D_r = 3.5'; D_{rw} = 3.5'

Substitute Type M grouted for Type H or Type VH riprap.

The best combination of riprap classification and slope will depend on many factors such as availability and cost of the various riprap classifications and right-of-way limitations. Remember to consider bedding and filter fabric costs. For the sake of this example select Type M riprap grouted at a 7:1' slope.

Step 3: Select length of downstream apron L_B = 20'

Step 4: Determine crest wall elevation.

Bottom width = 50', closest to 40'
V_n = 6 fps, halfway between 5 fps and 7 fps
Use P = 0.3'

7.4.4.3 Vertical Channel Drop Structures

The design chart for the vertical channel drop, Table No. 7.8, is based upon the height of the drop and the normal depth and velocity of the approach and exit channels. The channel must be prismatic throughout, from the upstream channel through the drop to the downstream channel.

The maximum (steepest) allowable side slope for the riprap stilling basin is 4:1. Flatter side slopes are allowable and encouraged when available right-of-way permits. The riprap should extend up the side slopes to a depth equal to 1-foot above the normal depth projected upstream from the downstream channel. The maximum fall allowed at any one drop structure is 4-feet from the upper channel bottom to the lower channel bottom, excluding the trickle channel.

A detailed description of the drop structure and the design procedure, going from upstream to downstream, is given below and shown on Figure No. 7.11.

1. Criteria

   a. Approach Depth: The upstream channels will normally be grass lined trapezoidal channels with trickle channels to convey normal low water flows. The maximum normal depth Y_n is 5-feet and the maximum normal velocity V_n is 7-feet/sec. for erosion resistant soils and 5-feet/sec. for easily eroded soils.

   b. Trickle Channel: The trickle channel shown in this case is a rectangular concrete channel. The concrete channel ends at the upstream end of the upstream riprap apron. A combination cut-off wall and foundation wall, to give the end of the trickle channel additional support, is provided. The water is allowed to "trickle" through the upstream apron and through the vertical wall. Riprap trickle channels would simply feather into the upstream apron.

   c. Approach Apron: A 10-foot long apron is provided upstream of the cut-off wall to protect against the increasing velocities and turbulence which result as the water approaches the vertical drop. Type M riprap can be used for this apron.

   d. Crest Wall: The vertical wall should have the same trapezoidal shape as the approach channel. The wall distributes the flow evenly over the entire width of the drop structure. This is important to prevent flow concentrations which would adversely affect the riprap basin.

Vertical Riprap Channel Drop Design Chart

Table No. 7.8

C (ft)	Vn (fps)	Yn&Yz (ft)	P (ft)	B (ft)	A (ft)	L_a (ft)	D (ft)	E (ft)	RIPRAP CLASS.
2	5	4	0.1	0.6	2.0	20	4	3	M
2	5	5	*	0.8	2.5	25	5	4	H
2	5; 7	4	0.1	0.8	2.5	20	5	4	H
2	5; 7	5	*	0.8	2.5	25	5	4	H
3	5	4	0.1	1.0	2.5	20	5	4	H
3	5	5	*	1.0	2.5	25	5	4	H
3	5; 7	4	0.1	1.0	2.5	20	5	4	H
3	5; 7	5	*	1.0	2.5	25	5	4	H
4	5	4	0.1	1.2	3.5	20	7	5	VH
4	5	5	*	1.2	3.5	25	7	5	VH
4	5; 7	4	0.1	1.4	3.5	20	7	6	VH
4	5; 7	5	*	1.4	3.5	25	7	6	VH

* See Table below to caculate P

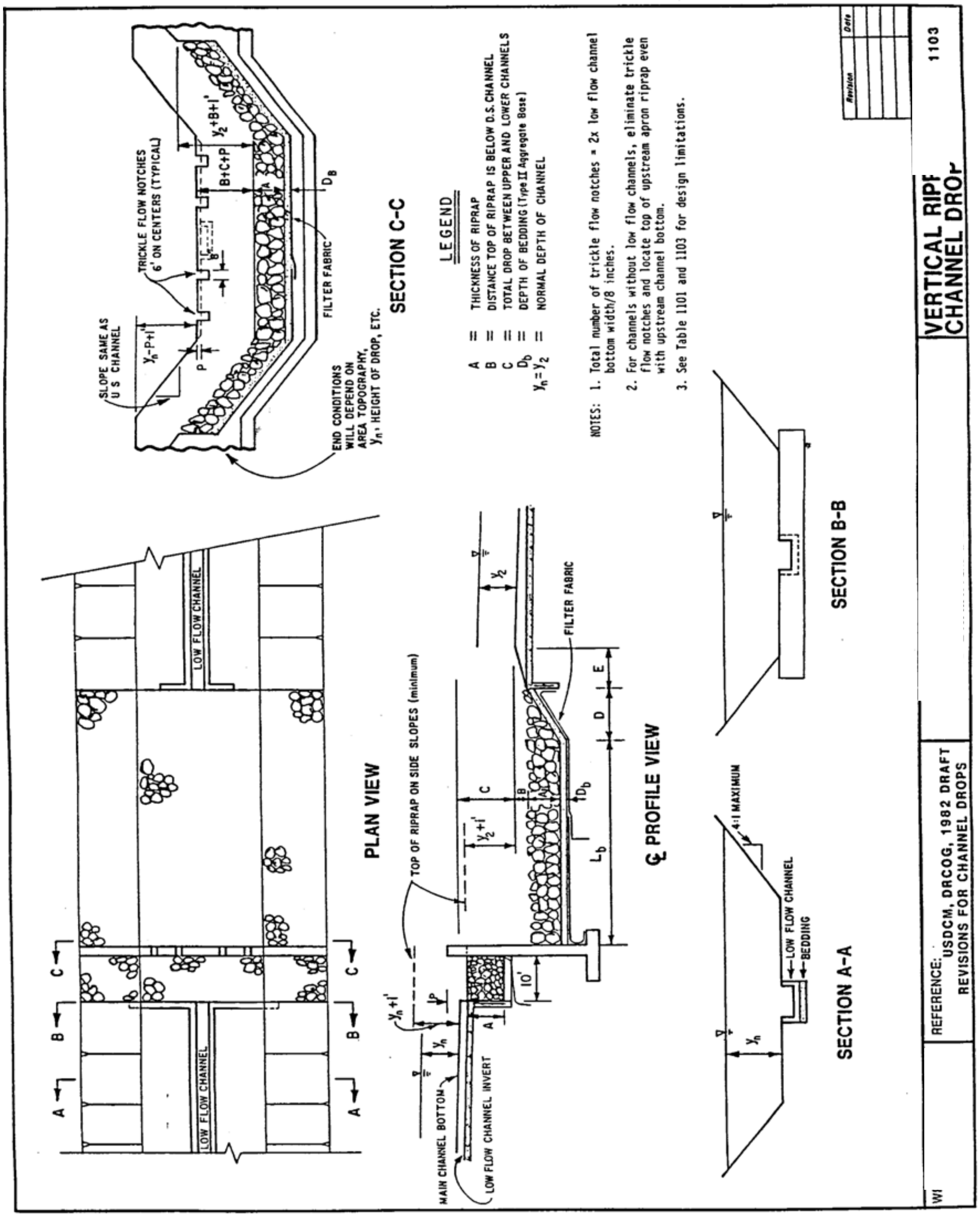
NOTES:
1. See Fig. 1103 for definition of symbols
2. See Section 705 for riprap gradation, classification and bedding requirements.
3. Maximum Allowable C = 4.0'
4. This chart is for ordinary riprap structures only. Other types of drop structures require their own hydraulic analysis. (See Section 1102.4).

CREST WALL ELEVATION CHART

BOTTOM WIDTH* (ft)	P@Vn = 5 fps (ft)	P@Vn = 7 fps (ft)
5	0.2'	0.2'
40	0.4'	0.2'
100	0.5'	0.3'

*Bottom Width of Approach Channel

Vertical Riprap Channel Drop
Figure No. 7.11



The trickle channel is ended at the upstream end of the upstream apron to prevent the trickle channel from concentrating additional water at a point during high flows, thus exceeding the design assumptions. The apron and the vertical wall combine to disperse the flow concentrated in the trickle channel. The trickle flows are allowed through the wall through a series of notches in order to prevent ponding. The voids in the riprap below the notch inverts are expected to silt in rapidly, or they can be filled at the time of construction.

The wall must be designed as a structural retaining wall. The top of the wall should be placed a distance P above the upstream channel bottom. This is done to create a higher water surface elevation upstream, thus reducing the drawdown effects normally caused by a sudden drop. P can be determined from Table No. 7.8.

e. Chute Apron: The riprap stilling basin is designed to force the hydraulic jump to occur within the basin, and is designed for essentially zero scour. The floor of the basin is depressed an amount B below the downstream channel bottom, excluding the trickle channel. This is done to create a deeper downstream sequent depth which helps keep the hydraulic jump in the basin. This arrangement will cause ponding in the basin. The trickle channel can, depending on the depth, relieve all or some of the ponding. The riprap can also be buried and vegetated to reduce the ponded area to a smaller size.

The riprap basin can be sized using Table No. 7.8. The way to use the table is to determine the required height of the drop C, the normal depths Y_n and Y_2. Both channels must have the same geometry and Y_2 must be equal to Y_n in order to use the table. Enter the row which contains the correct C, V_n and Y_n and Y_2 and select the riprap classification and all necessary dimensions from that row.

The riprap must be placed on bedding and filter fabric as shown in Figure No. 7.11. The riprap should extend up the channel side slopes a distance of Y_2 + 1' as projected from the downstream channel. The basin side slopes should be the same as those in the downstream channel (4:1 or flatter) up to the Y_2 + 1' location, above which riprap slopes as steep as 2:1 are allowable.

f. Exit Depth: The downstream channel should be the same as the upstream channel, including a trickle channel. For concrete trickle channels a foundation wall similar to the one used for the upstream trickle channel should be used. In some instances the wall may also be used to control seepage and piping.

Example No. 7.5: Vertical Riprap Channel Drop

Given: Q = 1600 cfs

Upstream and downstream channel dimensions

bottom width = 50 ft. S = .0043 ft./ft.
side slopes = 4:1 Y_c = 2.9 ft.
Y_n = 4.0 ft. V_n = 6.0 fps

Erosion resistant soils
Concrete trickle channel
Drop required = 3.0 ft.

Procedure:

Step 1: From Table No. 7.8, for C = 3.0', V_n = 6.0 fps and Y_n and Y_2 = 4.0'. Select the riprap designation and the riprap basin dimensions.

Riprap = Type H
B = 1.0'
A = 2.5'
L_B = 20'
D = 5.0'
E = 4.0'

Step 2: Determine P = 0.1 from Table No. 7.8

Step 3: Design retaining wall and finalize dimension

7.4.4.4 Other Types of Channel Drop Structures

When the unit discharge in the channel exceeds 35 cfs/ft., riprap drop structures will not be permitted. A different type of channel drop and extensive channel transitioning will be required. The trapezoidal section must first be transitioned into a vertical concrete channel. The flows are then accelerated by a sloping concrete chute and a concrete stilling basin is constructed to dissipate the energy. The channel is then transitioned back into the trapezoidal grass lined section.

The detailed hydraulic evaluation of the channel transitions, chute, and energy dissipators are beyond the scope of this Manual. The design engineer is referred to the technical references in Chapter 14 for additional information. A general discussion and design examples for energy dissipators are presented in Section 7.4.5.

7.4.5 Energy Dissipation Structures

7.4.5.1 Criteria for Use

Energy dissipators will be required at channel drops when the unit discharge exceeds 35 cfs/ft. and at the outlet of culverts or storm sewers when the velocity exceeds 16 fps. The dissipators shall be constructed of concrete and shall be one of the types of structures presented in this section.

7.4.5.2 Types of Energy Dissipators

Many stilling basins and energy-dissipating devices have been designed in conjunction with spillways, outlet works, and canal structures, utilizing blocks, sills, or other roughness elements to impose exaggerated resistance to the flow. The type of stilling basin selected is based upon hydraulic requirements, available space and cost. The hydraulic jump which occurs in a stilling basin has distinctive characteristics depending on the energy of flow which must be dissipated in relation to the depth of the flow.

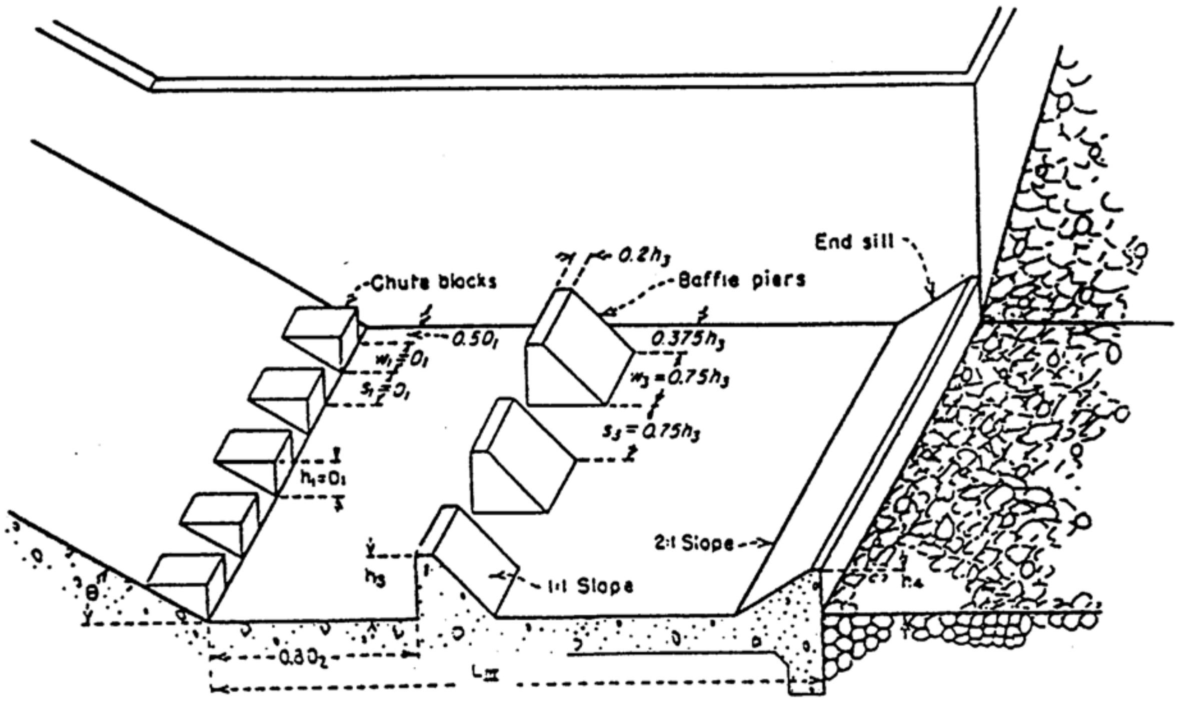
1. Short Stilling Basin (USBR Type III): The most effective way to shorten a stilling basin is to modify the jump by the addition of appurtenances in the basin. However, the appurtenances should be self-cleaning or nonclogging. The recommended design for a Type III stilling basin is shown in Figure No. 7.12. The chute blocks at the upstream end of a basin tend to corrugate the jet, lifting a portion of it from the floor to create a greater number of energy dissipating eddies. These eddies result in a shorter length of jump than would be possible without them, and tend to stabilize the jump. The baffle piers act as an impact dissipation device and the end sill is for scour control. The end sill has little or no effect on the jump. The only purpose of the end sill in a stilling basin is to direct the remaining bottom currents upward and away from the channel bed.

   This type of basin is recommended at the outlet of a sloping channel drop when there is adequate tailwater. For insufficient tailwater, USBR Type IX basin is recommended.

2. Baffled Apron Stilling Basin (USBR Type IX): Baffled aprons are used to dissipate the energy in the flow at a drop. They require no initial tailwater to be effective although channel bed scour is not as deep and is less extensive when the tailwater forms a pool into which the flow discharges. The chutes are constructed on an excavated slope, 2:1 or flatter, extending to below the channel bottom. Backfill is placed over one or more rows of baffles to restore the original streambed elevation. When scour or downstream channel degradation occurs, successive rows of baffle piers are exposed to prevent excessive acceleration of the flow entering the channel. If degradation does not occur the scour creates a stilling pool at the downstream end of the chute, stabilizing the scour pattern. The simplified hydraulic design of the baffled apron is shown on Figure No. 7.13.

USBR Type III Stilling Basin

Figure No. 7.12



NOTE: See Figure No. 7.15 for design data.

This type of basin is recommended for a channel drop where insufficient tailwater prevents the use of a Type III stilling basin. The basin can also be used for channel drops when adequate tailwater is available.

3. Impact Stilling Basin (USBR Type VI): This stilling basin is an impact-type energy dissipator, contained in a relatively small box-like structure, and requiring little or no tailwater for successful performance. The general arrangement of the basin for various discharges is shown on Figure No. 7.14. This type of basin is subjected to large dynamic forces and turbulence which must be considered in the structural design. The structure should be made sufficiently stable to resist sliding against the impact load on the baffle wall and must resist the severe vibrations. Riprapping should also be provided along the bottom and sides adjacent to the structure to avoid the tendency for scour of the outlet channel downstream from the end sill when shallow tailwater exists. This type of stilling basin is very effective at the outlet of storm drains or culverts where there is little or no tailwater.

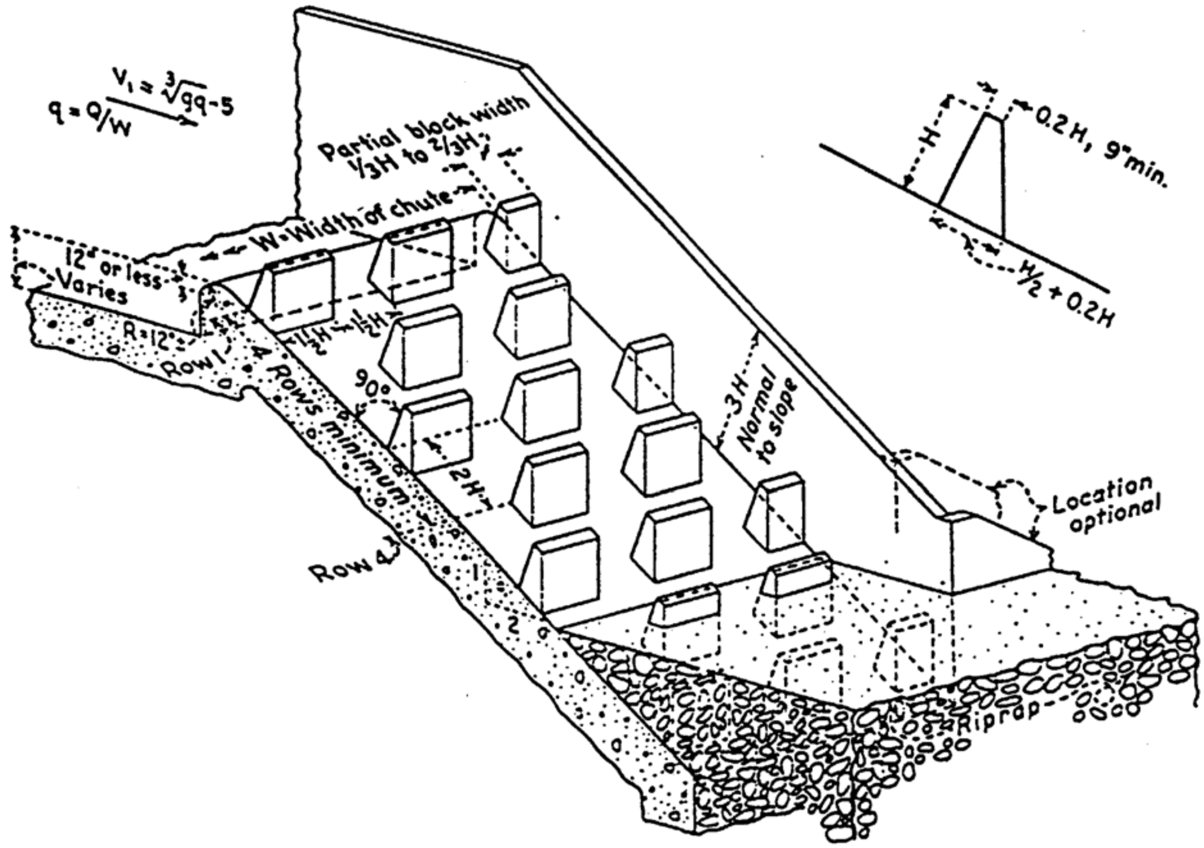
7.4.5.3 Hydraulic Design

1. Open Channel Type Basins: The three different stilling basin configurations can be divided into two categories, basins for spillways or channels (Type III or IX) and basins for pipe outlets (Type VI). A summary of the design data for Type III and IX basins is presented in Figure No. 7.15. The reader is referred to Reference 27 listed in Chapter 14 for a detailed discussion of the design requirements. However, the data in Figure No. 7.15 is generally sufficient for hydraulic design purposes.

2. Baffled Apron Stilling Basin (USBR Type IX): Baffled aprons are used to dissipate the energy in the flow at a drop. They require no initial tailwater to be effective although channel bed scour is not as deep and is less extensive when the tailwater forms a pool into which the flow discharges. The chutes are constructed on an excavated slope, 2:1 or flatter, extending to below the channel bottom. Backfill is placed over one or more rows of baffles to restore the original streambed elevation. When scour or downstream channel degradation occurs, successive rows of baffle piers are exposed to prevent excessive acceleration of the flow entering the channel. If degradation does not occur the scour creates a stilling pool at the downstream end of the chute, stabilizing the scour pattern. The simplified hydraulic design of the baffled apron is shown on Figure No. 7.13.

USBR Type IX Stilling Basin

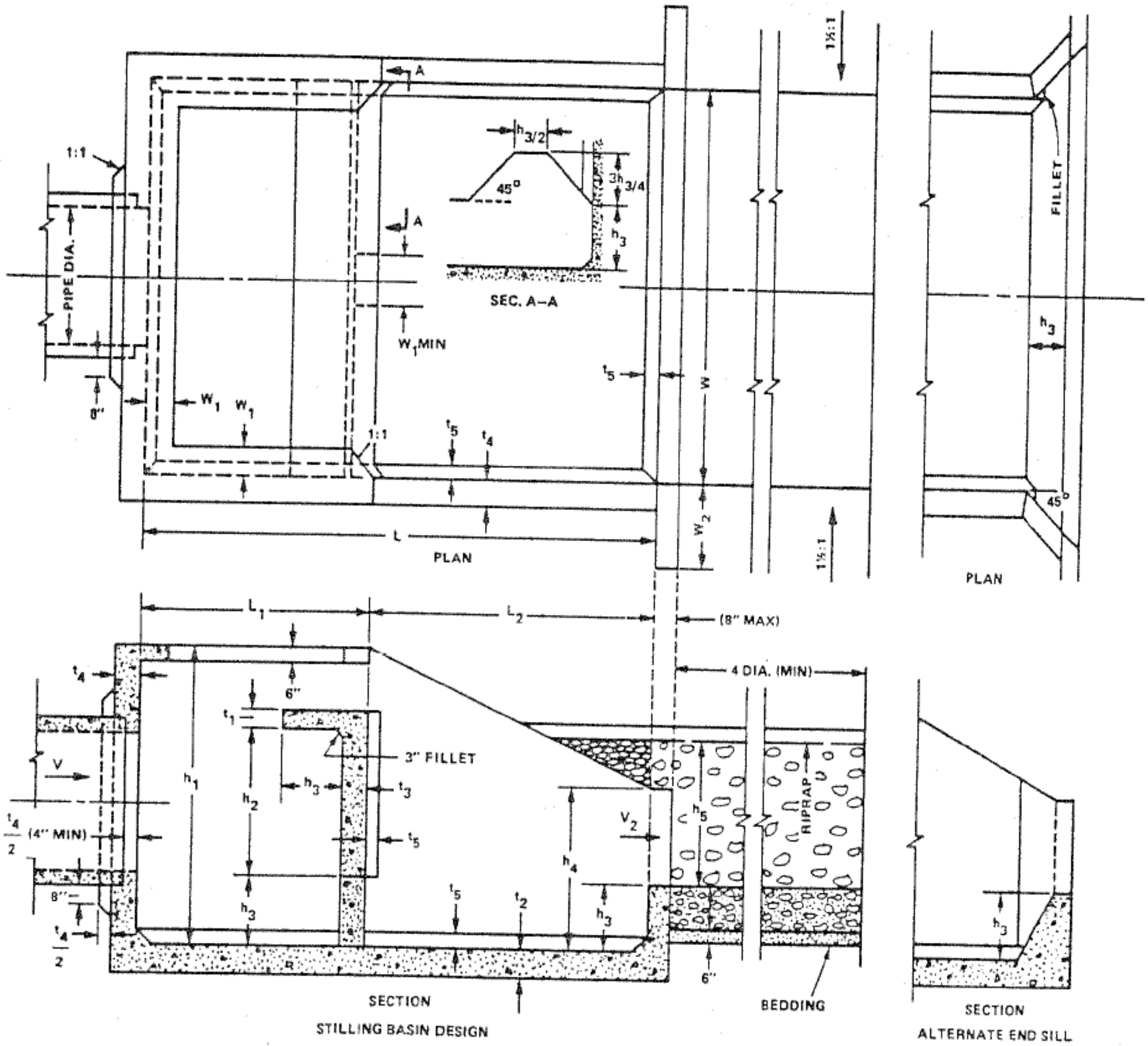
Figure No. 7.13



NOTE: See Figure No. 7.15(c) for design data.

USBR Type VI Stilling Basin

Figure No. 7.14



7.4.5.4 Pipe Outlet Type Basins

The requirement for a stilling basin for a culvert or storm sewer outlet is dependent upon the velocity of the flow. For velocities less than 7 fps a grassed channel with a small amount of riprap for protection against eddy current erosion is sufficient. For velocities less than 16 fps but greater than 7 fps, a formal riprap lining is required. For velocities greater than 16 fps, a Type VI stilling basin is recommended. The design procedure is illustrated by the following example:

Example No. 7.6: Impact Stilling Basin (USBR Type VI)

Given: Pipe Dia. = 48" RCP
Q = 214 cfs
V = 17 fps < 30 fps (upper limit)
Tailwater depth = 2.5 feet
Channel slope = 1.0 percent

Solution:

Step 1: Using the discharge (Q = 214 cfs) enter the DISCHARGE LIMITS portion of Figure No. 7.15 and read the maximum and minimum basin width.

$W_{min} = 12.5$ ft.
$W_{max} = 15.0$ ft.

Step 2: From the BASIC DIMENSION portion of Figure No. 7.15 and using the discharge Q = 214 cfs, interpolate for the basin dimensions. Note that for corresponding pipe size in the table is between a 54-inch and 60-inch diameter, which is larger than the example pipe size of 48-inches. The basin will therefore provide ample room for the example pipe.

W = 12' - 4"	b = 10' - 6"
H = 10' - 3"	c = 5' - 8"
L = 18' - 2"	d = 2' - 4"
A = 7' - 8"	g = 5' - 1"

Step 3: Provide minimum riprap downstream of structure a minimum distance of 3D (or 3 times the pipe rise). Compute the velocity at the structure outlet.

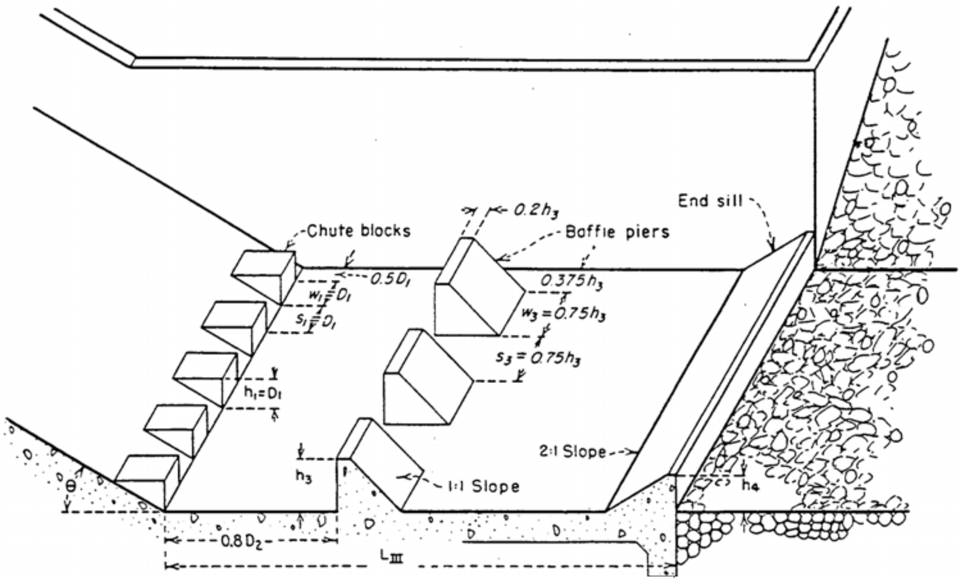
3D = 3 x 4' = 12' riprap length

A	=	W x tailwater depth
	=	12' - 4" x 2.5'
A	=	30.8 ft.^{2}
V	=	Q/A
V	=	6.9 fps

Design Data- USBR Type Stilling Basins

Figure No. 7.15 (a)

SHORT STILLING BASINS FOR CANAL STRUCTURES, SMALL OUTLET WORKS AND SMALL SPILLWAYS (BASIN III)



Jump and basin length reduced about 60 percent with chute blocks, baffle piers, and solid end sill. For use on small spillways, outlet works, small canal structures where V_1 does not exceed 50 – 60' per seconds and Froude number is above 4.5

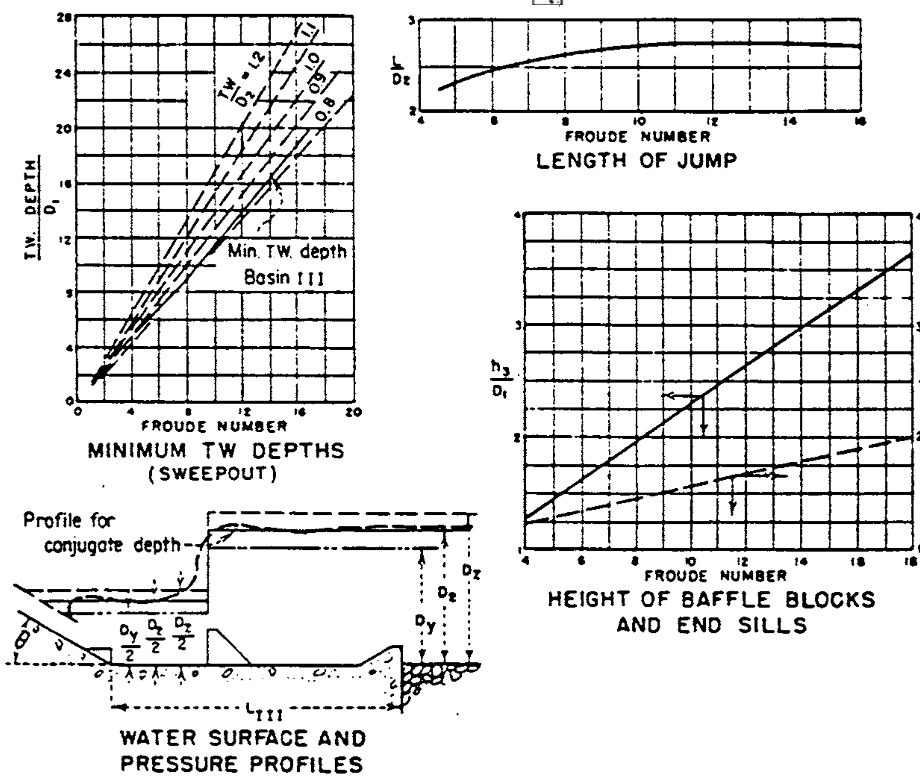
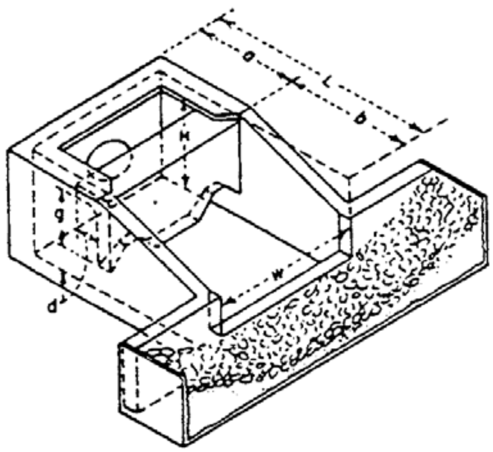


Figure No. 7.15 (b)

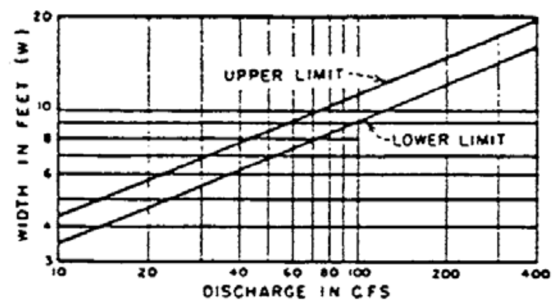
STILLING BASIN FOR PIPE ON OPEN CHANNEL OUTLET (BASIN VI)

For use on pipe or open channel outlets. Sizes and discharges from table V_1 should not exceed 30 feet per second. No tailwater required. Froude number usually 1.5 to 7 but not important
May substitute for Basin IV.
Energy loss greater than in comparable jump

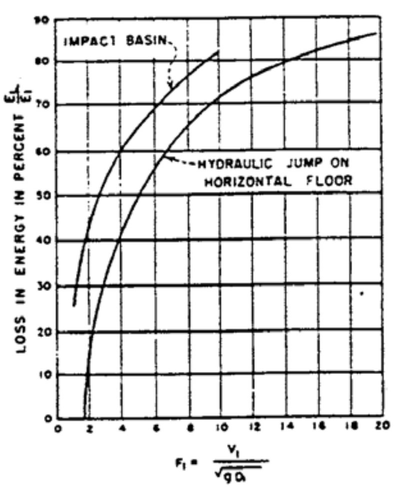


PIPE DIA IN	PIPE AREA SQ FT	Q	FEET AND INCHES W	H	L	a	b	c	d	g
18	1.77	21	5-6	4-3	7-4	3-3	4-1	2-4	0-11	2-1
24	3.14	38	6-9	5-3	9-0	3-11	5-1	2-10	1-2	2-6
30	4.91	59	8-0	6-3	10-8	4-7	6-1	3-4	1-4	3-0
36	7.07	85	9-3	7-3	12-4	5-3	7-1	3-10	1-7	3-6
42	9.62	115	10-6	8-0	14-0	6-0	8-0	4-5	1-9	3-11
48	12.57	151	11-9	9-0	15-8	6-9	8-11	4-11	2-0	4-5
54	15.90	191	13-0	9-9	17-4	7-4	10-0	5-5	2-2	4-11
60	19.63	236	14-3	10-9	19-0	8-0	11-0	5-11	2-5	5-4
72	28.27	339	16-6	12-3	22-0	9-3	12-9	6-11	2-9	6-2

BASIC DIMENSIONS



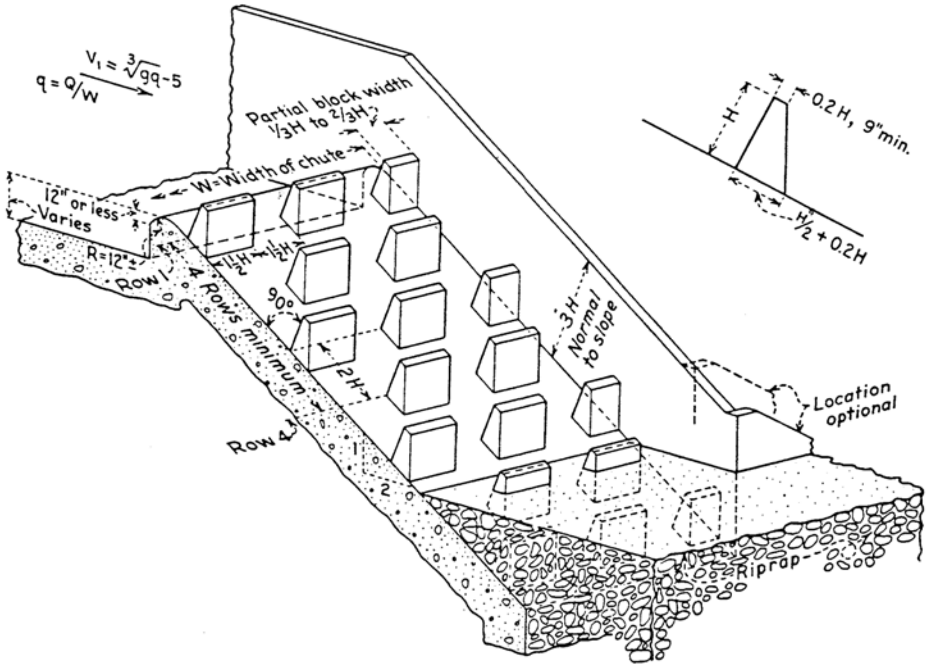
DISCHARGE LIMITS



COMPARISON OF ENERGY LOSSES

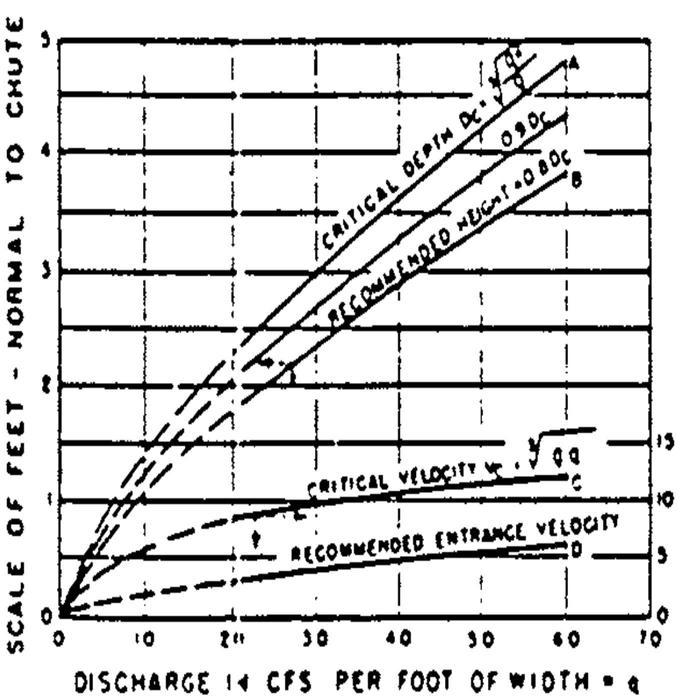
BAFFLED APRON FOR CANAL OR SPILLWAY DROPS (BASIN IX)

Figure No. 7.15 (c)



For use in flow ways where water is to be lowered from one level to another. The baffle piers prevent undue acceleration of the flow as it passes down the chute. Sice the flow velocities entering the downstream channel are relatively low, no stilling basin is required. The chute may be designed to discharge up to 60 cubic feet per second per foot of width, and the drop may be as high as structurally feasible.

BAFFLE PIER HEIGHTS AND ALLOWABLE VELOCITIES



SIMPLIFIED DESIGN PROCEDURE

The baffled apron should be designed for the maximum expected discharge, Q, up to 60 c.f.s per foot of width.

Entrance velocity $v = \sqrt[3]{(qd)}$

Baffle pier Height, H, should be about 0.8 D_c to 0.9D_c, Curve 8 above:

Baffle pier widths and spaces should be equal, up to 3/2 H, but not less than H.

The slope distance between rows of baffle peirs should be 2H, twice the baffle height H.

Four rows of baffle peirs are required to establish full control of the flow, although fewer frows have operated successfully. At least one row of baffles should be furied in the backfill

The chute training walls should be three times as high as the baffle piers

Riprap consisting of 6 0 to 12 inch stones should be placed at the downstream ends of training walls to prevent eddies form undermining the walls.

Step 4: Using the procedures outlined in Section 7.4.6.5 and Table No. 7.13, determine the riprap size required

$$VS^{0.17}/(S_s - 1)^{0.66} = 6.9\,(0.01)^{0.17}/(2.5-1)^{0.66} = 2.4$$

From Table No. 7.13, the minimum riprap size is VL, which must be buried. An alternate would be to use Type M riprap.

7.4.6 Channel Erosion Protection - Riprap

7.4.6.1 General

Riprap has proven to be an effective means to deter erosion along channel banks, in channel bottoms, upstream and downstream from hydraulic structures, at bends, at bridges, and in other areas where erosive tendencies exist. Riprap is a popular choice for erosion protection because the initial installation costs are often less than alternative methods for preventing erosion. However, the design engineer needs to bear in mind there are additional costs associated with riprap erosion protection since riprap installations require frequent inspection and maintenance. Wire enclosed riprap (gabions) in most cases requires complete renovation every 10 to 15 years.

The use of very light (VL) or light (L) types of riprap in an urbanized area have been found to be susceptible to vandalism. The lighter rock is easily displaced by hand and has been completely removed from the project site in some cases.

7.4.6.2 Ordinary Riprap

Ordinary riprap, or simply riprap, refers to a protective blanket of large loose stones, which are usually placed by machine to achieve a desired configuration. The term ordinary riprap has been introduced to differentiate loose stones from grouted riprap and wire enclosed rock, which are discussed later.

Many factors govern the size of the rock necessary to resist the forces tending to move the riprap. For the riprap itself, this includes the size and weight of the individual rocks, the shape of the stones (angular/fractured vs. smooth surface), the gradation of the particles, the blanket thickness, the type of bedding under the riprap, and the slope of the riprap layer. Hydraulic factors affecting riprap include the velocity, current direction, eddy action and waves.

Experience has shown that riprap failures generally result from undersized individual rocks in the maximum size range, an improper gradation of the rock which reduces the interlocking of individual particles and improper bedding for the riprap which allows leaching of channel particles through the riprap blanket.

1. Rock Properties: Rock used for riprap or wire enclosed riprap should be hard,

durable, angular in shape, and free from cracks, overburden, shale and organic matter. Neither breadth nor thickness of a single stone should be less than 1/3 the length and rounded stone should be avoided. The rock should sustain a loss of not more than 40 percent after 500 revolutions in an abrasion test (Los Angeles machine - ASTM C-535-69) and should sustain a loss of not more than 10 percent after 12 cycles of freezing and thawing (AASHTO test 103 for ledge rock procedure A). Rock having a minimum specific gravity of 2.65 is preferred; however, in no case should rock have a specific gravity less than 2.50. In lieu of testing requirements, rock obtained from county or ND Department of Transportation approved quarries may be used.

Classification and gradation for riprap are shown in Table No. 7.9 and are based on minimum specific gravity of 2.50 for the rock. Because of the relatively small size and weight, riprap types VL and L must be buried with native topsoil and revegetated to protect the rock from vandalism.

2. Grouted Riprap: Grouted riprap provides a relatively impervious channel lining which is less likely to be vandalized than dumped riprap. Grouted riprap requires less routine maintenance by reducing silt and trash accumulation and is particularly useful for lining low flow channels and steep banks. The appearance of grouted riprap is enhanced by exposing the tops of individual stones and by cleaning the projecting rocks with a wet broom. Grouted riprap should meet all the requirements for ordinary riprap except that the smallest rock fraction (smaller than the 10% size) should be eliminated from the gradation. A reduction of riprap size by one size designation is permitted for grouted rock (i.e., from Type M to Type L).

As with ordinary riprap, grouted riprap should be placed on an adequate bedding. The recommended minimum grout specifications include entrained air, a 28-day strength of at least 2400 pounds per square inch, and a high slump (5- to 7-inches) in order to penetrate either the full depth of the riprap layer or at least 2-feet where the riprap layer is thicker than 2-feet. Concrete having maximum aggregate size of 3/4-inches may be substituted for grout when using Type M riprap or larger riprap.

Weep holes should be provided at least every 4- to 6-feet at the toe of channel slopes and channel drops to reduce uplift forces on the grouted channel lining.

The grout shall be delivered to the place of final deposit by means that will insure uniformity and prevent segregation of the grout. Placing of grout shall be obtained by pumping under pressure through a 2-inch maximum diameter hose to insure complete penetration of the grout into the rock layer. A vibrator is to be employed near the nozzle during placement to aid the flow of the grout. The excess grout shall be removed by washing to leave a clean rock face exposed. The grouted riprap should resemble a hand placed stone wall or fireplace rock. Grout shall fill the voids to within approximately 4-inches of the riprap surface.

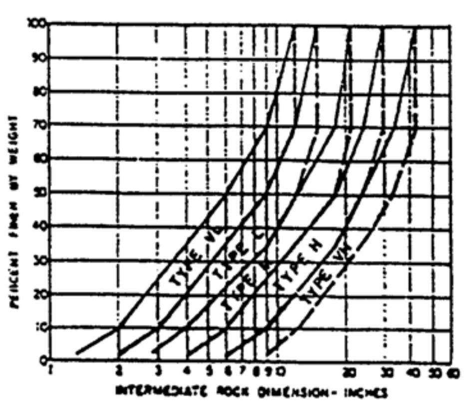
Classification and Gradation of Ordinary Riprap
Table No. 7.9

CLASSIFICATION AND GRADATION OF ORDINARY RIPRAP

RIPRAP DESIGNATION	% SMALLER THAN GIVEN SIZE BY WEIGHT	INTERMEDIATE ROCK DIMENSION (INCHES)	d_{50}* (INCHES)
Type VL	70-100	12	
	50-70	9	
	35-50	6	6**
	2-10	2	
Type L	70-100	15	
	50-70	12	
	35-50	9	9**
	2-10	3	
Type M	70-100	21	
	50-70	18	
	35-50	12	12
	2-10	4	
Type H	100	30	
	50-70	24	
	35-50	18	18
	2-10	6	
Type VH	100	42	
	50-70	33	
	35-50	24	24
	2-10	9	

*d_{50} = Mean particle size

**Bury types VL and L with native top soil and revegetate to protect from vandalism.



7.4.6.3 Wire Enclosed Rock

Wire enclosed rock must be approved for use by the City prior to construction for public facilities. Wire enclosed rock refers to rocks that are bound together in a wire basket so that they act as a single unit, usually referred to as a gabion. One of the major advantages of wire enclosed rock is that it provides an alternative in situations where available rock sizes are too small for ordinary riprap. Another advantage is the versatility that results from the regular geometric shapes of wire enclosed rock. The rectangular blocks and mats can be fashioned into almost any shape that can be formed with concrete. The durability of wire enclosed rock is generally limited by the service life of the galvanized binding wire which, under normal conditions, is considered to be about 15 years. Water carrying silt, sand, or gravel can reduce the service life of the wire; also water which rolls or otherwise moves cobbles and large stones breaks the wire with a hammer and anvil action and considerably shortens the life of the wire. The wire has been found to be susceptible to corrosion by various chemical agents and is particularly affected by high sulfate soils. If corrosive agents are known to be in the water or soil, a plastic coated wire should be specified.

Wire enclosed rock is not maintenance free and must be periodically inspected to determine whether the wire is sound. If breaks are found while they are still relatively small, they may be patched by weaving new strands of wire into the wire cage. Wire enclosed rock installations have been found to attract vandalism. Flat mattress surfaces seem to be particularly susceptible to having wires cut and stones removed. Where possible, mattress surfaces should be buried, as it has been found that wire enclosed rock buried under a few inches of soil is less prone to vandalism. Wire enclosed rock installations require inspection at least once a year under the best circumstances and may require inspection every three months in vandalism prone areas. Mattresses on sloping surfaces must be securely anchored to the surface of the soil.

Rock filler for the wire baskets should meet the rock property requirements for ordinary riprap. Minimum rock sizes and basket dimensions are shown in Table No. 7.10. The maximum stone size should not exceed 2/3 the basket depth or 12-inches, whichever is smaller.

7.4.6.4 Bedding Requirements

Long term stability of riprap and gabion erosion protection is strongly influenced by proper bedding conditions. A large percentage of all riprap failures are directly attributable to bedding failures. A properly designed bedding provides a buffer of intermediate sized material between the channel bed and the riprap to prevent piping of channel particles through the voids in the riprap. Two types of bedding are in common use: 1) a granular bedding filter, and 2) filter fabric.

Table No. 7.10

STANDARD GABION BASKETS (ENGLISH SIZES)

DRAINAGE MANUAL DESIGNATION	LETTER CODE OF SIZE	LENGTH	WIDTH	DEPTH	NUMBER OF DIAPHRAGMS	CAPACITY CUBIC YARDS	MINIMUM ROCK DIMENSION
G36	A	6'	x 3'	x 3'	1	2	4"
	B	9'	x 3'	x 3'	2	3	4"
	C	12'	x 3'	x 3'	3	4	4"
G18	D	6'	x 3'	x 1'-6"	1	1	4"
	E	9'	x 3'	x 1'-6"	2	1.5	4"
	F	12'	x 3'	x 1'-6"	3	2	4"
G12	G	6'	x 3'	x 1'	1	0.66	4"
	H	9'	x 3'	x 1'	2	1	4"
	I	12'	x 3'	x 1'	3	1.33	4"
			SLOPE MATTRESS				
SM9	T	10'	x 6'-6"	x 0'-9"	5	1.80	3"
	U	12'	x 6'-6"	x 0'-9"	6	2.16	3"

1. Granular Bedding: Two methods for establishing gradation requirements for granular bedding are described in this section. The first, a one or two layer bedding, shown in Table No. 7.11, is adequate for most ordinary riprap, grouted riprap or wire encased riprap applications. The second utilizes a design procedure developed by Terzaghi, which is referred to as the T-V (Terzaghi-Vicksburg) design (Reference 10, Chap. 14). The T-V filter criteria establishes an optimum bedding gradation for a specific channel soil. The latter requires channel soil information, including a gradation curve, while the Type I and Type II bedding specifications given in Table No. 7.11 are applicable whether or not soil information is available.

   The Type I and Type II bedding specifications shown in Table No. 7.11 were developed using the T-V filter criteria and the fact that bedding which will protect an underlying noncohesive soil with a mean grain size of 0.045 mm will protect anything finer. Since the T-V filter criterion provides some latitude in establishing bedding gradations, it was possible to make the Type I and Type II bedding specifications conform with most DOT aggregate specifications. The Type I bedding in Table No. 7.11 is designed to be the lower layer in a two layer filter for protecting fine grained soils and has a gradation identical to concrete sand specification AASHTO M-6. Type II bedding, the upper layer in a two layer filter, permits a slightly larger maximum rock fraction. When the channel is excavated in coarse sand and gravel (50 percent or more by weight retained on the No. 40 sieve), only the Type II filter is required, otherwise a two layer bedding (Type I topped by Type II) is required. Alternatively, a single 12-inch layer of Type II bedding can be used except at drop structures. For required bedding thickness, see Table No. 7.12. At drop structures a combination of filter fabric and Type II bedding is acceptable as an alternative to a two layer filter.

   The specifications for the T-V inverse filter relate the gradation of the protective layer (filter) to that of the bed material (base) by the following inequalities:

   $D_{15(filter)} \# 5\ d_{85(base)}$ (Equation 7.13)
   $4\ d_{15(base)} \# D_{15(filter)} \# 20\ d_{15(base)}$ (Equation 7.14)
   $D_{50(filter)} \# 25\ d_{50(base)}$ (Equation 7.15)

   where the capital "D" refers to the filter gain size and the lower case "d" to the base grain size. The subscripts refer to the percent by weight which is finer than the grain size denoted by either "D" or "d". For example, 15 percent of the filter material is finer than $D_{15(filter)}$ and 85 percent of the base material is finer than $d_{85(base)}$

Gradation for Granular Bedding

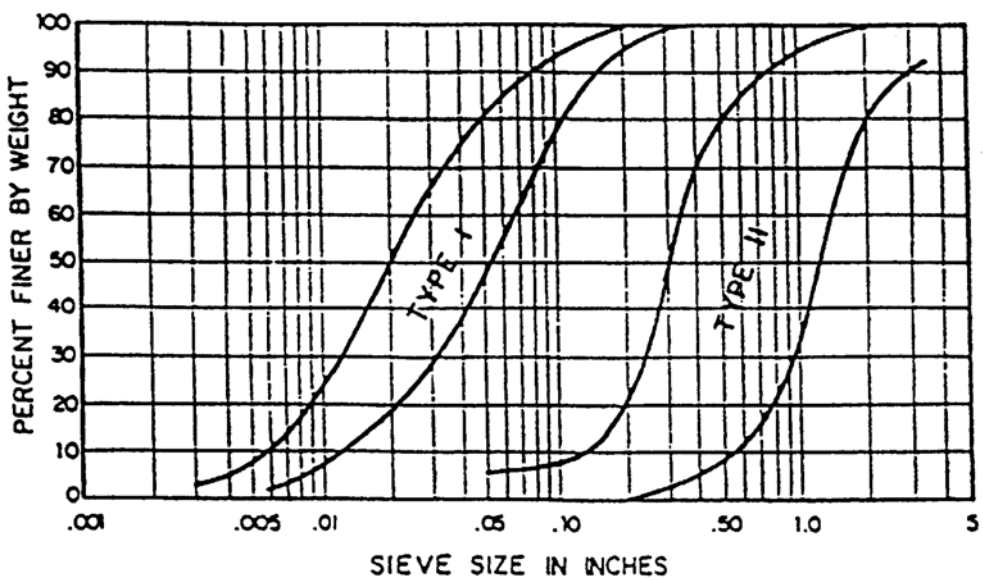
Table No. 7.11

U. S. STANDARD SIEVE SIZE	PERCENT WEIGHT BY PASSING TYPE I	SQUARE MESH SIEVES TYPE II
3"	-	90 - 100
1-1/2"	-	-
3/4"	-	20 - 90
3/8"	100	-
#4	95 - 100	0 - 20
#16	45 - 80	-
#50	10 - 30	-
#100	2 - 10	-
#200	0 - 2	0 - 3

NOTES:

1. For earth with less than 50% passing #40 sieve both Type I and Type II are required.
2. For earth with more than 50% passing #40 sieve, only Type II is required.

FILTER GRADATION LIMITS



Thickness Requirements for Granular Bedding

Table No. 7.12

RIPRAP DESIGNATION	MINIMUM BEDDING THICKNESS (INCHES)		
	FINE GRAINED SOILS		COURSE GRAINED SOILS
	TYPE I	TYPE II	TYPE II
VL, L	4	4	6
M	4	4	6
H	4	6	8
VH	4	6	8

1. Fine grained soils require a two layer filter as noted. A single layer of 12" Type II bedding may be substituted except at check drops.
2. For fine or coarse grained soils the filter requirements may be substituted with a single 4" or 6" layer of Type II bedding and a filter fabric (see fig. 709).
3. Fabric shall not be placed on slopes greater than 2.5:1 where riprap is to be constructed.
4. A course grained soil is defined as having 50% or more by weight retained on the #40 sieve.

2. Filter Fabric: Filter fabric is not a complete substitute for granular bedding. Filter fabric provides filtering action only perpendicular to the fabric and has only a single equivalent pore opening between the channel bed and the riprap. Filter fabric has a relatively smooth surface which provides less resistance to stone movement. As a result, filter fabric is restricted to slopes no steeper than 2.5h to lv. Tears in the fabric greatly reduce its effectiveness so that direct dumping of riprap on the filter fabric is not recommended and care must be exercised during construction. Nonetheless, filter fabric has proven to be an adequate replacement for granular bedding in many instances, especially where small quantities are required. Filter fabric provides an adequate bedding for channel linings along uniform mild sloping channels where leaching forces are primarily perpendicular to the fabric.

At drop structures and sloped channel drops, where seepage forces may run parallel with the fabric and cause piping along the bottom surface of the fabric, special care is required in the use and placement of the filter fabric. See page parallel with the fabric might be reduced by folding the edge of the fabric vertically downward about 2-feet (similar to a cutoff wall) at 12-foot intervals along the installation, particularly at the entrance and exit of the channel reach. Filter fabric should be lapped a minimum of 12 inches at roll edges with upstream fabric being placed on top of downstream fabric at the lap.

Fine silt and clay may clog the openings in the filter fabric, preventing free drainage and increasing failure potential due to uplift. For this reason, a double granular filter is recommended for fine silt and clay channel beds. See Figure No. 7.16 for details on acceptable use of filter fabric as bedding.

7.4.6.5 Ordinary Riprap Channel Linings

Design criteria applicable to ordinary and grouted riprap channel linings are presented in this section.

The Manning's roughness coefficient (n) for hydraulic computations may be estimated for ordinary riprap using:

$$n = .0395\, d_{50}^{0.17} \qquad \text{(Equation 7.16)}$$

in which d_{50} = the mean stone size in feet.

This equation does not apply to grouted riprap (n = .023 to .030), or to very shallow flow (hydraulic radius is less than or equal to 2 times the maximum rock size) where the roughness coefficient will be greater than indicated by the formula (see Table No. 7.5).

1. Rock Size and Lining Dimensions: Table No. 7.13 summarizes riprap requirements for a stable channel lining based on the following relationship which resulted from Smith and Murray's model studies (Reference 10, Chapter 14).

$$VS^{0.17} / (d_{50}^{0.5} (S_s - 1)^{0.66}) = 4.5 \quad \text{(Equation 7.17)}$$

in which,	V	=	mean channel velocity in feet per second
	S	=	longitudinal channel slope in feet per foot
	S_s	=	specific gravity of rock (minimum S_s = 2.50)
	d_{50}	=	rock size in feet for which 50% of the riprap by weight is smaller.

Using the riprap classification of Table No. 7.9 for the d_{50} values and specific gravity of 2.5, Equation 7.17 was rearranged to relate specifically to the riprap rock type (i.e., VL, L, M, H, and VH). Acceptable ranges of velocity and slope within the channel design criteria (i.e., velocity, Froude Number) was established and a direct relationship between the rock type and the flow parameters was developed. Table No. 7.13 presents the riprap requirements for channel linings as a function of slope and velocity.

Table No. 7.13 recognizes that rock size does not need to be increased for steeper channel side slopes, provided the side slopes are no steeper than 2h:1v (Reference 10, Chapter 14). Rock lined side slopes steeper than 2h:1v are not acceptable because of stability, safety, and maintenance considerations. Proper bedding is required both along the side slopes and the channel bottom for a stable lining.

The riprap blanket thickness should be at least 1.75 times d_{50} (at least 2.0 times d_{50} in sandy soils) and should extend up the side slopes at least 1 foot above the design water surface. At the upstream and downstream termination of a riprap lining, the thickness should be increased 50 percent for at least 3 feet to prevent undercutting.

2. Toe Protection: Where only the channel sides are to be lined, additional riprap is needed to provide for long-term stability of the lining. In this case, the riprap blanket should extend at least 3 feet below the existing channel bed and the thickness of the blanket below the existing channel bed increased to at least 3 times d_{50} to accommodate possible channel scour during floods (see Figure No. 7.17). For sandy soils, consult specific criteria for channels on sandy soils.

3. Channel Bends: The potential for erosion increases along the outside bank of a channel bend due to the acceleration of flow velocities on the outside part of the bend. Thus, it is often necessary to provide erosion protection in channels which otherwise would not need protection. In erosion resistant soils, extra protection is not required along bends where the radius is greater than 2 times the top width (as measured for the major flows), but in no case less than 100 feet.

FILTER FABRIC PLACEMENT DETAILS

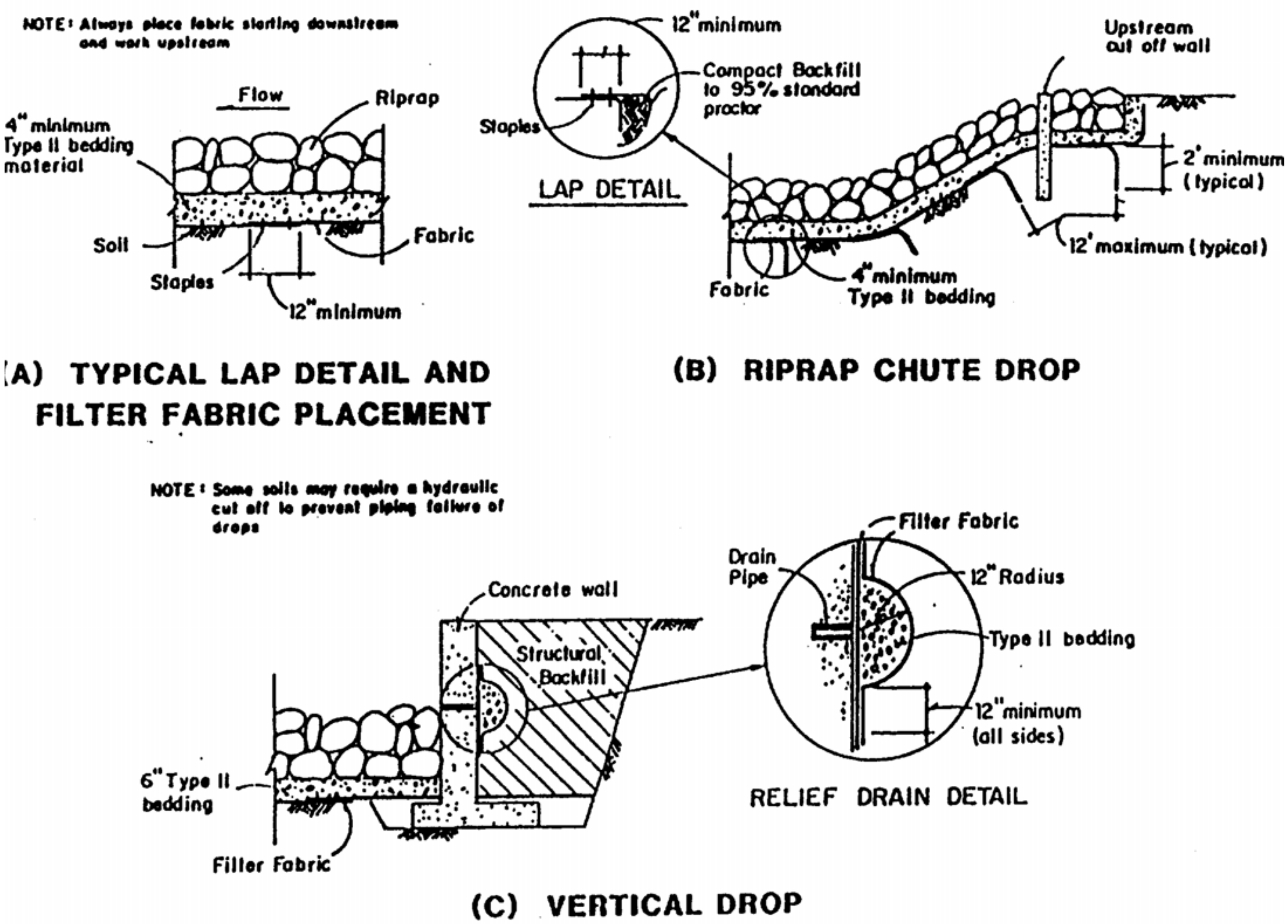


Figure No. 7.16

For bank protection requirements in sandy soils, consult the specific criteria for channels on sandy soils. However, for channels in erosion resistant soils not requiring riprap protection along straight sections, channel bends with radii smaller than stated above require Type L or SM9 riprap protection. Such riprap needs to be covered with native soil and revegetated in accordance with Section 7.4.3.1. The minimum allowable radius for a riprap lined bend is 1.2 times the top width of the design flow water surface and in no case less than 50 feet. The riprap protection should be placed along the outside of the bank and should extend downstream from the bend a distance equal to the length of the bend.

Where the mean channel velocity exceeds the allowable noneroding velocity so that riprap protection is required for straight channel sections, increase the rock size by one category (e.g., Type L to Type M) around bends having a radius less than the greater of the following: 2 times the top width, or 100 feet. The minimum allowable radius for a riprap lined bend in this case is also 1.2 times the top width of the design flow water surface.

4. Transitions: Scour potential is amplified by turbulent eddies in the vicinity of rapid changes in channel geometry such as at transitions and bridges. Table No. 7.13 may be used for selecting riprap protection for subcritical transitions (Froude Numbers 0.8 or less) by increasing the channel velocity by twenty percent (20%). Since the channel velocity varies through a transition, the maximum velocity in the transition should be used in selecting riprap size after it has been increased by 20%.

   Protection should extend upstream from the transition entrance at least 5 feet and extend downstream from the transition exit at least 10 feet.

7.4.6.6 Wire Enclosed Riprap Channel Linings

The geometric properties of wire enclosed rock permit placement in areas where ordinary riprap is either difficult or impractical to place. Proper design and construction is important to successful operation and lifetime performance. Figure No. 7.18 depicts some of the more common ways wire enclosed rock is configured along channel banks. However, wire enclosed riprap lining shall not be used for areas exposed to annual floods, and ordinary riprap is recommended whenever feasible.

The roughness coefficient for slope mattress linings varies from 0.025 to 0.033 depending on the predominant rock size. An n-value of 0.028 is recommended (see Table No. 7.5) based on a rock size of 4-inches. For gabion linings a larger value of n = 0.035 is recommended due to the larger rock size.

If wire enclosed rock lining is used, the toe must be protected by placing riprap at the toe. This is needed to protect against frequently occurring abrasion (see Figure No. 7.17).

Riprap Requirements for Channel Linings

Table No. 7-13

REQUIREMENTS FOR CHANNEL LININGS **

$VS^{0.17}/(S_s-1)^{0.66}$* (FEET 1/2 PER SECOND)	ROCK TYPE ***
1.4 to 3.2	VL
3.3 to 3.9	L
4.0 to 4.5	M
4.6 to 5.5	H
5.6 to 6.4	VH

* Use S sub-s = 2.5 unless the source of rock and the density are known at the time of design. V-in fps and S in ft/ft.

** Table valid only for Froude number of 0.8 or less and side slopes no steeper than 2h:1v.

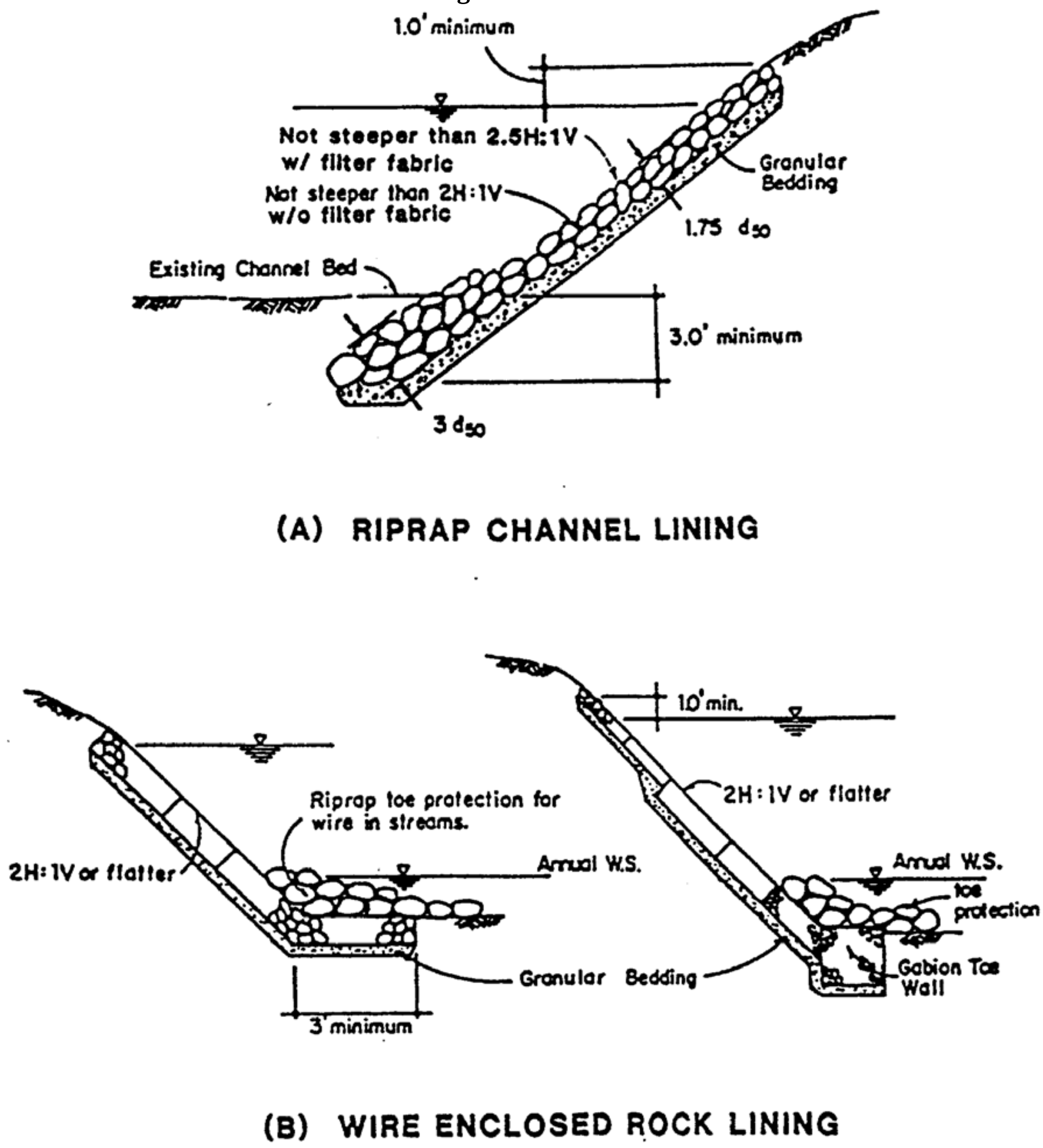
*** Type VL and L riprap shall be buried after placement to reduce vandalism.

SM9 slope mattress with toe protection may be substituted for Type VL or L riprap.

G12 gabion with toe protection may be substituted for Type M and Type H riprap.

Toe Protection for Rock Lined Channels

Figure No. 7.17



Where channel side slopes must exceed 2 h to 1v, gabion gaskets (G36) may be stacked to form a retaining wall as well as erosion protection along the channel banks as shown in Figure No. 7.18. Adjacent baskets should be tied together with heavy gauge wire and adequate protection against channel bed degradation must be provided at the toe of the lining. Stacked baskets must be sloped, or stepped into the bank as shown in Figure No. 7.18. Vertical stacking is not acceptable.

Channel linings should be tied to the channel banks with gabion (G36) counterforts at least every 12 feet. Counterforts should be keyed at least 12 inches into the existing banks with slope mattress linings (see Figure No. 7.18) and should be keyed at least 3 feet by turning the counterfort gabions end-wise when the lining is designed to serve as a retaining wall.

Mattresses and flat gabions on channel side slopes need to be tied to the banks by 2-inch diameter steel pipes driven 4 feet into tight soil (clay) and 6 feet into loose soil (sand) (see Figure No. 7.18). The pipes should be located at the inside corners of basket diaphragms along an upslope (highest) basket wall, so that the stakes are an integral part of the basket. The exact spacing of the stakes depends upon the configuration of the baskets, however, the following is the suggested minimum spacing: stakes every 6 feet along and down the slope, for slopes 2.5 to 1 and steeper; and every 9 feet along and down the slope for slopes flatter than 2.5 to 1. Counterforts are optional with slope mattress linings. Slope mattress staking, however, is required, whether or not counterforts are used.

7.4.6.7 Erosion Protection at Culvert Outlets

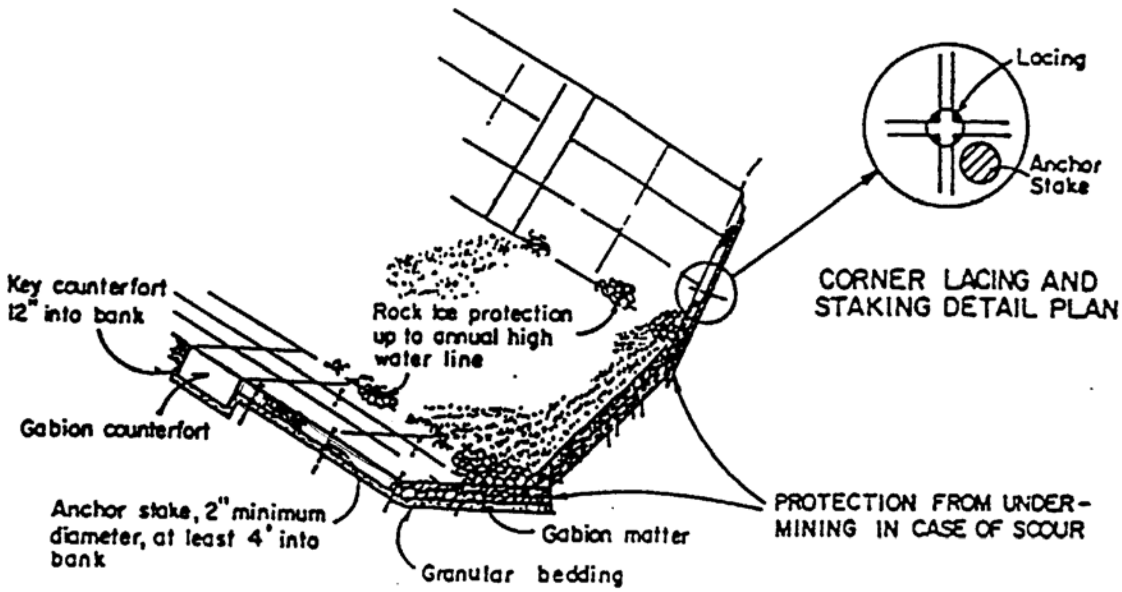
Scour resulting from highly turbulent rapidly decelerating flow is a common problem at conduit outlets. The following riprap protection is suggested for outlet Froude Numbers up to 2.5 (i.e., $(Q/(D^{2.5}))$ or $(Q/(WH^{1.5}))$ up to 14 where the outlet of the conduit slope is parallel with the channel gradient and the conduit outlet invert is flush with the riprap channel protection. Here Q is the discharge in cubic feet per second, D is the diameter of a circular conduit in feet, and W and H are the width and height of a rectangular conduit in feet.

1. Configuration of Protection: Figure No. 7.19 illustrates a typical riprap basin at a conduit outlet. The additional thickness of the riprap just downstream from the outlet is to assure protection from extreme flow conditions which might cause rock movement in this region. Note that protection is required under the conduit barrel and an end slope is provided to accommodate degradation of the downstream channel.

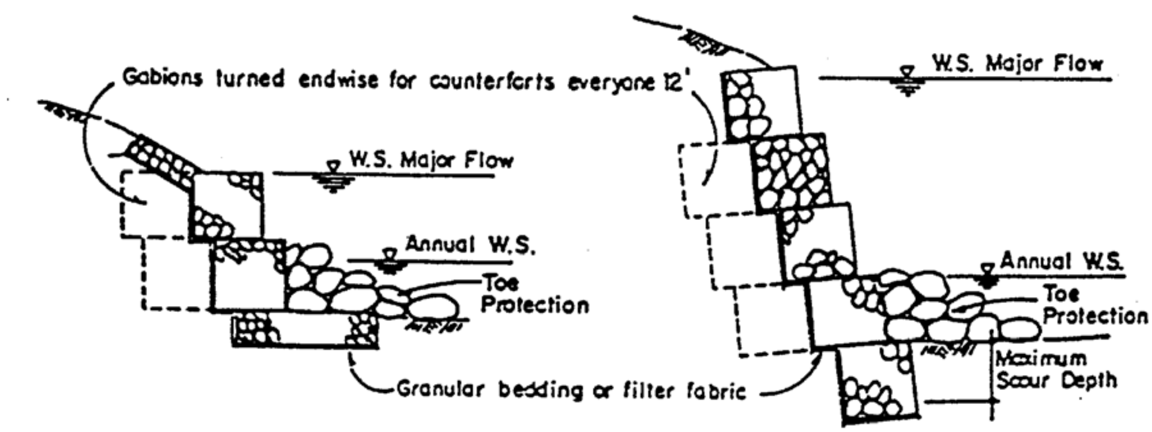
2. Rock Size: The required rock size may be selected from Figure No. 7.20 for circular conduits and from Figure No. 7.21 for rectangular conduits. Figure No. 7.20 is valid for $(Q/ (D^{2.5}))$ of 6.0 or less and Figure No. 7.21 is valid for $(Q/(WH^{1.5})$ of 8.0 or less. The parameters in these two figures are:

Gabion and Slope Mattress Lining Configurations

Figure No. 7.18



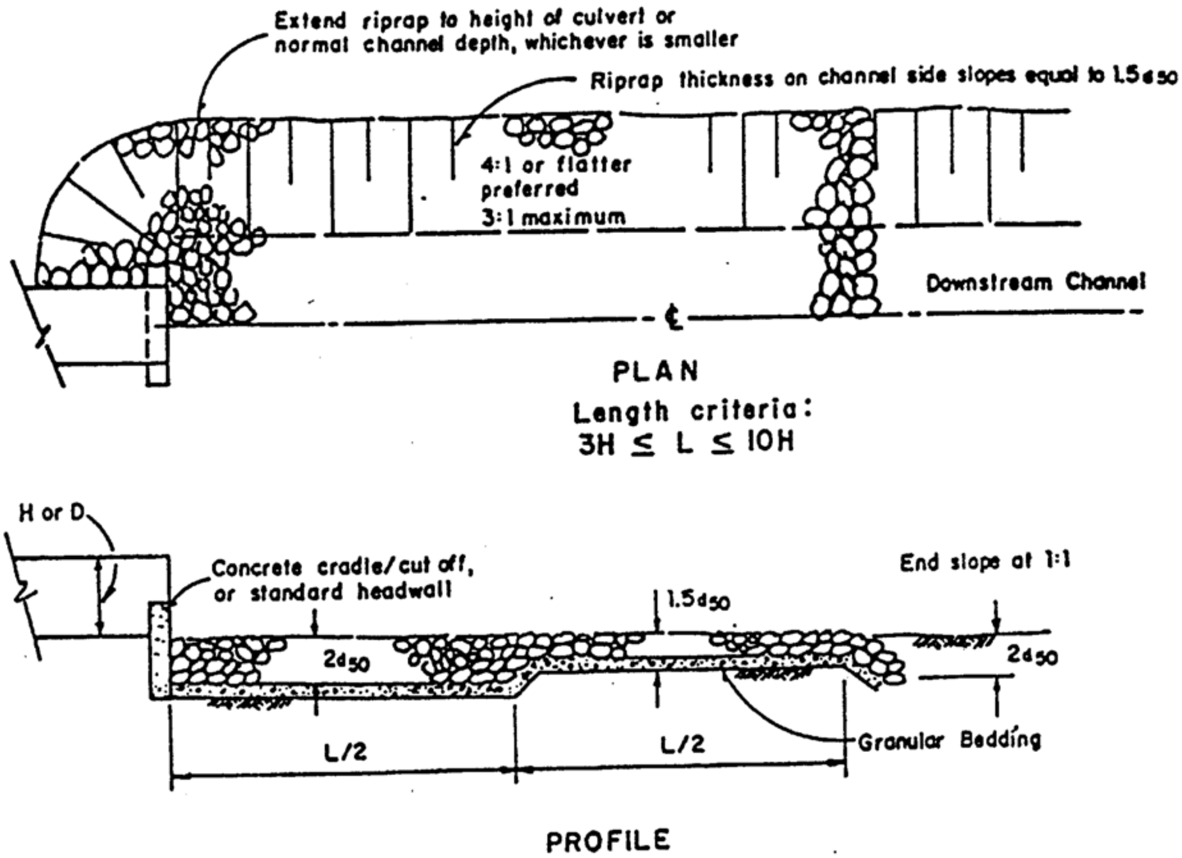
SLOPE MATTRESS LINING



GABION LINING

Conduit Outlet Erosion Protection

Figure No. 7.19



a. $(Q/(D^{1.5}))$ or $(Q/(WH^{0.5}))$ in which Q is the design discharge in cubic feet per second and D is a circular conduit diameter in feet and W and H are the width and height of a rectangular conduit in feet.

b. $Y_{t/D}$ or $Y_{t/H}$ in which Y_t is the tailwater depth in feet, D is the diameter of a circular conduit and H is the height of a rectangular conduit in feet. In cases where Y_t is unknown or a hydraulic jump is suspected downstream of the outlet, use $Y_{t/D} = Y_{t/H} = 0.40$ when using Figures No. 7.20 and 7.21.

c. The riprap size requirements in Figures No. 7.20 and 7.21 are based on the nondimensional parametric equations 7.18 and 7.19.
(Reference 10, Chapter 14)

Circular Culvert:

$$(d_{50}/D)(Y_t/D)^{1.2} / (Q/D^{2.5}) = 0.023 \qquad \text{(Equation 7.18)}$$

Rectangular Culvert:

$$(d_{50}/D)(Y_t/H) / (Q/WH^{1.5}) = 0.014 \qquad \text{(Equation 7.19)}$$

The rock size requirements were determined assuming that the flow in the culvert barrel is not supercritical. Equations 7.18 and 7.19 can be used when the flow in the culvert is less than pipe full and is supercritical if the value of D or H is modified for use with Figures No. 7.20 and No. 7.21. Whenever the flow is supercritical in the culvert, substitute the average depth (D_a) for D and average height (H_a) for H, in which D_a is defined as:

$$D_a = 1/2(D + Y_n) \qquad \text{(Equation 7.20)}$$

in which maximum D_a shall not exceed D, and

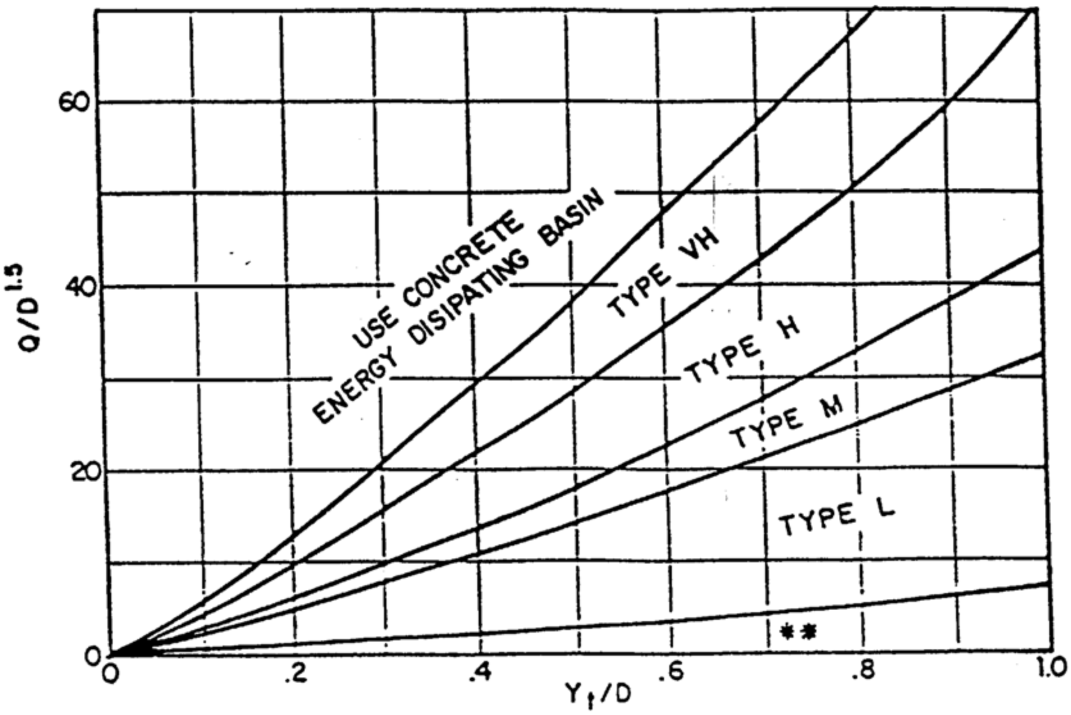
$$H_a = 1/2(H + Y_n) \qquad \text{(Equation 7.21)}$$

in which maximum H_a shall not exceed H, and

D_a = A parameter to be used in Figure No. 7.20 whenever the culvert flow is supercritical.
D = Diameter of a circular culvert in feet.
H_a = A parameter to be used in Figure No. 7.21 whenever the culvert flow is supercritical.
H = Height of a rectangular culvert in feet.
Y_n = Normal depth of supercritical flow in the culvert.

Riprap Erosion Protection at Circular Conduit Outlet

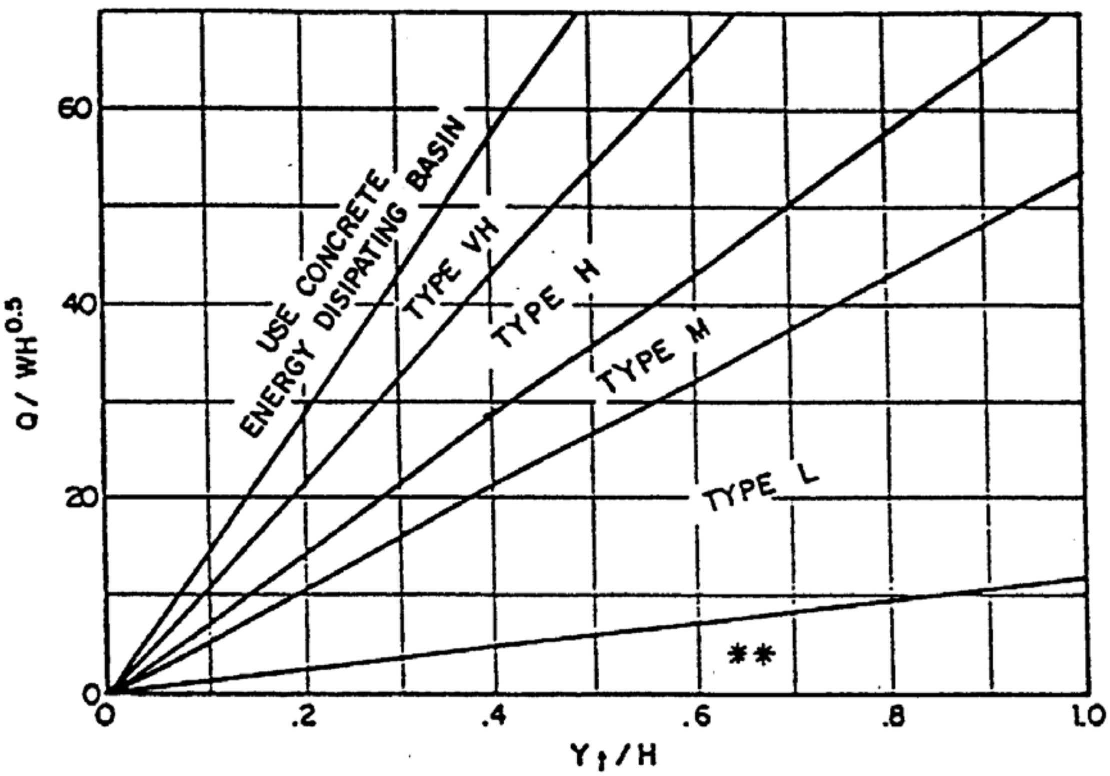
Figure No. 7.20



Use D_a instead of D whenever flow is supercritical in the barrel.
** Use Type L for a distance of 3D downstream.

Riprap Erosion Protection at Rectangular Conduit Outlet

Figure No. 7.21



Use H_a instead of H whenever culvert has supercritical flow in the barrel.
**Use Type L for a distance of 3H downstream.

3. Extent of Protection: The length of the riprap protection downstream from the outlet depends on the degree of protection desired. To prevent all erosion, the riprap must be continued until the velocity has been reduced to an acceptable value. For purposes of outlet protection during major floods, the acceptable velocity is set at 5 fps for very erosive soils, and 7 fps for erosion resistant soils. The rate at which the velocity of a jet from a conduit outlet decreases is not well known. For the procedure recommended here, the velocity decrease is assumed to be related to the angle of lateral expansions, σ, of the jet. The velocity is related to the expansion factor, $(1/(2\tan\sigma))$, which may be determined directly using Figures No. 7.22 and No. 7.23.

Assuming that the expanding jet has a rectangular shape:

$$L = (1/(2\tan\sigma))(A_t/Y_t - W) \qquad \text{(Equation 7.22)}$$

In which:

L	=	length of protection in feet,
W	=	width of the conduit in feet (use diameter for circular conduits),
Y_t	=	tailwater depth in feet,
σ	=	the expansion angle of the culvert flow.
A_t	=	Q/V (Equation 7.23)
Q	=	design discharge in cubic feet per second
V	=	the allowable non-eroding velocity in the downstream channel in feet per second.
A_t	=	required area of flow at allowable velocity in square feet.

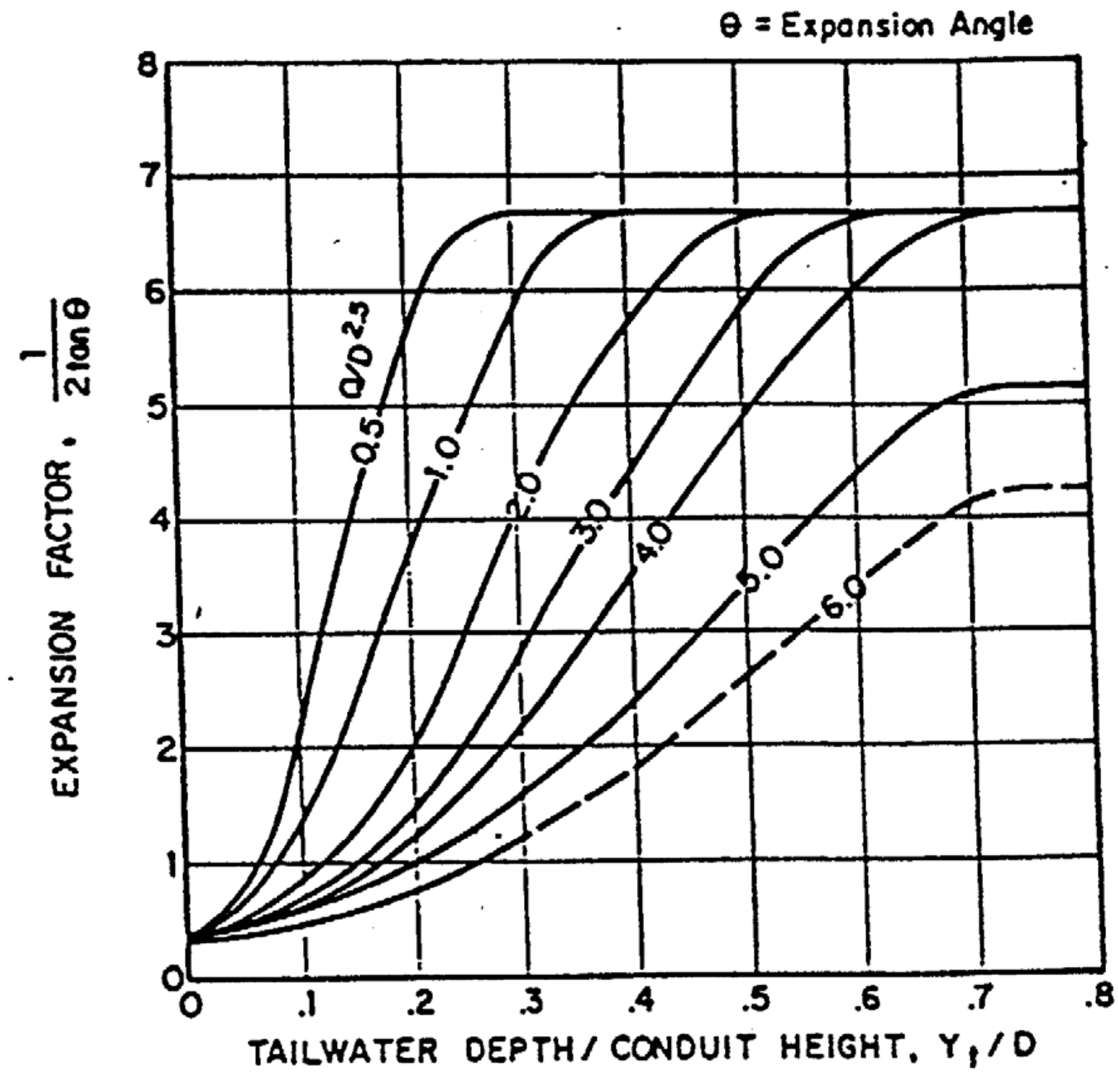
In certain circumstances, Equation 7.22 may yield unreasonable results. Therefore in no case should L be less than 3D or 3H, nor does L need to be greater than 10D or 10H whenever the Froude parameter $(Q/(WH^{1.5}))$ or $(Q/(D^{2.5}))$ is less than 8 or 6, respectively. Whenever the Froude parameter is greater than these maximums, increase the maximum L required by one-fourth D or H for each whole number the Froude parameter is greater than 8 or 6 for rectangular or circular pipe, respectively.

4. Multiple Conduits: The procedures outlined in the sections above can be used to design outlet erosion protection for multi-barrel culvert installations by hypothetically replacing the multiple barrels with a single hydraulically equivalent rectangular conduit. The dimensions of the equivalent conduit may be established as follows. First, distribute the total discharge, Q, among the individual conduits. Where all the conduits are hydraulically similar and identically situated, the flow can be assumed to be equally distributed, otherwise, the flow through each barrel must be computed. Next, compute the Froude parameter $(Q_i)/(D_i)^{2.5})$ (circular conduit) or $(Q_i)/(W_i)(H_i^{1.5})$ (rectangular conduit), where the subscript "i" indicates the discharge and dimensions associated with an individual conduit.

If the installation includes dissimilar conduits, select the conduit with the largest value of the Froude parameter to determine the dimensions of the equivalent conduit. Make the height of the equivalent conduit, H_e, equal to the height, or diameter, of the selected individual conduit. The width of the equivalent conduit, W_e, is determined by equating the Froude parameter from the selected individual conduit with the Froude parameter associated with the equivalent conduit, $(Q_e)/ (W_e)((H_e)^{1.5})$.

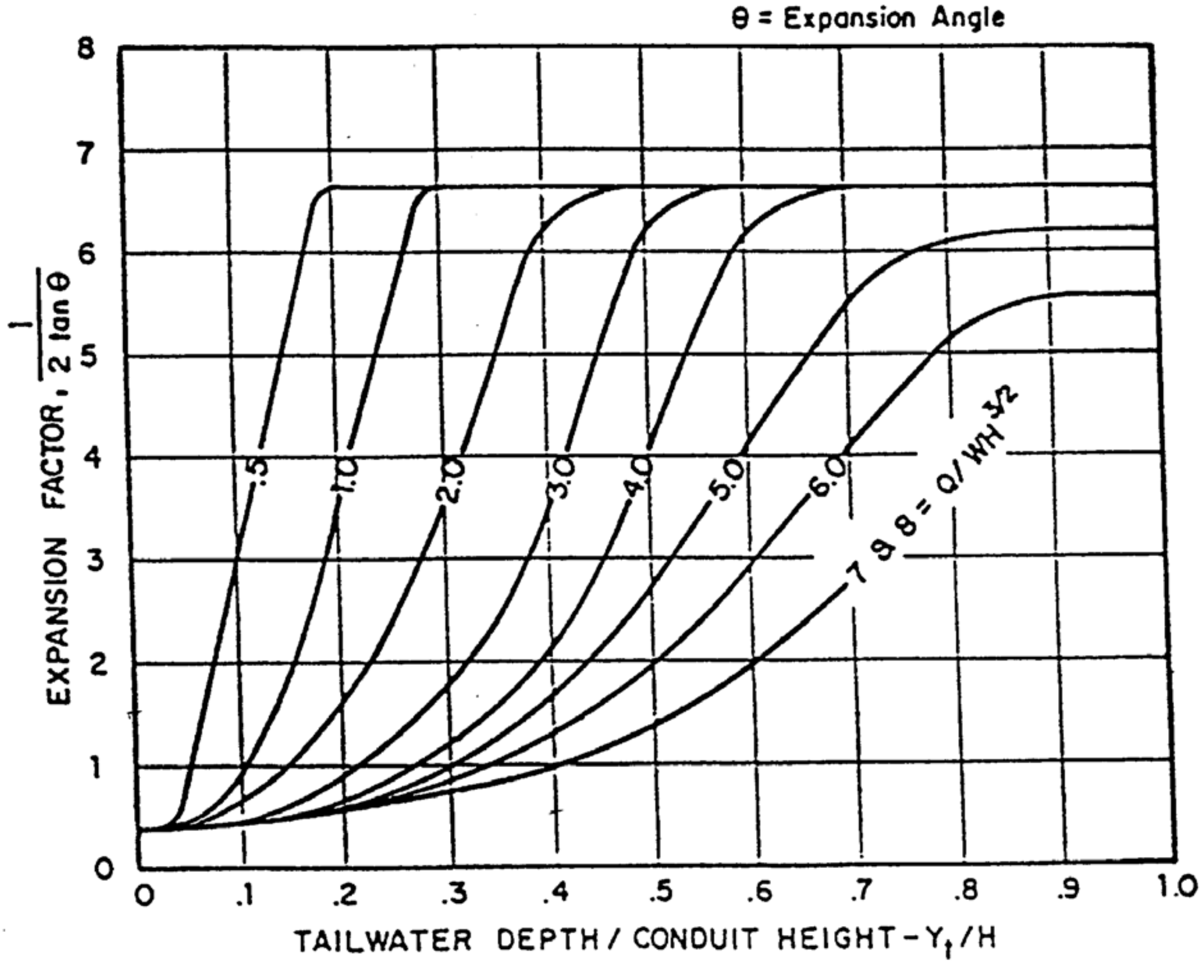
Expansion Factor for Circular Conduits

Figure No. 7.22



Expansion Factor for Rectangular Conduits

Figure No. 7.23



CHAPTER 8
CULVERT AND BRIDGE DESIGN

8.1 INTRODUCTION

The criteria presented in this section shall be used in the evaluation and design of culverts and bridges for public streets, roads, and highways. The review of all planning submittals (Chapter 2) will be based on the criteria presented herein.

Culverts are defined as a closed conduit for the passage of water under an embankment, such as a road, railroad, or canal. The distinction between a culvert and a storm sewer is the means by which flow enters the conduit, although culverts are also generally shorter in length than storm sewers. Runoff enters a storm sewer by means of stormwater inlets above the storm sewer, whereas runoff enters a culvert by an open channel, generally at a similar elevation. The geometry of the inlet area plays a major role in determining the required size or capacity of the culvert.

8.2 DESIGN STANDARDS FOR CULVERTS

8.2.1 Culvert Materials

All culverts within the City of Bismarck and its extraterritorial jurisdiction shall be constructed of reinforced concrete or corrugated steel. Aluminum or plastic culverts will not be allowed unless approved by the City Engineer. The materials, pipes, or appurtenances shall meet one or more of the following standards:

Culvert Standards

Material Pipe	Standard
Reinforced Concrete Pipe-Round	ASTM C-76 or AASHTO M-170
Reinforced Concrete Pipe-Elliptical	ASTM C-507 or AASHTO M-207
Reinforced Concrete Pipe-Joints	ASTM C-443 or AASHTO M-198
Reinforced Concrete Pipe-Arch	ASTM C-506 or AASHTO M-206
Precast Concrete Manholes	ASTM C-478 or AASHTO M-199
Precast Concrete Box Culverts	ASTM C-789/C-850 or AASHTO M-259/M-273
Concrete for Cast-in-place Culverts	CDOH 601
Corrugated Steel Pipe-Galvanized	AASHTO M-36
Corrugated Steel Pipe-Coated	AASHTO M-190
Structural Plate	AASHTO M-167

8.2.2 Culvert Location and Alignment

A culvert should ideally be located in the existing channel bed to minimize costs associated with structural excavation and channel work. However, this is not always possible. Some streambeds are sinuous and cannot accommodate a straight culvert. In other situations, a stream channel may have to be relocated to avoid the installation of an inordinately long culvert. When relocating a stream channel, it is best to avoid abrupt transitions in the stream alignment at either end of the culvert. Generally, it is best to align the culvert to fit as nearly as possible the natural alignment of the channel.

Culvert inverts should be set at the design grade of the inlet and outlet channel. If a channel needs to be cleaned and maintained from time to time, then the design channel grade may be lower than the existing grade. Culvert inverts set too low will result in sediment deposition within the culvert barrel and result in loss of capacity. Culvert inverts set too high above the design channel grade may result in objectionable ponding and silt deposition upstream.

Minimum culvert longitudinal grades shall be 1%. The maximum allowable length of culvert installations will be 200 feet, unless the additional length is approved by the City Engineer.

The culvert structural requirements shall be as follows:

1. Reinforced Concrete Pipe. RCP shall be a minimum strength of Class 2 under parking lots and shoulders and for 27 inch or larger diameters, Class 3 for diameters less than 24 inches under all streets and entrance pavement, and Class 5 under railroad tracks or jacked pipe.

2. Corrugated Steel Pipe. The required gage of CSP shall be approved by the City Engineer.

3. Structural Plate Pipe. The required gage for SPP or SPPA is to be approved by the City Engineer.

4. Minimum cover over the top of all culverts shall be 12" or as recommended by the manufacturer.

8.2.3 Physical and Structural Requirements

Culvert inverts should be set at the design grade of the inlet and outlet channel. If a channel needs to be cleaned and maintained from time to time, then the design channel grade may be lower than the existing grade. Culvert inverts set too low will result in sediment deposition within the culvert barrel and result in loss of capacity. Culvert inverts set too high above the design channel grade may result in objectionable ponding and silt deposition upstream.

The minimum culvert longitudinal grades shall be 1%. The maximum allowable length of culvert installations will be 200 feet, unless the additional length is approved by the City Engineer.

The culvert structural requirements shall be as follows:

1. Reinforced Concrete Pipe. RCP shall be a minimum strength of Class 3 under all streets and entrance pavement, and Class 5 under railroad tracks.

2. Corrugated Steel Pipe. The required gage of CSP shall be approved by the City Engineer.

3. Structural Plate Pipe. The required gage for SPP or SPPA is to be approved by the City Engineer.

4. Minimum cover over the top of all culverts shall be 12" or as recommended by the manufacturer.

8.2.4 Inlet and Outlet Requirements

For public roads and driveway culverts larger than 18", the culverts are to be designed with protection at the inlet and outlet areas. The culvert inlet shall include a headwall with wingwalls or a flared end section. Headwalls or end sections are to be located a sufficient distance from the edge of the shoulder or back of walk to allow for a maximum slope of 3 horizontal to 1 vertical to the back of the structure. This distance may be increased in order to satisfy recovery zone requirements of the NDDOT. The outlet area shall also include a headwall with wingwalls or an end section in addition to riprap protection as required in Chapter 7, Section 7.4.6.7. When outlet velocities exceed the limitations set forth in Section 8.2.5.1.4, an energy dissipator may be required.

8.2.5 Hydraulic Criteria

8.2.5.1 Culvert Sizing Criteria

There are three factors to be considered in the sizing of a culvert: the minimum design frequency, the minimum culvert size, and the allowable cross street flow in the street for which the culvert is being considered (allowable headwater elevation).

1. Design Frequency. The minimum design frequency for a culvert is dependent on the drainage classification of the street and the policy set forth in Chapter 3 - Stormwater Hydrology.

2. Minimum Size. The minimum culvert size shall be 18-inch diameter round or a 22" x 13" arch, or 23" x 14" elliptical.

3. Allowable Headwater. The maximum headwater for the 100-year design flow shall be 1.5 times the culvert diameter or culvert rise dimension for shapes other than round.

4. Allowable Cross Street Flow. When the flow in a channel exceeds the capacity of the culvert and overtops the cross street, the flow in the street must not exceed the limits set forth for the major storm (i.e., 100-year). If the cross street flow exceeds the limits for the minimum design frequency and/or the minimum culvert size, then the culvert must be increased until all criteria are met.

5. Velocity Limitations. In design of culverts both the minimum and maximum velocities must be considered. A minimum velocity of flow is required to assure a self-cleansing condition of the culvert. A minimum velocity in the culvert of 3-feet per second at the outlet is recommended.

The maximum velocity in a culvert is controlled by two factors, the channel protection provided at the outlet and the maximum allowable headwater. If the outlet velocities are less than 7 fps, then only the minimum amount of protection is required due to the eddy currents generated by the flow transition. As the outlet velocities increase, additional protection is required, such as more extensive riprap or an energy dissipation structure.

The limit placed on headwater at the entrance controls the maximum velocity that will occur in the culvert. If all the flow depth above the top of the culvert is converted to kinetic energy (velocity head), a theoretical maximum velocity is obtained.

The following requirements are placed on the outlet velocity of a culvert:

CULVERT OUTLET PROTECTION

F < 7 FPS	7 FPS < V < 16 FPS	V > 16 FPS
Minimum Riprap Protection	Riprap Protection or Energy Dissipator	Energy Dissipator

6. Hydraulic Data. The hydraulic data presented in Table No. 8.1 shall be used in the design and evaluation of culverts. The culvert capacity shall be computed using the Standard Form shown in Figure No. 8.1 or an equivalent.

8.3 CULVERT HYDRAULICS

8.3.1 Introduction

Presented in this section are the general procedures for hydraulic design and evaluation of culverts. The design engineer is assumed to possess a basic working knowledge of culvert hydraulics and is encouraged to review text books and other technical literature on the subject.

8.3.2 Inlet versus Outlet Control

There are two primary types of culvert flow, inlet control and outlet control. Under inlet control the cross-sectional area of the barrel, the inlet configuration or geometry, and the amount of headwater are all factors affecting capacity. Outlet control involves the additional consideration of the tailwater in the outlet channel and the slope, roughness and length of the barrel. Under inlet control conditions, the slope of the culvert is steep enough so that the culvert does not flow full and tailwater does not affect the flows.

8.3.3 Nomograph Methods

A. Inlet Control Condition. Inlet control for culverts may occur in two ways (see Figure No. 8.2).

1. Unsubmerged. The headwater is not sufficient to submerge the top of the culvert and the culvert inlet slope is supercritical. The culvert inlet acts like a weir (Condition A, Figure No. 8.2).

2. Submerged. The headwater submerges the top of the culvert but the pipe does not flow full. The culvert inlet acts like an orifice. (Condition B, Figure No. 8.2).

In the submerged inlet condition, the equation governing the culvert capacity is the orifice flow equation:

$$Q = C_d A \sqrt{2gh} \qquad \text{(Equation 8.1)}$$

where:	Q	=	Flow (cfs)
	C_d	=	Orifice coefficient
	A	=	Area (ft.^{2})
	g	=	Gravitational constant
	h	=	Head on culvert measured from centerline (ft.)

The orifice coefficient, C_d, varies with head on the culvert as well as the culvert type and entrance geometry. The inlet control rating for several culvert materials, shapes, and inlet configurations are presented in Figures No. 8.3 to No. 8.8. These nomographs were developed empirically by the pipe manufacturers, Bureau of Public Roads, and the Federal Highway Administration and are recommended for use in the Bismarck area, rather than the Equation 8.1, due to the uncertainty in estimating the orifice coefficient. The orifice flow equation, however, is used to size outlets for stormwater detention facilities. Refer to Chapter 9 for a discussion of detention outlets.

B. Outlet Control Condition. Outlet control will govern if the headwater is deep enough, the culvert slope sufficiently flat, and the culvert sufficiently long. There are three types of outlet control culvert flow conditions:

1. The headwater submerges the culvert top, and the culvert outlet is submerged under the tailwater. The culvert will flow full (Condition A, Figure No. 8.2).

2. The headwater submerges the top of the culvert and the culvert is unsubmerged by the tailwater (Condition B or C, Figure No. 8.2).

3. The headwater is insufficient to submerge the top of the culvert. The culvert slope is subcritical and the tailwater depth is lower than the pipe critical depth (Condition D, Figure No. 8.2).

The factors affecting the capacity of a culvert in outlet control include the inlet geometry and associated losses, the culvert material with friction losses, and the tailwater condition.

The capacity of the culvert is calculated using the conservation of energy principle (Bernoulli's Equation). An energy balance is determined between the headwater at the culvert inlet and at the culvert outlet, which includes the inlet losses, the friction losses, and the velocity head (see Figure No. 8.9). The equation is then expressed as:

$$H = h_e + h_f + h_v \qquad \text{(Equation 8.2)}$$

where:

H	=	total energy head (ft.)	
h_e	=	entrance head losses (ft.)	
h_f	=	friction losses (ft.)	
h_v	=	velocity head (ft.) = $V^2/2g$	(Equation 8.3)

For inlet losses the governing equation is:

$$h_e = K_e V^2/2g \qquad \text{(Equation 8.4)}$$

where K_e is the entrance loss coefficient. Typical inlet loss coefficients recommended for use are given in Table No. 8.1D.

Friction loss is the energy required to overcome the roughness of the culvert and is expressed as follows:

$$h_f = (29n^2L/R^{1.33})(V^2/2g) \qquad \text{(Equation 8.5)}$$

where:

n	=	Manning's coefficient (see Table No. 8.1)
L	=	Length of culvert (ft.)
R	=	Hydraulic radius (ft.)
V	=	Velocity of flow (fps)

Combining the Equations 8.2, 8.4, and 8.5 and simplifying the terms results in the following equation:

$$H = (K_e + 1 + 29n^2L/R^{1.33})\,(V^2/2g) \qquad \text{(Equation 8.6)}$$

Equation 8.6 can be used to calculate the culvert capacity directly when the culvert is flowing under outlet conditions A or B as shown on Figure No. 8.2. For conditions C or D, the HGL at the outlet is approximated by averaging the critical depth and the culvert diameter, which is used if the value is greater than the tailwater depth (Tw) to compute headwater depth (Hw).

A series of outlet control nomographs for various culvert materials and shapes have been developed by the pipe manufacturers, Bureau of Public Roads, and the Federal Highway Administration. The nomographs are presented in Figures No. 8.10 to No. 8.16. When rating a culvert, either the outlet control nomographs or Equation 8.6 can be used to calculate the headwater requirements. Critical flow depths for various culvert shapes are presented in Figures No. 8.17 to No. 8.21.

When using the outlet nomographs for corrugated steel pipe, the data must be adjusted to account for the variation in the n-value between the nomographs and the culvert being evaluated. The adjustment is made by calculating an equivalent length according to the following equation:

$L^1 = L\,(n^1/n)$ (Equation 8.7)

L^1 = Equivalent length

L = Actual length

n = Value of Manning's n-value shown on Figure No. 8.1 to 8.16

n^1 = Actual n-value of culvert

The actual n-value of the culvert can be obtained from Table No. 8.1. The n-value for the figures is listed at the bottom of the figure, along with typical length adjustment for various n-values.

8.3.4 Computer Software

The Federal Highway Administration has sponsored the development of a software package, HY8, which has the ability to analyze culvert installations of all types and configurations. The procedures followed are in accordance with Federal Highway Administration Hydraulic Design Series No. 5-Hydraulic Design of Highway Culverts. The nomographs contained in this chapter for the design of culverts have been extracted from HDS No. 5. Computer software performs the same analysis as with the nomograph procedure.

The computer software HY8 is able to handle very complex culvert installation problems. Multiple pipe installations consisting of different kinds of materials, different invert elevations, and different shaped sections are handled with little problem. In addition, the Engineer can analyze road overtopping situations and scour protection at culvert outlets.

8.4 DESIGN STANDARDS FOR BRIDGES

8.4.1 State Standards

Design standards for bridges within the City of Bismarck and its extraterritorial jurisdiction will be in accordance with the bridge design and construction standards published by the North Dakota Department of Transportation. Included with these standards are criteria for geometric design of bridges, structural design, foundation design, and hydraulic design.

8.4.2 Bridge Sizing Criteria

In addition to the criteria set forth in NDDOT Standards, the following criteria shall apply:

Low chord shall be a minimum of 1-foot above the 100-year water surface elevation or above the energy grade line (EGL), whichever is greater. The waterway section at the bridge shall be sized so as not to cause a significant rise (1-foot) in the 100-year water surface elevation or cause flow to accelerate to velocities sufficient to scour and undermine the bridge abutments and wingwalls. The City Engineer reserves the right to require lower allowable stage increases to protect upstream property.

8.4.3 Velocity Limitations

The velocity limitations through the bridge opening are controlled by the potential abutment scour and subsequent erosion protection provided. Using the more readily available riprap Type M for the channel lining and/or protection of the abutments and wingwalls, then the maximum channel velocity is between 8 to 10 fps depending on channel slope. For consistency with the culvert design and as a practical limit on the flow energy, a maximum velocity of 10 fps shall be allowed through a bridge.

8.4.4 Hydraulic Analysis

The design calculations for all bridges must be prepared and certified by a Professional Engineer Registered in the State of North Dakota. The procedures for design as outlined in the publication "Hydraulics of Bridge Waterways" shall be used for the design and supplemented by a HEC-2 Backwater Analysis to verify the resulting hydraulic data.

8.4.5 Inlet and Outlet Configuration

The design of all bridges shall include adequate wingwalls of sufficient length to prevent abutment erosion and to provide slope stabilization from the embankment to the channel. Erosion protection on the inlet and outlet transition slopes shall be provided to protect from the erosive forces of eddy current.

Hydraulic Date for Culverts

Table No. 8.1

(A) Manning's n-values for Corrugated Steel Pipe

Corrugations	Annular	Helical						
	2 2/2" x 1/2"	1 1/2" x 1/2" [11,12]		2 2/3" x 1/2"				
	All Diam.	8"	10"	12"	18"	24"	36"	48"
Unpaved	0.024	0.012	0.014	0.011	0.014	0.016	0.019	0.020
25% Paved	0.021					0.015	0.017	0.020
Fully Paved	0.012					0.012	0.012	0.012

Corrugations	Annular 3" x 1"	Helical					
	All Diam.	36"	48"	54"	60"	56"	72"
Unpaved	0.027	0.021	0.023	0.023	0.024	0.025	0.025
25% Paved	0.023	0.019	0.020	0.020	0.021	0.022	0.022
Fully Paved	0.012	0.012	0.012	0.012	0.012	0.012	0.012

(B) Manning's n-values for Structural Plate Metal Pipe

Corrugations 6"x2"	Diameters			
	5 ft	7 ft	10 ft	15 ft
Plain-unpaved	0.033	0.032	0.030	0.028
25% Paved	0.028	0.027	0.026	0.024

(B) Manning's n-values for Concrete Pipe/Culvert

TYPE	n-VALUE
Pre-Cast	0.012
Cast in Place	--
With Steel Forms	0.013
With Wood Forms	0.015

Hydraulic Date for Culverts

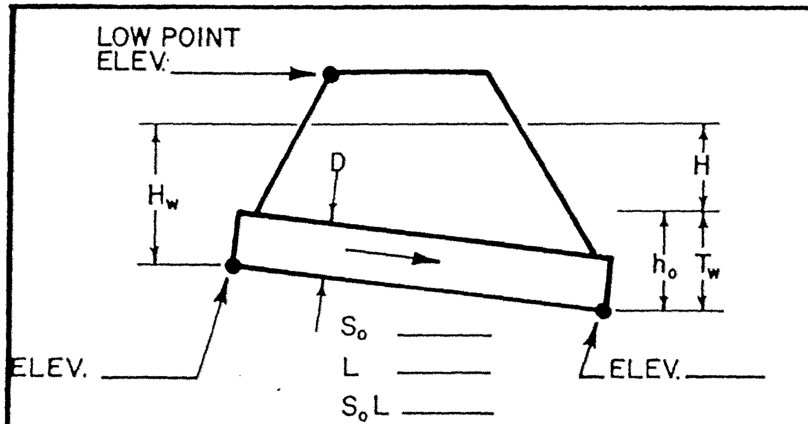
Table No. 8.1

Type of Entrance	Entrance Coefficient, Ke
Pipe	
Headwall	
Grooved edge	0.20
Rounded edge (0.15D radius)	0.15
Rounded edge (0.25D radius)	0.10
Square edge (cut concrete and CMP	0.40
Headwall & 45° Wingwall	0.20
Grooved edge	0.35
Square edge	
Headwall with Parallel Wingwalls Spaced 1.25D apart	
Grooved edge	0.30
Square edge	0.40
Beveled edge	0.25
Projecting Entrance	
Grooved edge (RCP)	0.25
Square edge (RCP)	0.50
Sharp edge, thin wall (RCP)	0.90
Sloping Entrance	
Mitered to conform to slope	0.70
Flared-end Section	0.50
Box, Reinforced Concrete	
Headwall Parallel to Embankment (no wingwalls)	
Square edge on 3 edges	0.50
Rounded on 3 edges to radius of 1/12 barrel dimension	0.20
Wingwalls at 30° to 75° to barrel	
Square edge at crown	0.40
Crown edge rounded to radius of 1/12 barrel dimension	0.20
Wingwalls at 10° to 30° to barrel	
Square edged at crown	0.50
Wingwalls parallel (extension of sides)	
Square edged at crown	0.70

NOTE: The entrance loss coefficients are used to evaluate the culvert or sewer capacity operating under outlet control.

CULVERT RATING

PROJECT: ____________ LOCATION: ____________ STATION: ________



CULVERT DATA

TYPE: ____________ n: ______

INLET: ____________ Q_{FULL}: ______

K_e: ____________ V_{FULL}: ______

OUTLET CONTROL EQUATIONS

(1) $H_w = H + h_o - LS_o$

(2) For $T_w < D$; $h_o = \frac{d_c + D}{2}$ or T_w (whichever is greater)

$T_w > D$; $h_o = T_w$

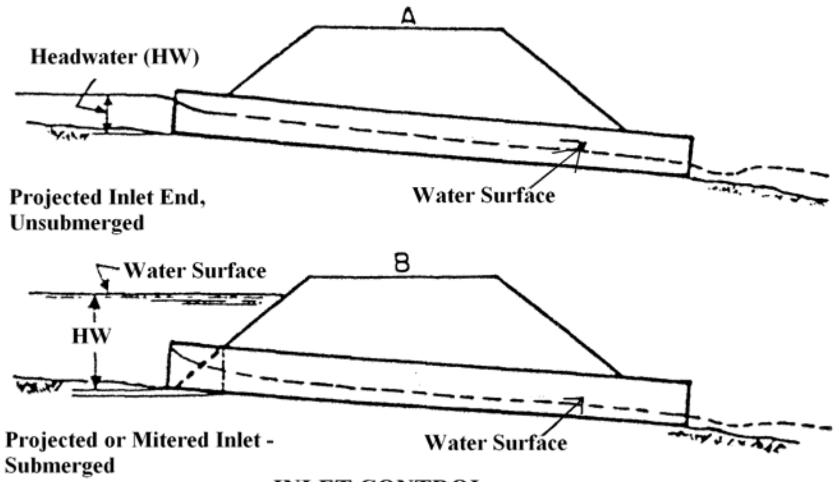
(3) For Box Culvert: $d_c = 0.315(Q/B)^{2/3} \le D$

Q	INLET CONTROL		OUTLET CONTROL						CONT.	CONTROL	ELEV.
					$T_w < D$		$T_w > D$				
	$\frac{H_w}{D}$	H_w	H	T_w	d_c	$\frac{d_c+D}{2}=h_o$	h_o	H_w	H_w		
1	2	3	4	5	6	7	8	9	10	11	12

Figure No. 8.1

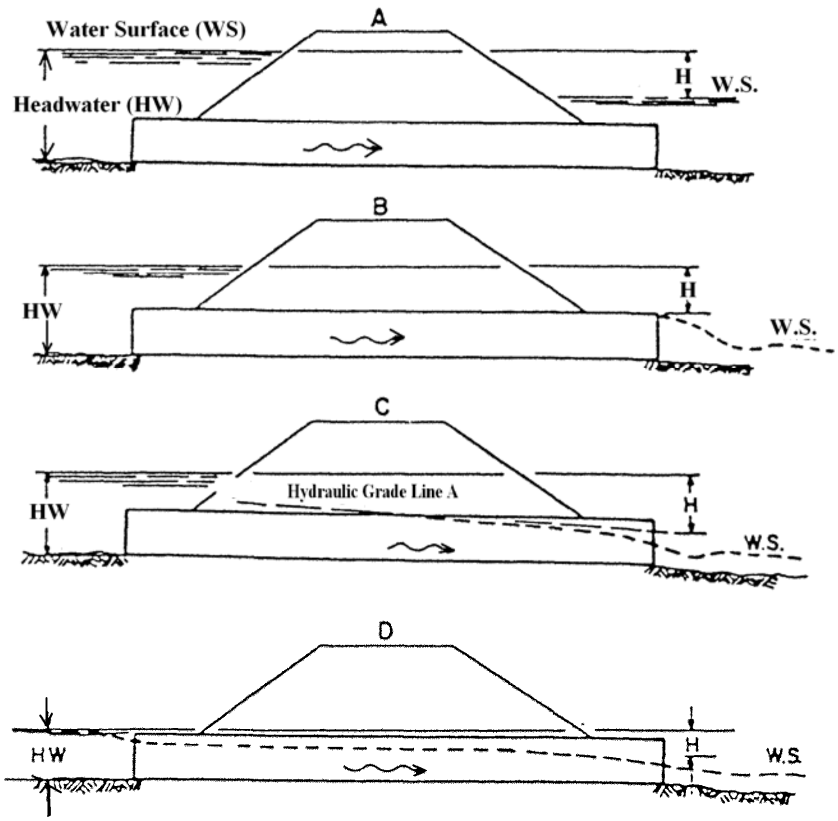
INLET AND OUTLET CONDITION FOR CULVERTS

Figure No. 8.2



INLET CONTROL

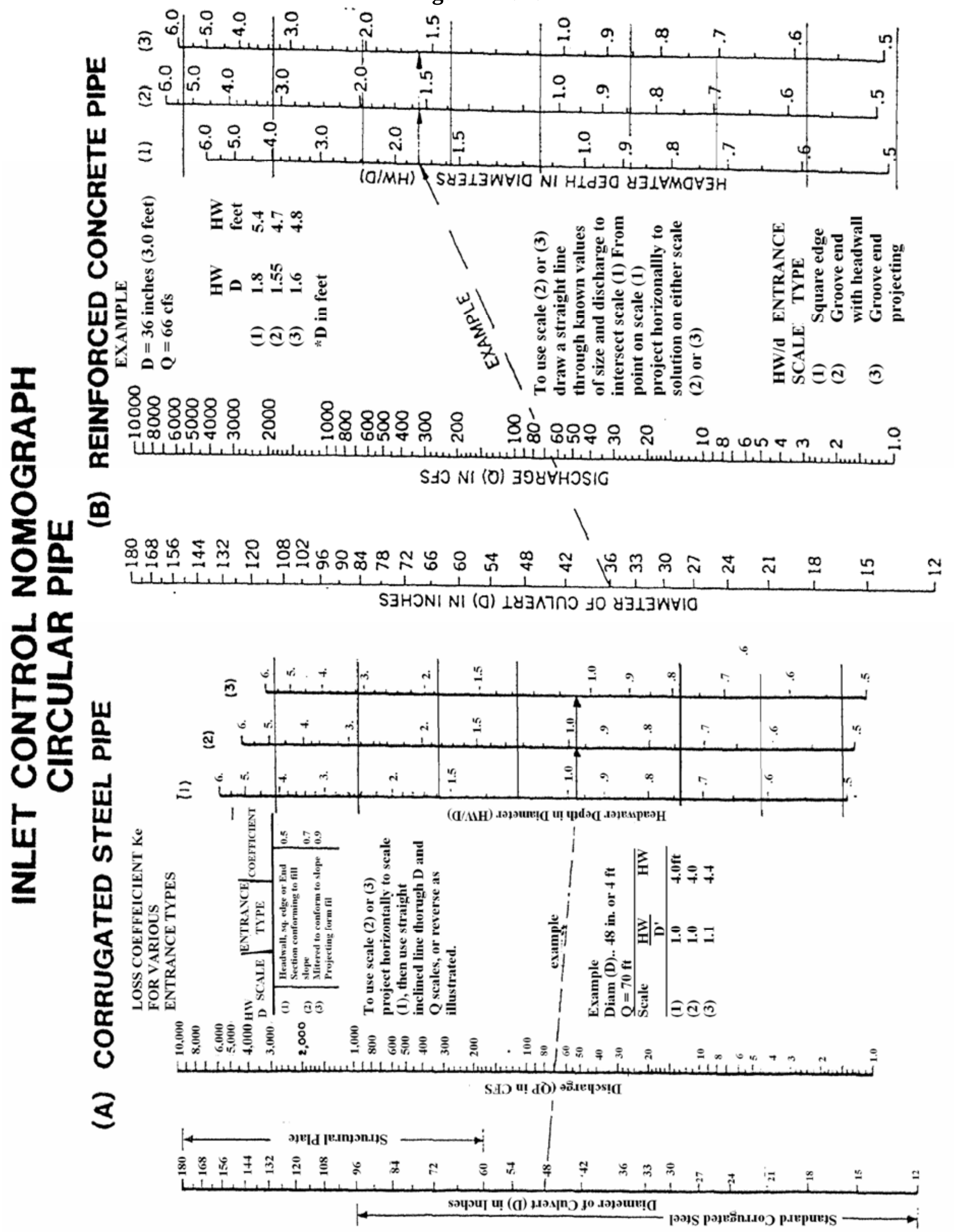
Fig. 4-12 Inlet control is on of the 2 Major tpes of culvert flow.
Condition A with unsubmerged culvert inlet is preferred to the submerged end.
Slope, roughness and length of culvert barrel are no consideration.



OUTLET CONTROL

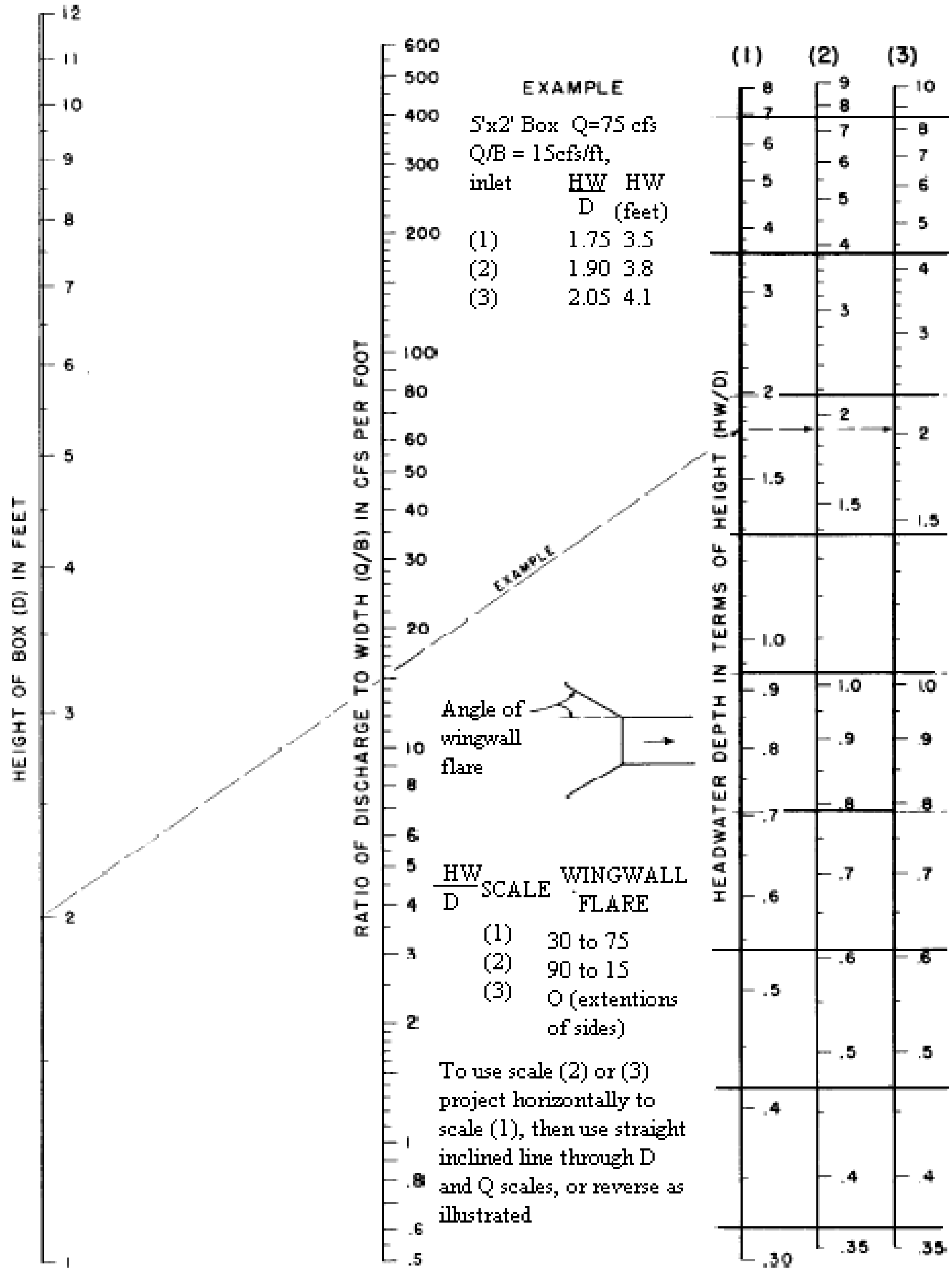
Fig. 4-13 *Outlet control* involves these factors: cross-sectional area of barrel; inlet "geometry"; ponding; tailwater; and slope, roughness and length of culvert barrel.

Figure No. 8.3

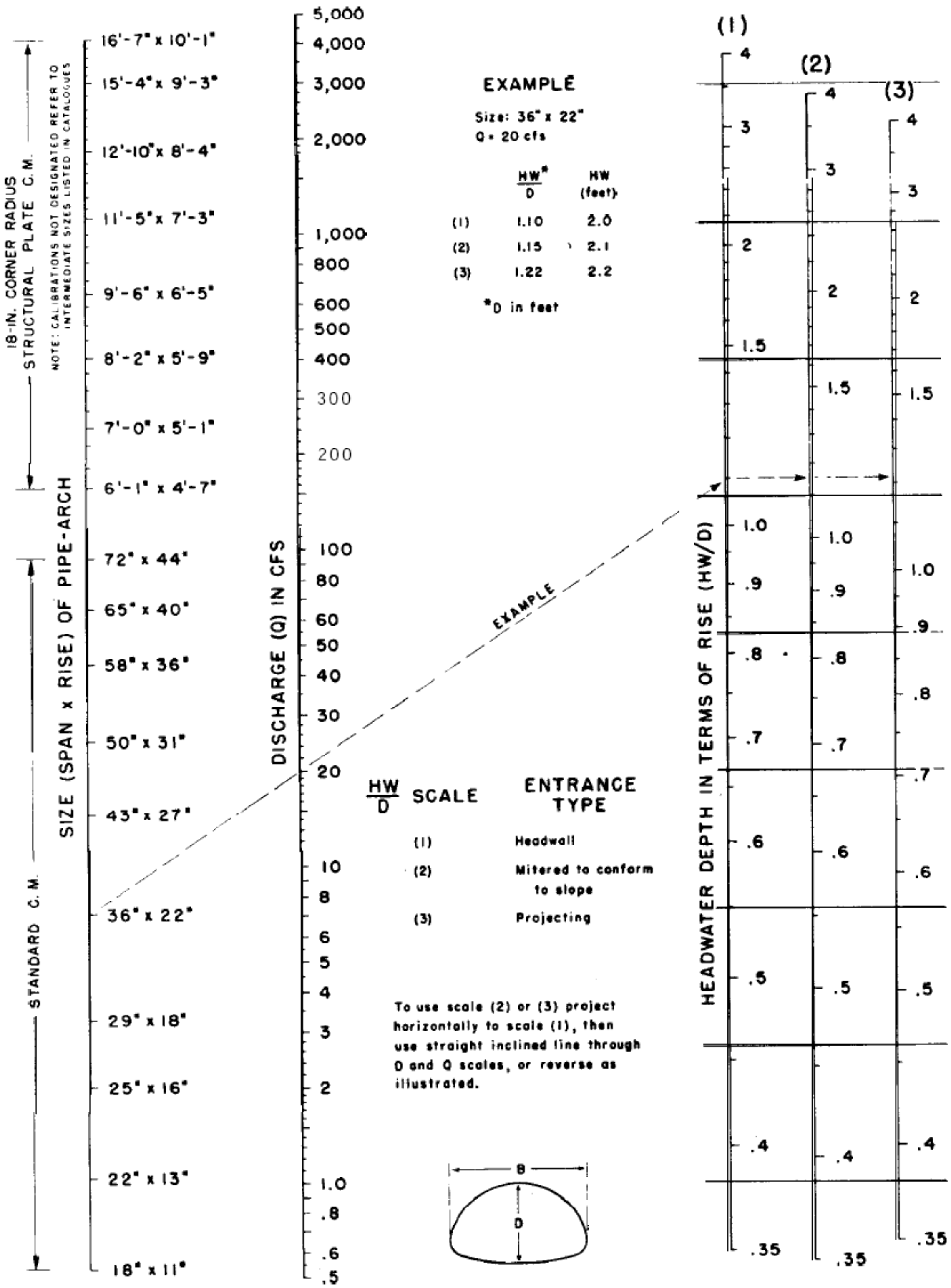


INLET CONTROL NOMOGRAPH
CIRCULAR PIPE
(A) CORRUGATED STEEL PIPE
(B) REINFORCED CONCRETE PIPE
LOSS COEFFEICIENT Ke FOR VARIOUS ENTRANCE TYPES
HW/D SCALE
ENTRANCE TYPE
COEFFICIENT
(1) Headwall, sq. edge or End Section conforming to fill slope 0.5
(2) Mitered to conform to slope 0.7
(3) Projecting form fil 0.9
To use scale (2) or (3) project horizontally to scale (1), then use straight inclined line thorugh D and Q scales, or reverse as illustrated.
Example
Diam (D).. 48 in. or 4 ft
Q = 70 ft
Scale HW/D' HW
(1) 1.0 4.0ft
(2) 1.0 4.0
(3) 1.1 4.4
example
Structural Plate
Standard Corrugated Steel
Diameter of Culvert (D) in Inches
Discharge (QP in CFS
Headwater Depth in Diameter (HW/D)
EXAMPLE
D = 36 inches (3.0 feet)
Q = 66 cfs
HW/D HW feet
(1) 1.8 5.4
(2) 1.55 4.7
(3) 1.6 4.8
*D in feet
To use scale (2) or (3) draw a straight line through known values of size and discharge to intersect scale (1) From point on scale (1) project horizonallly to solution on either scale (2) or (3)
HW/d SCALE ENTRANCE TYPE
(1) Square edge
(2) Groove end with headwall
(3) Groove end projecting
DIAMETER OF CULVERT (D) IN INCHES
DISCHARGE (Q) IN CFS
HEADWATER DEPTH IN DIAMETERS (HW/D)

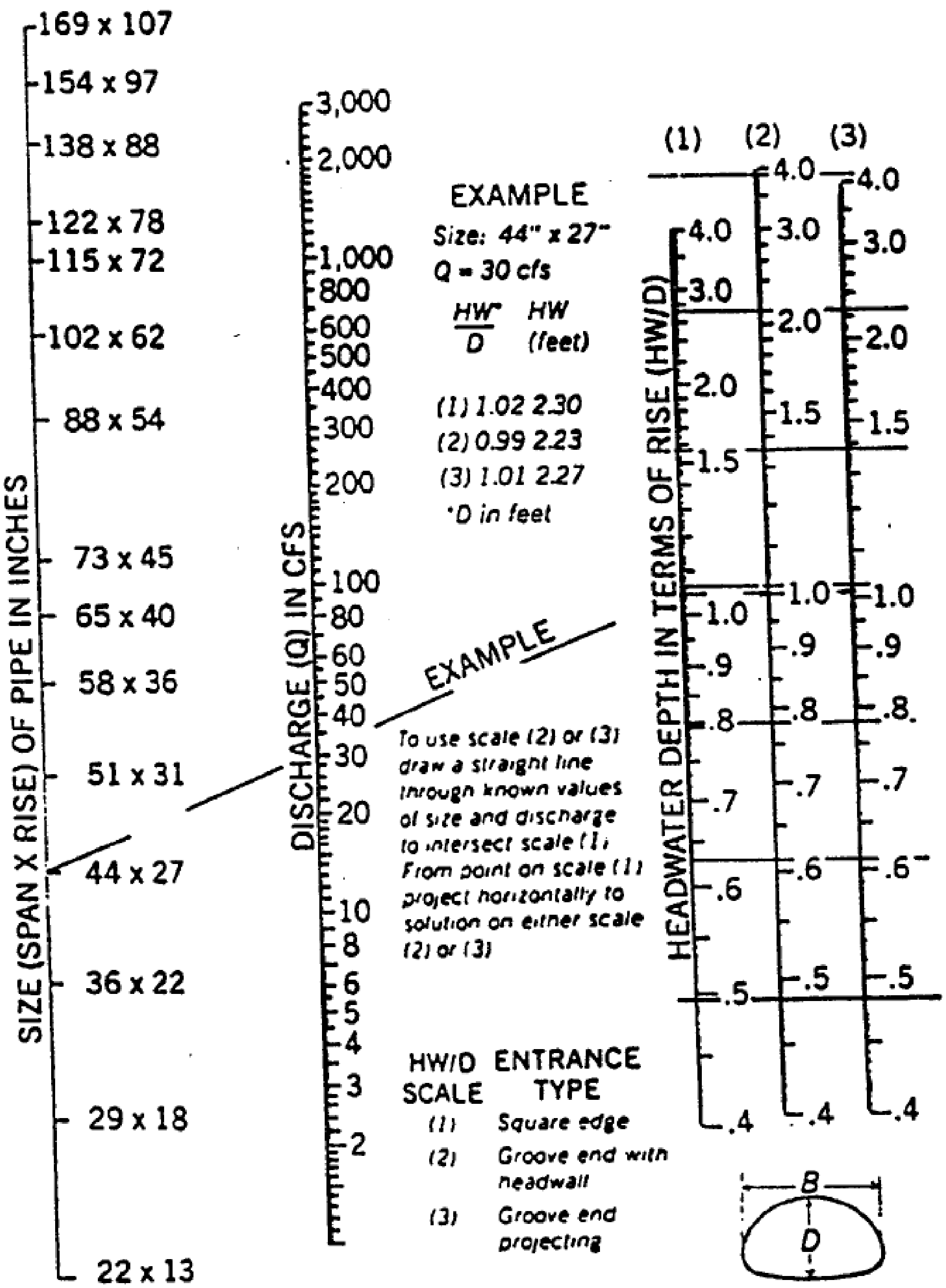
Inlet Control Nomograph
Box Culverts
Figure No. 8.4



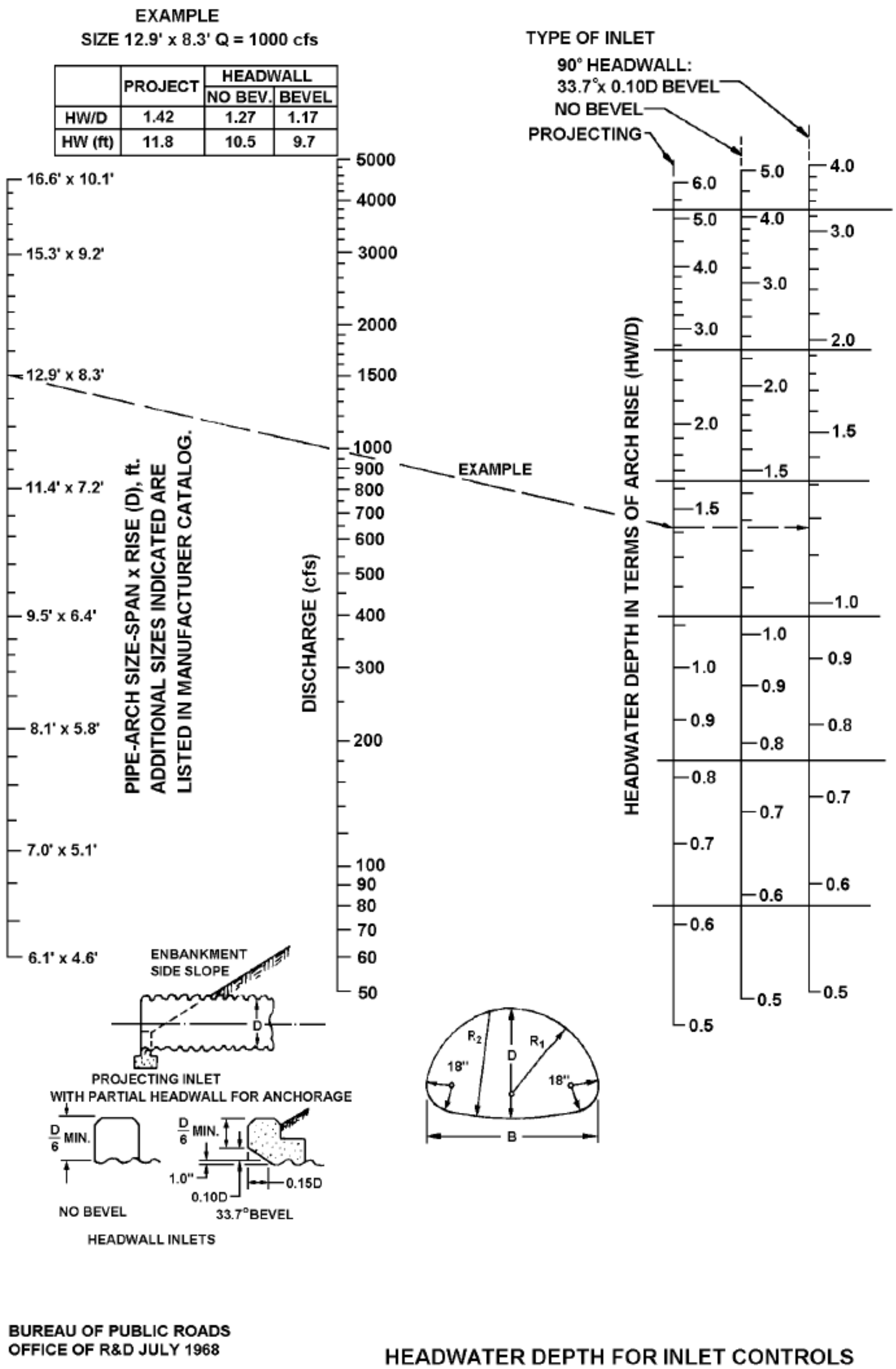
Inlet Control Nomograph
CSP Arch
Figure No. 8.5



Inlet Control Nomograph
RCP Arch
Figure No. 8.6



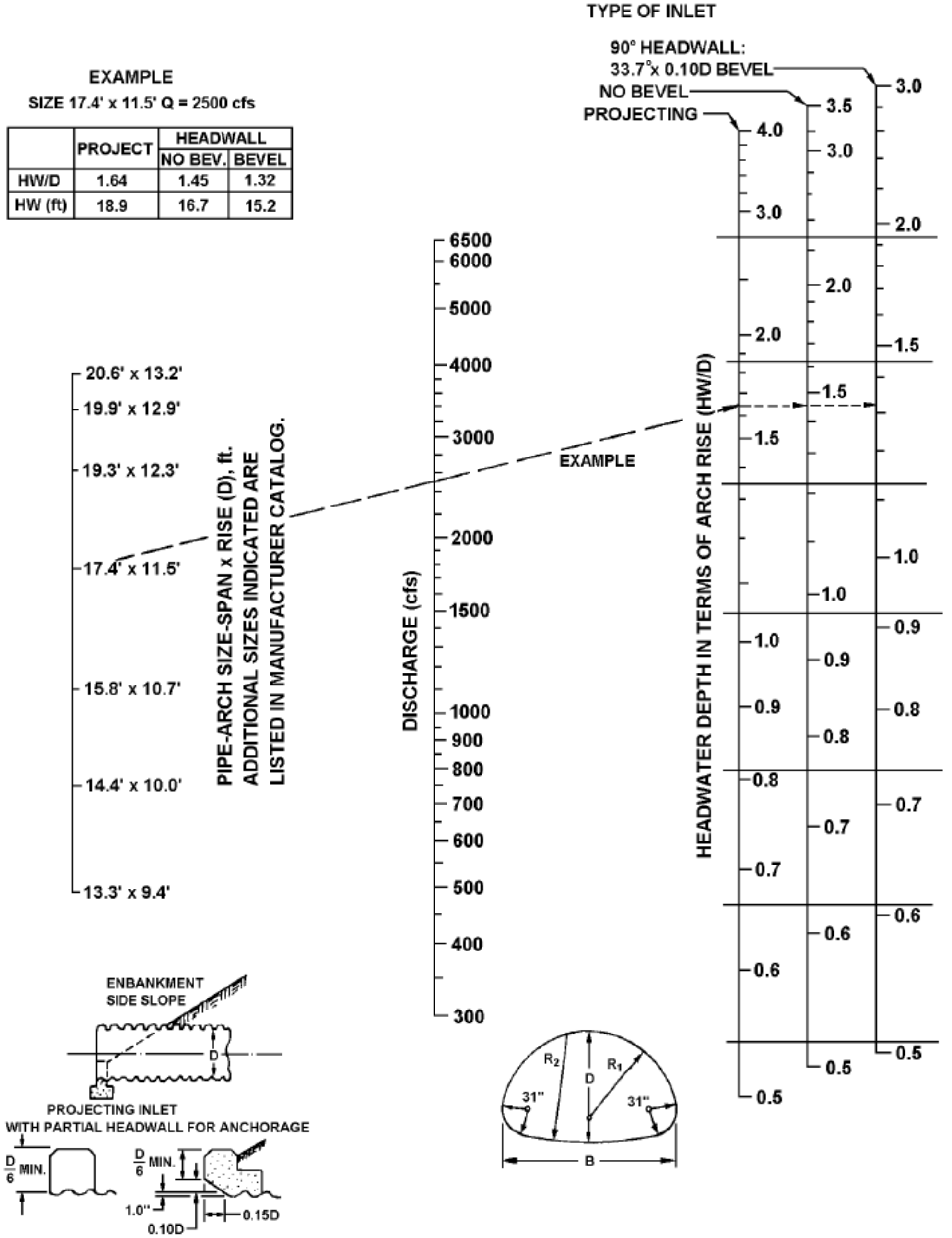
Inlet Control Nomograph SPP ARCH
Figure No. 8.7 (a)



	PROJECT	HEADWALL	
		NO BEV.	BEVEL
HW/D	1.42	1.27	1.17
HW (ft)	11.8	10.5	9.7

HEADWATER DEPTH FOR INLET CONTROLS
STRUCTURAL PLATE PIPE-ARCH CULVERTS
18 in. RADIUS CORNER PLATE
PROJECTING OR HEADWALL INLET
HEADWALL WITH OR WITHOUT EDGE BEVEL

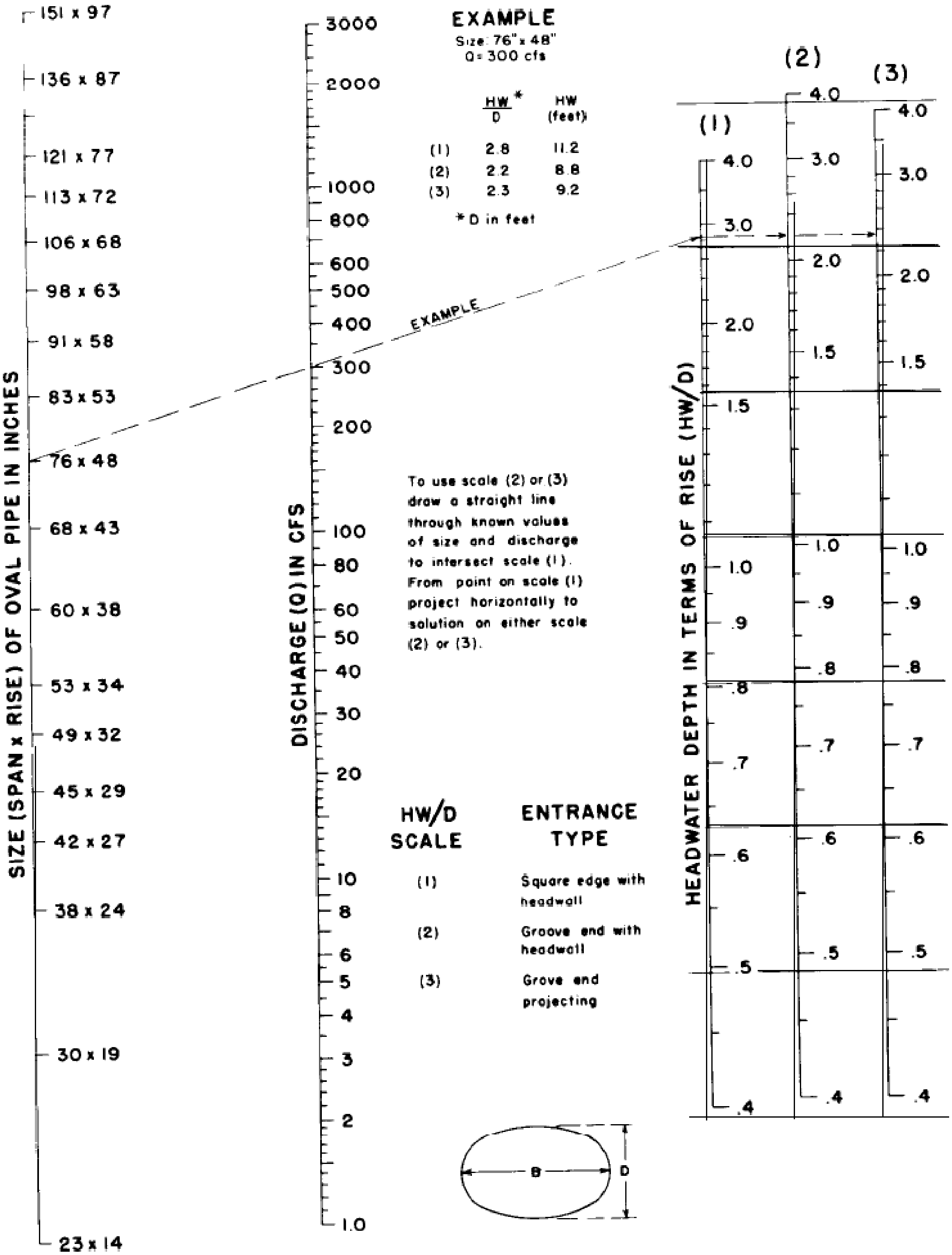
Inlet Control Nomograph SPP ARCH
Figure No. 8.7 (b)



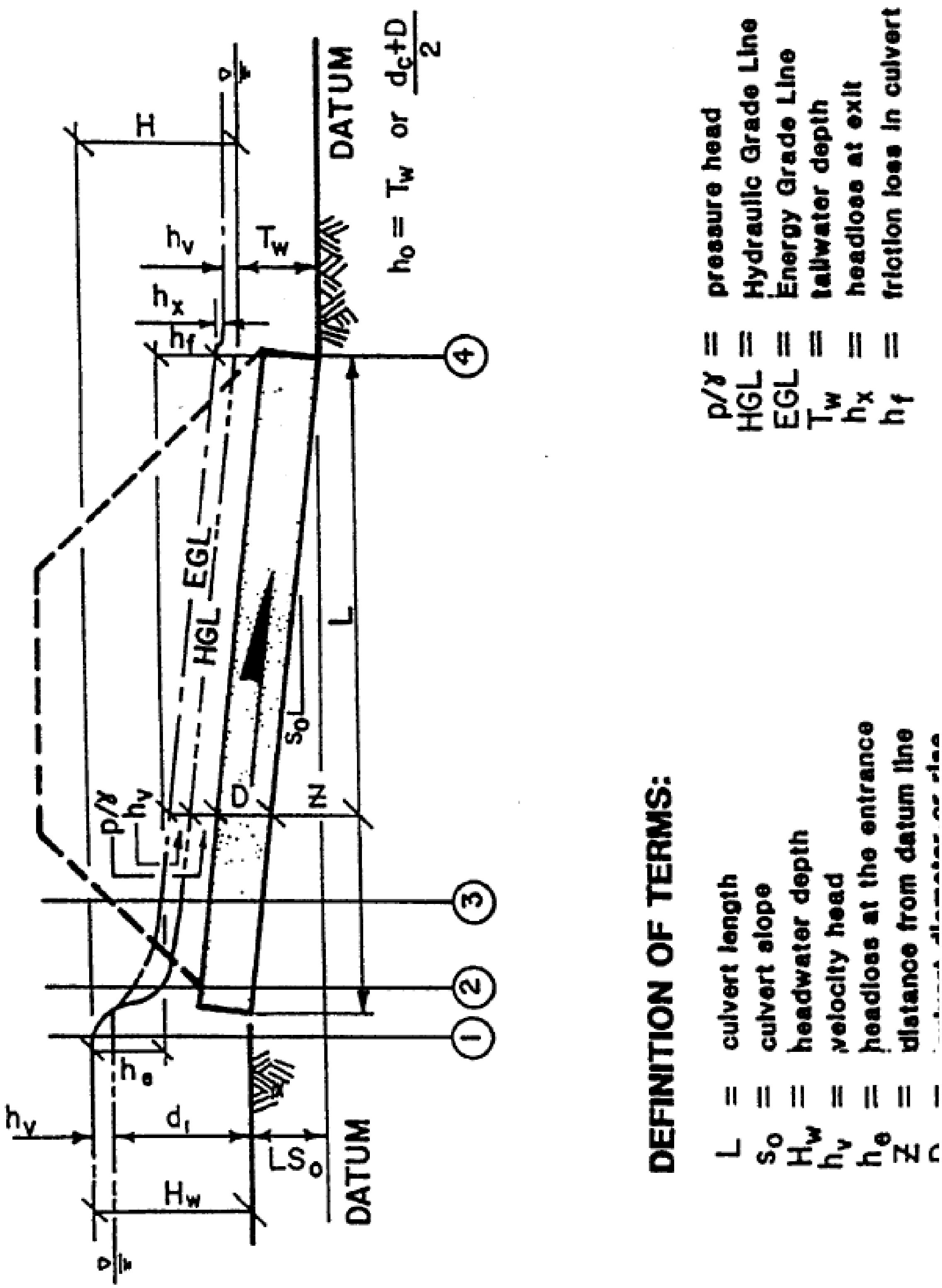
	PROJECT	HEADWALL	
		NO BEV.	BEVEL
HW/D	1.64	1.45	1.32
HW (ft)	18.9	16.7	15.2

HEADWATER DEPTH FOR INLET CONTROL
STRUCTURAL PLATE PIPE-ARCH CULVERTS
31 in. RADIUS CORNER PLATE
PROJECTING OR HEADWALL INLET
HEADWALL WITH OR WITHOUT EDGE BEVEL

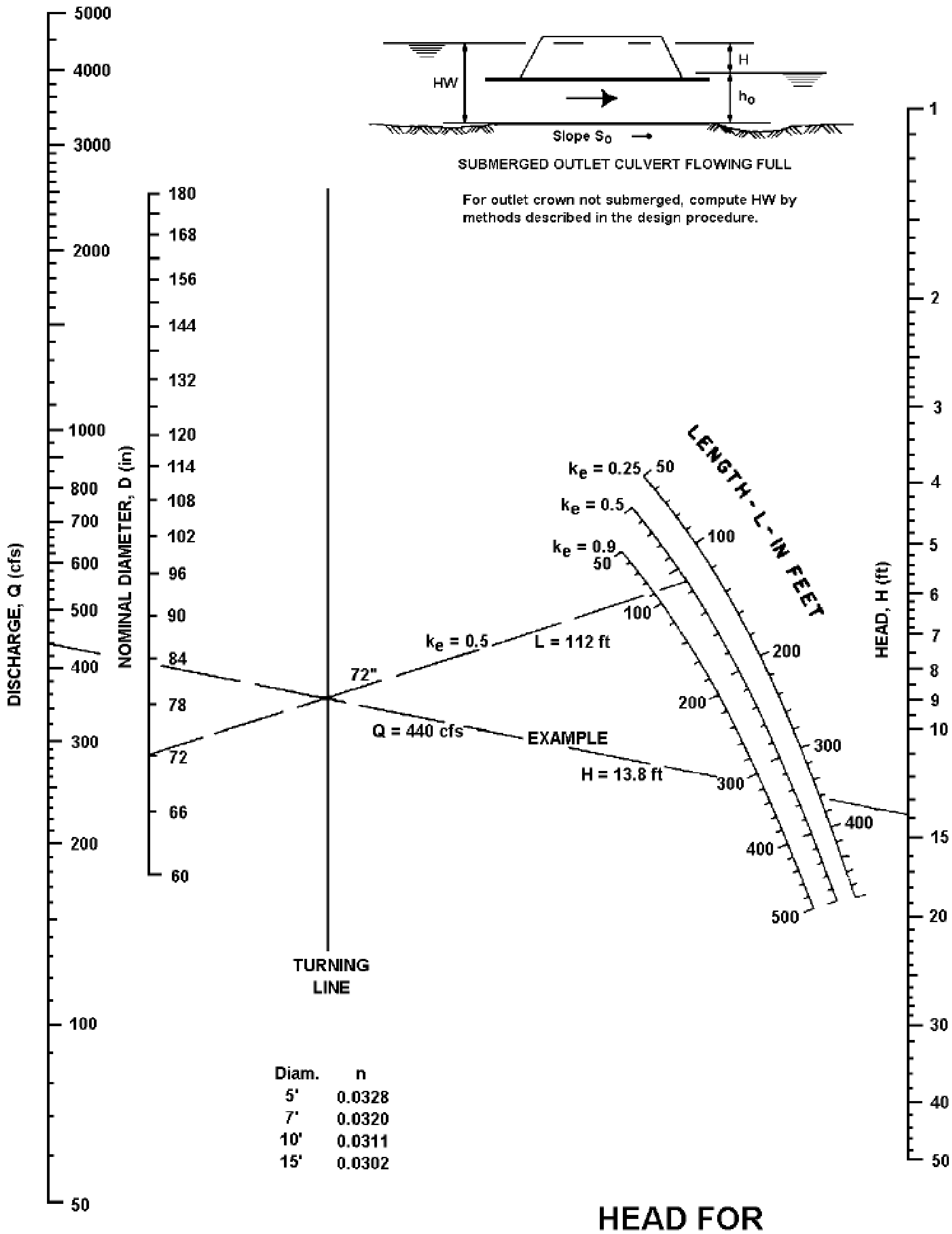
Inlet Control Nomograph
RCP Ellipse
Figure No. 8.8



Hydraulics of a Culvert Under Outlet Condition
Figure No. 8.9



Outlet Control Nomograph Circular CSP
Figure No. 8.10 (a)

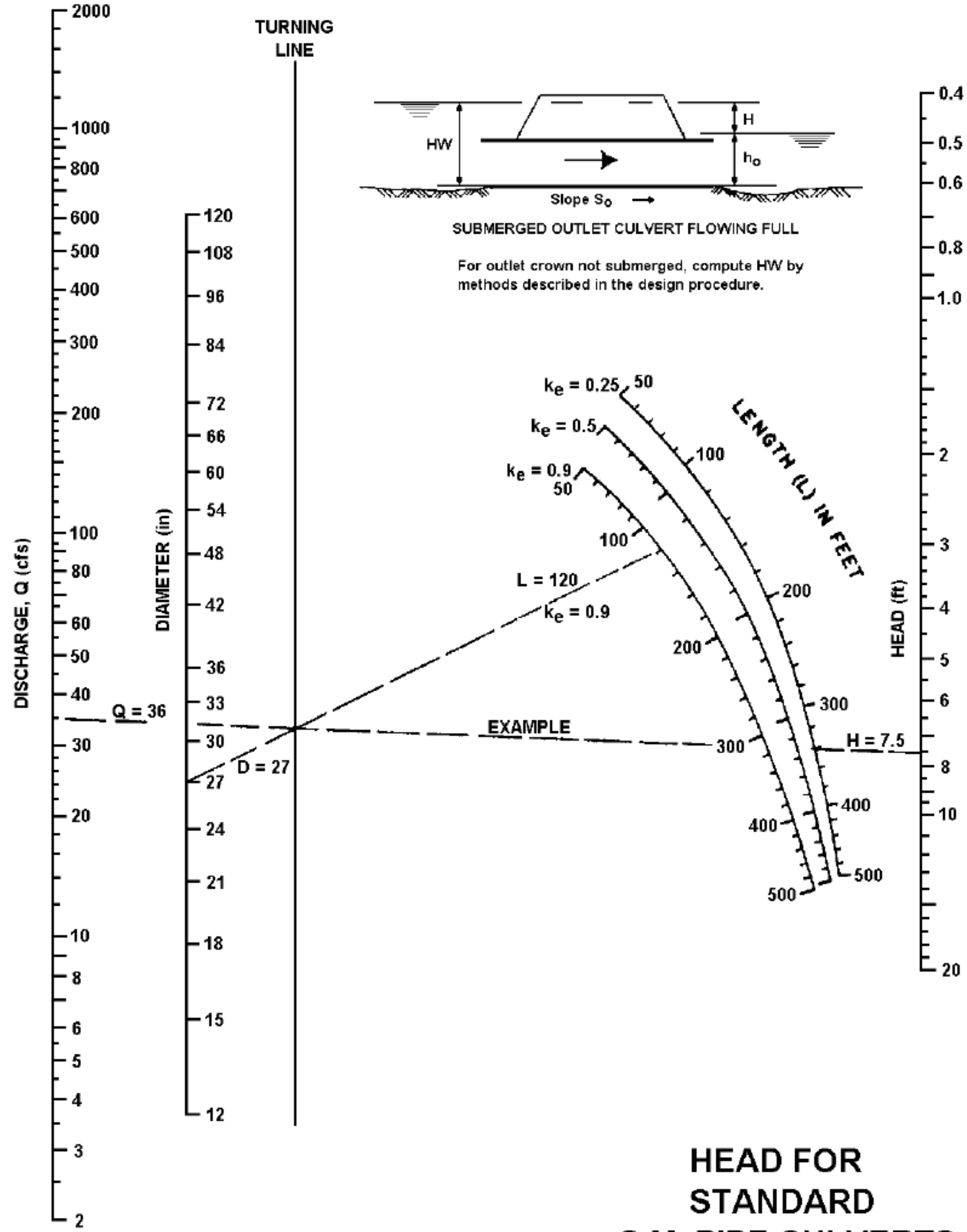


Diam.	n
5'	0.0328
7'	0.0320
10'	0.0311
15'	0.0302

BUREAU OF PUBLIC ROADS JAN. 1963

HEAD FOR STRUCTURAL PLATE CORR. METAL PIPE CULVERTS FLOWING FULL
n = 0.0328 TO 0.0302

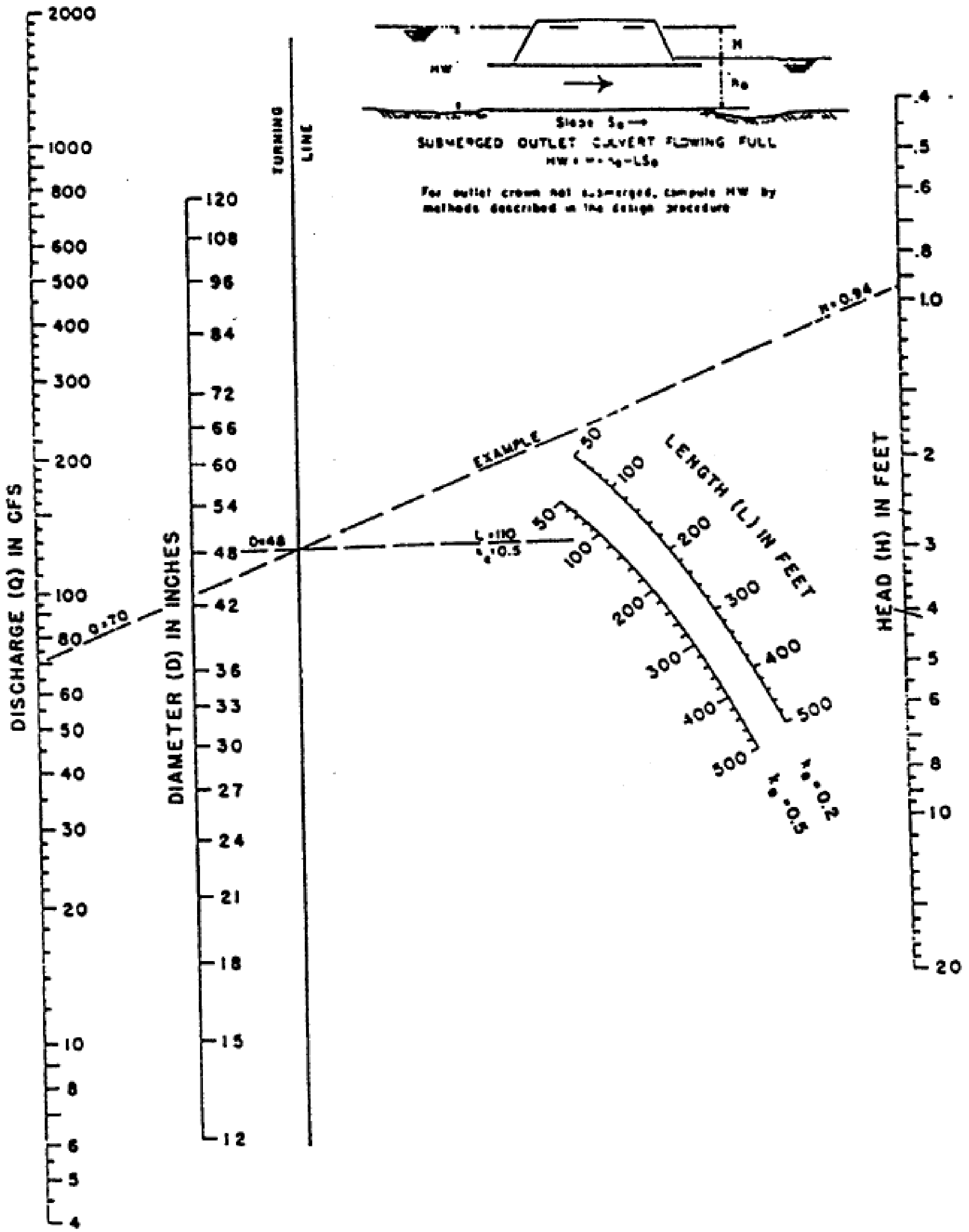
Outlet Control Nomograph Circular CSP
Figure No. 8.10 (b)



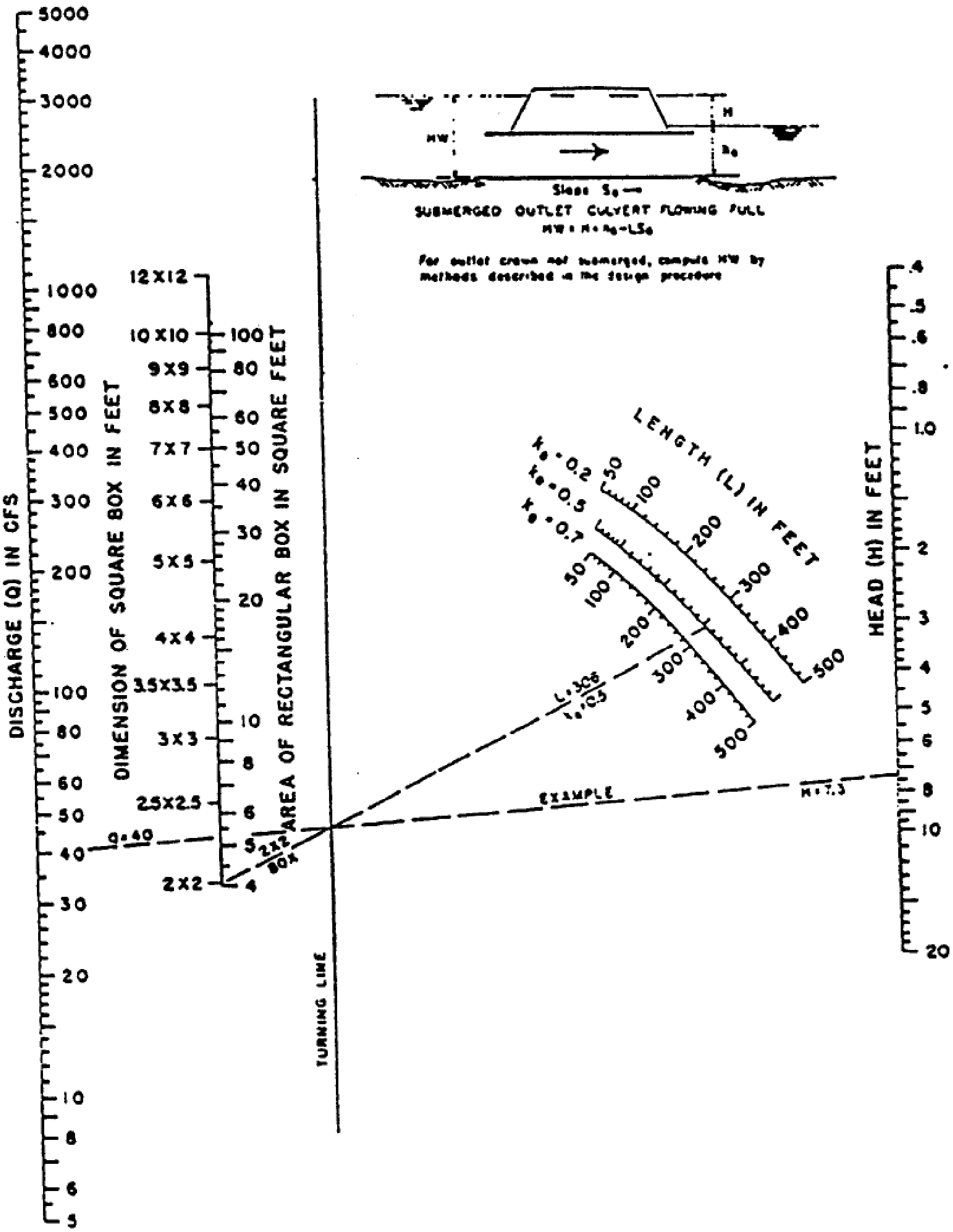
HEAD FOR
STANDARD
C.M. PIPE CULVERTS
FLOWING FULL

Pipe Diam. In Feet	Roughness Factor		Length Adjustment Factor (n' / n) ^2
	Curves Based on n min	Actual n'min	
5'	0.0328	0.033	1.0
7'	0.032	0.037	1.0
10'	0.0311	0.03	0.93
15'	0.0302	0.028	0.86

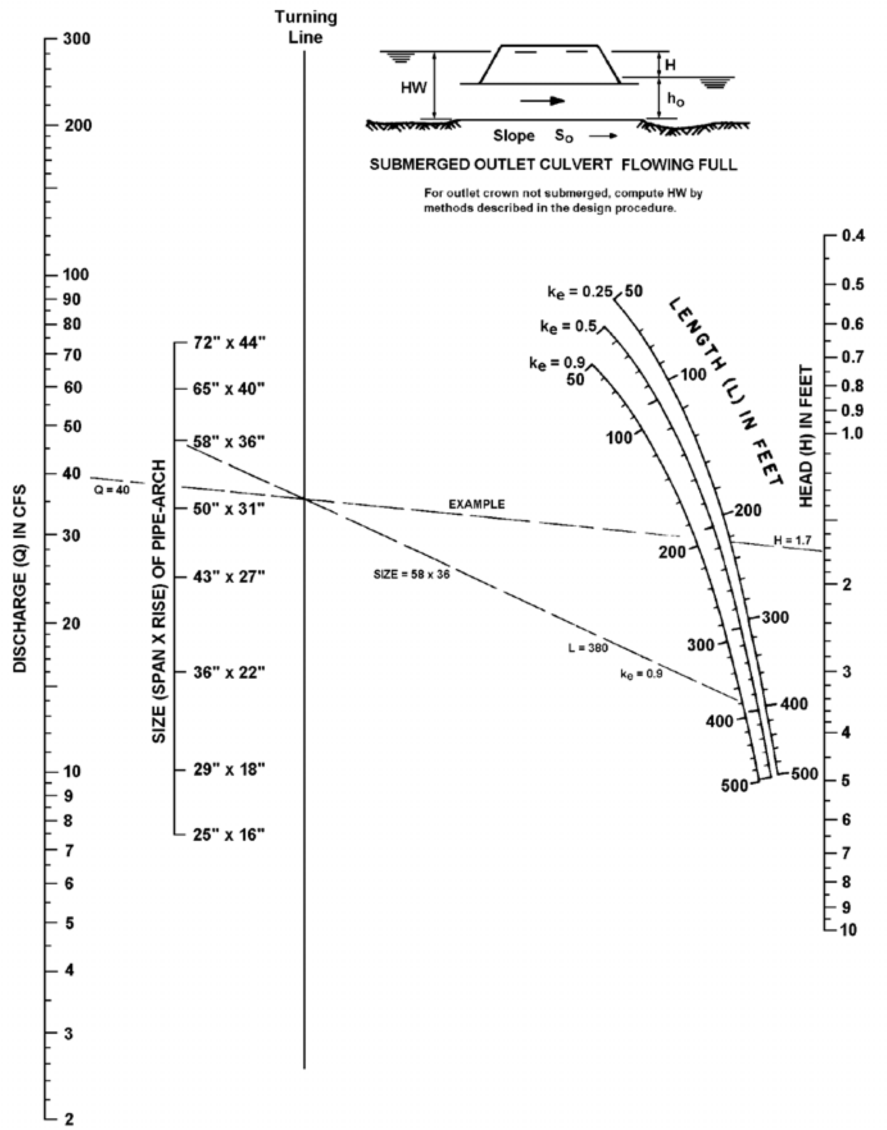
Outlet Control Nomograph
Circular RCP
Figure No. 8.11



Outlet Control Nomograph
Box Culverts
Figure No. 8.12



Outlet Control Nomograph
CSP Arch
Figure No. 8.13



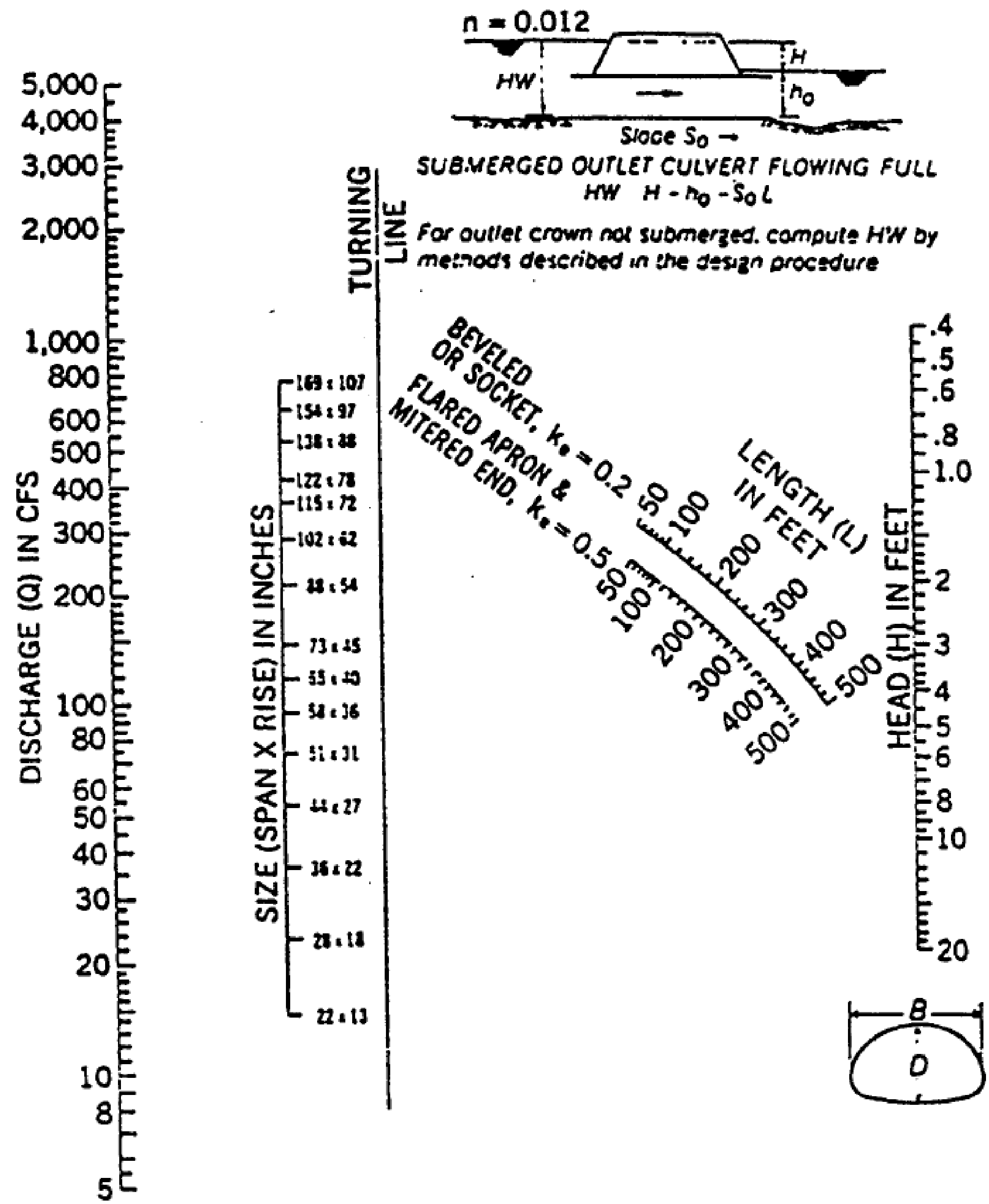
HEAD FOR
STANDARD C.M. PIPE-ARCH CULVERTS
FLOWING FULL
n = 0.024

BUREAU OF PUBLIC ROADS JAN. 1963

Length Adjustment for Improved Hydraulics

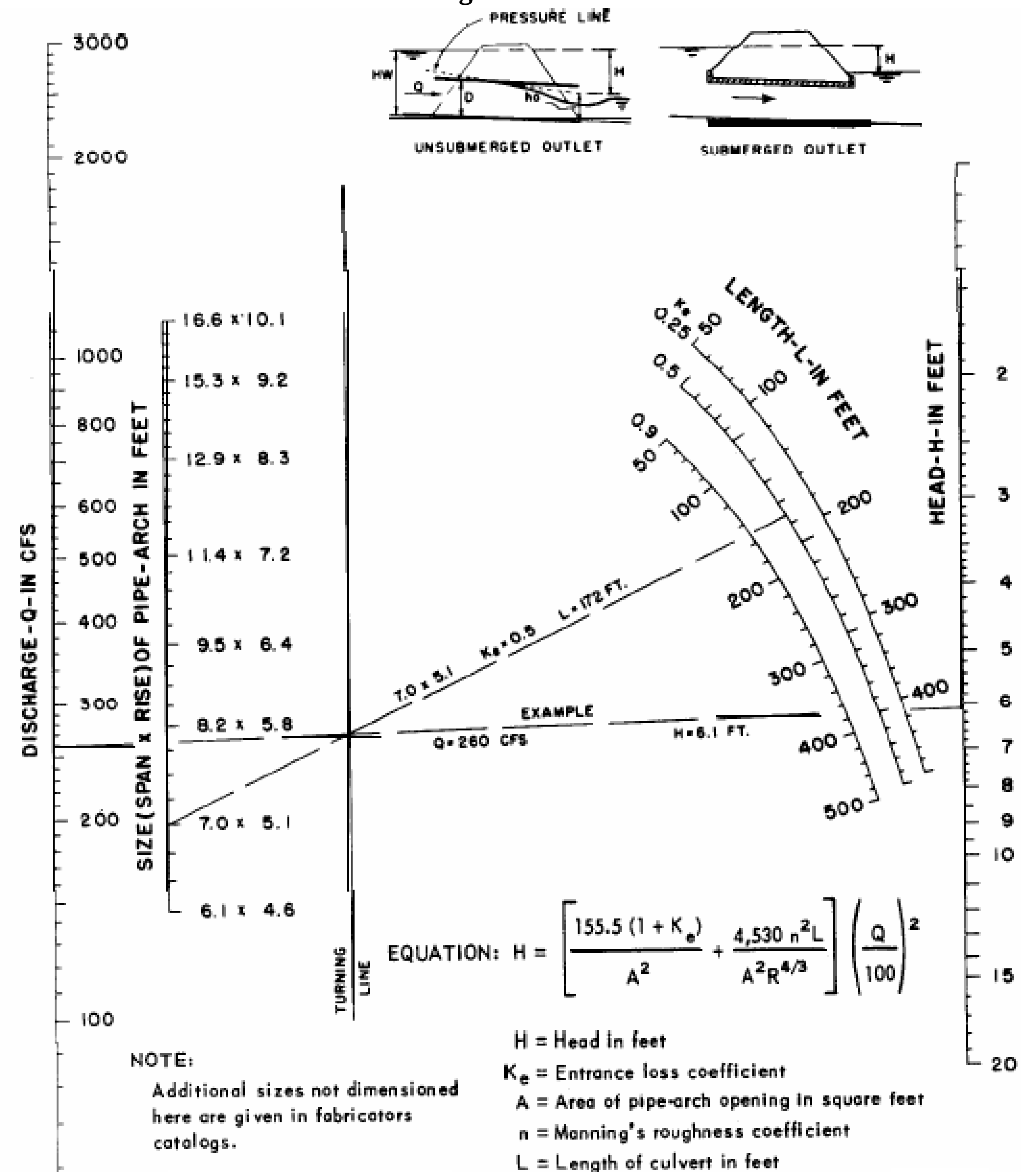
Pipe Diameter in Inches	Roughness Factor n' for Helical Corr.*	Length Adjustment Factor $\left(\frac{n'}{n}\right)^2$
12	.011	.21
24	.016	.44
36	.019	.51
48	.020	.70

Outlet Control Nomograph
RCP Arch
Figure No. 8.14



Outlet Control Nomograph SPP Arch

Figure No. 8.15

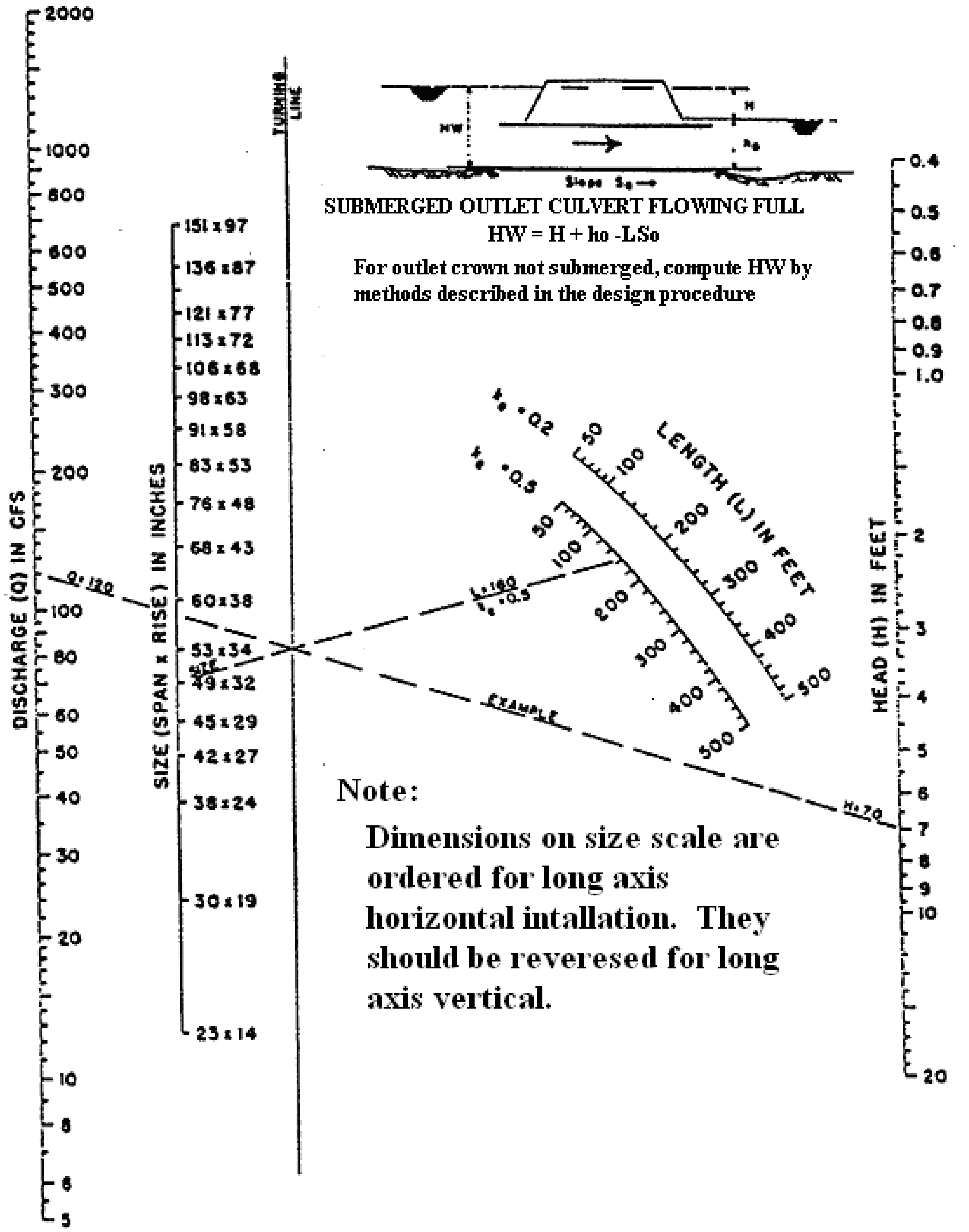


Outlet Control. Head for structural plate pipe - arch culvert with 18-in. corner radius - with submerged outlet and flowing full. For 31 in corner radius use structure sizes with equivalent areas on the 18-in. corner radius scale.

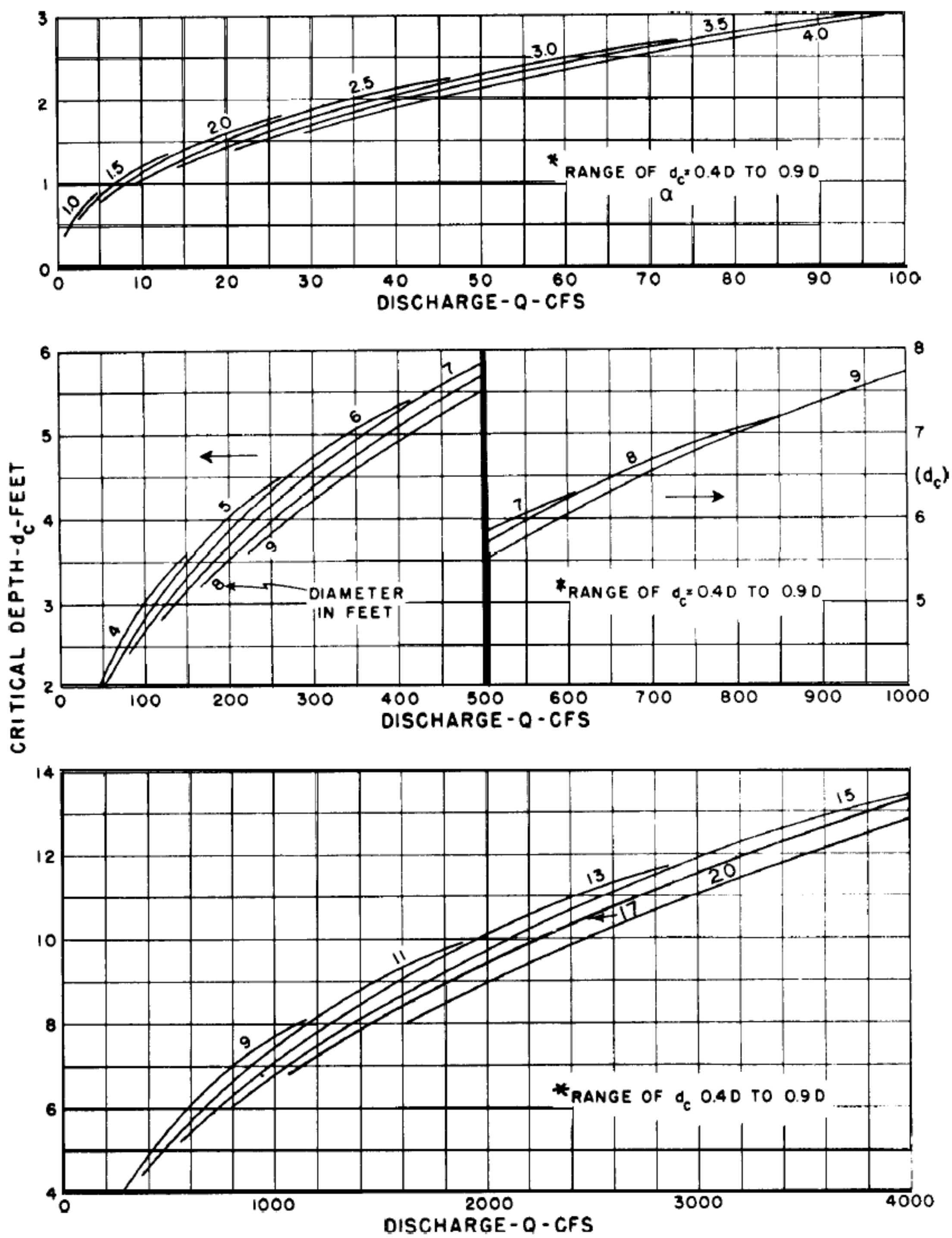
Length Adjustment for Improved Hydraulics

Pipe Arch Size In Feet	Roughness Factor: Curves Based on n min	Roughness Factor: Actual n' min	Length Adjustment Factor (n' / n) ^2
6.1 x 4.6	0.0327	0.0327	1.0
8.1 x 5.8	0.0321	0.032	1.0
11.4 x 7.2	0.0315	0.03	0.907
16.6 x 10.1	0.0306	0.028	0.837

Outlet Control Nomograph
RCP Ellipse
Figure No. 8.16

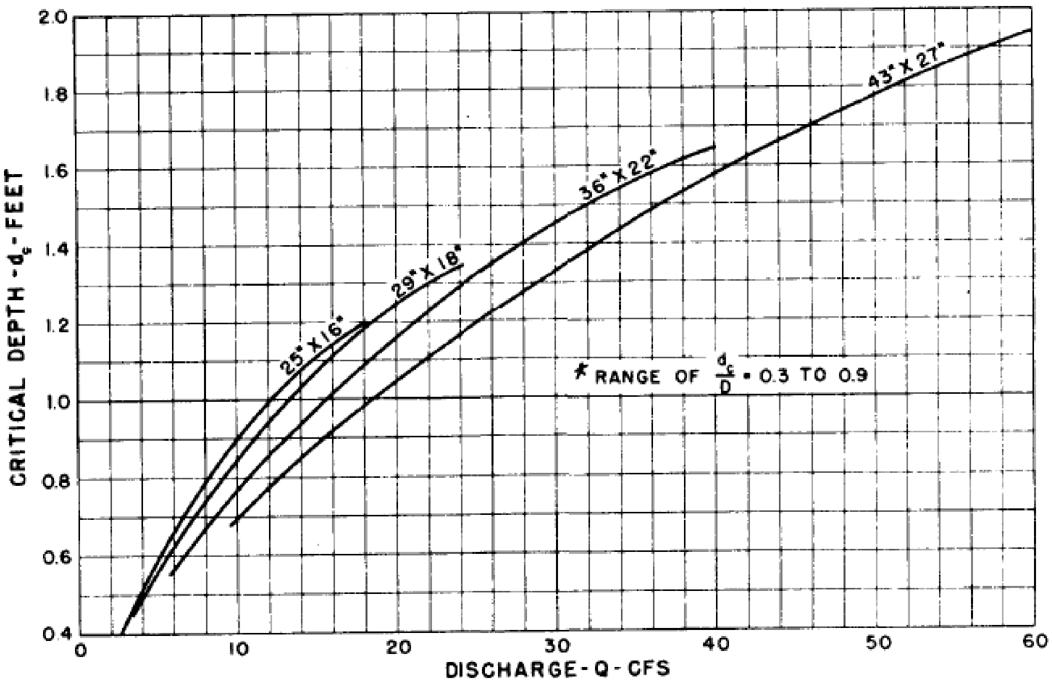


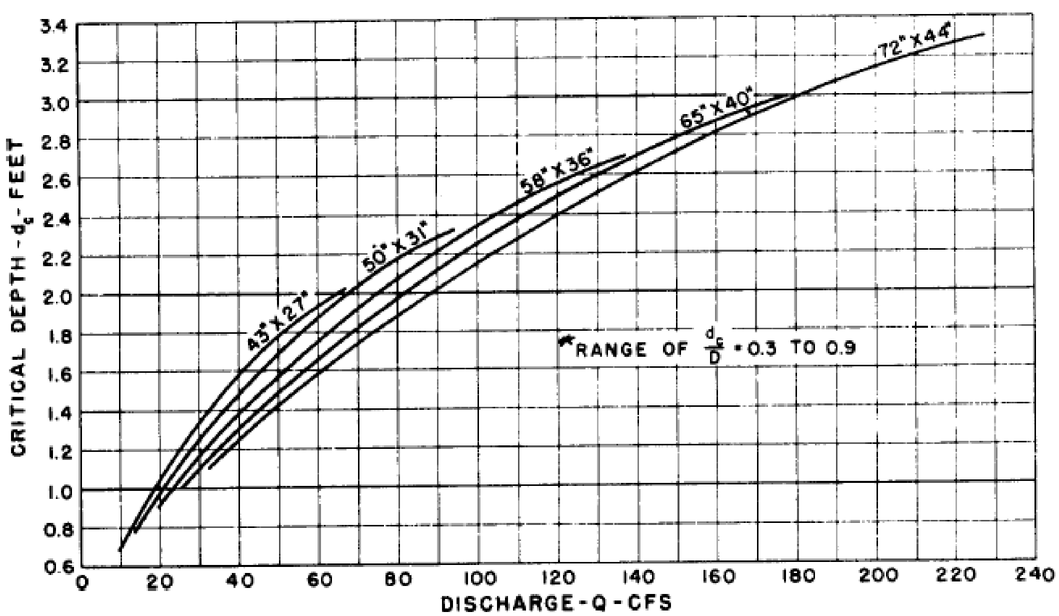
Critical Depth Curves
Circular Pipe
Figure No. 8.17



*NOTE: FOR VALUES OF d_c ABOVE CURVE, USE d_c = D

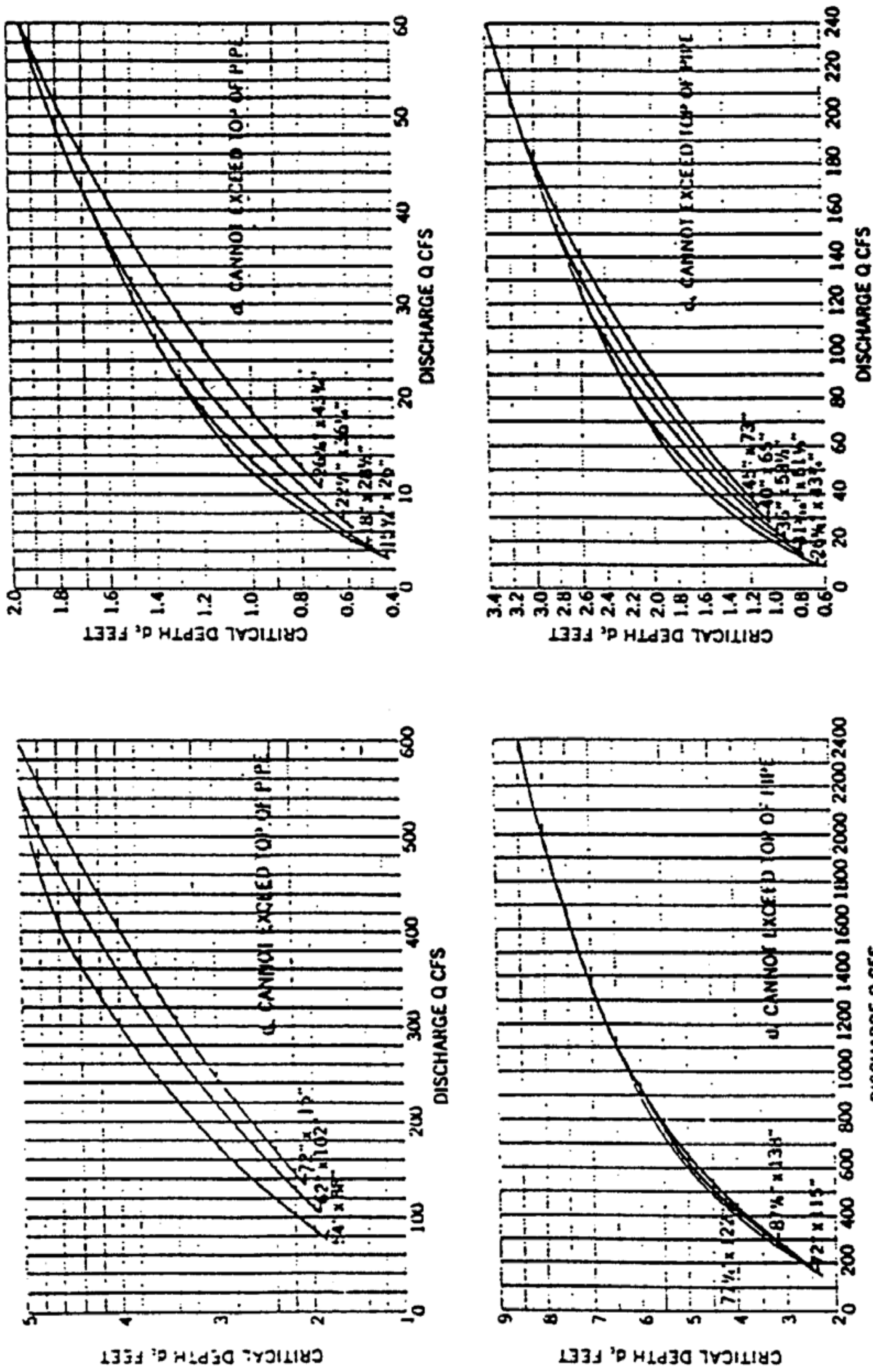
Critical Depth Curves
CSP Arch
Figure No. 8.18



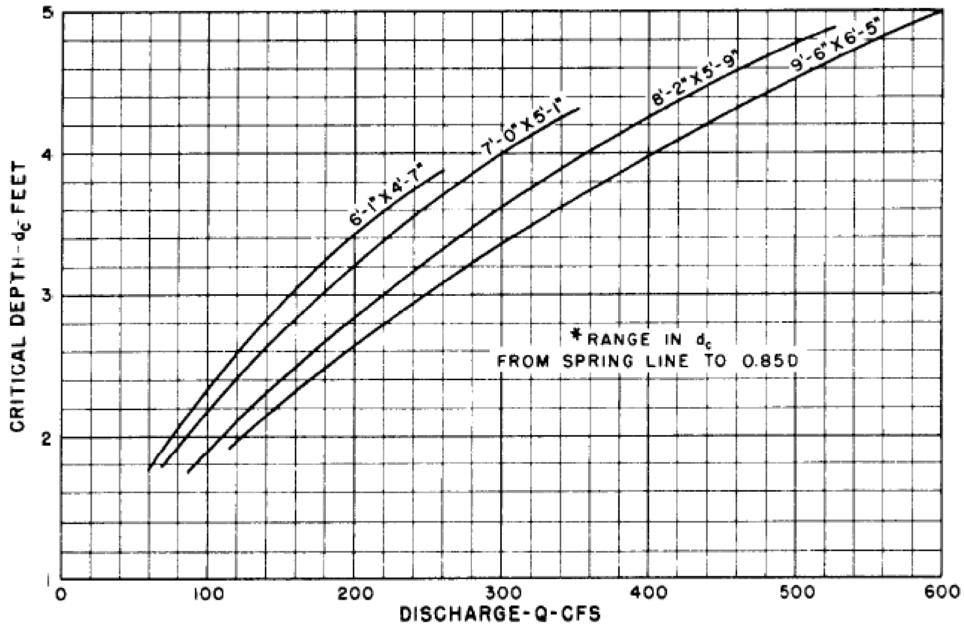


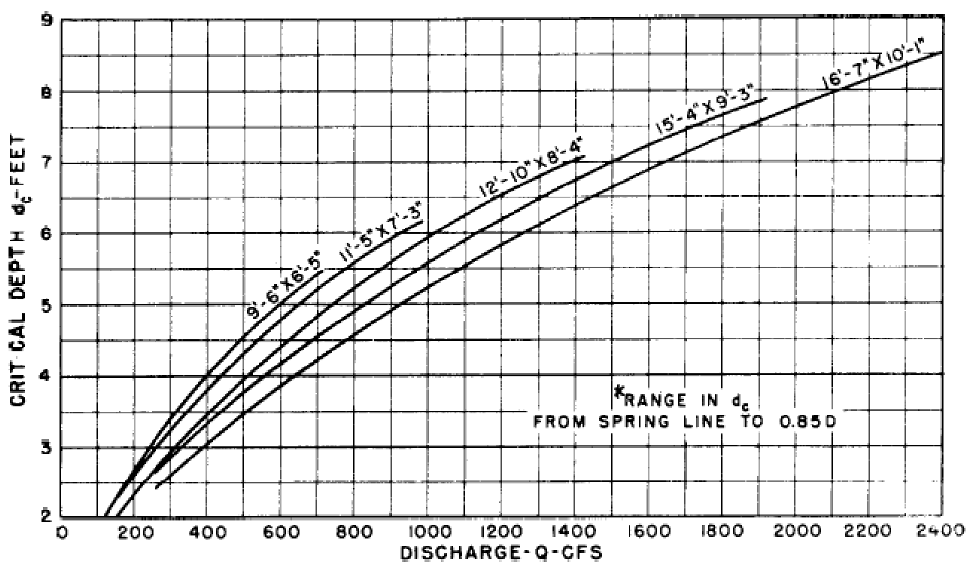
*NOTE: FOR VALUES OF d_c ABOVE CURVE, USE $d_c = D$

Critical Depth Curves
RCP Arch
Figure No. 8.19



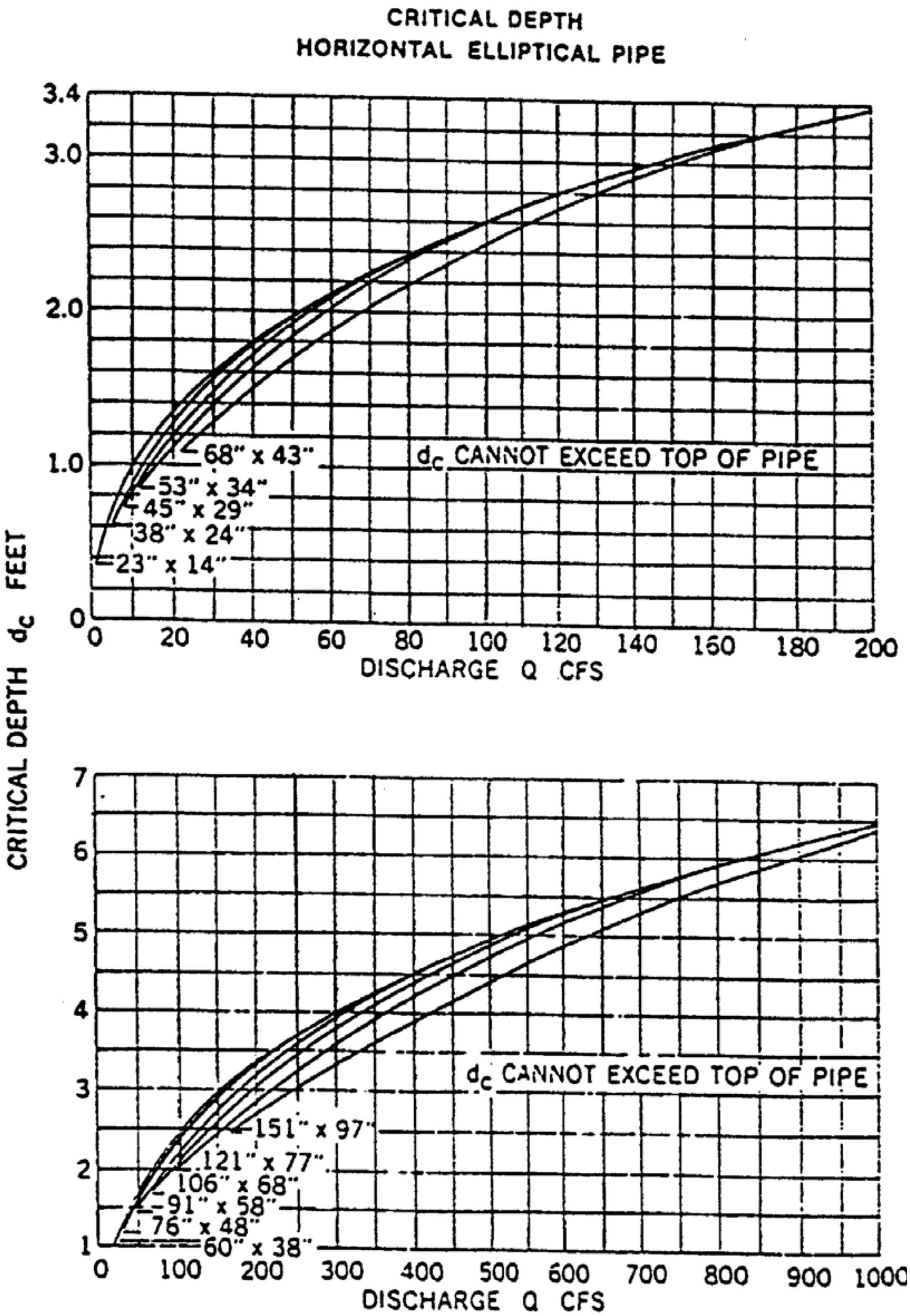
Critical Depth Curves
SSP Arch
Figure No. 8.20





*NOTE: FOR VALUES OF d_c ABOVE
CURVE, USE d_c = D

Critical Depth Curves
RCP Ellipse
Figure No. 8.21



CHAPTER 9
STORMWATER DETENTION/RETENTION

9.1 INTRODUCTION

The criteria presented in this section shall be used in the design and evaluation of all detention facilities for the City of Bismarck. The review of all planning submittals (Chapter 2) will be based on the criteria presented in this section.

The main purpose of a detention facility is to store the excess stormwater runoff associated with an increased basin imperviousness and to discharge this excess at a rate similar to the rate experienced from the basin without development. The value of stormwater runoff detention as part of the urban system has been explored by many individuals, agencies, and professional societies. Detention is considered a viable method to reduce urban drainage costs. Temporarily detaining a few acre-feet of runoff can significantly reduce downstream flood hazards as well as pipe and channel requirements in urban areas. Storage also provides for sediment and debris collection which helps to keep streams and rivers cleaner. Because detention basins slow down the flow of water, particles carried with the water begin to settle out within the pond and thereby enhance water quality. The water quality aspects of stormwater detention basins are discussed in Chapter 10 of this Manual.

There are two types of detention facilities defined for use within the City of Bismarck and its extraterritorial limits. Local detention is defined as detention provided to serve only the area in question (on-site) and no areas outside the development area boundaries (off-site). Local detention facilities consist of parking lot storage, rooftop storage, or small grassy areas with a capacity of usually less than one (1) acre-foot of storage. In some cases, however, local detention facilities can be larger for correspondingly larger developments. When properly designed and maintained, local detention can provide significant benefits for the initial or minor storm drainage system. The benefits of local detention for the major system have been studied and preliminary data evaluations do indicate significant benefits can be achieved.

Regional detention facilities provide storage for both on-site and off-site runoff. Generally, the off-site contributing drainage area consists of several upstream developments comprising a significant drainage area. Because the regional detention facility serves many developments, the operation and management is usually retained by the City or a local water management authority. Because of the public nature of such facilities, they are usually incorporated into the park system or greenbelt area. Regional detention can provide significant benefits for both minor and major storm drainage systems by controlling a greater volume of storm runoff. These regional facilities also require a more formalized maintenance program than local facilities and, therefore, the detention function is more likely to be preserved.

In some instances, the detention facility effect on flood peaks can be detrimental or have no significant benefit at all. Local and even regional detention adjacent to a major drainage is an example. Also, detention in the lower reaches of a drainage basin may not be desirable. This detention can aggravate downstream drainage problems by holding back storm runoff and allowing the upper basin runoff to combine and increase the total basin flood peaks. These effects can be difficult to determine and are highly dependent upon the design storm assumptions.

Stormwater impoundments are also classified as detention or retention facilities. In a detention facility, the temporary storage of floodwater is usually released by a measured but uncontrolled outlet. Detention facilities typically flatten and spread out the inflow hydrograph, lowering the peak. Retention storage is provided in a facility without a positive outlet, or with a specially regulated outlet, where all or a portion of the inflow is stored for a prolonged period. Infiltration basins are a common type of retention facility. Ponds that maintain water permanently, with freeboard provided for flood storage, are probably the most common type of retention facility.

Detention and retention facilities can further be subdivided as follows:

1. On-Stream Storage: This is a facility that intercepts the stream flow directly. On-stream storage occasionally is provided as an on-site facility, though it is more often an integral part of a watershed or regional stormwater management plan.

2. On-Site Facilities: In providing for on-site facilities, special attention must be given to the design of outlet structures for controlling runoff from rooftops, parking lots, and swales. Because runoff volumes from such areas are small, the required outlets are also small, which increases the potential for plugging by debris. Also, the outlet must release temporarily stored water in a reasonable amount of time. As an example, parking lots must drain relatively fast in order not to be a nuisance. Rooftop storage must be designed so as to provide for structural safety if the outlets are plugged.

3. Off-Stream Storage: This type of storage involves the diversion of flow out of the stream into a separate storage facility. A typical example is a side channel spillway that diverts storm flows from the stream into a storage impoundment.

4. Conveyance Storage: Conveyance storage is an often neglected form of storage, because it is dynamic and requires channel storage routing analysis to identify it. Slower flowing conveyance caused by flatter slopes or rougher channels can markedly retard the buildup of flood peaks and alter the time response of the tributaries in a watershed.

5. Wet Basins or Infiltration Basins: Wet basins are detention basins designed to maintain a permanent pool of water. In most aspects, their design is similar to other detention basins except for the permanent pool. Wet basins are used for aesthetic or water quality enhancement, or for the maintenance of fish and wildlife. All outlets are above the normal level of the pool. Infiltration basins resemble other detention basins in most respects, though they may be built without outlets. They may retain flood flows for a long period of time, for the purpose of encouraging infiltration into the groundwater.

Also presented in this section, along with the technical criteria, is the general procedure for design and evaluation of both the local detention and regional detention facilities. Any special design conditions which cannot or are not defined by these criteria shall be reviewed by the City Engineer before proceeding with design.

9.2 DESIGN STANDARDS

9.2.1 State Water Commission

9.2.1.1 State Engineer's Office

The North Dakota State Engineer, pursuant to Chapter 61-04 in Sections 61-16.1-38 and 61-16.1-53, North Dakota Century Code, and the North Dakota State Water Commission, pursuant to Section 61-02-14 of the North Dakota Century Code, have the power, authority, and general jurisdiction to regulate, control, and supervise the construction and operation of all dams within the State of North Dakota, both public and private, which they deem necessary or advisable. Also, the State Water Commission has the power and authority to work with the United States and any of its departments, agencies, or officers with regards to dams, and to do what is necessary to carry out the expressed and applied purposes of Chapter 61-02, North Dakota Century Code. For the purposes of administration, the State Engineer requires the following permits:

1. Construction Permit

   No dams which are capable of retaining or storing more than 12 1/2 acre-feet of water may be constructed in North Dakota without a permit. An application for the construction of a dam must be submitted first to the State Engineer. The application must be on State Water Commission Form No. 110 and must include complete plans and specifications for the dam or reference to standard specifications. Upon receipt, the State Engineer will review the application, except in unique circumstances, the State Engineer will complete his initial review within 45 days. He will forward the application, plans and specifications, along with any changes or conditions, to the Water Resource District within which the contemplated dam is located. Within 45 days, the Water Resource District shall consider the application and information forwarded by the State Engineer and return it to the State Engineer with its decision and comments regarding suggested changes, conditions, and modifications. The State Engineer makes the final decision and forwards that decision to the applicant and the Water Resource District (61-16.1-38, NDCC).

2. Water Permit

   If water is to be appropriated for beneficial uses as a result of the construction of a dam, a conditional water permit must be obtained prior to the initiation of the construction. An application form No. 108 and instruction booklet, for the purpose of acquiring the conditional water permit, may be obtained from the State Engineer’s office. Chapter 61-04, NDCC, establishes the procedures that must be followed prior to acquiring a conditional water permit.

3. Other Permits

The construction of a dam or detention structure may require several other permits. The Corps of Engineers may require permits under the authority of Section 404 of the Clean Water Act and Section 10 of the Rivers and Harbors Act. The North Dakota Health Department requires a permit to discharge water into a stream. The State Engineer also requires a temporary water permit for water used during construction. Additional information for these permits can be obtained by contacting the respective agencies.

9.2.1.2 Dam Safety Standards

The role of individual states concerning dam safety varies from state to state. States do have regulatory authority over dam safety, and require the owner of dams to be responsible in the design and construction, operation, maintenance and repair of dams. The state's role in dam safety is to regulate the construction or enlarging of dams, repair, alteration, maintenance, operation, transfer of ownership, and abandonment in such a manner as to best provide for public health, safety and welfare. States authority for dam safety is contained within Chapter 61 of the North Dakota Century Code. At the federal level, the 1972 National Dam Safety Act, Public Law 92-367, required an inventory of all nonfederal dams and a one time inspection of these dams nationwide. In general, the Corps of Engineers has been responsible for caring out these inspections. In the state of North Dakota, the State Water Commission is primarily responsible for dam safety in all aspects. This includes design, construction, operation and maintenance.

9.2.2 Embankment

9.2.2.1 Side Slopes and Slope Stability

Slopes on earthen embankments less than 10' in height shall not be steeper that 4 (horizontal) to 1 (vertical). For embankments greater than 10' in height, the slopes shall be such as to maintain slope stability. The City Engineer should be contacted for additional requirements. All earthen slopes shall be covered with topsoil and revegetated with grass. Criteria for grass seeding are provided in Section 7.4.3.1. Slopes on riprap earthen embankments shall not be steeper than 3 (horizontal) to 1 (vertical). For grassed detention facilities, the minimum bottom slope shall be 0.5% measured perpendicular to the trickle channel. For parking lot detention, the maximum bottom slope shall be 5%.

9.2.2.2 Seepage Control

A seepage control system should be provided for all dams and embankments where a reservoir pool will be maintained for an appreciable length of time, regardless of the depth of pond, or embankment, or foundation materials. The system should control seepage through both the embankment and the foundation and may involve measures such as downstream pervious zones, interior drainage trenches, drainage blankets, pressure relief drains, toe drains, impervious cutoff trenches, upstream and impervious blankets, and other systems.

Any pipe, conduit, wall, or structure passing through the embankment should have a sufficient number of anti-seepage collars (or devices) to increase the length of the seepage path by a minimum of 15%, with a minimum of 2 anti-seepage collars (or devices) for each spillway, pipe, wall or structure.

9.2.2.3 Material and Compaction Requirements

All fill material for embankments shall be approved by the project engineer before placement. The fill should be placed so that compacted layers do not exceed 6" in depth. Fill shall be compacted by sheepsfoot or pneumatic rollers or their equivalent. The layers shall be compacted in such a manner so that laminations do not occur between layers. Special compaction methods shall be used at pipes, wall, abutments, or structures passing through the dam to assure compaction at these areas equal to that in the rest of the embankment.

No trees, branches, stumps, roots, brush, or other vegetative organic material or frozen soil, ice, snow, or other frozen materials shall be placed anywhere within the embankment during construction. No fill shall be placed on a frozen foundation or on frozen fill.

All fill materials in an earthen embankment shall be compacted to 95% standard proctor maximum density. However, this may vary depending on material type and embankment properties. Embankment fill materials shall be compacted as close to the optimum moisture content as possible. However, this may also vary depending upon material type and desired embankment properties.

9.2.2.4 Freeboard and Depth Requirements

In general, it is difficult to set specific minimum freeboard allowances because of the many factors involved in such determinations. The design engineer will have to assess the critical parameters for each project and develop its minimum requirement. Some projects are reasonably safe without freeboard allowance because they are designed for overtopping, or other factors which minimize possible overtopping. Conversely, freeboard allowances of several feet may be necessary to provide a safe condition. Parameters that should be considered include the duration of high water levels in the pond during the design flood; effective wind pitch and reservoir depth available to support wave generations; the probability of high wind speed occurring from a critical direction; the potential wave runup on the dam or embankment based on roughness and slopes; the ability of the dam to resist erosion from overtopping waves; and ice damage.

For some facilities, however, minimum freeboard requirements have been determined. The minimum freeboard required for grassed and parking lot detention facilities is 1.0 feet above the computed 100-year water surface elevation. The minimum required freeboard for rooftop detention is 3" above the computed 100-year water surface elevation.

The maximum depth from water surface to outlet invert for parking lot detention is 6" for the 10-year design and 18" for the 100-year design. The maximum depth from water surface to outlet invert for rooftop detention is 3". There is no maximum depth for grassed detention facilities as this parameter will be governed by the site topography.

9.2.2.5 Embankment Protection

All embankment surfaces shall be protected from water, wind, and ice erosion. The protection shall cover all areas subject to wave action under normal, severe, or unusual operating conditions. Areas of a dam subject to concentrated foot and vehicle traffic shall have permanent erosion protection installed, such as paths, sidewalks or steps constructed of concrete, asphalt or rock. If a vegetative cover is to be relied upon for erosion protection, the establishment of the cover shall be considered an integral part of the construction operation. Whenever a detention pond uses an embankment to contain water, the embankment shall be protected from catastrophic failure due to overtopping. Overtopping can occur when the pond outlets become obstructed or when a storm exceeding a 100-year event occurs. Failure protection for the embankment may be provided in the form of a buried heavy riprap layer on the entire downstream face of the embankment, or a separate emergency spillway having a minimum capacity of twice the maximum release rate for the 100-year storm. Structures shall not be permitted in the path of the emergency spillway or overflow. The invert of the emergency spillway shall be set equal to or above the 100-year water surface elevation.

9.2.2.6 Foundation Design

Subsurface explorations and foundations in borrow areas (drill holes, test pits, outer holes, etc.) are required for all dams regardless of size or hazard classification. The explorations should be extensive enough to firmly establish whether a safe dam can be built at the site and to reveal the subsurface soil, rock, and ground water conditions. Minimum investigations should include at least three explorations along the centerline of the dam. At least one exploration should be made at the deepest part of the depression across which the dam will be built, and also at the location of any principal or emergency spillways. At least one exploration should extend to a depth equal to the proposed height of the dam. Large, or hazardous dams require much more extensive investigations. Subsurface explorations for long dams are required at periodic intervals along the length of the dam but their spacing is dependent upon the uniformity of the subsurface soils found at the site.

The importance of adequate foundation treatment is emphasized by the fact that approximately 40% of all earth fill failures are attributed to foundation failures. Foundations should be designed for strength, stability, seepage and settlement. Foundation preparation shall include stripping to include sod, topsoil, all organic matter and all other unsuitable material. All dams should contain a cutoff or keytrench to bond the impervious zone of the embankment to the foundation and for seepage control. Foundations should be wide enough to allow free movement of excavation and compaction equipment. Rock foundations should be grouted to a depth equal to the height of the dam, unless geologic conditions dictate otherwise.

9.2.3 Outlet Works

9.2.3.1 General

Every detention facility shall be provided with sufficient discharge capacity and/or storage capacity, to accommodate the spillway design flood without endangering the safety of the dam. An adequate spillway system shall mean that the dam has sufficient capacity to pass back to back spillway design floods occurring within a reasonable period of time.

The embankment spillway system may be either a principal spillway or principal spillway in combination with an emergency spillway. An emergency spillway is usually required when the principal spillway is controlled or a conduit. Emergency spillways are discussed in the next section. The principal spillway should be a permanent non-erodible type of construction. Its principal spillway or outlet works and associated flood storage should have sufficient capacity to pass frequently recurring floods and thus reduce the use of the emergency spillway.

Outlet works shall include a satisfactory means of dissipating the flow at its outlet to minimize the risk of erosion to the dam, downstream roads, bridges, or other structures and to assure conveyance of flow without endangering the natural environment of the stream. Inlets to conduit spillway systems must be protected by a trash rack to prevent clogging and shall be designed with antivortex devices to prevent detrimental vortexing unless evidence can be presented establishing that they are not necessary. Anti-seepage collars are required for most conduits. On any conduit where a blockage by animals can occur, they may have to be protected by an animal guard.

Each embankment or dam should have the means to permit emergency drainage of the reservoir or pond to an acceptable level within a reasonable period of time. In determining an acceptable level and a reasonable time, consideration will be given to the damage potential posed by possible failure, risk and nature of potential failure capability and stability of available drainage courses to convey the waters released in the event of an emergency dewatering, and influence of rapid drawdown on the stability of the dam.

9.2.3.2 Outlet Configuration

Figures No. 9.1, 9.2, and 9.3 depict four examples for detention pond outlet configurations. These configurations are not meant to be all inclusive. There are many more outlet configurations which are possible, depending upon the ingenuity of the design engineer.

A Type 1 outlet consists of a graded drop inlet, outlet pipe, and an overflow weir in the pond embankment. The control for the 10-year discharge shall be at the throat of the outlet pipe under the headwater as determined on Figure No. 9.1. The grate must be designed to pass the 10-year flow with a minimum of 50% blockage (i.e., twice the 10-year flow). Since the minimum size of the outlet pipe is 18", then a control orifice at the entrance of the pipe may be required to control the discharge of the design flow.

The difference between the 100-year and 10-year discharge is released by an overflow weir or spillway. The surcharge on the outlet pipe will be included in the total discharge when sizing the overflow weir or spillway. If sufficient pond depth is available, the drop inlet and grate can be replaced by depressed inlet with a headwall and trash rack. Depression of the inlet is required to reduce nuisance backup flow into the pond during trickle down flows. The maximum trash rack opening dimension shall be equal to the minimum opening in the orifice plate.

A Type 2 outlet consists of a drop inlet with an orifice controlled inlet for the 10-year discharge and a crest overflow and pipe inlet control for the 100-year discharge. Type II outlet configuration is also shown on Figure No. 9.1. The control for the 10-year discharge occurs at the orifice opening for the head as shown on the figure. The control for the 100-year discharge occurs at the throat of the outlet as shown on the figure. However, the difference between the 100-year and 10-year discharge must pass over the weir and therefore the weir must give adequate length. The effective weir length (L) occurs for three sides of the box. To insure the 100-year control occurs at the throat of the outlet pipe a 50% increase in the required weir length is recommended. Section 7.3.6.1 in Chapter 7 has a discussion of weir flow. In addition, the outlet pipe must have an adequate slope to ensure throat control in the pipe.

A Type 3 outlet is shown in Figure 9.2. The Type 3 outlet is similar to the Type 2 outlet except that the trash rack covers the entire outlet works. The 10-year and 100-year discharge control points are the same as the control points as the Type 2 outlet.

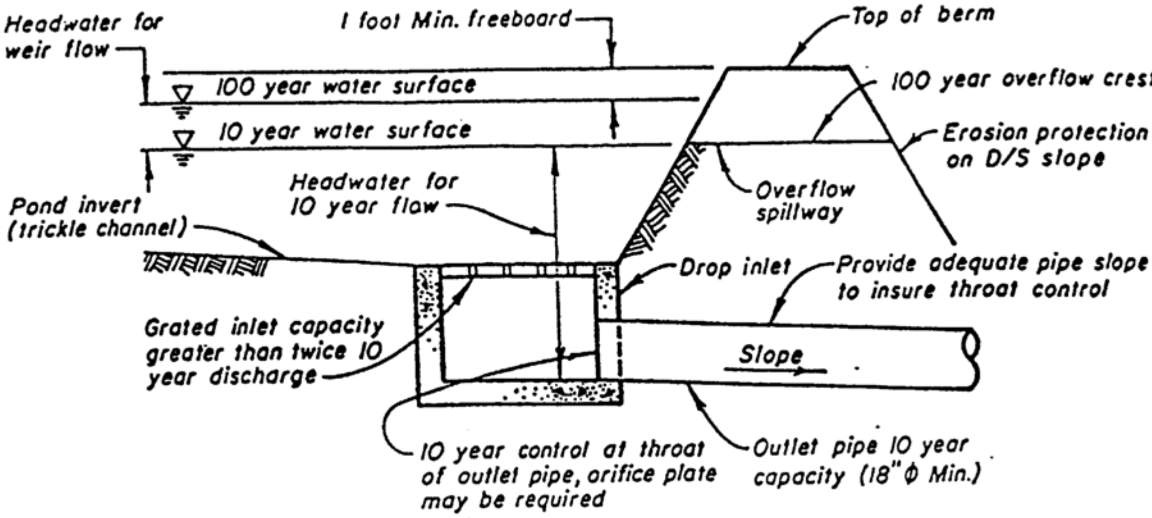
Figure No. 9.3 shows the configuration of a Type 4 outlet. Type 4 outlet is designed for use with parking lot detention. The outlet consists of a weir curve opening for the 10-year flow control and a weir overflow section for the 100-year flow control. The outlet channel shall be riprapped to prevent erosion of the berms or undermining of the curb and asphalt parking lot.

There are many other outlet configurations that can be utilized in conjunction with stormwater detention facilities. The ability of outlet facilities to convey the design discharges for the project to function as intended must be documented in the Stormwater Management Plan Report. Unique outlet structures and configurations must receive the approval of the City Engineer.

9.2.3.3 Structure Materials

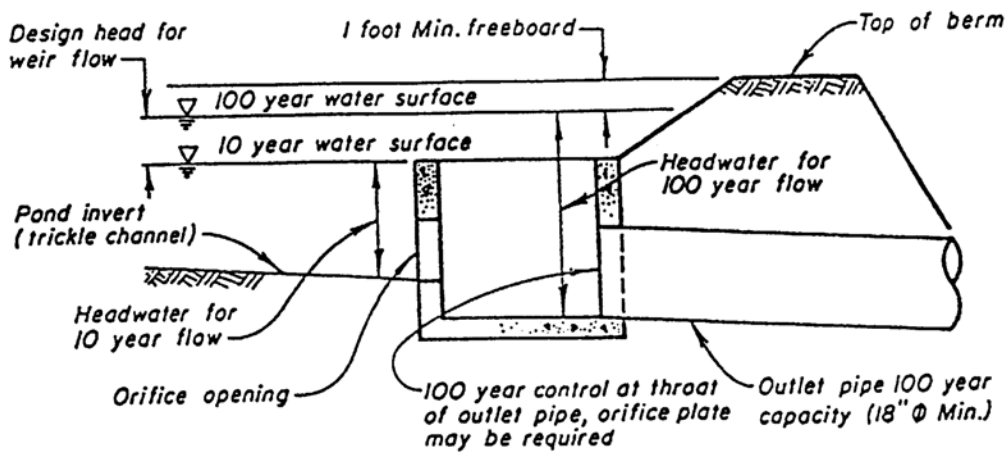
The main conduit materials for outlet works can include RCP or CSP culvert. Special structures such as weirs or inlet boxes shall be constructed of reinforced concrete. Such structures shall be sufficient to resist sliding, overturning, and uplift. In the case of corrugated steel products, some manufacturer's provide predesigned drop inlets and risers, trash racks and antivortex walls, and anti-seepage collars. These special standard designs were developed by the Natural Resource Conservation Service.

DETENTION POND OUTLET CONFIGURATIONS



TYPE I OUTLET

No Scale

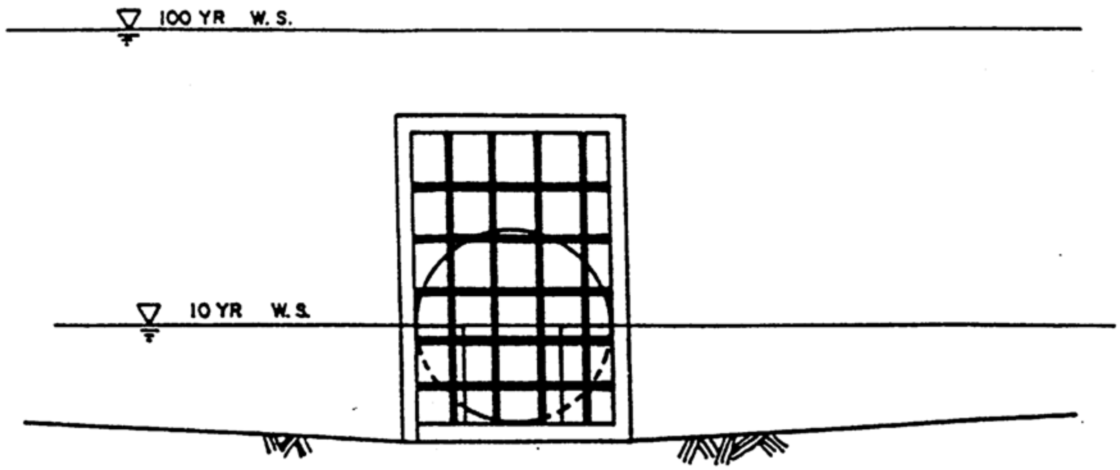


TYPE 2 OUTLET

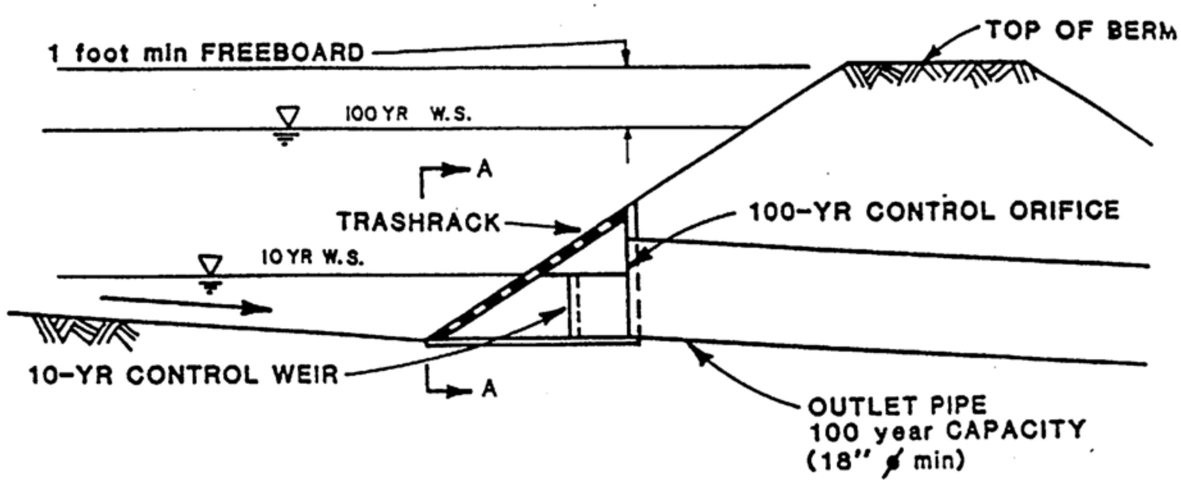
No Scale

Figure No. 9.1

DETENTION POND OUTLET CONFIGURATIONS



SECTION A-A



TYPE 3 OUTLET
NO SCALE

Figure No. 9.2

DETENTION POND OUTLET CONFIGURATIONS

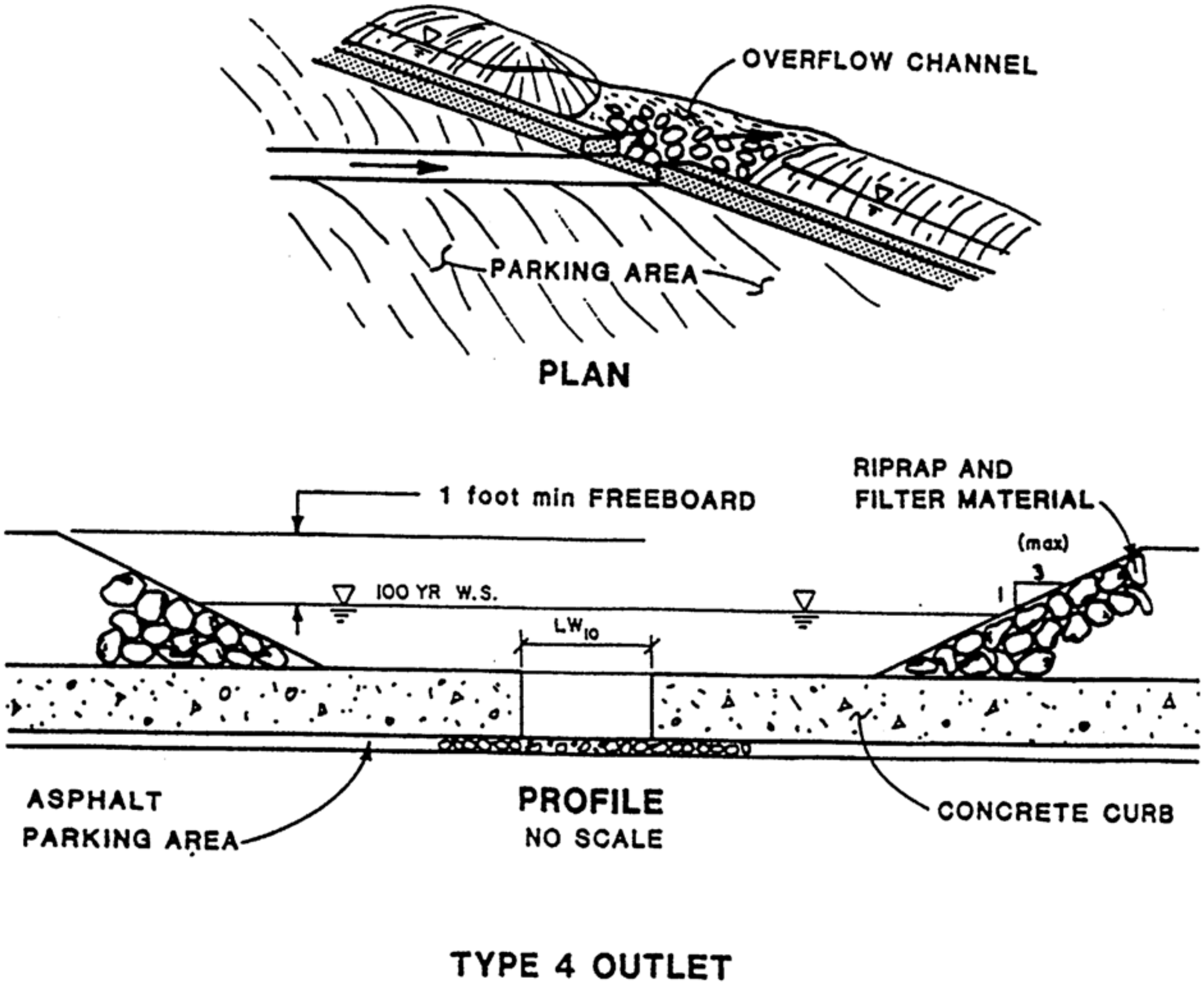


Figure No. 9.3

9.2.3.4 Structural Requirements

All outlet works structural components shall be capable of resisting design embankment loads, lateral earth pressure forces, and anticipated hydraulic loading. Numerous references are available for determining the class or gauge of the outlet pipe conduit. These are usually in the form of tables or charts for various loading conditions. Special concrete structures shall be designed in accordance with standard criteria for the design of concrete hydraulic structures. Numerous references are available for use in the design of these facilities from the Natural Resources Conservation Service, the Bureau of Reclamation, or the Corps of Engineers.

9.2.4 Emergency Spillway

9.2.4.1 General

The purpose of an emergency spillway is to prevent the embankment or dam from suffering a catastrophic failure due to overtopping. As was pointed out earlier, overtopping can occur when the pond outlets become obstructed or when a larger than a 100-year storm event occurs. The inlet or low point in an emergency spillway should be set equal to or above the 100-year water surface elevation. Emergency spillway discharges should be controlled or directed away from the dam so as not to endanger the stability of the embankment and should be returned to the natural water course far enough downstream to prevent erosion of the embankment toe.

9.2.4.2 Configuration

Emergency spillways, whenever possible, should be located away from the dam in an undisturbed area. Topographic saddles generally make good sites. If the embankment is also used as a roadway, then a depression in the roadway well away from the principle spillway conduit can be made to function as an emergency spillway. The layout and profile of vegetative spillways should provide a maximum bulk of material to ensure safety against breaching at the spillway during the passage of the emergency spillway flood.

9.2.4.3 Design Considerations

Excavated spillways consist of three elements: the intake channel, the control channel and the exit channel. The relationship between the water surface elevation in the pond and the discharge through the emergency spillway should be evaluated by computing the head losses in the inlet channel upstream of the control section. If a control section is not used, the spillway should be evaluated by computing the water surface profile through the full length of the spillway. Subcritical flow exists in the inlet channel. The flow conditions change to critical in the control section.

It is recommended that the inlet channel be level for a minimum distance of 30 feet upstream of the control section. This level part of the inlet channel should be the same width as the exit channel and its centerline should be straight and coincide with the centerline of the exit channel. A curved centerline is permissible in the inlet channel upstream of the level section, but it must be tangent with the level section.

The centerline of the exit channel must be straight and perpendicular to the control section for a distance equal to one half of the maximum base width of the dam. Curvature may be introduced below this point, provided that the flowing water will not impede on the dam should the channel fail at the curve. The exit channel should be as long as reasonably practical. It should be of sufficient slope to ensure supercritical flow for all discharges greater than 25% of the spillway design flood. The maximum velocity limitations for vegetated channels shall apply to the exit channel.

9.3 HYDRAULIC DESIGN CRITERIA

9.3.1 Local ("On-site") Detention

1. Equation Detention Method: The "traditional" method of computing detention storage requirements utilizes the Rational Method to size on-site detention for watersheds that are too small for analysis using the more rigorous methodologies such as TR20, HEC-1, TR55, etc. Because of the inexact nature of this procedure, errors can easily occur in determining the appropriate release rate(s) to maintain "historical" or "undeveloped" flow rates. The procedure outlined in this section uses allowable maximum unit area release rates and minimum volume requirements at these release rates for sizing detention ponds. This methodology is referred to as the Equation Detention Method. The method is based on actual modeling results and represents the average of conditions that will be encountered. However, the simplification of the process and the consistency in the detention analysis are considered to compensate for the site specific differences that may occur.

   Local detention facilities are to be designed using the "equation detention method" if flows only from the subject property are routed through the detention facility(s). There may be more than one local detention facility on the subject property. If the facilities are sequential (i.e., flows routed from one facility to another), then the sequential detention procedure shall be utilized (Section 9.3.1-3). Any undetained runoff shall meet the requirements of Section 9.3.1-2.

   For the "equation detention method", the minimum required volume and the maximum release rates at the ponding depths corresponding to the volumes shall be determined using the following equations:

   Minimum Detention Volume:

   $V = KA$ (Equation 9.1)

   For the 100-year,

   $K_{100} = (1.78I - 0.002I^2 - 3.56)/1000$ (Equation 9.2)

   For the 10-year,

   $K_{10} = (0.95I - 1.90)/1000$ (Equation 9.3)

where	V	=	required volume for the 100- or 10-year storm (acre-feet)
	I	=	Developed basin imperviousness (%)
	A	=	Tributary area (acres)

Maximum Release Rates:

$$Q_{100} = 1.00A \qquad \text{(Equation 9.4)}$$
$$Q_{10} = 0.24A \qquad \text{(Equation 9.5)}$$

Where Q_{100}, Q_{10} = release rates for the 100- and 10-year storms, respectively (cfs).

Example No. 9.1: Equation Detention Method

Given: A basin that has the following characteristics:
Basin Area (A) = 23 acres
Basin Imperviousness (I) = 55%
Off-site Area = 0 acres
Undetained Area = 0 acres
One detention facility

Required: 100-year and 10-year storage volumes and release rates.

Solution:

Step 1: Determine K_{100} using Equation 9.2

$$\begin{aligned} K_{100} &= (1.78\ I - 0.002\ I^2 - 3.56)/1000 \\ &= (1.78\ (55) - 0.002(55)^2 - 3.56)/1000 \\ &= 0.0883 \end{aligned}$$

Step 2: Determine K_{10} using Equation 9.3

$$\begin{aligned} K_{10} &= (0.95I - 1.90)/1000 \\ &= (0.95 \times 55 - 1.90)/1000 \\ &= 0.0504 \end{aligned}$$

Step 3: Determine minimum required 100-year storage volume using Equation 9.1

$$\begin{aligned} V &= KA \\ &= 0.0883 \times 23 \\ &= 2.03 \text{ acre-feet } (88{,}500 \text{ ft.}^3) \end{aligned}$$

Step 4: Repeat Step 3 for 10-year storage

$$V = KA$$
$$= 0.0504 \times 23$$
$$= 1.16 \text{ acre-feet } (50{,}500 \text{ ft.}^3)$$

Step 5: Determine maximum allowed 100-year release rate

$$Q_{100} = 1.00\ A$$
$$= 1.00 \times 23$$
$$= 23 \text{ cfs}$$

Step 6: Repeat Step 5 for 10-year release rate

$$Q_{10} = 0.24A$$
$$= 0.24 \times 23$$
$$= 5.5 \text{ cfs}$$

2. Compensating Detention Procedure: Local detention facilities are to be designed using the "compensating detention procedure" if any runoff is to flow undetained from the subject property. There may be more than one local detention facility on the site. The compensating detention procedure requires that the total release rates from the detained and undetained areas be equal to the allowable release rates from the total site. Therefore, the more undetained runoff, the less the allowable detention facility release rate. The limit on the undetained area is 5 percent or 5 acres, whichever is less.

   Compensating detention is to be computed as follows:

   a. Minimum Detention Volumes: The minimum detention volumes shall be calculated using Equations 9.1 through 9.3. The imperviousness and area parameters for the basin contributing the undetained runoff shall be included in the minimum detention volume calculation as if this basin was tributary to the detention facility.

   b. Maximum Release Rates: The maximum release rates shall be calculated in two steps. First, the allowable release rates from the whole basin (detention and undetained areas) shall be computed using Equations 9.4 and 9.5. Then, the maximum release rates from the detention facility are set equal to the maximum release rates from the whole site minus the runoff rates from the undetained area.

Example No. 9.2: Compensating Detention Method

Given: Total Basin Area (A) = 23 acres
Basin Imperviousness = 55%
Off-site Area = 0 acres
Undetained Area = 0.8 acres
10-year Peak Runoff from Undetained Area = 2.4 cfs
100-year Peak Runoff from undetained Area = 4.2 cfs
One detention facility

Required: 100-year and 10-year storage volumes and release rates

Solution:

Step 1: Compare the undetained area to the allowable undetained area:

Allowable undetained area = 5% of total area
= 0.05 x 23 = 1.15 acres
= 0.8 acres < 1.15 acres - acceptable

Step 2: Determine minimum required 100-year and 10-year storage volumes.
Some as Step 1 and 4 in Example No. 9.1

V_{100} = 2.03 acre-feet
V_{10} = 1.16 acre-feet

Step 3: Determine 100-year and 10-year allowable release rates from the total site.

Q_{100} = 1.00 A
= 1.00 x 23
= 23 cfs

Q_{10} = 0.24 A
= 0.24 x 23
= 5.5 cfs

Step 4: Determine maximum allowable 100-year release rate from the detention facility

Q (detention) = Q (total site) - Q (undetained)
= 23 - 4.2
= 18.8 cfs

Step 5: Repeat Step 4 for 10-year release rate

Q (detention) = Q (total site) - Q (undetained)
= 5.5 - 2.4 = 3.1 cfs

3. Sequential Detention Procedure: Local detention facilities are to be designed using the "sequential detention procedure" if any storm runoff is detained by two or more detention facilities in sequence before leaving the subject property. The sequential detention method accounts for the inherent decrease in efficiency of two sequential detention facilities versus one facility by considering the released runoff from one facility to be equivalent to runoff from an incremental area tributary to the second facility. Thus, the storage volume of the second facility is increased to accommodate the incremental area runoff. By minimizing the second detention facility's release rate, the volumes of any additional sequential facilities are minimized.

   Sequential detention facilities are to be designed using the Standard Form shown in Figure No. 9.4. The form is divided into two parts: Singular Detention and Sequential Detention. The singular detention part is for listing and computing the parameters associated with a single detention facility. Each facility is analyzed using the "equation detention method" criteria and the "compensating detention procedure" criteria, if required.

   The sequential detention part of the form evaluates the combined effect of the detention facilities. The results of the second part computations will yield the minimum volume required and the maximum release rates allowed for each detention facility. The description of Figure No. 9.4 is as follows:

   Col. 1: Facility Number: Designated number of the detention facility being analyzed.

   Col. 2: Basin Area: Area of basin (sub-basin) tributary to the detention facility not including any area tributary to an upstream detention facility.

   Col. 3: (Q_I): Peak inflow in cfs from the area described in Column 2.

   Col. 4: IMP %: Percent imperviousness of the area described in Column 2 to be used in Equations 9.2 and 9.3.

   Col. 5: K: K-factor calculated from Equations 9.2 and 9.3 and the percent imperviousness (IMP %) in Column 4.

   Col. 6: ($Q_{I/A}$): Peak inflow (Q_I) in Column 3 divided by the area (A) in Column 2.

   Col. 7: ΣQ: Peak inflow into detention facility computed by summation of the peak inflow in Column 3 and the maximum release rate from the detention facilities just upstream in Column 10.

SEQUENTIAL DETENTION CALCULATION

SUBDIVISION ______________________________

CACULATED BY ____________ DATE __________

		SINGULAR DETENTION					SEQUENTIAL DETENTION			
	FACILITY NUMBER	BASIN AREA (A) Ac	Q_i CFS	IMP %	K Ft	Q_i/A CFS/Ac	ΣQ CFS	Z Ac	S_m Ac-Ft	Q_m CFS
	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
10-YEAR										

100-YEAR										

$\Sigma Q = Q_i +$ Previous Q_m

$Z = \Sigma Q/(Q_i/A)$

$S_m = KZ$

Figure No. 9.4

Col. 8: Z: Equivalent inflow area computed by dividing Column 7 by Column 6 ($\Sigma Q/ (Q_I/A)$).

Col. 9: Minimum S_m: Minimum allowed storage volume for the respective detention facility computed using Equation 9.1 and the parameters in Column 5 (K-factor) and Column 8 (Z = A).

Col. 10: Maximum Q_m: Maximum allowed release rate for the respective detention facility computed using Equations 9.4 and 9.5, and the Z parameter in Column 8.

Example No. 9.3: Sequential Detention Procedure

Given: Drainage basin shown on Figure No. 9.5. The following sub-basin parameters:

SUB-BASIN	AREA ACRES	& IMP	Q_{10} CFS	Q_{100} CFS
A-1	11	40	21.0	47.0
A-2	7	70	23.0	41.0
A-3	10	40	17.0	37.0
A-4	16	50	29.0	57.0
B-1	9	45	23.0	47.0

Undetained Area = 0 acres

Off-site flow from B-1 is routed around the development.

Required: 10-year and 100-year storage volumes and release rates for all detention facilities.

Solution:

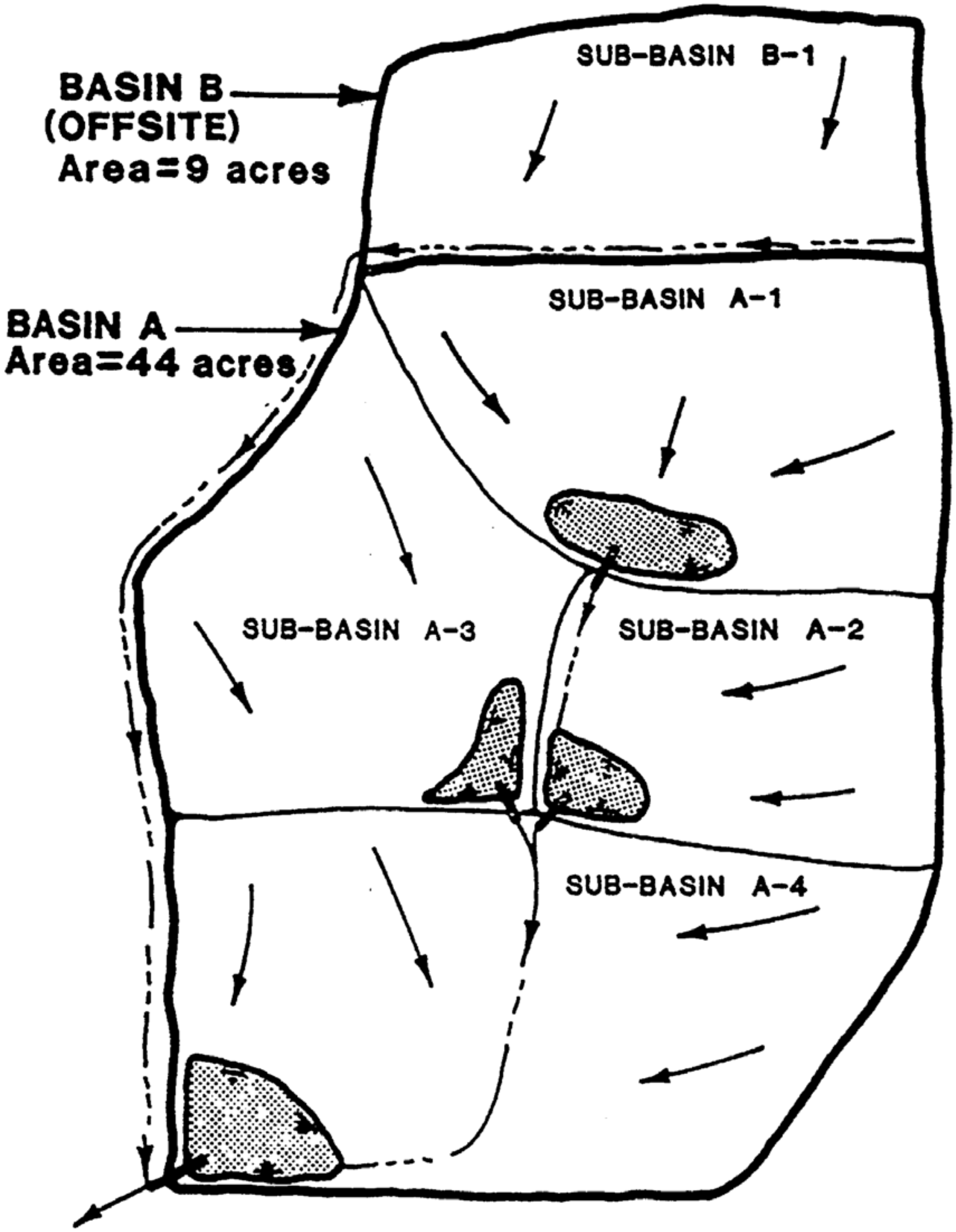
Step 1: Using the Standard Form for Sequential Detentions, fill in the given 10-year sub-basin parameters in Columns 1, 2, 3, and 4 for the uppermost detention facility.

Step 2: Compute K_{10} = (0.95I - 1.90)/1000
= (0.95 (40) - 1.90)/1000
= 0.0361

Step 3: Divide Column 3 by Column 2 and enter the result in Column 6

$Q_I/A = 21/11 = 1.909$ cfs/acre

Example No. 9.3
Figure No. 9.5
Sequential Detention Method



Example No. 9.3
Figure No. 9.6

SEQUENTIAL DETENTION CALCULATION

SUBDIVISION ____________
CACULATED BY AJL DATE Feb. 24, 1984

	FACILITY NUMBER (1)	SINGULAR DETENTION: BASIN AREA (A) Ac (2)	Q_I CFS (3)	IMP % (4)	K Ft (5)	Q_I/A CFS/Ac (6)	SEQUENTIAL DETENTION: ΣQ CFS (7)	Z Ac (8)	S_m Ac-Ft (9)	Q_m CFS (10)
10-YEAR	A-1	11	21	40	0.0361	1.909	21.0	11.0	0.40	2.6
	A-2	7	23	70	0.0646	3.286	25.6	7.8	0.50	1.9
	A-3	10	17	40	0.0361	1.700	17.0	10.0	0.36	2.4
	A-4	16	29	50	0.0456	1.813	33.3	18.4	0.84	4.4
100-YEAR	A-1	11	47	40	0.0644	4.272	47.0	11.0	0.71	11.0
	A-2	7	41	70	0.1112	5.857	52.0	8.9	0.99	8.9
	A-3	10	37	40	0.0644	3.700	37.0	10.0	0.64	10.0
	A-4	16	57	50	0.0804	3.563	75.9	21.3	1.71	21.3

Step 4: Compute the 10-year peak inflow to the detention facility. For this facility, the peak inflow is equal to the peak inflow from sub-basin A-1 or 21.0 cfs. Enter the result in Column 7.

Step 5: Divide Column 7 by Column 6 and enter the result in Column 8

$$Z = \text{sum of } (Q/Q_I/A))$$
$$= 21.0/1.909$$
$$= 11.0 \text{ acres}$$

Step 6: Compute the minimum required 10-year storage volume as shown in Example 9.1 using Z = A and K_{10} from Column 5. Enter the result in Column 9.

$$V = KZ$$
$$= 0.0361 (11.0) = 0.40 \text{ acre-feet } (17{,}300 \text{ ft.}^3)$$

Step 7: Compute the minimum allowable 10-year release rate as shown in Example 9.1 using Z = A and enter the result in Column 10.

$$Q_{10} = 0.24Z$$
$$= 0.24 (11.0)$$
$$= 2.6 \text{ cfs}$$

Step 8: Repeat Steps 1 through 3 for the next detention facility

$$K_{10} = 0.0646$$
$$Q_i/A = 3.286$$

Step 9: Repeat Step 4 but include the peak outflow from detention facility A-1.

$$Q = 23 + 2.6$$
$$= 25.6 \text{ cfs}$$

Step 10: Repeat Steps 5 through 7

$$Z = 7.8$$
$$S_m = 0.50 \text{ acre-feet}$$
$$Q_m = 1.9 \text{ cfs}$$

Step 11: Repeat Steps 1 through 7 for the remaining detention facilities for both the 10-year and 100-year floods.

9.3.2 Regional Detention

Minimum required volumes and maximum allowable release rates for regional detention facilities shall be determined by hydrograph methods commonly used for analysis of floods and stormwater runoff. Typical hydrograph methods which are recognized and generally accepted include TR55-Urban Hydrology for Small Watersheds (National Resource Conservation Services), TR20 - Computer Programs for Project Formulation Hydrology (National Resource Conservation Service), and HEC-1 - Flood Hydrograph Package (Corps of Engineers).

The hydrograph methodology uses hydrographs computed from an acceptable hydrograph generation package (TR20, TR55, HEC-1) to determine the minimum storage volume required for each detention facility. The generated hydrographs are routed using methods discussed in several hydrology general reference texts (i.e., Viessman et. al., Linsley et. al., Ponce, and McCuen). When including off-site basins that discharge from off-site detention facilities, the generated hydrographs are routed through the facilities using the as-built storage volumes and release rates for the facility. The maximum allowable release rates per on-site detention facilities shall be equal to the generated historic peak runoff at the respective design points. The minimum required storage volume for on-site detention facilities shall be the storage volumes computed from routing the generated hydrographs with the maximum release rates to the historic peak runoff at the respective design points.

9.3.3 Detention Method Selection

This chapter has outlined several possible detention outlet configurations and methodologies for the analysis and design of detention facilities. Methods outlined herein are not meant to be all inclusive. Stormwater detention concepts are constantly evolving through new technology and new developments. The project engineer is not limited to the outlet configuration or methodologies suggested herein. The project engineer is cautioned, however, to secure prior approval of the City Engineer before using any unique methodology not cited herein.

9.4 STORMWATER MANAGEMENT REPORT REQUIREMENTS

The analysis and design of stormwater detention facilities and the selection of facility components shall be thoroughly documented within the Stormwater Management Plan Report. A copy of the project engineer's calculations for the stormwater detention facilities shall include:

1. Inflow Hydrographs: Stage - Storage, Stage - Discharge, and Routing Curves; and such other routing computations which are the basis of the design.

2. Calculations of allowable discharge, required storage volumes and calculated ponding elevations.

3. Such other calculations and material require to complete the design.

4. Certification by a Professional Engineer Registered in the State of North Dakota that the stormwater detention facility design and calculations were performed by the engineer, or under the engineer's supervision, and that the facilities and design meet the criteria of this chapter.

CHAPTER 10
WATER QUALITY PROTECTION

10.1 INTRODUCTION

10.1.1 Hydrologic Impacts

There are two main reasons why urban development increases pollutant loads in runoff. First, the volume and rate of runoff are typically increased as an area is developed, providing a larger capacity to transport pollutants. Second, some materials are typically made more available for loss in runoff as the intensity of land use increases.

When an undeveloped area changes to support urban land uses, drastic changes in the local hydrology result. As land is covered with roads, buildings, and parking lots, the amount of rainfall that can infiltrate into the soil is reduced. This increases the volume of runoff from the watershed. Figure No. 10.1 shows the relationship of runoff, infiltration, and evaporation for watersheds with varying degrees of impervious cover. Typical impervious cover percentages are shown in Table 10.1.

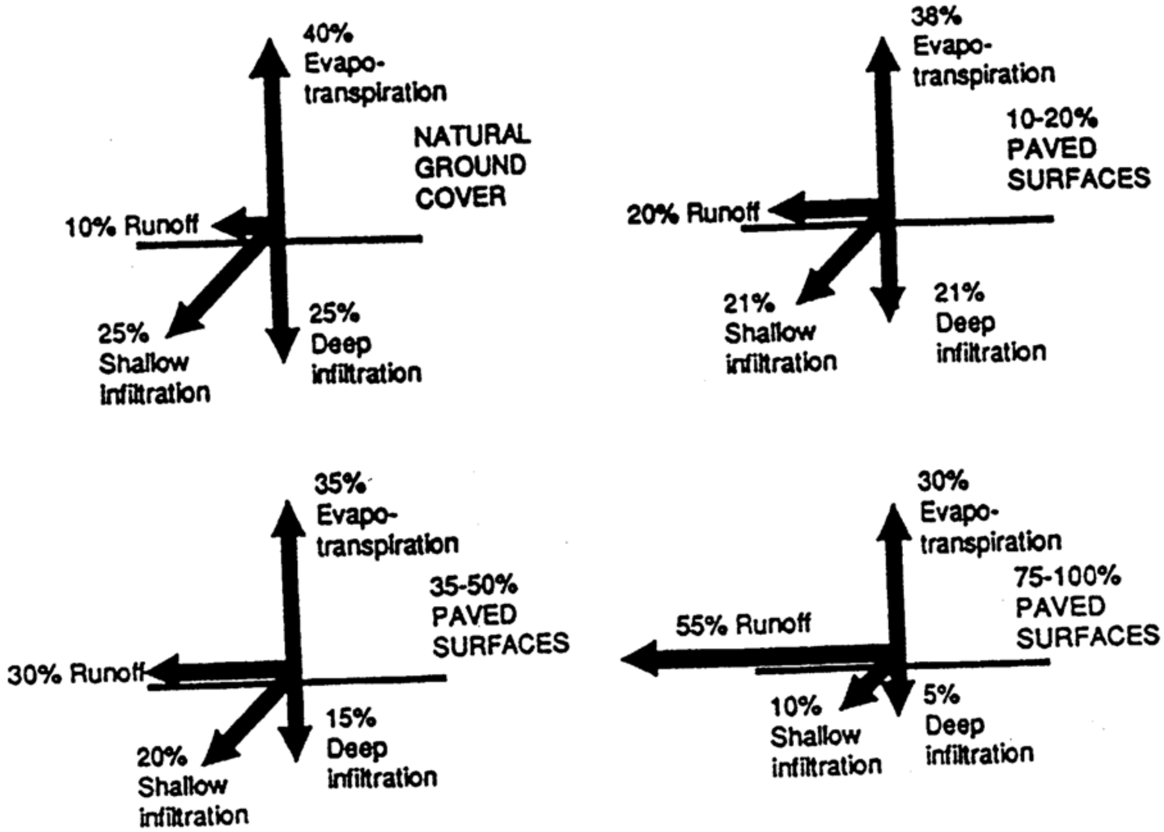
Table No. 10.1
Typical Impervious Area Percentages

Land Use	Percent Impervious Cover
Business District of Shopping Center	95-100
Residential, High Density	45-60
Residential, Medium Density	35-45
Residential, Low Density	20-40
Open Areas	0-10

When an urban area is developed, natural drainage patterns are modified as runoff is channeled into road gutters, storm sewers, and paved channels. These modifications increase the velocity of runoff, which decreases the time required to carry it to the mouth of the watershed. This results in higher peak discharges and shorter times to reach peak discharge. The increased volume of runoff after development is significant because of the increased pollutant loading it can deliver, as well as potential flooding and channel erosion problems.

10.1.2 Urban Nonpoint Source Pollutants

The major nonpoint source pollutants include sediment, nutrients, trace metals, oxygen-demanding substances, toxic chemicals, bacteria, hydrocarbons and chloride.



IMPACT OF PAVED SURFACES ON RUNOFF

Figure No. 10.1

1. <u>Sediment</u>. Sediment is made up of tiny soil particles that are washed or blown into lakes and streams. It is considered to be one of the most damaging pollutants. Sediment fills in road ditches, streams, lakes, rivers, and wetlands. It can affect aquatic life by smothering fish larvae and eggs. Suspended soil particles cause water to look cloudy or turbid. Excessive turbidity reduces light penetration in water, impairs site feeding fish, clogs fish gills, and increases drinking water treatment costs. Fine sediment also acts as a vehicle to transport other pollutants including nutrients, trace metals, and hydrocarbons to nearby surface waters. Runoff from construction sites is by far the largest source of sediment in urban areas under development. Another major source of sediment is streambank erosion, which is accelerated by increases in peak rates and volumes of runoff due to urbanization.

2. <u>Nutrients</u>. Nutrients are also a major concern for surface water quality because of the effect it can have on lakes and rivers. Nutrients, especially phosphorous and nitrogen, can cause algal blooms and excessive aquatic plant growth in lakes and rivers. Of the two, phosphorous is usually the limiting nutrient that controls the growth of algae. As phosphorous loadings rise, the potential for algal blooms and accelerated lake eutrophication also increases. The ammonium form of nitrogen can also have severe effects of surface water quality.

   The ammonium is converted to nitrate and nitrite forms of nitrogen in a process called nitrification. This process consumes large amounts of oxygen and can result in fish kills by lowering dissolved oxygen levels in water. These conditions can impair many important uses of waters, including recreation, fish habitat, and water supply. The nitrate form of nitrogen is very soluble and is found naturally at low levels in water. When nitrogen fertilizer is applied to lawns and other areas in excess of plant needs, nitrates can leach below the root zone, eventually reaching groundwater. Water contaminated with high levels of nitrates presents a clear health hazard to young infants whose formula is prepared with it.

3. <u>Trace Metals</u>. Trace metals are a water quality concern because of the toxic effects they can have on aquatic life. The most common trace metals found in urban runoff are lead, zinc, and copper. Fallout from automobile emissions is a major source of lead in urban areas. Lead and zinc in urban runoff have also been associated with application of sand and salt on roads. The toxicity of trace metals in runoff varies with the hardness of the receiving water. As the total hardness of the water increases, the threshold concentration levels for adverse biological effects increases. Many of these metals become attached to the fine sediment and are carried with it until the sediment settles out. When these materials settle out, they can accumulate over a period of time to levels that are harmful to aquatic life.

4. Oxygen-Demanding Substances. While land animals extract oxygen from the air, aquatic life depends on oxygen dissolved in water. When organic matter is consumed by microorganisms, dissolved oxygen is consumed in the process. After it rains, urban runoff can deposit large quantities of oxygen-demanding substances in lakes and streams. The biochemical oxygen demand (BOD) of typical urban runoff is on the same order of magnitude as the effluent from the efficiently run secondary wastewater treatment plant. This can create a pulse of high oxygen demand during storm runoff that can totally deplete oxygen supplies in shallow, slow moving, or poorly flushed waters. Oxygen depletion is a common cause of fish kills. In urban areas, pet wastes, street litter, and organic matter are common sources of oxygen-demanding substances.

5. Bacteria. Bacteria levels are most severe in the summer when temperatures are most favorable for the reproduction. Studies have found that total coliform counts exceeded EPA water quality criteria almost every time it rained. The coliform bacteria that are detected may not be a health risk in themselves, but are often associated with other pathogens. Pet and other animal wastes are common sources of bacteria in urban areas.

6. Hydrocarbons. Petroleum-driven hydrocarbons are commonly found in urban runoff. These materials initially float on water and create the familiar rainbow-colored film. Hydrocarbons have a strong affinity for sediment and quickly become absorbed to it. Hydrocarbons are then transported with sediment and settle out with it. Hydrocarbons are a concern because they are known to be toxic to aquatic organisms at relatively low concentrations. Common sources of hydrocarbons are spillage at oil storage and fueling facilities, leakage from crank cases, and improper disposal of drain oil.

7. Chloride. During winter conditions, salt is sometimes used to melt ice from roads, parking lots, and sidewalks. Because it is extremely soluble, almost all salt applied ends up in surface or groundwater. If the concentration of chloride becomes too high, it can be toxic to many fresh water organisms. Normal application of salt to roads for de-icing is unlikely to create toxic conditions due to elevated chloride levels. However, there have been numerous documented cases of surface and groundwater contamination caused by runoff from inadequately protected stockpiles of salt and sand-salt mixtures.

10.1.3 Pollutant Delivery Process

Understanding the pollutant delivery process is fundamental to nonpoint source pollution control. There are three steps in the delivery process: availability, detachment, and transport. Most substances must go through this entire chain before they can become pollutants. Breaking this chain at any step will prevent a substance from being delivered to receiving waters. Some pollutants are more readily controlled at a particular step in the delivery process. A basic understanding of this process and the characteristics of the pollutants in question helps to target Best Management Practices (BMPs) so they can prevent delivery most effectively.

1. Availability. Obviously a material must be available before it can become a potential pollutant. The quantity of the material in the environment and its characteristics determine the degree of availability. In an urban environment, the quantity of certain pollutants in the environment is a function of the intensity of the land use. For instance, a high density of automobile traffic makes a number of potential pollutants, such as lead and hydrocarbons more available. Control methods, such as street sweeping, would reduce the availability of these pollutants. Availability of a material such as fertilizer is a function of the quantity and the manner in which it is applied. Applying fertilizer in quantities that exceed plant needs leaves the excess nutrients available for loss to surface or groundwater. Reducing the availability of fertilizers through reduced use and proper application is the best way to control nonpoint source pollution from these materials.

2. Detachment. Detachment is the process in which materials are dislodged from their original location and become mobile. The attachment process can be either physical or chemical. Most physical detachment is the result of raindrop impact or overland flow. A chemical detachment involves dissolving soluble materials or ion exchange processes. Control of pollutant delivery in the detachment phase is most practical for materials such as sediment when erosion control practices are used to prevent the detachment of soil particles. Once the soil particles are detached, coarser particles can be trapped effectively by sediment control practices. However, fine soil particles are not readily trapped except by detention practices with very long detention times.

3. Transport. Transport is the final phase of the delivery process. Transport involves moving the material from its point of detachment to receiving water. In urban areas, a large part of the runoff is transported to receiving waters over impermeable surfaces such as streets or in storm sewers. This results in very efficient transport of pollutants to receiving waters once they are detached. For many urban nonpoint source pollutants, especially those associated with sediment, interrupting transport is the most practical way to prevent their delivery to receiving waters. Detention or infiltration practices can be effective for interrupting transport from the pollutants.

10.2 CONTROLS FOR WATER QUALITY PROTECTION

10.2.1 Objectives

The control of stormwater runoff quality can be characterized as more an engineering art than a science. Design criteria for pollutant removal are a continuous evolving process. However, it is possible to establish some general objectives toward which design efforts may be directed in order to achieve a significant reduction in the pollution load to downstream areas including:

1. The most effective runoff quality controls reduce peak and volume - these are generally infiltration controls.

2. The next most effective controls reduce runoff peak - These controls generally involve storage.

3. For small runoff events (those with return intervals of less than two years), the runoff should be retarded by detention in order to control downstream erosion.

4. The most obnoxious pollutants in urban runoff are settleable; however, appreciable amounts of nutrients and some heavy metals are dissolved and require treatment.

5. Design events for runoff quality are small frequent events (smaller than the one year runoff event). This design approach is significantly different than that for the design of systems to control runoff quantity, which is usually designed for large infrequent runoff events (10, 25, 50, or 100-year event).

10.2.2 Source Controls

Source controls emphasize the prevention and reduction of nonpoint pollution by eliminating the opportunity for pollutants on the land surface to be entrained into surface runoff. Being alert to the presence of chemicals and pollutants in areas subjected to rainfall or runoff is the first step in source control. Modifying the site plan and/or drainage plan is the second step toward eliminating these sources.

Source control is a difficult matter. Homeowners generally are not restricted in the application of nutrients, herbicides, and pesticides to their yards. Chemical spillages in roadways, dustfall, etc., are also difficult to control. Nonetheless, attention to good housekeeping practices while designing the layout of the surface drainage system can significantly reduce the inevitable sources, and Best Management Practice (BMP) programs now being introduced promise to reduce or eliminate some sources and their resultant impacts on water quality. Typical housekeeping BMPs are as follows:

1. Fertilizer Management. Fertilizer management involves control of the rate, timing, and method of fertilizer application in urban areas so that plant nutrient needs are met while minimizing the chance of polluting surface or groundwater. This practice is directed at control of phosphorous and nitrogen and runoff from landscaped areas. Fertilizer management can be an effective practice for the control of nutrients from landscaped areas. Significant nutrient loads can result from overapplication of lawn fertilizer in urban areas. Restricting the amount of fertilizer applied to the quantity needed for plant growth will minimize the potential for surface or groundwater contamination.

2. Litter Control. Litter control involves the removal of litter from streets and other surfaces before runoff or wind moves these materials to surface waters. This practice will prevent litter from becoming pollution as well as improving the aesthetics of the area. A major source of phosphorous in urban runoff is leaves and lawn clippings. Removing these materials before they can enter surface waters can reduce phosphorous loadings significantly. In addition to leaves and lawn clippings, litter that should be controlled includes pet wastes, trash, oil, and chemicals. There are two categories of litter control programs: source reduction and removal programs. Source reduction includes recycling, public education, and litter container placement. Litter removal programs include refuse and leaf collection, street cleaning, and catch basin cleaning.

3. Catch Basin Cleaning. Catch basins are chambers or sumps installed in storm sewers, usually at the curb, which allows surface runoff to enter the sewer. These catch basins have a low area intended to retain sediment. By trapping coarse sediment, the catch basin prevents trapped solids from clogging the sewer or being washed into receiving waters. However, these low areas must be cleaned out periodically to maintain their sediment trapping ability.

4. Street Sweeping. Street sweeping involves the removal of grit, debris, and trash from urban impervious areas such as streets, parking lots, and sidewalks. Streets are normally swept with either a mechanical broom sweeper or a vacuum sweeper. If these materials are removed from the streets where they deposited, they are no longer available for lost urban runoff. In most cases, the prime reason for street sweeping is for aesthetics and urban housekeeping, rather than water quality benefits. The dominant influence on effectiveness of street sweeping appears to be the frequency of sweeping and the interval between storms. Semiannual street sweeping programs are recommended to remove debris after spring snowmelt and after leaves fall in the autumn.

5. De-icing Chemical Use and Storage. A tremendous amount of de-icing chemicals are used each winter on roads, parking lots, and sidewalks. Sodium chloride (salt) is the main chemical used. Proper use and storage of salt will reduce the chance of high chloride concentration in runoff that may damage the environment. In order to prevent chloride from entering surface or groundwater, the following practices should be used at stockpile locations:

   - All piles should be covered with polyethylene if not stored in a shed. All sand/salt piles should be moved to empty salt sheds or covered during the spring and summer.

   - Any runoff from stockpiles should be contained.

10.2.3 Site Controls

Site controls are generally those controls that attempt to reduce runoff rate and volume at or near the point where the rainfall hits the ground surface. The following types of site controls are common:

1. Minimization of Directly Connected Impervious Area. Directly connected impervious area (DCIA) is defined as the impermeable area that drains directly to the improved drainage system (i.e., paved gutter, improved ditch, or pipe). The minimization of DCIA is by far the most effective method of runoff quality control because it delays the concentration of flows into the improved drainage system, and maximizes the opportunity for rainfall to infiltrate at or near the point at which it strikes the ground. It should be kept in mind that the aim of this practice is minimization of the runoff peak and volume for small storms.

2. Swales and Filter Strips. Swales, or grassed waterways, and filter strips are among the oldest stormwater control measures and have been used along side streets and highways for many years. A swale is a shallow channel which has the following characteristics:

    a. Side slopes are flatter than 3:1.
    b. Contains contiguous areas of standing or flowing water only following rainfall.
    c. It is planted with and contains vegetation suitable for soil stabilization, stormwater treatment, and nutrient update. The filter strip is simply a strip of land across which stormwater from a street, parking lot, rooftop, etc., flows before entering adjacent receiving waters.

    For small storms, both swales and filter strips remove pollutants from stormwater by slowing the water and settling or filtering out solids as the water travels over the grassed area, and by allowing infiltration into the underlying soil. In general, the higher the flow rate, the lower the efficiency. Thus, low velocity and shallow depth are key design criteria. A swale designed with a low bottom slope and check dams will perform much more efficiently than one without check dams. Raised driveway culverts are very effective as check dams. For maximum efficiency of pollutant removal during small storms, a trapezoidal swale with as large a bottom width as can be fitted into the site plan is desirable, since this will maximize the amount of runoff and contact with the vegetation and soil.

    Design flows are calculated using equations for open channel flow, and for small frequent runoff events. For filter strips, there are two primary design considerations. The first is to minimize the grade and the direction of flow (i.e., terracing). The second is to make certain that the water flowing across the strip is introduced at the upstream side of the strip in such a way that it flows across the strip in sheet flow and does not channelize.

3. <u>Porous Pavement</u>. Porous pavements have excellent potential for use on streets and parking areas. When properly designed and carefully installed and maintained, porous pavements can have load bearing strength and longevity similar to conventional pavements. In addition, a porous pavement can help reduce the amount of land needed for stormwater management to preserve the natural water balance at a given site, and to provide a safer driving surface that offers better skid resistance and reduced hydroplaning. A porous pavement is only feasible on sites with permeable soils, fairly flat slopes, and relatively deep water table levels. Where winter conditions are, severe, additional considerations should be given to the structural integrity of a porous pavement under the freeze-thaw conditions.

4. <u>Infiltration Devices</u>. Infiltration devices are those stormwater quality control measures that completely capture runoff from the water quality design storm and allow it to infiltrate into the ground. They are the most effective stormwater quality control device that can be implemented, but they can only be used in situations where the captured volume of water can infiltrate into the ground before the next storm, and where they will not cause structural or groundwater pollution problems. Infiltration devices can be either above ground infiltration basins, or buried infiltration trenches. Among their advantages are that they can help to minimize alterations to the natural water balance of a site, can be integrated into a site's landscaped and open areas, and, if carefully designed, can serve larger developments.

   Infiltration basins function to temporarily store stormwater until it infiltrates the surrounding ground through the bottom and sides of the basin. They can be constructed on-line or off-line with respect to the normal drainage path. When a basin is constructed on-line, it is designed to capture the water quality design storm entirely. When a larger storm occurs, it overflows the basin, which then serves as a detention pond for those larger events. Off-line infiltration basins are designed to divert the more polluted first flush of stormwater out of the normal path and hold it for later water quality treatment. Off-line infiltration devices are designed to store a selected volume of stormwater for a specified period of time with a predetermined infiltration rate.

   In general, site planning considerations for infiltration trenches are the same as for infiltration basins. Trench bottoms should be at least four feet above the seasonal high water table level to prevent mounding of groundwater, consequent loss of infiltration capacity, and insect problems. Trenches should be inspected frequently within the first few months of operation and regularly thereafter. Such inspections should be done after large storms to check for ponding.

10.2.4 Detention Controls

10.2.4.1 Dry Pond Detention.

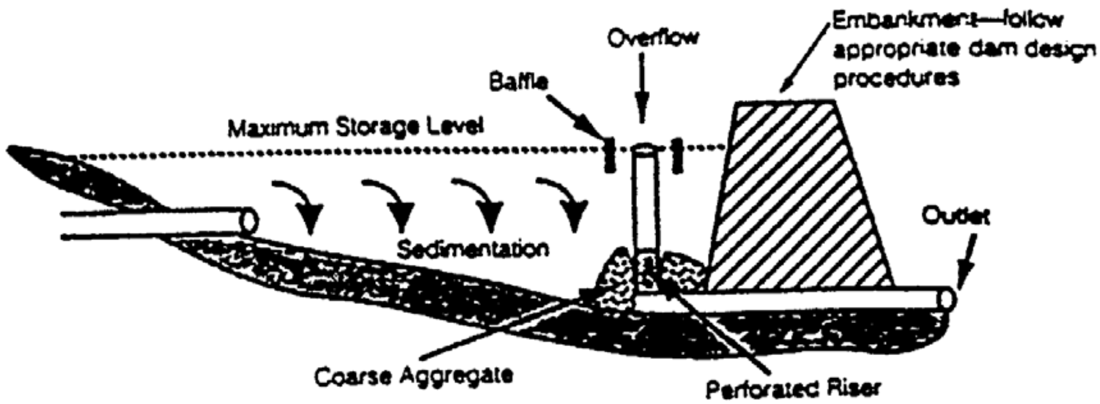
Detention basins designed for flood peak reduction only, without any provision for a permanent pool, is the most common type of detention basin used. These basins generally are not as effective in removing pollutants as wet basins, to be described later. This is primarily due to the short hydraulic residence time. For basins with detention times of less than 12 hours, no more than 10% of stormwater pollutants are removed, and in fact, there may be even a negative pollutant removal rate due to the washout of pollutants captured in earlier small storms. Figure 10.2 presents a schematic of a typical dry detention pond and summarizes typical performance characteristics.

Performance can be improved, from the standpoint of water quality control, if certain modifications are made to the design which differs from the typical dry detention basin. For example, the use of swales (as pretreatment conveyances), and land infiltration will greatly improve the pollutant removal effectiveness and life of a dry detention system. Another possibility is to incorporate a shallow marsh around the outlet structure, as shown in Figure No. 10.3. It should be noted, however, that the addition of a marsh makes this control device no longer a dry detention basin but rather a transitional device between a dry and wet detention system (i.e., extended detention pond system).

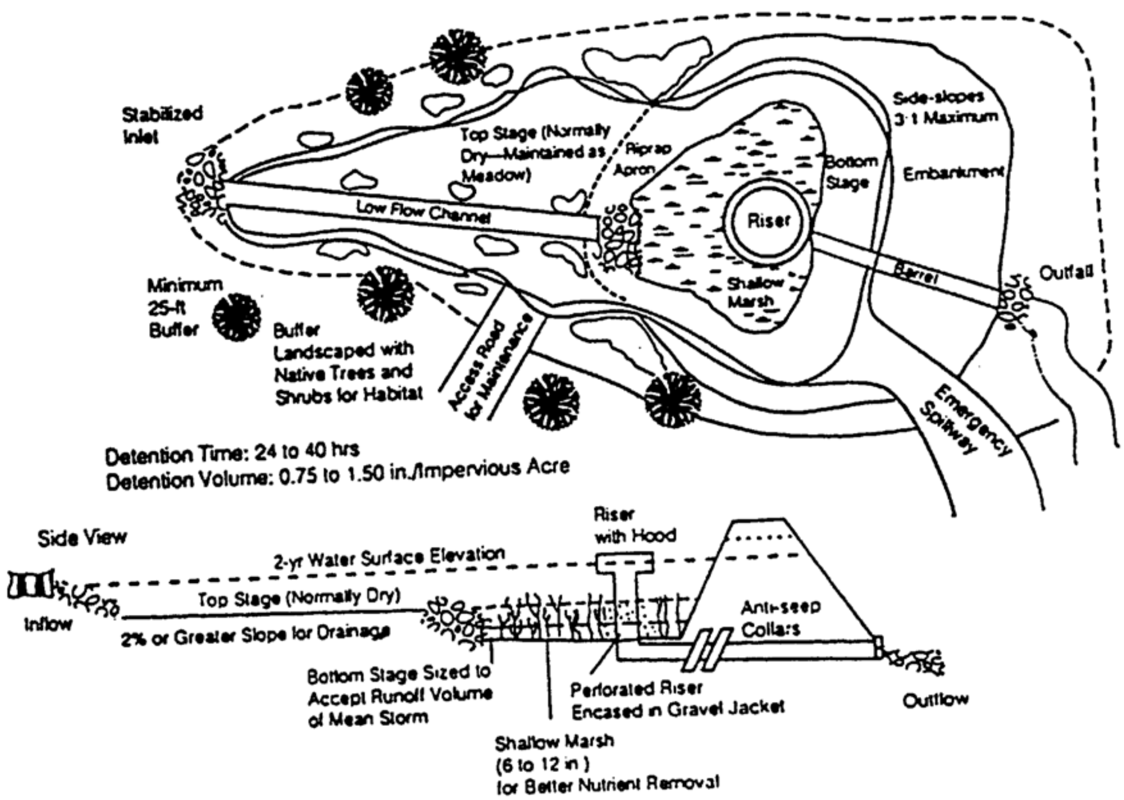
10.2.4.2 Wet Pond Detention.

The wet pond detention system consists of a permanent water pool, an overlying zone in which design runoff volume temporarily increases the depth of the pool while it is stored and released at the allowed peak discharge rate, and a shallow littoral zone (the biological filter). Figure No. 10.4 illustrates the basic components of a wet pond detention system that is used for both flood control and water quality enhancement. Figure No. 10.5 shows an example of a wet detention pond sized for a two to four week residence time. During storms, runoff replaces treated waters detained within the permanent pool after the previous storm, thus making the permanent water pool volume and the vegetated littoral zone of utmost importance for water quality enhancement. Flood detention ponds are often used in series with swale inter-connectors. If properly designed and maintained, they can provide effective flood and water quality protection, as well as an aesthetically pleasing wildlife habitat.

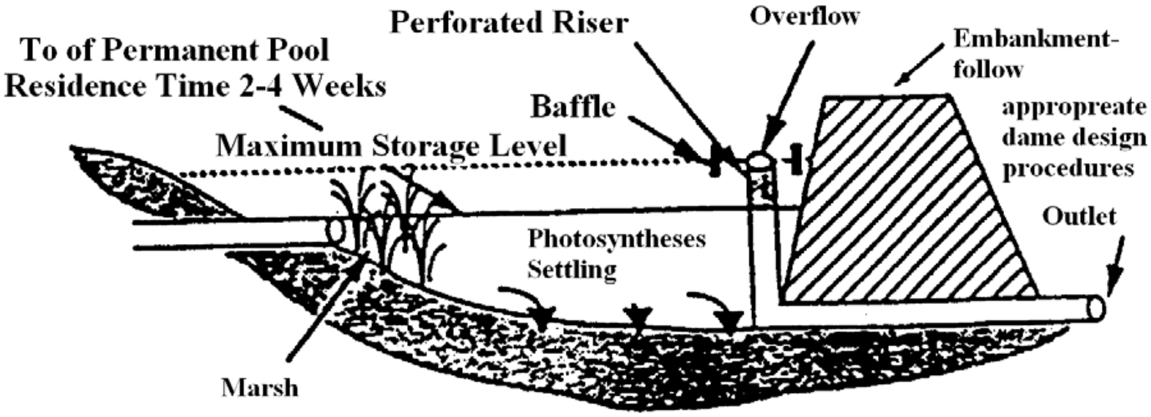
The removal of stormwater pollutants in a wet detention pond system is accomplished by a number of physical, chemical, and biological processes. Gravity settling is used to remove the heavier particles. Chemical flocculation occurs when heavier sediment particles overtake and coalesce with smaller, lighter particles to form larger particles. Biological removal of dissolved stormwater pollutants includes uptake by aquatic plants and metabolism by phytoplankton, and microorganisms that inhabit the bottom sediments. Removal of dissolved pollutants primarily occurs during the relatively long quiescent period between storms. Accordingly, the permanent water pool is especially vital, since it permits treatment between storms, reduces runoff energy, and provides a habitat for aquatic plants and algae. Aerobic conditions at the bottom of the permanent pool will maximize the uptake of dissolved pollutants (nutrients, metals) by bottom sediments, and will minimize release from the sediments into the water.



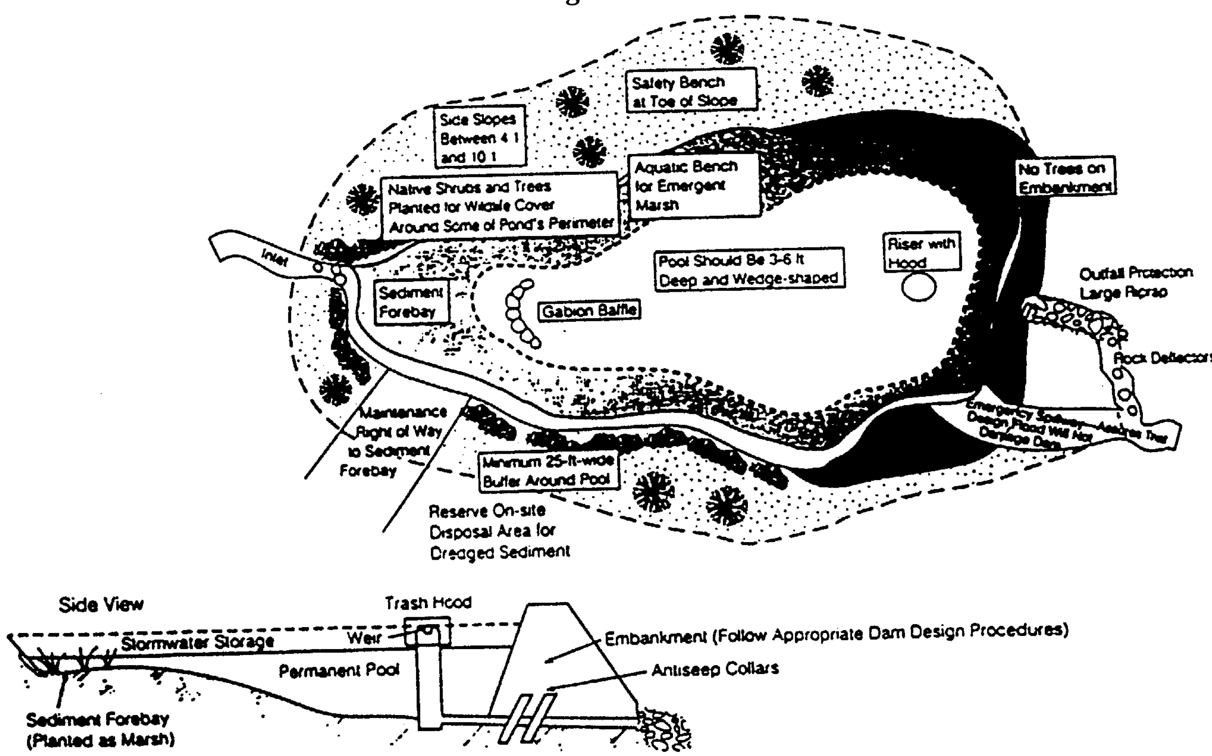
Dry Detention Pond
Figure 10.2



Extended Detention Pond
Figure 10.3



Wet Detention Pond
Figure 10.4



Typical Design of Wet Detention Pond
Figure 10.5

There are three critical criteria in determining how efficiently the wet detention pond system will work. The first critical criterion is the volume of the permanent pool, which should be sufficient to provide two to four weeks of detention time so that algae can grow. The second key criterion is the depth of the permanent pool, which should be greater than four to six feet, but less than 10 to 15 feet, so that the water remains wind-mixed and the bottom sediment stays aerobic. If the bottom sediment turns anaerobic, it will release nutrients into the overlying pool, and these nutrients will be washed out in the next rainstorm. The third critical criterion involves the shallow littoral zone, typically concentrated around inflow points and the outflow. This is one of the very important components of the system, since the aquatic plants within this zone provide the biological assimilation of dissolved stormwater pollutants. The littoral zone should cover at least 30% of the pond's surface area and should have a gentle slope (6:1 or shallower) to a depth of two feet below the control elevation. Another benefit of the flat side slopes is enhanced public safety, especially for children.

Wet ponds should not be considered in areas where the underlying soil infiltration rate will not allow water to remain in the pond on a permanent basis or where the rainfall is not sufficient to sustain the necessary volume in the pool. The use of several detention ponds in series, or the separation of the pond into multiple cells, will enhance pollutant removal and lessen maintenance tasks.

10.2.4.3 Wetland Detention

Wetland treatment removal of water pollutants involves passing runoff through a natural wetland or a constructed wetland to remove or treat the pollutants. Wetlands provide favorable conditions for removal of pollutants from urban runoff through sedimentation and also provide an intense pool of biological activity to use nutrients during the growing season. Wetlands can provide water quality enhancement through sedimentation, filtration, absorption, and biological processes as well as natural flood protection.

Wetlands can be very effective for trapping urban pollutants; however, urban runoff can overload and degrade a natural wetland. The solids that are trapped from urban runoff are deposited in the wetland where they may be difficult to remove at a later date. Given many unknowns still associated with this treatment practice, only relatively isolated wetlands or constructed wetlands should be used for stormwater management. Care should be taken to minimize changes in the hydro-period (the time that water remains at a particular level in the wetland - which determines the form, function, and nature of the wetland) caused by the addition of stormwater. Finally, pretreatment practices are needed to reduce oil, grease, and sediment levels to protect the wetland.

10.3 IMPLEMENTATION OF STORMWATER QUALITY CONTROLS

Integrating urban runoff quality controls into a stormwater management plan requires creativity. In the ponding phase, the design engineer must constantly have in mind the four objectives of stormwater management. These are: 1) surface drainage; 2) flood control; 3) erosion and sediment control; and 4) control of pollutants in the runoff.

The approach to stormwater quality control is to minimize the adverse impacts of stormwater through a coordinated system of source controls and site controls. The stormwater management system can be viewed as a treatment train in which the individual treatment practices are viewed as cars. The first car is source controls, followed by site controls. Grassed swales, detention ponds, wetlands, and other devices add to the train. The more controls incorporated into the system, the better the performance of the train.

A good stormwater quality management system will provide acceptable results if the following concepts are kept in mind:

1. Design runoff quality controls to capture small storms.

2. Design to maximize sediment removal, and removal of other pollutants.

3. The most effective method for reducing urban runoff pollution is to minimize directly connected impervious area (DCI).

4. Infiltration devices are the most efficient, but also the most difficult, to maintain.

5. Dry detention is the easiest to design and operate, and efficiency is satisfactory if properly designed.

6. Wet detention is more difficult to design and maintain but more efficient than dry detention, and often more aesthetically desirable.

CHAPTER 11
EROSION AND SEDIMENTATION CONTROL

11.1 INTRODUCTION

Each year excessive quantities of soil erode from certain areas that are undergoing development for non-agricultural uses such as housing developments, industrial areas, recreational facilities and roadways. In addition, soils and debris are being deposited on streets from excavation and construction sites. The resulting sediment clogs storm sewers and road ditches, causes dust and mud on streets and leaves deposits of silt in creeks and streams, which is considered a major water pollutant.

The purpose of this chapter is to develop guidelines to minimize soil erosion and sedimentation from occurring on urban developing land by requiring proper provisions for surface and subsurface water disposal and the protection of soil surfaces during and after construction in order to promote the safety, public health, convenience, and general welfare of the community.

These guidelines are not necessarily intended to replace local ordinances or state and federal requirements, but are intended to help developers, private consultants and others who will be developing urban lands. They should be useful in planning for soil and water conservation as an integral part of development plans, to minimize erosion and sedimentation problems on land undergoing urban development.

11.2 DESIGN CRITERIA

11.2.1 General

Design criteria for erosion and sediment control measures and materials should be in accordance with, but not limited to, the following:

1. National Pollutant Discharge Elimination System (NPDES) of the Environmental Protection Agency and as administered by the North Dakota State Health Department.

2. North Dakota Department of Transportation Standard Specifications for Road and Bridge Construction (current edition).

3. The American Association of State Highways and Transportation Officials (AASHTO) "A Guide for Highway Landscape and Environmental Design" (current edition).

4. Other recognized design manuals for erosion and sedimentation control (i.e., Natural Resource Conservation Service, publications, etc.).

5. Conflict - In case of a conflict between the above design standards, the City Engineer should be contacted for clarification.

11.2.2 <u>Definitions</u>

1. Accelerated Soil Erosion. The increased loss of land and surface (soil) occurring as the result of a land distributing activity.

2. Non-Erosive Velocity. The velocity of water movement that will not cause accelerated soil erosion.

3. Stabilization. Erosion control placement or covering of soil to insure its resistance to soil erosion, sliding or other earth movement.

4. Permanent Soil Erosion Control. Those control measures which are installed or constructed to control erosion on a permanent basis and which are maintained on a continuous basis after completion of a project.

5. Temporary Soil Erosion Control. Interim control measures which are installed or constructed for the temporary control of soil erosion until the permanent soil erosion control are established. Examples are stabilized crop seeding, sediment traps, silt dikes, moistening of soils, etc.

6. Sediment Traps and Basins

   a. A sediment trap ponding area is generally constructed of earth to provide an embankment across a drainage swale. The maintenance period of a sediment trap is based on when the exposed area has been stabilized.

   b. A sediment basin is a longer term temporary or permanent ponding area with a controlled release structure. A series of small sediment traps may be substituted for a sediment basin upon the approval of the City Engineer.

11.2.3 <u>Requirements</u>

1. Any land disturbing activity shall be conducted in such a manner so as too effectively reduce accelerated soil erosion and resulting sedimentation.

   Before these activities commence, including site grubbing, erosion controls should be installed on the downhill portion of the site or where erosion is a potential problem.

2. Sediment traps, perimeter dikes, sediment barriers (such as straw bale barriers and silt

fencing), and other measures intended to trap sediment within the site, drainage outlets to the major stormwater system or retention facilities must be constructed as a first step in grading and be made functional before upslope land disturbance takes place. Earthen structures such as dams, dikes, and diversions must be mulched within 14 calendar days of installation. Earthen structures that will remain in place for a period exceeding one year must be seeded and mulched. Construction of earthen structures should be scheduled so as to allow prompt seeding during appropriate seasons.

3. Concentrated stormwater runoff from disturbed drainage areas with ten acres of total tributary area or greater should pass through a temporary sediment basin providing 3,600 cubic feet of storage per acre drained. If 3,600 cubic feet per area drained is not attainable, combinations of silt fences, multiple sediment traps, or equivalent sediment controls are required for all side slopes and downslope boundaries of the construction area. For 10 or fewer acres disturbed, the same controls are required except for the sediment basin, which is optional.

   Velocity dissipation devices shall be used to provide a non-erosive velocity flow to maintain existing hydraulic conditions of water courses.

   Sediment traps will require periodic maintenance including sediment removal whenever one-half of the sediment storage volume is filled.

   The site must be inspected by a qualified person (provided by the discharger) once every seven calendar days and within 24 hours after a rainfall of 0.5" or greater. Reports summarizing the inspections shall be made and retained as part of the storm water pollution prevention plan until project termination.

4. All land disturbing activities should be designed, constructed, and completed in such a manner that the exposure time of disturbed land should be limited to the shortest possible period of time. Once grading operations are completed in areas of work stoppages for a period of 21 or more calendar days, the stabilization must be initiated by the 14th calendar day. In extremely environmentally sensitive areas, a 72 hour stabilization period is to be used.

5. Sediment caused by accelerated soil erosion shall be removed within 48 hours after the heavy precipitation, snowmelt or strong winds which caused it.

6. Any temporary or permanent facility designed and constructed for the conveyance of water around, through, or from the land disturbing activity shall be designed to limit the water flow to a non-erosive velocity.

7. Temporary soil erosion control facilities shall be removed and areas of land

disturbance graded and stabilized with permanent soil erosion control measures pursuant to approved plans and specifications.

8. Erosion and maintenance shall be minimized by use of acceptable side slopes, rounded and blended with natural terrain; serrated cut slopes; drainage channels designed with due regard to width, depth, slopes, alignment, and protective treatment; inlets located and spaced with erosion control in mind; prevention of erosion at culvert outlets; proper facilities for ground water interception; dikes, berms, and other protective devices; sedimentation devices to trap sediment at strategic locations; and protective ground covers and planting.

9. All temporary and permanent erosion sedimentation control practices must be maintained by the Owner. Silt fences will require periodic replacement. Sediment basins will require periodic sediment removal. All facilities must be inspected following heavy precipitation or snow melt.

10. Maximum suggested allowable rates of soil loss are as follows:

    a. Agricultural and Horticultural Lands - 1 to 5 tons/acre/year depending on soils.
    b. Non-agricultural Lands - 5 tons/acre/year.
    c. Construction Sites - 5 tons/acre/year.

    To predict soil loss the "United States Department of Agricultural Handbook Number 537" provides one method. The procedure is founded on an empirical soil loss equation called the Universal Soil Loss Equation (USLE). The equation is believed to be applicable wherever numerical values of its factors are available. Tables and charts as presented in the Handbook Number 537 are to be used in the equation.

    The Universal Soil Loss Equation (USLE) is an erosion model designed to predict the longtime average soil losses due to runoff from specific field areas. Widespread field use has substantiated its usefulness and validity for this purpose. It is also applicable for such non-agricultural conditions as construction sites.

    With appropriate selection of its factor values, the equation will compute the average soil loss for a year. It computes the soil loss for a given site as the product of six major factors whose most likely values at a particular location can be expressed numerically. Erosion variables reflected by these factors vary considerably about their means from storm to storm, but effects of the random fluctuations tend to average out over extended periods. Because of the unpredictable short-time fluctuations in the levels of influential variable, the soil loss equation is substantially less accurate for prediction of specific events than for prediction of longtime averages.

    The soil equation is:

$$A = R \times K \times (LS) \times C \times P \qquad \text{(Equation 11.1)}$$

where

A	=	is the computed soil loss per unit area, expressed in the units selected for K and for the period selected for R. In practice, these are usually so selected that they compute A in tons per acre per year, but other units can be selected.
R	=	the rainfall and runoff factor, is the number of rainfall erosion index units, plus a factor for runoff from snowmelt or applied water where such runoff is a significant.
K	=	the soil erodability factor, is the soil loss rate per erosion index unit for a specified soil.
LS	=	the slope-length factor and slope-steepness factor is the ratio of soil loss from the field slope and length to that from a 72.6 ft. length, and a 9 percent slope, under identical conditions.
C	=	the cover and management factor, is the ratio of soil loss from an area with specified cover and management to that from an identical area in open cut.
P	=	the support practice factor, is the ratio of soil loss with a support practice. This value is usually equal to one for construction or development areas.

11.3 DESIGN REQUIREMENTS

11.3.1 Temporary Erosion and Sedimentation Control

1. Current criteria for temporary erosion and sedimentation control apply to all projects which require temporary measures to control soil erosion. By specification the contractor should be required to construct and maintain all erosion and sediment devices shown on the plans and/or deemed necessary by the design engineer to effectively control pollution of waterways and sedimentation adjacent to property.

2. During project design, temporary erosion and sedimentation control measures shall be

considered and reviewed as to location and type. Temporary erosion and sedimentation control measures and intended uses include but are not limited to the following:

a. Sediment Trap, Silt Dikes and Ditches:

(1) Set permanent inlet elevation of drainage structures and provide silt basins below the permanent elevation. Place sediment traps ahead of drainage structures approximately 200 feet spacing for ditch grades 1% to 2% and the vertical height of the foreslope is 5 feet or less. The maximum drainage area for a sediment trap is 2.5 acres at a storage volume of 3,600 cubic feet per acre of disturbed drainage area. Multiple sediment traps may be used to store sediment for drainage areas larger than 2.5 acres. As an example, a site drains 7.5 acres which would require three sediment traps at a total storage volume of 27,000 cubic feet. One single sediment basin can be used for the same volume in this example. Normally, a sediment basin should be used when the disturbed drainage area equals or exceeds 10 acres.

Concentrated stormwater runoff leaving a development site must be routed through sediment traps and discharged directly into a public stormwater system, wherever practical.

Temporary channels or pipes leaving the site should be capable of conveying the runoff from a 10-year storm without overtopping its banks, surcharging or eroding during development of the site in question. Permanent channels or retention facilities must be complete-in-place prior to completion of the site development. Temporary channels should not be in place for a period exceeding two years prior to the construction of a permanent facility.

(2) Silt dikes may be required when the vertical height of a construction foreslope is five feet or less and the existing ground slopes away from the foreslope in rural areas. A silt fence shall be incorporated at the outlet end of the silt dike. For urban areas, with slopes at 2% or greater, a temporary silt dike should be placed parallel to the street to intercept flows for drainage areas 10 acres or greater.

(3) Silt ditches shall be required when the drainage from the

adjacent ground flows onto the roadway right-of-way. Generally, the silt ditch is placed at the right-of-way line.

b. Silt Fence: Used when vertical height in cut or fill areas is over five feet or where five acres and greater of concentrated or sheet flows with slopes at 2% or greater drain to the roadway right-of-way.

(1) Place a silt fence near the right-of-way line or a minimum of 10 feet from the back of curb or toe of the foreslope.

(2) Locate diagonal and individual silt fences near right-of-way line in approximately 200 foot segments with lower end skewed toward the roadway to intercept small concentrations of diagonal sheet flow. The silt fence should be perpendicular to the flow.

(3) Place silt fence around all inlets in conjunction with the construction of the inlet.

(4) Other locations of silt fences may be necessary for the control of erosion and may be required by the City Engineer.

c. Silt Fence as Ditch Checks: Construct at right angles to flow and intercept slope area where possible.

Ditch Grade	Approximate Spacing
1% to 2%	300 ft.
2% to 3%	200 ft.
3% to 4%	100 ft.
4% and greater	temporary sediment basins

d. Others: Other temporary controls may include, seeding, mulching, sodding, straw bale dikes, diversions and other innovative practices or modifications to specified practices may be used if approved by the City Engineer.

11.3.2 Permanent Erosion and Sediment Control

1. Current specifications for permanent erosion and sedimentation control apply to all projects. A permanent vegetative cover should be established before the end of the construction season or temporary erosion control is to be maintained throughout the winter months until spring planting is established. Vegetation shall not be considered established until ground cover is achieved, and is mature enough to control soil erosion satisfactorily and to survive severe weather conditions. All temporary erosion and sedimentation control measures shall be disposed of within 30 days after final site stabilization is achieved.

2. During project design, permanent erosion and sedimentation control measures shall be reviewed as to location and type. Permanent erosion and sedimentation control measures and intended uses include, but are not limited to the following:

   a. Seeding

   One form of permanent erosion and sedimentation control is seeding fertilizing and mulching. Seed shall be sown only at times of the year when temperature, moisture, and climatic conditions will promote germination and plant growth. Normal permanent seeding is established within at least 45 days. If seeding is not established in this period, the exposed areas shall be reseeded.

   b. Sodding

   Sodding and fertilizing may be used for erosion and sedimentation control such as for a buffer strip adjacent to adjoining property. Sod may also be placed in ditch bottoms and in areas where vegetation is needed quickly. Sodding must be staked in slopes greater than 10%. The maximum allowable sodded slope is 6:1 (17%).

   c. Riprap

   A layer of various sized rocks (called riprap) is a popular method of controlling erosion. The rough, angular surfaces and the variety of sizes help the rocks fit tightly together to form a dense barrier. Reference is made to Chapter 7 of this manual for a detailed discussion of rock riprap design requirements.

d. Gabions

Gabions (rock-filled wire baskets or mattresses) can be used in situations where the stream banks are too steep for rip-rap or for specific control limits. They are particularly effective for protecting the submerged part of the bank. The layer of rock-filled baskets will continue to provide protection while adjusting itself to shifting in the stream bed. Reference is made to Chapter 7 of this manual for a detailed discussion of gabion design requirements.

e. Fabric Blanket

A fabric blanket is a flexible, mesh-like material through which vegetation can grow. It is usually about 1/2-inch thick, and consists of tightly coiled and intertwined fibers of either nylon or excelsior. In the beginning, the fabric alone protects the bank. As plants grow, their roots intertwine with the fabric making an even stronger barrier.

A fabric blanket is normally used when vegetation alone can't provide enough protection, but the erosion isn't serious enough to justify more expensive control measures. Since it needs vegetation to be effective, it works best on the upper bank - above the level that is normally submerged. It also works well on streams which are dry for part of the year. The blanket system has been used on slopes as steep as 1:1, but normal use is 2:1 and less.

The manufacturer of the fabric blanket will provide instructions on how to install it and limits of use. Sometimes it is buried in the face of the bank a few inches. It can also be spread on top of the soil, or used as a cover to hold down mulch.

No matter how it is used, the fabric must be securely anchored in position. The sections of fabric should be overlapped three to six inches, and staked to the bank. The ends of the fabric are usually buried in 12-inch deep trenches.

f. Other

Other permanent erosion prevention measures may include, erosion control mats, wood excelsior, jetties and other innovative practices or modifications to specified practices may be used if approved by the City Engineer.

11.4 STORMWATER DISCHARGE PERMIT REQUIREMENTS

11.4.1 General

In October 1990, the Environmental Protection Agency (EPA) approved the Final Stormwater Rule under the National Pollutant Discharge Elimination System (NPDES). Under this rule, qualified projects are required to have stormwater discharge permits. The EPA's original stormwater regulations require that construction activities that disturb five or more acres of land after October 1, 1992, be authorized by NPDES Permit, administered through the North Dakota Department of Health and Consolidated Laboratories. The North Dakota Department of Health defines disturbed soil as any soil that is exposed to erosive forces (i.e., wind or water). Anyone conducting a construction activity that disturbs five or more acres of total land area is required to apply for coverage under the NPDES Construction Stormwater Permit. Construction activity includes the following:

- Clearing
- Grading
- Excavation
- Road building
- Construction of:
  - Residential houses
  - Office buildings
  - Commercial facilities
  - Industrial buildings
  - Landfills
  - Airports
  - Feedlots

11.4.2 Basic Requirements

Stormwater NPDES Permits require that an erosion and sedimentation control plan be developed for the project. The plan is in turn composed of temporary erosion and sediment control plan and a permanent erosion and sediment control plan. These erosion control plans must contain Best Management Practices (BMPs) developed to meet the goals of each plan. An excellent reference for developing erosion and sedimentation control plans is "Stormwater Management for Construction Activities: Developing Pollution Prevention Plans and Best Management Practices" prepared by the EPA and distributed by the National Technical Information Service (telephone 703-487-4650). This manual was developed to assist NPDES applicants with developing temporary and permanent erosion and sedimentation control plans.

In North Dakota, applications for an NPDES Stormwater Discharge Permit are initiated by completion of "Notice of Intent to Obtain Coverage under NPDES General Permit for Stormwater Discharges Associated with Industrial Activity". This form must be completed and submitted to: North Dakota Department of Health and Consolidated Laboratories, Division of Water Quality, PO Box 5520, Bismarck, ND 58502-5520 (telephone 701-221-5210).

11.5 EROSION AND SEDIMENTATION CONTROL PLAN

11.5.1 Temporary Erosion and Sedimentation Control Plan

11.5.1.1 Goal

The goal of the Temporary Erosion and Sedimentation Control Plan is to prevent sediment from entering waters of the state during construction. The owner shall incorporate Best Management Practices (BMPs) into the project's final plans and specifications, which are designed to meet this goal. It is the owner's responsibility to select the appropriate BMPs which satisfy these requirements.

11.5.1.2 Assigning Responsibility

When developing bidding documents or other contracts, the owner must identify who will implement and manage the erosion and sedimentation control BMPs before and during construction; and ensure that the plan will be implemented and stay in effect until the construction project is complete, and the entire site has undergone final stabilization. In addition, the final plans and specifications must clearly identify who will be responsible for the maintenance requirements.

11.5.1.3 Plan Contents

The Temporary Erosion and Sedimentation Control Plan, if developed as a document separate from the project's final plans and specifications, must be prepared for the proposed project. The plan must contain appropriate BMPs and contain standard plates and/or specifications of these BMPs.

1. Standard plates and/or specifications must be provided for all BMPs selected by the designer to be used on the project, and at a minimum, must include the following:
    a. perimeter sediment control
    b. placement and type of temporary cover.

2. Where applicable, standard plates and/or specifications must also be provided for the following:
    a. horizontal slope grading
    b. proposed stabilized vehicle entrances
    c. temporary sedimentation basins
    d. storm sewer pipe outlet energy dissipation
    e. storm sewer inlet control
    f. erosion and sedimentation control requirements for stockpile areas.

11.5.1.4 Final Plans and Specifications

The above standard plates and/or specifications are to be incorporated into the project's final plans and specifications. In addition, the final plans and specifications shall clearly denote:

1. Location and type or the procedures to establish the location and type of all erosion and sedimentation control BMPs.

2. Existing and final grades, including dividing lines and direction of flow for all stormwater runoff drainage areas located within the project limits.

3. Locations of areas not to be disturbed or areas where construction will be staged to minimize duration of exposed soil areas.

4. All waters of the state, including existing wetlands identified on the National Wetlands Inventory Map, within one-half mile from the exposed construction area which will receive direct stormwater runoff from the construction site during construction.

   Where waters of the state, including wetlands, which will receive the direct runoff, will not fit on a plan sheet, they shall be identified with an arrow, indicating both direction and distance.

5. Timing for installation of all erosion and sedimentation control BMPs.

11.5.1.5 Plan Retention

The owner shall keep a copy of the Temporary Erosion and Sedimentation Control Plan and all changes to it for three years after completion of the construction project.

11.5.1.6 Changes to the Temporary Erosion and Sedimentation Control Plan

Changes in the plan made during construction to accommodate phased construction, sequenced work, timing issues, or changed site conditions are allowable.

11.5.2 Permanent Erosion and Sedimentation Control Plan

11.5.2.1 Goal

The goal of the Permanent Erosion and Sedimentation Control Plan is to protect North Dakota water resources from pollutants generated from a project's ultimate development's impervious surfaces, change in land use, or changed ground cover.

11.5.2.2 <u>Assigning Responsibility</u>

When developing bidding documents or other contract, the owner must identify who will maintain the water quality management BMPs until construction is complete, all maintenance activities are complete and the site has undergone final stabilization.

11.5.2.3 <u>Plan Contents</u>

The Permanent Erosion and Sedimentation Control Plan must be prepared for the proposed project, and may be developed as a separate document from the final plans and specifications. The play must contain appropriate BMPs which satisfy the above goal, and contain standard plates and/or specifications of these BMPs. These standard plates and specifications must be incorporated into the project's final plans and specifications. At a minimum, the plan must contain:

1. Land feature changes (in acres) for both before and after construction:

    a. Total project area;
    b. Total impervious surface area of the project;
    c. Total impervious area of the project;
    d. Total estimated impervious surface area of ultimate development;
    e. Total estimated pervious area of ultimate development;

2. Standard plates and/or specifications of permanent erosions and sediment control BMPs below:

    a. <u>Sediment Control</u>

       Where a project's ultimate development replaces surface vegetation with one or more acres of cumulative impervious surface and all runoff has not been accounted for in a local unit of government's existing storm water management plan or practice, the runoff shall be discharged to a sedimentation basin prior to entering waters of the state.

        1) Proposed Development

           Except as provided in 2) below ("Reconstruction or Work on Existing Roadways"), the wet sedimentation basin shall be based on the project's ultimate development and comply with the following requirements:

            a) The basin's hydraulic volume shall be efficient to capture a 1/2 inch of runoff from the impervious watershed area.

b) Basins shall also provide a minimum of 250 $ft.^3$ dead sediment storage volume below the basin's hydraulic volume/impervious acre drained.

c) Basin inlets shall be placed above the sediment storage volume.

d) Basin outlets shall be designed to remove all suspended solids greater than five microns with a settling velocity of 1.3×10^{-4} ft/sec.

e) Basin outlets shall also be designed to prevent short circuiting and the discharge of floating debris.

f) Basins must provide spillways to accommodate storm events in excess of the basin's hydraulic design.

2) Reconstruction or Work on Existing Roadways

While recommended, the above provision for a sedimentation basin will not be required for work on existing roadways where:

1) the drainage area is served by an existing storm sewer which is day lighted off the road's right-of-way or,

2) proximity to bedrock or vertical relief precludes it, or,

3) existing right-of-way precludes it.

For these situations, however, the owner will be required to incorporate other sedimentation or treatment devices (i.e., grass swales, smaller sediment basins, etc.).

b. <u>Permanent Erosion Control</u>

1) All drainage ditches construction to drain water from the site after construction is complete must be stabilized.

2) All pipe outlets must be provided with permanent energy dissipation where the pipe's outlet velocity will exceed the permanent cover's erosive velocity.

c. Treatment

The owner is required to provide treatment of storm water through the use of BMPs such as grass swales, wetlands constructed for the purpose of treating storm water, and the planting or development of emergent vegetation around the perimeter of the wet sedimentation basin's sediment storage volume.

11.5.2.4 FINAL PLANS AND SPECIFICATIONS

The above standard plates and/or specifications are to be incorporated into the project's final plans and specifications. In addition, the final plans and specifications shall clearly denote:

1. Location and type of all permanent erosion and sediment control BMPs.

2. The plan sheets must clearly identify all waters of the state, including wetlands identified on the National Wetlands Inventory Map within the one-half mile from the construction area which will receive direct storm water runoff from the construction site after construction is complete.

   Where the water of the state which will receive the direct runoff and will not fit on the plan sheet, the resource shall be identified with an arrow, indicating both direction and distance.

3. Methods to be used for final stabilization of all exposed soil areas. For linear utility and roadway projects, final stabilization is not required on agricultural land which will be tilled within one year of project completion.

11.5.2.5 PLAN RETENTION

The owner shall keep a copy of the Permanent Erosion and Sedimentation Control Plan and all changes to it for three years after completion of the construction project.

11.5.2.6 CHANGES TO THE PERMANENT EROSION AND SEDIMENTATION CONTROL PLAN

Changes in the plan made during construction to accommodate changed site conditions are allowable.

CHAPTER 12
STORMWATER/DRAINAGE EASEMENTS

12.1 GENERAL REQUIREMENTS

1. All easements must be mutually exclusive. Easements shall be identified as public utility and drainage easements.

2. All easements for installation of storm sewer pipe should be at least 30 feet wide. The pipe shall be placed only on one side of a joint property line to allow for placement of excavated earth if maintenance is required. Storm sewers placed in easements shall be located 7.5 feet from the easement line. In situations where the design engineer can clearly demonstrate that an easement less than 30 feet is adequate (considering depth of storm sewer, stockpile area and maintenance access), the City Engineer may consider such a request.

3. No landscaping except grass may be placed in the easement.

4. No permanent structure may be placed in the easement.

5. Fencing will not be allowed in Surface Water Drainage easements.

6. The easement agreement must state that any temporary structures (including paving and fencing) placed in the easement will be removed by the owner of the land when requested by the City Engineer so that maintenance can be performed. The owner of the land must agree to hold the City harmless for the reinstallation or replacement of structures removed from the easement.

7. Storm sewer pipe installation in easements between single-family residential lots may be approved at the discretion of the City Engineer to drain a cul-de-sac or other isolated areas which might otherwise not be drained.

12.2 EASEMENT FOR STORM SEWER

The following easement form may be used where a storm sewer is to be constructed but no surface drainage way is required to carry stormwater flows in excess of the capacity of the storm sewer. The width must accommodate maintenance equipment and excavated earth to repair the storm sewer.

* * * * * * * * * *

EASEMENT FORM FOR STORM SEWER

KNOW ALL PERSONS BY THESE PRESENTS:

That ____

____

_

(hereinafter called "Grantor") in consideration of the sum of _______________________________

____

_

to be paid by the ___________________________ upon final approval and acceptance of this Easement by the _____________________________, do hereby convey unto the City of Bismarck, a municipal corporation (hereinafter called the "City") a perpetual Easement for Storm Sewer Right-of-Way under, over, through and across the following described real estate:

(hereinafter called "Easement Area") for the purpose of the City constructing, reconstructing, repairing, enlarging and maintaining a storm sewer, together with necessary appurtenances thereto, under, over, through and across said Easement Area.

This Easement shall be subject to the following terms and conditions:

1. ERECTION OF STRUCTURES PROHIBITED. Grantor shall not erect any landscaping or structure over or within the Easement Area without obtaining the prior written approval of the City Engineer.

2. CHANGE OF GRADE PROHIBITED. Grantor shall not change the grade, elevation or contour of any part of the Easement Area without obtaining the prior written consent of the City Engineer.

3. RIGHT OF ACCESS. The City shall have the right of access to the Easement Area and have all rights of ingress and egress reasonably necessary for the use and enjoyment of the Easement Area as herein described, including but not limited to, the right to remove any unauthorized obstructions or structures placed or erected on the Easement Area.

4. EASEMENT RUNS WITH LAND. This Easement shall be deemed to run with the land and shall be binding on Grantor and on Grantor's successors and assigns.

5. APPROVAL BY THE CITY. This Easement shall not be binding until it has received the final approval and acceptance by the City.

Grantor does HEREBY COVENANT with the City that Grantor holds said real estate described in this Easement by title in fee simple; that grantor has good and lawful authority to convey the same; and said Grantor covenants to WARRANT AND DEFEND the said premises against the lawful claims of all persons whomsoever.

Each of the undersigned hereby relinquishes all rights of dower, homestead and distributive share, if any, in and to the interests conveyed by this Easement.

Words and phrases herein including acknowledgment hereof shall be construed as in the singular or plural number, and as masculine or feminine gender, according to the context.

Signed this _____ day of ________________, ______.

____

____

____

STATE OF NORTH DAKOTA, COUNTY OF ________________, ss

On this _____ day of ________________, ________, before me the undersigned, a Notary Public in and for the State of North Dakota, personally appeared

to me known to be the identical persons named in and who executed the same as their voluntary act and deed.

____

Notary Public in and for __________ County, North Dakota.

I, ____, do hereby certify that the within and foregoing Easement was duly approved and accepted by the

____________________________________ by Resolution and Roll Call No. ___________, passed

on the _____ day of ________________, ______, and this certificate is made pursuant to authority

contained in said Resolution.

Signed this _____ day of ________________, _______.

____

Authorized Signature of _____________________________

* * * * * * * * * *

12.3 EASEMENT FOR STORM SEWER AND SURFACE WATER DRAINAGE

Easements for storm sewer and surface water drainage shall be used where there is a storm sewer placed within a drainage way, manmade or natural, and where the drainage way must be maintained to carry stormwater flow in excess of the capacity of the storm sewer. The easement width must accommodate and contain the width and depth of flow from a 100-year storm. The easement width must also allow for maintenance equipment. The following easement form may be used for an easement for storm sewer and surface water drainage.

* * * * * * * * * *

EASEMENT FOR STORM SEWER AND SURFACE WATER DRAINAGE

KNOW ALL PERSONS BY THESE PRESENTS:

That
____
____
____
____

(hereinafter called "Grantor") in consideration of the sum of ________________________________ to be paid by the __________________________________ upon final approval and acceptance of this Easement by the ____________________________________, do hereby CONVEY unto the City of Bismarck, a municipal corporation (hereinafter called "City"), a perpetual Storm Sewer Easement and a perpetual Easement for Surface Water Drainage under, over, through and across the following described real estate:

(hereinafter called "Easement Area") for the purpose of the City constructing, reconstructing, repairing, enlarging and maintaining a storm sewer, together with necessary appurtenances thereto, under, over, through and across said Easement Area as well as for the purpose of the City constructing, reconstructing, repairing, grading, and maintaining the surface of said Easement Area in a manner that will permit the free and unobstructed flow of surface water over the Easement Area.

1. <u>ERECTION OF STRUCTURES PROHIBITED</u>. Grantor shall not erect any structures over or within the Easement Area without obtaining the prior written approval of the City Engineer.

2. <u>OBSTRUCTIONS PROHIBITED</u>. Grantor shall not erect or cause to be placed on the Easement Area any structure, material, device, thing, or matter which could possibly obstruct or impede the normal flow of surface water over the Easement Area without obtaining the prior written approval of the City Engineer.

3. <u>CHANGE OF GRADE PROHIBITED</u>. Grantor shall not change the grade, elevation or contour of any part of the Easement Area without obtaining the prior written consent of the City Engineer.

4. <u>RIGHT OF ACCESS</u>. The City shall have the right of access to the Easement Area and have all rights of ingress and egress reasonably necessary for the use and enjoyment of the Easement Area as herein described, including but not limited to, the right to remove any unauthorized obstructions or structures placed or erected on the Easement Area.

5. <u>EASEMENT RUNS WITH LAND</u>. This Easement shall be deemed to run with the land and shall be binding on Grantor and on Grantor's successors and assigns.

6. <u>APPROVAL BY CITY</u>. This Easement shall not be binding until it has received the final approval and acceptance by the City by Resolution.

Grantor does HEREBY COVENANT with the City that Grantor holds said real estate described in this Easement by title in fee simple; that grantor has good and lawful authority to convey the same; and said Grantor covenants to WARRANT AND DEFEND the said premises against the lawful claims of all persons whomsoever.

Each of the undersigned hereby relinquishes all rights of dower, homestead and distributive share, if any, in and to the interests conveyed by this Easement.

Words and phrases herein including acknowledgment hereof shall be construed as in the singular or plural number, and as masculine or feminine gender, according to the context.

Signed this _____ day of ________________, ______.

____

____

____

STATE OF NORTH DAKOTA, COUNTY OF ________________, ss

On this _____ day of _______________, ______, before me the undersigned, a Notary Public in and for the State of North Dakota, personally appeared

to me known to be the identical persons named in and who executed the same as their voluntary act and deed.

____

Notary Public in and for __________ County, North Dakota.

I, ____, do hereby certify that the within and foregoing Easement was duly approved and accepted by the City by Resolution and Roll Call No. ___________.

Signed this _____ day of _______________, _______.

____

Authorized Signature of ____________________________

* * * * * * * * * *

12.4 EASEMENT FOR SURFACE WATER DRAINAGE

Surface water drainage easements shall have an adequate width to accommodate a 100-year storm and maintenance equipment. In addition, temporary easements may be required for construction. The following form may be used for drainage easements where a storm sewer is not required.

* * * * * * * * * *

EASEMENT FOR SURFACE WATER DRAINAGE

KNOW ALL PERSONS BY THESE PRESENTS:

That

(hereinafter called "Grantor") in consideration of the sum of

____

to be paid by the ____________________________________ upon final approval and acceptance of

this Easement by the _______________________________________, do hereby CONVEY unto the City of Bismarck, a municipal corporation (hereinafter called "City"), a perpetual easement for Surface Water Drainage under, over, through and across the following described real estate:

(hereinafter called "Easement Area") for the purpose of the City constructing, reconstructing, repairing, grading, and maintaining the surface of said Easement Area in a manner that will permit the free and unobstructed flow of surface water over the Easement Area.

1. ERECTION OF STRUCTURES PROHIBITED. Grantor shall not erect any structures over or within the Easement Area without obtaining the prior written approval of the City Engineer.

2. OBSTRUCTIONS PROHIBITED. Grantor shall not erect or cause to be placed on the Easement Area any structure, material, device, thing, or matter which could possibly obstruct or impede the normal flow of surface water over the Easement Area without obtaining the prior written approval of the City Engineer.

3. CHANGE OF GRADE PROHIBITED. Grantor shall not change the grade, elevation or contour of any part of the Easement Area without obtaining the prior written consent of the City Engineer.

4. RIGHT OF ACCESS. The City shall have the right of access to the Easement Area and have all rights of ingress and egress reasonably necessary for the use and enjoyment of the Easement Area as herein described, including but not limited to, the right to remove any unauthorized obstructions or structures placed or erected on the Easement Area and the right to improve, repair, and maintain the Easement Area in whatever manner necessary to provide adequate and proper drainage and to protect the public health, safety and general welfare.

5. EASEMENT RUNS WITH LAND. This Easement shall be deemed to run with the land and shall be binding on Grantor and on Grantor's successors and assigns.

6. APPROVAL BY CITY. This Easement shall not be binding until it has received the final approval and acceptance by the City by Resolution and Roll Call No. ________.

Grantor does HEREBY COVENANT with the City that Grantor holds said real estate described in this Easement by title in fee simple; that Grantor has good and lawful authority to convey the same; and said Grantor covenants to WARRANT AND DEFEND the said premises against the lawful claims of all persons whomsoever.

Each of the undersigned hereby relinquishes all rights of dower, homestead and distributive share, if any, in and to the interests conveyed by this Easement.

Words and phrases herein including acknowledgment hereof shall be construed as in the singular or plural number, and as masculine or feminine gender, according to the context.

Signed this _____ day of _______________, ______.

____

____

____

STATE OF NORTH DAKOTA, COUNTY OF ________________, ss

On this _____ day of _______________, ______, before me the undersigned, a Notary Public in and for the State of North Dakota, personally appeared

____
____
_

to me known to be the identical persons named in and who executed the same as their voluntary act and deed.

____

Notary Public in and for __________ County, North Dakota.

I, ____, do hereby certify that the within and foregoing Easement was duly approved and accepted by the City by Resolution and Roll Call No. ___________.

Signed this _____ day of _______________, ______.

____

Authorized Signature of ____________________________

* * * * * * * * * *

CHAPTER 13
OPERATION AND MAINTENANCE

13.1 INTRODUCTION

All stormwater management systems must be operated and maintained. A well maintained stormwater management system will be ready to convey the runoff from the next storm event with minimal damage to stormwater management facilities. A poorly maintained stormwater management system may not be able to function at its design conveyance and could be damaged by the runoff. The increases of potential repair costs and liability exposure are less obvious, but no less serious. Minor repairs can often prolong the service life of a facility, and can reduce the costs of future major repairs and/or replacement.

Because stormwater management systems function intermittently and seldom at full capacity, it is all too easy to defer maintenance and repair activities. The stormwater management system must be actively maintained. That is, it is too late to repair damage from the last storm or to prevent maintenance from a storm about to commence. Maintenance and repair activities can generally be categorized under two categories: (1) scheduled maintenance and repair (includes mowing, trash pickup and other routine activities associated with a normal wear of the drainage facilities), (2) unscheduled repair (includes the repair of erosion damage, structure repair, and other activities which occur on an infrequent and nonperiodic basis).

The owner of stormwater management facilities should establish a routine maintenance inspection program once the facility has been constructed and placed in service. Inspections should be conducted on an annual or semiannual basis, as well as following major storm events. Provisions must be made for funding of both scheduled and unscheduled maintenance and repair activities.

13.2 OPERATION AND MAINTENANCE OBJECTIVES

A thorough stormwater management system operation and maintenance program will provide for scheduled maintenance activities and will also accommodate necessary unscheduled work. The general goals of an operation and maintenance program should include the following:

1. Participate in stormwater management project planning and in design review to facilitate maintenance activities.

2. Participate in construction progress meetings to determine if maintenance oriented facilities are being built as called for in the design.

3. Inspect the facilities regularly to monitor their effectiveness and need for repairs.

4. Prevent stormwater management systems from falling into visual disrepair.

5. Reduce life/cycle cost through effective design review, timely maintenance activities, documentation of crew and equipment productivity, and an analysis of repair costs and longevity.

6. Repair deteriorated facilities before major damage or failure occurs.

7. Have stormwater management systems repaired, cleaned, and ready to function before the next storm event arrives.

8. Repair and maintain stormwater management facilities as necessary to insure they are capable of operating at full designed conveyance capability.

13.3 OPEN CHANNEL STORMWATER FACILITIES

13.3.1 Design and Construction Stage

1. Access

   Vehicle access is vital to the operation and maintainability of a stormwater management system. Ramps leading into channels and/or all-weather trails parallel along the system are desirable and should be included in the design. Access ramps and trails should have traffic control barriers to keep unwanted traffic from using the trails while still allowing pedestrian movement.

2. Side Slopes

   Grass lined channels must have slopes that are steep enough to drain toward the channel and yet are gentle enough to allow vegetation to establish and to permit mowing and clean-up activities. Generally, side slopes of 4:1 or flatter are desirable from the standpoint of mowing.

3. Vandalism

   Stormwater management facilities can be an attractive nuisance and can be damaged by those who use the area. Preventive measures may be necessary to keep graffiti off walls, to keep rock riprap from being relocated, or to keep gabion baskets from being cut open.

4. Localized Erosion

   There are several locations that can suffer erosion and subsequently need increased maintenance work. Proper design will reduce the likelihood of erosion problems. A practical review of the project plans may reveal the need for additional erosion protection in the following places:

a. At all transitions, such as changes in channel cross-section or channel lining.
b. At the outside of curves where flow velocities are higher.
c. At the outlet of all tributary storm sewer pipes.
d. On the bank opposite all tributary pipes and channels.
e. Downstream from drop structure energy dissipation basins.
f. Downstream of bridges and culverts.

5. <u>Trash Racks</u>

Normally, these structures do exactly what they are designed to do, catch debris. The bars should be spaced to allow small debris to pass through yet catch large material. Arrange the bars to facilitate cleaning and to allow debris to float out of the water as water level rises.

6. <u>Sediment Traps</u>

If called for in the design, sediment traps will certainly need regular silt removal to protect the downstream facilities. The sediment traps effectively reduce downstream maintenance needs.

13.3.2 <u>Post Construction and Operation Stage</u>

1. <u>Mowing</u>

In urbanized areas, the drainage channel should be mowed often enough to control weeds and to show community responsibility. For native grass vegetation in a semi-arid climate, 3 to 6 cuttings per year are satisfactory. The City should monitor those areas mowed by others such as the Park District, business or home owners.

2. <u>Debris Control</u>

Debris blockage at drainage structures often contributes to flooding problems. Trash racks and debris traps help reduce the problem if they function properly and are regularly cleaned. Regular debris removal along the length of the channel system also helps. This should include trimming and thinning of trees as they encroach on the stormwater channel or if they have become overgrown.

3. <u>Inspection</u>

Inspection at least annually of drainage facilities including storm sewer/drainage easements will detail long-term changes in the system and will highlight needed maintenance work. Inspections should also be done following major storm events.

4. Silt Removal

Some silt accumulation in stilling basins and around channel obstructions is inevitable, and is harmless in limited amounts. Silt should be removed if it is severe enough to alter the water surface or to affect the function of stormwater conveyance facilities such as drop structures. Silt accumulations can also cause trouble by supporting undesirable or obstructive vegetation.

5. Trail and Access Repair

An annual effort to repair damaged trail sections and access ramps will result in guaranteed maintenance access and better pedestrian use. This maintenance activity is the responsibility of the owner/operator of the trail system.

13.3.3 Rehabilitation Stage

The regular stormwater facility inspection program will alert maintenance personnel when a system is in need of repairs. If the problems are repaired promptly, the facility can be returned to service with little threat of further damage or failure. Typical problem areas that signal need for rehabilitation are as follows:

1. Hard Lined Trickle /Low Flow Channels - Local undermining of the structure or secondary channel erosion parallel to the main channel.

2. Soft Lined Trickle/Low Flow Channels - Random bank failure and bottom degradation is unsafe or threatens other improvements.

3. Tributary Channels and Pipe Outlets - Erosion from the receiving channel leading back to the tributary outlet and/or erosion under the outlet structure.

4. Drop Structures and Grade Control Structures - Erosion damage in and around the energy dissipation basin and around the outside edges of the structure. Physical damage can occur to the structure in the form of uplifted or depressed concrete, broken gabion baskets, and displaced riprap.

5. Channel Banks - Channel bottom and sides should be maintained to their original slopes. Bank protection such as riprap, slope paving, or retaining walls can be undermined by local scour or general degradation if not "toed-in" deep enough. Grass-lined banks can lose vegetation cover and can suffer spot erosion that may quickly worsen.

13.4 PIPED STORMWATER FACILITIES

13.4.1 Design and Construction Stage

1. Access

   Manpower and equipment access for the length of the pipe to system must be available. Publicly owned right-of-way or easements are normally sufficient. However, it is imperative that these easements connect to a public right-of-way, other easements, or receiving waters and are wide enough for maintenance activities. The easement language must be restrictive enough to prohibit undesirable activities on the land surface above the pipe facilities.

2. Erosion Protection

   The inlets and outlets to pipe systems are subject to high velocities. Adequate protection will usually take the form of riprap aprons, energy dissipation structures or concrete headwalls, wingwalls and aprons. Steep earth slopes at inlet and outlet transitions frequently need short walls to hold the soil in place.

3. Trash Racks

   This is one of the most frequent problem areas. The design should consider the potential debris source upstream. Designers should assume at least 50% blockage of a trash rack when designing for the maximum storm runoff event. Another rule of thumb requires a trash rack to have four times the clear opening area of the pipe being protected.

4. Manholes

   Manhole location and spacing are discussed in Chapter 4, Section 4.6 of this manual. Manholes need to be accessible in all-weather conditions. Be sure access is available to all pipes of a multi-barrel system. Drop manholes can be especially difficult when designing for adequate access and safety.

5. Record Drawings

   It is an absolute necessity to obtain record drawings of the completed project and to maintain them for future reference.

13.4.2 Post Construction and Operation

1. Curb Inlet Cleaning

   Because of their location and shape, inlets often trap sediment and debris. They should be cleaned twice a year to ensure their proper function. If only one cleaning is possible, it should occur prior to the rainy season.

2. Debris Control

   Trash racks should be cleaned regularly to keep accumulations from forming. In-pipe debris should be removed if it is large enough to create a flow obstruction.

3. Overflow Channel Maintenance

   If the pipe system was designed with a surcharge or overflow channel (easement), it deserves occasional attention. It must be kept clear of debris, excessive vegetation or sedimentation, as well as monitored for alterations to the channel by adjacent owners. In general, it should be maintained as an open channel to be ready to function when called upon.

4. Inspection

   A regular in-pipe inspection of pipe drainage systems will detail long term changes and point out needed maintenance work such as debris removal or pipe joint patching. Special attention is necessary to insure the safety of the inspection personnel if the pipe is long. Small pipes and pipes that carry continuous flow can be viewed with automated equipment. Inspections should be done following major runoff or windy events. Inlet grates should be checked for clogging, and catch basins and pipes for sediment/waste blockage.

13.4.3 Rehabilitation Stage

1. Inlet and Outlet Structures

   Local erosion due to high velocities, lack of protection, or transition turbulence will need repair and correction.

2. Trench Backfill

   Subsidence of the trench, which can result from poor initial compaction or from pipe or joint failure, needs to be brought to grade. Earth settlement around manholes is a frequent indicator of compaction problems.

3. Pipe Joints

The first sign of problems in a piped stormwater system usually shows at the pipe joints. Spalled concrete, cracks, distorted pipe geometry, backfill movement, and water inflow occur at the joints and are precursors of greater problems to come. If such problems are noted at pipe joints, immediate repair action must be initiated to prevent further and accelerated deterioration of the system.

13.5 STORMWATER DETENTION AND RETENTION FACILITIES

13.5.1 Design and Construction Stage

1. Access

The detention structure and impoundment area should be accessible to maintenance equipment for the removal of silt and debris, and for repair of inevitable minor erosion problems. Easements and/or rights-of-way are required to allow access to the detention structure and impoundment area by the owner or agency responsible for maintenance.

2. Safety

If children are apt to play in the vicinity of the detention structure and impoundment area, installation of an attractive fence or preferably, landscaping that will discourage entry (thick, thorny shrubs) along the periphery of the detention facilities may be advisable. The detention facility is situated at a lower grade and adjacent to a highway installation of a guard rail would be in order. Access to the pond bottom and outlet is especially important.

3. Side Slopes

Because of aesthetic requirements and the need for frequent mowing, the bank slope, bank protection, and vegetation type are important design criteria. Flat bank slopes on the order of 4:1 or flatter facilitate mowing and aid in reducing rill erosion. It is important that appropriate vegetation be established as soon as possible after construction.

4. Drawdown Capability

It is important that detention facilities with permanent pools have provisions for complete drainage in order to facilitate silt removal operations. The frequency of sediment removal will vary among facilities, depending upon the original volume set aside for sediment, the rate of accumulation, drainage area erosion control measures and aesthetic appearance of the pond.

5. Silt Storage

All detention facilities should be designed with sufficient depth to allow for accumulation of sediment over several years prior to its removal.

6. Wet/Permanent Pool Basins

Wet basins should be of sufficient depth to discourage excessive aquatic vegetation on the bottom of the basin. Adequate dissolved oxygen supply in ponds (to minimize deterioration of water quality) can be maintained by artificial aeration. Use of fertilizer and pesticides adjacent to the pond should be carefully controlled.

7. Secondary Uses

Secondary uses of a detention facility in an impoundment area that would be incompatible with sediment deposits should not be planned unless a high level of maintenance will be provided. For example, planning a combination tennis court/detention basin may not be advisable downstream from an area prone to soil erosion. If the bottom of the detention basin is to be used for other uses, it needs special consideration to minimize periods of wetness. It may be advisable to provide an underground tile drainage system, if active recreation is contemplated and a flat bottom is required.

8. Outlet Structures

To reduce maintenance, outlet structures should be designed with nonmoving parts (i.e., pipes, box culverts, orifices, and weirs). Manually and/or electrically operated gates should be avoided. To reduce maintenance, outlets should be designed with openings as large as possible, compatible with the depth-outflow curve desired and with water quality, safety, and aesthetic objectives in mind. One way of doing this is to use a larger outlet pipe and construction of an orifice or v-notch weir on the headwall to reduce outflow rates. Outlets should be robustly designed to lessen chances of damage from debris or vandalism. The use of thin steel plates or sharp crested weirs is best avoided because of the potential for accidents, especially with children. Thin plate orifices must be protected with trash racks.

9. Miscellaneous Considerations

French drains or other small underground detention facilities are almost impossible to maintain and should not be used where sediment loads are apt to be high. Underground tanks or conduits designed for detention should be sized and designed to permit entrance of mechanized equipment to remove accumulated sediment.

13.5.2 Post Construction and Operation Stage

The most important routine function of maintenance for stormwater detention facilities are cutting grass and weeds, removing sediment, repairing any erosion, and cleaning out debris. These operations must be planned and adjusted to local conditions, such as erodability of upstream soils, prevalence of insect vectors, and the type of outlets utilized in a particular facility.
To maintain aesthetic appeal, floating debris must be removed from the pool surface after a storm. Therefore, to remove the debris, access must be provided for vehicles and boats. Accumulated sediment and weed growth can be removed by dredging or by excavating equipment after dewatering.
Where pumping facilities are required, the pumphouse should be designed to provide security and resistance to vandalism, which could include fencing and vandal/resistant doors and locks. Between storm events, maintenance should include lubrication and operation of the pumps on a regular schedule to ensure they will function when needed.

Outlets that are protected by a trash rack will accumulate trash during and between storm events. To facilitate outlet operation and maintenance, trash racks should be curved or inclined so that debris tends to ride up as the water level rises. Such design leaves the rack clear and allows easier cleaning during a storm event. The periodic removal of debris from orifice trash racks is the single most important maintenance aspect of any effective stormwater detention program.

Outlet structures may be partially or completely plugged by a build-up of deposited sediment, by floating plant growth, and by vegetation growing in the sediment. Sediment deposition is a natural occurrence in basins and periodic removal of vegetation and sediment is necessary to insure that the intended hydraulic function of the outlet is not impeded. Such activities should be anticipated during design so that both maintenance access and a nearby waste and disposal site are provided.

Proper design of the outlet structure can minimize the need for maintenance of the discharge end. High velocity outflow can erode the downstream outlet, foreslope, and channel. A well designed and constructed energy dissipater, surrounded by large, well graded riprap on the foreslope and downstream channel can do much to reduce the maintenance needed. Deep toewalls to resist scour (undercutting) should also be provided.

All portions of the outlet structure must be accessible to vehicles, equipment, and personnel between and during storm events. This includes the floor of the basin as well as ramps to points above the upstream and downstream sides of the outlet structure.

Provisions to remove particulate pollution are closely related to the desirability of prolonged retention of flood flows for flood damage reduction. From the viewpoint of capital investment, the additional water quality control function is obtained virtually without cost. However, prolonged retention of storm flows for either purpose requires a small outlet, which entails additional maintenance. The grass is cut periodically as a part of general grounds maintenance. At the same time, debris should be removed in front of the trash racks protecting small orifices.

13.5.3 Rehabilitation Stage

With regular inspection, components of a stormwater detention facility in need of repair will become readily obvious. If the problems are repaired promptly, the facility can return to service with little threat of further damage or failure. Listed below are several of the typical problem areas that signal the need for rehabilitation:

1. Upstream and Downstream Slopes

   The condition of the upstream and downstream slopes of a detention facility provides many warnings of possible future problems. Such warnings include deficient grass cover, erosion or slides, trees growing on the slope, longitudinal and transverse cracks, deficient riprap protection, and visual depressions, bulges or settlements. In addition, burrow holes of small animals can lead to major piping problems for the embankment. Immediate action must be taken to investigate and carry out necessary repairs to correct obvious problems.

2. Outlet Works

   Concrete drainage structures in need of repair exhibit spalling, cracking, erosion, scaling, and exposed reinforcement. Pipe joints may show displacement or offset, loss of joint material, and leakage. Grass-lined overflow spillways should be inspected for erosion and sloughing of slopes and the crest.

3. Terminal Structure and Outlet Channel

   Terminal structures or energy dissipaters constructed of concrete may show spalling, cracking, erosion, scaling, or exposed reinforcement. The outlet channel should be inspected for erosion and backcutting, sloughing, obstructions, and the need for riprap protection.

CHAPTER 14
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Chapter 15

FORMS

CITY OF BISMARCK ENGINEERING DEPARTMENT *

Phone (701) 355-1505
221 North Fifth Street
P.O. Box 5503
Bismarck, ND 58506-5503

http://www.bismarck.org/index.aspx?nid=17

1) Stormwater Management Plan and Permit Process Flow Chart

2) City of Bismarck Stormwater Plan/Permit Application Checklist

NORTH DAKOTA DEPARTMENT OF HEALTH * Phone (701) 328-5210

Division of Water Quality
1200 Missouri Avenue
P.O. Box 5520
Bismarck, ND 58502-5520

http://www.ndhealth.gov/WQ/Storm/StormWaterHome.htm

1) Notice of Intent to Obtain Coverage under NDPES General Permit for Stormwater Discharges Associated with Construction Activity

2) Notice of Intent to Obtain Coverage under NDPES General Permit for Stormwater Discharges Associated with Industrial Activity

NORTH DAKOTA STATE WATER COMMISSION * Phone (701) 328 -2752

North Dakota State Water Commission
900 East Boulevard
Bismarck, ND 58505-0187

http://www.swc.state.nd.us/4dlink9/4dcgi/redirect/index.html

1) Application/Notification to Construct of Modify Dam, Dike, Ring Dike, or Other Water Resource Facility

2) Application to Drain

3) Application for Authorization to Construct a Project within Island and Bed of Navigable Streams and Water

UNITED STATES ARMY CORPS OF ENGINEERS * Phone (701) 255 -0015

North Dakota Regulatory Office
U.S. Army Corps of Engineers
1513 South 12th Street
Bismarck, ND 58504

1) Corps of Engineers Section 404 - Section 10 Application

***** ***Information on specific application procedures and requirements are available directly from each agency. Developers are encouraged to contact these agencies to determine the need to apply for these permits or to comply with the applicable federal, state and local regulations.***

Chapter 16

CURRENT FEE SCHEDULE

The City of Bismarck Current Fee Schedule is available through the City Engineering Department at the following address:

Engineering
221 N. 5th St.
Bismarck, ND 58501

Phone: 701-355-1505
Fax: 701-222-6593
Email: bisengd@nd.gov

Or go to the Community Development Department under Planning then Land Use/Development Process. http://www.bismarck.org/index.aspx?NID=1020

Click on:

City/ETA Site Plan Application & Checklist - pdf file
Revised 6/2008

City/ETA Site Plan Requirements - pdf file
Revised 12/2009

Chapter 17

STORMWATER CONSTRUCTION SPECIFICATIONS

The City of Bismarck Construction Specifications are available through the City Engineering Department at the following address:

Engineering
221 N. 5th St.
Bismarck, ND 58501

Phone: 701-355-1505
Fax: 701-222-6593
Email: bisengd@nd.gov

Or go to our Engineering Department website under Contractors, Vendors & Consultants, click on Construction Specs & Standard Details.

http://www.bismarck.org/index.aspx?NID=312

Chapter 18

STANDARD DETAILS

The City of Bismarck Standard Details are available through the City Engineering Department at the following address:

Engineering
221 N. 5th St.
Bismarck, ND 58501

Phone: 701-355-1505
Fax: 701-222-6593
Email: bisengd@nd.gov

Or go to our Engineering Department website under Contractors, Vendors & Consultants, click on Construction Specs & Standard Details.

http://www.bismarck.org/index.aspx?NID=312

See City of Bismarck Standard Details 800-1206 pdf's

Chapter 19
STORMWATER MANAGEMENT ORDINANCE

The City of Bismarck Standard Management Ordinance is available through the City Attorney Office at the following address:

City Attorney
221 North 5th Street
Bismarck, ND 58501-4028

Contact:
Arla Lawler

Phone: (701) 355-1343
Fax: (701) 222-6470
Email: alawler@nd.gov

Or See City of Bismarck City Attorney Website

http://www.bismarck.org/index.aspx?NID=90

Title 14 - Zoning.pdf - pdf file (527 KB)
Posted on: 12/29/2005