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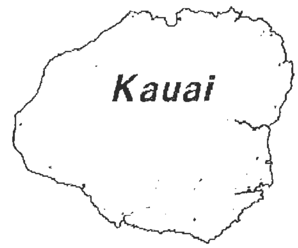

STORM WATER RUNOFF SYSTEM MANUAL

JULY 2001

DEPARTMENT OF PUBLIC WORKS
COUNTY OF KAUAI

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STORM WATER RUNOFF SYSTEM MANUAL

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Approved:

CESAR C. PORTUGAL, COUNTY ENGINEER

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SECTION 1 - GENERAL PROVISIONS

1.1 Title: This manual together with all future changes and amendments shall be known as the *Department of Public Works (DPW), County of Kauai, Storm Water Runoff System Manual,* hereinafter referred to as the MANUAL.

1.2 Jurisdiction: The MANUAL shall apply to all land within the jurisdictional boundaries of the County of Kauai (*County*) which includes the Islands of Kauai and Niihau. The MANUAL shall apply to all facilities constructed within County rights-of-way, easements dedicated for public use and to privately-owned systems that are part of the required infrastructure improvements for a subdivision. The runoff detention and water quality requirements will apply to privately-owned and maintained facilities that are required to meet drainage design requirements of the County. These facilities include but are not limited to closed conduits, inlets, manholes, culverts, swales, detention basins, channels and other facilities approved by the County Engineer.

1.3 Purpose and Effect: The MANUAL has been prepared to guide County Engineers and other interested parties in the general features required for the design of storm drainage facilities in the County of Kauai.

The MANUAL is not intended to limit the initiative and resourcefulness of the engineer in developing drainage plans, or be viewed as maximum limits in design criteria. More stringent criteria should be used where such is indicated.

1.4 Review and Approval: The County will review all drainage submittals for compliance with the MANUAL. An approval by the County does not relieve the owner, engineer, or designer of the responsibility for ensuring that the calculations, plans, specifications, construction and as-built drawings are in compliance with the MANUAL, and that the necessary or desired drainage objectives will be accomplished.

1.5 Interpretation: In the interpretation and application of the provisions of the MANUAL the following shall govern:

a. In its interpretation and application, the provisions shall be regarded as the minimum requirements for the protection of the public health, safety and welfare of the residents of the County of Kauai.

b. If other laws, ordinances, or regulations cover the same subject as the MANUAL, the stricter requirement shall apply.

c. The MANUAL shall not abrogate or annul any permits or approved drainage reports, construction plans, easements, or covenants issued before the effective date of the MANUAL.

1.6 Variance:

Authority

The County Engineer may grant variances from the provisions of this Section only in unusual situations as set forth in this Manual.

Standards

Variances from the terms of this Manual shall be requested only for the following reasons:

a. Situations where strict compliance with the Manual may not serve to protect the public health and safety.

b. Situations which require additional analysis outside the scope of the Manual for which the additional analysis shows that deviation from the Manual will not be detrimental to public health and safety.

c. Hydrologic and/or hydraulic conditions which cannot be adequately addressed by strict compliance with the Manual.

In no case will a variance be granted for conditions which are created by improper site planning (i.e. lack of adequate space allocations) or when it will provide the applicant with special privileges not enjoyed by other properties in the vicinity. The County Engineer shall indicate the particular evidence that supports the granting of the variance.

Application

An application for a variance shall be filed by a registered Civil Engineer. The application shall contain the information required pursuant to this Manual and other information justifying the issuance of the variance.

Procedure

(a) All variances shall be submitted in writing to the County Engineer for approval.

(b) The County Engineer shall inform the applicant within thirty (30) calendar days from receipt of the written request as to whether the variance request is a complete application containing the required scientific and technical information necessary for the variance consideration, unless a mutually agreed-to extension is obtained from the County Engineer.

(c) Upon receipt of a completed written application, the County Engineer shall either grant or deny the variance request within sixty (60) calendar days. Upon the determination of the County Engineer that a variance may be granted consistent with the requirements of this Manual, the Variance shall be issued to the applicant on such terms and conditions, as the facts may warrant. The County Engineer shall append conditions that achieve a substantial equivalent or alternative to the regulation from which the variance is sought.

1.7 Definitions: For the purpose of the MANUAL, unless it is plainly evident from the context that a different meaning is intended, certain words and phrases used herein are defined as follows:

a. *Best Management Practices (BMPs)* shall mean temporary and permanent practices that are consistent with the requirements of Title 11, Chapter 55 of the Hawaii Administrative Rules as administered by the State Department of Health (DOH) Clean Water Branch and selected by the engineer to mitigate water quality impacts due to development.

b. *Building Setback Line* shall mean: the Flood Boundary as defined by Flood Emergency Management Agency (FEMA) Flood Insurance Study (FIS). Where no Flood Boundary has been defined by a FEMA FIS or COE Flood Study:

- 1 foot above the Flood Elevation as calculated by an engineer utilizing FEMA-approved methods for riverine Flood Insurance Studies.
- 3-foot above the Flood elevation as calculated by an engineer utilizing normal depth computations.

c. *Comprehensive Zoning Ordinance* shall mean Chapter 8 of the Kauai County Code, the Comprehensive Zoning Ordinance.

d. *County Engineer* shall mean the Engineer in charge of the Department of Public Works, County of Kauai.

e. *Detention Facilities* shall mean facilities such as detention basins or wetlands used to provide the temporary impoundment, or detention, of runoff, to control runoff rates.

f. *Development* shall be as defined in the Comprehensive Zoning Ordinance of the County of Kauai.

g. *Drainage Basin* shall mean the area of land from which storm runoff flows to a drainageway.

h. *Drainageway* shall mean any drainage flow path consisting of natural channels, gullies, valleys, flood plains, wetlands or other waterways.

i. *Engineer* shall mean a Licensed Professional Civil Engineer currently registered by the State of Hawaii to practice Engineering in the discipline of Civil Engineering.

j. *Finished Floor Elevation* shall mean the elevation of the lowest habitable floor of any structure.

k. *Floodway, Flood Fringe and Flood Elevation* shall be as defined by the Federal Emergency Management Agency (FEMA) on the FEMA Flood Insurance Rate Map (FIRM).

l. *Flow Pattern and Rate* shall mean the calculated flow pattern and rate of flow of stormwater runoff as determined by the Engineer utilizing topographic information and hydrologic and hydraulic engineering analysis.

m. *Local Drainage System* shall mean a system that consist of streets, swales, curb and gutter, inlets, closed conduits, culverts, channels and other drainage facilities required to convey the minor storm runoff from a tributary area of less than 100 acres to a major drainageway.

n. *Major Drainageway or Major Drainage System* shall mean any drainage flow path consisting of natural channels, gullies, large storm drain trunk lines, regional detention areas and other facilities for major storm runoff from a tributary area of 100 acres or more.

o. *Peak Flow* shall mean the maximum instantaneous flow during a given storm event as calculated by approved hydrologic methods.

p. *Predevelopment* shall mean the existing condition of a property prior to any development activity.

q. *Property and Right-of-Way Lines* shall mean the boundaries or lot lines shown on a subdivision map that depict the separation between two properties or land parcels.

r. *Storm Event* shall mean the estimated amount of rainfall, given in inches, that is expected to occur during a storm with a certain duration and recurrence interval.

s. *Subdivision* shall mean all activities as defined in the Subdivision Ordinance of the County of Kauai.

t. *Ultimate Development* shall mean the future development of a drainage basin to conform to the land uses as shown on the Kauai County General Plan.

u. *Volume of Runoff* shall mean the total amount of runoff for a given storm event as calculated by approved hydrologic methods.

v. *Watershed* is a system of contiguous drainage basins that ultimately flow to the ocean.

SECTION 2 - DRAINAGE PLANNING AND SUBMITTALS

2.1 Review Process: All subdivision and development shall be required to submit drainage reports, plans, construction drawings, specifications and as-built information in conformance with the requirements of the MANUAL, when required as a condition for approval by the Planning Director and/or County Engineer.

2.2 Drainage Report: The drainage report shall be prepared, stamped, and signed by an Engineer and shall be submitted to the County for review and approval. Reports shall be cleanly and clearly reproduced and legible throughout. Blurred or unreadable portions of the report will be deemed unacceptable and will require resubmittal.

2.2.1 Drainage Report Contents: The purpose of the drainage report is to define the nature of the proposed development or project, and to describe all existing conditions and proposed facilities needed to conform to the requirements of the MANUAL.

The drainage report shall include:

a. General Location and Description:

1. General Location Map(s) shall include:
   - Name of town
   - Tax Map Key (TMK)
   - Names of local streets within and adjacent to the proposed subdivision or project.
   - Identification of major and local drainageways, facilities, and/or easements within and adjacent to the proposed subdivision or project.
   - Names of surrounding developments.
   - Flood Information

- Property Boundaries

2. Description of Property shall include:
   - Area of property in acres
   - Ground cover (type of trees, shrubs, vegetation, general soil conditions, topography, and average slope)
   - Major drainageways
   - General project description
   - Proposed land use

b. Hydrologic map(s) and data for the existing drainage condition. The map shall contain contours and show the boundaries of the drainage areas, existing drainage facilities, concentration points, and existing flow patterns. Data and calculations shall show discharges and other information relative to the existing drainage condition including ground cover and soils information. A description of the existing drainage patterns and systems shall also be included in the data and calculations.

c. Hydrologic map(s) and data for the proposed onsite and offsite drainage improvements. The map and data shall show the existing and proposed contours, property boundaries, boundaries of the drainage areas, concentration points, discharges, proposed street and lot flow pattern, overflow routes and a description of the proposed drainage improvements.

d. The drainage report will describe the onsite and offsite drainage improvements for the proposed development. The onsite and offsite drainage improvement plan with supporting calculations shall include:

   1. Plan and profile of proposed onsite and offsite drainage improvements.

2. Drainage sub-areas and discharges.

3. Catch basin/drain inlet interception and bypass rates.

4. Street flooding or dry pavement widths.

5. Design flows between manholes and catch basin inlets.

6. Hydraulic grade lines in culverts, manholes, and catch basin inlets.

7. Hydraulic grade lines and velocities at outlet structures.

8. Detention basin hydrology and hydraulics.

9. Drainageway and building setback lines and/or floodway, flood fringe, and flood elevation lines, where applicable.

10. Description of changes to existing drainage patterns on adjacent and downstream properties and "unreasonable risk".

e. Conclusions:

1. Compliance with the MANUAL.

2. The Drainage Concept will not adversely affect adjacent and downstream properties. Existing drainage conditions, including flow patterns and peak flow rates, shall not create an unreasonable risk to adjacent and downstream properties as a result of a subdivision or development project. Flow patterns and peak flow rates shall be controlled by proper grading and drainage facilities.

2.3 Construction Plans and Specifications: One (1) set of construction plans and specifications shall be submitted for review and approval. The construction plans shall be stamped and signed by an Engineer. Plans for storm drainage improvements shall include the following:

2.3.1 Plan and Profile: Plan and profile construction drawings of proposed drainage system installation, inlets and manholes with pertinent elevations, dimensions, types and horizontal and vertical controls shown.

2.3.2 Property and Right-of-Way Lines: Property and right-of-way lines, easements, existing and proposed structures, walls, fences and other land features.

2.3.3 Drainage Structures: Drainage structures such as lined channels, ditches and swales, box culverts, manholes, bridges and headwalls (proposed and existing) with cross-sections.

a. Structural Design Data:

1. Structural design computations for all drainage structures, other than pipes used within the limits of current loading tables and structures shown in the Standard Details of the County of Kauai, Department of Public Works.

2. Information pertinent to the design, such as boring data, soils report, etc.

3. Upon the completion of construction of major structures, submit pertinent data such as pile driving logs, pile tip elevations, etc.

2.3.4 Detention Basin: Detention basin grading, outlet and inlet locations, and landscaping, with cross-sections or contours sufficient to verify volumes.

2.3.5 Outlet Devices: Outlet control devices for basin, spillways and energy dissipators shall be shown in detail.

2.3.6 Maintenance: Maintenance access considerations, as required.

2.3.7 Grading and Erosion Control:
For Clearing and Grubbing, and Lot Grading, an Erosion Control Plan is required that utilizes BMP's for erosion control during construction. Temporary sediment basin(s) may be required that provides for the storage of 0.5 inch of sediment per acre of land to be cleared and grubbed or graded.

Permanent erosion and sediment control will also be required for developments greater than 2 acres. Permanent sediment basin(s) or other erosion and sediment control facilities will be required to store 0.5 inch of sediment per acre of impervious surfaces, per storm event. These facilities must be maintained after storm events so that the required erosion and sediment control is available for the next storm event.

2.3.8 Elevations: Finished floor elevations of structures and water surface elevations.

2.3.9 Current Flood Plain: Relation of site to current 100-year flood plain boundaries as defined by FEMA or the requirements of this MANUAL.

SECTION 3 - DRAINAGE POLICY

3.1 Introduction: The provisions for adequate drainage are necessary to preserve and promote the general health, welfare, and economic well being of the County. Since drainage affects all governmental jurisdictions and private property, the County has developed Drainage Policies for implementation by this MANUAL. The following Subsections describe the County of Kauai's policies.

3.2 Basic Principles:

a. Natural drainageways shall be used for storm runoff drainageways wherever possible.

b. Channelization or alteration of natural drainageways should be minimized.

c. In general, natural streams, rivers and tributaries shall not be replaced with lined drainage or closed systems, except at roadway crossings.

d. The diversion of storm runoff from one watershed to another shall be avoided. However, the transfer of drainage flow from one watershed to another is an alternative in some instances. Proposals for such transfers may be approved only when there is no other viable alternative and where detention will be provided to maintain predevelopment flow pattern and rates.

e. Existing drainage conditions, including flow patterns and peak flow rates, shall not create an unreasonable risk to adjacent and downstream properties, as a result of a subdivision or development. Flow patterns and peak flow rates shall be controlled by proper grading and drainage facilities.

f. Minor storm events shall be the 2-year recurrence interval storm. Major storm events shall be the 100-year recurrence interval storm.

g. Drainage facilities shall be designed to handle the storm flow from a 100-year recurrence interval storm when the drainage area is greater than or equal to 100 acres.

h. Where there are no downstream drainage systems and/or if the downstream drainage system does not have sufficient drainage capacity, the upstream owner shall install drainage facilities (such as detention basins) to maintain both the 2-year and 100-year storm flows at or below the predevelopment flow rates and conditions.

i. Detention basins shall be installed to maintain storm flow discharges to downstream systems at or below predevelopment peak flow rates and to regulate runoff volume discharge rates. Detention basins to be dedicated to the County shall also be developed in conjunction with parks or open lands to serve as a multi-purpose use facilities for both recreation and drainage.

Detention facilities shall be required to keep peak storm flow rates leaving the site to predevelopment levels and to detain the increased volume of runoff due to the proposed development, when the proposed project exceeds two (2) acres in size. This increased volume of runoff is intended to be released slowly after the storm event for water quality detention compliance. This requirement also applies to projects 2 acres or less in size if the downstream drainage system cannot accommodate the increase in storm flows from the project and existing structures are subject to drainage or flooding problems.

j. Bridges, box culverts, arch culverts and other structures used to cross a drainageway must convey storm water under open channel flow conditions and with sufficient freeboard and width to pass debris that may be expected to be conveyed through the structure(s). For local drainage systems and drainage areas smaller than 2 acres, the structures shall be designed to accommodate the 2-year recurrence interval storm flow. For drainage areas larger than 2 acres, the structures shall be designed to accommodate the 2-year or 100-year recurrence interval storm flow, depending on the size of the drainage basin.

k. Debris racks and/or deflectors are recommended when floating debris is expected to be conveyed by the drainageway.

l. Drainage structure flow rates are recommended to be increased by a "bulking factor" when floating debris and fine and coarse detritus are anticipated and there is no upstream debris control structure. The Engineer should use best judgement on bulking factors based on the based on the characteristics of the upstream basin.

Existing 100-year flow	= 200 cfs
Bulking factor	= 1.5
Q	= 200 x 1.5
Q	= 300 cfs

m. The County has adopted the Federal Emergency Management Agency (FEMA) Flood Insurance Rate Maps and a flood ordinance to regulate the construction of structures within floodplains. Buildings shall be protected from the 100-year storm and will not be permitted in the floodway and drainageway and building setback lines. A drainage report with the conveyance compensation measure (no-rise) calculations shall be submitted for approval for placement of a single building (structure) or another simple encroachment within an adopted regulatory floodway. Floodways, drainageways and building setback lines shall be established by drainage studies that are prepared by the County, FEMA, or engineer in accordance with this Manual, and shall be reserved to convey major storm events. Floodways, flood fringes, drainageways, and building setback lines may be adjusted by additional drainage studies that provide the scientific information to justify the adjustment. When the flood area is delineated on the FEMA Flood Insurance Rate Maps, any drainage studies prepared to justify an adjustment to the floodways and flood fringes shall comply with FEMA Guidelines for Map Revisions and approved by FEMA. All buildings are recommended to have finished floor elevations at least 1 foot above the flood level of the 100-year storm as defined by FEMA or the County's flood ordinance.

n. Storm drainage facilities shall be designed to require the least amount of maintenance as determined by the Engineer and reviewed by the County Engineer. Access to storm drainage facilities shall be provided for all drainage facilities. An easement shall be provided for all underground drainage systems. Surface drainage improvements such as swales and channels shall be constructed within a right-of way. Surface drainage improvements must include fencing and access. Maintenance of private drainage facilities shall be the responsibility of the owner of the land/development.

Easement and Right of Way required for maintenance access are as follows:

1. Easements shall have a minimum width of 10 feet for drainage culverts.

2. Unless prohibited by the topography, new drainage facility alignments in residential subdivisions must be located adjacent and parallel to property lines so that required drainage easements can be situated

along property lines. Drainage easements shall be located entirely on one property and not split between adjacent properties. If the easement is to serve as an overflow route for runoff from a major storm event, a concrete overflow swale shall be constructed in the easement. The overflow swale shall be designed to convey the 100-year recurrence interval storm.

3. Channel Right-of-Way. The channel width shall be sufficient to provide the required waterway area for the design storm as determined in accordance with the MANUAL. The total right-of-way shall include a 15-foot wide maintenance road along both banks where the top width of the channel exceeds 50 feet, and along one bank where the top width is 50 feet or less. The maintenance road along the channel shall be topped with 6 inches of crushed coral or base course and treated with bituminous material. In lieu of a maintenance road, for normally dry channels, access ramps or other suitable alternative measures to facilitate maintenance may be provided.

4. Detention and Debris Basins. A 10-foot wide access road shall be provided along the perimeter of the basin. In lieu of a maintenance road, for normally dry channels, access ramps or other suitable alternative measures to facilitate maintenance may be provided.

Drainage easements shall be shown an all subdivision maps and construction drawings.

o. Drainage design shall include an overflow route for the 100-year recurrence interval or more infrequent storms. The overflow route shall maintain the pre-development flow pattern and discharge point(s) and must not expose existing or proposed structures on neighboring landowners to unreasonable risk of foreseeable harm.

3.3 **Drainage Planning and Design**: The design criteria presented herein are intended to establish guidelines, standards, and methods for effective drainage planning and design. The County may revise and update criteria as necessary to reflect advances in the field of storm drainage engineering and water resource management. The following are the County of Kauai drainage planning and design policies:

a. The Local Drainage System shall be designed to convey the peak runoff of a minor storm event with minimal disruption to the development. A minor storm event shall be the 2-year recurrence interval storm for drainage areas less than 100 acres.

b. The Major Drainage System shall be designed to convey the peak runoff from the 100-year recurrence interval storm to minimize health and life hazards, damage to structures, and interruption to traffic and services.

c. Recurrence interval: The County of Kauai shall require that all developments include the planning, design, and implementation for both the local and major drainage systems in accordance with the following recurrence intervals:

Recurrence Intervals:

1. For local drainage systems with drainage areas less than 100 acres, Tm (recurrence interval) = 2 years, unless otherwise specified.

2. For local drainage systems with drainage areas less than 100 acres with sump or tailwater effect, and for the design of roadway culverts

and bridges utilizing static head at entrance, Tm (recurrence interval) = 2 years.

3. For local drainage systems with drainage areas greater than 100 acres and major drainage systems, Tm (recurrence interval) = 100 years.

d. Peak Flows: The United States Geological Survey (USGS) stream gage data shall be used to determine peak flows. If stream gage data is unavailable, the Natural Resources Conservation Service (NRCS) Technical Release 55 (TR-55) and/or Technical Release 20 (TR-20) are recommended to develop hydrographs and peak flows for storm runoff. The NRCS software computer programs, utilizing the TR-55 and TR-20 methods, may be used to facilitate the analysis.

The following criteria shall be used to determine storm runoff quantities:

1. For drainage areas less than 100 acres the Rational Method may be used.

2. For drainage areas larger than 100 acres and 2,000 acres or less, the NRCS Hydrograph Analysis, TR-55 or TR-20, may be used.

3. For drainage areas larger than 2,000 acres the NRCS Hydrograph Analysis TR-20, may be used.

e. Street conveyance is an integral part of the storm drainage system and may be used for transporting storm runoff up to the allowable street flooding design limits listed below. Street rights-of-way (ROW) are to be used as the drainage channel to convey frequent and infrequent storm runoff.

The allowable limits for street flooding shall be as follows:

Allowable Street Flooding for Minor Storm Runoff (2-year recurrence interval storm)

Street Classification	Allowable Flooding
1) Collector, Minor and Dead End	Flow may spread up to the crown of the street. (Water depth at the crown shall be zero.)
2) Major and Arterial	Flow spread must leave two 10-foot lanes free of water. (Ten feet dry on either side of the street crown.)

Allowable Street Flooding for Major Storm Runoff (100-year recurrence interval storm)

Street Classification	Allowable Flooding
1) All Streets	Streets can be flooded up to an elevation that residential, public, commercial, and industrial buildings shall not be inundated. There should be a minimum of 1 foot freeboard between the water surface elevation of the major storm and the finish lower floor elevation of structures.

On streets with grassed swales, concrete paved swales shall be provided where road grade equals or exceeds 7%. Inlets shall be installed at the downstream terminus of all paved swales.

On streets with lots below the roadway elevation, the lot grading shall incorporate a 1-foot high berm along the lower side of the road right-of-way, when the finished floor of a new building cannot be constructed 1-foot above the 100-year flood level on the street. Driveway ramps will be designed to maintain the 1-foot high of this berm across the driveway.

f. Storm runoff detention is recommended to maintain storm runoff to peak pre-development rates. Only dry detention basins will be accepted by the County for dedication. Additional capacity for sediment and debris shall be provided onsite.

Exemptions from the storm runoff detention requirement will be as follows:

1. Development or redevelopment of a total area of less than 2 acres.

2. Development of an area that drains into a drainageway which is capable of conveying the storm runoff based on the ultimate development of the drainage basin.

g. Best Management Practices: Best Management Practices shall be incorporated in construction activities and development to reduce the impacts of polluted runoff to the environment. The recommended erosion control and development measures are as follows:

1. Construction Activities
   i. Reduce erosion and to the extent practicable, retain sediment onsite during and after construction.

   ii. Erosion and Sedimentation Control Plan shall be developed.

iii. Erosion control recommendations:

- Schedule clearing or grading work during dry times of the year.
- Construct project in stages.
- Clear only areas essential for construction.
- Locate potential pollutant sources away from steep slopes, water bodies and critical areas.
- Keep the graded areas damp at all times.
- Provide dust screens along residential homes.
- Provide cutoff or interceptor channels to collect runoff and sediment.
- Revegetate graded sites within 30 calendar days after final grading is completed.
- Temporarily revegetate and/or provide soil stabilization to minimize erosion and dust problems at graded sites if grading will be suspended for 30 days or more.
- Provide stable base area where vehicles and equipment can be washed before leaving the project site.
- Limit grading area to 10-acre increments. The next increment shall not commence unless the earlier increment's grassing of the bared site is growing and there is an irrigation system and maintenance program to ensure that plant growth is fully established.

iv. Sediment Control Recommendations:

- Install temporary cutoff swales or ditches to direct storm flows and sediment to temporary sedimentation basin.
- Install temporary sediment and or permanent detention basin(s).
- Install filter fabric fences.

- Install gravel filter berms.
- Install inlet protection measures.
- Drain storm runoff through vegetated filter strips.

2. Development

   i. Detention basin shall have water quality control provisions.
   ii. Structural improvements of natural channels shall be avoided.
   iii. Storm runoff shall be discharged into natural grass watercourses.

These practices shall be incorporated into the drainage planning and design, and included in the Drainage Report and the construction drawings.

SECTION 4 - HYDROLOGY

4.1 Introduction: This Section describes the criteria and methodology for determining the storm runoff design peak flows and volumes to be used in the preparation of storm drainage studies, plans, and facility design hydrographs.

4.2 Rainfall: Design storms and time intensity frequency maps developed by the U.S. Weather Bureau in its Technical Paper No. 43, "Rainfall-Frequency Atlas of Hawaiian Islands" have been adopted by the County. All hydrologic analyses shall utilize the rainfall data presented herein for calculating storm runoff. Procedures for calculating storm runoff are presented as follows:

4.3 *Storm Event* Recurrence Interval: The recurrence interval shall be in accordance with Section 3 of this MANUAL.

4.4 Rational Method: The formula Q = CIA shall be used to determine quantities of flow rate, in which:

Q	=	flow rate, in cubic feet per second;
C	=	runoff coefficient;
I	=	rainfall intensity, in inches per hour for a duration equal to the time of concentration; and
A	=	drainage area, in acres.

The maximum limit of application of the Rational Method is 100 acres.

4.4.1 Runoff Coefficient: The runoff coefficient shall be determined from Table 1. The runoff coefficient is in accordance with the land uses described in the County's Comprehensive Zoning Ordinance and storm recurrence interval.

For drainage areas with more than one type of land use, a weighted value of the runoff coefficient shall be computed.

4.4.2 Time of Concentration and Travel Time, T_c: For overland sheet flow of less than 300 feet the T_C shall be determined using Plate 1, Watercourse Slope/Velocity Chart developed by the National Resource Conservation Service TR-55 for overland flow. The time of concentration shall be computed by the travel velocity using Plate 1 for overland flow. Travel time is the ratio of the flow length to flow velocity. Travel time in drainage channels and culverts shall be determined by computing travel velocity of the channel or culvert using Manning's Equation. For distinct consecutive flow areas, a summation of the T_C shall be used. Travel path, length, slope and type of flow shall be shown on the hydrologic map. Overland sheet flow will be allowed up to a maximum distance of 300 feet. Beyond 300 feet, flow is considered a shallow concentrated flow, and the grassed waterway line shall be used to estimate travel velocity.

4.4.3 Rainfall Intensity: The rainfall intensity of a drainage area shall be determined by the following procedure:

a. Select the appropriate 1-hour rainfall value from Plate 3 or 4 for the design recurrence interval. Design recurrence interval shall be as designated in Section 3 of this MANUAL.

b. Determine the Intensity Correction Factor by entering in the required time of concentration in Plate 2. Apply the correction factor to the 1-hour rainfall intensity to obtain the corrected rainfall intensity of the desired duration.

4.5 Natural Resources Conservation Service (NRCS) Hydrograph Analysis: The County has adopted the procedure for computing the peak flows and plotting hydrographs as outlined in the NRCS, National Engineering Handbook, Section 4, Hydrology, April 21, 1993 Supplement (Urban Hydrology for Small Watersheds [TR-55 Version 2.10] and computer program for Project Formulation Hydrology [TR-20 Version 2.04]), or latest revision thereof. For soil type information on Hawaii soils, the Erosion and Sediment Control Guide for Hawaii (NRCS, March 1981) or latest revision thereof, may be used along with the Soil Survey of Islands of Kauai, Oahu, Maui, Molokai and Lanai, State of Hawaii (NRCS, 1972). The NRCS computer program TR-55 or TR-20 with the Type I Rainfall Distribution for Hawaii may be used.

4.6 Storm Flow Analysis: The Engineer shall determine the predevelopment and post-development storm runoff from the project. Detention is required to keep storm runoff rates to predevelopment conditions for all subdivisions and developments larger than two (2) acres when the downstream drainageways do not have the capacity to handle flows from the ultimate development for the watershed.

The Engineer shall maintain predevelopment drainage flow patterns at the project boundaries. The diversion of storm runoff from one watershed to another shall be avoided. Additionally, the study on the diversion must document that the extended peak flows and runoff volumes will not present downstream drainage flood problems. Either NRCS TR-55 or TR-20 is recommended to be used to obtain the inflow and outflow hydrographs for the design of the detention basin(s).

SECTION 5 - DESIGN STANDARDS

5.1 General Conditions: The design and capacity of a drainage system shall be predicated on the following conditions:

a. On the basis of the runoff resulting from the design storm, the system shall convey surface runoff without damage to street facilities, structures or ground, and shall not result in reduced traffic speeds and operation on local streets. Flooding allowances shall be in accordance with Section 3 - Drainage Policy.

b. Runoff events exceeding the design storm must be conveyed with the minimum amount of damage to structures.

c. System must have maximum reliability of operation with minimum maintenance and upkeep requirements.

d. System must be adaptable to future expansion, if necessary, with minimum additional cost.

e. Where sump conditions exist a safety measure such as a designated overflow facility (overflow swale) shall be provided to prevent flooding of adjacent lots in the event the design capacity of the street drainage system is exceeded.

f. Roadway bridges shall be designed to pass the design runoff as open channel flow with a minimum freeboard as specified in the attached Plate 7 Freeboard Requirements. Multiple-span road crossings shall have minimum clear span of 30 feet, unless otherwise permitted by the County Engineer. Where possible, the roadway shall be designed to form a sag vertical curve with a low point at the drainageway, crossing with minimum grades to confine and control overflow at the crossing. Roadway bridges shall be designed to include only deck and roadway to meet the required roadway elevations. Fill material shall not be used to meet roadway elevations above the deck.

g. Outlets for closed conduits or drains emptying into open channels shall be designed to point downstream at an angle of 45° or less.

h. Subsurface drains shall be installed wherever recommended by the Engineer or where ground water is encountered or may be present during wet weather.

i. Drainage culverts and outlet structures shall be installed at the bottom of the hillside to prevent slope erosion below the outlet.

j. Lots abutting streams and open channels should be graded to drain towards the drainageway to the extent practicable, if lot grading is to be done.

5.2 Closed Conduits: Closed conduits are typically part of the local drainage system and are required when the other surface aspects of the local drainage system, roadside swales, and curb and gutter, do not have the capacity to maintain allowable flood limits.

5.2.1 Sizes and Gradients: The sizes and gradients will be determined by the Manning formula:

$$V = \frac{1.486}{n} R^{2/3} S^{1/2}$$

$$Q = AV$$

where;	V	=	flow velocity, in feet per second (ft/s)
	Q	=	flow rate, in cubic feet per second (cfs)
	A	=	flow area, in square feet (sq. ft.)
	R	=	hydraulic radius, in feet
			R = A/P, where P is the wetted perimeter, in feet
	S	=	slope of energy gradient, in feet per foot (ft/ft)
	n	=	Manning's roughness coefficient

Graphical solutions of this formula for circular pipes are found in Plates 8 to 16.

The following limitations for closed conduits shall apply:

a. Minimum size pipe: 18 inches nominal inside diameter

b. Minimum velocity: 2.5 feet per second (fps)

c. In general, pipe sizes shall not decrease in the direction of flow.

5.2.2 Materials and "n" Values: The following pipes are acceptable for storm drain construction, together with the roughness coefficients to be used in the solution of the Manning formula:

Material	n
Reinforced Concrete Pipe	0.013
Spiral Ribbed Aluminum Pipe*	0.012
Corrugated Metal Pipe*	0.024
High Density Polyethylene (HDPE) and Polyvinylchloride (PVC) Pipe**	0.012

*All metal pipes must have a service life of 40 to 50 years. Pipe ends need to be corrugated, and corrugated aluminum bands must be used for connections. Soil test chart for pH, resistivity and pipe service life shall be submitted.

**HDPE and PVC pipes shall be as specified by the Department of Transportation of the State of Hawaii

5.2.3 Loading:

5.2.3.1 Reinforced Concrete, *Spiral Ribbed Aluminum, High Density Polyethylene and Polyvinylchloride Pipe.*

a. Minimum pipe cover in County and private roadways, driveways and other areas with vehicular traffic (based on the current, "Standard Specification for Highway Bridges," AASHTO) shall be two feet.

   Should there be a need for a pipe cover of less than 2'-0", the Engineer shall submit a structural design for review and approval. The decision to allow such design will be made by the County Engineer.

b. Minimum pipe cover in easement areas without vehicular traffic shall be 1'-6".

c. Maximum permissible depth will be determined from current loading tables in pipe handbooks for the respective pipes.

d. All pipes shall be installed as specified in the Hawaii Standard Specifications for Roads, Bridge, and Public Works Construction.

e. Drain pipes installed along the longitudinal axis of the roadway shall be located within the roadway shoulder.

f. Pipe thickness and maximum cover shall conform to the pipe manufacturer's recommendations for the particular application.

g. Crushed rock shall be used for bedding and backfilling up to the top of HDPE and PVC pipes.

h. Sand bedding shall be used for spiral ribbed aluminum pipes.

5.2.3.2 Other Closed Conduits: Minimum cover for all other closed conduits shall be two feet. Structures shall be designed to support all loads at the planned depths.

5.2.4 Manholes and Inlets:

5.2.4.1 Manholes:

a. Location: Manholes shall be located at all changes in pipe size and changes in alignment or grade, and at all junction points.

b. Spacing: Maximum manhole spacing for pipes 36 inches or less in diameter, or box drains with the smallest dimension of 36 inches or less, shall be 250 feet, and manholes shall be kept within the right-of-way. Maximum manhole spacing for larger pipes and box drains shall be 500 feet.

c. Special Details: Bottoms of manholes and inlets serving as manholes shall be sloped to drain from inlet to outlet pipe as shown in the Standard Details for Public Works Construction, Department of Public Works, County of Kauai.

5.2.4.2 Inlets (Catch Basins): There are two types of inlets used in the County of Kauai: grate inlets and side opening inlets (catch basins). 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 throughout the inlet and, therefore, ponding does not occur at the inlet. The sump condition exists whenever water is restricted or ponds; i.e., the inlet is located at a low point. A sump condition can occur at a change in grade of the street from negative to positive, or at an intersection due to the crown slope of a cross street.

a. Location: Inlets are encouraged to be located at the upstream side of intersections, in sumps and where required by the street flooding limits of Section 3.3, Paragraph e.

b. Spacing: As required to meet allowable street flooding criteria.

c. Types:

1. Side Opening Catch Basins: For gutter grades up to 4%, standard 10-foot curb inlets with a depressed gutter shall be used. For grades 4% and greater, 10-foot long deflector inlets shall be used.

2. Grate Inlet Catch Basins: Standard single or double P-1- 7/8 - 4 grates shall be used.

5.2.4.3 Inlet Hydraulics: The procedure and basic data used to define the capacities of the standard inlets are presented on the Inlet Capacity Charts. The procedure consists of determining the theoretical flow interception by the inlet and the bypass flow. To account for the effects which decrease inlet capacity, such as debris plugging, pavement overlaying, and variations in design assumptions, inlet capacities in the Design Charts are reduced by 25 percent for sump conditions and 15 percent for all other conditions.

5.2.5 Pipe System Analysis: The pipe system shall be analyzed by sections from the lowest outlet to manhole, manhole to manhole, or manhole to inlet, and shall continue upstream. The design flow shall be used in determining the hydraulic gradient of the system. Full consideration of the tailwater, entrance control, outlet control and critical flow conditions shall be made.

a. Pipe flow characteristics consisting of normal depth and critical depth shall be shown for each section of culvert between manholes, catch basins and inlet and outlet structures for pipes flowing full or partially full. The principles of flow of water in closed conduits shall be used.

b. Flow shall be considered ponded at each manhole, catch basin and inlet and outlet structure. The ponded flow will have negligible velocity. Inlet and outlet headwater depth shall be determined at each manhole, catch basin and inlet and outlet structure using a square edged entrance control charts. See Plates 19 through 23.

5.2.5.1 Hydraulic Gradient Computations: The hydraulic grade line represents the water surface elevation of flow in a hydraulic system. It is: (1) a hypothetical line connecting points to which water will rise in manholes and junctions throughout the system during the design flow; or (2) the level of flowing water at any point along an open channel.

It shall be determined starting at the downstream end of the proposed drainage system and proceeding upstream by adding the friction losses and manhole losses of the system.

The hydraulic gradient for the design flow shall be at least one foot below the top of the manhole cover, or one foot below the top of grate or gutter flow-line grade.

a. Beginning Elevation: The elevation of the hydraulic gradient at the downstream end outlet shall be selected according to the following conditions:

1. Connection to existing drainage system - determined from the hydraulic gradient computations for the existing drain;

2. Discharge into a stream - determined from the flow conditions of the stream;

3. Submerged tailwater condition - begin at the tailwater elevation; and

4. Freefall condition (conduit) - begin at the crown of the proposed drain.

b. Friction Loss: Friction loss shall be determined based on the following equation:

$$h_f = S_f(L)$$

where:

h_f = head loss due to friction, in feet

S_f = friction slope from Manning's formula

$$S_f = \frac{(nv)^2}{2.208R^{4/3}}$$

L = length of pipe or channel, in feet

The friction loss shall be calculated for the design flow with the pipe flowing full or partially full.

c. Headwater: Each manhole, catch basin and junction structure shall be considered to be ponded at its upstream culvert inlet. Headwater shall be calculated based upon inlet or outlet control using a square edged entrance. The higher headwater shall be considered to be the ponded water surface elevation.

d. Hydraulic Gradient: The hydraulic gradient of drainage systems shall be determined by calculating the headwater elevation at each manhole, catch basin or inlet and outlet structures. Based upon the headwater elevation at these structures, the hydraulic gradient for sections of culverts between these structures will indicate full-flow or open channel flow conditions.

5.2.6 Special Details: The following structures shall be installed where required:

a. Headwalls, aprons and cut-off walls at drain inlets and outlets.

b. Energy dissipators at outlets when the velocity exceeds 5 feet per second unless the outlet discharges on to non-erosive ground such as solid rock, ***or*** rip-rap designed to resist high velocities.

c. Debris control structures.

d. Guardrails at headwalls and inlets, where the headwall or inlet presents a hazard to vehicular traffic or pedestrians.

e. Rock chutes or grade structures.

5.3 Open Channels: The channels in the County of Kauai are defined as either natural or manmade. Natural channels include all water courses that have occurred naturally by the erosion process. Manmade channels are those constructed or developed by human effort such as irrigation canals and flumes, roadside ditches or swales, streets, and lined channels. General criteria and guidelines for open channel flow are presented in this section as follows:

5.3.1 Channel Hydraulics: Use the Manning Formula to determine the required waterway areas where uniform flow can be assumed.

The channel total depth shall include design water depth and minimum freeboard allowances. Design water depth shall include the rise in water surface caused by curves and junctions.

For the natural channel/stream, the hydraulic gradient shall be determined by utilizing the standard-step backwater analysis method, HEC-2 software method, or sample calculation in Appendix A, taking into consideration losses due to changes in velocity of channel cross section, drops, waterway openings, or obstructions. The hydraulic gradient shall be shown on all applicable construction drawings.

Should the channel be required to accommodate the 100-year recurrence interval storm, a flood analysis shall be performed to determine the 100-year flow, based on the design 100-year, 24-hour recurrence interval storm (Plate 6) by the NRCS method and shown on all applicable construction drawings.

5.3.2 Permissible Velocities and "n" Values: The following is a list of "n" values for open channels and maximum permissible velocities. Maximum velocities shall be based upon design flow quantities.

Unlined Channels	Manning's "n"	Maximum Velocity (fps)
Rock	0.035	10
Ledge coral or lime stone	0.025	10
Earth with vegetation (grassed)	0.035	5

Lined Channels	Manning's "n"	Maximum Velocity (fps)
Conc., trowel finish	0.013	No limitation
Conc., smooth wood forms	0.015	No limitation
Gunite	0.020	20
Grouted riprap & Cement Rubble Masonry (CRM)	0.025	20
Conc., street curb and gutter	0.015	No limitation
Asphalt Paved Street	0.015	(Roadway grade limitation)

Velocities between 5 feet per second and 10 feet per second will be permitted in material such as cemented gravel, hard pan, or mud rock depending upon its hardness and resistance to scouring. Borings and material samples shall be submitted for evaluation before velocities exceeding 5 feet per second will be permitted.

5.3.3 Channel Lining:

a. All drainage channels shall be fully lined for maintenance purposes. Private unlined drainage channels will be allowed if integrated into a multi-purpose drainage facility under the control of the owner.

b. All fill sections shall be lined. The lining shall be continuous and include side slopes and invert, with appropriate cut-off walls.

c. Total depth of channel lining will include design water depth and freeboard.

d. Attention shall be given to construction details of linings such as thickness, reinforcement, expansion and construction joints, cut-off walls, water-tight joints, placement of reinforcement, etc. Where the channel discharges into streams or other channels outside of the limits of a development, velocity reducing and transition structures shall be constructed to minimize erosion and overtopping of banks and subsequent flooding of downstream areas.

e. At bends where channel velocities are supercritical, rectangular channels shall be used, unless otherwise permitted by the County Engineer.

5.3.4 Freeboard: In designing open channels, freeboard must be provided to allow for surface roughness, wave action, air bulking, and splash and spray. These phenomena depend on the energy content of the flow. For water flowing at velocity v feet per second and depth d feet, the kinetic energy per foot of width per second is equal to $(wvd)(v^2/2g) = wdv^3/2g$, where w is the unit weight of water (62.4 pounds per cubic foot).

This kinetic energy can be converted to potential energy to lift the water surface when flow is stopped or changing direction as a function of depth and velocity of flow. The U.S. Bureau of Reclamation has developed an empirical expression, equations 1 and 2, to express a reasonable indication of desirable freeboard in terms of depth and velocity as follows:

1. Freeboard, in feet $= 2.0 + 0.025v\sqrt[3]{d}$

for $(d > 5'$ and $d < d_c)$

2. *Freeboard, in feet $= 1.0 + 0.20d + 0.025v\sqrt[3]{d}$*

for $(d \geq 5'; 5' > d > d_c; V < 20$ fps) where v is the velocity in feet per second, d is the depth of flow in feet, and where d_c is defined as the critical depth in feet. For discharges less than 30 cfs, the channel shall be designed for 100% greater capacity than the design discharge. The velocity of flow can be computed by dividing the design discharge by the cross-sectional area of flow. For convenience of the application, equations 1 and 2 are shown graphically in Plate 7.

5.3.5 Junctions: Junctions shall be designed as much as possible to channel branch flows at a small angle (i.e.. less than 45 degrees) with the main flow direction to reduce velocity and momentum resistance components, deposition of debris, and erosion of banks.

5.3.6 Bends and Superelevation: Changes in direction of flow shall be made with smoothly curved channel walls allowing for superelevation in water surface. Curves will nearly always require additional depth. Trapezoidal channels for supercritical velocities are not recommended. Curve radii should be sufficiently great to limit superelevation of the water surface to one foot above computed depth of flow or at straight reach 10% of water surface width, whichever is the least. The amount of superelevation for simple curves may be determined as follows:

a. Trapezoidal Channel
   Subcritical velocity:

$$e = \frac{V^2(b + 2zd)}{(gR - 2zV^2)}$$

b. Rectangular Channel

Subcritical velocity:

$$e = \frac{V^2 b}{2gR}$$

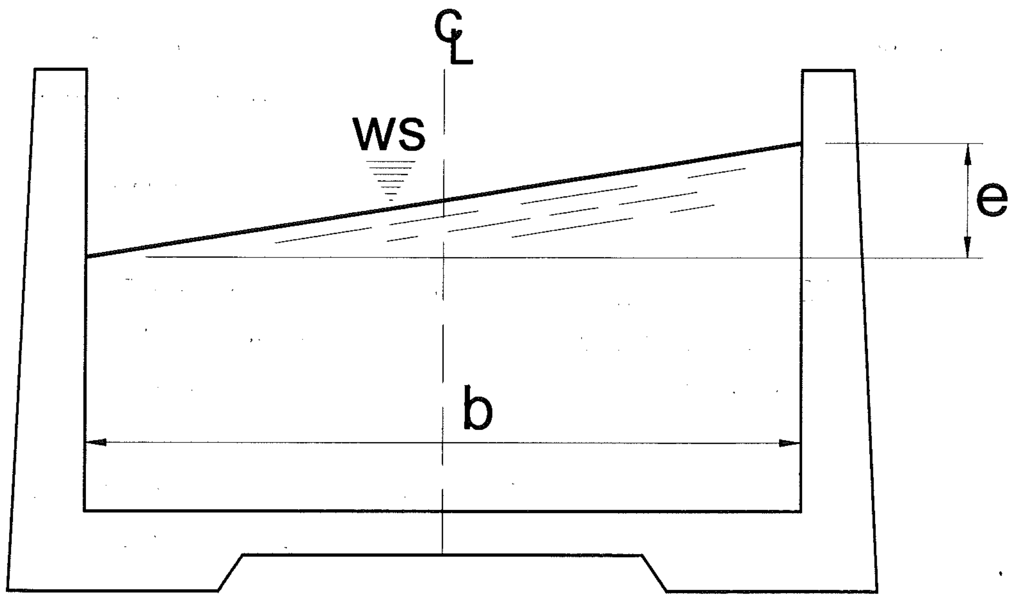
Supercritical velocity

$$e = \frac{V^2 b}{gR}$$

Supercritical velocity - compound curve:

$$e = \frac{V^2 b}{2gR}$$

The compound curve is a simple curve of radius R preceded and followed by sections of simple curves with radii of 2R and lengths of $\frac{b}{\tan \beta}$, where $\sin \beta = \frac{\sqrt{g d_m}}{V}$.



Water Surface Superelevation Showing "e"

Figure 5.1 Superelevation Within an Open Channel

Where:	e	=	maximum difference in elevation of water surface between channel sides, feet (ft)
	z	=	co-tangent of bank slope
	d	=	normal depth, feet (ft)
	b	=	channel bottom width, feet (ft)
	R	=	radius of curve to centerline, feet (ft)
	g	=	acceleration due to gravity, feet per second squared (fps^2)
	V	=	normal velocity, feet per second (fps)
	d_m	=	mean depth, feet (ft)

5.3.7 Transitions:

a. The maximum angle between channel centerline and transition walls should be 12.5 degrees.

b. Sharp angles in alignment of transition structure should be avoided.

5.4 Streets: Portions of urban and rural streets, specifically gutters and roadside ditches/swales, are part of the Local Drainage System. When drainage in the street exceeds allowable limits, a closed conduit system or open channel is required to convey the excess flows. The streets can also be a part of the Major Drainage System when they carry flows in excess of the minor storm (refer to Section 3.3.e).

Design criteria for the collection and movement of storm water on public streets are based on a reasonable frequency and magnitude of traffic interference. Depending on the character of the street, certain traffic lanes can be fully inundated during the minor design storm (2-year recurrence interval storm). However, during less frequent, more intense storms (any storm with a recurrence interval greater than the 2-year storm up to the 100-year storm), runoff may fully inundate traffic lanes as well as land adjacent to the roadway. Proper design will keep water levels below finish floor elevations of buildings or homes. The primary function of the streets for the Local Drainage System is to convey the nuisance flows quickly and efficiently to the closed conduit or open channel drainage system without significant interference to traffic movement. For the Major Drainage System, the function of the streets is to provide an emergency passageway for the flood flows with minimal damage to the development.

5.4.1 Hydraulic Computation: The allowable minor storm capacity of each curb and gutter street section shall be calculated using the modified Manning's formula:

$$Q = 0.56\left(\frac{Z}{n}\right)S^{1/2}d^{8/3}$$

where;	Q	=	discharge in cfs
	Z	=	inverse of cross slope ($1/S_X$), where S_X is the cross slope of the gutter and pavement (ft/ft)
	d	=	depth of water at face of curb (feet)
	S	=	longitudinal grade of street (ft/ft)
	n	=	Manning's roughness coefficient

The allowable street flow capacity was computed using a symmetrical street section in accordance with the "Standard Details for Public Works Construction," Department of Public Works, County of Kauai. The allowable street flow capacity for non-symmetrical streets shall be determined using the above equation. Street flow capacity calculations shall be submitted to the County for critical locations of the non-symmetrical streets. The computed street capacity must never exceed the allowable street capacity presented in the MANUAL.

For convenience, allowable street flow capacity for standard symmetrical grassed swale, and curb and gutter street cross sections are shown on the charts "Street Capacity (Half-Section)" (Plates 17 and 18).

5.5 Culverts: A culvert is defined as a conduit for the passage of surface drainage water under a highway, street, or embankment (except for detention outlets).

5.5.1 Culvert Hydraulics: The procedures and basic data to be used for the hydraulic evaluation of culverts shall be in accordance with Plates 19 through 23.

5.5.2 Inlet and Outlet Configuration: All culverts are to be designed with headwalls and wingwalls, or with flared-end sections at the inlet and outlet. Flared-end sections will only be allowed on pipes with diameters of 42 inches (or equivalent) or less.

Headwalls, wingwalls, and flared-end sections should be designed and constructed to use the existing land forms of the site and blend with the natural landscape. Additional protection in the form of riprap will also be required at the inlet and outlet due to the potential scouring velocities.

5.5.3 Velocity Considerations: In the design of culverts, both minimum and maximum velocities must be considered. A minimum velocity of 2.5 feet per second shall be provided to assure a self-cleansing action of the culvert.

The maximum velocity is dictated by the channel conditions at the outlet. If the outlet velocities are less than 5 fps for grassed channels, then the minimum amount of protection is required due to eddy currents generated by the flow transition. Higher outlet velocities will require substantially more protection.

5.6 Hydraulic Structures: The following structures shall be installed where required. These structures should be designed based on site specific situations. It is therefore recommended that reference be made to reliable published documents for their design procedures.

a. Energy Dissipators: Energy dissipators shall be used to dissipate energy where necessary, and to transition the flow from a lined channel to normal flow in an unlined channel. Recommended energy dissipators include either: impact stilling basins or an outlet structure with baffle blocks. Engineers can use discretion and initiative to select different alternatives provided that supporting data and documentation are submitted. Examples of energy dissipators are shown in Appendix A.

b. Debris barriers: Debris barriers should be provided upstream of an intake to prevent clogging. Where required, boulder basins shall be provided upstream of the debris barrier.

c. Vertical drop structures

d. Bridges and spillways

e. Outlet streambed protection (bed lining)

f. Headwalls, aprons and cutoff walls at drain inlets and outlets

g. Guardrails at headwalls and inlets, where the headwalls and inlets represent a hazard to vehicular traffic or pedestrians.

5.7 Detention: Detention basins shall be constructed where required to control discharges to downstream areas. The allowable discharge for developed conditions shall be equal to or less than the peak discharge under predeveloped conditions. Generally, detention facilities will not reduce the total volume of runoff, but simply redistribute the volume of runoff over a longer period of time.

a. Detention basins intended to continuously hold water are considered to be wet retention basins. Wet retention basins are not acceptable for dedication to the County.

b. Detention basins and reservoirs must be approved by State of Hawaii, Department of Land and Natural Resources and comply with the Department of Land and Natural Resources, Dam Safety Requirements, if volume is greater than 15 acre-feet, and dam height is greater than 25 feet, or if the storage volume is greater than 50 acre-feet and height is greater than 6 feet. No application is required for volumes less than 15 acre-feet and heights less than 6 feet. All detention basins shall be approved by the County Engineer.

c. Design Criteria:

1. Detention basins shall be sized using the 2-year, 24-hour recurrence interval rainfall for local drainage systems and the 100-year, 24-hour recurrence interval rainfall for major drainage systems.

2. Primary and emergency overflow spillway shall pass the 100-year stormwater discharge without causing overflows at any location which is not intended to receive such flow.

3. Outlet Releasing Capacity: Maximum discharge shall be controlled to predeveloped rates or less. Detained runoff volume must be discharged slowly over a reasonable period after the storm event.

4. Embankment Barrier Top Width: Minimum 14 foot width.

5. Embankment Slope: Interior side not steeper than 3:1
Exterior side not steeper than 2:1

6. Freeboard: Minimum 1 foot above major stormwater surface elevation.

7. Maintenance: A 10-foot wide access road to a ramp leading to the bottom of the detention basin shall be provided for maintenance.

8. Detention basins will not be included as land area that is required for park dedication.

9. Detention basins that will have a total depth of 24 inches or less for the 2-year, 24-hour recurrence interval storm can be dedicated to the County. Detention basins with water depths greater than 24 inches for the for the 2-year, 24-hour recurrence interval storm shall be fenced and remain a private facility. An agreement that assures perpetual maintenance of private basins will be required for privately owned detention basins.

10. The developer/designer is encouraged to utilize the detention basin as a multi-purpose facility (i.e. park or playfield) for the 100-year, 24-hour recurrence interval storm.

11. Detention Basin Report: A detention basin report shall be provided for review and approval by the County Engineer. The report shall include:

    Inflow hydrograph
    Stage, storage and outflow data
    Flood routing calculations

    Procedures: The report shall include maps, sketches, graphs, calculations and technical data regarding the following:

    i. Inflow Hydrograph (pre-development and post development)
        Runoff Curve Number
        Time of Concentration
        Rainfall

ii. Stage, Storage and Outflow
Depth versus Storage
Depth versus Outflow
Outlet Structure Characteristics

iii. Hydrograph Routing
Storage Indication Curve
Routing Form
Overflow Provisions

A sample detention basin calculation is provided in Appendix A.

5.8 **Water Quality**: Water quality protection shall be provided during construction with temporary sedimentation basins or flow-through based water quality control measures. Flow-through measures include infiltration, vegetated swales, bioretention filters and other types of filters.

Temporary sediment basin(s) will be provided for sites greater than 2 acres, to provide for the storage of 0.5 inch of sediment per acre of land to be cleared and grubbed or graded. Outflows from the sediment basin shall utilize methods to control sediment discharge approved by the County Engineer.

Water quality protection shall also be provided when detention basins are installed for controlling storm water discharge and pollutant control after the project has been developed. Permanent erosion and sediment control will be required for developments greater than 2 acres. Permanent sediment basin(s) or other erosion and sediment control facilities will be required to store 0.5 inch of sediment per acre of impervious surfaces, per storm event. These facilities must be maintained after storm events so that the required erosion and sediment control capacity is available for the next storm event. The detention basin may also serve as temporary sedimentation basins, provided that the required sediment volume is included in the basin design.

If a detention basin is required for the project, it shall also be used for water quality control. The water quality control portion of the detention basin shall be sized for a runoff volume that is calculated by one-half inch of runoff per acre that is made impermeable by the development, and methods utilized to control sediment discharge shall be approved by the County Engineer. The water quality control portion of the detention basin should be designed to drain in a 4-hour time period or longer after the storm event.

Design Plates & Tables

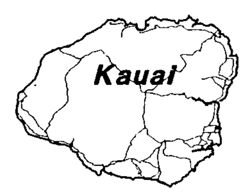


Table 1

TYPICAL RUNOFF COEFFICIENTS FOR BUILT-UP AREAS

LAND USE OR SURFACE CHARACTERISTICS	AVERAGE* PERCENT IMPERVIOUS	STORM FREQUENCY "C" 2	100
Business:			
General Commercial	90	0.82	0.84
Neighborhood Commercial	70	0.60	0.80
Residential:			
R-1	10	0.20	0.40
R-2	20	0.38	0.55
R-4	50	0.43	0.70
R-6	50	0.45	0.75
R-10	50	0.50	0.80
R-20	50	0.55	0.80
5 Acre Lot	8	0.15	0.30
Industrial:			
Limited Industrial	80	0.71	0.82
General Industrial	90	0.80	0.90
Parks, Cemeteries:	7	0.10	0.45
Playgrounds:	13	0.15	0.50
Schools:	50	0.45	0.70
Streets:			
Paved	100	0.87	0.93
Unpaved	95	0.80	0.90
Driveways and Walks:	96	0.87	0.93
Roofs:	90	0.80	0.90
Lawns, Sandy Soil:	0	0.00	0.20
Lawns, Clayey Soil:	0	0.05	0.50

NOTE: (These Rational formula coefficients may not be valid for large basins. These coefficients are also average values and may require adjustments depending on the surface characteristics, soil type, slope, infiltration, evaporation, depression storage, etc. The Engineer shall use sound engineering judgement in selecting the proper coefficient(s).) For composite drainage areas compute "weighted" Rational formula coefficient(s).

*** Average impervious areas do not correlate directly to allowable impervious area.**

Table 2 - Drain Inlet Capacity for Sump Conditions

Q' (Inlet Flow capacity) =
3 x (Grate Perimeter in ft.) x (Max depth from crown of street in ft.)$^{(3/2)}$

***Calculation Note: Q' values shown were reduced by 25% to account for clogging.**

Inlet Type	Grate Perimeter (ft.)
G2	9.9
G3	15.8
G4	13.1

Roadway Class	Max. Depth from Crown of Street (ft.)
Collector	0.723
Minor	0.583
Dead-End	0.533

Roadway Classification	Inlet Type	Maximum Inlet Capacity (Q') in cfs
Collector	G2	13.7
Collector	G3	21.9
Collector	G4	18.1
Minor	G2	9.9
Minor	G3	15.8
Minor	G4	13.1
Dead-End	G2	8.7
Dead-End	G3	13.8
Dead-End	G4	11.5

Table 3 - Catch Basin (CB) Capacity for Sump Condition

Q_i (Inlet Flow Capacity) = C x h x L x (2 x g x d_0) $^{0.5}$

C = Constant = 0.67
h = Orafice throat width = 0.4167 ft.
L = Length of throat
g = Acceleration due to gravity =32.16 ft/(sec^2)
d_0 = Effective head on Center of Orafice Throat = d_i - (h/2)sin (79.27)
d_i = Depth at lip of curb opening

***Calculation Note: Q_i values shown were reduced by 25% to account for clogging.**

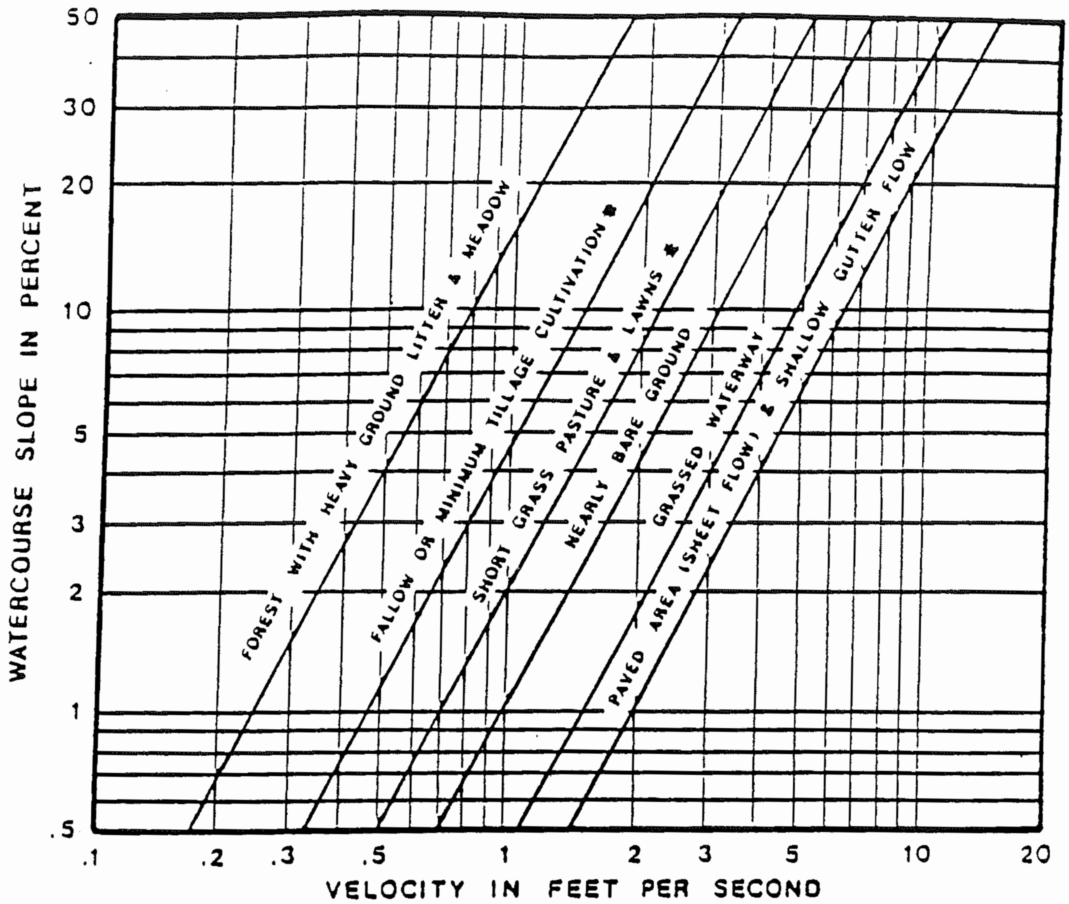
Roadway Class	d_i (ft.)	d_0 (ft.)
Collector	0.39	0.53
Minor	0.27	0.41
Dead-End	0.23	0.37

CB Type	Length (L) in feet
A	10.00
B	10.00
C	10.00
D	3.50
E	16
F	16.5

Roadway Classification	CB Type	Maximum Inlet Capacity (Q_i) in cfs
Collector	A	12.2
Collector	B	12.2
Collector	C	12.2
Collector	D	4.3
Collector	E	19.6
Collector	F	20.2
Minor	A	10.8
Minor	B	10.8
Minor	C	10.8

Roadway Classification	CB Type	Maximum Inlet Capacity (Q_i) in cfs
Minor	D	3.8
Minor	E	17.2
Minor	F	17.7
Dead-End	A	10.2
Dead-End	B	10.2
Dead-End	C	10.2
Dead-End	D	3.6
Dead-End	E	16.3
Dead-End	F	16.9

Plate 1



ESTIMATE OF AVERAGE FLOW VELOCITY FOR USE WITH THE RATIONAL FORMULA.

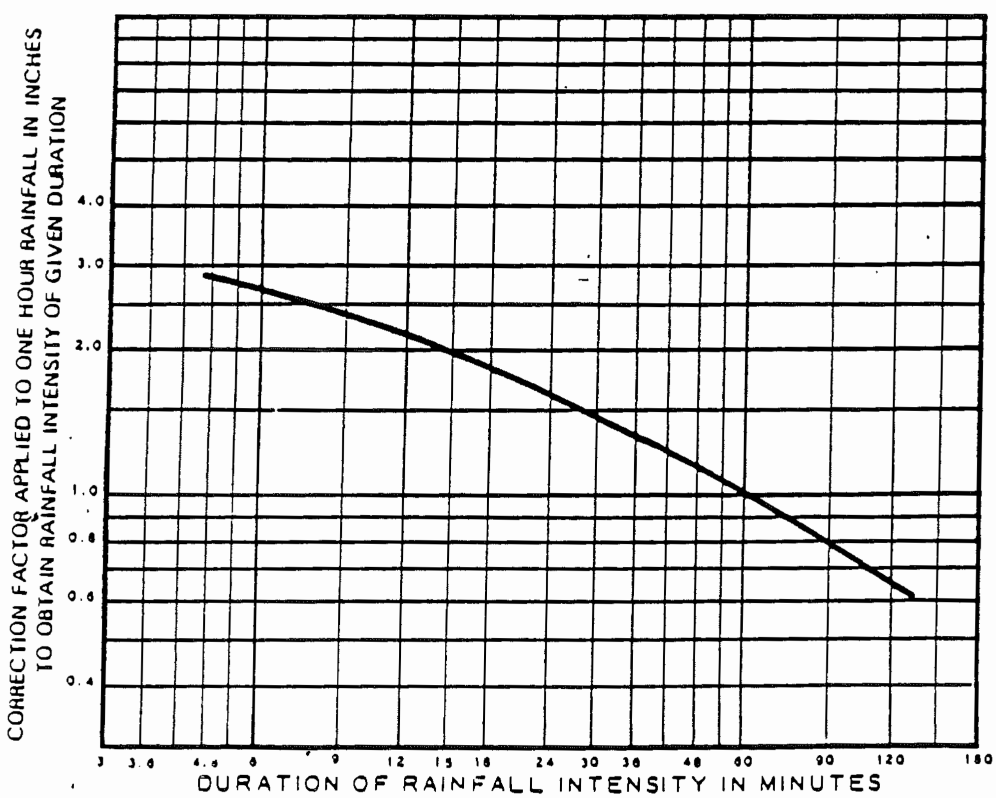


Plate 2

CORRECTION FACTOR

FOR CONVERTING 1 HR. RAINFALL TO RAINFALL INTENSITY OF VARIOUS DURATIONS

TO BE USED FOR AREA LESS THAN 100 ACRES

Plate 3

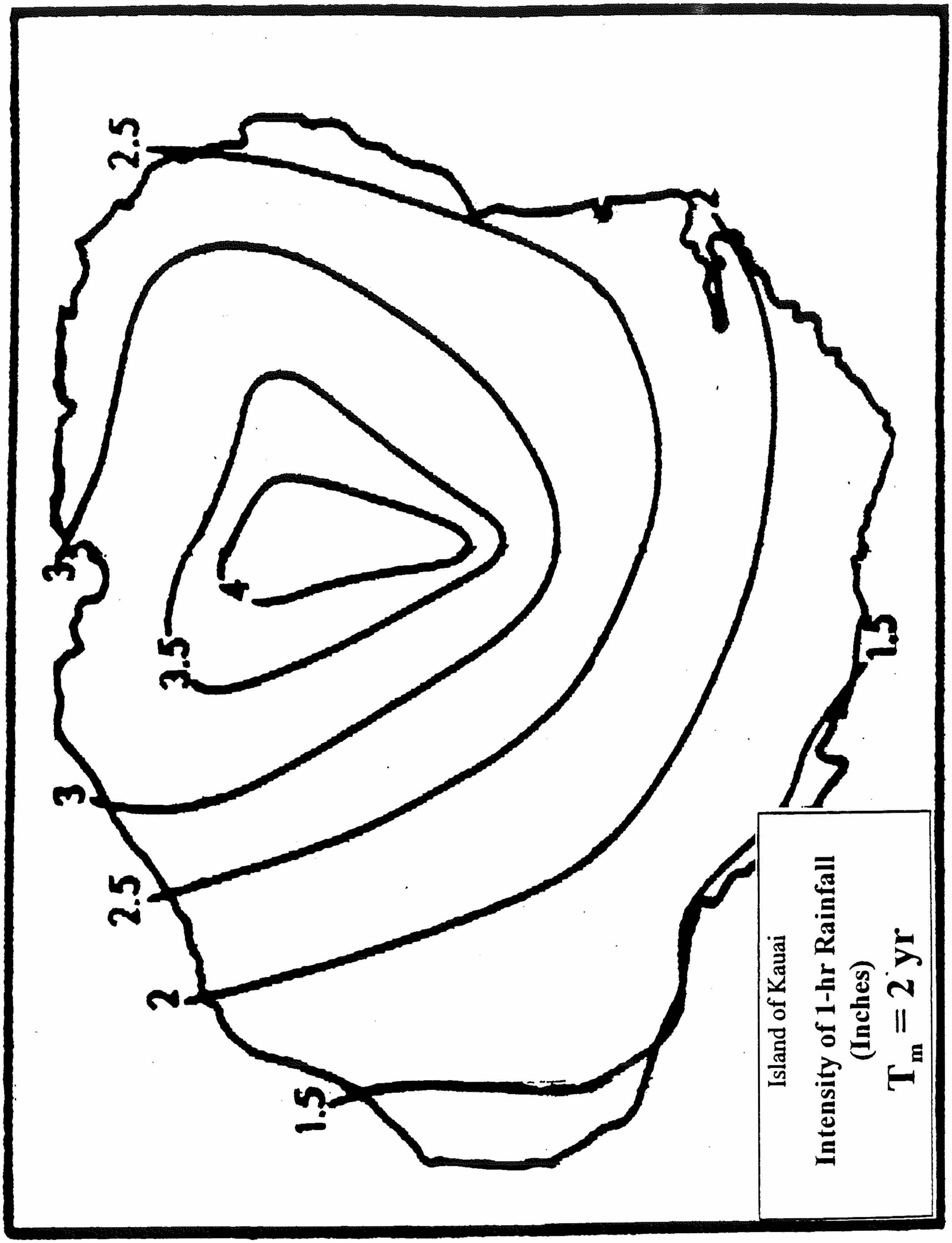
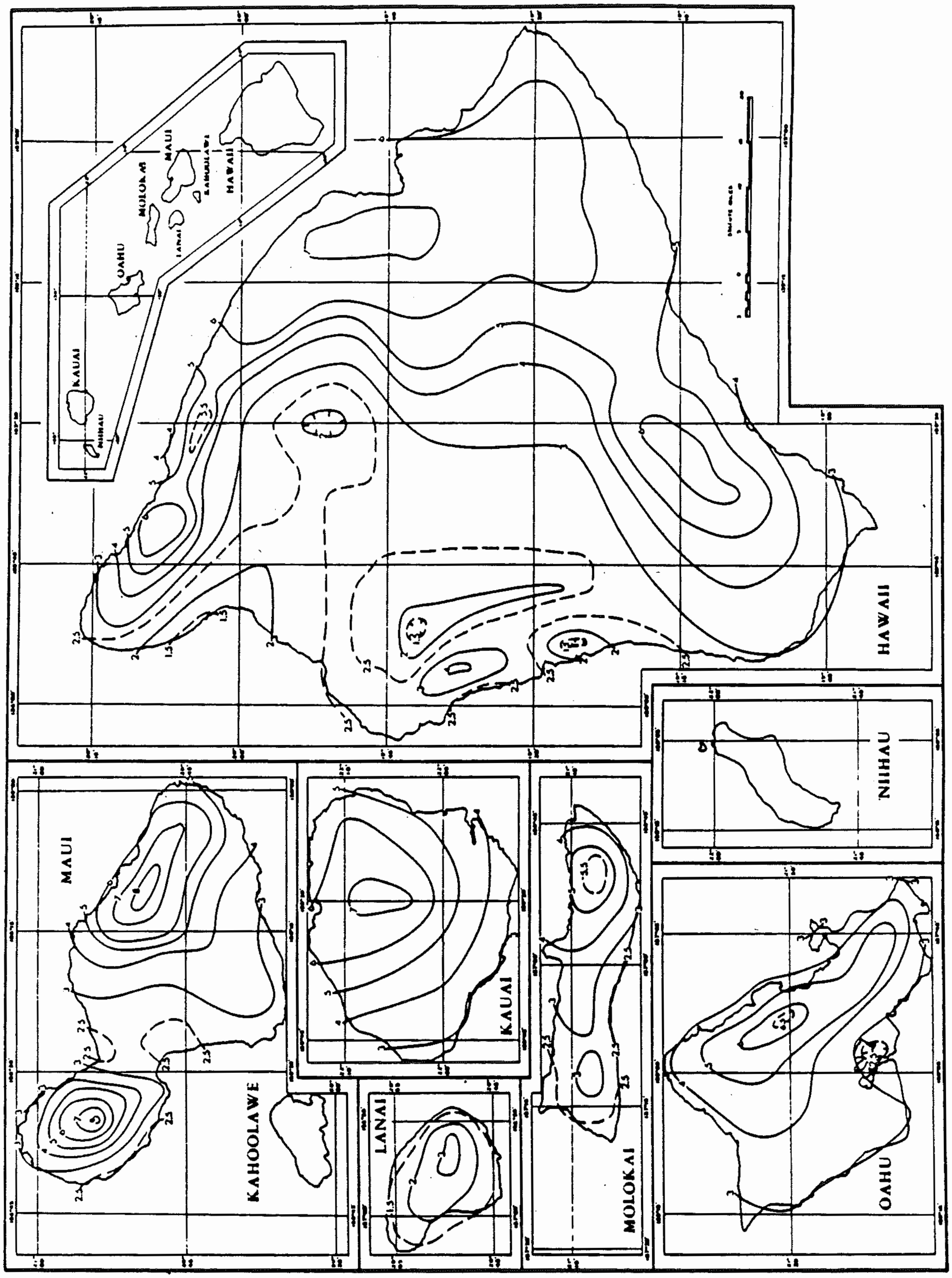
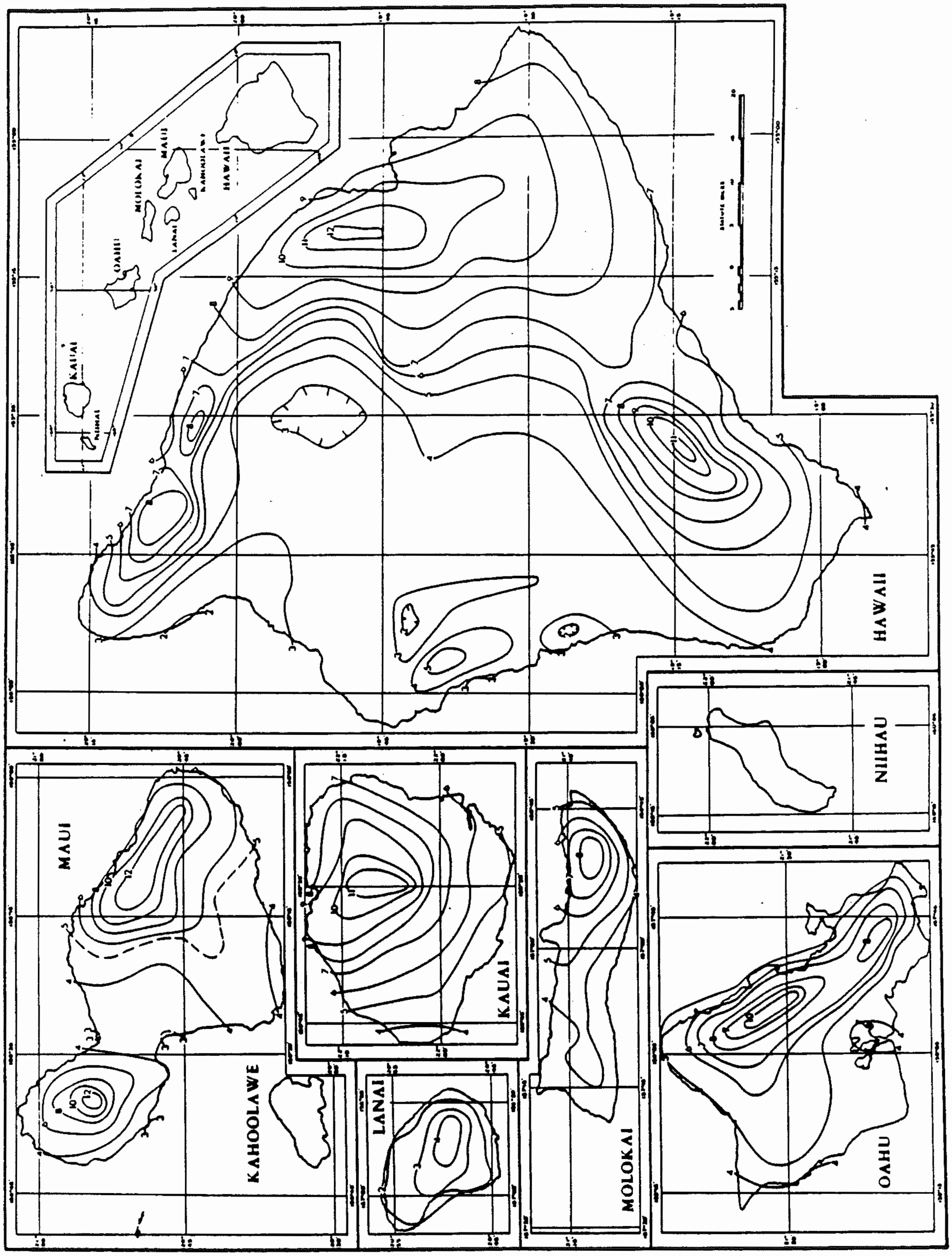

Island of Kauai
Intensity of 1-hr Rainfall
(Inches)
$T_m = 2$ yr
2.5
3.5
3
2.5
2
1.5
3.5
4
1.5

Plate 4



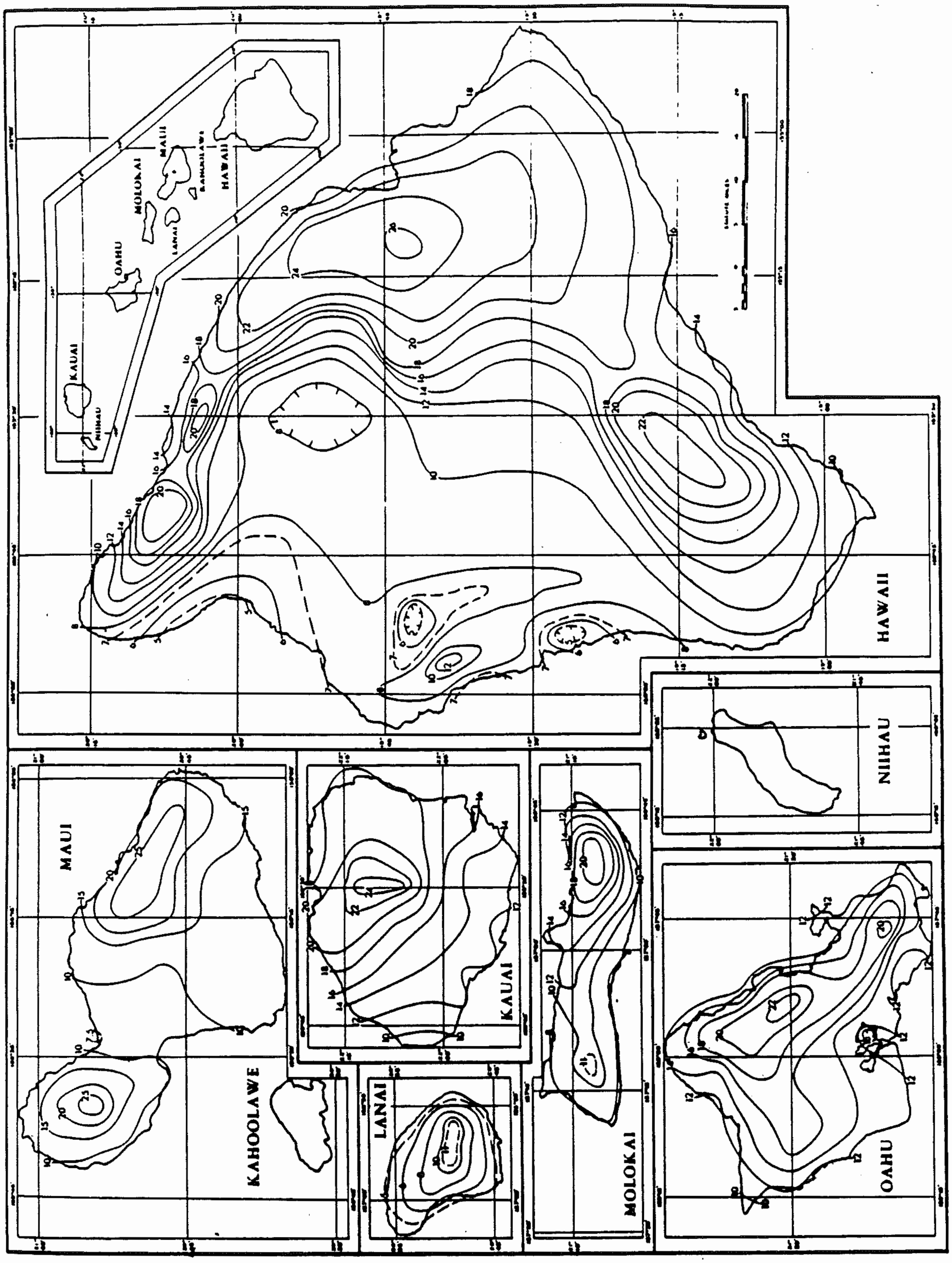
100–Year 1–Hour Rainfall (in.)

Plate 5



2-Year 24-Hour Rainfall (in.)

Plate 6



100-Year 24-Hour Rainfall (in.)

FREEBOARD ALLOWANCES　　Plate 7

FREEBOARD IN FEET:

1. $2.0 + 0.025\ V\ \sqrt[3]{d}$

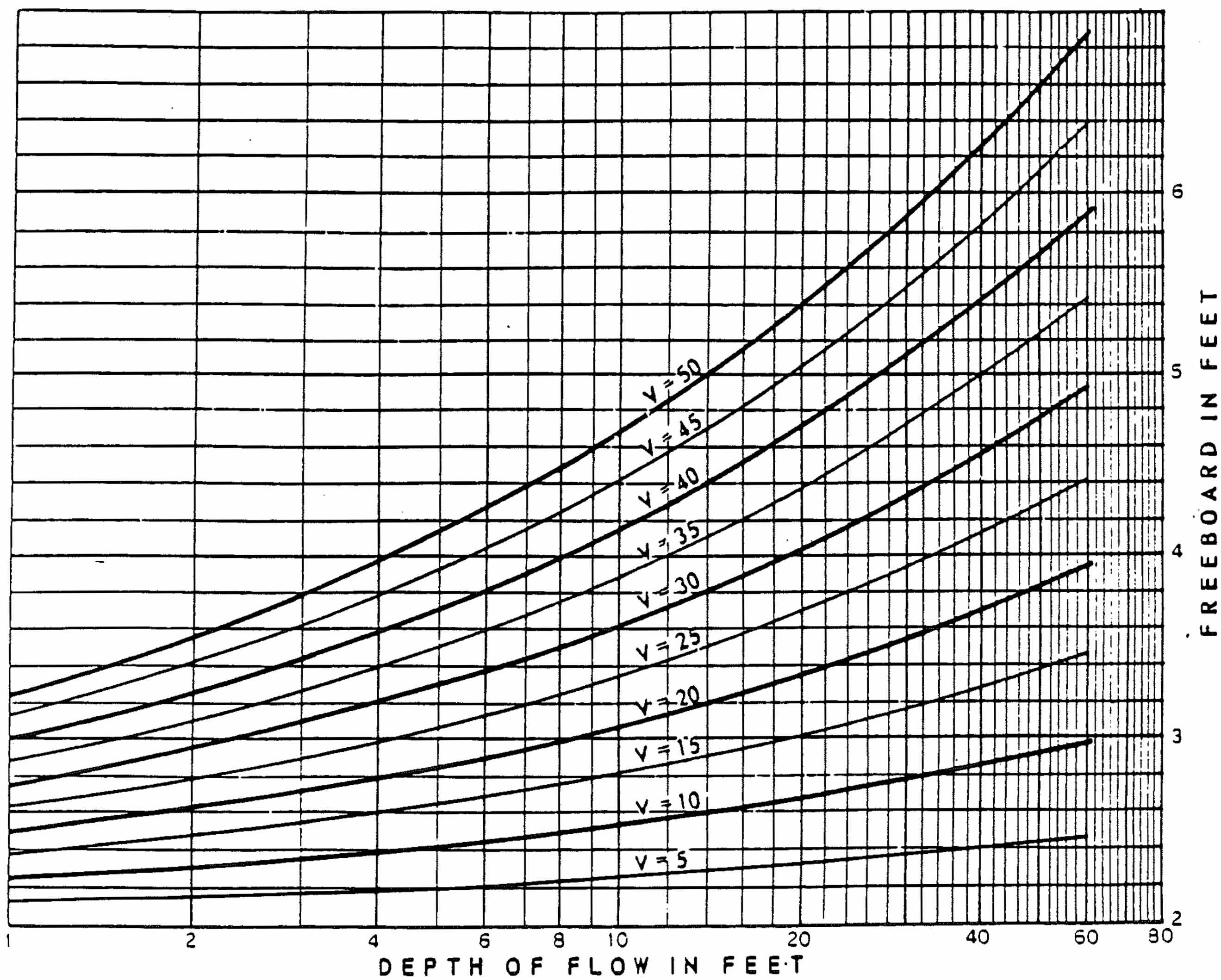
2. $d \geqq 5'$; $5' > d > d_c$; $V < 20$

   $1.0 + 0.20d + 0.025\ V\ \sqrt[3]{d}$

Where V = Velocity, in feet per second

d = Depth of flow, in feet

NOTE: For discharges less than 30 cfs, channel shall be designed for 100% greater capacity than the design discharge.



Pipe Flow Charts

The following pipe flow charts have been derived by the *U.S. Public Roads Administration, Division Two, Washington, D.C.* These charts are designed to enable direct solution of the Manning formula for circular pipes flowing full and for uniform part-full flow in circular pipes. The "n" scales of 0.013 and 0.024 have been inserted to facilitate the use of these charts for storm drainage systems in Honolulu. The following examples will help to explain the use of the pipe flow charts.

EXAMPLES

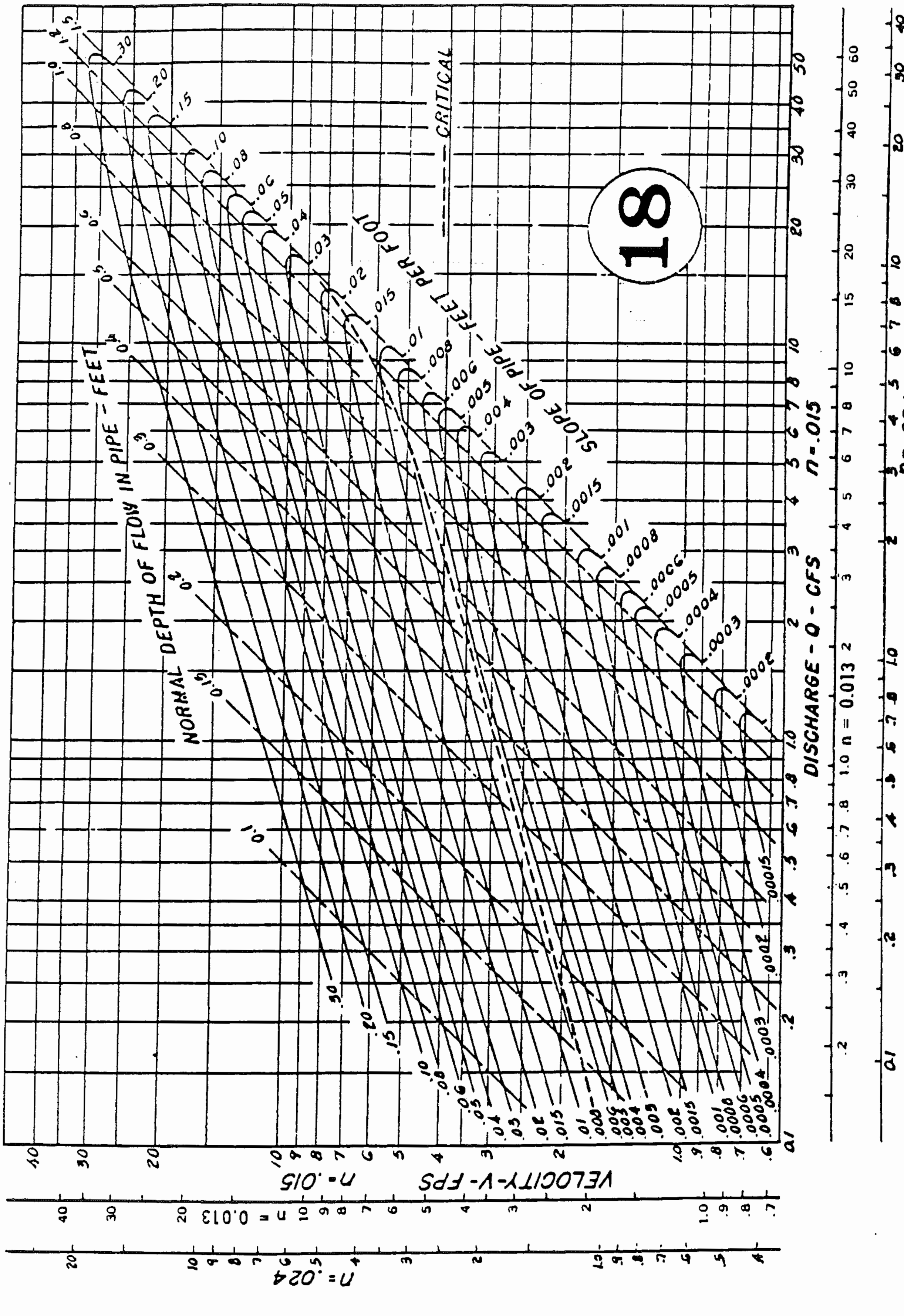
A. Determine the depth and velocity of flow in a long 30-inch pipe, n = 0.013, on a 0.5-percent slope (S_o = 0.005) discharging 25 cfs. Enter the 30-inch diameter chart at Q = 25 on n = 0.013 scale, follow up to intersection with line for slope S_o = 0.005, and read normal depth d_n = 1.8 feet and normal velocity V = 6.7 fps.

To find critical depth, enter chart Q = 25 on n = 0.015 scale, and read critical depth d_c = 1.7 feet at intersection with dotted critical curve. Also critical velocity V_c = 7.0 fps. (Note: Critical depth and velocity would be the same, regardless of pipe roughness).

B. Determine friction slope for a 30-inch corrugated metal pipe, n = 0.024, on a slope S_o = 0.008 ft/ft with a discharge Q = 25 cfs. Enter the 30-inch diameter chart at Q = 25 on n = 0.024 scale. Note that this ordinate falls to the right of the 0.008 slope line, therefore, the pipe will flow full. Read friction slope S_f = 0.012 at the line for depth equal to pipe diameter.

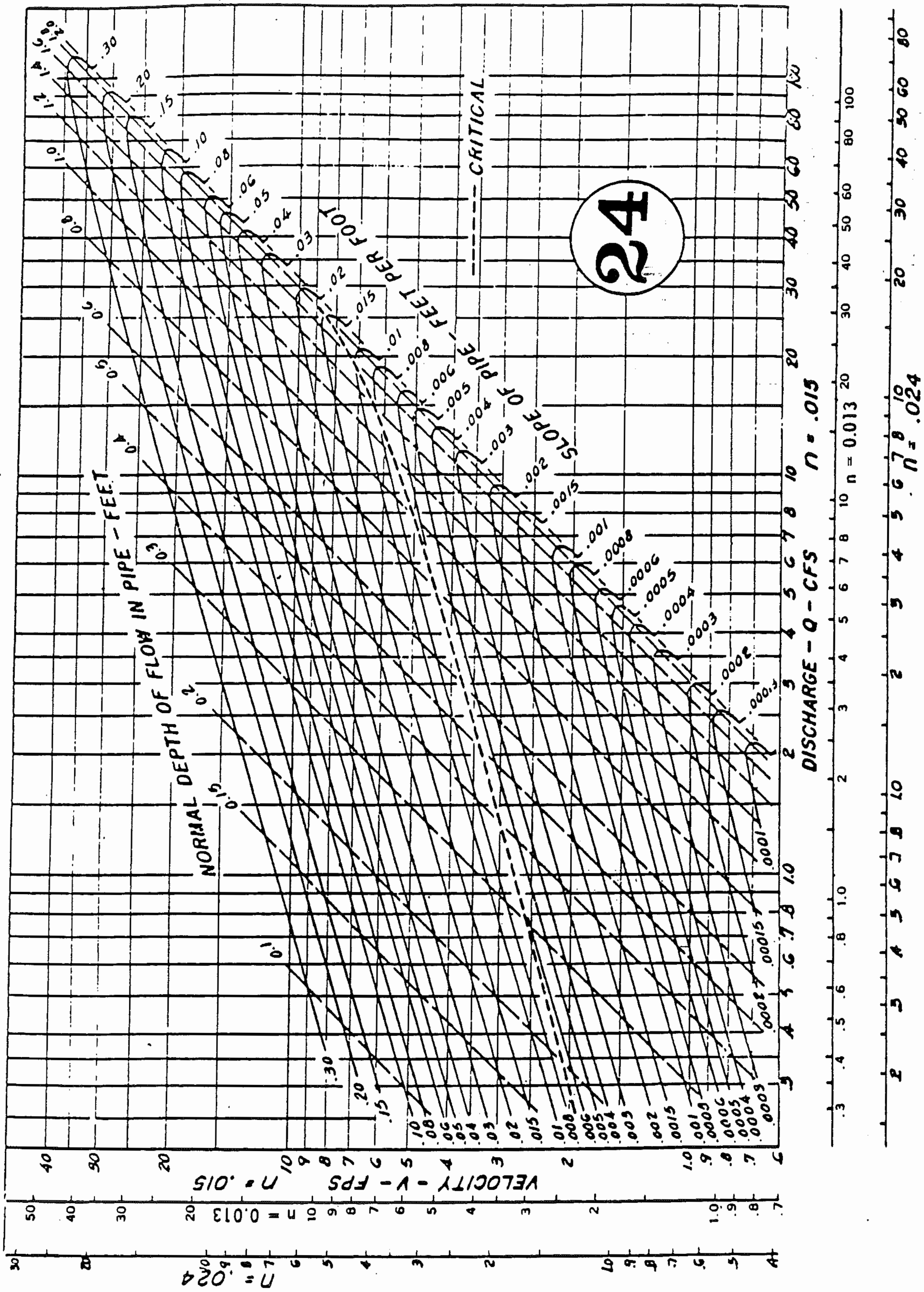
(Note: $Q = 25 \times \frac{0.024}{0.015} = 40$ cfs on the Q-scale for n = 0.015.)

Plate 8



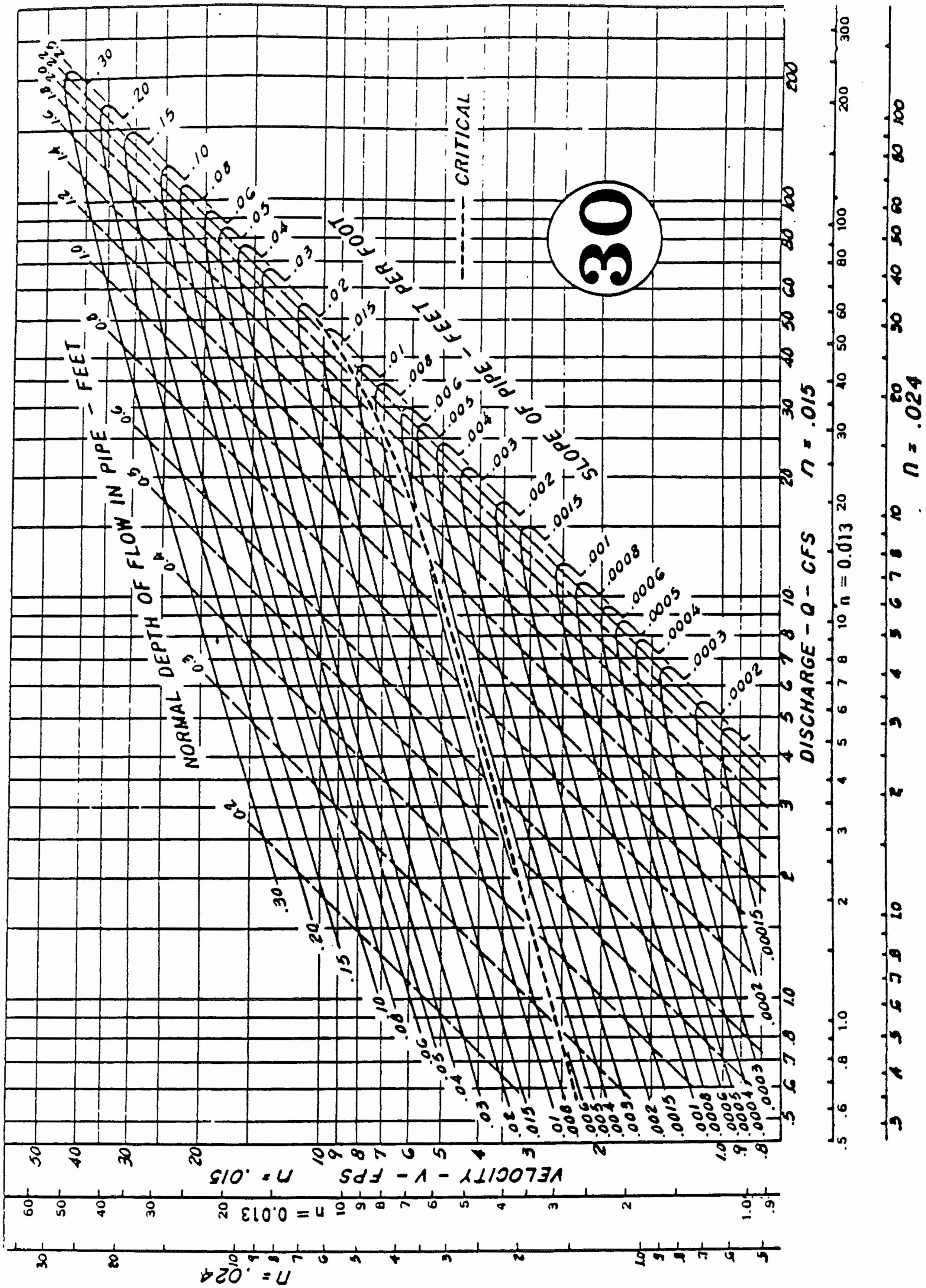
Pipe Flow Chart **18** inch Diameter

Plate 9



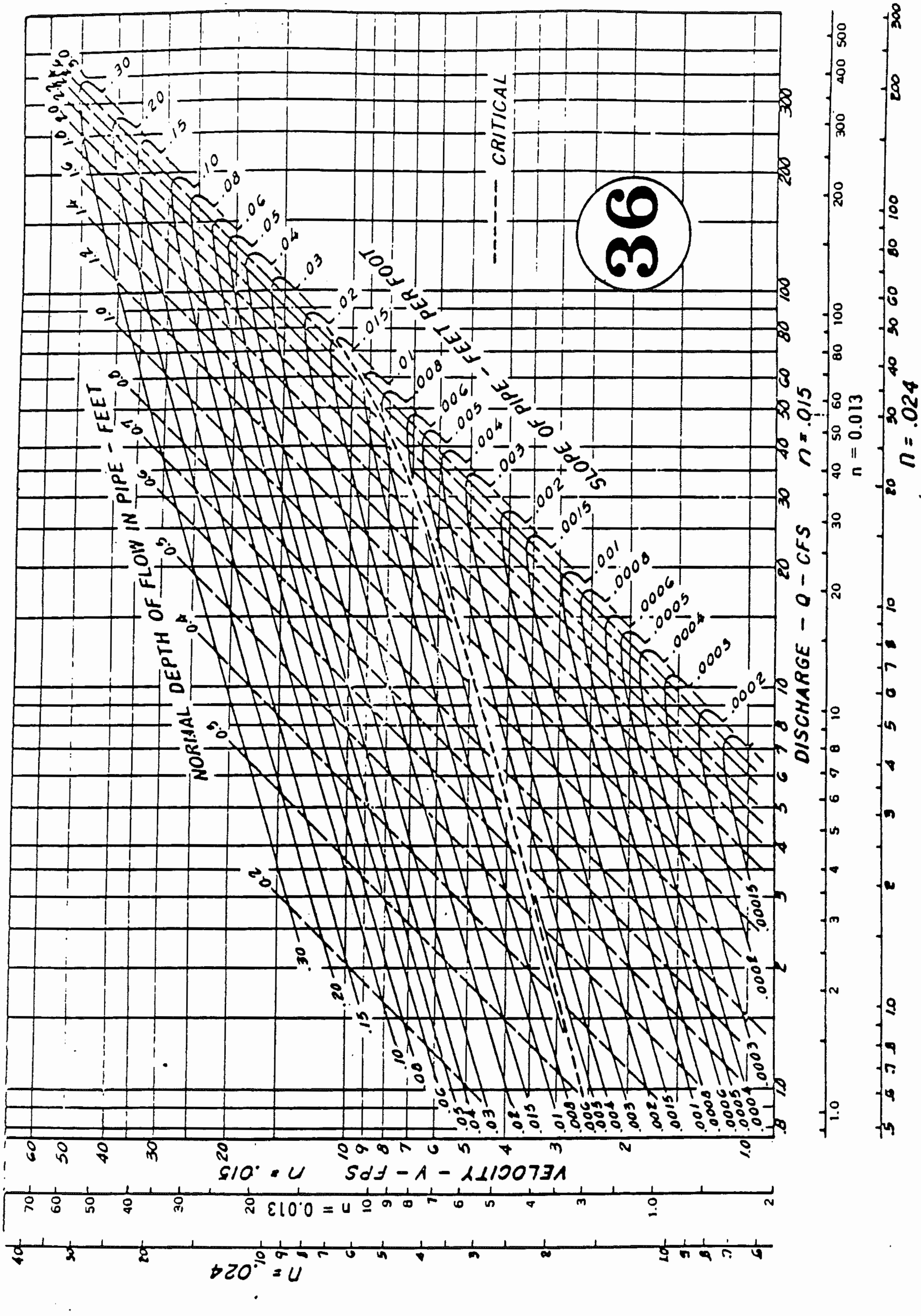
Pipe Flow Chart 24 inch Diameter

Plate 10



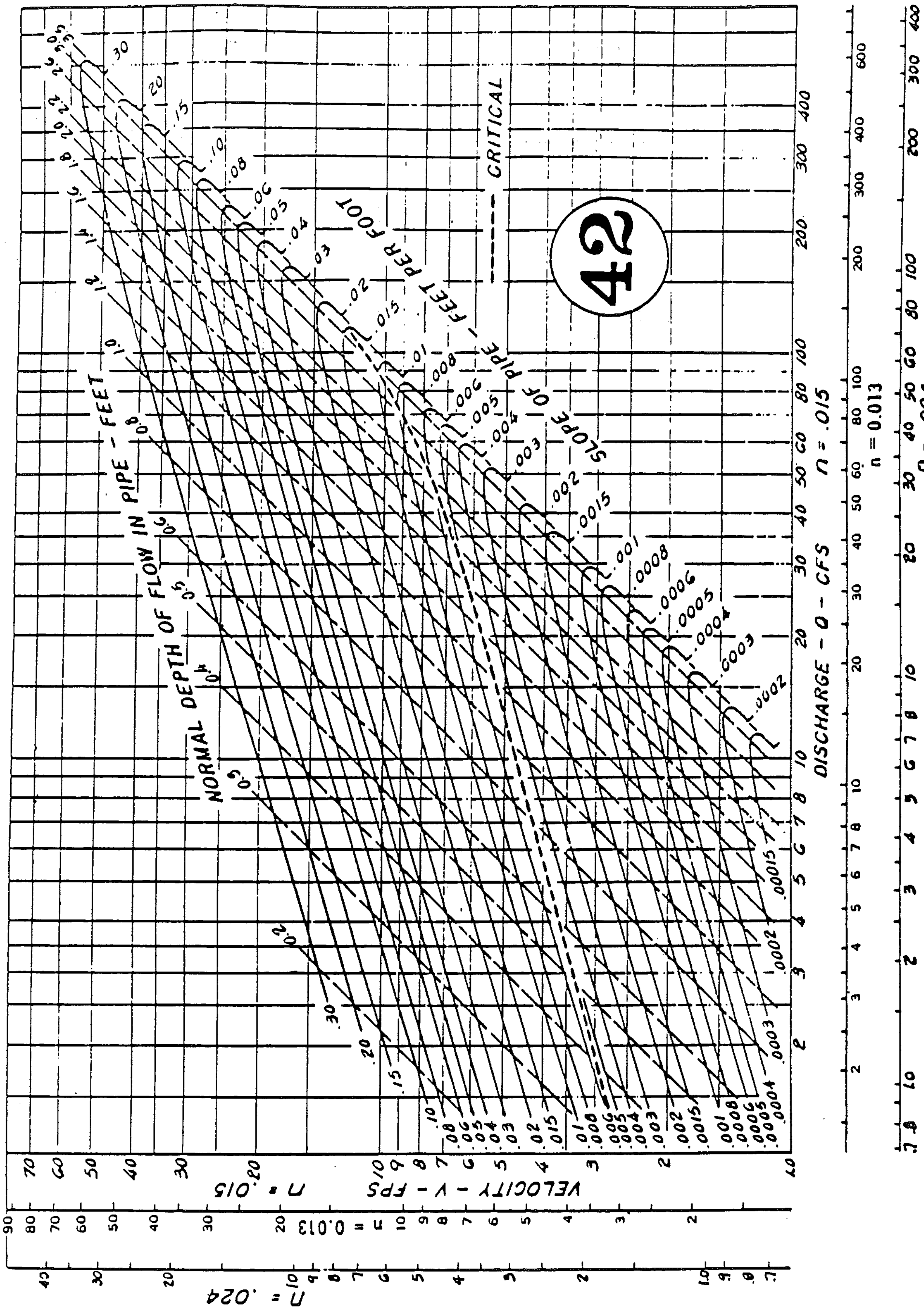
Pipe Flow Chart 30 inch Diameter

Plate 11



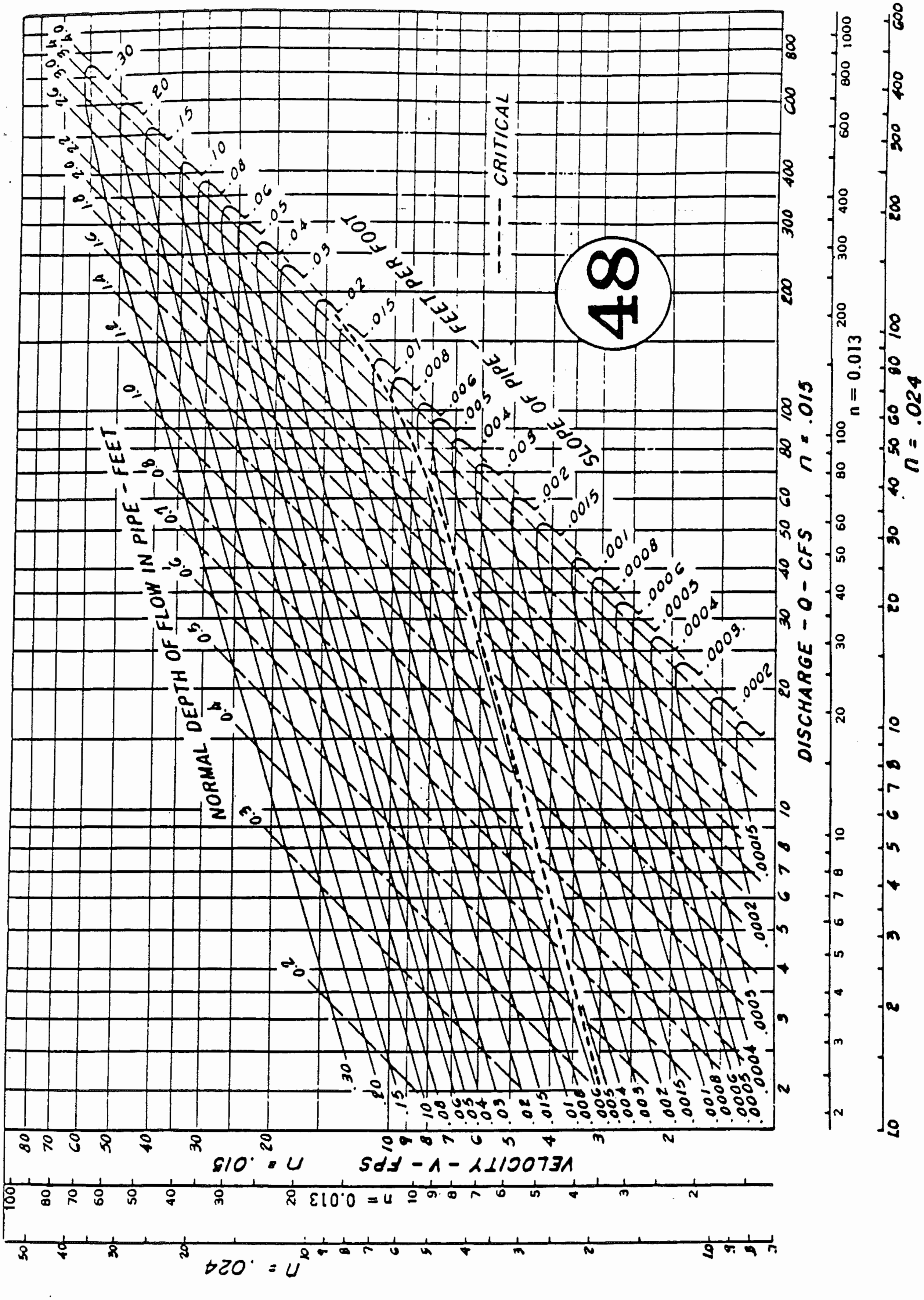
Pipe Flow Chart 36 inch Diameter

Plate 12

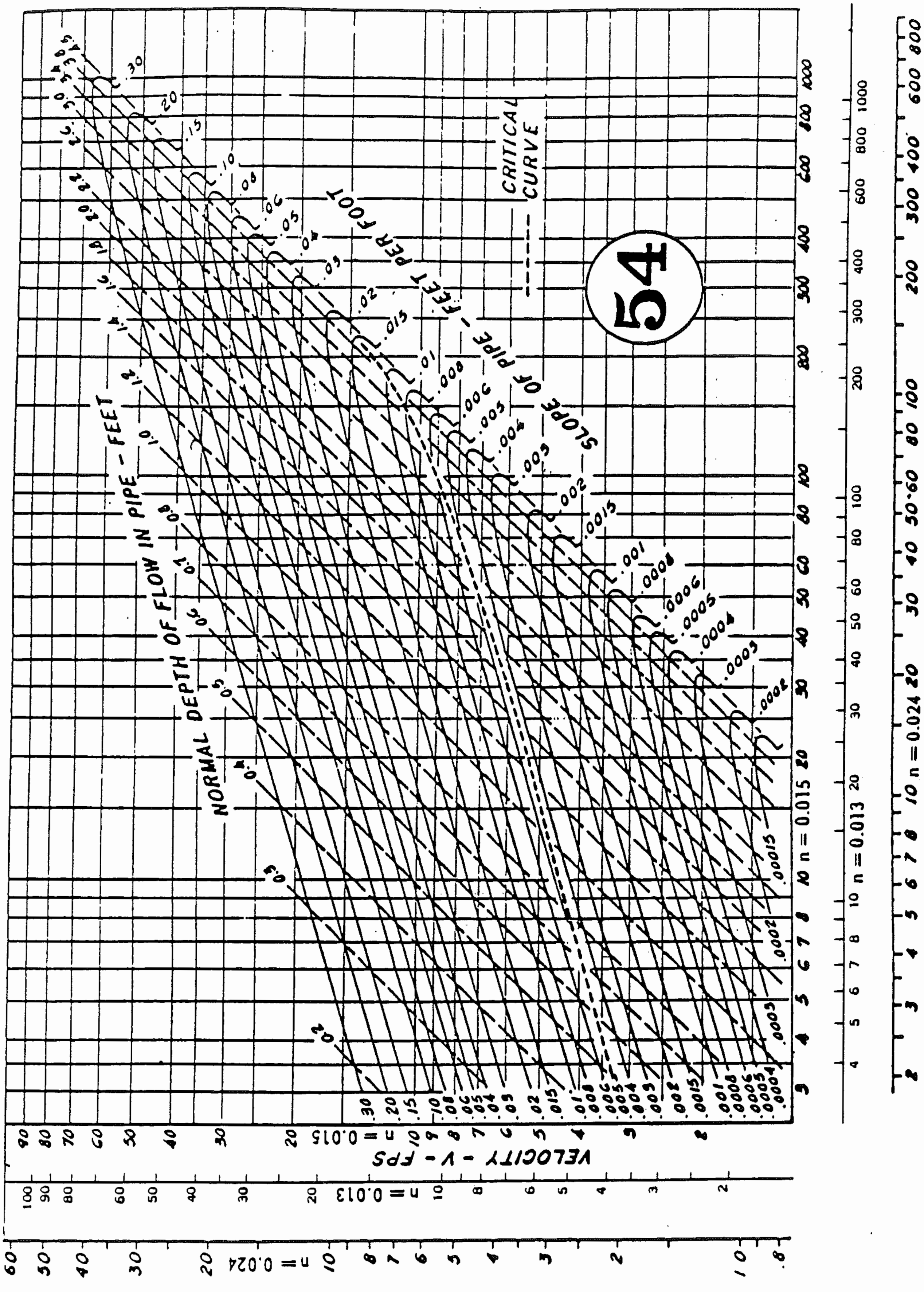


Pipe Flow Chart 42 inch Diameter

Plate 13

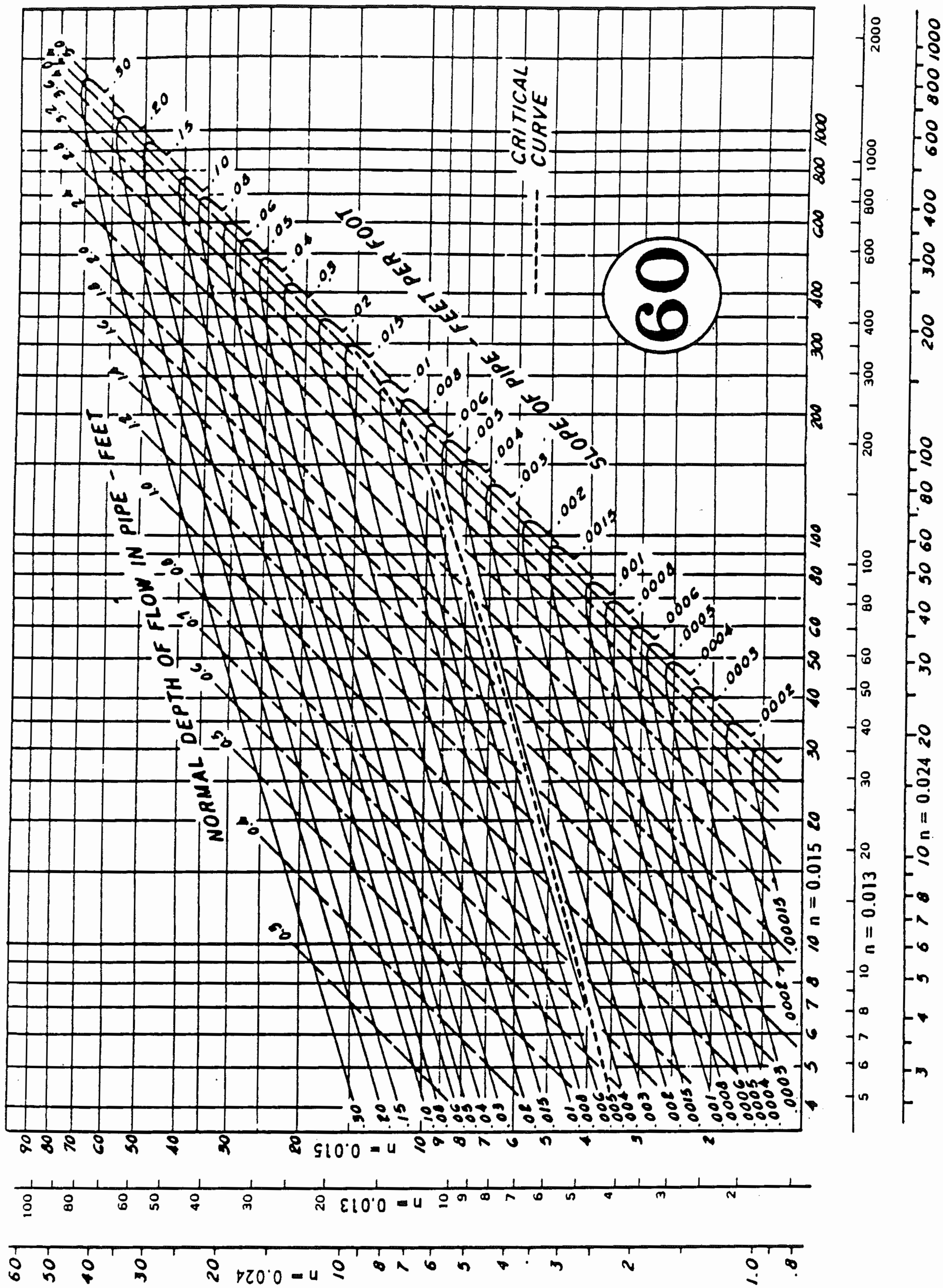


Pipe Flow Chart 48 inch Diameter



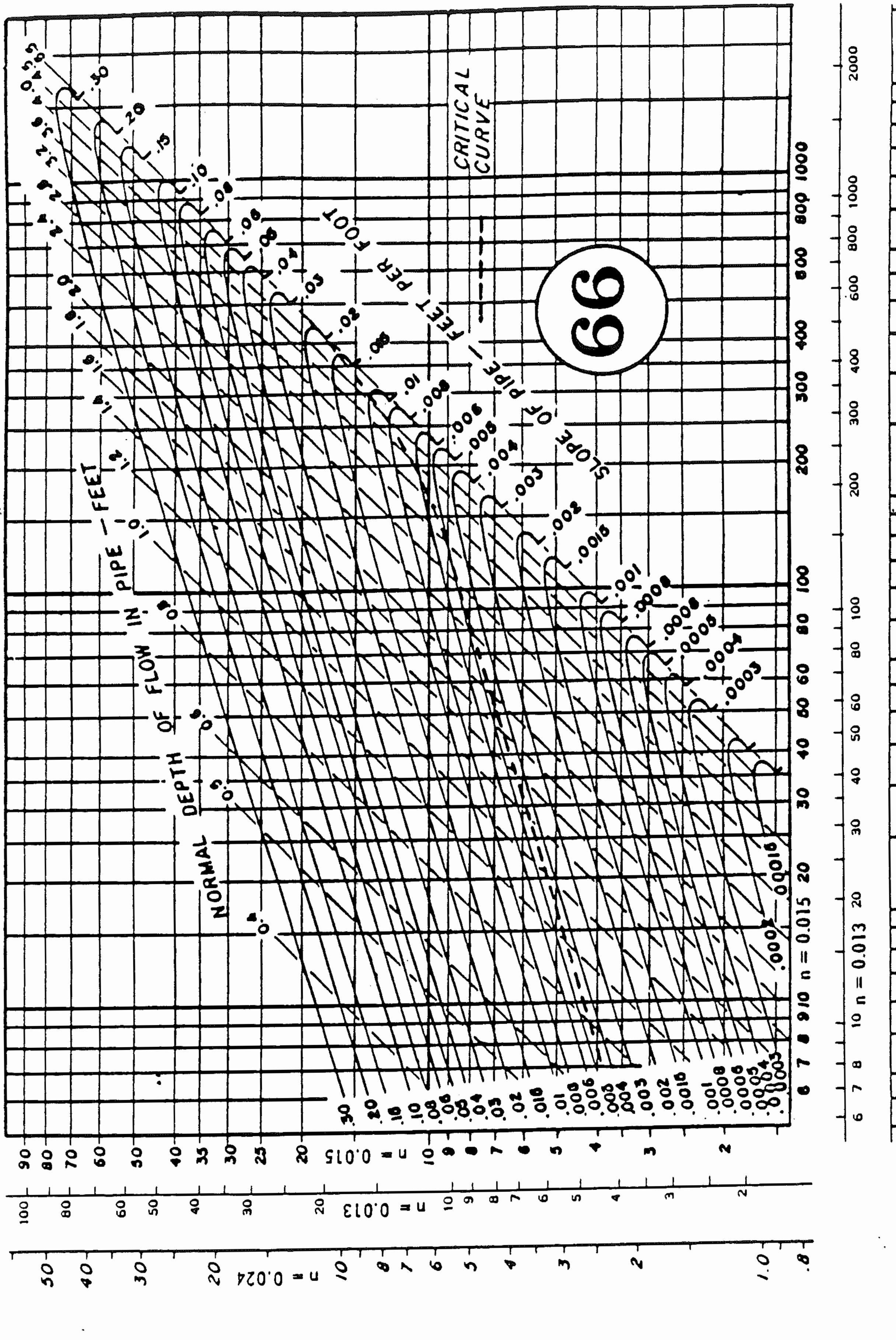
Pipe Flow Chart 54 inch Diameter

Plate 15



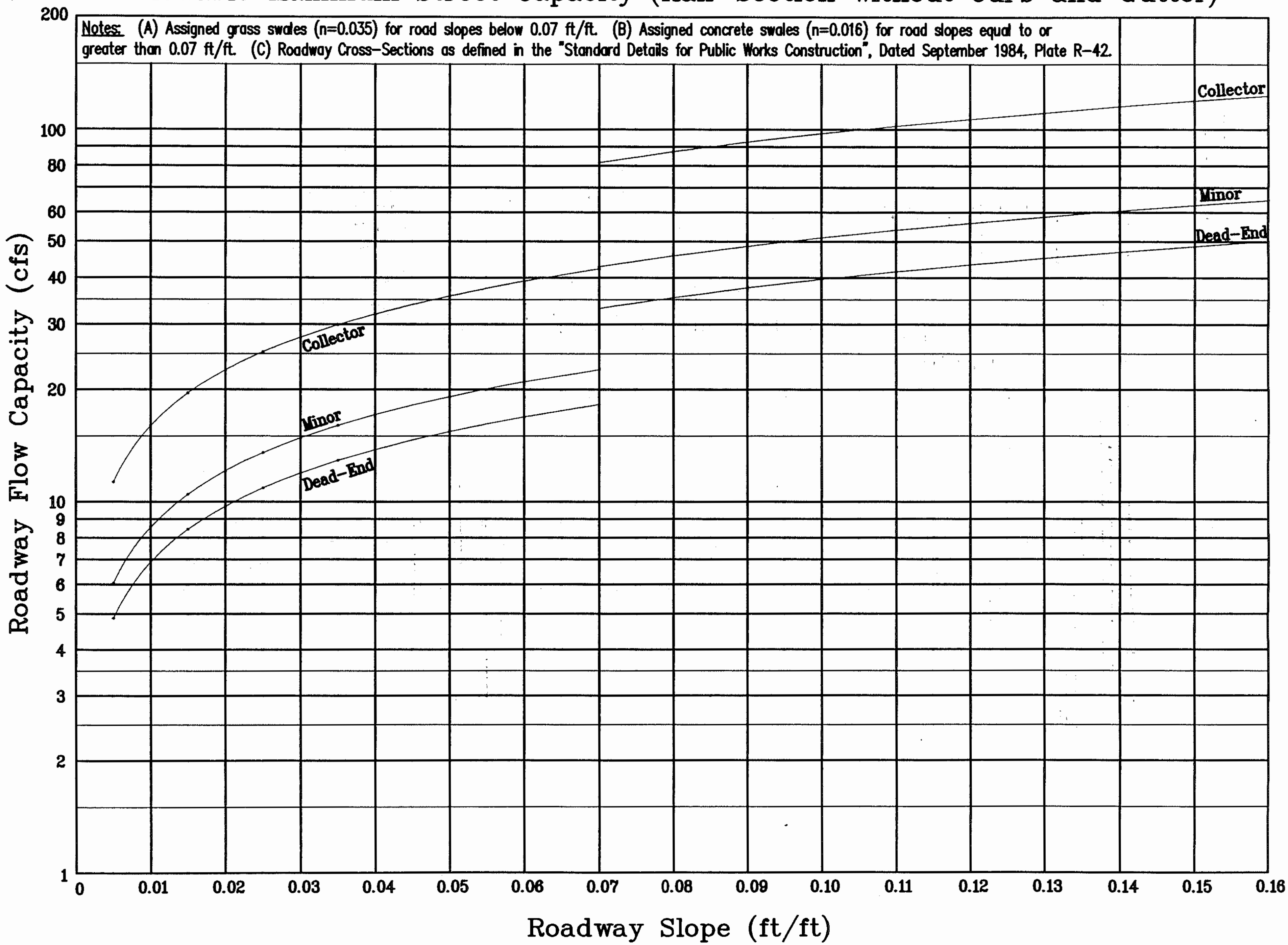
Pipe Flow Chart 60 inch Diameter

Plate 16



Pipe Flow Chart 66 inch Diameter

Allowable Maximum Street Capacity (Half–Section without Curb and Gutter)



Allowable Maximum Street Capacity (Half-Section with Curb and Gutter)

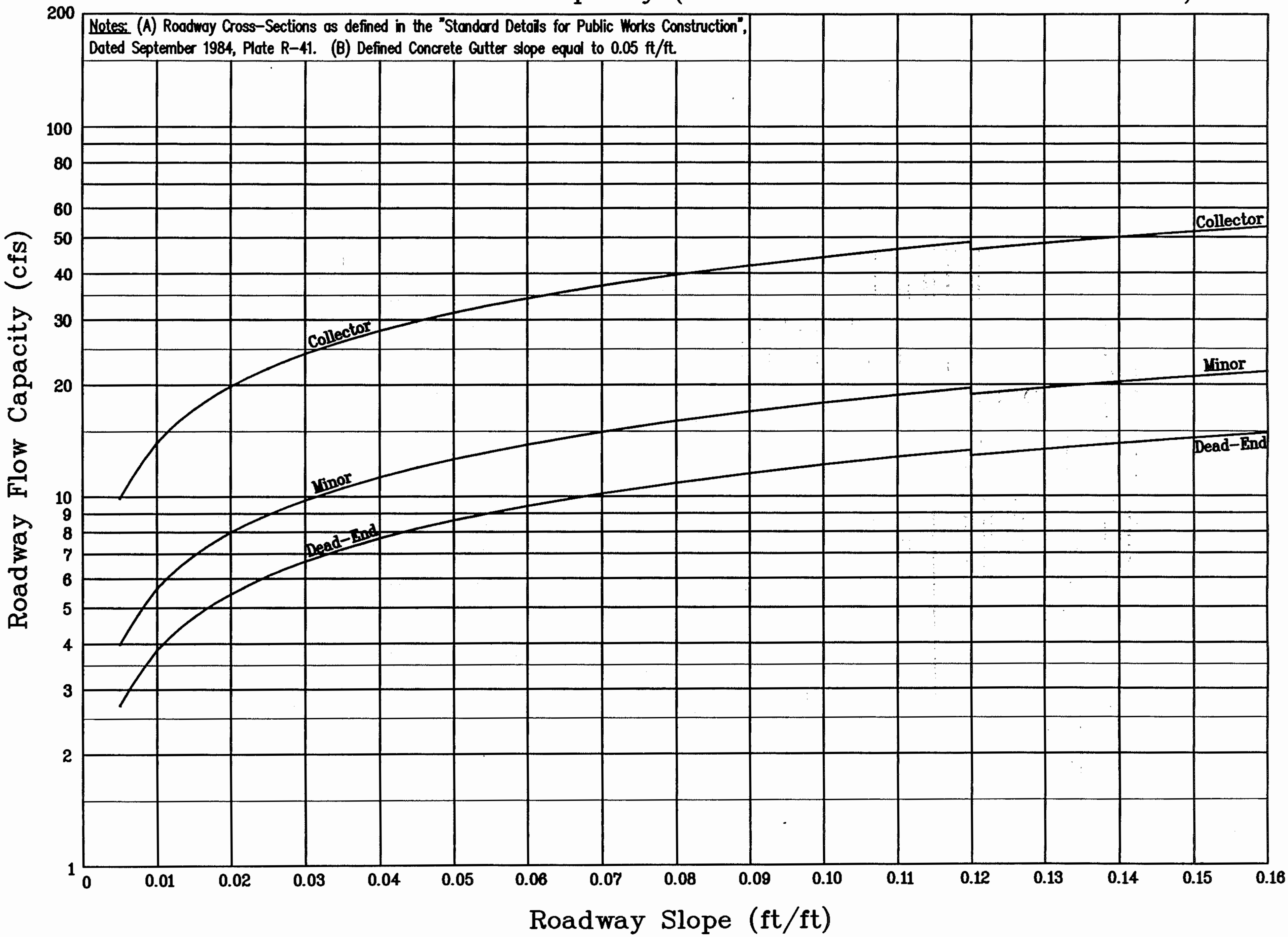
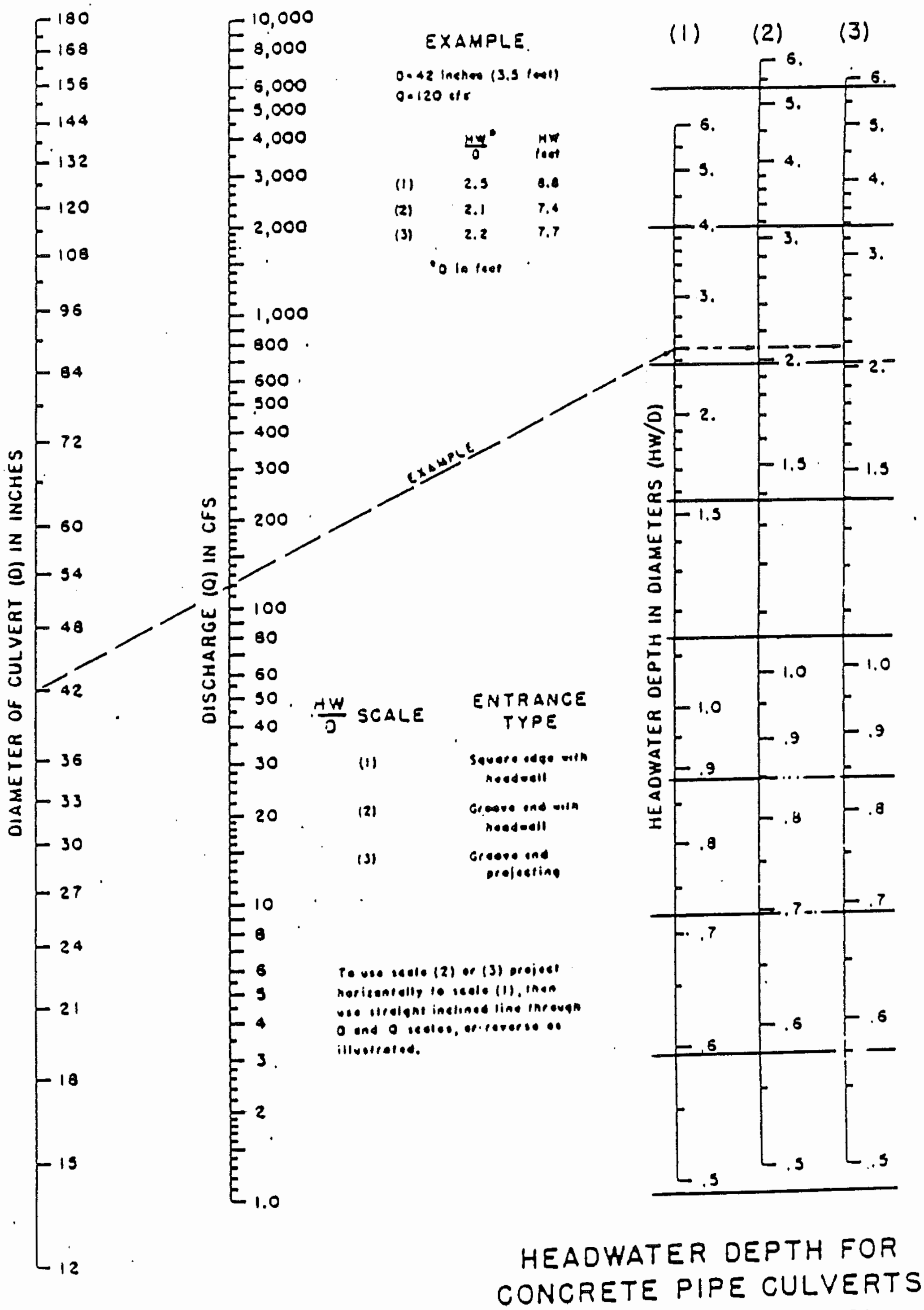


Plate 19



HEADWATER DEPTH FOR
CONCRETE PIPE CULVERTS
WITH INLET CONTROL

BUREAU OF PUBLIC ROADS JAN. 1963

NOMOGRAPH FOR BOX CULVERTS WITH ENTRANCE CONTROL

Plate 20

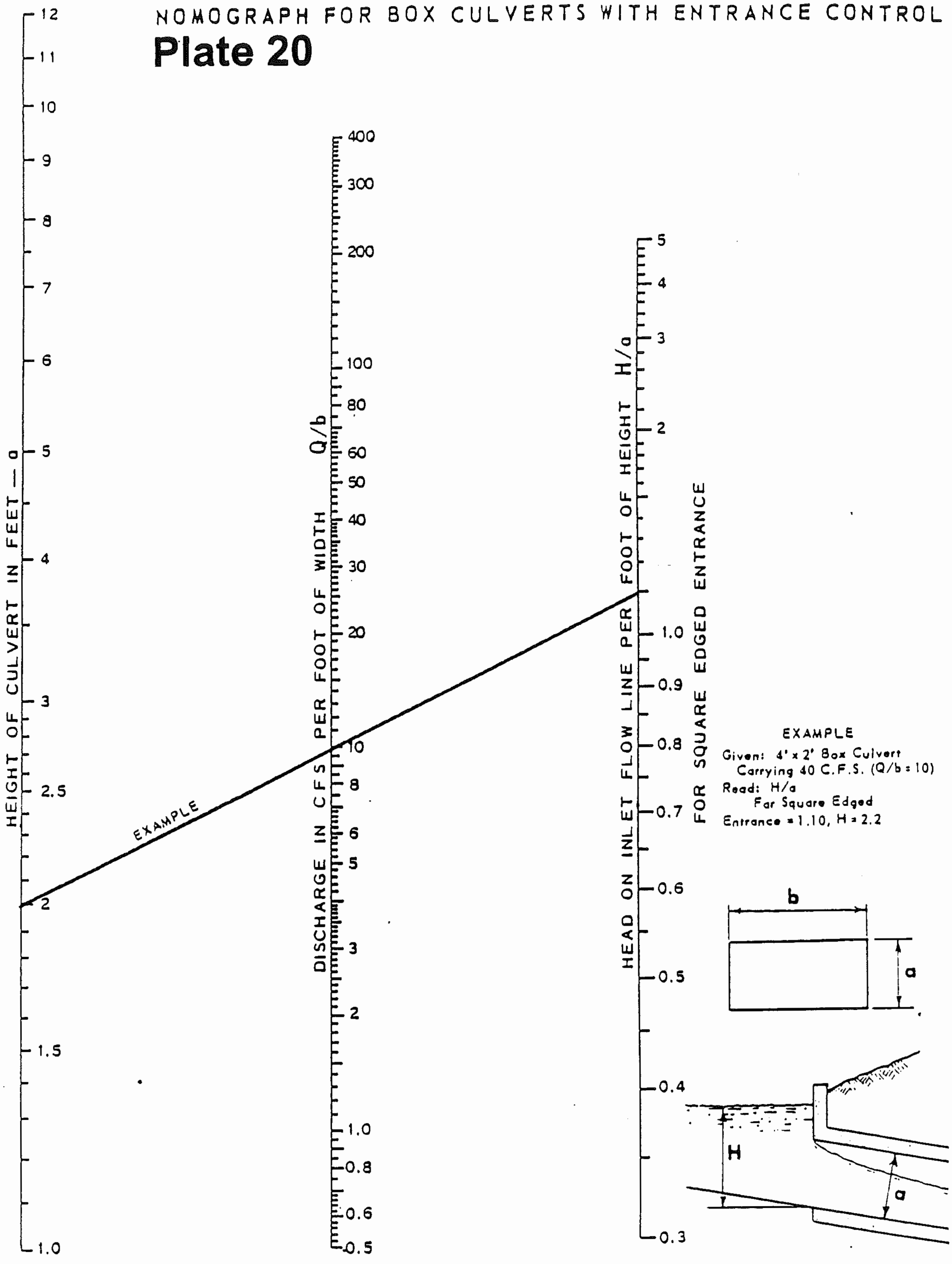
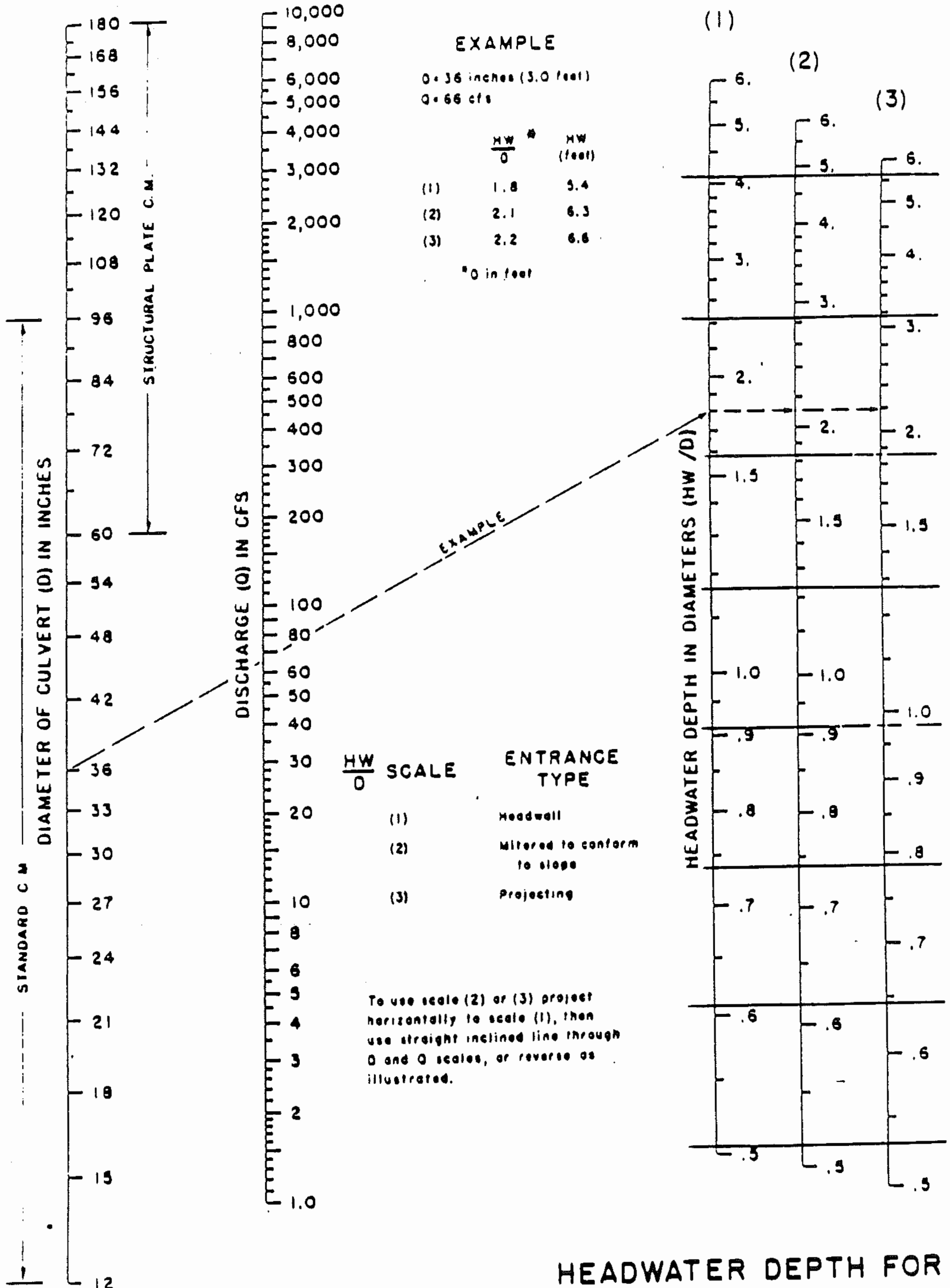


Plate 21



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

BUREAU OF PUBLIC ROADS JAN. 1963

Plate 22

HEAD FOR CONCRETE PIPE CULVERTS FLOWING FULL, n = 0.012

adapted from Bureau of Public Roads chart 1051.1

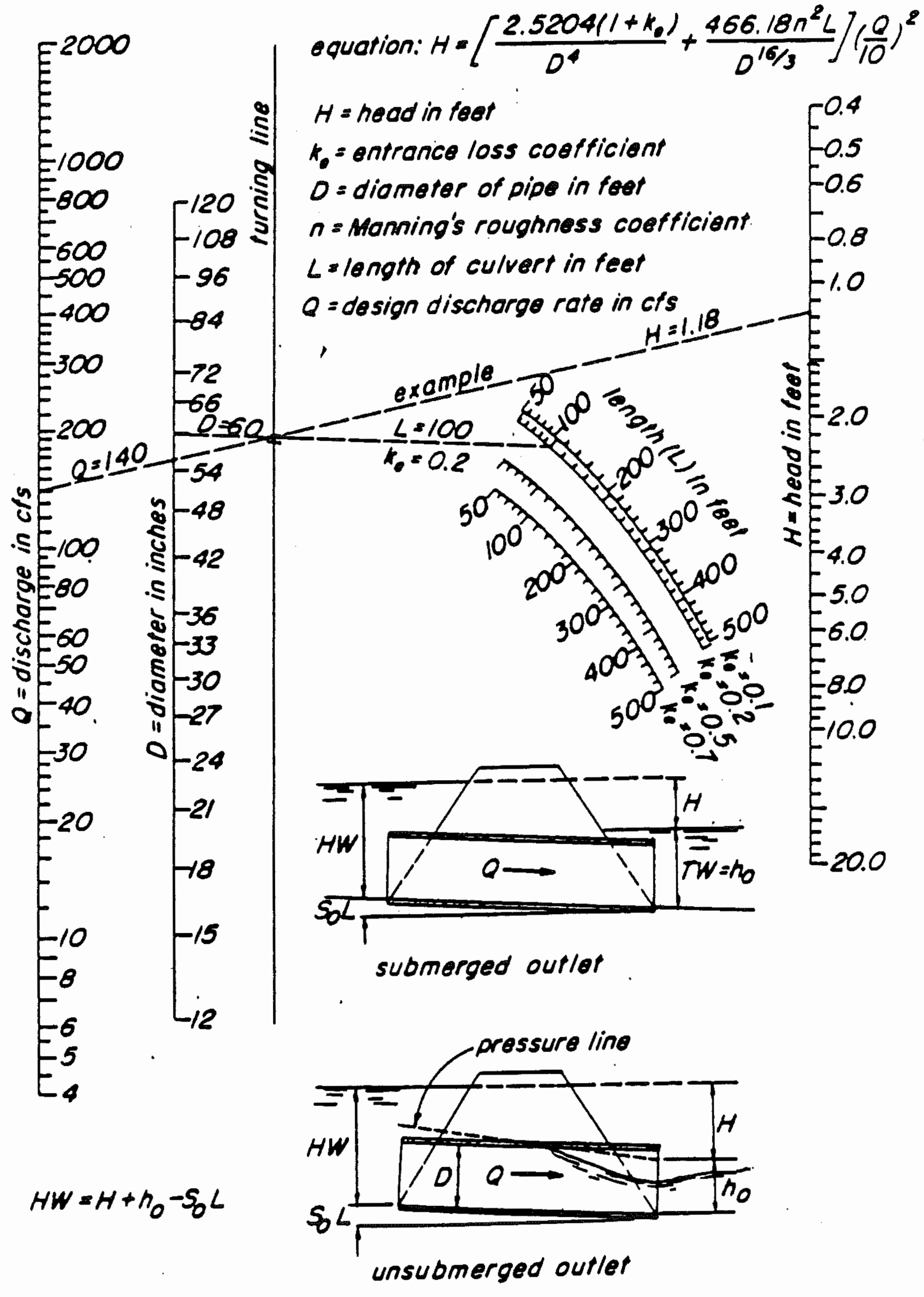


Plate 23

HEAD FOR C.M. PIPE CULVERTS FLOWING FULL, n=0.024

adapted from Bureau of Public Roads chart 1052

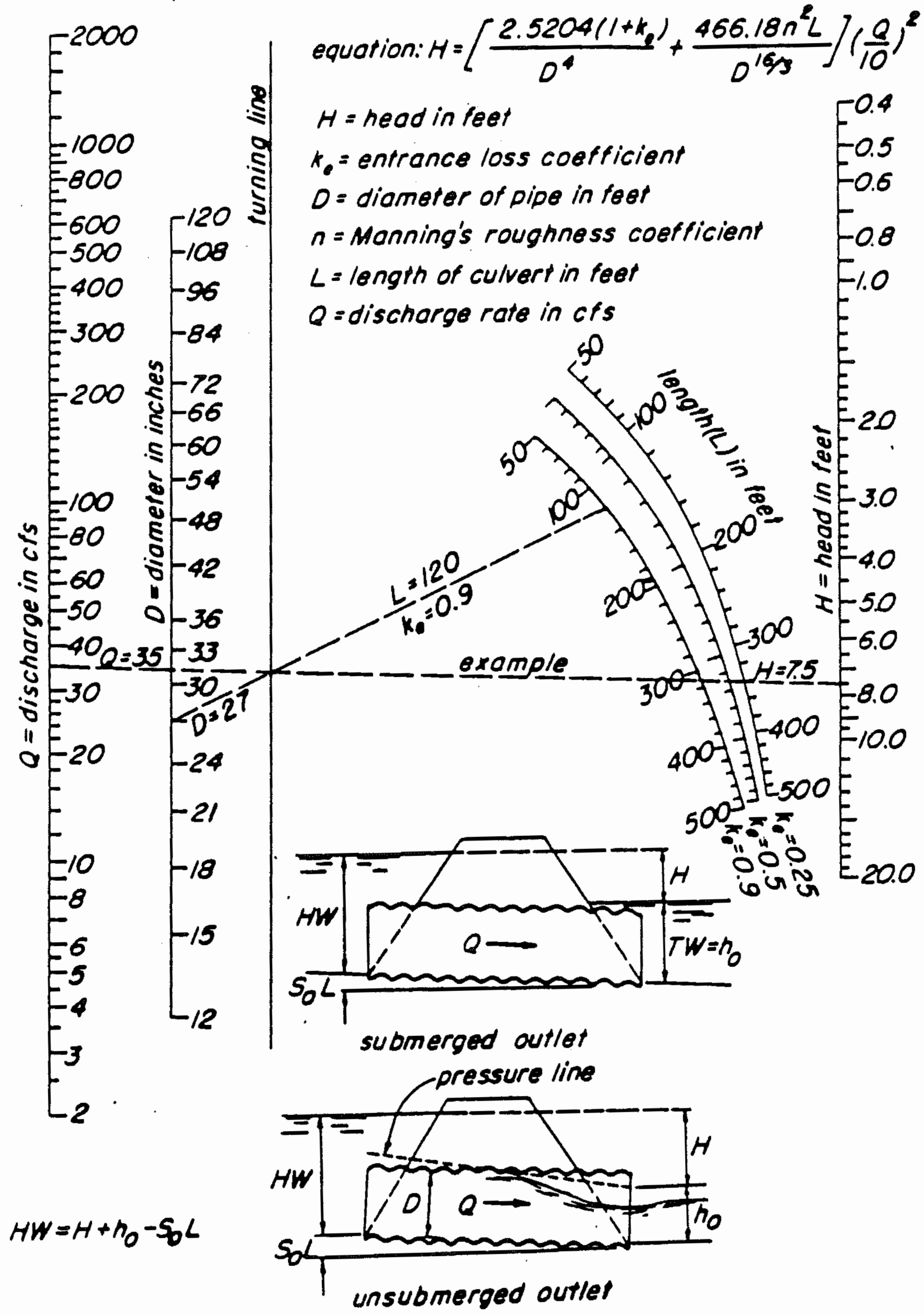
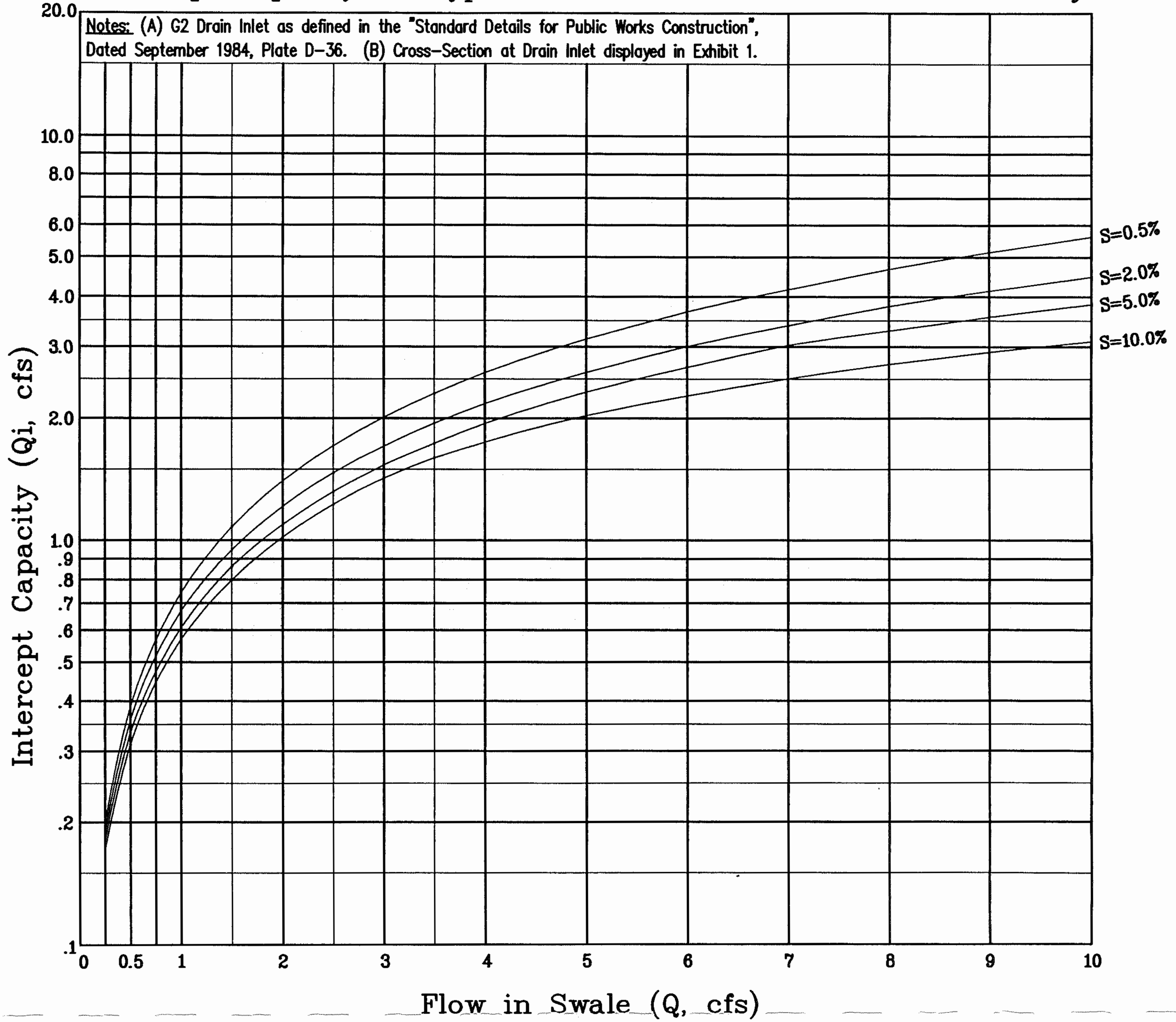


Fig. C-1

Intercept Capacity for Type G2 Drain Inlet on a Collector Roadway



Intercept Capacity for Type G2 Drain Inlet on a Minor Roadway

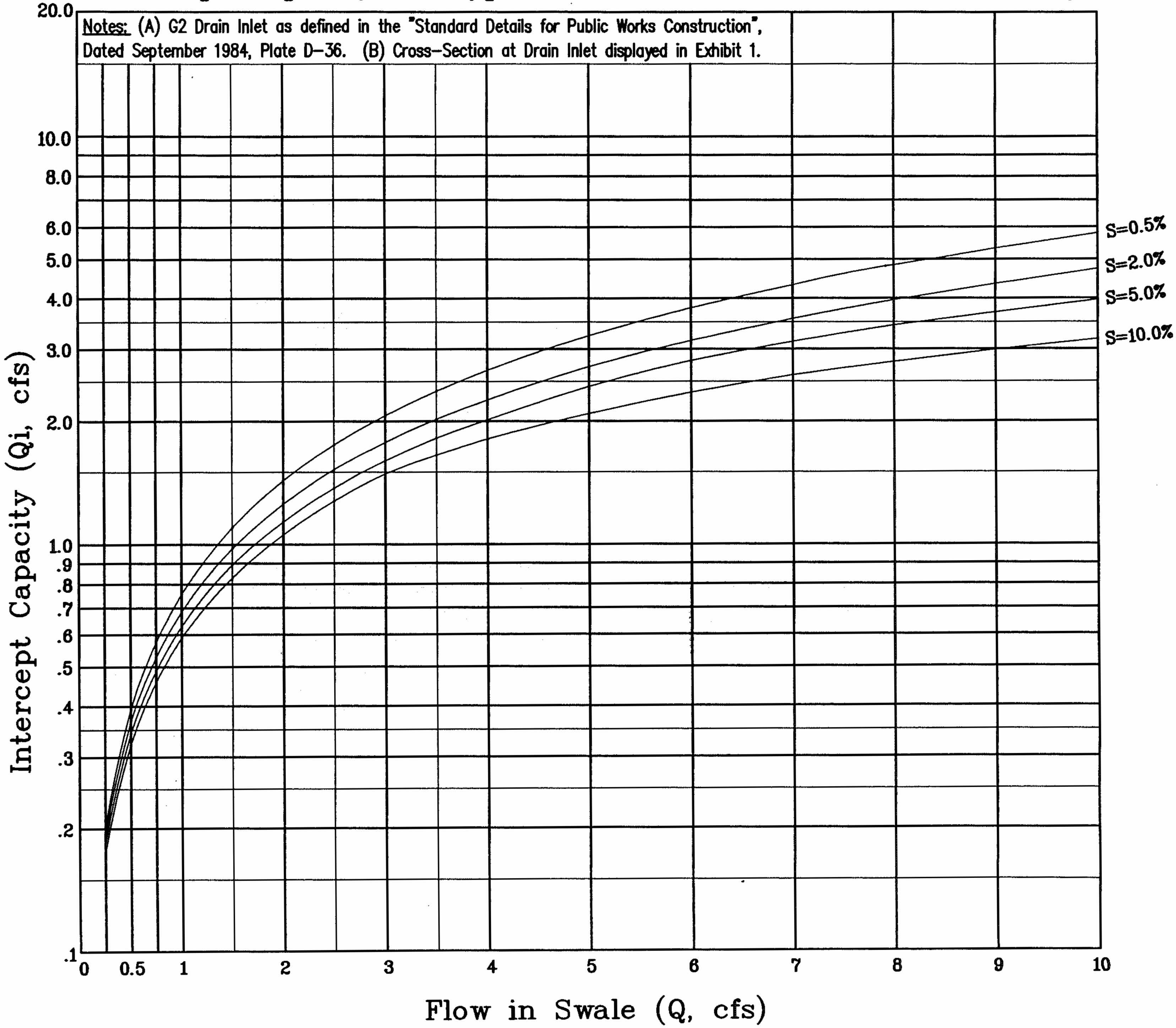


Plate 26

Intercept Capacity for Type G2 Drain Inlet on a Dead-End Roadway

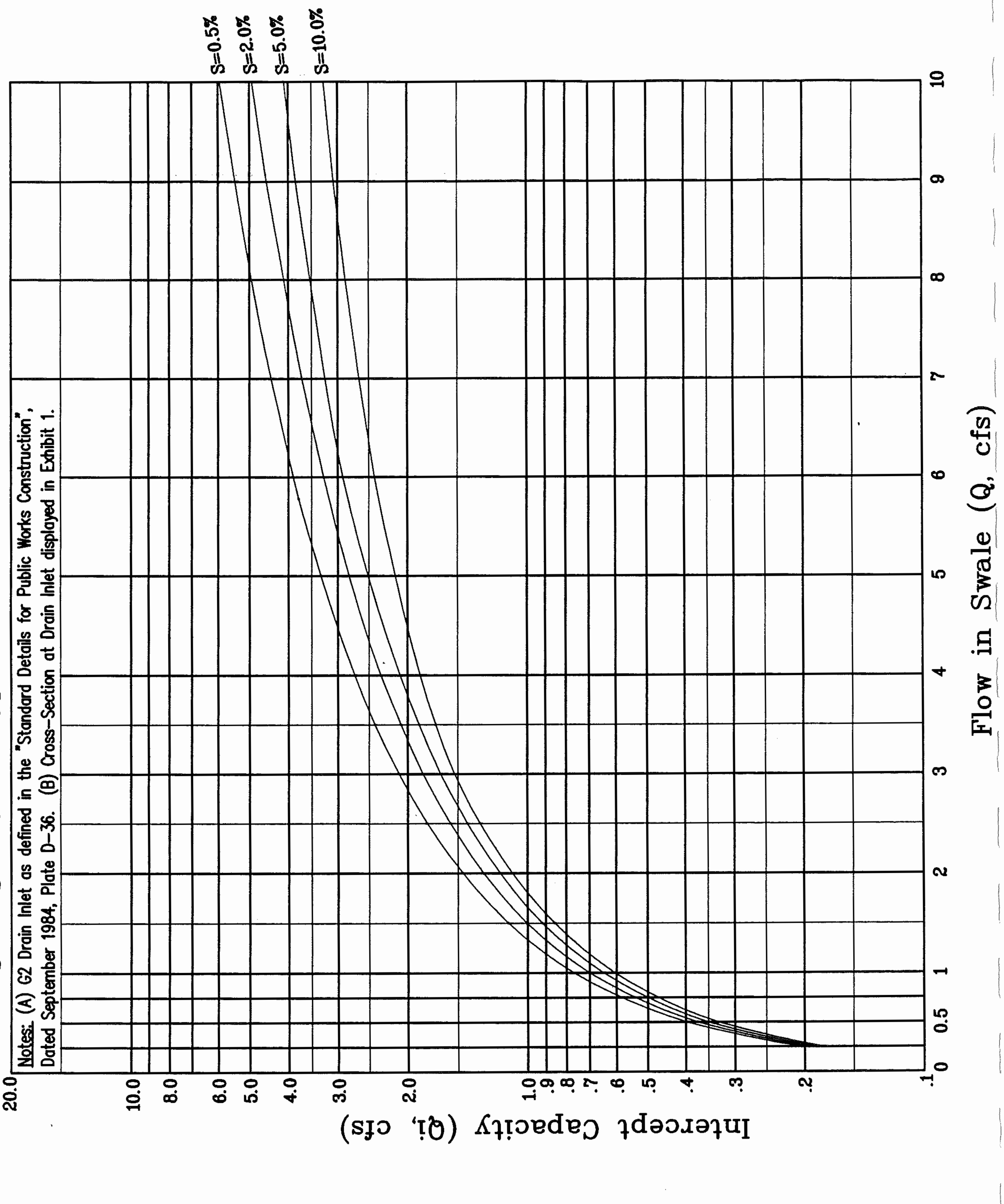
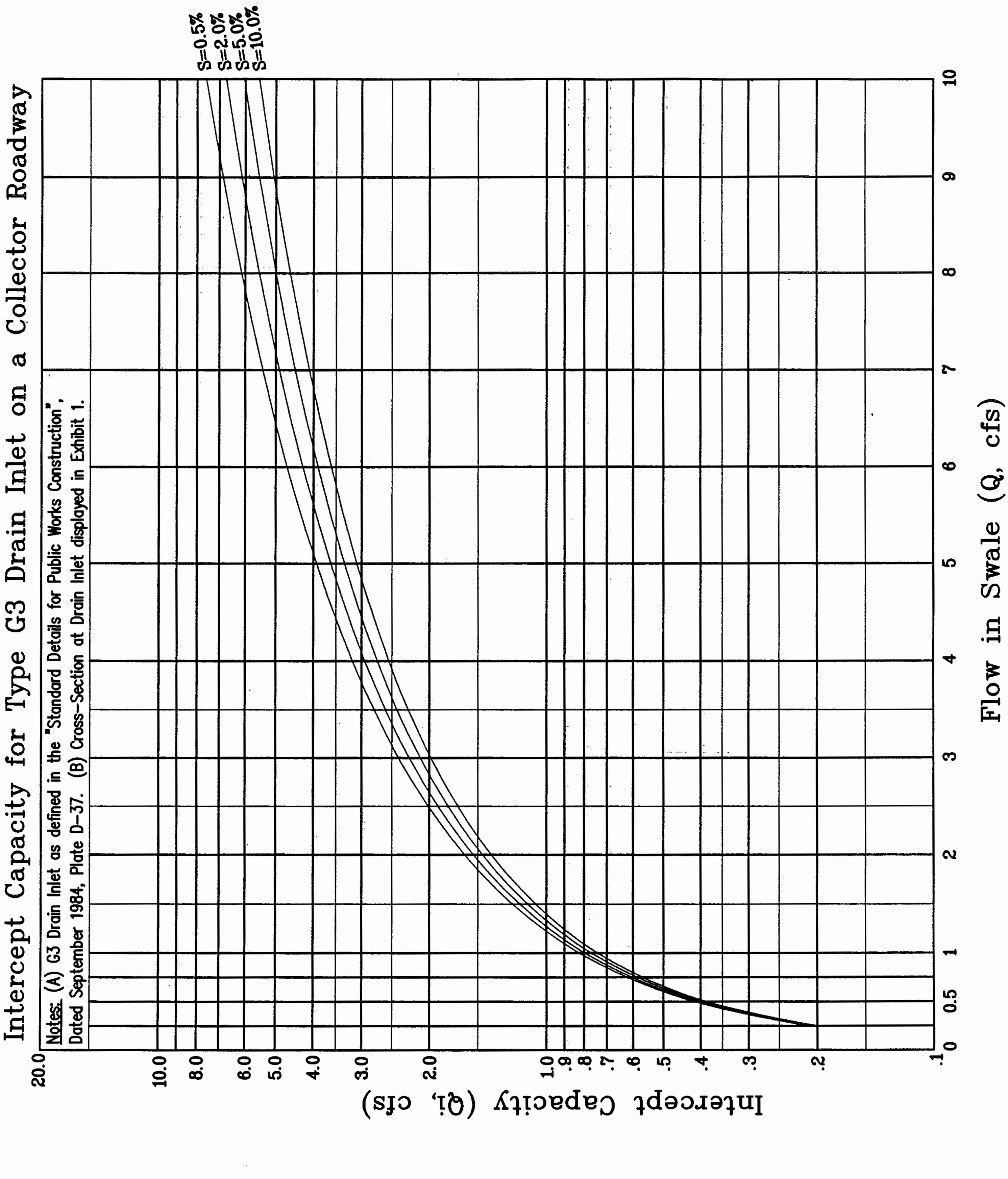


Plate 27



Intercept Capacity for Type G3 Drain Inlet on a Collector Roadway
Notes: (A) G3 Drain Inlet as defined in the "Standard Details for Public Works Construction", Dated September 1984, Plate D-37. (B) Cross-Section at Drain Inlet displayed in Exhibit 1.
S=0.5%
S=2.0%
S=5.0%
S=10.0%
Intercept Capacity (Qi, cfs)
20.0
10.0
8.0
6.0
5.0
4.0
3.0
2.0
1.0
.9
.8
.7
.6
.5
.4
.3
.2
.1
Flow in Swale (Q, cfs)
0
0.5
1
2
3
4
5
6
7
8
9
10

Intercept Capacity for Type G3 Drain Inlet on a Minor Roadway

Notes: (A) G3 Drain Inlet as defined in the "Standard Details for Public Works Construction", Dated September 1984, Plate D-37. (B) Cross-Section at Drain Inlet displayed in Exhibit 1.

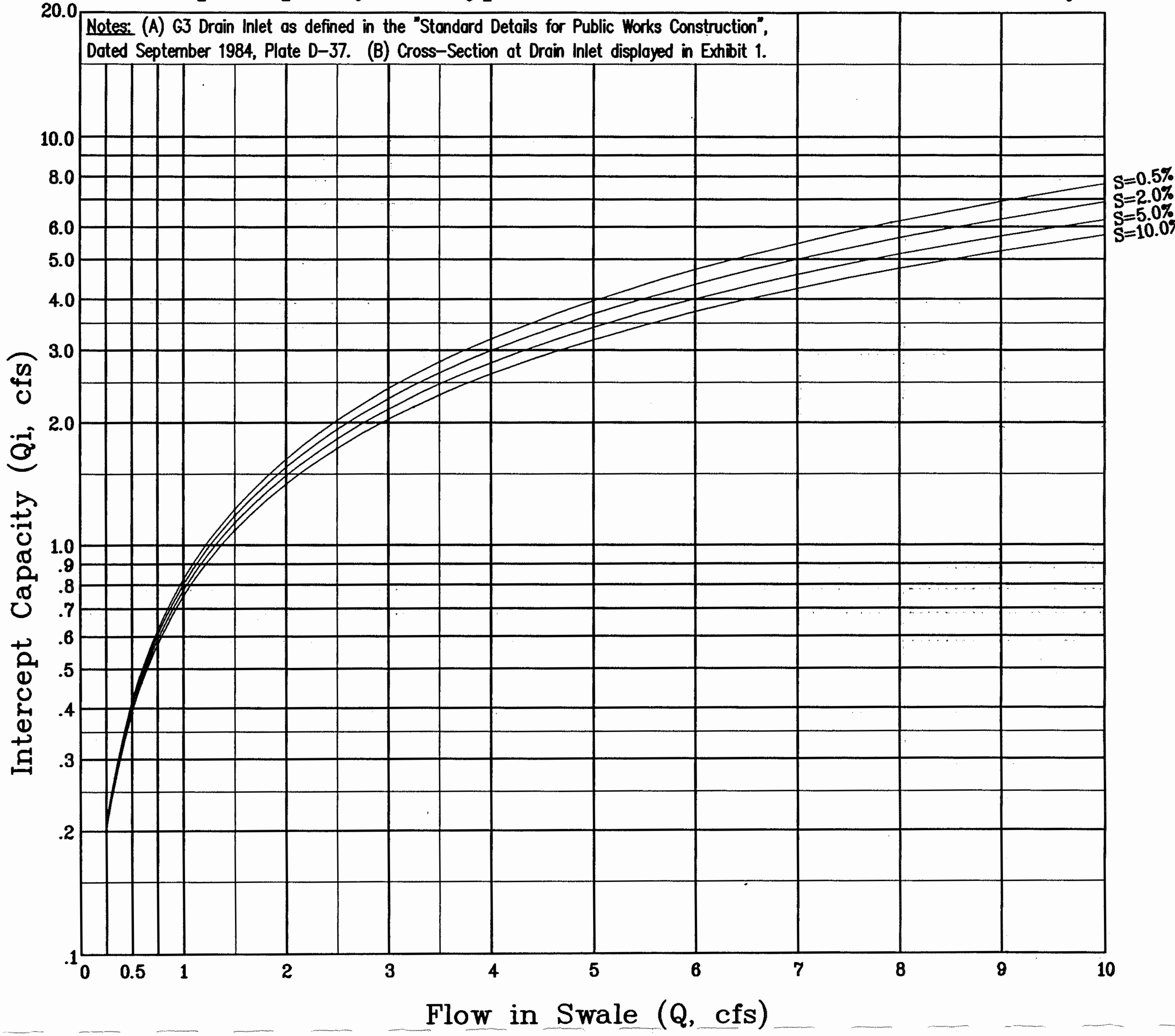


Plate 29

Intercept Capacity for Type G3 Drain Inlet on a Dead-End Roadway

Notes: (A) G3 Drain Inlet as defined in the "Standard Details for Public Works Construction", Dated September 1984, Plate D-37. (B) Cross-Section at Drain Inlet displayed in Exhibit 1.

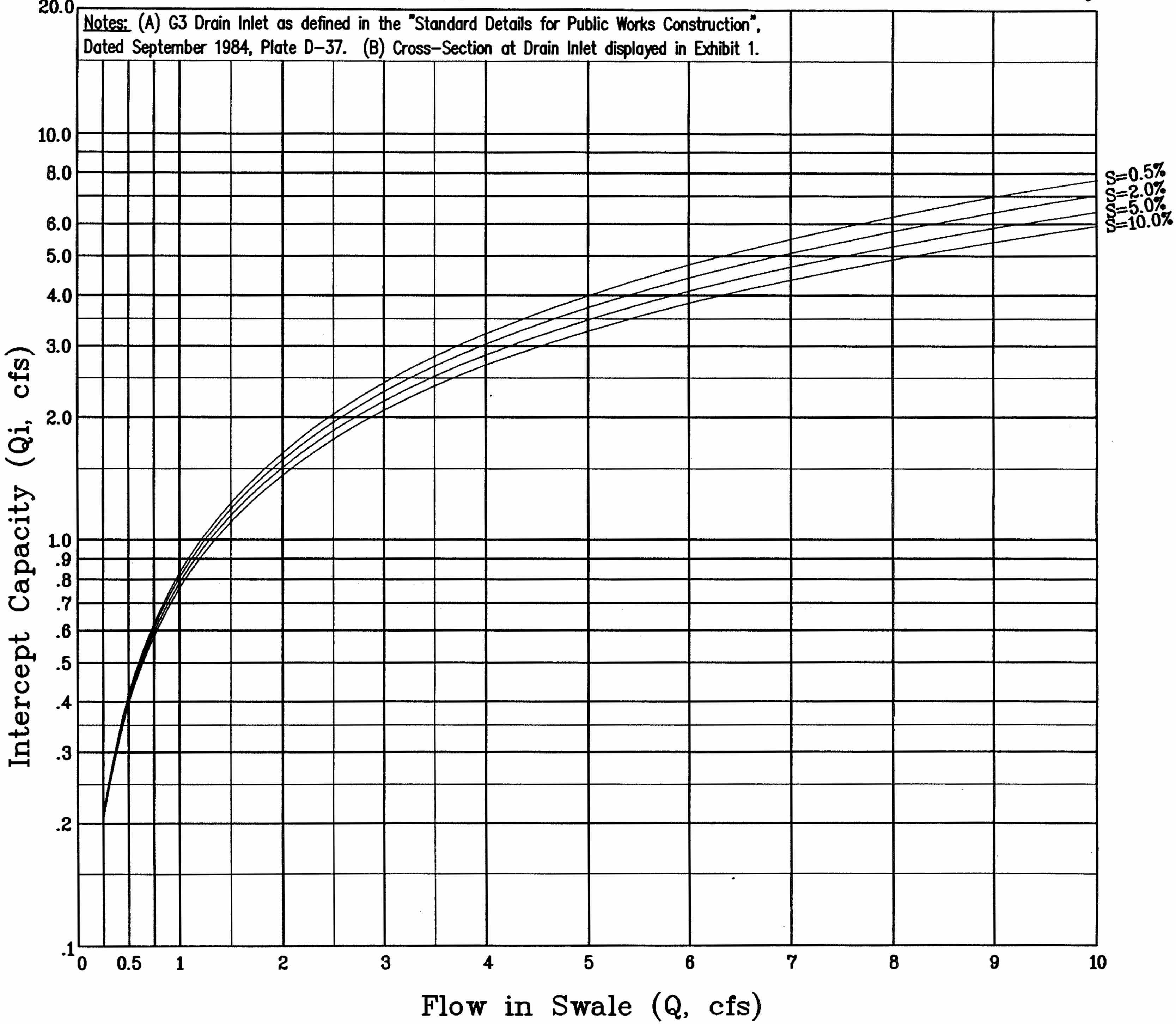
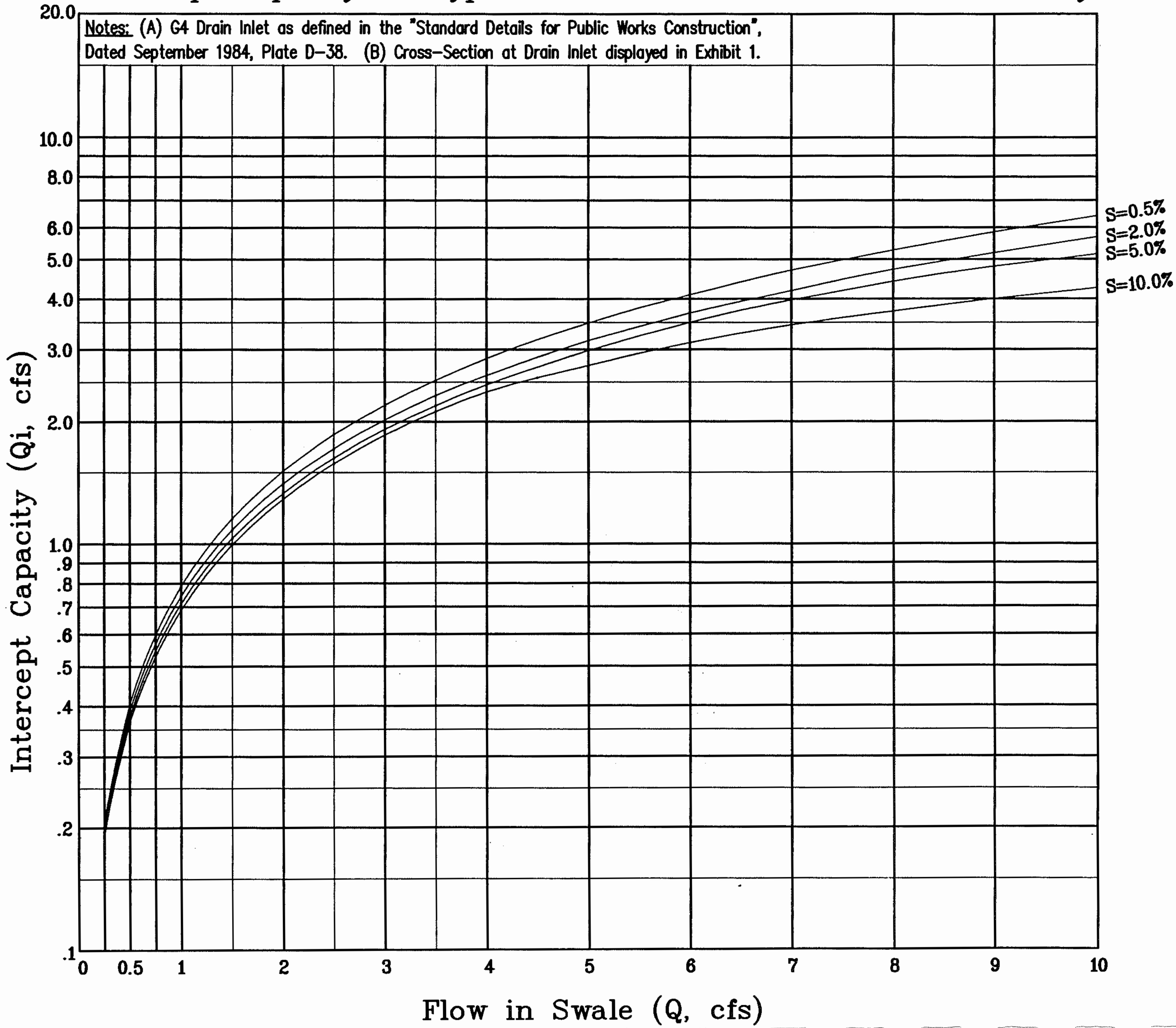


Plate 30

Intercept Capacity for Type G4 Drain Inlet on a Collector Roadway



Intercept Capacity for Type G4 Drain Inlet on a Minor Roadway

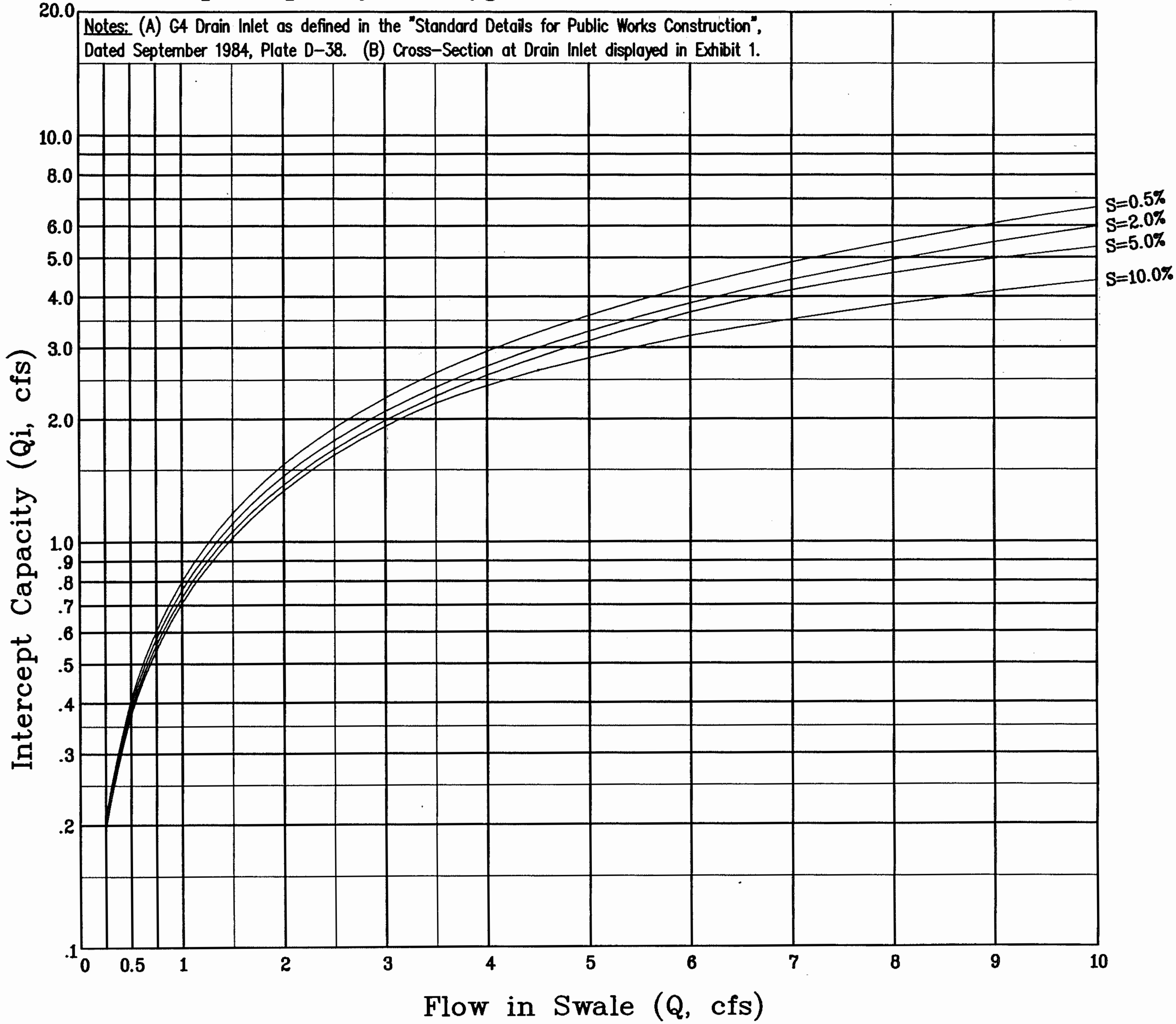


Plate 32

Intercept Capacity for Type G4 Drain Inlet on a Dead-End Roadway

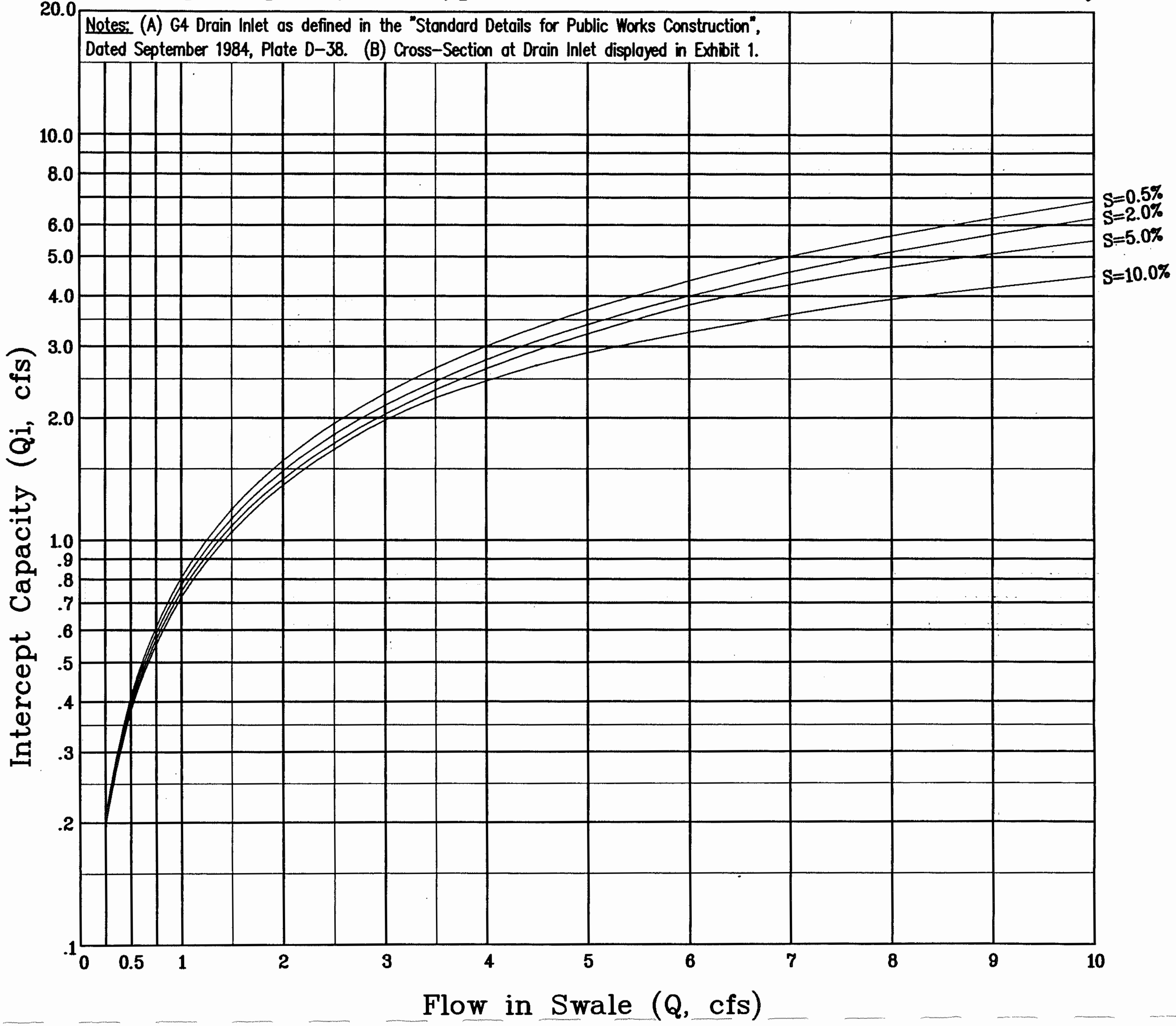
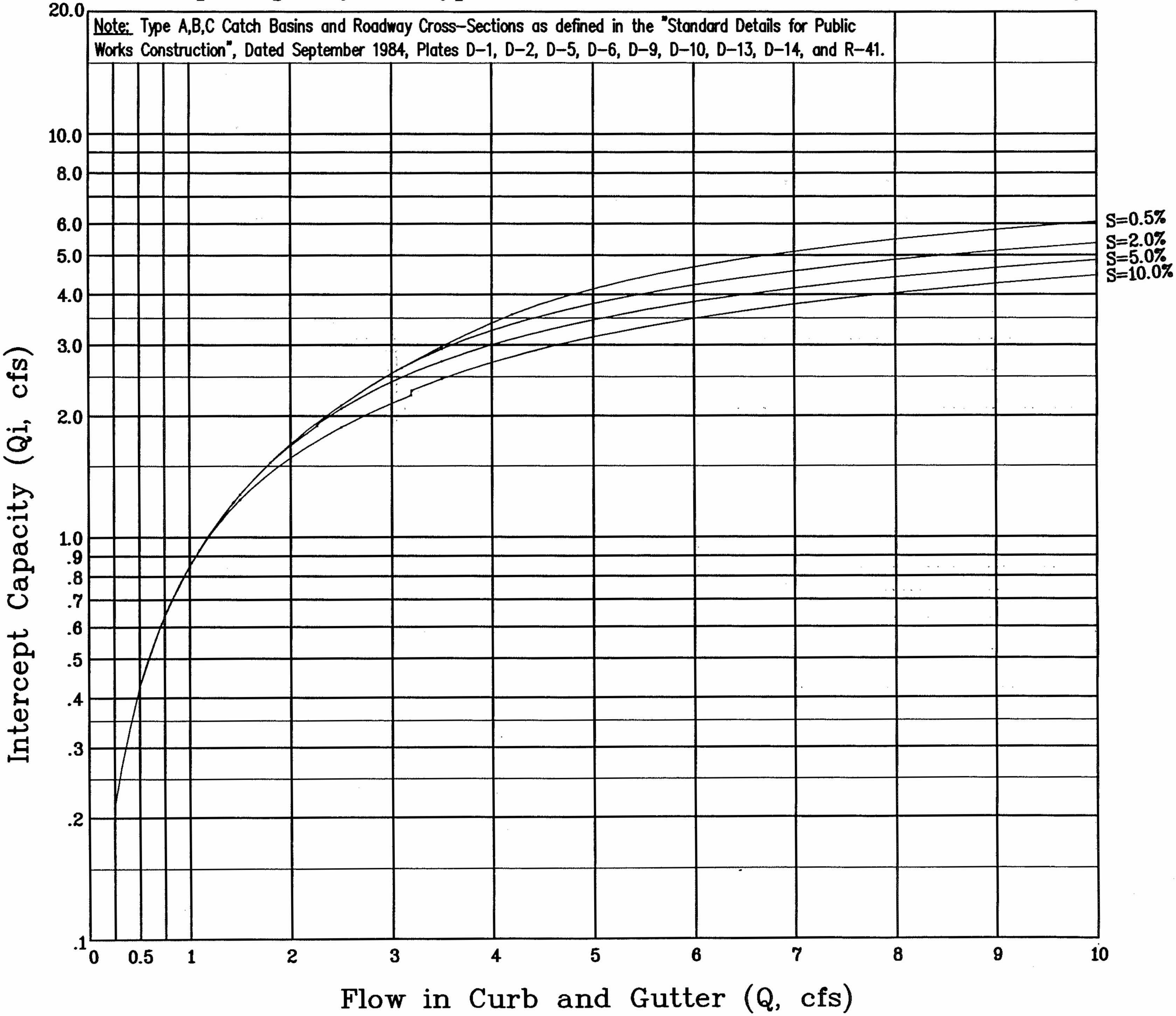


Plate 33

Intercept Capacity for Type A,B,C Catch Basin on a Collector Roadway



Intercept Capacity for Type D Catch Basin on a Collector Roadway

Note: Type D Catch Basin and Roadway Cross-Sections as defined in the "Standard Details for Public Works Construction", Dated September 1984, Plates D-11, D-12, D-13, D-14, and R-41.

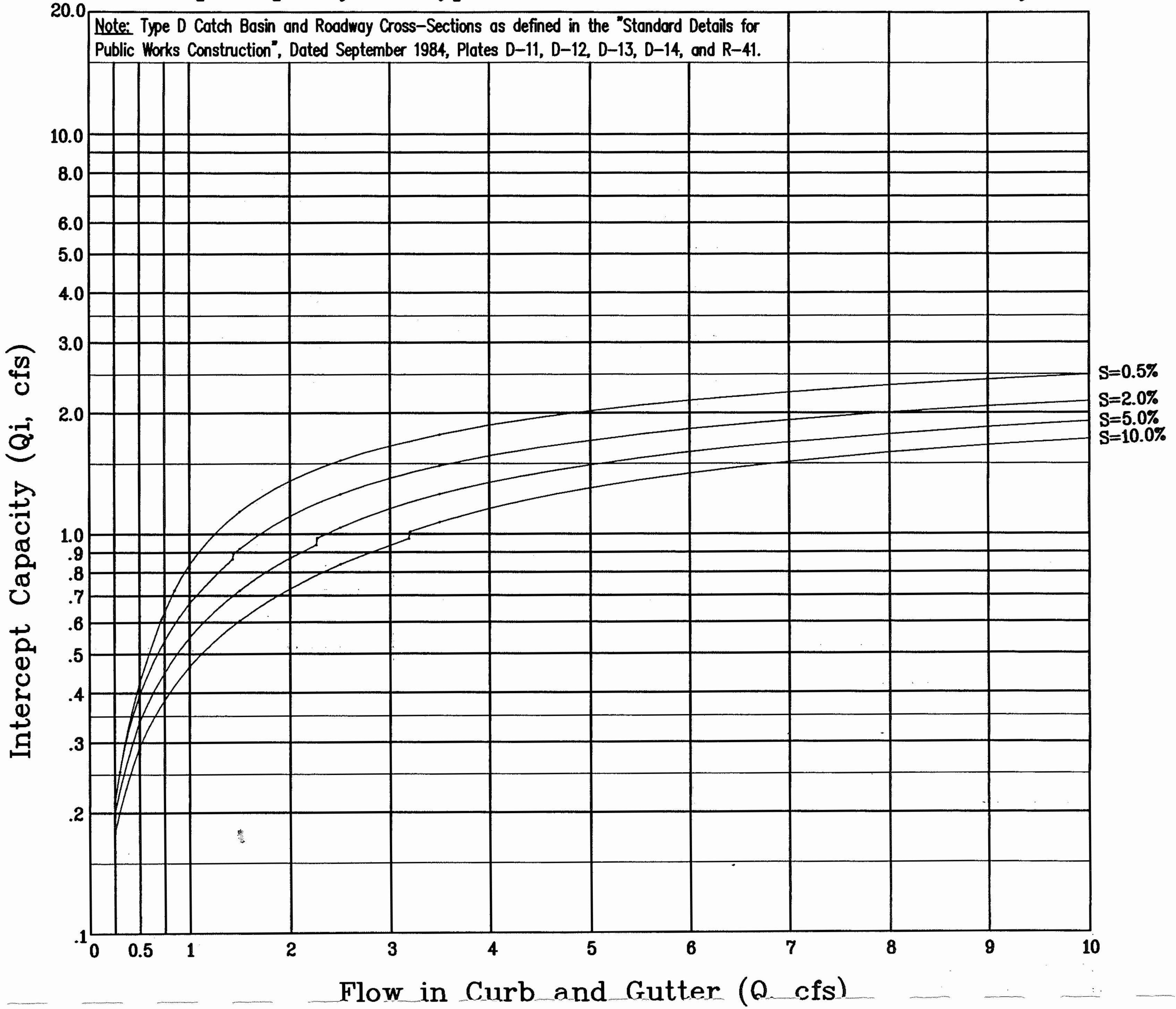
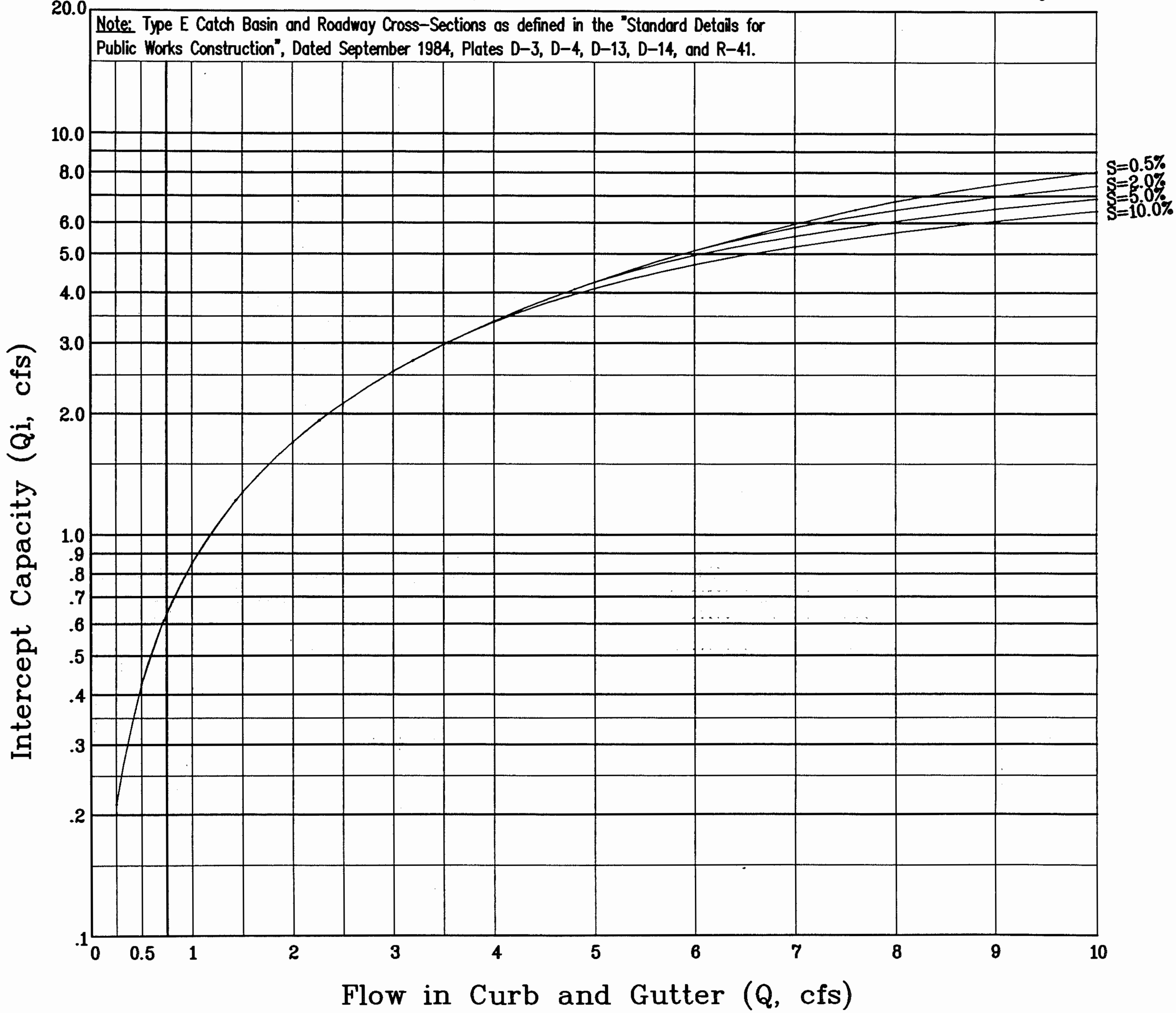


Plate 35

Intercept Capacity for Type E Catch Basin on a Collector Roadway



Intercept Capacity for Type F Catch Basin on a Collector Roadway

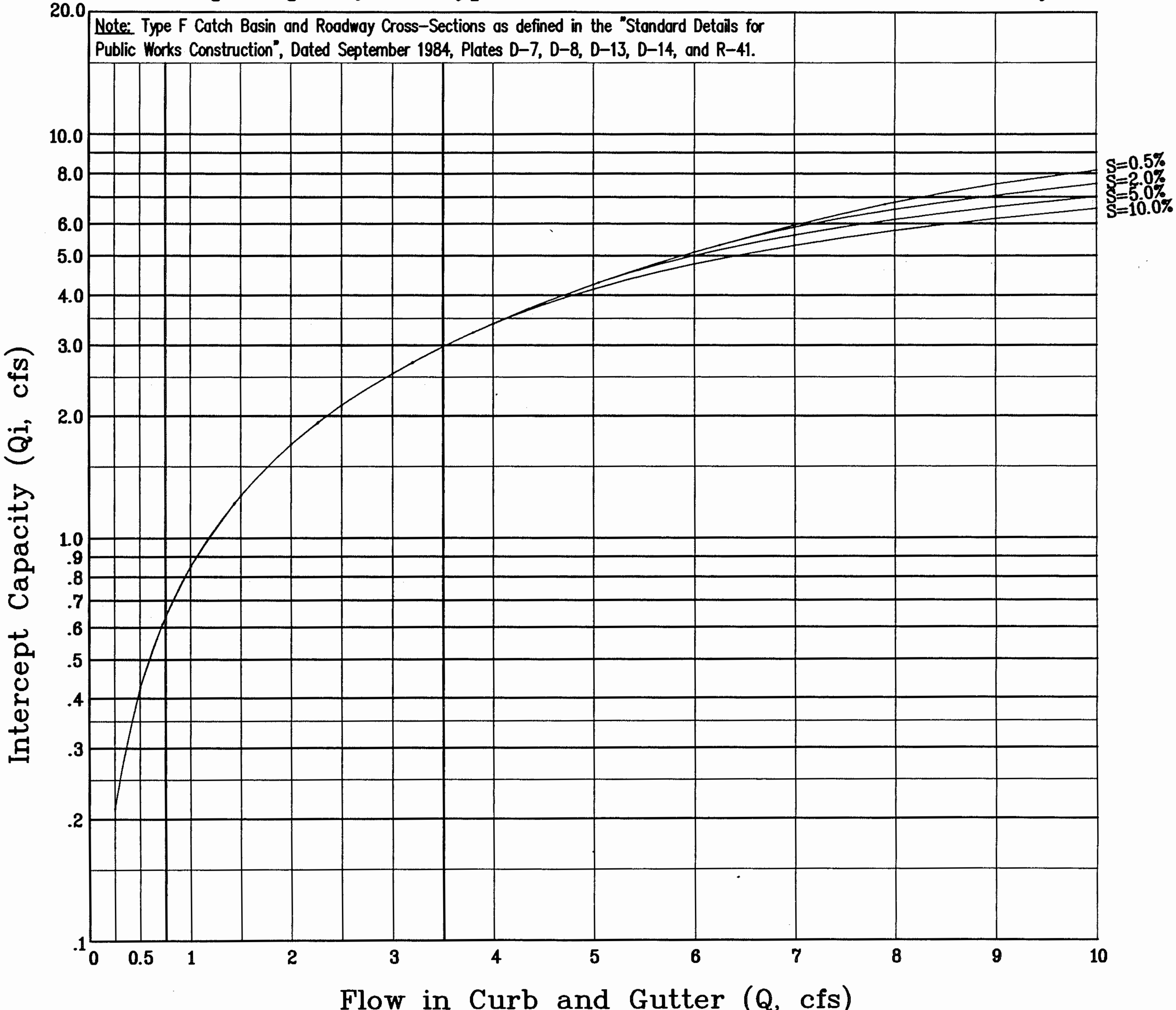


Plate 36

Plate 37

Intercept Capacity for Type A,B,C Catch Basin on a Minor Roadway

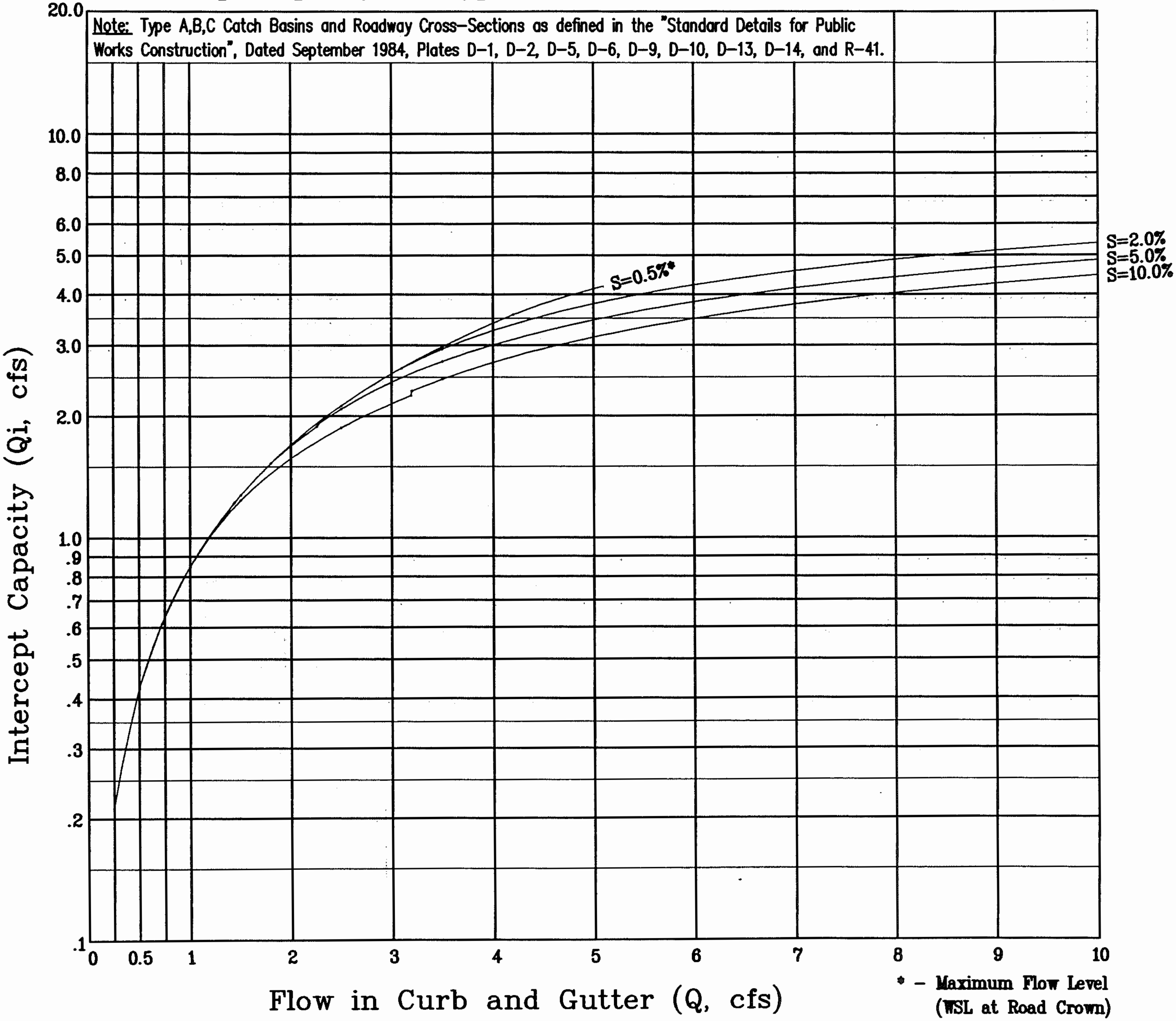
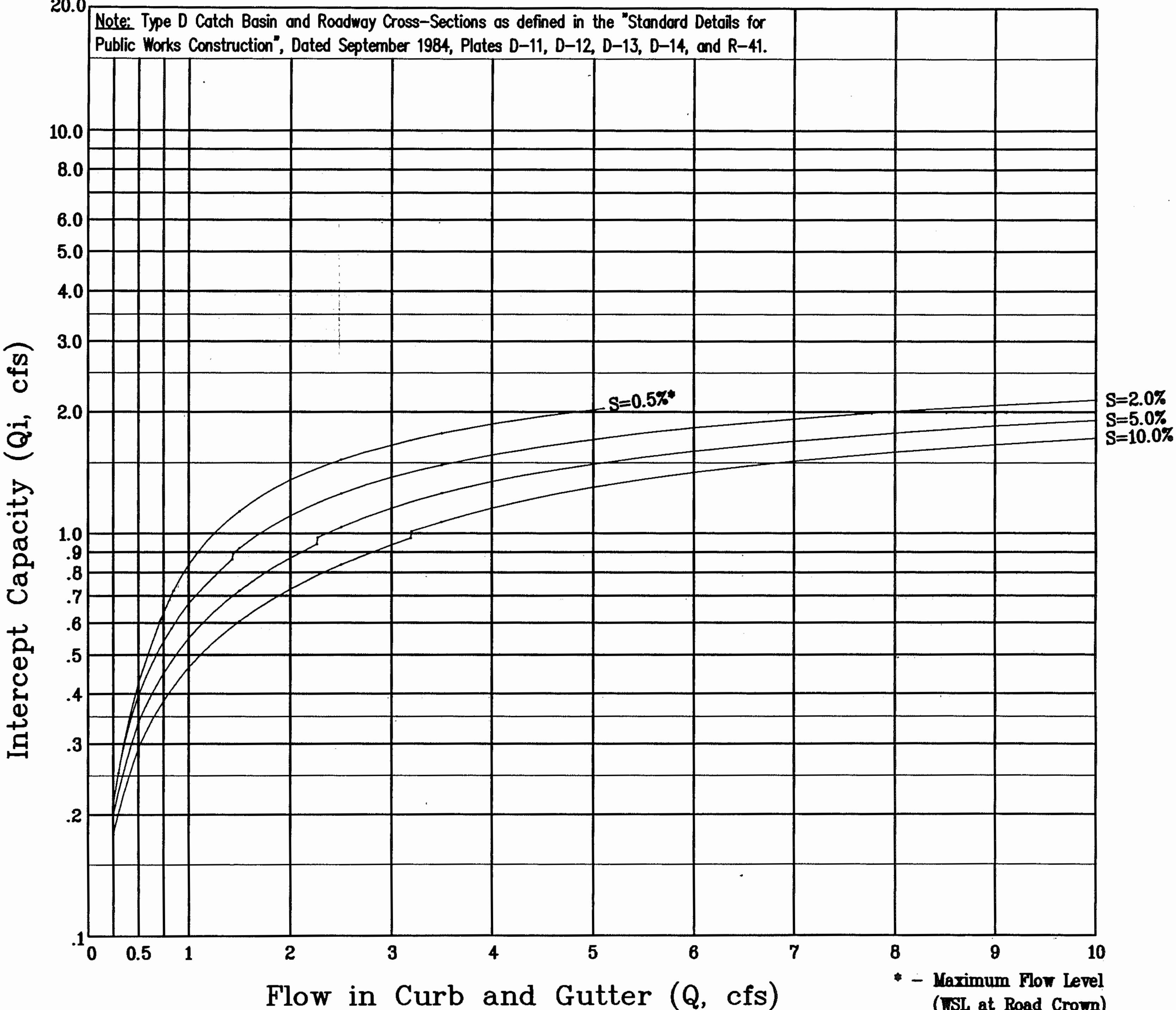


Plate 38

Intercept Capacity for Type D Catch Basin on a Minor Roadway

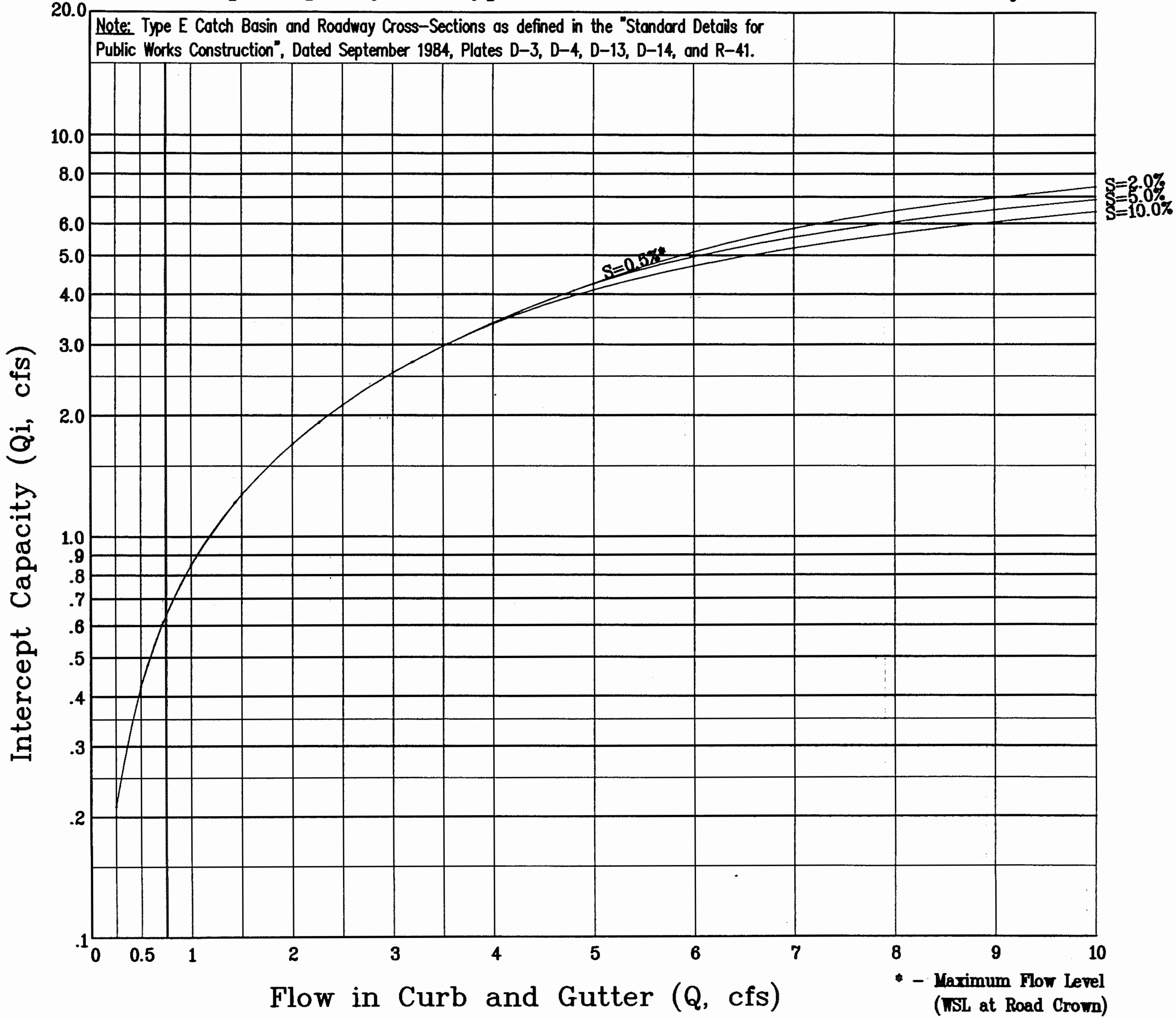
Note: Type D Catch Basin and Roadway Cross-Sections as defined in the "Standard Details for Public Works Construction", Dated September 1984, Plates D-11, D-12, D-13, D-14, and R-41.



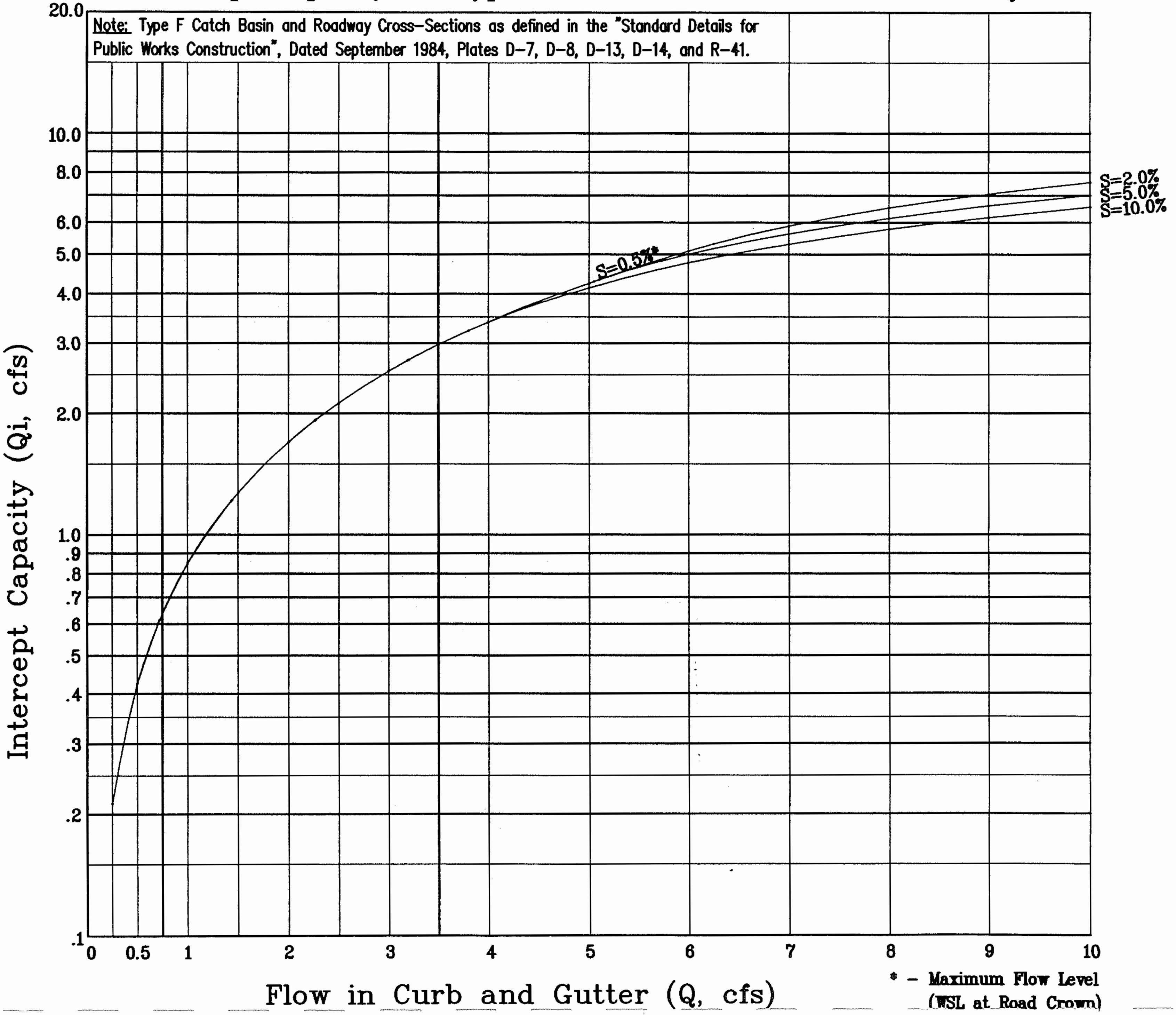
* – Maximum Flow Level (WSL at Road Crown)

Plate 39

Intercept Capacity for Type E Catch Basin on a Minor Roadway



Intercept Capacity for Type F Catch Basin on a Minor Roadway



Intercept Capacity for Type A,B,C Catch Basin on a Dead-End Roadway

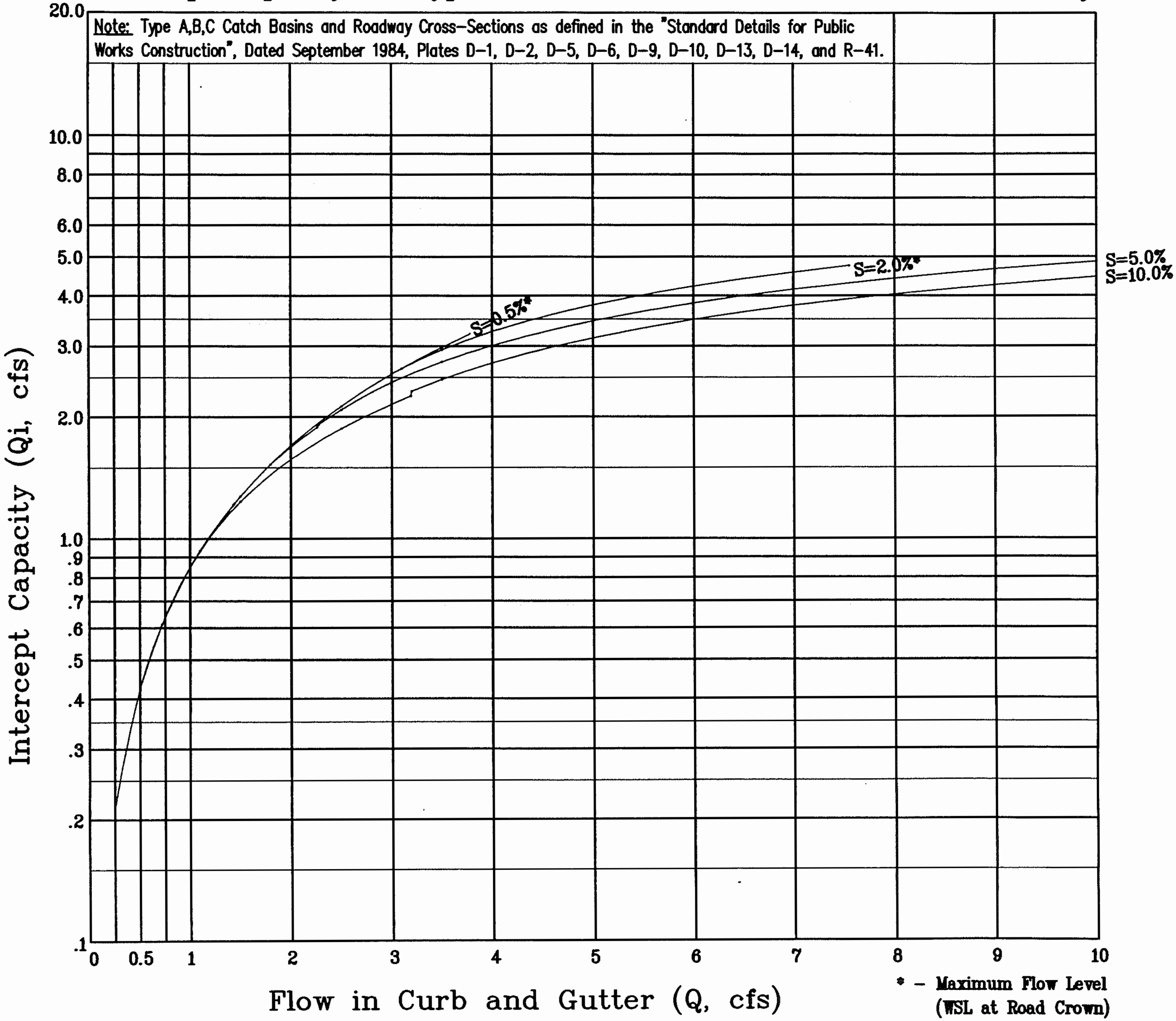
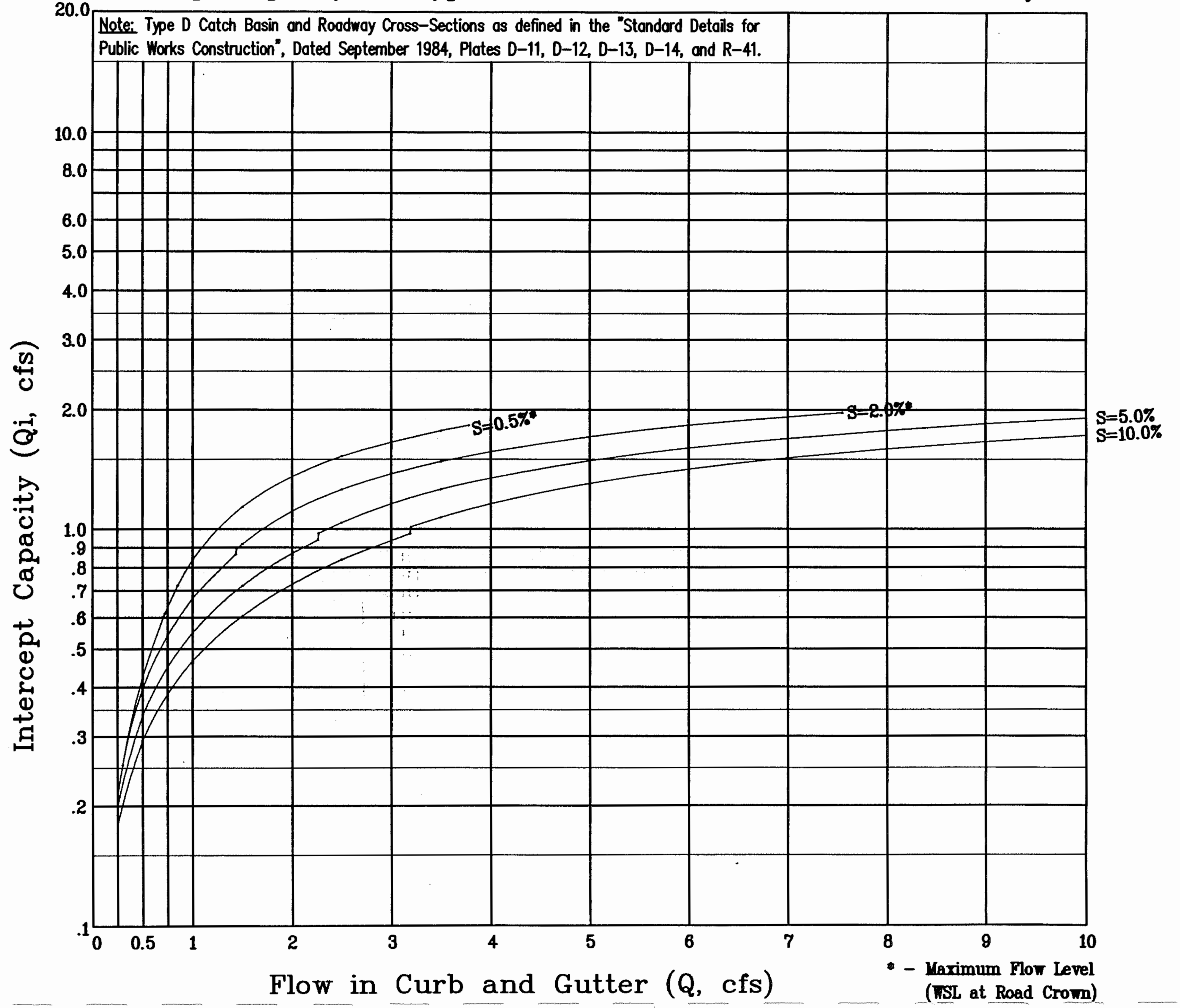


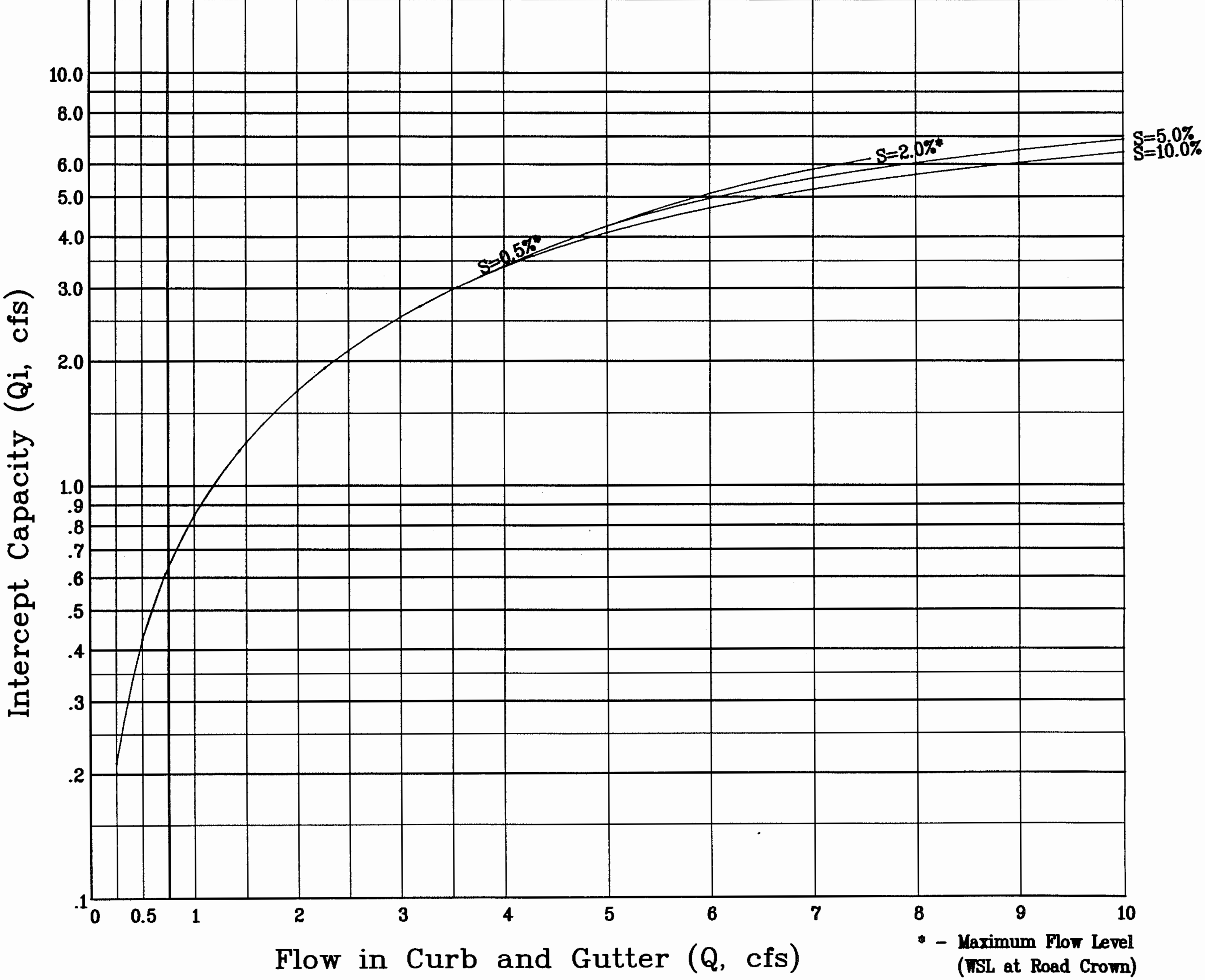
Plate 42

Intercept Capacity for Type D Catch Basin on a Dead-End Roadway



Intercept Capacity for Type E Catch Basin on a Dead-End Roadway

Note: Type E Catch Basin and Roadway Cross-Sections as defined in the "Standard Details for Public Works Construction", Dated September 1984, Plates D-3, D-4, D-13, D-14, and R-41.



Intercept Capacity for Type F Catch Basin on a Dead-End Roadway

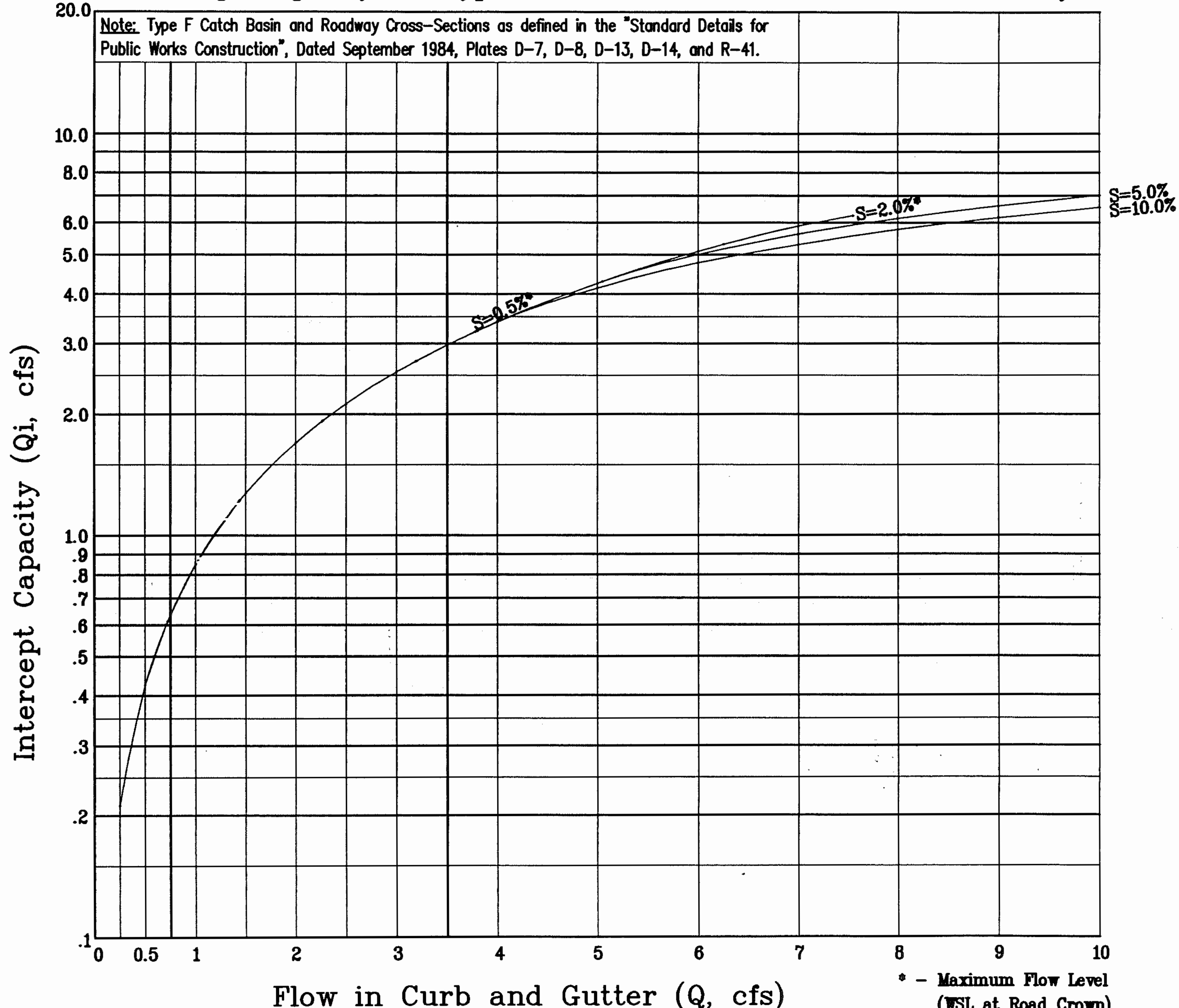


Plate 45

ROCK PROTECTION AT OUTFALLS						
Discharge Velocity at Design Flow (fps)		REQUIRED PROTECTION				
Greater than	Less than or equal to	Minimum Dimensions				
		Type	Thickness	Width	Length	Height
0	5	Rock Lining[1]	1 foot	Diameter + 6 feet	8 feet or 4 x Diameter, whichever is greater	Crown + 1 foot
5	10	Rip Rap[2]	2 feet	Diameter + 6 feet or 3 x Diameter, whichever is greater	12 feet or 4 x Diameter, whichever is greater	Crown + 1 foot
10	20	Gabion Outfall	As Required	As Required	As Required	Crown + 1 foot
20	N/A	Engineered Energy Dissipater Required				

(1) Rock lining shall be quarry spalls with gradation as follows:

Passing 8-inch square sieve: 100%

Passing 3-inch square sieve: 40% max.

Passing 3/4-inch square sieve: 0% max.

(2) Rip rap shall be reasonably well graded with gradation as follows:

Maximum stone size, 24" (nominal diameter)

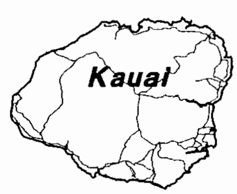
Median stone size, 16"

Minimum stone size, 4"

Note: Rip rap sizing governed by side slopes on outlet channel, assumed to be approximately 3:1.

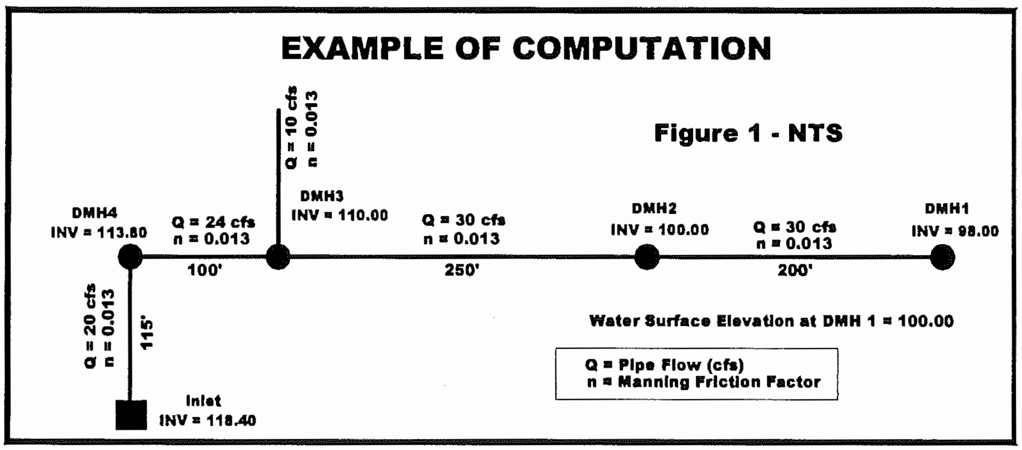
Source: Surface Water Design Manual, King County, Washington, February 1996

Appendix A - Examples



EXAMPLE: PIPE SYSTEM ANALYSIS

Given:



Problem: Determine pipe sizes and hydraulic gradients for Figure 1.

Solution:

<u>Determine Pipe Sizes</u>

Make a preliminary determination of all pipe sizes for the data given in Figure 1 using pipe flow charts. As an example on how to use the chart, assume that for an 18" pipe, it was determined through investigation and analysis of the area that the flow (Q) in the pipe was found to equal 20 cfs. The normal depth of flow in the pipe was determined to be two-thirds full or 12 inches, and the Manning Roughness Constant (n) was given as 0.013 based on the material of the pipe. From the pipe flow chart for an 18" pipe, the given Q, n, and normal depth of flow values would match to a pipe slope (S) of 0.06 or 6%. If this 6% slope is acceptable to the drainage flow and site conditions of the project, an 18" pipe can be used for the drain. After the size of the pipe is determined, the same pipe flow chart can be used to determine the flow velocity (V) in the chart by extending a straight line to the left vertical axis.

For this exercise, the values for V and S values have already been determined and are listed in Figure 2.

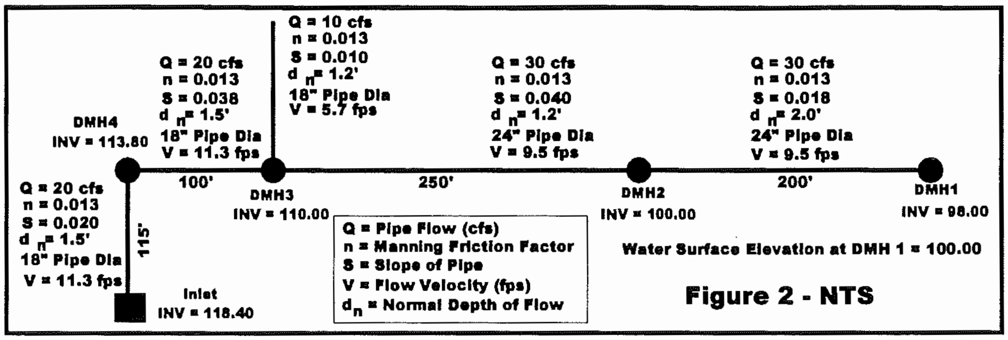


Figure 2 - NTS

Compute Hydraulic Gradient

1. The controlling grade at DMH 1 is 100.00, as noted in Figure 1 as the water surface elevation. Begin by identifying locations along the alignment for entrance and exit losses.

2. With the selected pipe size between DMH 1 and DMH 2 having a 24" diameter, and an S = 0.010, compute the head loss in the pipe by the formula:

(1) $h = SL$ or
(2) $h_f = S_f L$, which ever condition controls

h = Elevation Head Loss
h_f = Friction Head Loss
S = Slope of the pipe
S_f = Friction Slope (Used when pipe flowing full)
L = Length of pipe or channel

The 24" pipe between DMH 1 and DMH 2 is flowing full. This can be verified by matching the given Q, n, and V values to the corresponding depth of flow value on the pipe flow chart and comparing the depth of flow value to the diameter of the pipe. If the depth of flow value is equal to the diameter of the pipe, the pipe is flowing full. Based on the values V=9.5 fps, n=0.013, and Q = 30 cfs, the friction slope or S_f value taken from the pipe flow chart is equal to 0.018.

Substituting into equation (2), the head loss between DMH 1 and DMH 2 is calculated as follows:

$$\mathbf{h_f = S_f L = (0.018)(200') = 3.60\ feet}$$

The downstream hydraulic gradient at DMH 2 is equal to the controlling grade at DMH 1 plus the head loss or

100.00 + 3.60 = <u>103.60</u>

3. Since the pipe is flowing full, compute the upstream hydraulic gradient at DMH 2 by computing the ponded water surface elevation at DMH 2. This is accomplished by adding the invert of DMH 2 to the headwall value obtained from Plate 19. This should be based on D = 24", Q=30 cfs, and a square edge entrance type. From Plate 19, HW/D = 2.35 and HW = 4.70 feet.

   The upstream hydraulic gradient at DMH 2 is:

   100.00 + 4.70= <u>104.70</u>

4. Compute the head loss elevation between DMH 2 and DMH 3 from equation (1)

   h = SL = (0.040)(250) = 10.00 feet

   Since the pipe is not flowing full as determined by the pipe flow chart, the elevation head loss and the normal depth must be added to the invert of DMH 2. Therefore, the downstream hydraulic gradient at DMH 3 is:

   100.00 + 10.00 + 1.2 = <u>111.20</u>

   (where D_n = 1.2 as given in Fig. 1)

5. Compute the upstream hydraulic gradient at DMH 3 by computing the ponded water surface elevation at DMH 3.

   Inlet control loss for Q = 30 cfs, D = 24" from Plate 19

   $$\mathbf{\frac{HW}{D} = 2.35}$$

H = Headwater Depth D = Diameter of Pipe

   Thus H = 4.70 feet

   The upstream hydraulic gradient at DMH 3 is:

   110.00 + 4.70 = <u>114.70</u>

6. With the selected pipe size between DMH 3 and DMH 4 having an 18" diameter, and an S = 0.038, compute the head loss in the pipe by the formula:

$h_f = S_f L = (0.038)(100') = 3.80$ feet

The downstream hydraulic gradient at DMH 4 is:

114.70 + 3.80 = <u>118.50</u>

since the tailwater condition of the pipe is submerged.

7. Inlet control losses for Q=20 cfs, D=18" is

HW/D = 4.20
H = 6.30 feet

The upstream hydraulic gradient due to inlet control at DMH 4 is

113.80 + 6.30 = <u>120.10</u>

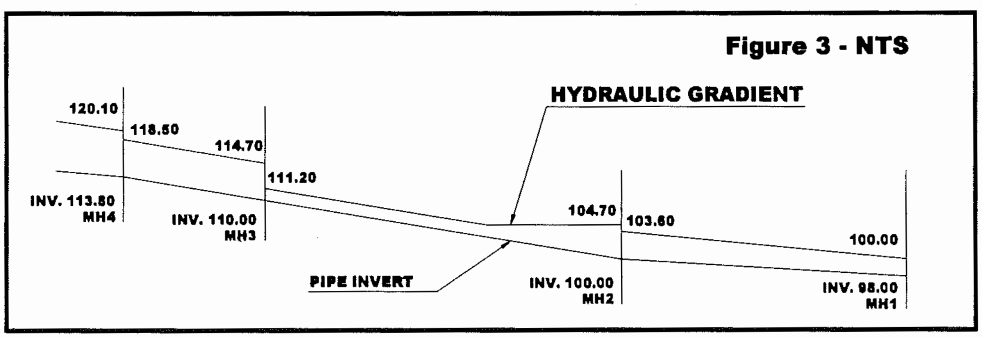


Figure 3 - NTS

EXAMPLE: DETENTION BASIN CALCULATION

SITUATION:

A 120-acre forest-land watershed, with a single runoff discharge point, is planned to be developed into a residential subdivision (38% impervious area). The development is to include a single detention basin in order to control runoff and avoid any increases to existing peak discharge rates. Use the peak discharge due to the 2-year, 24-hour storm and the 100-year, 24-hour storm.

SOLUTION:

Using the Standard SCS 24-hour Type I Dimensionless Hydrograph, the existing inflow hydrographs and peak discharge rates for the 2-year, 24-hour and 100-year, 24-hour storms can be determined. The 2-year and 100-year, 24-hour rainfall hydrographs for the developed conditions were also determined.

Prior to detention basin design, it is best to estimate the storage volume required to reduce "developed" peak discharge rates to pre-development levels. Based on the TR-55 (Second Ed., June 1986) graph for the detention basin routing of rainfall Type I, a good estimation of required storage volume can be found.

Design a detention basin to meet detention time and outflow requirements. Utilize an outlet feature (i.e: weir, flume, discharge pipe, etc...) that provides a suitable stage-discharge curve for the 2-year storm and if necessary for the 100-year storm as well.

1. Determine the peak runoff rates for the existing condition.

   Watershed Area

   A = 120 acres (or 0.1875 sq. mi.)

   Runoff Curve Number

   CN = 60 (as determined by soil type and watershed characteristics)

   Time of Concentration

   T_c = 0.35 hr. (based on watershed length and slope)

   Rainfall

   $P_{2\text{-}24}$ = 6.0 inches (2-yr, 24-hr storm)

   $P_{100\text{-}24}$ = 14.0 inches (100-yr, 24-hr storm)

   The resulting peak $Q_{2\text{-}24}$ = 74 cfs and the peak $Q_{100\text{-}24}$ = 398 cfs.

2. Determine the peak discharge rates for the developed condition.

Watershed Area

$A = 120$ acres (or 0.1875 sq. mi.)

Runoff Curve Number

$CN = 75$ (as determined by soil type and watershed characteristics)

Time of Concentration

$T_c = 0.20$ hr. (based on watershed length and slope)

Rainfall

$P_{2\text{-}24} = 6.0$ inches (2-yr, 24-hr storm)

$P_{100\text{-}24} = 14.0$ inches (100-yr, 24-hr storm)

Using the same methods, the resulting peak $Q_{2\text{-}24} = 201$ cfs and the peak $Q_{100\text{-}24} = 683$ cfs.

3. The designer is expected to be able to incorporate the proposed detention facility into the layout of the proposed development as well as consult with hydraulic reference manuals to determine the outflow characteristics for given outlet structures.

For this example, the peak inflow and outflow data for the 2-year and 100-year storms are entered into the TR-55 chart and the approximate detention basin storage volumes of 4.6 acre-feet and 12.8 acre-feet are determined.

A detention basin is selected, utilizing the estimated storage volumes along with the required discharges. The stage-discharge relationship is then entered into the routing forms to test the performance of the outlet controls. The following equations are used to determine the outlet control discharges:

$$Q_{Holes} = 9CA\,(2gH)^{1/2}$$
$$Q_{Weir} = CLH^{3/2}$$
$$Q_{Orifice} = CA(2gH)^{1/2}$$
$$Q_{Berm} = 3.21\,LH^{1.5} + 2.4\,H^{2.5}$$

After each iterative routing, the size of the detention basin and outlet controls are adjusted to meet the pre-development conditions. A 2-stage detention basin, comprising of a vertical perforated pipe with 2-inch diameter holes at 4-inch centers around its circumference and a hooded riser outlet, is selected. The 3.94-acre detention basin has the following characteristics:

Stage (ft)	Discharge (cfs)	Storage (ac-ft)
0.0	0.00	0.000
0.5	5.00	1.984
1.0	11.88	3.997
1.5	32.78	6.039
2.0	72.27	8.111
2.5	118.92	10.212
3.0	159.46	12.344
3.5	247.09	14.506
4.0	394.41	16.699

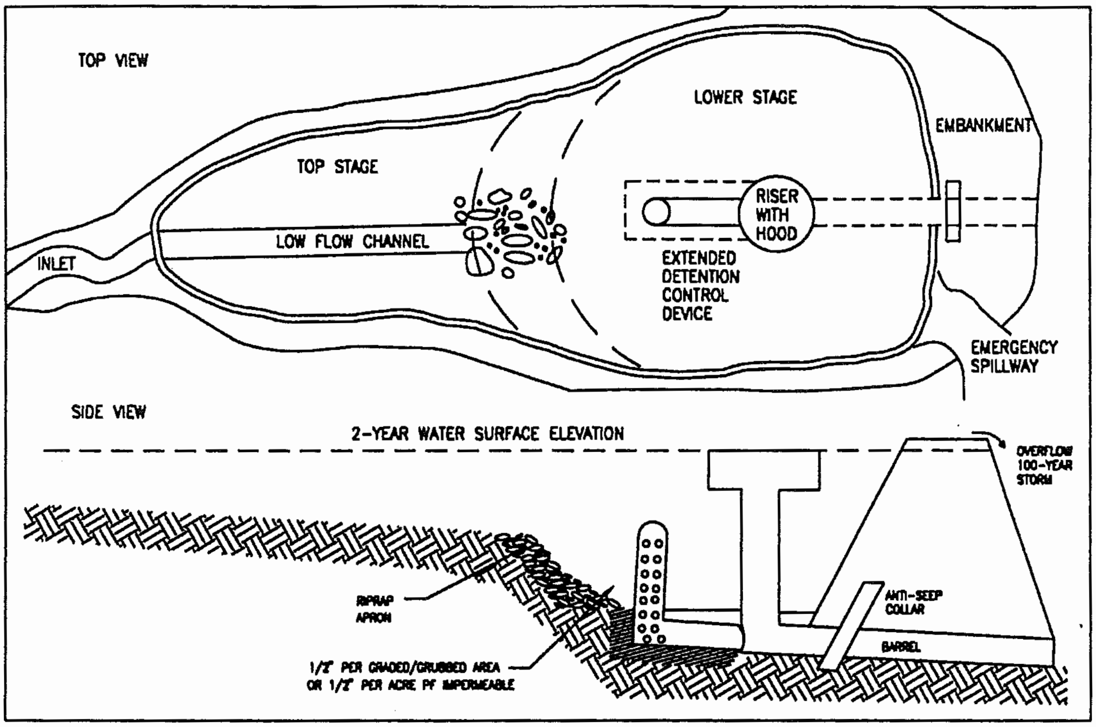
The routed peak discharge rates determined by the routing forms are:

$Q_{2\text{-}24}$ = 72 cfs at elevation 2.1 ft and storage of 8.54 ac-ft

$Q_{100\text{-}24}$ = 394 cfs at elevation 4.7 ft

The resulting peak discharges are less than or equal to the "existing" peak discharges for the given storms and the design is accepted.

It should be noted that retention of runoff volume may be incorporated into the design of the detention basin. In that case, it is left up to the designer to determine the available volume of the detention basin at the beginning of the storm as it relates to the storage-discharge relationship. Additional storage capacity for sediment and debris are included in the design of the detention basin, to provide adequate water quality measures.


TOP VIEW
LOWER STAGE
EMBANKMENT
TOP STAGE
RISER WITH HOOD
LOW FLOW CHANNEL
INLET
EXTENDED DETENTION CONTROL DEVICE
EMERGENCY SPILLWAY
SIDE VIEW
2-YEAR WATER SURFACE ELEVATION
OVERFLOW 100-YEAR STORM
RIPRAP APRON
ANTI-SEEP COLLAR
BARREL
1/2" PER GRADED/GRUBBED AREA
OR 1/2" PER ACRE PF IMPERMEABLE

STAGE vs DISCHARGE

DATA:

Area est.	171600 sf	
Basin Bottom Dimension:	440 ft	
Basin Bottom Dimension:	390 ft.	
Bottom area of Basin	171600 sf	3.94 acres
Perforated Pipe/Orifice Dia.	60.00 in.	
Diameter Holes	2.00 in	
Perforated Pipe Length	1.00 ft	
Area of Pipe	19.63 ft	
Circum. of Pipe/Size of Weir	15.71 ft	
Area of Hole	0.0218 ft	
Spacing between holes	4.00 in	
Number of Holes	47.00	
Coefficient of Discharge	0.65	
Coefficient of Horz.Orifice	0.60	Based on the ASCE Continuing Education Services
Thickness of Pipe	6.00 in	
Weir Length	60.00 ft	
TAN	3.000	3:1 side slopes
Coefficient of Weir	0.600	
Hydrograph Time Interval:	6 min.	
Coefficient of Broad-Crested Weir		Based on Table 5-3 Handbook of Hydraulics

CALCULATIONS:

Elevation (msl)	Depth (ft)	Row 1 Perf Pipe Q (cfs)	Row 2 Perf Pipe Q (cfs)	Row 3 Perf Pipe Q (cfs)	Orifice Riser Q (cfs)	Weir Riser Q (cfs)	Weir or Orifice Control Q (cfs)	Total 2-yr Control Q, cfs	Berm Weir Q (cfs)	Total Outlet Q (cfs)
196.0	0.0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
196.1	0.1	0.69	0.00	0.00	0.00	0.00	0.00	0.69	0.00	0.69
196.2	0.2	1.83	0.00	0.00	0.00	0.00	0.00	1.83	0.00	1.83
196.3	0.3	2.49	0.00	0.00	0.00	0.00	0.00	2.49	0.00	2.49
196.4	0.4	3.01	0.00	0.00	0.00	0.00	0.00	3.01	0.00	3.01
196.5	0.5	3.45	1.54	0.00	0.00	0.00	0.00	5.00	0.00	5.00
196.6	0.6	3.84	2.29	0.00	0.00	0.00	0.00	6.13	0.00	6.13
196.7	0.7	4.20	2.85	0.00	0.00	0.00	0.00	7.05	0.00	7.05
196.8	0.8	4.53	3.31	1.20	0.00	0.00	0.00	9.04	0.00	9.04
196.9	0.9	4.83	3.72	2.07	0.00	0.00	0.00	10.62	0.00	10.62
197.0	1.0	5.12	4.09	2.67	0.00	0.00	0.00	11.88	0.00	11.88
197.1	1.1	5.39	4.42	3.16	29.90	1.39	1.39	14.37	0.00	14.37
197.2	1.2	5.65	4.73	3.59	42.28	3.93	3.93	17.91	0.00	17.91
197.3	1.3	5.90	5.03	3.97	51.78	7.23	7.23	22.12	0.00	22.12
197.4	1.4	6.14	5.30	4.31	59.79	11.60	11.60	27.36	0.00	27.36
197.5	1.5	6.37	5.57	4.63	66.85	16.22	16.22	32.78	0.00	32.78
197.6	1.6	6.59	5.82	4.93	73.23	22.49	22.49	39.82	0.00	39.82
197.7	1.7	6.80	6.06	5.21	79.10	28.33	28.33	46.41	0.00	46.41
197.8	1.8	7.01	6.29	5.48	84.56	37.09	37.09	55.87	0.00	55.87
197.9	1.9	7.21	6.51	5.74	89.69	44.26	44.26	63.72	0.00	63.72
198.0	2.0	7.40	6.73	5.98	94.54	52.15	52.15	72.27	0.00	72.27
198.1	2.1	7.60	6.94	6.21	99.16	60.17	60.17	80.91	0.00	80.91
198.2	2.2	7.78	7.14	6.44	103.57	68.55	68.55	89.92	0.00	89.92

CALCULATIONS:

Elevation (msl)	Depth (ft)	Row 1 Perf Pipe Q (cfs)	Row 2 Perf Pipe Q (cfs)	Row 3 Perf Pipe Q (cfs)	Orifice Riser Q (cfs)	Weir Riser Q (cfs)	Weir or Orifice Control Q (cfs)	Total 2-yr Control Q, cfs	Berm Weir Q (cfs)	Total Outlet Q (cfs)
198.3	2.3	7.96	7.34	6.66	107.79	77.30	77.30	99.26	0.00	99.26
198.4	2.4	8.14	7.53	6.87	111.86	86.39	86.39	108.93	0.00	108.93
198.5	2.5	8.31	7.72	7.08	115.79	95.81	95.81	118.92	0.00	118.92
198.6	2.6	8.49	7.90	7.27	119.59	105.55	105.55	129.21	0.00	129.21
198.7	2.7	8.65	8.08	7.47	123.27	115.59	115.59	139.80	0.00	139.80
198.8	2.8	8.82	8.26	7.66	126.84	125.94	125.94	150.67	0.00	150.67
198.9	2.9	8.98	8.43	7.84	130.32	136.58	130.32	155.56	0.00	155.56
199.0	3.0	9.13	8.60	8.02	133.70	147.50	133.70	159.46	0.00	159.46
199.1	3.1	9.29	8.76	8.20	137.00	158.70	137.00	163.25	6.11	169.37
199.2	3.2	9.44	8.92	8.37	140.23	170.17	140.23	166.97	17.36	184.33
199.3	3.3	9.59	9.08	8.54	143.38	181.91	143.38	170.60	32.03	202.62
199.4	3.4	9.74	9.24	8.71	146.46	193.90	146.46	174.15	49.50	223.65
199.5	3.5	9.89	9.39	8.87	149.48	206.14	149.48	177.63	69.46	247.09
199.6	3.6	10.03	9.54	9.03	152.44	218.63	152.44	181.05	91.66	272.71
199.7	3.7	10.17	9.69	9.19	155.35	231.37	155.35	184.40	115.96	300.35
199.8	3.8	10.31	9.84	9.34	158.20	244.34	158.20	187.69	142.22	329.91
199.9	3.9	10.45	9.98	9.49	161.00	257.55	161.00	190.92	170.36	361.29
200.0	4.0	10.59	10.12	9.64	163.75	270.98	163.75	194.10	200.30	394.41
200.7	4.69	11.48	11.06	10.62	181.61	369.66	181.61	214.76	451.75	666.51

STAGE vs STORAGE

DATA:

Area	171600 sf
Basin Bottom Dimension, ft	440 ft
Basin Bottom Dimension, ft	390 ft
Design 2-yr Depth	2 ft
Side Slopes	3:1 H:V

Hydrograph Time Interval: 6 min.
(To be incorporated with Rainfall Hydrograph for Routing Purposes)

CALCULATIONS:

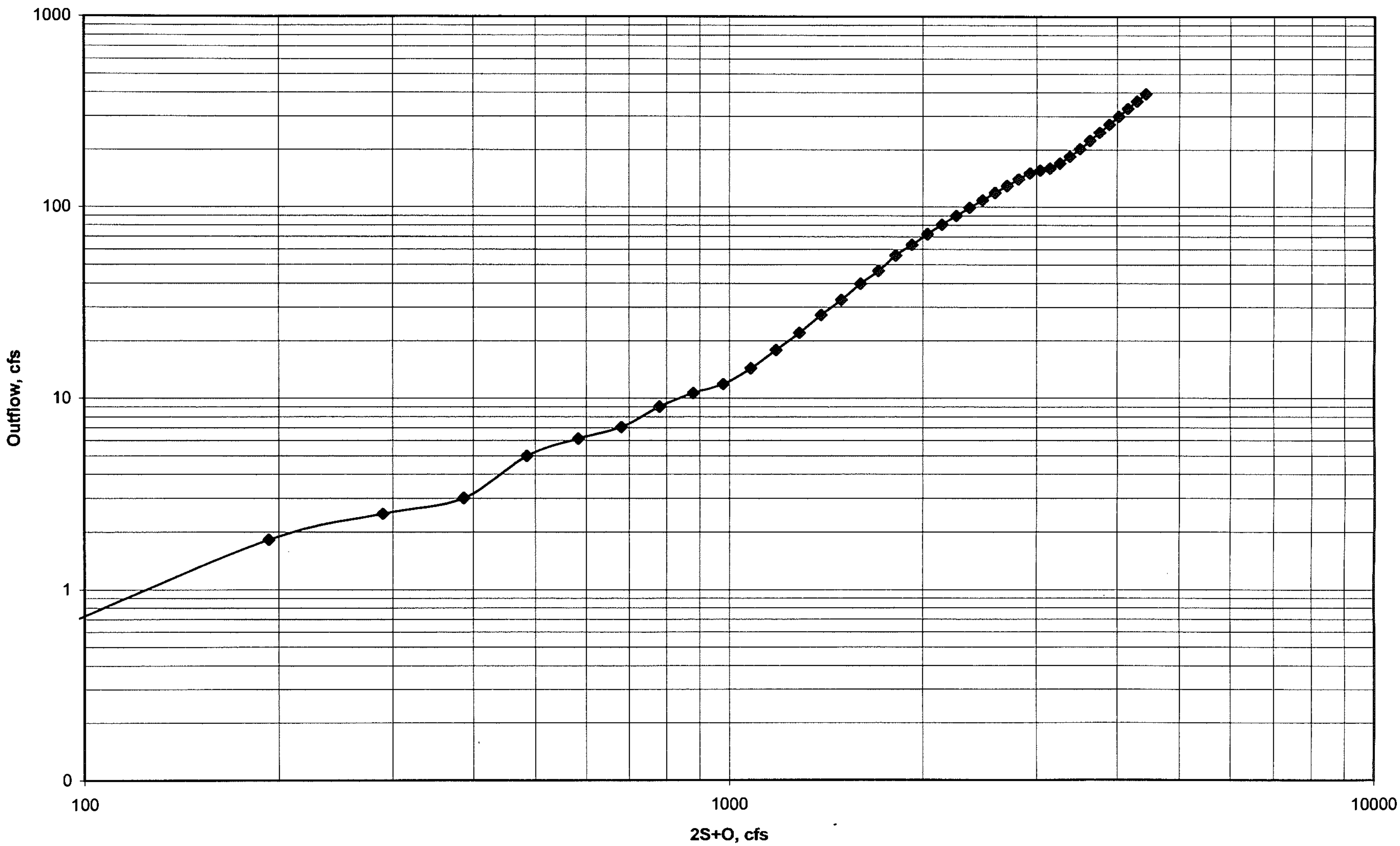
Elevation ft	Depth ft	Basin Area acres	Average Basin Area acres	Increment Depth ft	Increment Volume cu.ft	Cumulative Volume cu.ft	Total Volume acre-ft
196.0	0.000	171600		0.0	0	0	0.00
196.100	0.100	172098	171849	0.1	17185	17185	0.40
196.200	0.200	172597	172348	0.1	17235	34420	0.79
196.300	0.300	173097	172847	0.1	17285	51704	1.19
196.400	0.400	173598	173348	0.1	17335	69039	1.59
196.500	0.500	174099	173848	0.1	17385	86424	1.99
196.600	0.600	174601	174350	0.1	17435	103859	2.39
196.700	0.700	175104	174852	0.1	17485	121344	2.79
196.800	0.800	175607	175355	0.1	17536	138880	3.19
196.900	0.900	176111	175859	0.1	17586	156466	3.60
197.000	1.000	176616	176364	0.1	17636	174102	4.00
197.100	1.100	177122	176869	0.1	17687	191789	4.41
197.200	1.200	177628	177375	0.1	17737	209526	4.82
197.300	1.300	178135	177881	0.1	17788	227315	5.23
197.400	1.400	178643	178389	0.1	17839	245153	5.64
197.500	1.500	179151	178897	0.1	17890	263043	6.05
197.600	1.600	179660	179406	0.1	17941	280984	6.46
197.700	1.700	180170	179915	0.1	17992	298975	6.88
197.800	1.800	180681	180425	0.1	18043	317018	7.29
197.900	1.900	181192	180936	0.1	18094	335111	7.71
198.000	2.000	181704	181448	0.1	18145	353256	8.12
198.100	2.100	182217	181960	0.1	18196	371452	8.54
198.200	2.200	182730	182474	0.1	18247	389700	8.96
198.300	2.300	183244	182987	0.1	18299	407998	9.38
198.400	2.400	183759	183502	0.1	18350	426348	9.81
198.500	2.500	184275	184017	0.1	18402	444750	10.23
198.600	2.600	184791	184533	0.1	18453	463203	10.65
198.700	2.700	185308	185050	0.1	18505	481708	11.08
198.800	2.800	185826	185567	0.1	18557	500265	11.51
198.900	2.900	186345	186086	0.1	18609	518874	11.93
199.000	3.000	186864	186604	0.1	18660	537534	12.36
199.100	3.100	187384	187124	0.1	18712	556247	12.79
199.200	3.200	187905	187644	0.1	18764	575011	13.23
199.300	3.300	188426	188165	0.1	18817	593828	13.66
199.400	3.400	188948	188687	0.1	18869	612696	14.09
199.500	3.500	189471	189210	0.1	18921	631617	14.53
199.600	3.600	189995	189733	0.1	18973	650590	14.96
199.700	3.700	190519	190257	0.1	19026	669616	15.40
199.800	3.800	191044	190781	0.1	19078	688694	15.84
199.900	3.900	191570	191307	0.1	19131	707825	16.28
200.000	4.000	192096	191833	0.1	19183	727008	16.72
200.100	4.100	192623	192360	0.1	19236	746244	17.16
200.200	4.200	193151	192887	0.1	19289	765533	17.61
200.300	4.300	193680	193415	0.1	19342	784874	18.05
200.400	4.400	194209	193944	0.1	19394	804269	18.50
200.500	4.500	194739	194474	0.1	19447	823716	18.95
200.600	4.600	195270	195004	0.1	19500	843217	19.39
200.700	4.700	195801	195536	0.1	19554	862770	19.84

ROUTING CURVE SUMMARY

CALCULATIONS:

Elevation (msl)	Storage Vol. (ac-ft)	Outlet O Q (cfs)	O+2S (cfs)
196.0	0.000	0.00	0.00
196.1	0.395	0.69	96.16
196.2	0.790	1.83	193.05
196.3	1.187	2.49	289.74
196.4	1.585	3.01	386.56
196.5	1.984	5.00	485.13
196.6	2.384	6.13	583.14
196.7	2.786	7.05	681.19
196.8	3.188	9.04	780.61
196.9	3.592	10.62	879.90
197.0	3.997	11.88	979.15
197.1	4.403	14.37	1079.91
197.2	4.810	17.91	1182.00
197.3	5.219	22.12	1285.05
197.4	5.628	27.36	1389.41
197.5	6.039	32.78	1494.24
197.6	6.451	39.82	1600.98
197.7	6.864	46.41	1707.54
197.8	7.279	55.87	1817.27
197.9	7.694	63.72	1925.68
198.0	8.111	72.27	2035.07
198.1	8.529	80.91	2144.85
198.2	8.948	89.92	2255.27
198.3	9.368	99.26	2366.32
198.4	9.790	108.93	2477.99
198.5	10.212	118.92	2590.27
198.6	10.636	129.21	2703.15
198.7	11.061	139.80	2816.61
198.8	11.488	150.67	2930.65
198.9	11.915	155.56	3039.01
199.0	12.344	159.46	3146.66
199.1	12.774	169.37	3260.62
199.2	13.205	184.33	3379.93
199.3	13.637	202.62	3502.86
199.4	14.071	223.65	3628.83
199.5	14.506	247.09	3757.50
199.6	14.942	272.71	3888.65
199.7	15.379	300.35	4022.13
199.8	15.818	329.91	4157.82
199.9	16.258	361.29	4295.63
200.0	16.699	394.41	4435.47
200.7	19.776	666.51	5452.24

Routing Curve



Watershed Desc: Developed Conditions -2yr/24hr

Hydrograph Routing Calculations

Time Hr	(1) I_1 cfs	(2) I_1+I_2 cfs	(3) 2S-O cfs	(4) O_2+2S_2 cfs	(5) Elev ft
0	0.000	0.000	0.000	0.000	196.00
0.1	0.000	0.000	0.000	0.000	196.00
0.2	0.000	0.000	0.000	0.000	196.00
0.3	0.000	0.000	0.000	0.000	196.00
0.4	0.000	0.000	0.000	0.000	196.00
0.5	0.000	0.000	0.000	0.000	196.00
0.6	0.000	0.000	0.000	0.000	196.00
0.7	0.000	0.000	0.000	0.000	196.00
0.8	0.000	0.000	0.000	0.000	196.00
0.9	0.000	0.000	0.000	0.000	196.00
1	0.000	0.000	0.000	0.000	196.00
1.1	0.000	0.000	0.000	0.000	196.00
1.2	0.000	0.000	0.000	0.000	196.00
1.3	0.000	0.000	0.000	0.000	196.00
1.4	0.000	0.000	0.000	0.000	196.00
1.5	0.000	0.000	0.000	0.000	196.00
1.6	0.000	0.000	0.000	0.000	196.00
1.7	0.000	0.000	0.000	0.000	196.00
1.8	0.000	0.000	0.000	0.000	196.00
1.9	0.000	0.000	0.000	0.000	196.00
2	0.000	0.000	0.000	0.000	196.00
2.1	0.000	0.000	0.000	0.000	196.00
2.2	0.000	0.000	0.000	0.000	196.00
2.3	0.000	0.000	0.000	0.000	196.00
2.4	0.000	0.000	0.000	0.000	196.00
2.5	0.000	0.000	0.000	0.000	196.00
2.6	0.000	0.000	0.000	0.000	196.00
2.7	0.000	0.000	0.000	0.000	196.00
2.8	0.000	0.000	0.000	0.000	196.00
2.9	0.000	0.000	0.000	0.000	196.00
3	0.000	0.000	0.000	0.000	196.00
3.1	0.000	0.000	0.000	0.000	196.00
3.2	0.000	0.000	0.000	0.000	196.00
3.3	0.000	0.000	0.000	0.000	196.00
3.4	0.000	0.000	0.000	0.000	196.00
3.5	0.000	0.000	0.000	0.000	196.00
3.6	0.000	0.000	0.000	0.000	196.00
3.7	0.000	0.000	0.000	0.000	196.00
3.8	0.000	0.000	0.000	0.000	196.00
3.9	0.000	0.000	0.000	0.000	196.00
4	0.000	0.000	0.000	0.000	196.00
4.1	0.000	0.000	0.000	0.000	196.00
4.2	0.000	0.000	0.000	0.000	196.00
4.3	0.000	0.000	0.000	0.000	196.00
4.4	0.000	0.000	0.000	0.000	196.00
4.5	0.000	0.000	0.000	0.000	196.00
4.6	0.000	0.000	0.000	0.000	196.00
4.7	0.000	0.000	0.000	0.000	196.00
4.8	0.000	0.000	0.000	0.000	196.00
4.9	0.000	0.000	0.000	0.000	196.00
5	0.000	0.000	0.000	0.000	196.00
5.1	0.000	0.000	0.000	0.000	196.00
5.2	0.000	0.000	0.000	0.000	196.00
5.3	0.000	0.000	0.000	0.000	196.00
5.4	0.000	0.000	0.000	0.000	196.00
5.5	0.000	0.004	0.000	0.004	196.00
5.6	0.004	0.041	0.004	0.045	196.00
5.7	0.038	0.159	0.045	0.204	196.00
5.8	0.121	0.357	0.204	0.561	196.00
5.9	0.236	0.608	0.561	1.168	196.00
6	0.372	0.888	1.168	2.056	196.00
6.1	0.516	1.184	2.056	3.240	196.00

Watershed Desc: Developed Conditions -2yr/24hr

Hydrograph Routing Calculations

Time Hr	(1) I_1 cfs	(2) I_1+I_2 cfs	(3) 2S-O cfs	(4) O_2+2S_2 cfs	(5) Elev ft
6.2	0.668	1.506	3.240	4.746	196.00
6.3	0.839	1.879	4.746	6.625	196.00
6.4	1.040	2.295	6.625	8.920	196.00
6.5	1.255	2.731	8.920	11.651	196.00
6.6	1.476	3.207	11.651	14.858	196.00
6.7	1.731	3.748	14.858	18.606	196.00
6.8	2.017	4.314	18.606	22.919	196.00
6.9	2.297	4.896	22.919	27.815	196.00
7	2.599	5.536	27.815	33.351	196.00
7.1	2.937	6.227	33.351	39.578	196.00
7.2	3.289	6.897	39.578	46.475	196.00
7.3	3.608	7.542	46.475	54.017	196.00
7.4	3.934	8.195	54.017	62.211	196.00
7.5	4.261	8.829	62.211	71.041	196.00
7.6	4.568	9.456	71.041	80.497	196.00
7.7	4.888	10.107	80.497	90.604	196.00
7.8	5.219	10.745	90.604	101.349	196.00
7.9	5.525	11.377	99.968	111.345	196.10
8	5.851	12.038	109.964	122.002	196.10
8.1	6.187	12.759	120.621	133.380	196.10
8.2	6.572	13.711	131.999	145.710	196.10
8.3	7.139	15.058	144.329	159.386	196.10
8.4	7.919	16.860	158.005	174.865	196.10
8.5	8.941	19.008	173.484	192.492	196.10
8.6	10.068	21.321	191.111	212.432	196.20
8.7	11.253	23.805	208.778	232.583	196.20
8.8	12.552	26.509	228.929	255.438	196.20
8.9	13.957	29.423	251.784	281.207	196.20
9	15.466	32.541	277.553	310.095	196.20
9.1	17.075	35.916	305.115	341.031	196.30
9.2	18.841	39.813	336.052	375.865	196.30
9.3	20.973	44.484	370.886	415.370	196.30
9.4	23.511	49.829	409.350	459.179	196.40
9.5	26.318	55.750	453.160	508.910	196.40
9.6	29.432	65.235	498.917	564.152	196.50
9.7	35.803	86.664	554.159	640.822	196.50
9.8	50.861	132.474	628.553	761.027	196.60
9.9	81.614	227.988	746.933	974.921	196.70
10	146.375	347.504	953.674	1301.178	196.90
10.1	201.129	387.111	1256.937	1644.048	197.30
10.2	185.982	329.557	1564.405	1893.962	197.60
10.3	143.575	258.596	1782.221	2040.817	197.80
10.4	115.021	209.636	1896.286	2105.923	198.00
10.5	94.616	173.583	1961.392	2134.975	198.00
10.6	78.968	145.758	1990.445	2136.203	198.00
10.7	66.791	124.762	1991.672	2116.434	198.10
10.8	57.971	109.830	1971.903	2081.734	198.00
10.9	51.859	99.315	1937.203	2036.517	198.00
11	47.456	91.411	1891.987	1983.398	198.00
11.1	43.956	85.043	1855.963	1941.006	197.90
11.2	41.087	80.098	1813.570	1893.669	197.90
11.3	39.011	76.520	1781.928	1858.448	197.80
11.4	37.509	73.748	1746.708	1820.456	197.80
11.5	36.239	71.449	1708.715	1780.165	197.80
11.6	35.210	69.531	1687.349	1756.880	197.70
11.7	34.321	67.831	1664.065	1731.896	197.70
11.8	33.510	66.254	1639.081	1705.334	197.70
11.9	32.743	64.632	1625.691	1690.323	197.70
12	31.889	62.975	1610.680	1673.655	197.60
12.1	31.087	61.398	1594.012	1655.410	197.60
12.2	30.312	59.973	1575.767	1635.740	197.60
12.3	29.661	58.854	1556.096	1614.950	197.60

Watershed Desc: Developed Conditions -2yr/24hr

Hydrograph Routing Calculations

Time Hr	(1) I_1 cfs	(2) I_1+I_2 cfs	(3) 2S-O cfs	(4) O_2+2S_2 cfs	(5) Elev ft
12.4	29.193	57.803	1535.307	1593.110	197.60
12.5	28.610	56.567	1527.547	1584.114	197.50
12.6	27.957	55.437	1518.551	1573.988	197.50
12.7	27.481	54.478	1508.425	1562.903	197.50
12.8	26.998	53.506	1497.340	1550.846	197.50
12.9	26.509	52.523	1485.282	1537.806	197.50
13	26.014	51.530	1472.242	1523.772	197.50
13.1	25.515	50.527	1458.209	1508.735	197.50
13.2	25.011	49.514	1443.172	1492.686	197.50
13.3	24.503	48.493	1437.972	1486.465	197.50
13.4	23.990	47.463	1431.751	1479.214	197.40
13.5	23.473	46.540	1424.500	1471.040	197.40
13.6	23.066	45.563	1416.326	1461.889	197.40
13.7	22.496	44.324	1407.175	1451.498	197.40
13.8	21.827	43.152	1396.785	1439.937	197.40
13.9	21.325	42.247	1385.223	1427.470	197.40
14	20.922	41.271	1372.756	1414.027	197.40
14.1	20.349	40.136	1359.313	1399.449	197.40
14.2	19.787	39.248	1344.735	1383.984	197.40
14.3	19.461	38.622	1339.743	1378.365	197.30
14.4	19.161	38.036	1334.124	1372.160	197.30
14.5	18.875	37.590	1327.920	1365.509	197.30
14.6	18.714	37.342	1321.269	1358.611	197.30
14.7	18.628	37.213	1314.370	1351.583	197.30
14.8	18.585	37.155	1307.343	1344.498	197.30
14.9	18.570	37.024	1300.257	1337.281	197.30
15	18.454	36.731	1293.040	1329.771	197.30
15.1	18.277	36.458	1285.531	1321.989	197.30
15.2	18.180	36.311	1277.748	1314.059	197.30
15.3	18.131	36.125	1269.819	1305.943	197.30
15.4	17.993	35.796	1261.703	1297.499	197.30
15.5	17.803	35.616	1253.259	1288.874	197.30
15.6	17.813	35.524	1244.634	1280.158	197.30
15.7	17.711	35.251	1244.342	1279.593	197.20
15.8	17.540	35.103	1243.778	1278.881	197.20
15.9	17.563	35.029	1243.065	1278.094	197.20
16	17.467	34.766	1242.279	1277.045	197.20
16.1	17.299	34.505	1241.229	1275.734	197.20
16.2	17.206	34.363	1239.918	1274.281	197.20
16.3	17.157	34.174	1238.466	1272.639	197.20
16.4	17.017	33.838	1236.824	1270.662	197.20
16.5	16.821	33.532	1234.847	1268.379	197.20
16.6	16.711	33.362	1232.564	1265.926	197.20
16.7	16.651	33.273	1230.110	1263.383	197.20
16.8	16.622	33.232	1227.568	1260.800	197.20
16.9	16.611	33.102	1224.985	1258.087	197.20
17	16.492	32.799	1222.272	1255.071	197.20
17.1	16.308	32.511	1219.255	1251.766	197.20
17.2	16.203	32.350	1215.951	1248.300	197.20
17.3	16.146	32.145	1212.485	1244.630	197.20
17.4	15.999	31.796	1208.815	1240.611	197.20
17.5	15.797	31.597	1204.795	1236.392	197.20
17.6	15.800	31.489	1200.577	1232.066	197.20
17.7	15.688	31.195	1196.250	1227.446	197.20
17.8	15.507	31.030	1191.630	1222.660	197.20
17.9	15.523	30.941	1186.845	1217.786	197.20
18	15.418	30.658	1181.970	1212.628	197.20
18.1	15.240	30.378	1176.813	1207.191	197.20
18.2	15.138	30.220	1171.375	1201.595	197.20
18.3	15.082	30.014	1165.780	1195.794	197.20
18.4	14.932	29.660	1159.978	1189.638	197.20
18.5	14.727	29.336	1153.823	1183.158	197.20

Watershed Desc: Developed Conditions -2yr/24hr

Hydrograph Routing Calculations

Time Hr	(1) I_1 cfs	(2) I_1+I_2 cfs	(3) 2S-O cfs	(4) O_2+2S_2 cfs	(5) Elev ft
18.6	14.608	29.150	1147.343	1176.492	197.20
18.7	14.541	29.047	1147.753	1176.800	197.10
18.8	14.505	28.993	1148.061	1177.055	197.10
18.9	14.488	28.849	1148.315	1177.165	197.10
19	14.361	28.529	1148.426	1176.955	197.10
19.1	14.168	28.225	1148.216	1176.441	197.10
19.2	14.056	28.050	1147.702	1175.752	197.10
19.3	13.993	27.831	1147.013	1174.844	197.10
19.4	13.838	27.465	1146.105	1173.570	197.10
19.5	13.627	27.132	1144.831	1171.963	197.10
19.6	13.505	26.939	1143.224	1170.163	197.10
19.7	13.434	26.830	1141.424	1168.253	197.10
19.8	13.395	26.771	1139.514	1166.285	197.10
19.9	13.376	26.621	1137.546	1164.167	197.10
20	13.245	26.295	1135.428	1161.723	197.10
20.1	13.049	25.984	1132.984	1158.968	197.10
20.2	12.934	25.803	1130.228	1156.032	197.10
20.3	12.869	25.579	1127.293	1152.872	197.10
20.4	12.710	25.207	1124.132	1149.339	197.10
20.5	12.497	24.989	1120.600	1145.590	197.10
20.6	12.493	24.864	1116.851	1141.715	197.10
20.7	12.371	24.551	1112.976	1137.527	197.10
20.8	12.179	24.368	1108.788	1133.156	197.10
20.9	12.189	24.263	1104.417	1128.680	197.10
21	12.075	23.962	1099.941	1123.903	197.10
21.1	11.887	23.663	1095.164	1118.827	197.10
21.2	11.776	23.489	1090.088	1113.577	197.10
21.3	11.713	23.267	1084.838	1108.104	197.10
21.4	11.554	22.893	1079.365	1102.258	197.10
21.5	11.339	22.673	1073.519	1096.193	197.10
21.6	11.334	22.546	1067.454	1089.999	197.10
21.7	11.211	22.228	1061.260	1083.489	197.10
21.8	11.017	22.042	1054.750	1076.792	197.10
21.9	11.025	21.934	1053.031	1074.966	197.00
22	10.909	21.628	1051.205	1072.833	197.00
22.1	10.719	21.325	1049.072	1070.398	197.00
22.2	10.607	21.147	1046.637	1067.785	197.00
22.3	10.541	20.921	1044.024	1064.946	197.00
22.4	10.380	20.543	1041.185	1061.728	197.00
22.5	10.163	20.320	1037.968	1058.288	197.00
22.6	10.157	20.189	1034.527	1054.716	197.00
22.7	10.032	19.868	1030.956	1050.823	197.00
22.8	9.836	19.678	1027.063	1046.741	197.00
22.9	9.842	19.567	1022.980	1042.547	197.00
23	9.725	19.257	1018.787	1038.044	197.00
23.1	9.532	18.951	1014.283	1033.234	197.00
23.2	9.418	18.770	1009.473	1028.243	197.00
23.3	9.351	18.540	1004.483	1023.023	197.00
23.4	9.189	18.159	999.262	1017.421	197.00
23.5	8.970	17.932	993.660	1011.593	197.00
23.6	8.963	17.799	987.832	1005.631	197.00
23.7	8.836	17.474	981.870	999.344	197.00
23.8	8.638	17.281	975.583	992.864	197.00
23.9	8.643	17.168	969.104	986.272	197.00
24	8.524	16.855	962.511	979.366	197.00

Watershed Desc: Developed Conditions -100yr/24hr

Hydrograph Routing Calculations

Time	(1)	(2)	(3)	(4)	(5)
	I_1	I_1+I_2	2S-O	O_2+2S_2	Elev
Hr	cfs	cfs	cfs	cfs	ft
0	0.000	0.000	0.000	0.000	196.00
0.1	0.000	0.000	0.000	0.000	196.00
0.2	0.000	0.000	0.000	0.000	196.00
0.3	0.000	0.000	0.000	0.000	196.00
0.4	0.000	0.000	0.000	0.000	196.00
0.5	0.000	0.000	0.000	0.000	196.00
0.6	0.000	0.000	0.000	0.000	196.00
0.7	0.000	0.000	0.000	0.000	196.00
0.8	0.000	0.000	0.000	0.000	196.00
0.9	0.000	0.000	0.000	0.000	196.00
1	0.000	0.000	0.000	0.000	196.00
1.1	0.000	0.000	0.000	0.000	196.00
1.2	0.000	0.000	0.000	0.000	196.00
1.3	0.000	0.000	0.000	0.000	196.00
1.4	0.000	0.000	0.000	0.000	196.00
1.5	0.000	0.000	0.000	0.000	196.00
1.6	0.000	0.000	0.000	0.000	196.00
1.7	0.000	0.000	0.000	0.000	196.00
1.8	0.000	0.000	0.000	0.000	196.00
1.9	0.000	0.000	0.000	0.000	196.00
2	0.000	0.000	0.000	0.000	196.00
2.1	0.000	0.000	0.000	0.000	196.00
2.2	0.000	0.000	0.000	0.000	196.00
2.3	0.000	0.000	0.000	0.000	196.00
2.4	0.000	0.000	0.000	0.000	196.00
2.5	0.000	0.000	0.000	0.000	196.00
2.6	0.000	0.000	0.000	0.000	196.00
2.7	0.000	0.002	0.000	0.002	196.00
2.8	0.002	0.083	0.002	0.085	196.00
2.9	0.081	0.396	0.085	0.481	196.00
3	0.315	0.988	0.481	1.469	196.00
3.1	0.673	1.797	1.469	3.266	196.00
3.2	1.124	2.760	3.266	6.026	196.00
3.3	1.635	3.801	6.026	9.827	196.00
3.4	2.166	4.905	9.827	14.732	196.00
3.5	2.740	6.085	14.732	20.817	196.00
3.6	3.345	7.280	20.817	28.097	196.00
3.7	3.935	8.445	28.097	36.542	196.00
3.8	4.510	9.639	36.542	46.180	196.00
3.9	5.129	10.907	46.180	57.088	196.00
4	5.778	12.170	57.088	69.257	196.00
4.1	6.391	13.441	69.257	82.698	196.00
4.2	7.050	14.711	82.698	97.410	196.00
4.3	7.661	15.910	96.029	111.939	196.10
4.4	8.249	17.141	110.558	127.699	196.10
4.5	8.893	18.295	126.318	144.613	196.10
4.6	9.402	19.313	143.232	162.545	196.10
4.7	9.911	20.417	161.164	181.581	196.10
4.8	10.506	21.669	180.200	201.869	196.10
4.9	11.163	22.918	198.215	221.133	196.20
5	11.755	24.063	217.480	241.543	196.20
5.1	12.309	25.137	237.889	263.026	196.20
5.2	12.828	26.158	259.373	285.530	196.20
5.3	13.329	27.137	281.876	309.013	196.20
5.4	13.808	28.083	304.034	332.117	196.30
5.5	14.275	29.129	327.138	356.267	196.30
5.6	14.854	30.230	351.287	381.517	196.30
5.7	15.376	31.373	376.538	407.911	196.30
5.8	15.997	32.535	401.891	434.426	196.40
5.9	16.539	33.575	428.407	461.982	196.40
6	17.037	34.534	455.962	490.496	196.40
6.1	17.497	35.435	480.503	515.938	196.50

Watershed Desc: Developed Conditions -100yr/24hr

Hydrograph Routing Calculations

Time Hr	(1) I_1 cfs	(2) I_1+I_2 cfs	(3) 2S-O cfs	(4) O_2+2S_2 cfs	(5) Elev ft
6.2	17.938	36.598	505.944	542.542	196.50
6.3	18.660	38.382	532.549	570.931	196.50
6.4	19.722	40.572	560.938	601.509	196.50
6.5	20.850	42.726	589.240	631.966	196.60
6.6	21.876	45.051	619.697	664.747	196.60
6.7	23.174	47.830	652.478	700.308	196.60
6.8	24.656	50.585	686.213	736.799	196.70
6.9	25.930	53.192	722.704	775.897	196.70
7	27.262	56.078	761.802	817.880	196.70
7.1	28.815	59.159	799.809	858.969	196.80
7.2	30.344	61.845	840.898	902.742	196.80
7.3	31.501	64.121	881.495	945.617	196.90
7.4	32.621	66.333	924.370	990.703	196.90
7.5	33.712	68.315	966.942	1035.257	197.00
7.6	34.603	70.160	1011.496	1081.656	197.00
7.7	35.557	72.112	1052.917	1125.030	197.10
7.8	36.555	73.929	1096.291	1170.220	197.10
7.9	37.374	75.666	1141.481	1217.147	197.10
8	38.291	77.552	1181.331	1258.883	197.20
8.1	39.260	79.742	1223.067	1302.810	197.20
8.2	40.482	83.193	1258.569	1341.762	197.30
8.3	42.711	88.734	1297.521	1386.255	197.30
8.4	46.023	96.504	1342.014	1438.519	197.40
8.5	50.482	105.744	1383.805	1489.548	197.40
8.6	55.262	115.340	1434.834	1550.174	197.50
8.7	60.078	125.273	1484.611	1609.884	197.50
8.8	65.195	135.751	1530.241	1665.992	197.60
8.9	70.556	146.678	1586.349	1733.027	197.60
9	76.122	157.992	1640.211	1798.204	197.70
9.1	81.870	169.900	1705.389	1875.288	197.70
9.2	88.029	183.519	1763.548	1947.067	197.80
9.3	95.490	199.820	1819.632	2019.452	197.90
9.4	104.330	218.197	1892.017	2110.214	197.90
9.5	113.867	238.058	1965.683	2203.741	198.00
9.6	124.191	270.913	2041.912	2312.824	198.10
9.7	146.722	347.612	2132.988	2480.599	198.20
9.8	200.889	508.607	2262.737	2771.344	198.40
9.9	307.717	826.865	2512.927	3339.792	198.60
10	519.147	1201.705	3001.053	4202.757	199.10
10.1	682.557	1297.810	3542.932	4840.742	199.80
10.2	615.253	1082.829	4051.927	5134.756	200.00
10.3	467.576	836.120	4345.941	5182.060	200.00
10.4	368.544	667.068	4393.245	5060.313	200.69
10.5	298.524	544.296	4271.498	4815.794	200.00
10.6	245.771	451.190	4026.979	4478.169	200.00
10.7	205.419	381.850	3689.354	4071.204	200.00
10.8	176.431	332.813	3470.494	3803.308	199.70
10.9	156.382	298.365	3309.132	3607.497	199.50
11	141.983	272.632	3202.251	3474.882	199.30
11.1	130.648	252.095	3106.222	3358.316	199.20
11.2	121.446	236.198	3019.577	3255.775	199.10
11.3	114.751	224.616	2936.862	3161.477	199.10
11.4	109.864	215.609	2842.564	3058.174	199.00
11.5	105.745	208.137	2747.044	2955.181	198.90
11.6	102.392	201.886	2653.837	2855.723	198.80
11.7	99.494	196.358	2576.131	2772.490	198.70
11.8	96.864	191.255	2514.073	2705.329	198.60
11.9	94.391	186.086	2446.912	2632.998	198.60
12	91.695	180.869	2395.165	2576.034	198.50
12.1	89.174	175.928	2358.172	2534.100	198.40
12.2	86.754	171.461	2316.238	2487.698	198.40
12.3	84.706	167.900	2269.836	2437.736	198.40

Watershed Desc: Developed Conditions -100yr/24hr

Hydrograph Routing Calculations

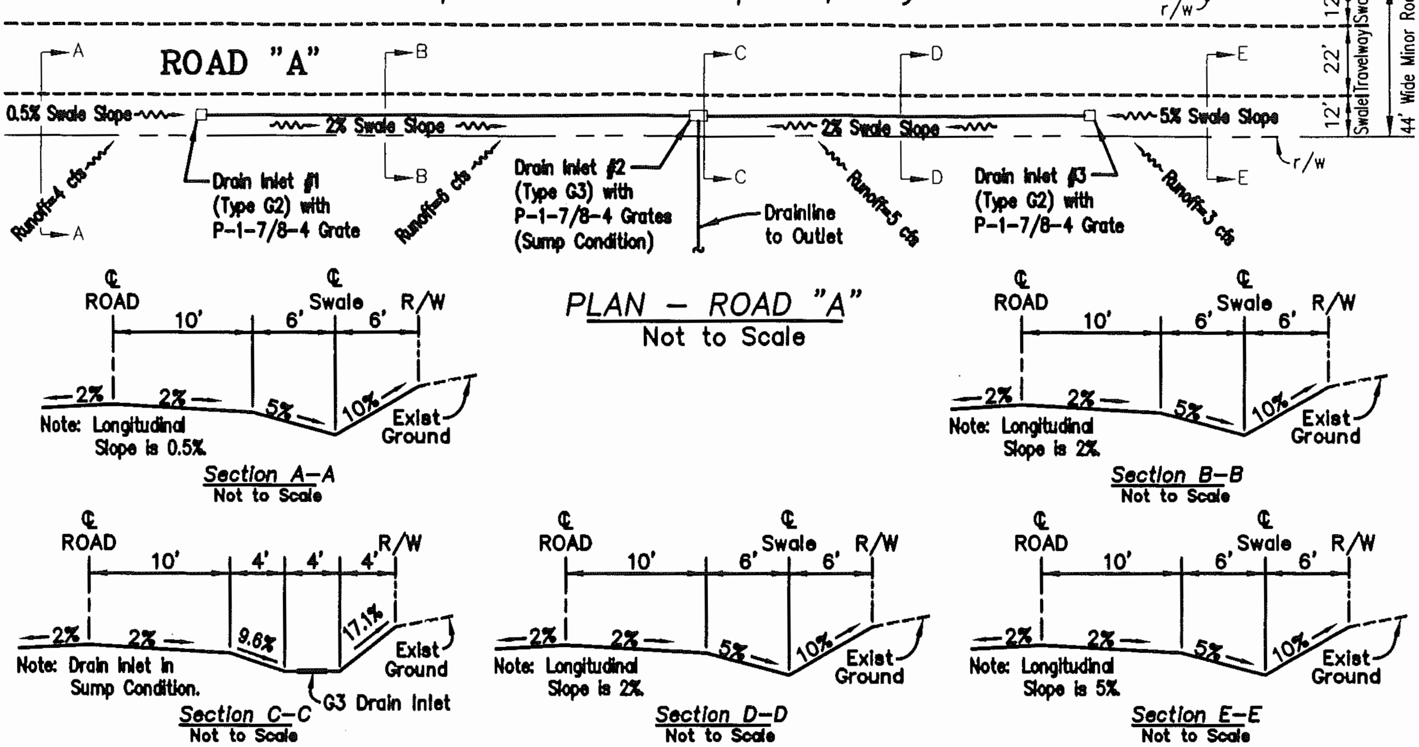
Time Hr	(1) I_1 cfs	(2) I_1+I_2 cfs	(3) 2S-O cfs	(4) O_2+2S_2 cfs	(5) Elev ft
12.4	83.193	164.564	2239.213	2403.777	198.30
12.5	81.371	160.730	2205.255	2365.984	198.30
12.6	79.359	157.221	2186.148	2343.369	198.30
12.7	77.862	154.219	2163.532	2317.752	198.20
12.8	76.357	151.203	2137.915	2289.118	198.20
12.9	74.846	148.174	2109.281	2257.455	198.20
13	73.328	145.134	2077.619	2222.753	198.20
13.1	71.806	142.085	2060.923	2203.008	198.10
13.2	70.279	139.027	2041.178	2180.205	198.10
13.3	68.748	135.960	2018.375	2154.335	198.10
13.4	67.212	132.886	1992.505	2125.391	198.10
13.5	65.673	130.121	1980.860	2110.981	198.00
13.6	64.448	127.222	1966.450	2093.673	198.00
13.7	62.775	123.607	1949.142	2072.749	198.00
13.8	60.832	120.193	1928.218	2048.411	198.00
13.9	59.361	117.532	1903.880	2021.413	198.00
14	58.172	114.687	1893.977	2008.665	197.90
14.1	56.516	111.412	1881.229	1992.641	197.90
14.2	54.896	108.830	1865.206	1974.036	197.90
14.3	53.934	106.981	1846.600	1953.581	197.90
14.4	53.047	105.253	1826.146	1931.399	197.90
14.5	52.206	103.915	1803.964	1907.879	197.90
14.6	51.710	103.130	1796.139	1899.269	197.80
14.7	51.420	102.675	1787.528	1890.203	197.80
14.8	51.255	102.420	1778.463	1880.882	197.80
14.9	51.164	101.963	1769.142	1871.105	197.80
15	50.799	101.067	1759.364	1860.432	197.80
15.1	50.268	100.226	1748.691	1848.917	197.80
15.2	49.958	99.737	1737.176	1836.913	197.80
15.3	49.779	99.138	1725.172	1824.311	197.80
15.4	49.359	98.154	1712.570	1810.724	197.80
15.5	48.795	97.578	1717.909	1815.487	197.80
15.6	48.783	97.246	1722.672	1819.918	197.80
15.7	48.463	96.421	1708.177	1804.598	197.80
15.8	47.958	95.939	1711.783	1807.722	197.70
15.9	47.981	95.663	1714.906	1810.569	197.70
16	47.681	94.869	1717.754	1812.623	197.80
16.1	47.188	94.085	1719.808	1813.893	197.80
16.2	46.897	93.627	1721.078	1814.704	197.80
16.3	46.729	93.043	1721.889	1814.932	197.80
16.4	46.314	92.063	1722.117	1814.180	197.80
16.5	45.749	91.165	1721.365	1812.530	197.80
16.6	45.416	90.638	1719.715	1810.353	197.80
16.7	45.222	90.332	1717.537	1807.870	197.80
16.8	45.110	90.159	1715.054	1805.214	197.80
16.9	45.049	89.745	1712.398	1802.143	197.70
17	44.696	88.864	1709.328	1798.192	197.70
17.1	44.168	88.024	1705.377	1793.401	197.70
17.2	43.856	87.530	1700.586	1788.116	197.70
17.3	43.674	86.921	1695.301	1782.222	197.70
17.4	43.248	85.922	1689.407	1775.329	197.70
17.5	42.675	85.332	1682.514	1767.846	197.70
17.6	42.657	84.986	1675.031	1760.017	197.70
17.7	42.329	84.144	1667.202	1751.346	197.70
17.8	41.815	83.647	1658.530	1742.178	197.70
17.9	41.832	83.357	1649.362	1732.720	197.70
18	41.525	82.547	1639.904	1722.452	197.70
18.1	41.022	81.747	1629.636	1711.383	197.70
18.2	40.725	81.275	1618.568	1699.843	197.70
18.3	40.550	80.676	1620.199	1700.876	197.60
18.4	40.126	79.680	1621.232	1700.912	197.60
18.5	39.554	78.767	1621.269	1700.036	197.60

Watershed Desc: Developed Conditions -100yr/24hr

Hydrograph Routing Calculations

Time Hr	(1) I_1 cfs	(2) I_1+I_2 cfs	(3) 2S-O cfs	(4) O_2+2S_2 cfs	(5) Elev ft
18.6	39.213	78.226	1620.392	1698.618	197.60
18.7	39.012	77.908	1618.975	1696.883	197.60
18.8	38.895	77.724	1617.239	1694.963	197.60
18.9	38.829	77.298	1615.320	1692.618	197.60
19	38.469	76.403	1612.974	1689.377	197.60
19.1	37.934	75.549	1609.734	1685.283	197.60
19.2	37.615	75.043	1605.639	1680.682	197.60
19.3	37.428	74.422	1601.039	1675.461	197.60
19.4	36.995	73.410	1595.818	1669.228	197.60
19.5	36.415	72.484	1589.584	1662.068	197.60
19.6	36.070	71.935	1582.425	1654.360	197.60
19.7	35.865	71.610	1574.716	1646.327	197.60
19.8	35.745	71.421	1566.683	1638.104	197.60
19.9	35.676	70.989	1558.461	1629.450	197.60
20	35.313	70.088	1549.807	1619.895	197.60
20.1	34.775	69.229	1540.252	1609.481	197.60
20.2	34.454	68.719	1529.838	1598.557	197.60
20.3	34.264	68.093	1532.993	1601.087	197.60
20.4	33.829	67.075	1521.443	1588.519	197.60
20.5	33.246	66.469	1522.955	1589.424	197.50
20.6	33.223	66.110	1523.861	1589.970	197.50
20.7	32.887	65.250	1524.407	1589.657	197.50
20.8	32.364	64.739	1524.094	1588.833	197.50
20.9	32.376	64.436	1523.269	1587.706	197.50
21	32.061	63.611	1522.142	1585.753	197.50
21.1	31.550	62.795	1520.189	1582.984	197.50
21.2	31.245	62.309	1517.421	1579.730	197.50
21.3	31.064	61.697	1514.167	1575.864	197.50
21.4	30.633	60.685	1510.301	1570.986	197.50
21.5	30.052	60.082	1505.423	1565.504	197.50
21.6	30.029	59.723	1499.941	1559.664	197.50
21.7	29.693	58.862	1494.100	1552.962	197.50
21.8	29.169	58.349	1487.399	1545.747	197.50
21.9	29.180	58.044	1480.184	1538.228	197.50
22	28.864	57.215	1472.664	1529.879	197.50
22.1	28.351	56.396	1464.316	1520.712	197.50
22.2	28.045	55.908	1455.149	1511.057	197.50
22.3	27.863	55.293	1445.493	1500.786	197.50
22.4	27.430	54.277	1435.223	1489.500	197.50
22.5	26.847	53.671	1434.786	1488.457	197.50
22.6	26.823	53.309	1433.743	1487.052	197.40
22.7	26.486	52.445	1432.338	1484.783	197.40
22.8	25.959	51.929	1430.069	1481.998	197.40
22.9	25.970	51.622	1427.284	1478.906	197.40
23	25.652	50.790	1424.192	1474.982	197.40
23.1	25.138	49.968	1420,268	1470.237	197.40
23.2	24.830	49.478	1415.523	1465.001	197.40
23.3	24.647	48.860	1410.287	1459.147	197.40
23.4	24.213	47.842	1404.433	1452.274	197.40
23.5	23.629	47.232	1397.560	1444.793	197.40
23.6	23.604	46.869	1390.079	1436.948	197.40
23.7	23.265	46.002	1382.234	1428.236	197.40
23.8	22.737	45.484	1373.522	1419.006	197.40
23.9	22.747	45.175	1364.292	1409.467	197.40
24	22.428	44.341	1354.753	1399.094	197.40

Example – Inlet Intercept Capacity in Swale



The plan and cross-sectional views, shown above, are graphical illustrations of Road "A". The drainage system used on Road "A", a minor roadway, consists of two – Type G2 flanking drain inlets (Drain Inlets #1 and #3), one – Type G3 drain inlet in a sump (Drain Inlet #2), and an outlet (not shown). Storm runoffs of 4 cfs and 3 cfs are directed to Drain Inlet #1 (DI #1) and Drain Inlet #3 (DI #3) respectively, while storm runoffs totaling 11 cfs are directed to Drain Inlet #2 (DI #2). Roadside swales that channel runoff to DI #1, DI #2, and DI #3 have slopes of 0.5%, 2% and 5%, respectively.

1) What is the street capacity (half-section) upstream of DI #1 ? Will runoff overtop the roadway crown in this section of Road "A" ?
   - **A) Using Section A-A and Plate 16 (Allowable Maximum Street Capacity (Half-Section without Curb and Gutter)): Minor Roadway Flow Capacity when slope is 0.5% = 6.0 cfs.**
   - **B) The maximum flow upstream of DI #1 is 4 cfs. Since 4 cfs is less than 6.0 cfs, runoff will NOT overtop the roadway crown.**

2) What is the street capacity (half-section) upstream of DI #3 ? Will runoff overtop the roadway crown in this section of Road "A" ?
   - **A) Using Section E-E and Plate 16 (Allowable Maximum Street Capacity (Half-Section without Curb and Gutter)): Minor Roadway Flow Capacity when slope is 5% = 19.1 cfs.**
   - **B) The maximum flow upstream of DI #3 is 3 cfs. Since 3 cfs is less than 19.1 cfs, runoff will NOT overtop the roadway crown.**

3) How much flow does DI #1 intercept ? How much flow bypasses DI #1 ?
   - **A) Using Plate 24 (Intercept Capacity for Type G2 Drain Inlet on a Minor Roadway): Flow Intercept Capacity (when Swale Flow is 4 cfs and Swale Slope is 0.5%) = 2.7 cfs.**
   - **B) Bypass Flow = 4 cfs (Swale Flow) – 2.7 (Intercept Flow) = 1.3 cfs.**

4) How much flow does DI #3 intercept ? How much flow bypasses DI #3 ?
   - **A) Using Plate 24 (Intercept Capacity for Type G2 Drain Inlet on a Minor Roadway): Flow Intercept Capacity (when Swale Flow is 3 cfs and Swale Slope is 5%) = 1.6 cfs.**
   - **B) Bypass Flow = 3 cfs (Swale Flow) – 1.6 (Intercept Flow) = 1.4 cfs.**

5) What is the street capacity (half-section) between DI #1 and DI #2 ? Will runoff overtop the roadway crown in this section of Road "A" ?
   - **A) Using Section B-B and Plate 16 (Allowable Maximum Street Capacity (Half-Section without Curb and Gutter)): Minor Roadway Flow Capacity when slope is 2% = 12.1 cfs.**
   - **B) The maximum flow between DI #1 and DI #2 is the sum of the runoff from the drainage area (6 cfs) and the flow that bypassed DI #1 (1.3 cfs) or 7.3 cfs. Since 7.3 cfs is less than 12.1 cfs, runoff will NOT overtop the roadway crown.**

6) What is the street capacity (half-section) between DI #3 and DI #2 ? Will runoff overtop the roadway crown in this section of Road "A" ?
   - **A) Using Section D-D and Plate 16 (Allowable Maximum Street Capacity (Half-Section without Curb and Gutter)): Minor Roadway Flow Capacity when slope is 2% = 12.1 cfs.**
   - **B) The maximum flow between DI #3 and DI #2 is the sum of the runoff from the drainage area (5 cfs) and the flow that bypassed DI #3 (1.4 cfs) or 6.4 cfs. Since 6.4 cfs is less than 12.1 cfs, runoff will NOT overtop the roadway crown.**

7) How much runoff is directed to DI #2 ? Will runoff overtop the roadway crown at Section C-C of Road "A" ?
   - **A) 11 cfs (Drainage area flow) + 1.3 cfs (Bypass flow from DI #1) + 1.4 cfs (Bypass flow from DI #3) = 13.7 cfs.**
   - **B) Using Table 2 (Drain Inlet Capacity for Sump Condition): Intake Capacity for a Minor Roadway with Type G3 Drain Inlet = 15.8 cfs. Since 13.7 cfs is less than 15.8 cfs, runoff will NOT overtop the roadway crown.**

DRAIN INLET IN SUMP CONDITION

References:

(1) Federal Highway Administration, Drainage of Highway Pavements, Hydraulic Engineering Circular No. 12, dated March 1984. (HEC-12)

(2) Standard Details for Public Works Construction, dated September, 1984.

(3) Exhibit 1 in this Drainage Manual (Trapezoid Cross-Sections at G2, G3, G4 Drain Inlets for Dead-End, Minor, Collector Roadways).

Basically, a drain inlet in a sump condition operates as a weir when water surface levels above the inlet are low, as an orifice when water surface levels above the inlet are high, and both weir and orifice for transitional water surface levels. Chart 11 in the HEC-12 Manual graphically shows the drain inlet operating in its weir, orifice, and transitional phases. Chart 11 graphs, Discharge (Q) versus Depth of Water above Inlet (d), for drain inlets having varying perimeters (when operating as a weir) and varying clear opening area (when operating as an orifice).

From the County Standard Details, D-36, D-37, D-38, the following perimeter values for type G2, G3, G4 Drain Inlets were obtained: G2 = 9.9', G3= 15.8', G4 = 13.1'. When these perimeter values were applied to Chart 11, it was evident that the G2, G3, and G4 inlets functioned as a WEIR for water surface depths up to approximately 0.9'.

This drainage manual states that flooding up to the crown of the roadway is acceptable and when combined with Exhibit 1 the following limitations were established:

Maximum allowable water surface depth for Collector Roadways is 0.72' (as measured from the roadway crown to the top of the drain inlet).

Maximum allowable water surface depth for Minor Roadways is 0.58' (as measured from the roadway crown to the top of the drain inlet).

Maximum allowable water surface depth for Dead-End Roadways is 0.53' (as measured from the roadway crown to the top of the drain inlet).

Note: As stated earlier, G2, G3, G4 inlets operate as weirs for water surface depths up to approximately 0.9', well above the maximum roadway depths defined above.

From the above, we know that G2, G3, G4 drain inlets on all of our Dead-End, Minor, Collector Roadways function as a WEIR when the water surface depths are at their maximum allowable height. Using this premise, the "Allowable Maximum Street Capacity (Half-Section without Curb and Gutter)" Table was generated using the WEIR equation shown below.

$$Q\ (\text{in}) = (0.75)*(3)*(\text{Inlet Perimeter})*(\text{Roadway Crown to Top of Drain Inlet Depth})^{3/2}$$

Note: Above formula assumes a 25% clogging factor.

DRAIN INLET ON CONTINUOUS GRADE

References:

(1) Federal Highway Administration, Drainage of Highway Pavements, Hydraulic Engineering Circular No. 12, dated March 1984. (HEC-12)

Note: Example 23 and 24 in the HEC-12 Manual (pp.93-96) was used as a guide in developing a procedure for calculating the intercept capability of a drain inlet on a continuous grade.

(2) Standard Details for Public Works Construction, dated September, 1984.

(3) Exhibit 1 in this Drainage Manual (Trapezoid Cross-Sections at G2, G3, G4 Drain Inlets for Dead-End, Minor, Collector Roadways).

Basically, the amount of runoff a drain inlet can intercept is composed of two components, a frontal intercept component and a side intercept component. The procedure for computing the amount of runoff a drain inlet can intercept was comprised of the following five steps: (A) Compute the ratio of frontal flow to total flow in a trapezoidal gutter. (B) Compute the ratio of total frontal flow intercepted to total frontal flow. (C) Compute ratio of side flow intercepted to total side flow. (D) Compute the overall efficiency of the inlet (combines the frontal flow and side flow ratios). (E) Compute the runoff intercepted by the inlet and factor in a 15% clogging factor. The procedure is performed on collector, minor, dead-end roadways, with G2, G3, G4 drain inlets, using slopes of 0.5%, 2%, 5%, 10%, and swale flows (i.e. flow directed at the drain inlet) between 0.25 cfs to 10 cfs. In all, data for nine graphs was generated using this procedure.

Ratio of Frontal Flow to Total Gutter Flow

$e_0 = w/(B+(d/2)*(z_1+z_2))$

Where: (A) Trapezoidal cross-sections at G2,G3,G4 drain inlets on various roadways are defined by Exhibit 1 in the Drainage Manual.

(B) w = width of drain inlet as defined in the County Standard Details D-36, D-37, D-38.

(C) B = the base width (i.e. flat section) of the trapezoidal section, as defined in Exhibit 1.

(D) d = depth of flow in the trapezoidal section, as calculated using Manning's formula (n=0.015).

(E) z = grade of side inclines of the trapezoidal section, as defined in Exhibit 1.

Ratio of Frontal Flow Intercepted to Total Frontal Flow

If (v) is less than or equal to (v_0) then:
$Rf = 1$

If (v) is greater than (v_0) then:
$Rf = 1 - (0.09)*(v-v_0)$

Where: (A) v = velocity of flow in the trapezoidal section, as calculated using Manning's formula (n=0.015).

(B) v_0 = gutter velocity where splash-over first occurs (Using the County Standard Details and Chart 7 in the HEC-12 manual, it was determined that v_0 for G2,G4 drain inlet grates = 6.13 ft/s and v_0 for G3 drain inlet grate = 9.2 ft/s).

Average Ratio of Side Flow Intercepted to Total Side Flow

First Find: $Rsx_1 = 1/(1+(0.15*v^{1.8})/(s_{x1} *L^{2.3}))$
Next Find: $Rsx_2 = 1/(1+(0.15*v^{1.8})/(s_{x2} *L^{2.3}))$

Last Find: $Rsx = (Rsx_1 + Rsx_2)/2$

Where: (A) Dimensions of drain inlets are defined in the County Standard Details D-36, D-37, D-38.

(B) v = velocity of flow in the trapezoidal section, as calculated using Manning's formula (n=0.015).

(C) s_{x1} = the slope of side incline that abuts the travelway in the trapezoidal drain inlet section, as defined in Exhibit 1.

(D) s_{x2} = the slope of side incline that abuts the right of way in the trapezoidal drain inlet section, as defined in Exhibit 1.

(E) L = Length of the grate which was determined to be 2.95' for G2, G4 and 5.90', for G3.

(F) Rsx = Average Ratio of Side Flow Intercepted to Total side flow.

Overall Efficiency of Drain Inlet (Intercept Efficiency of Frontal and Side Flows)

$E = (Rf*e_0) + Rsx*(1-e_0)$

Where: (A) The first term is the ratio of frontal flow intercepted to the total frontal flow multiplied by ratio of frontal flow to total gutter flow.

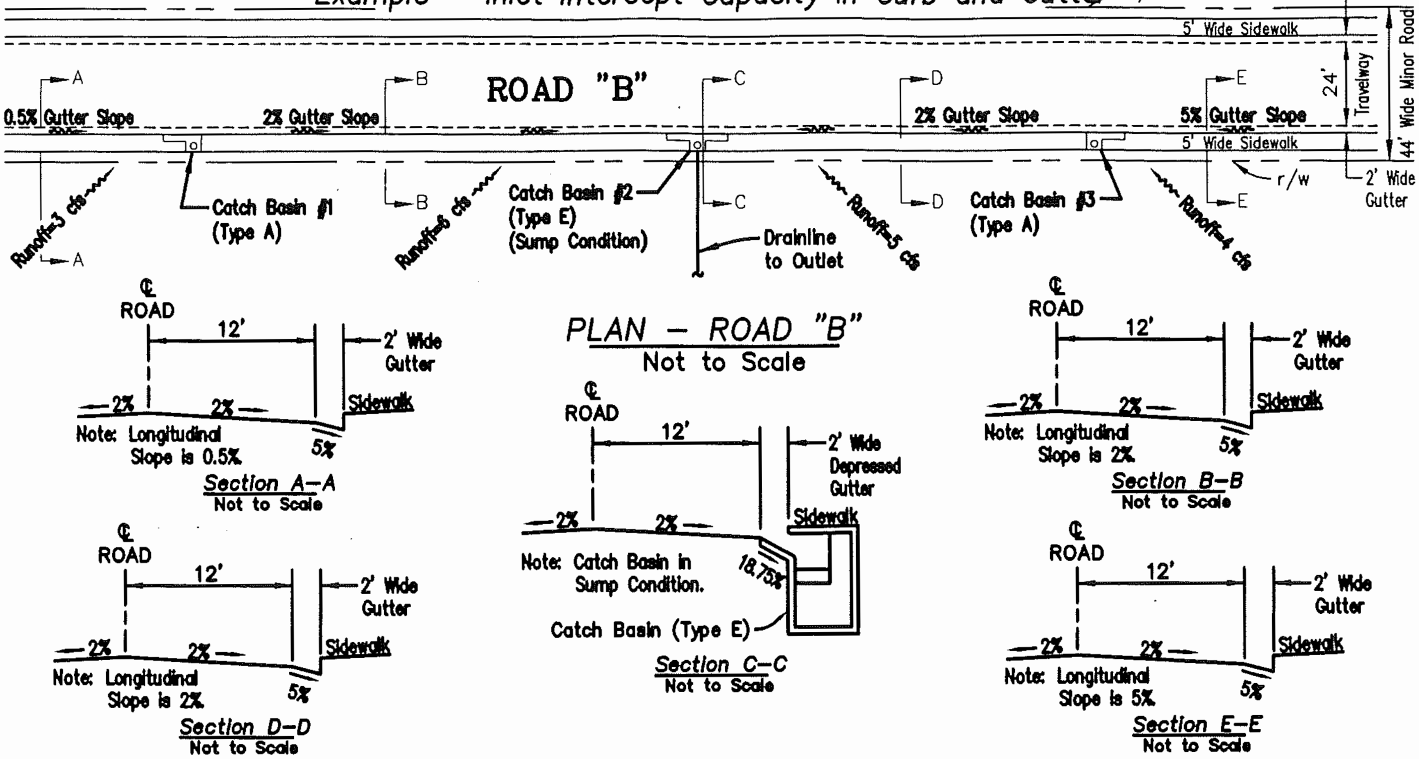
(B) The second term is the ratio of side flow intercepted to the total side flow multiplied by the ratio of side flow to total gutter flow.

Runoff Intercepted by Drain Inlet after Factoring in Clogging

$Q_{in} = (Q_{swale})*(E)*(0.85)$ (Assumes a 15% Clogging Factor)

Where: (A) Q_{in} = Flow intercepted by the Drain Inlet (i.e. the Graphed Value).
(B) Q_{swale} = Swale Flow (i.e. Flow Directed to the Drain Inlet).
(C) E = Intercept Efficiency of Frontal and Side Flows.
(D) 85% is the efficiency of the inlet after accounting for a 15% Clogging Factor.

Example – Inlet Intercept Capacity in Curb and Gutter



The plan and cross-sectional views, shown above, are graphical illustrations of Road "B". The drainage system used on this curb and gutter minor roadway, consists of two – Type A flanking catch basins (Catch Basins #1 and #3), one – Type E catch basin in a sump (Catch Basin #2), and an outlet (not shown). Storm runoffs of 3 cfs and 4 cfs are directed to Catch Basin #1 (CB #1) and Catch Basin #3 (CB #3) respectively, while storm runoffs totaling 11 cfs are directed to Catch Basin #2 (CB #2). Street gutters which channel runoff to CB #1, CB #2, and CB #3 have longitudinal slopes of 0.5%, 2% and 5%, respectively.

1) What is the street capacity (half-section) upstream of CB #1 ? Will runoff overtop the roadway crown in this section of Road "B" ?
   A) Using Section A-A and Plate 17 (Allowable Maximum Street Capacity (Half-Section with Curb and Gutter)):
      Minor Roadway Flow Capacity when slope is 0.5% = 4.0 cfs.
   B) The maximum flow upstream of CB #1 is 3 cfs. Since 3 cfs is less than 4.0 cfs, runoff will NOT overtop the roadway crown.
2) What is the street capacity (half-section) upstream of CB #3 ? Will runoff overtop the roadway crown in this section of Road "B" ?
   A) Using Section E-E and Plate 17 (Allowable Maximum Street Capacity (Half-Section with Curb and Gutter)):
      Minor Roadway Flow Capacity when slope is 5% = 12.6 cfs.
   B) The maximum flow upstream of CB #3 is 4 cfs. Since 4 cfs is less than 12.6 cfs, runoff will NOT overtop the roadway crown.
3) How much flow does CB #1 intercept ? How much flow bypasses CB #1 ?
   A) Using Plate 36 (Intercept Capacity for Type A, B, C Catch Basin on a Minor Roadway):
      Flow Intercept Capacity (when Gutter Flow is 3 cfs and Longitudinal Gutter Slope is 0.5%) = 2.5 cfs.
   B) Bypass Flow = 3 cfs (Gutter Flow) – 2.5 (Intercept Flow) = 0.5 cfs.
4) How much flow does CB #3 intercept ? How much flow bypasses CB #3 ?
   A) Using Plate 36 (Intercept Capacity for Type A, B, C Catch Basin on a Minor Roadway):
      Flow Intercept Capacity (when Gutter Flow is 4 cfs and Longitudinal Gutter Slope is 5%) = 3.0 cfs.
   B) Bypass Flow = 4 cfs (Gutter Flow) – 3.0 (Intercept Flow) = 1.0 cfs.
5) What is the street capacity (half-section) between CB #1 and CB #2 ? Will runoff overtop the roadway crown in this section of Road "B" ?
   A) Using Section B-B and Plate 17 (Allowable Maximum Street Capacity (Half-Section with Curb and Gutter)):
      Minor Roadway Flow Capacity when slope is 2% = 8.0 cfs.
   B) The maximum flow between CB #1 and CB #2 is the sum of the runoff from the drainage area (6 cfs) and the flow that bypassed CB #1 (0.5 cfs) or 6.5 cfs. Since 6.5 cfs is less than 8.0 cfs, runoff will NOT overtop the roadway crown.
6) What is the street capacity (half-section) between CB #3 and CB #2 ? Will runoff overtop the roadway crown in this section of Road "B" ?
   A) Using Section D-D and Plate 17 (Allowable Maximum Street Capacity (Half-Section with Curb and Gutter)):
      Minor Roadway Flow Capacity when slope is 2% = 8.0 cfs.
   B) The maximum flow between CB #3 and CB #2 is the sum of the runoff from the drainage area (5 cfs) and the flow that bypassed CB #3 (1.0 cfs) or 6.0 cfs. Since 6.0 cfs is less than 8.0 cfs, runoff will NOT overtop the roadway crown.
7) How much runoff is directed to CB #2 ? Will runoff overtop the roadway crown at Section C-C of Road "B" ?
   A) 11 cfs (Drainage area flow) + 0.5 cfs (Bypass flow from CB #1) + 1.0 cfs (Bypass flow from CB #3) = 12.5 cfs.
   B) Using Table 3 (Catch Basin Capacity for Sump Condition): Intake Capacity for a Minor Roadway with Type E Catch Basin = 17.2 cfs. Since 12.5 cfs is less than 17.2 cfs, runoff will NOT overtop the roadway crown.

CATCH BASIN IN SUMP CONDITION

References:

(1) Federal Highway Administration, Drainage of Highway Pavements, Hydraulic Engineering Circular No. 12, dated March 1984. (HEC-12)

(2) Standard Details for Public Works Construction, dated September 1984.

Like drain inlets, catch basin in a sump condition operates as a weir when flow depths are low and as an orifice when flow depths are high. And like roadways with drain inlets, roadways with catch basin may be flooded up to the roadway crown (per this drainage manual). The above, combined with County Standard Details, R-41, D-1 to D-14, establishes the following limits:

1) Maximum allowable water surface depth on a Collector Roadways is 0.735 (as measured from the roadway crown to the lip of the curb opening).
2) Maximum allowable water surface depth on a Minor Roadways is 0.615 (as measured from the roadway crown to the lip of the curb opening).
3) Maximum allowable water surface depth on a Dead-End Roadways is 0.575 (as measured from the roadway crown to the lip of the curb opening).
4) Type A,B,C has an opening 10' long, Type D has an opening of 3.5, Type E has an opening of 16', and Type F has an opening of 16.5.
5) The curb opening (height) for all catch basins is 5" or 0.417'.

Note: When the water surface depth is at its maximum level (at the roadway crown) the curb opening will be totally submerged.

From the County Standard Details D-1 to D-14 and from viewing Figure 21 in the HEC-12 manual, the typical catch basin for Kauai will have a "limited" or more specifically, an "inclined" throat. Interestingly, the HEC-12 manual (p.76) states that a limited or inclined throat reduces the capacity of the curb opening by causing it to go into orifice flow at depths less than the height of the opening. Base on this HEC-12 observation and the fact that the curb openings will be totally submerged when water surface depth is at the roadway crown the following premise was made: "All catch basins (A, B, C, D, E, F) on all of roadways (Dead-End, Minor, Collector), will functions as an ORIFICE when the water surface level is at its maximum allowable surface depth (i.e. at the crown of roadways)." Using this premise, the "Allowable Maximum Street Capacity (Half-Section with Curb and Gutter)" Table was generated using the ORIFICE Equation shown below:

$$Q\ (in) = (0.75)*(C)*(H)*(L)*(2*G*d0)^{1/2} \qquad \text{(Assumes a 25\% clogging factor)}$$

Where: $C = 0.67$ (Orifice Coefficient), H = Curb Opening = 0.417', $G = 32.16\ ft/s^2$
L = Length of Opening (specified in the County Standard Details),
$d0$ = Effective Head on Center of Orifice Throat
$d0 = di - (H/2)*\sin\theta = di - (.0417'/2)*\sin 79.27^{\circ} = di - 0.205'$

Where: di is the depth of water as measured from the crown of the roadway to the lip of the curb opening. Angle θ (See Figure 21 in HEC-12 manual) is defined by the dimensions specified in the County Standard Details.

CATCH BASIN ON CONTINUOUS GRADE

References:

(1) Federal Highway Administration, Drainage of Highway Pavements, Hydraulic Engineering Circular No. 12, dated March 1984. (HEC-12)

Note: Example 11 in the HEC-12 Manual (pp.63-64) was used as a guide in developing a procedure for calculating the intercept capability of a catch basin on a continuous grade.

(2) Standard Details for Public Works Construction, dated September, 1984.

The general theory in computing the intercept capacity of a catch basin, under a known set of flow conditions (flow, roadway slope, roadway cross-section), is to first calculate the theoretical length of the catch basin (opening) necessary to intercept 100% of the flow. If this theoretical length is less than or equal to the actual length of the catch basin (opening) being proposed, then it is assumed that the entire flow will be intercepted; however, if this theoretical length is greater than the actual length of the catch basin (opening) being proposed, then it is assumed that only a fractional amount of flow will be intercepted by the proposed catch basin. The basic procedure in calculating the intercept capacity of a catch basin consists the following steps. First, there are three basic starting points in this analysis depending on what portion of the roadway runoff is flowing: (1) Gutter Flow Only, (2) Flow in Between Curb and Roadway Crown (Gutter and Travelway Flow), and (3) Flow Beyond the Roadway Crown. Case (3) terminates all analysis, because it violates one of the mandates in this drainage manual (i.e. only flooding up to the roadway crown is acceptable). For Cases (1) and (2), the analysis continues by calculating the theoretical catch basin length necessary to intercept 100% of the flow. If the theoretical catch basin length is greater than the actual catch basin length, then a fractional intercept factor or efficiency factor is calculated. If the theoretical catch basin length is less than or equal to the actual catch basin length then it is presumed that the entire flow has been intercepted and its efficiency is 100%. To complete the analysis, clogging is factored into the catch basin's intercept capacity. The procedure is performed on collector, minor, dead-end roadways, with A,B,C,D,E,F catch basins, using slopes of 0.5%, 2%, 5%, 10%, and runoff flows (i.e. flow directed at the catch basin) between 0.25 cfs to 10 cfs. In all, data for twelve graphs was generated using this procedure.

Starting Point: What Section(s) of the Roadway is Runoff Flowing

With known slopes, swale flows (i.e. flow directed at the catch basin), and roadway sections (see County Standard Details R-41, D-13, D-14), Manning's formula (n=0.015 for A.C. pavement, n=0.016 for concrete gutter), was used to determine the section(s) of the roadway runoff was flowing.

Gutter Flow Only

When flow is confined to the concrete gutter, the theoretical catch basin (opening) length necessary to intercept 100% of the flow is defined in the following way:

$L_T = (0.6)*(Q^{0.42})*(s^{0.3})*(1/n*s_x)^{0.6}$

Where: A) Q = Flow in the gutter directed to catch basin.
B) s = Longitudinal slope of the gutter.
C) n = Manning's "n", used n=0.016 for concrete gutter.
D) s_x = 18.75% (Cross Slope of concrete gutter per County Standard Details D-13 and D-14).

Having solved for L_T, the next step is to check how theoretical catch basin length compares to the actual catch basin length.

Flow Between the Curb and Roadway Crown (Gutter and Travelway Flow)

When runoff is flowing in the gutter and on the travelway, an equivalent cross-slope (s_e) must be determined prior to determining the theoretical catch basin length necessary to intercept 100% of the flow. (s_e) is determined in the following way:

Having calculated the total flow width (computed in the first step) and knowing that the standard gutter width is 2', the A.C. pavement flow width is derived from simple subtraction (T_s = total flow width – 2', where T_s = A.C. pavement flow width). Next, T_s is used to determine the amount of flow on the A.C. pavement.

$Q_S = (0.56/n)*(s_X^{1.67})*(s^{0.5})*(T_s^{2.67})$

Where: A) Q_S and T_S are the A.C. pavement Flow and Flow Width, Respectively.
B) s_X = 2% (Pavement Cross Slope per County Standard Detail R-43).
C) s = Longitudinal slope of the Roadway.
D) n = Manning's "n", used n=0.015 for A.C. Pavement.

The next step is to find the ratio of gutter flow to total flow. The simplest explanation is:
Gutter Flow + Pavement Flow = Total Flow, or if expressed in terms of ratios,
Gutter Flow/Total Flow + Pavement Flow/Total Flow = 1, using algebra
Gutter Flow/Total Flow = 1 – Pavement Flow/Total Flow, or if expressed symbolic form:

$E_0 = 1 - (Q_S/Q)$

Where: A) E_0 = Ratio of Gutter Flow to Total Flow.
B) (Q_S/Q) = Ratio of Pavement Flow to Total Flow.

From the above the equivalent cross-slope (s_e) is calculated in the following way:

$s_e = s_x + (s'_w * E_0)$

Where: A) s_X = 2% (Pavement cross slope per County Standard Detail R-43)
B) s'_w = 18.75% (Gutter cross slope per County Standard Details D-13 and D-14)

Having defined the equivalent cross-slope, the theoretical catch basin (opening) length necessary to intercept 100% of the flow can be determined using the following equation:

$$L_T = (0.6)*(Q^{0.42})*(s^{0.3})*(1/n*s_e)^{0.6}$$

Where: A) Q = Flow Directed to the Catch Basin.
B) s = Longitudinal Slope of the Roadway.
C) n = Manning's "n", used n=.0155 (Average of Concrete Gutter and A.C. Pavement).
D) s_e = Equivalent Cross Slope.

Theoretical Catch Basin Length Greater Than The Actual Catch Basin Length

If the theoretical catch basin length is greater than the actual length of the proposed catch basin, then only a fractional portion of the total flow will be intercepted by the proposed catch basin. The efficiency of the proposed catch basin is determined by the following equation:

$$E = 1 - (1 - L/L_T)^{1.8}$$

Where: A) L = Actual Catch Basin Opening Length
B) L_T = Theoretical Catch Basin Opening Length Necessary to Intercept 100% of Flow

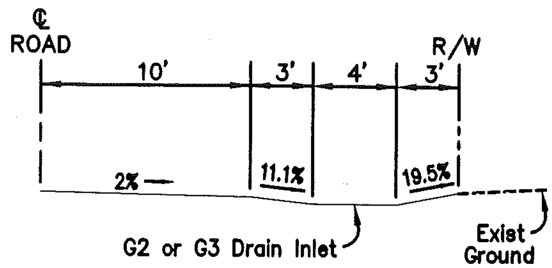
Note: If the theoretical catch basin length is less or equal to the actual catch basin length then it is presumed that 100% of the flow has been intercepted by the proposed catch basin and efficiency (E) is 1.

Runoff Intercepted by Catch Basin after Factoring in Clogging

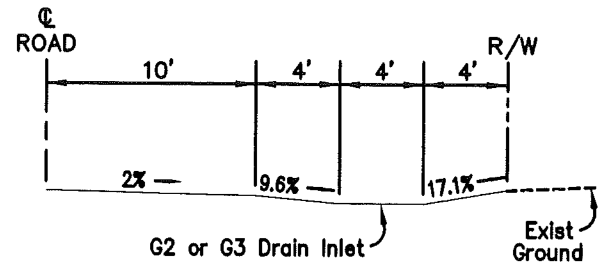
$Q_{in} = (Q)*(E)*(0.85)$ (Assumes a 15% Clogging Factor)

Where: (A) Q_{in} = Flow intercepted by the Proposed Catch Basin (i.e. the Graphed Value)
(B) Q = Flow Directed to the Catch Basin
(C) E = Intercept Efficiency of the proposed Catch Basin
(D) 85% is the efficiency of Catch Basin after accounting for a 15% Clogging Factor

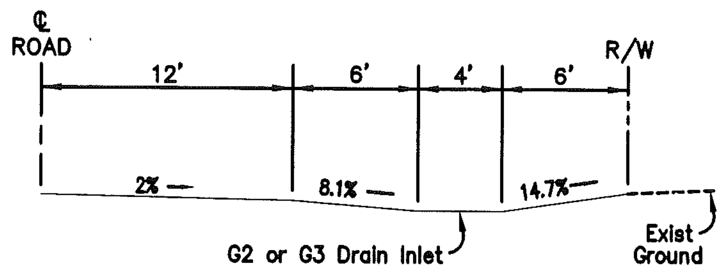
Exhibit 1 – Trapezoidal Cross-Sections at G2, G3, G4 Drain Inlet for Dead-End, Minor, Collector Roadways



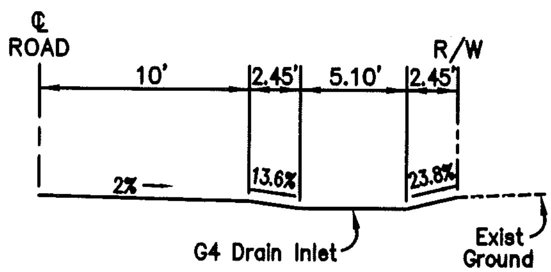
Dead-End Roadway without Curb and Gutter
and Type G2 or G3 Drain Inlet
Not to Scale



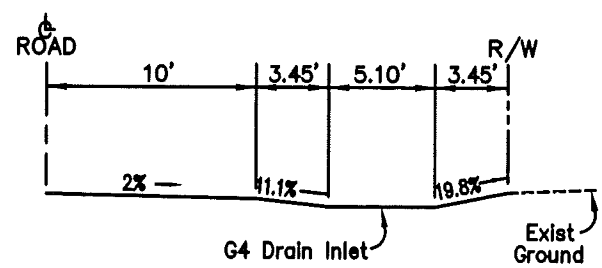
Minor Roadway without Curb and Gutter
and Type G2 or G3 Drain Inlet
Not to Scale



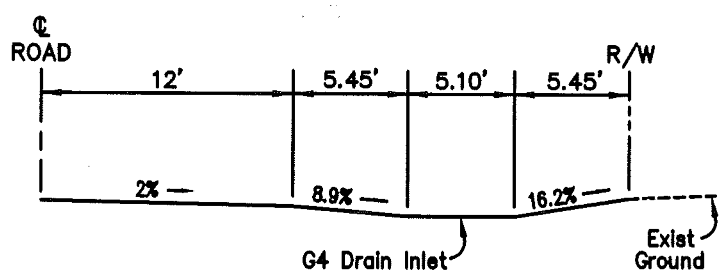
Collector Roadway without Curb and Gutter
and Type G2 or G3 Drain Inlet
Not to Scale



Dead-End Roadway without Curb and Gutter
and Type G4 Drain Inlet
Not to Scale



Minor Roadway without Curb and Gutter
and Type G2 or G3 Drain Inlet
Not to Scale



Collector Roadway without Curb and Gutter
and Type G2 or G3 Drain Inlet
Not to Scale

NOTES:

1) Sections shown includes a 1" local depression at the inlet, which is the current County DPW Standard.
2) Standard Sections (without depression or drain inlet) for the Dead-End, Minor, and Collector Roadways are defined in the County Standard Detail R-42.
3) G2, G3, and G4 Drain Inlet dimensions are defined in the County Standard Details D-36, D-37, and D-38.

Example — Impact Type Energy Dissipator (a.k.a. Impact–Basin Outlet Structure)

Note: The following example and discussion are excerpts from the (U.S. Department of Transportation (Federal Highway Administration), 1983, The Hydraulic Design of Energy Dissipators for Culverts and Channels, Hydraulic Engineering Circular No. 14, pp. VIII–C–1 to VIII–C–9.)

Design Procedures (Refer to Figure VIII–C–1)

1. Use of the impact–basin outlet structure is limited to installations where the velocity at the entrance to the stilling basin does not exceed 50 feet per second and discharge is less than 400 cfs.
2. From the maximum discharge and velocity, compute the flow area at the end of the approach pipe. Compute y_e for a rectangular section of equivalent area twice as wide as the depth of flow, $y_e = (A/2)^{1/2}$.
3. Compute the Froude number Fr and the energy at the end of the pipe H_0. Enter the curve on figure VIII–C–2 and determine the required width of basin W.
4. With W known, obtain the remaining dimensions of the impact basin structure from table VIII–C–1.

Example Problem: Given: D = 48 inches, S_0 = 0.15, Q = 300 cfs, n = 0.015, v_0 = 40 ft/s, y_0 = 2.3 feet

Find: USBR Impact Basin dimensions for use at the outlet of a concrete pipe.

Solution: Since Q is less than 400 cfs and v_0 less than 50 ft/s, the impact basin structure may be tried at this site.

1) Compute y_e:
   Since, $A = Q/v_0 = 300/40 = 7.5$ sq. ft.
   $y_e = (A/2)^{1/2} = (7.5/2)^{1/2} = 1.94$ feet

2) Compute Fr and H_0 and find W:
   $Fr = v_0/(gy_e)^{1/2} = 40/(32.2 \times 1.94)^{1/2} = 5.05$
   $H_0 = y_e + v_0^2/2g = 1.94 + (40)^2/64.4 = 26.8$ feet
   From Figure VIII–C–2: $H_0/W = 1.68$
   $W = 26.8/1.68 = 16.0$ feet

3) From table VIII–C–1 select remaining dimensions:

   Design a second baffle wall dissipator at the end of a long rectangular concrete channel 4 feet wide using the same depth of flow = 2.3 feet, s = 0.15 and n = 0.015 as in the previous example and compare results.

   The discharge for the rectangle channel flowing at a depth of 2.3 feet will be 375 cfs. Computations and comparison with the first example are tabulated below.

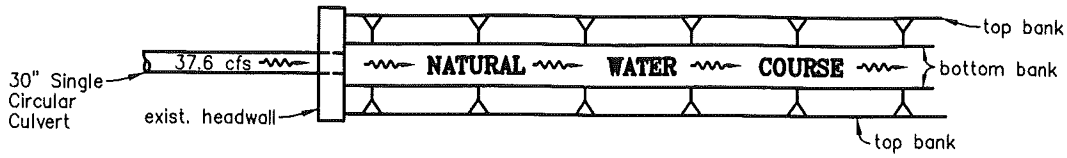
CHANNEL			CIRCULAR	RECTANGULAR
Depth of Flow	y_0	ft.	2.3	2.3
Area of Flow	A	sq. ft.	7.50	9.20
Discharge	Q	c.f.s.	300	375
Velocity	v_0	ft/s	40.	40.9
Flow Depth (Rect. Section)	y_e	ft.	1.9	2.3
Velocity Head	$v_0^2/2g$	ft.	24.9	26.
$H_0 = y_e + v_0^2/2g$		ft.	26.8	28.3
$Fr = v_0/(gy_e)^{1/2}$		--	5.05	4.75
From Figure VIII–C–2	H_0/W	--	1.68	1.55
Width of Basin	W	ft.	16.0	18.3
$H_L/H_0(100)$ – Low Tailwater			67%	65%

   Entering Table VIII–C–1 with W = 16 feet and W = 18 feet, the remaining dimensions of the two dissipators can be read directly in feet and inches. The basin width is taken to the nearest half foot, while the other dimensions are read to the nearest inch. This degree of accuracy is sufficient.

   VIII–C–1 "Hydraulic Design of Stilling Basins," Journal of the Hydraulics Division, A.S.C.E., Paper 1406, October 1957.

4) Other design considerations:
   A) If entrance conduits slopes are greater than 15°, a horizontal section of at least four conduit widths long should be provided immediately upstream of the impact basin structure.
   B) When a hydraulic jump is expected to form in the downstream end of the pipe and the entrance is submerged, a vent about one–sixth the pipe diameter should be installed at a convenient location upstream from the jump.
   C) For erosion reduction and better basin operation, use the alternative end sill and 45° wingwall design shown in Figure VIII–C–1.
   D) For protection against undermining, a cutoff wall should be added at the end of the basin.
   E) Riprap should be placed downstream of the basin for a length of at least four conduit widths.
   F) The sill should be set as low as possible to prevent degradation downstream. For best performance, the downstream channel should be at the same elevation as the top of the sill. A slot should be placed in the end sill to provide for drainage during periods of low flow.
   G) To provide structural support and aid in priming the device, a short support should be placed under the center of the baffle wall.

Example — Baffle Blocks and End Weir Outlet Structure

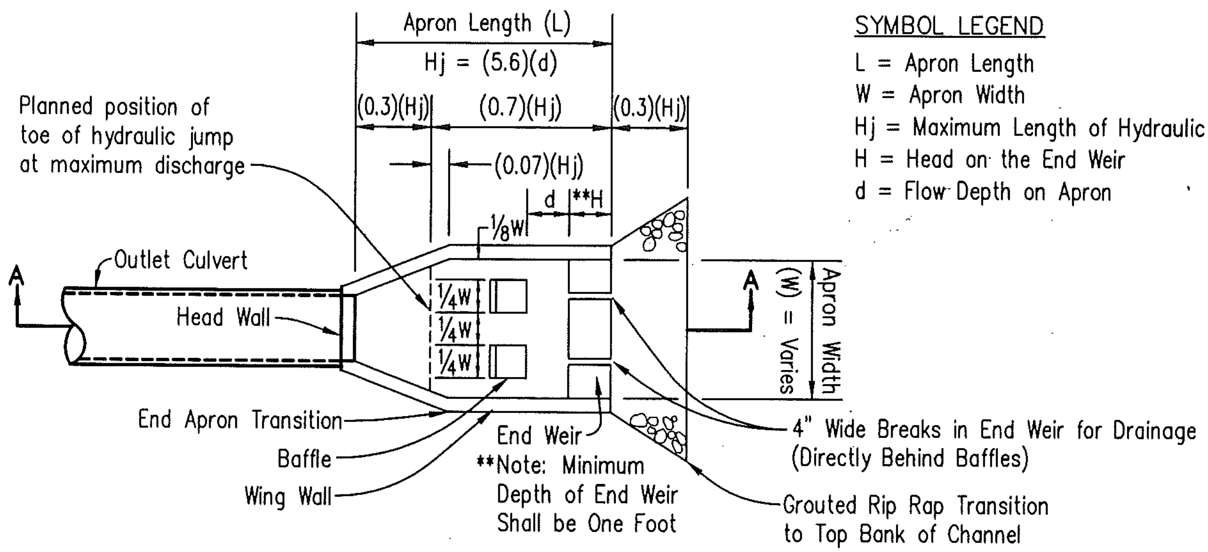


PLAN — Existing Natural Water Course
Not to Scale

The plan view, shown above, is a graphical illustration of a 30" circular culvert flowing into a Natural Water Course. Calculations show that flow from the 30" single circular culvert has a rate of 37.6 cfs, a pipe flow depth of 1.8' and a velocity of 9.92 ft/s. Design an outlet structure with baffle blocks and an end weir that would mitigate the erosive velocity from the 30" circular culvert (flow velocity should be decreased below 5 ft/s).

Background:

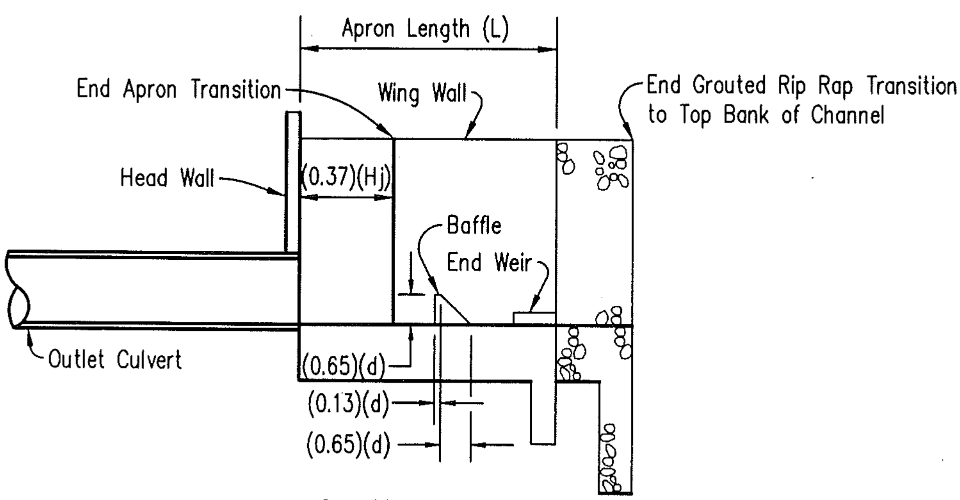
The (Portland Cement Association, 1964, Handbook of Concrete Culvert Pipe Hydraulics, Illinois, Portland Cement Association, pp. 139–164) was used as a guide for the solution to this problem. The Plan and Cross-Sectional views shown below illustrates the basic baffle blocks and end weir outlet structure configuration. And as stated in the above mentioned reference, "the proportions shown are intended to serve only as a guide in establishing minimum dimensions. Departures may be made without seriously affecting the performance of the apron".



SYMBOL LEGEND

L = Apron Length
W = Apron Width
Hj = Maximum Length of Hydraulic Jump
H = Head on the End Weir
d = Flow Depth on Apron

PLAN — Outlet
Not to Scale



Section A–A
Not to Scale

CASE (I) – Let the apron width and the outlet culvert width be equal.

(I) Given: Culvert Flow (Q) = 37.6 cfs, Culvert Flow Depth (dn) = 1.8', Outlet Diameter (D) = 2.5', Apron Width (W) = 2.5', Relative Depth of Flow in Pipe = (dn/D) = 0.72, Energy Discharge Factor (qc) = $(Q)/(D)^{5/2}$ = 3.80

(A) Using the given (qc), (dn), and Figure 61 we determine that the Pressure plus Momentum factor $(P+M)/(wD^3)$ = 0.93

(B) Using the Value from (A) and Figure 62a we find that the Relative Flow Depth on the Apron = (d/D) = 1.29 or since the Outlet Diameter (D) is 2.5', the Apron Flow Depth (d) = 1.29*2.5 = 3.23'

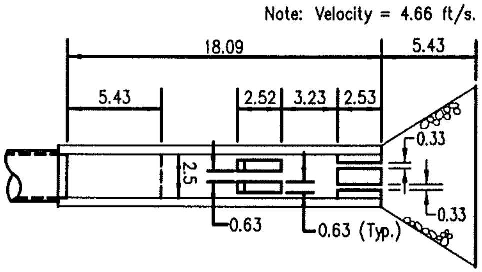
(C) Using Culvert Flow (Q), Apron Width (W), and Apron Flow Depth (d), the Apron Flow Velocity (Va) = Q/dW = 4.66 ft/s

(D) The Head on the End Weir (H), can be calculated using (Q), Apron Width (W), Apron Flow Velocity (Va), and the discharge formula over a broad-crested weir $(Q = 3*W(H+1.2*Va^2/2*32.2)^{3/2})$. Substituting the known values into the formula and solving for (H), the Head on the End Weir (H) = $(Q/3*W)^{2/3}-(.01863*Va^2)$ = 2.53'

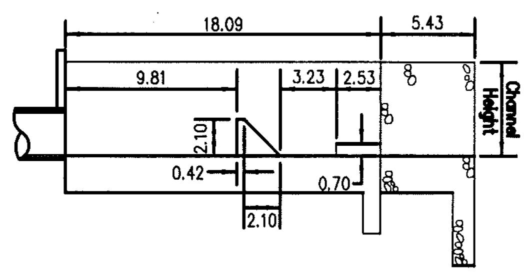
(E) Having enough information, the Baffle Blocks and End Weir Outlet Structure is Dimensioned as Follows:

(1) Apron Length = Max. Hydraulic Jump (Hj) = (5.60*d) = (5.60*3.23) = 18.09'
(2) Distance from Outlet to Toe of Hydraulic Jump (T) = (0.30*Hj) = (0.30*18.09) = 5.43'
(3) Distance from (T) to End of Apron Transition = 0 (No Transition Needed when Apron Width = Outlet Culvert Width)
(4) End Weir Height = Apron Flow Depth (d) – Head on Weir (H) = 3.23' – 2.53' = 0.70'
(5) Depth of End Weir = Weir Head (H) = 2.53'
(6) Vacant Space Between End Weir and Baffle Blocks = Apron Flow Depth (d) = 3.23'
(7) Baffle Block Width = 0.25 * Apron Width = 0.25*2.5 = 0.625'
(8) Baffle Block Height = 0.65 * Apron Flow Depth (d) = 0.65*3.23 = 2.10'
(9) Baffle Block Top Flat Depth = 0.13 * Apron Flow Depth (d) = 0.13*3.23 = 0.42'
(10) Baffle Block Total Depth = Baffle Block Top Flat Depth + Baffle Block Height = 2.10+0.42= 2.52'
(11) Minimum Horizontal Distance for Rip Rap Transition to Top Channel Bank = (0.30*Hj) = (0.30*18.09) = 5.43'

Note: Velocity = 4.66 ft/s.



PLAN - Outlet
Not to Scale



Section
Not to Scale

CASE (II) – For comparison purposes, let the apron width be three times the width of the outlet culvert.

(II) Given: Culvert Flow (Q) = 37.6 cfs, Culvert Flow Depth (dn) = 1.8', Outlet Diameter (D) = 2.5', Apron Width (W) = 7.5', Relative Depth of Flow in Pipe = (dn/D) = 0.72, Energy Discharge Factor (qc) = $(Q)/(D)^{5/2}$ = 3.80

(A) Using the given (qc), (dn), and Figure 61 we determine that the Pressure plus Momentum factor $(P+M)/(wD^3)$ = 0.93

(B) Using the Value from (A) and Figure 62c we find that the Relative Flow Depth on the Apron = (d/D) = 0.76 or since the Outlet Diameter (D) is 2.5', the Apron Flow Depth (d) = 0.76*2.5 = 1.90'

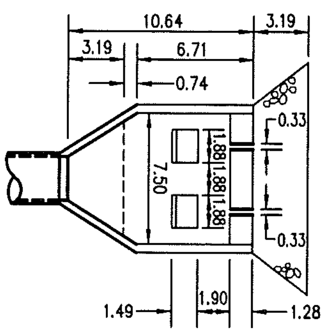
(C) Using Culvert Flow (Q), Apron Width (W), and Apron Flow Depth (d), the Apron Flow Velocity (Va) = Q/dW = 2.64 ft/s

(D) The Head on the End Weir (H), can be calculated using (Q), Apron Width (W), Apron Flow Velocity (Va), and the discharge formula over a broad-crested weir $(Q = 3*W(H+1.2*Va^2/2*32.2)^{3/2})$. Substituting the known values into the formula and solving for (H), the Head on the End Weir (H) = $(Q/3*W)^{2/3}-(.01863*Va^2)$ = 1.28'

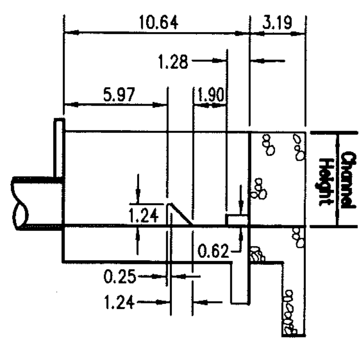
(E) Having enough information, the Baffle Blocks and End Weir Outlet Structure is Dimensioned as Follows:

(1) Apron Length = Max. Hydraulic Jump (Hj) = (5.60*d) = (5.60*1.90) = 10.64'
(2) Distance from Outlet to Toe of Hydraulic Jump (T) = (0.30*Hj) = (0.30*10.64) = 3.19'
(3) Distance from (T) to End of Apron Transition = (0.07*Hj) = (0.07*10.64) = 0.74'
(4) End Weir Height = Apron Flow Depth (d) – Head on Weir (H) = 1.90' – 1.28' = 0.62'
(5) Depth of End Weir = Weir Head (H) = 1.28'
(6) Vacant Space Between End Weir and Baffle Blocks = Apron Flow Depth (d) = 1.90
(7) Baffle Block Width = 0.25 * Apron Width = 0.25*7.5 = 1.88'
(8) Baffle Block Height = 0.65 * Apron Flow Depth (d) = 0.65*1.90 = 1.24'
(9) Baffle Block Top Flat Depth = 0.13 * Apron Flow Depth (D) = 0.13*1.90 = 0.25'
(10) Baffle Block Total Depth = Baffle Block Top Flat Depth + Baffle Block Height = 1.24+0.25 = 1.49'
(11) Minimum Horizontal Distance for Rip Rap Transition to Top Channel Bank = (0.30*Hj) = (0.30*10.64) = 3.19'

Note: Velocity = 2.64 ft/s.



PLAN - Outlet
Not to Scale



Section
Not to Scale

In comparison, both Case (I) and Case (II) reduces the flow velocity in the channel below the intended goal of 5 ft/s. In Case (II) the channel velocity was reduced significantly below 5 ft/s; however, design constraints could preclude Case (II) as a viable solution.

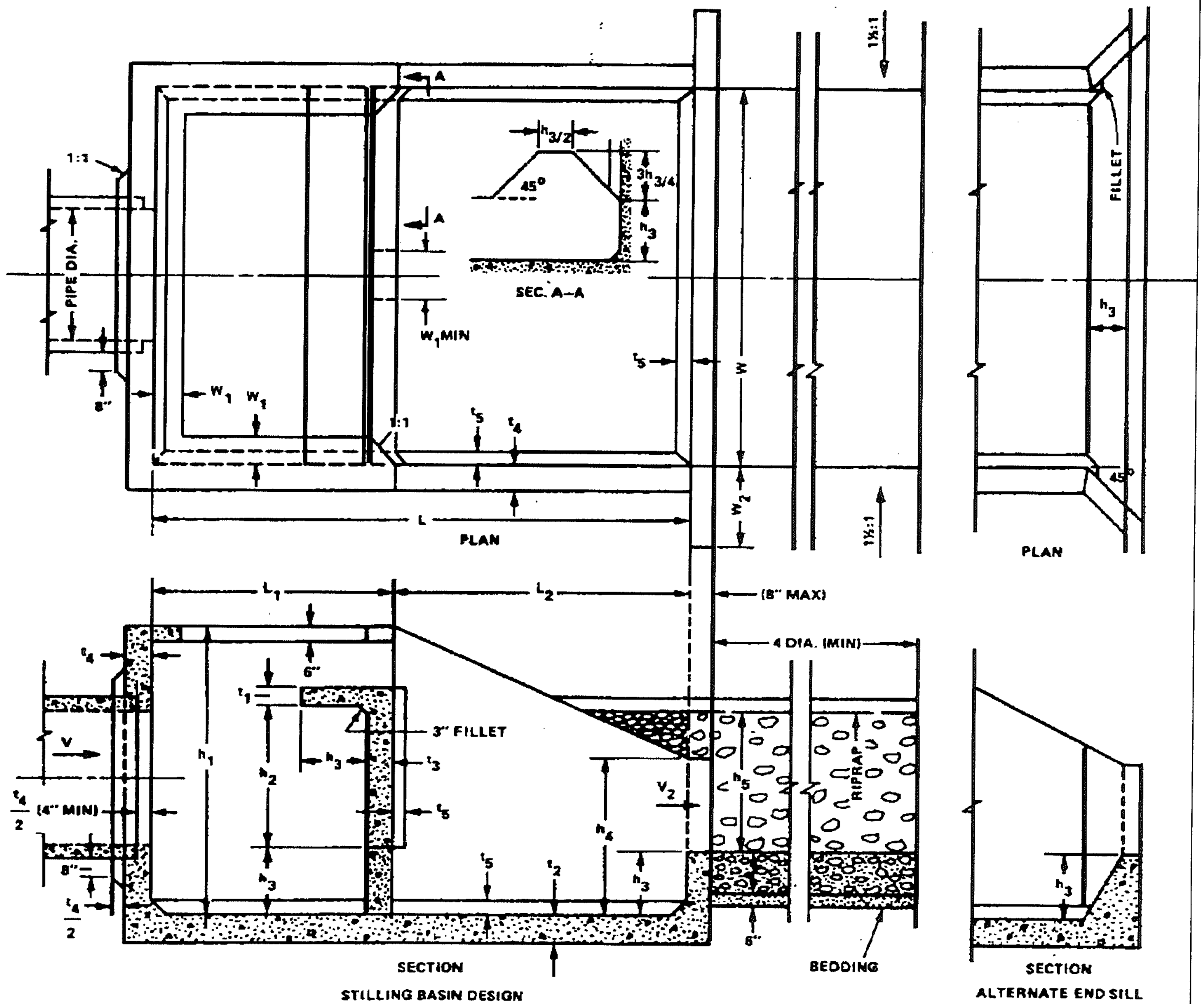


FIGURE VIII–C–1. BAFFLE–WALL ENERGY DISSIPATOR – USBR TYPE VI

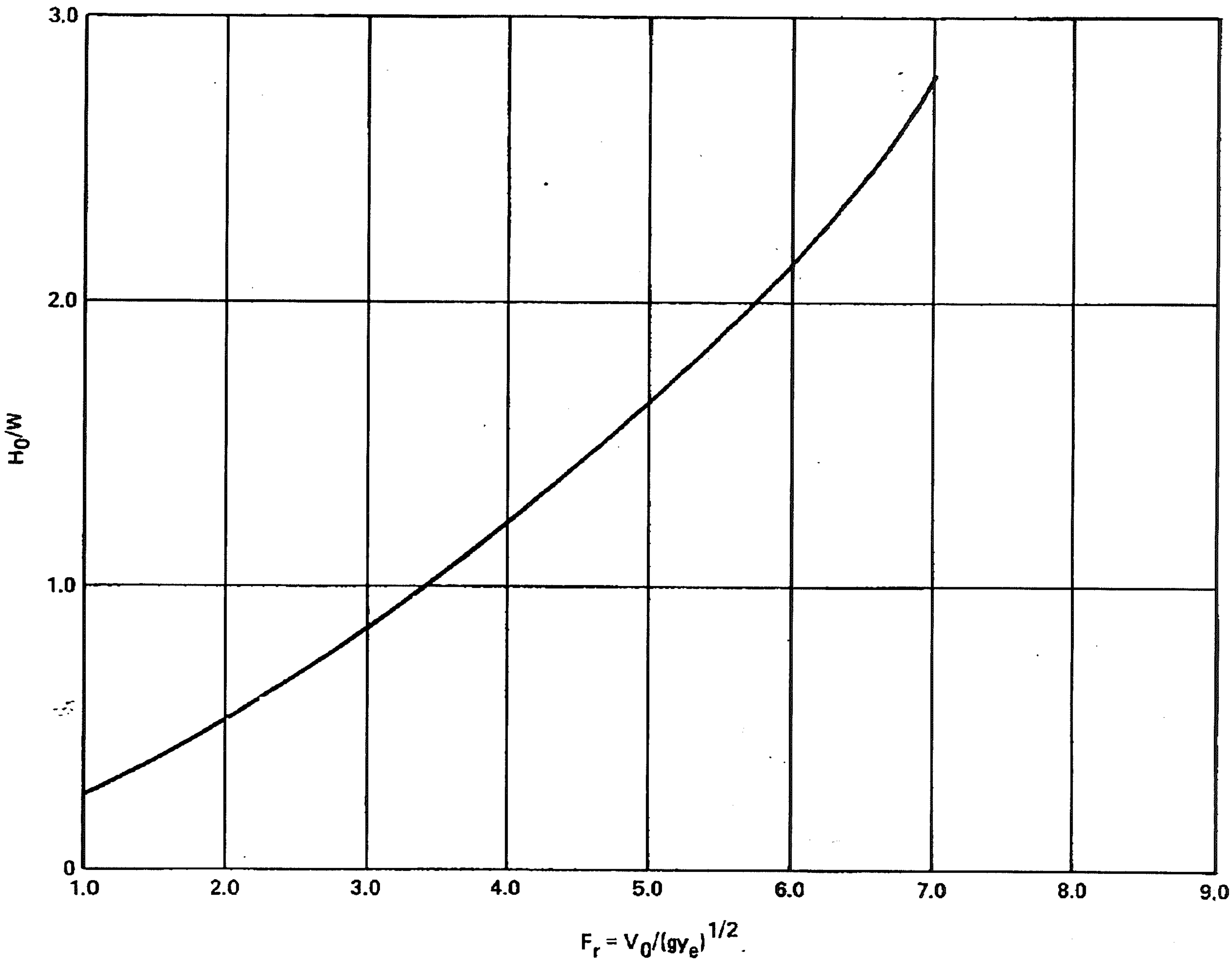


FIGURE VIII–C–2. DESIGN CURVE – BAFFLE WALL DISSIPATOR

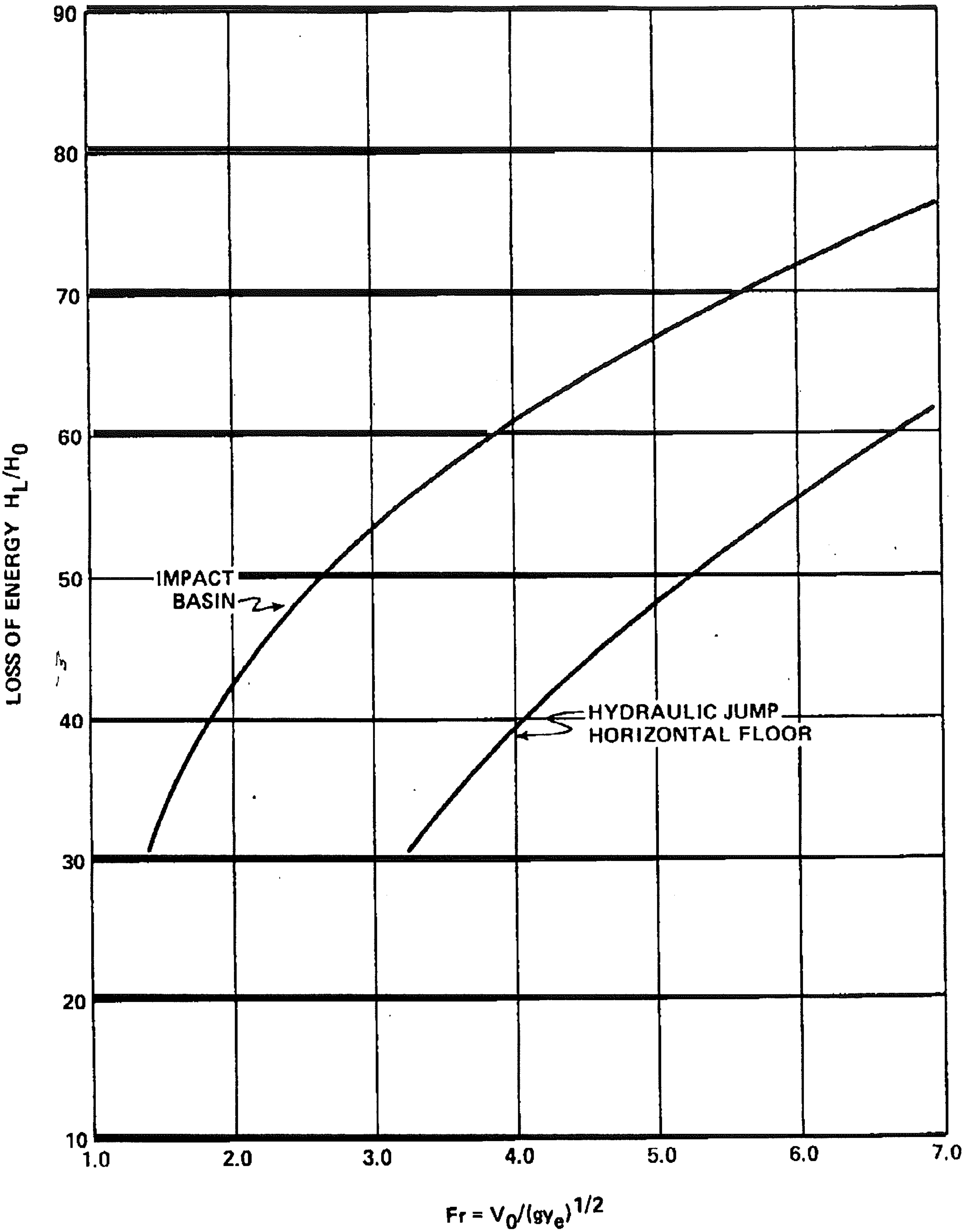


FIGURE VIII–C–3. ENERGY LOSS–IMPACT BASIN–HYDRAULIC JUMP

BAFFLE WALL DISSIPATOR - TABLE VIII-C-1.

Dimensions of basin in feet and inches

W	h_1	L	h_2	h_3	L_1	L_2	h_4	W_1	W_2	t_3	t_2	t_1	t_4	t_5
4-0	3-1	5-5	1-6	0-8	2-4	3-1	1-8	0-4	1-1	0-6	0-6	0-6	0-6	0-3
5-0	3-10	6-8	1-11	0-10	2-11	3-10	2-1	0-5	1-5	0-6	0-6	0-6	0-6	0-3
6-0	4-7	8-0	2-3	1-0	3-5	4-7	2-6	0-6	1-8	0-6	0-6	0-6	0-6	0-3
7-0	5-5	9-5	2-7	1-2	4-0	5-5	2-11	0-6	1-11	0-6	0-6	0-6	0-6	0-3
8-0	6-2	10-8	3-0	1-4	4-7	6-2	3-4	0-7	2-2	0-7	0-7	0-6	0-6	0-3
9-0	6-11	12-0	3-5	1-6	5-2	6-11	3-9	0-8	2-6	0-8	0-7	0-7	0-7	0-3
10-0	7-8	13-5	3-9	1-8	5-9	7-8	4-2	0-9	2-9	0-9	0-8	0-8	0-8	0-3
11-0	8-5	14-7	4-2	1-10	6-4	8-5	4-7	0-10	3-0	0-9	0-9	0-8	0-8	0-4
12-0	9-2	16-0	4-6	2-0	6-10	9-2	5-0	0-11	3-0	0-10	0-10	0-8	0-9	0-4
13-0	10-0	17-4	4-11	2-2	7-5	10-0	5-5	1-0	3-0	0-10	0-11	0-8	0-10	0-4
14-0	10-9	18-8	5-3	2-4	8-0	10-9	5-10	1-1	3-0	0-11	1-0	0-8	0-11	0-5
15-0	11-6	20-0	5-7	2-6	8-6	11-6	6-3	1-2	3-0	1-0	1-0	0-8	1-0	0-5
16-0	12-3	21-4	6-0	2-8	9-1	12-3	6-8	1-3	3-0	1-0	1-0	0-9	1-0	0-6
17-0	13-0	22-6	6-4	2-10	9-8	13-0	7-1	1-4	3-0	1-0	1-1	0-9	1-0	0-6
18-0	13-9	23-11	6-8	3-0	10-3	13-9	7-6	1-4	3-0	1-1	1-1	0-9	1-1	0-7
19-0	14-7	25-4	7-1	3-2	10-10	14-7	7-11	1-5	3-0	1-1	1-2	0-10	1-1	0-7
20-0	15-4	26-7	7-6	3-4	11-5	15-4	8-4	1-6	3-0	1-2	1-2	0-10	1-2	0-8

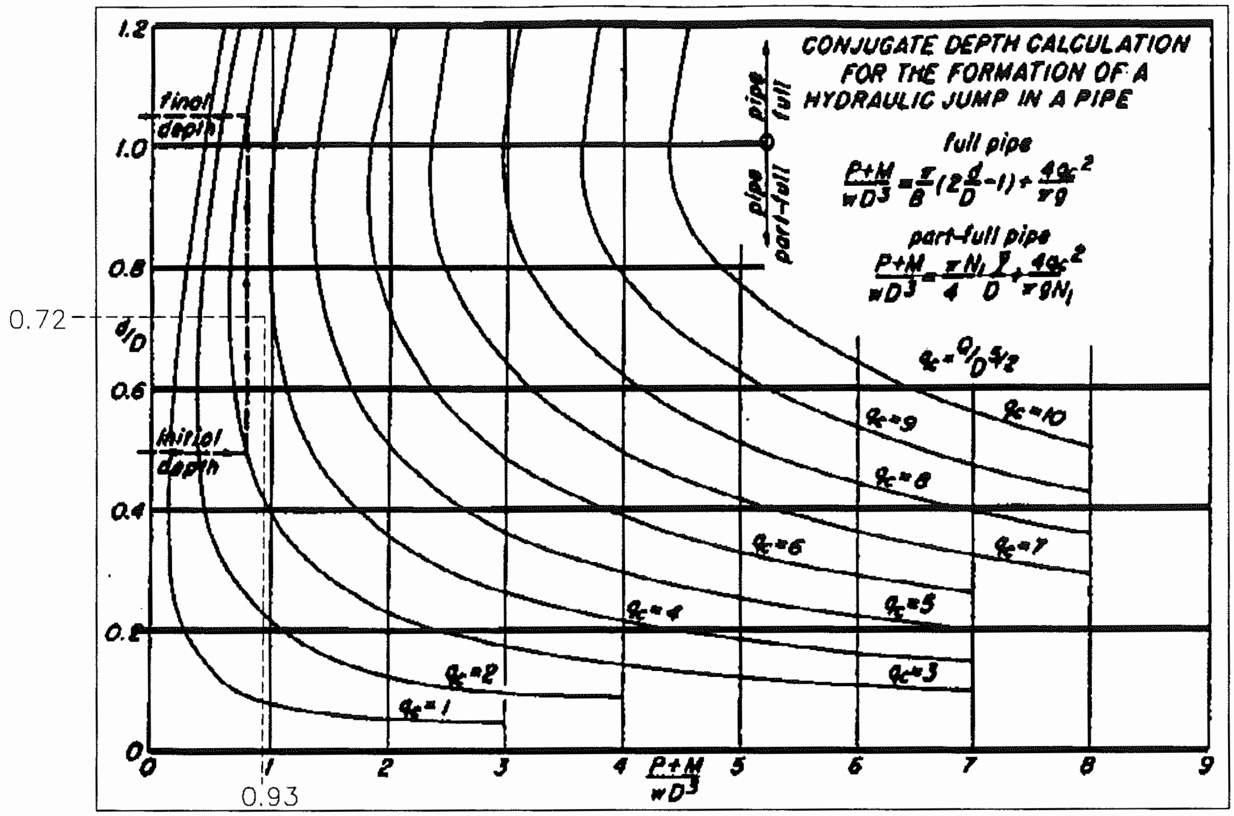


Figure 61. - Pressure plus momentum for various depths of flow in circular pipe

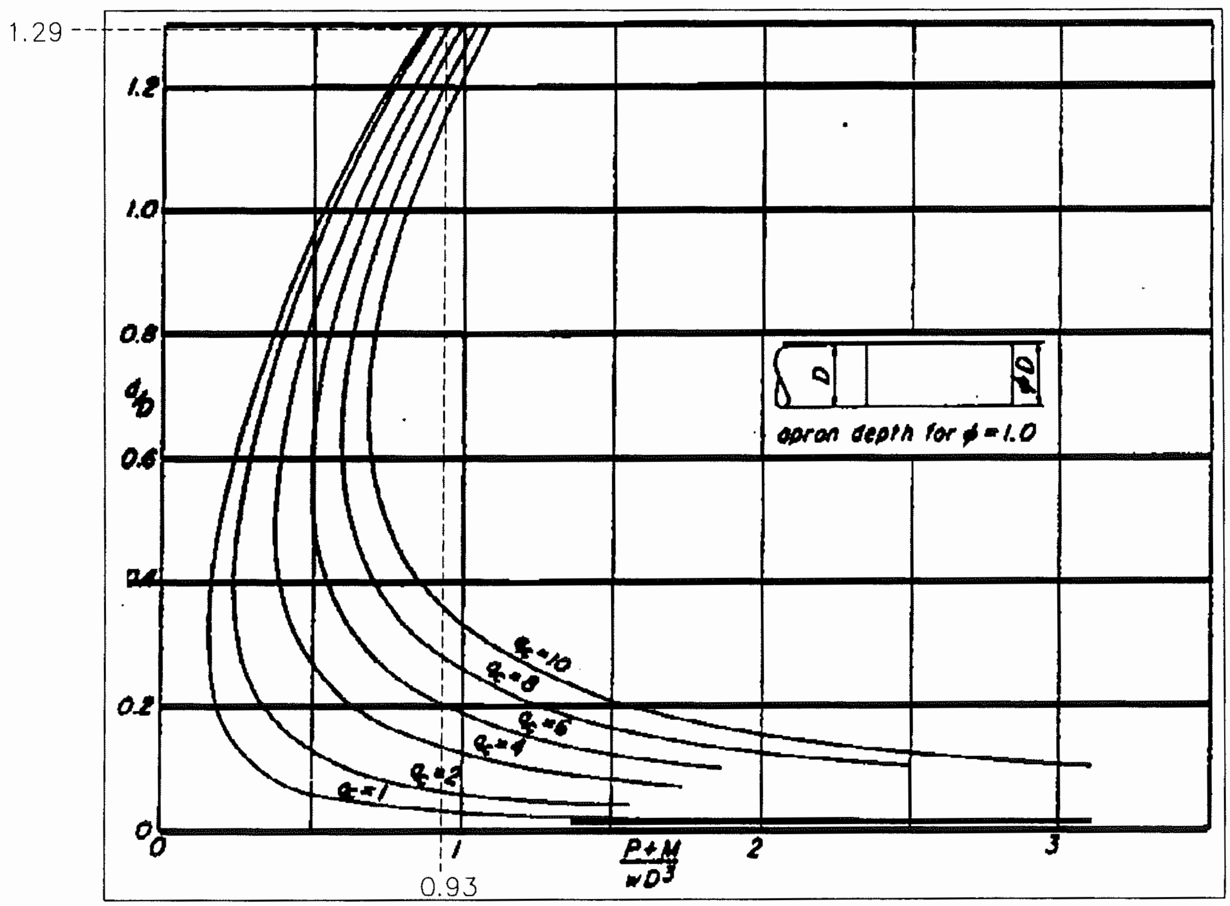


Figure 62a. - Apron depth for end of apron width equal to D

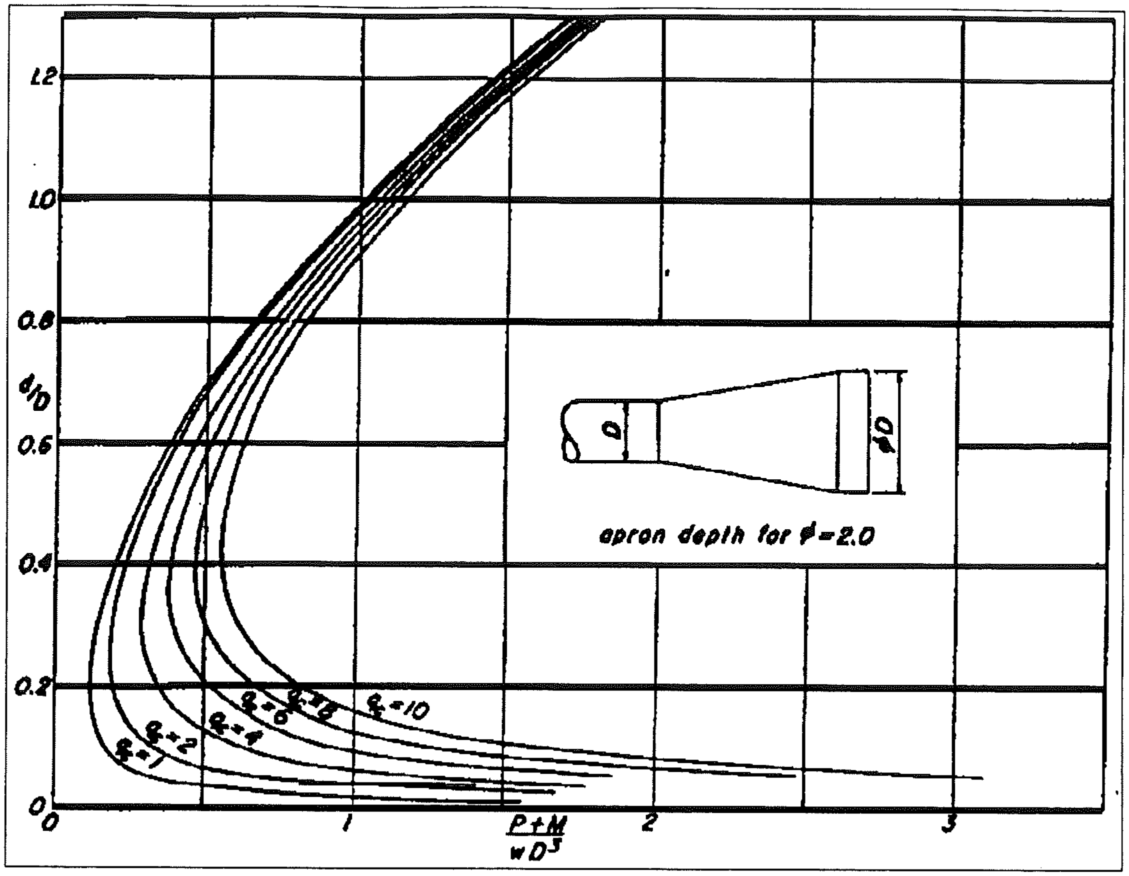


Figure 62b. - Apron depth for end of apron width equal to 2D

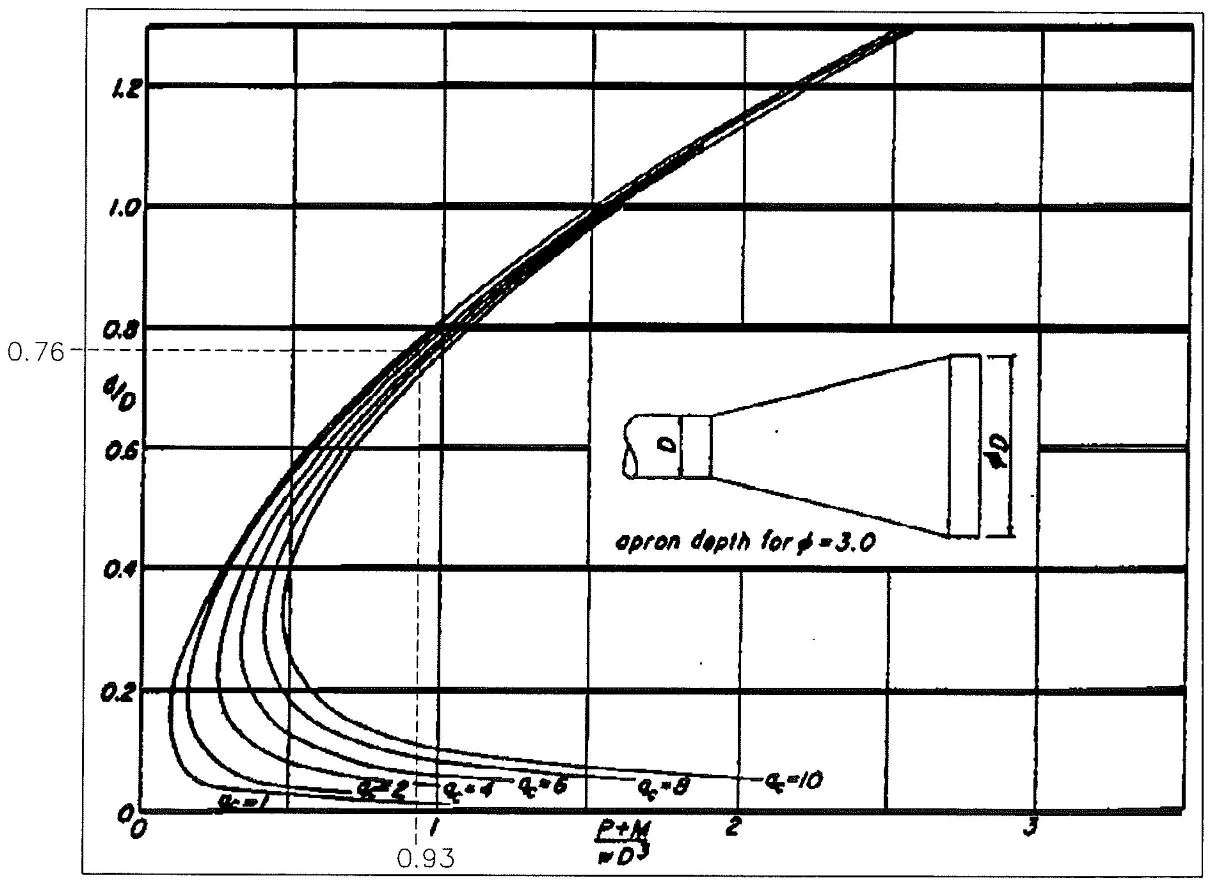


Figure 62c. - Apron depth for end of apron width equal to 3D

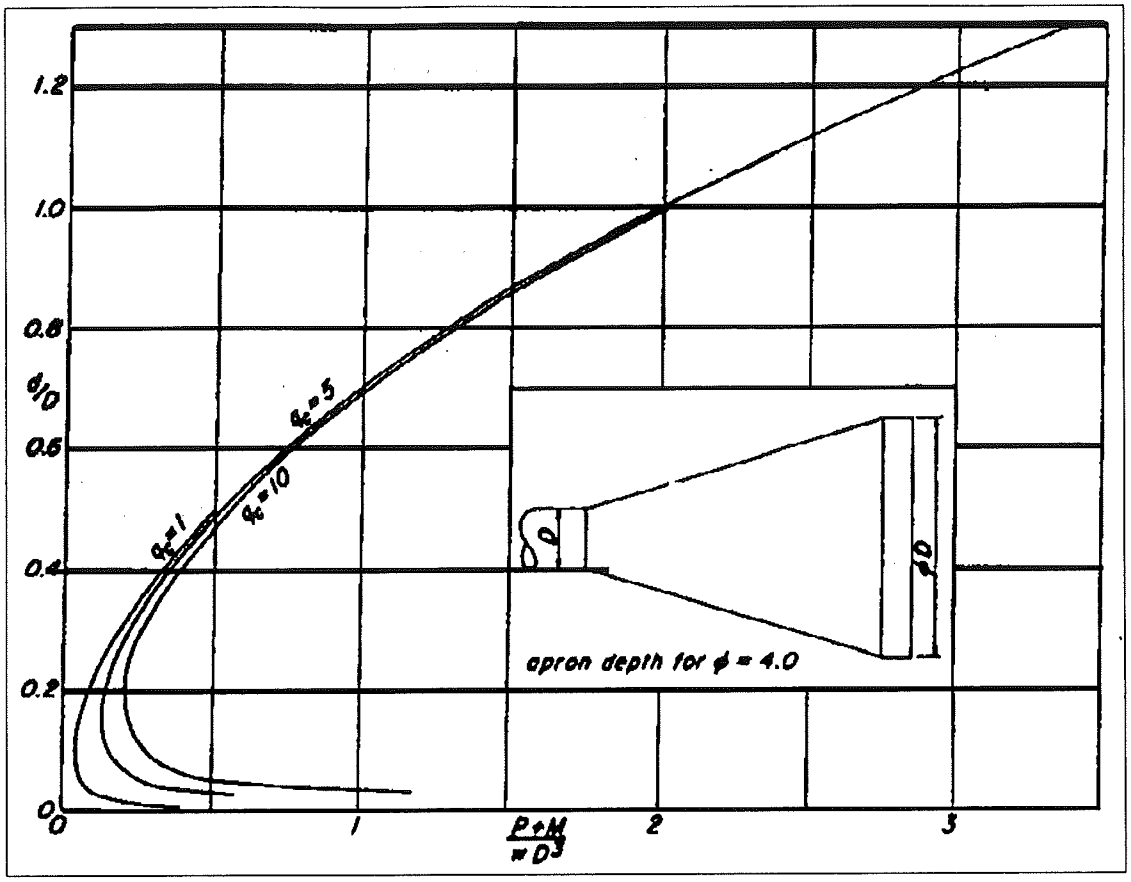
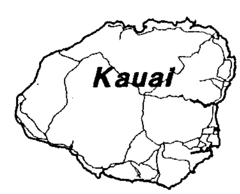


Figure 62d. - Apron depth for end of apron width equal to 4D

Appendix B - Submittal Forms



HYDRAULIC SUMMARY SHEET

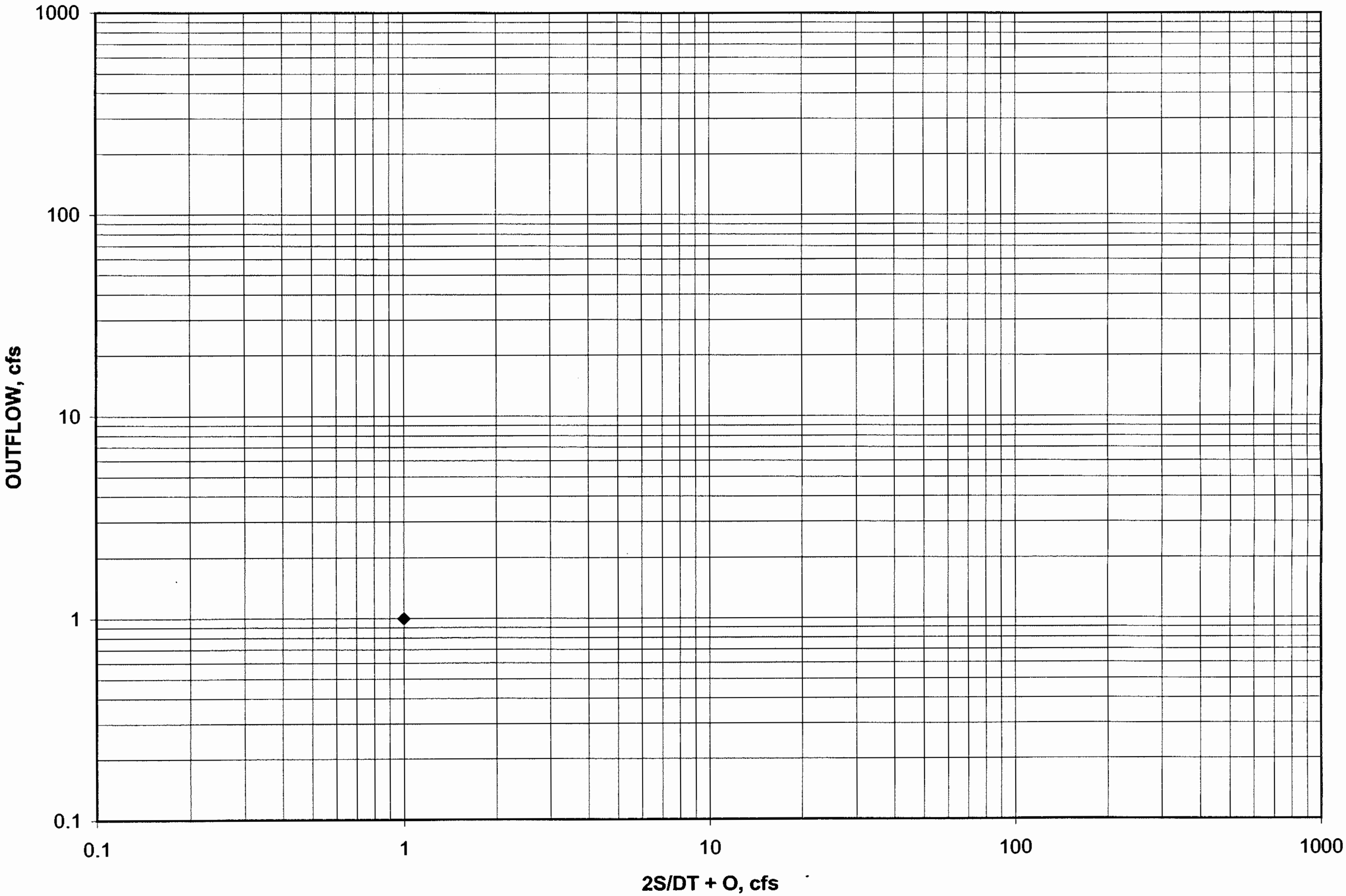
PROJECT:
DRAINLINE:

DATE:
COMPUTED BY:
CHECKED BY:

Inlet	Time of Concentration	Rainfall	Correction Factor	Runoff	Accumulated Runoff	Normal Depth	Critical Depth	Normal Velocity	Pipe Size	Pipe Slope	Friction Slope	Invert Elevation	Friction Loss	Headwater	Hydraulic Grade Elevation		Finish Grade Elevation	Remarks
															Down	Up		

ONSITE DRAINAGE CALCULATION
COUNTY OF KAUAI

	RUNOFF								CURB AND GUTTER		SWALE	INLET			
LOCATION	AREA acres	C	I	Q_d cfs	Carryover cfs	Total, Q cfs	RDWY. SLOPE	$Q_t/S_g^{1/2}$	FLOOD WIDTH, ft	FLOW DEPTH, ft	DEPTH ft	CAPACITY cfs	BYPASS cfs	INLET TYPE	REMARKS



1000
100
10
1
0.1
OUTFLOW, cfs
0.1
1
10
100
1000
2S/DT + O, cfs

DEPTH STORAGE

ELEVATION	DEPTH	BASIN AREA	AVERAGE BASIN AREA	INCREMENT OF DEPTH	INCREMENT OF VOLUME	TOTAL VOLUME
feet	feet	acres	acres	feet	acre-feet	acre-feet

DEPTH OUTFLOW SUMMARY

ELEVATION	DEPTH	Q1	Q2	Q3	TOTAL Q
feet	feet	cfs	cfs	cfs	cfs

ROUTING CURVE SUMMARY

2S/T + O = 240S + O

ROUTING PERIOD = .1 hour

ELEVATION	S (STORAGE)	O (OUTFLOW)	2S/T + O
feet	acre-feet	cfs	csf

ROUTING FORM

1 LINE	2 TIME	3 INFLOW (2)	4 INFLOW (1) + INFLOW (2)	5 2S(1)/t - (1)	6 2S(2)/t + O(2)	7 OUTFLOW, O(2)