Santa Clara County
California

Drainage Manual
2007

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Drainage Manual
Adopted August 14, 2007

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This edition of the Santa Clara County Drainage Manual has been prepared to provide engineers with the requirements for the design of storm water facilities in the County. The manual sets forth County standards for storm drainage design in accordance with the County's subdivision and land development regulations. It is being made available to public and private engineers to provide consistent design procedures throughout the County. For more information, please contact the Land Development Engineering Department.

Very truly yours,

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and Development Services

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1. INTRODUCTION

1.1 Purpose

The Office of Development Services has prepared this drainage manual to provide a framework for the various hydraulic and hydrologic analyses necessary to plan and design storm drainage and flood control facilities within Santa Clara County. By providing this tool to landowners, developers, engineers and other agencies, Santa Clara County anticipates that when used in conjunction with other agency manuals and design criteria, the information contained herein will help produce consistent and equivalent results.

This edition of the Drainage Manual is an update to the manual published in March 1966. As predicted nearly forty years ago, tremendous urbanization within the Santa Clara Valley has strained many storm drainage and flood protection systems. Continuing development and redevelopment within the county will only exacerbate potential impacts to storm drainage infrastructure. This manual is thus intended to provide methodologies to evaluate the impact of development on storm drainage infrastructure and design drainage facilities and to accommodate planned growth and redevelopment.

Consistent design and evaluation criteria for storm drainage systems help the Office of Development Services and other agencies review storm drain and flood protection designs and impact statements for projects throughout Santa Clara County, both within and outside of incorporated areas. This manual identifies the multiple design standards, methods of analyses, and engineering tools required for the planning and design of storm drainage systems and flood control facilities within the County.

1.2 Santa Clara County

Santa Clara County encompasses 1,315 square miles at the southern end of San Francisco Bay. The county's population (approximately 1.7 million people in 2000, representing about one-quarter of the Bay Area's total population) makes it the largest of the nine Bay Area counties and the fifth largest in California.

Neighboring counties include San Mateo to the northwest, Alameda and San Joaquin to the north, Stanislaus to the east, Merced to the southeast, San Benito to the south, and Santa Cruz to the west.

1.3 Topography

Santa Clara County's land forms are characterized by sub-parallel coastal mountain ranges with intervening valleys. Major topographical features include the Santa Clara Valley, which is framed by the Diablo Range to the east, the Santa Cruz Mountains to the west; and the Baylands in the northwest adjacent to San Francisco Bay. Elevations range from sea level at the bay to 4,372 feet at Copernicus Peak in the Diablo Mountain Range and to 3,806 feet at Loma Prieta in the Santa Cruz Mountains.

The valley floor was formed over millions of years from rainfall, runoff, and eroding sediment from the defining mountain ranges. The width of the valley floor varies from approximately 14 miles in the north to less than 1 mile centrally, and about 5 miles at the southern end.

Rolling hills surround the relatively flat and fertile Santa Clara Valley. The Diablo Range, consisting mainly of grassland, chaparral and oak savannah, dominates the entire eastern half of the county. Along the western spine, the Santa Cruz Mountains contain rolling grasslands and oak-studded foothills, mixed hardwoods and dense evergreen forests. The higher elevations of the Santa Cruz Mountains are characterized by redwood forests, steep slopes and active earthquake faults. Areas of geologic instability are prevalent in both mountain ranges.

In the northwestern corner of the county on the San Francisco Peninsula, Baylands areas adjacent to the southern San Francisco Bay waters consist mostly of vast salt evaporation ponds, significant portions of which are undergoing restoration to salt marsh and wetlands.

1.4 Climate

The county's regional climate is Mediterranean, generally remaining temperate throughout the year due to the area's geography and proximity to the Pacific Ocean. Temperatures in the Santa Clara Valley range from 35 to 60 degrees Fahrenheit in the winter and 50 to 100 degrees Fahrenheit during the summer. Mean annual precipitation ranges from 10 inches in the inland valley areas to 56 inches at the top of the Santa Cruz Mountains.

1.5 Land Uses

Santa Clara Valley from San Jose north to Palo Alto (North Valley) is extensively urbanized, housing about 90 percent of the County's residents. Santa Clara County

south of San Jose remains predominantly rural, with the exception of the Cities of Gilroy and Morgan Hill, and the small unincorporated community of San Martin. Low density residential developments are also scattered throughout the southern valley and foothill areas.

Urban growth within Santa Clara County is primarily concentrated on the valley floor with some development located in the foothills. Over 30 percent of the Santa Clara Valley is residential with an additional 5 percent occupied by commercial industries, many of which are electronics and computer companies.

1.6 Growth

Urbanization in the second half of the twentieth century changed Santa Clara County from an area of relatively isolated agricultural communities to one of continuous urban development with both suburbs and emerging urban cores. This trend is expected to continue in the first part of the twenty-first century; although to preserve remaining open spaces, many land use agencies are proposing more intense development within established urban areas to meet an increasing population.

Between 1980 and 1990, the county grew by about 200,000 people (16 percent); and by the year 2000, another 185,000 people made Santa Clara County their home (12 percent growth). Planners generally predict that the County's population will continue to grow, but at a slower rate. According to the Association of Bay Area Governments, by 2010, Santa Clara County's population is projected to increase by nearly 200,000 people to almost 1.9 million, and to exceed 2 million by 2020. North County areas are expected to grow the most in terms of absolute population, but South County areas are likely to grow at faster rates.

1.7 Drainage Design

The owner of a proposed development or redevelopment is ultimately responsible for the design of the proposed drainage works to dispose of stormwater runoff from or through an area without endangering lives or property, to the extent that it can be economically justified without unmitigated environmental impact.

Designers are solely responsible for the evaluation of public safety, for providing appropriate levels of economic protection, for ensuring maintainability of drainage facilities, for assessing the environmental impacts, and for implementing the analytical procedures most appropriate for the project at hand.

This manual is not intended to supplant the judgment of qualified registered engineers with regard to storm drain analyses, evaluation, or design. Rather, the manual is to be used primarily for the standardization of design and review practices by the Office of Development Services and other cities and agencies within the county who choose to use this manual, or portions thereof, as an evaluation tool. The computation of design flows and the review of planned drainage facilities by the Development Services staff, however, will be in conformance with the procedures outlined herein.

Since the County may assume maintenance responsibilities for completed projects, the Office of Development Services must be assured that proposed storm drain designs or flood protection projects will not require excessive maintenance, and that there are no known jurisdictional obstacles to project maintenance.

2. GENERAL DRAINAGE POLICIES

Policies and procedures introduced in this chapter shall be used by the Office of Development Services for the design and review of drainage systems that fall within its direct jurisdiction. In the absence of other specific design or planning guidance from another governing body, these policies are intended to apply throughout Santa Clara County.

This chapter defines flood protection and drainage terminology; establishes general procedures for hydrologic analyses and design; describes required levels of protection; summarizes recurrence intervals for design discharges and water surface elevations; lists recommended calculation procedures for various categories of projects based on drainage area size; and describes a general framework of regulatory considerations that may be applicable to flood protection and drainage projects.

2.1 Flood Protection and Drainage Terminology

Terms used with regularity throughout this manual are defined below.

Annual Series – A general term for a set of any kind of data in which each item is the maximum or minimum in a year. This manual deals most often with annual maximum runoff and precipitation (rainfall).

Design Discharge or Design Flow – The design discharge or flow is defined as the maximum flow that a structure or system of structures is expected to pass. Usually, the drainage or flood protection system is expected to safely pass the design discharge without causing damage to property or people. Design discharge is typically expressed as a peak rate of flow, with English units of cubic feet per second (cfs), and metric units of cubic meters per second (cms).

Design Storm – The design storm is defined as the temporal (time variant) distribution of a specified design rainfall depth (inches) that is a function of the storm duration (hours) and frequency (years). Table D-1 and Figure D-1 in Appendix D provide tabular and graphical representations of the adopted 24-hour incremental distribution (i.e. rainfall pattern) for Santa Clara County. This pattern is based on the three-day December 1955 rainfall event, still considered to be the storm of record for Northern California.

Flood Frequency – The frequency of flood and precipitation events are described in practice and in this manual using complementary terminology. "Exceedance frequency" refers to the probability of any individual precipitation or runoff event in any water year exceeding a certain threshold. Ten-percent peak discharge is that flow rate with a ten percent chance of being equaled or exceeded during any water year. A "one percent, 6-hour precipitation depth" is that 6-hour depth of rainfall that has a one percent probability (chance) of being equaled or exceeded during any water year.

An alternate terminology used in practice is the "recurrence interval," which is also referred to as a "return period." This terminology uses a number of years to specify a flood event. For example, a "ten-year peak discharge" would be that discharge expected to be equaled or exceeded once every ten years *on the average*. The "100-year, 6-hour precipitation depth" is the 6-hour depth of rainfall expected to be equaled or exceeded at a location once every 100 years *on the average*. Annual hydrologic events are considered to be independent of one another; so there is a finite probability of exceeding the 100-year runoff in back-to-back water years. Experiencing a rainfall or runoff event of a certain magnitude does not lessen the chance (probability) of experiencing another event of equal or greater magnitude in subsequent years.

These two frequency terminologies are related as the reciprocal of one another. The ten-percent event (0.10 probability) is equivalent to the ten-year event because the reciprocal of 0.10 is 10. Similarly the one-percent event (0.01 probability) is the 100-year event.

Freeboard – The vertical distance between an elevation of interest (e.g. water surface, hydraulic grade line, or energy grade line) and the elevation of containment, such as the top of stream bank, street grade, or floodwall. Freeboard is intended to provide for a factor of safety in the design of stormwater storage and conveyance facilities.

Frequency Interval – The frequency interval (or recurrence interval) of a peak flow is the number of years, on average, in which the specified flow is expected to be equaled or exceeded one time. Exceedance probability and frequency interval are mathematically inverse of each other; thus, an exceedance probability of 0.01 is equivalent to a frequency interval of 100 years. For example, a peak flow with a 100-year frequency interval will, on average, be equaled or exceeded once every 100 years and has an exceedance probability of 0.01 (a 1-percent chance of being exceeded in a given year). Frequency intervals refer to the average number of occurrences over a long period of time; for example, a 100-year flood is statistically expected to occur about 10 times in a 1,000-year period, rather than exactly once every 100 years. Additionally, it should be noted that

the occurrence of a flood of a given frequency interval in a given year does not affect the probability of such a flood occurring again the next year.

Watershed (or Drainage Area) – The total area that drains overland to a particular "point of interest." In this manual there are three categories of "drainage area." A Large Drainage Area drains an area greater than 200 acres. A Small Drainage Area drains an area less than or equal to 200 acres. A Very Small Drainage Area, a subset of Small, drains an area less than or equal to 50 acres.

2.2 General Procedure

Standards for design are expressed as the return period of the design flow. That is, the planning, analysis, and design of drainage and flood control facilities begin by establishing a required level of protection. From a hydrologic standpoint, this involves ascertaining the recurrence interval to be used. A level of protection is also associated with providing freeboard above the design water surface elevation. The design water surface elevation is also referred to as the design hydraulic grade line (HGL).

As described in this chapter, specific hydrologic methods used to calculate design discharges, size facilities, and to establish HGLs depend on the area tributary to the subject flood protection or drainage facility, as does the recurrence interval used for analysis or design.

Projects in Santa Clara County shall be designed such that the stormwater runoff generated from the 10-year design storm is conveyed in the storm drainage system (underground pipes and/or stable open channels) and the stormwater runoff generated from the 100-year design storm is safely conveyed away from the project site without creating and/or contributing to downstream or upstream flooding conditions.

2.3 Flood Protection Levels

The following levels of flood protection are considered to be the minimum acceptable for new projects within Santa Clara County.

Ten-year runoff shall be contained by a storm drain system consisting of underground pipe and/or stable open channels. New storm sewers and channels shall be designed to safely convey the 10-year storm without surcharge. Existing storm drain facilities may be used to convey flow, as long as one foot of freeboard meeting the criteria set forth in Table 2-1 is satisfied for the new ten-year design flow.

Table 2-1: Freeboard Criteria for Existing Storm Drain Systems

System Type	Freeboard Criterion (1 foot)
Storm Drains in Paved Areas	From HGL to Nearest Flow Line (Gutter for Streets)
Storm Drains in Unpaved Areas	From HGL to Nearest Manhole or Field Inlet Rim Elevation
Open Channels	From HGL to Lowest Adjacent Bank Elevation

As shown in Table 2-1 existing storm drain facilities shall not be considered adequate unless they are capable of conveying the 10-year runoff with one foot of freeboard between the 10-year water surface elevation and the nearest gutter elevation (storm drains in paved areas), nearest manhole or filed inlet rim elevation (storm drains in unpaved areas) or lowest adjacent bank elevation (open channels). The failure of the existing storm drainage system to provide the required freeboard may necessitate storm drainage system upgrades and/or replacement and/or on-site detention as a condition of approval for proposed development.

Flows in excess of the ten percent flood up to the one percent flood shall be conveyed in the streets, provided that development is not subject to flooding. Excess stormwater volume may also be detained in open space areas and parking lots, provided that development is not subject to flooding.

A safe release shall be provided for the design 100-year flow. Within urbanized areas, the 100-year discharge may be carried by a combination of a storm drain system and surface flow on the street, as long as the hydraulic grade line is contained within street rights-of-way. Under no circumstances shall the energy grade line (water surface plus velocity head) exceed the finished floor elevation of any structure, including garages.

Gravity drainage is to be utilized insofar as practicable. Areas draining to a closed sump shall be drained utilizing pumping facilities provided with standby power and automatic transfer switches.

Improvements to facilities that are part of a FEMA Flood Insurance Study must be designed to contain the FEMA 100-year water surface elevation using FEMA criteria as discussed in Chapter 5.

At the project engineers' or County's discretion, more stringent design standards for flood protection levels may be applied on a case-by-case basis if the return periods or freeboard given above fail to provide sufficient public safety, if the economic consequences of project failure warrant increased flood protection, or if more stringent standards are judged to be in the public's or project's best interest.

Proposals for projects with design return periods or freeboard less than those provided above may be considered only when supported by clear evidence that economic considerations warrant reduced standards. In these cases, project sponsors must select appropriate design criteria on the basis of economic analyses and demonstrate that the proposed return period and freeboard provide adequate protection. Such analyses shall be submitted for approval by the Department of Planning and Development Services. In no case will economic considerations take precedence over public safety or over providing adequate flood protection to other properties.

2.4 Hydrologic Calculations

2.4.1 Small Drainage Areas

The Rational Method (Chapter 3) may be utilized to estimate peak discharges for site projects with tributary areas less than or equal to 200 acres, where storage effects are not significant. The Rational Method is generally considered acceptable to determine peak design discharges for small urban catchments. Despite falling under the catchment area criterion for smaller site projects, the Rational Method should not be used for tributary areas less than or equal to 200 acres if: the watershed has a large percentage of pervious area with widely varying soil types, or there is substantial surface storage considered to effectively reduce the peak discharge or, with the exception of Very Small Drainage Areas, when detention basin computations are required.

2.4.2 Large Drainage Areas

The Unit Hydrograph Method described in Chapter 4 shall be used to estimate peak discharges for site projects with tributary areas greater than 200 acres, or where runoff storage is a significant consideration. The Unit Hydrograph Method shall also be utilized on catchments with a high percentage of pervious area or with widely varying soil types, and to evaluate all but the simplest detention basins (i.e., those with very small drainage areas).

2.4.3 Major Projects

While the Unit Hydrograph Method described in Chapter 4 is one means of providing design discharge estimates for projects involving major flood protection or drainage facilities, methodologies to estimate design flows from streamflow records, regression analyses and other stochastic techniques are beyond the scope of this drainage manual. For projects utilizing or impacting such facilities, planners and designers are referred to the Santa Clara Valley Water District for further guidance.[1]

2.5 Projects Falling Within Santa Clara Valley Water District Jurisdiction

Any project that requires a permit from the Santa Clara Valley Water District must comply with all processes and procedures as put forth by that District. Currently the District has permit jurisdiction along all watercourses in the County with a drainage area greater than 320 acres (1/2 square mile.) The District currently also has permit jurisdiction fifty (50) feet from the top of bank along each side of a watercourse whose drainage area is greater than 320 acres. Up-to-date District requirements can be found by contacting the District or on the District's web site: www.valleywater.org.

2.6 Projects Requiring an NPDES Permit

As authorized by the Federal Clean Water Act, the National Pollutant Discharge Elimination System (NPDES) permit program controls water pollution by regulating point sources that discharge pollutants into waters of the United States. In California, the Regional Water Quality Control Boards are responsible for enforcing the federal regulations through issuance of waste discharge requirements. The California Regional Water Quality Control Board, San Francisco Bay Region (Regional Board) has jurisdiction over waters in and around the Santa Clara Valley.

In order to comply with these requirements, Santa Clara County, the Santa Clara Valley Water District and 13 cities in the Santa Clara Valley have joined together to form the Santa Clara Valley Urban Runoff Pollution Prevention Program (Program). The participating members of the program (Dischargers) are required to implement the stormwater pollution management measures outlined in the Santa Clara Valley Urban Runoff Management Plan (Management Plan) to control the quality of the stormwater entering their storm drainage systems. The Management Plan establishes a framework for management of stormwater discharges, sets forth the Program's objectives, and contains performance standards required of each of the Dischargers.

Current NPDES regulations can be found on the web site www.scvurppp.org or by inquiring at the applicable planning or building permit authority for any project.

2.7 CEQA Regulations

The California Environmental Quality Act (CEQA) requires an environmental review of projects proposed to be undertaken or requiring approval by State and Local government agencies. The goal of the CEQA review process is to identify the significant environmental impacts of projects and to either avoid the impacts or mitigate for them.

CEQA requires that an environmental checklist form be completed in order to address whether the project would affect a number of environmental factors, including hydrology and water quality. Hydrologic and water quality issues that must be addressed in the current CEQA environmental checklist include:

- Violation of any water quality standards or waste discharge requirements
- Substantial depletion of groundwater supplies or interference with groundwater recharge
- Substantial alteration of existing drainage patterns resulting in substantial on- or off-site erosion or siltation
- Substantial alteration of existing drainage patterns resulting in substantial on- or off-site flooding
- Creation or contribution of runoff water which would exceed the capacity of existing or proposed stormwater drainage systems or provide substantial additional sources of polluted runoff
- Other substantial degradation of water quality
- Placement of housing within a 100-year flood hazard area as mapped on a federal Flood Hazard Boundary or Flood Insurance Rate Map or other flood hazard delineation map
- Placement of structures within a 100-year flood hazard area which would impede or redirect flood flows
- Exposure of people or structures to a significant risk of loss, injury or death involving flooding, including flooding as a result of the failure of a levee or dam

Each of the items addressed in the CEQA environmental checklist are evaluated to determine whether the project results in no impact, less than significant impact, less than significant impact with mitigation incorporation, or potentially significant impact. If a determination of "less than significant impact with mitigation incorporation" is sought, the project applicant may be required to determine feasible mitigation alternative(s) and

demonstrate that the mitigation alternative(s), once incorporated into the project, result in less than significant environmental impacts. In terms of hydrology and water quality, mitigation alternatives may include, but not be limited to: water quality ponds, detention ponds, retention ponds, and low-flow diversions for riparian habitat and/or wetland restoration.

Up-to-date CEQA requirements may be obtained by requesting information from the responsible land use agency or building permit-granting agency.

2.8 Other Permits

Permits from local, state, regional and federal agencies may be required to complete the construction, operation and maintenance of storm drainage and flood protection facilities. The County bears no responsibility for determining the permits required for any given project. Agencies, acts and permits listed in this chapter are common to typical storm water related projects, but are not intended to be all-inclusive.

Examples of such permits are: U.S. Army Corps of Engineers Section 404 of the Clean Water Act; National Marine Fisheries Section 10(a)(1)(B) of the Endangered Species Act; and State of California Department of Fish and Game Sections 1601 and 1603 Streambed Alteration Permit. Requirements for these and other necessary permits should be determined by contacting the permitting agency.

2.9 Projects Requiring Detention Storage

Detention storage may be required to mitigate for the loss of existing storage within a watershed; increased stormwater runoff due to increased imperviousness within a watershed; or the degradation of water quality resulting from proposed development within a watershed. Wherever possible, the County encourages regional detention facilities maintained by a public agency rather than individual site facilities on private property.

Existing storage within a watershed may take the form of depression storage, swales, natural channels, and increased roughness due to vegetation within the watershed. Development within a watershed may reduce the amount of storage by clearing the area and increasing the amount of impervious surfaces.

An increase in impervious surfaces (e.g. roofs and pavement) resulting from development within a watershed may lead to an increase in the amount of surface water

runoff generated from the project site (both peak rate – and thus velocity – and volume). These increases are generally attributed to the loss of infiltrative capacity of the soil and the loss of vegetation within the watershed. They may also be the function of improved drainage and lost storage.

A portion of the rainfall that is infiltrated into the ground is absorbed by the soil and the remaining infiltrated rainfall flows through the soil as subsurface flow, including interflow and groundwater. Precipitation is also intercepted by vegetation and released back to the atmosphere through transpiration. Finally, a portion of the rainfall is captured in depression storage within the watershed. Collectively, infiltration, interception and depression storage are referred to as "hydrologic losses."

Development within a watershed may also lead to degradation of the quality of the stormwater runoff from the project site. The increase in impervious surface due to development can result in more pollutants being discharged directly to the storm drainage system and its receiving waters. Pursuant to current NPDES requirements discussed in Section 2.6, new development or significant redevelopment shall be required to incorporate site design practices that reduce the impacts of development on water quality. The incorporation of landscape-based measures, including detention storage, is one such site design measure.

As previously discussed, unit hydrograph analyses are required for large site projects with catchments larger than 200 acres, and where detention storage is a significant component. Analyzing storm runoff hydrographs allows for a more detailed accounting of the hydrologic processes occurring within a watershed, including hydrologic losses, natural storage, routing, and the evaluation of detention systems. Hydrograph analyses shall be used to analyze the effects of detention storage for project sites draining more than 50 acres. For project sites with drainage areas smaller than 50 acres, a modified Rational Method approach suitable for analyzing the effects of detention storage described in Chapter 6 may be used.

A summary of the hydrologic method criteria is shown in Table 2-2.

Table 2-2: Summary of Hydrologic Method Criteria

A. If projects require review, approval, or permit issuance by the Santa Clara Valley Water District, then the current hydrologic calculation methods approved by the District shall be used. Generally, these types of projects include:

- Projects that outfall directly into a SCVWD facility;
- Projects being constructed within 50 feet of a SCVWD facility;
- Major projects which involve major flood protection or drainage facilities, or which may affect major flood protection or drainage facilities.

B. If projects do not require SCVWD approval and they meet all of the following bulleted criteria, then the hydrologic method as noted is to be used:

- No detention being used;
- No substantial surface storage effects;
- No large areas of pervious soils.

Size of Drainage Area	Hydrologic Method to be Used
200 Acres or Less (Small Drainage Area)	Rational Method May Be Used
More Than 200 Acres (Large Drainage Area)	Unit Hydrograph Method Must Be Used

C. If projects do not require SCVWD approval and they meet any of the following bulleted criteria, then the hydrologic method as noted is to be used:

- Detention being used
- Substantial surface storage effects
- Large areas of pervious soils

Size of Drainage Area	Hydrologic Method to be Used
50 Acres or Less (Very Small Drainage Area)	Modified Rational Method May Be Used (APWA or ASCE Methods)*
More Than 50 Acres	Unit Hydrograph Method Must Be Used

*Refer to Chapter 6, Section 6.3.1

NOTE: If professional consultants have any questions about what design method should be used, they should contact staff at the Development Services Office before proceeding with the design

2.10 Project Elevation Datum

Currently there are a number of different vertical datum systems in use in Santa Clara County. These include:

- National Geodetic Vertical Datum of 1929 (NGVD 29)
- North American Vertical Datum of 1988 (NAVD 88)
- Mean Lower Low Water (MLLW)
- Numerous local city datum systems

The engineer must be cognizant of various datum systems and be certain that all elevations used for tides, channel cross-sections, tailwater elevations, storm drain system inverts and rim elevations, and other relevant hydrologic data are on consistent vertical datum systems. Engineers should also be aware that conversions between datum systems are not universal, and depend upon site specific locations. The County assumes no responsibility for errors in datum.

2.11 Data Access

This report and all attachments can be found on the official Santa Clara County Web site and may be downloaded free of charge. A selection of Internet resources for obtaining data from the U. S. Geological Survey (USGS) and the National Resources Conservation Service (NRCS) for hydrologic analysis is shown below. The URLs are current at the time of publication, but are of course, subject to change over time.

<u>Digital Elevation Model (DEM) Data:</u>
USGS, *The National Map* Seamless Data Distribution System:
http://seamless.usgs.gov/
USGS, San Francisco Bay Area 24K DEM Index Page:
http://bard.wr.usgs.gov/htmldir/dem_html/index.html

<u>Land Use Data:</u>
USGS, 1:100,000-scale Land Use-Land Cover for California:
http://edcwww.cr.usgs.gov/glis/hyper/guide/1_250_lulcfig/states100k/CA.html

<u>Soils Data:</u>
NRCS, State Soil Geographic (STATSGO) Database:
ftp://ftp.ftw.nrcs.usda.gov/pub/statsgo/dos/arc/data/

1. Download California STATSGO data (ca.zip).
2. Unzip file. You will need all folders and files.

3. In GIS, ADD THEME. The STATSGO data is an ArcInfo coverage. It should be called CA in the "Spatial" folder.
4. The "Comp.dbf" file contains hydrologic soil group (A, B, C, or D) information; add it to your GIS project. The file should be in the "Spatial" folder.
5. Join "Comp.dbf" to the CA theme using "Muid" as the linking attribute (common field name).
6. The "HYDGRP" field in "Comp.dbf" is the hydraulic soil group.

Surface Water Features Spatial Data:
USGS, National Hydrography Dataset:
http://nhd.usgs.gov/

Chapter Endnotes

[1]Santa Clara Valley Water District, Hydrology and Geology Unit. 1998; *Draft Hydrology Procedures*

3. RATIONAL METHOD OF PEAK FLOW ESTIMATION

One primary hydrologic value of interest when evaluating and designing a storm drain collection system is the peak rate of flow that each element must carry. In a highly urbanized area characterized by relatively small watersheds with largely impervious areas, the Rational Method has a long history of usefulness for flood peak estimation and stormwater conveyance system design, where a full hydrograph is not required.

3.1 Use of Rational Method

The Rational Method is used to predict peak flows for small drainage areas that can be either natural or developed. The Rational Method can be used for culvert design, pavement drainage design, storm drain design, and some stormwater facility design. The Rational Method can provide satisfactory estimates for relating peak discharge to rainfall intensity by the formula:

$$Q_T = kCi_T A \qquad (3\text{-}1)$$

Where:	Q	=	peak discharge (cfs)
	T	=	recurrence interval (years)
	k	=	1.008 (most often rounded to 1)
	C	=	a dimensionless runoff coefficient
	i	=	the design rainfall intensity (inches per hour) for a duration equal to the time of concentration for the basin
	A	=	drainage area (acres)

3.2 Underlying Assumptions and Limitations on Use

The Rational Method is based on the premise that under constant rainfall intensity, peak discharge occurs at the basin outlet when the entire area above the outlet contributes runoff. Known as the "time of concentration," this value is defined as the time required for runoff to travel from the most hydraulically distant point (at a drainage divide such as a ridge) to the outlet. When using the Rational Formula, its underlying assumptions should be understood and verified for applicability to site conditions:

1. The frequency interval of the computed peak flow is that of the design rainfall intensity.

2. Rainfall is spatially uniform over the catchment being considered.
3. Rainfall intensity is uniform throughout the duration of the storm.
4. Storm duration, as associated with the peak discharge, is equal to the time of concentration (rainfall intensity averaging time) of the drainage area being considered.

The Rational Method has been shown to provide reasonable estimates for peak discharges on small catchments where storage effects are not significant. The Rational Method is not recommended for drainage areas larger than 200 acres in Santa Clara County, for any catchment where ponding or storage within the catchment might affect the peak discharge, or for catchments that utilize drainage facilities, particularly if they involve storage. It may be possible, in some cases, to adjust the runoff coefficient, C, in the Rational Method formula to account for storage within the catchment; however, this is not recommended since the Rational Method only provides an estimate of the peak discharge, not a discharge hydrograph. A modified Rational Method approach may be used to analyze the effects of detention storage for project sites with drainage areas less than 50 acres.

3.3 Estimating Runoff Coefficients

In the Rational Method, a lumped parameter, C, is used to convert precipitation into direct runoff. This parameter models all of the watershed variables (e.g., infiltration, depression storage, vegetation, evapotranspiration, etc.) that cause only a certain percentage of precipitation to flow off of the catchment as runoff. Estimated values of peak discharge, therefore, are heavily influenced by the selection of runoff coefficients.

Runoff coefficients to be used in analysis and design are listed in Table 3-1 for various land use conditions, ground covers, and hydrologic soil groups (HSG). Runoff coefficients are calibrated to generally match results obtained when using the US Department of Agriculture Soil Conservation Service (SCS but now Natural Resources Conservation Service) Curve Number methodology for representative 200-acre watersheds, which is the threshold for using the unit hydrograph method. Runoff coefficients for watersheds with more than one land use or soil type shall be weighted based on area. Coefficients listed for "shrub land" may be considered for chaparral, an oak-grass complex, and other non-agricultural rural areas. Engineering judgment shall be used to modify runoff coefficients for unique land uses not listed.

In Table 3-1 Soil Types B, C and D are based on the SCS classification of HSG. This designation is a standard designation used by the SCS and has been defined for Santa Clara County in existing SCS publications. D-type soils are less permeable than are C-type soils, which are, in turn, less permeable than B-type soils.

Table 3-1: Runoff Coefficients for Rational Formula

Land Use	C for Soil Type		
	B	C	D
Low Density Residential	0.30	0.40	0.45
Medium Density Residential	0.50	0.55	0.60
High Density Residential	0.70	0.70	0.75
Commercial	0.80	0.80	0.80
Industrial	0.70	0.75	0.75
Parks	0.20	0.30	0.35
Agricultural	0.15	0.35	0.40
Urban Open Space	0.10	0.35	0.45
Shrub Land	0.10	0.20	0.30
Paved / Impervious Surface	0.85	0.85	0.85

The Rational Method implies that this ratio is fixed for a given drainage basin. Studies have shown, however, that the coefficient may vary with respect to prior wetting and seasonal conditions (antecedent moisture). It has also been observed that as rainfall intensity increases, soil permeability decreases. One may sense that runoff coefficients should increase with rainfall intensity.

Applying such non-linearities over relatively small urbanized drainage basins does not necessarily improve hydrologic precision enough to offset the more difficult computations, so using a constant runoff coefficient is standard in Santa Clara County. For watersheds with significant variation in antecedent moisture conditions, soil types, or other complexities, however; the hydrograph method described in Chapter 4 should be employed regardless of basin size.

3.4 Time of Concentration

Effective use of the Rational Formula also depends upon the computation of the time of concentration. As indicated previously, the time of concentration is defined as the travel time of a drop of water from the most hydraulically remote point in the contributing area to the point where the discharge is being determined. The travel time for the water to move down the catchment can include overland flow time and the travel time in street gutters, roadside swales, storm sewers, drainage channels, small streams, and other drainage ways.

3.4.1 3.4.1 Natural Watersheds

For natural watersheds with flow primarily in channels after an initial overland flow distance typically equal to about 300 feet, the Kirpich formula may be used:

$$t_c = 0.0078\left(\frac{L^2}{S}\right)^{0.385} + 10 \qquad (3\text{-}2)$$

Where: t_c = time of concentration (minutes)

L = maximum length of travel from headwater to outlet (feet)

S = effective slope along L (feet per foot)

The concept of effective slope is illustrated by Figure 3-1, showing how the slope of the main hydraulic length is determined by plotting a profile of the channel from the outlet to the divide along the main channel and the primary upstream overland flow path. A straight slope line is drawn on the profile so that the area under the line is the same as that under the profile.

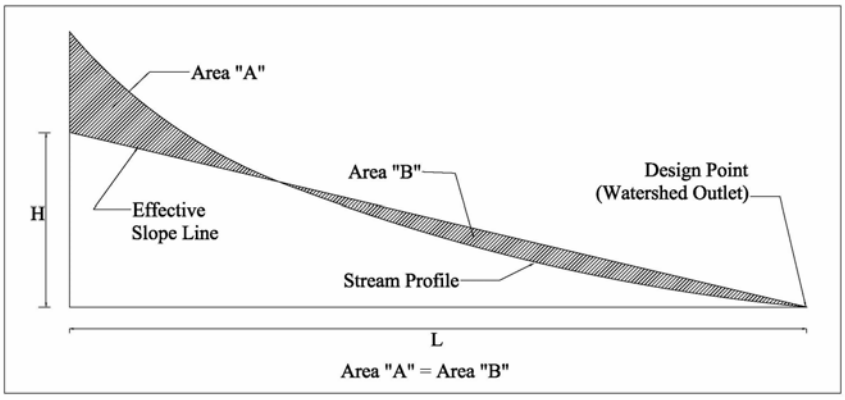


Figure 3-1: Computation of S, Effective Stream Slope

3.4.2 Urbanized Drainage Basins

Runoff from urbanized basins travels in three phases:

1. Initial overland flow represents rainfall collecting on roof tops and making its way to an impervious surface, where runoff begins in earnest. This value is assumed to be ten minutes where a substantial area is drained, and five minutes when only street or parking lot sections are drained.
2. Gutter flow represents the sheet flow of runoff over pavement, other impervious surfaces (e.g. street gutters), or pervious surfaces toward an initial collection point in the storm drain system. Calculations for this portion of the initial travel time shall be based upon nomographs included as Figure A-1 in Appendix A.
3. Pipe flow in a storm drain collection system shall be calculated by dividing the distance between design points by the average flow velocity in the subject reach of pipe. (More detailed procedures regarding storm drain hydraulics are provided in Chapter 5 of this manual.) Since flow velocities may change between inlets or manholes due to changes in flow rate, pipe size, roughness, or slope, the total pipe flow time must be calculated as the sum of the time increments for each section of storm drain.

The total time of concentration to be used in the Rational Method calculation is the sum of the overland flow time plus any travel time in pipes, gutters, swales, or channels leading down to the point where the discharge is being determined.

3.5 Rainfall Intensity

Calculated times of concentration are used to establish the average rainfall intensity to be applied uniformly over the watershed to produce its peak discharge for a specified return period using the Rational Formula.

The Santa Clara Valley Water District's Return Period-Duration-Specific (TDS) Regional Equation has been used to establish a relationship between precipitation depth and Mean Annual Precipitation for various storm frequencies (return periods).

Mean Annual Precipitation isohyets for Santa Clara County have been excerpted from the Santa Clara Valley Water District[1] and presented as Figure A-2 in Appendix A. In areas where the isohyet gradient is relatively constant, particularly for smaller basins, the Mean Annual Precipitation value used to select an IDF curve or apply Equation 3-3 may be selected at the watershed's centroid, or weighted center of an area. In larger areas, particularly those with rapidly changing Mean Annual Precipitation contours,

such as mountain slopes, the catchment's representative Mean Annual Precipitation value should be obtained by true areal averaging. The true areal averaging method may also be most appropriate for larger watersheds of unusual shapes such as: crescents, inverted pear, extremely elongated, or bent.

Twenty-four-hour storm durations are used to establish rainfall intensity-duration-frequency (IDF) curves provided in Appendix B. The TDS Regional Equation is given by:

$$x_{T,D} = A_{T,D} + \left(B_{T,D} MAP\right) \quad (3\text{-}3)$$

Where: $x_{T,D}$ = precipitation depth for a specific return period and storm duration (inches)

T = return period (years)

D = storm duration (hours)

$A_{T,D}$, $B_{T,D}$ = coefficients from Tables B-1 and -2 (dimensionless)

MAP = Mean Annual Precipitation (inches)

The precipitation intensity, $i_{T,D}$ is given by:

$$i_{T,D} = \frac{x_{T,D}}{D} \quad (3\text{-}4)$$

Intensity-Duration-Frequency (IDF) curves for Mean Annual Precipitation (MAP) values of 15-, 20-, 25-, 30-, 35-, and 40-inches are provided for users' convenience in Appendix B. Intensity curves for 2-, 5-, 10-, 25-, 50-, and 100-year return periods are shown for each of the IDF curves. Interpolation may be used to obtain IDF relationships for MAP values between those shown in Appendix B, or Equations 3-3 and 3-4 can be applied directly.

3.6 Rational Method Application

Calculation sheets to assist manual users with Rational Method computations are provided in Appendix C. Generally, the Rational Formula is applied by breaking up the storm drain system and watershed of interest into discrete points of concentration. Land uses and soil types within the individual catchments are tabulated by area, and component runoff coefficients are obtained using Table 3-1. A composite runoff coefficient is obtained by areal weighting. Times of concentration for each catchment are calculated using the procedures described herein, and using the appropriate IDF curve

from Appendix B, rainfall intensity is input into the Rational Formula to provide the estimated peak discharge at each point of concentration.

For small areas, particularly in the upstream reaches of a system, it may be that the highest peak flow will be computed by using a shorter time of concentration from an individual catchment instead of the cumulative time of concentration resulting from travel in the upstream collection system. The peak flow computed from a smaller area, but more intense rainfall intensity due to the short time of concentration, may control the design in the most upstream portion of the drainage system, but as the design moves downstream to larger areas, usually the cumulative effective area (C x A) and longer time of concentration result in higher flows. Whichever combination of cumulative C x A and cumulative time of concentration produces the highest peak discharge at any given point shall govern the design of the drainage system immediately downstream of that point.

Chapter End Notes

[1]Santa Clara Valley Water District, *Mean Annual Precipitation Map, San Francisco and Monterey Bay Region [map]*. 1:250,000. 1988.

4. HYDROGRAPH METHOD

4.1 Applicability

The hydrograph method is generally used when analyzing larger watersheds where the Rational Method should not be applied. The hydrograph method allows the user to account for hydrologic losses, including evaporation, transpiration, infiltration, surface routing, storage within the watershed, and varying antecedent moisture conditions. In addition, the hydrograph method allows for the analysis of complex drainage facilities, including diversions and detention ponds. In practice, this method allows for the development of a flood hydrograph using a design storm, an appropriate infiltration or loss rate technique, and a synthetic unit hydrograph.

The basic process of the hydrograph method includes:

- Simulating rainfall from a specified storm event
- Simulating rainfall losses due to interception and infiltration
- Simulating the overland flow into creeks, channels, or pipes to provide a runoff hydrograph at concentrated points
- Routing the hydrograph through creeks, channels, or pipes
- Routing the hydrograph through detention basins or reservoirs

Detailed explanation of the hydrograph method can be found in the *Handbook of Applied Hydrology* by V. T. Chow,[1] *Hydrology for Engineers* by Linsley, Kohler, and Paulhus,[2] the ASCE Manual and Report on Engineering Practice No. 28, *Hydrology Handbook*,[3] and other appropriate references.

A hydrograph analysis is required for drainage areas greater than 200 acres, or if the drainage area is greater than 50 acres and includes a detention basin or storage reservoir. For areas less than 200 acres or those smaller areas that do not require a detention basin or storage reservoir, the hydrograph method may be applied, but should be compared to the results obtained by application of the Rational Method described in Chapter 3.

4.2 Computer Programs

HEC-1 and HEC-HMS are hydrologic modeling computer programs developed by the Hydrologic Engineering Center (HEC) of the U. S. Army Corps of Engineers. Both programs are designed to compute rainfall-runoff hydrographs and route the hydrographs through channels, pipes, detention basins, and reservoirs. HEC-1 is a

DOS-based program that receives its input data via a text file. It was originally released in 1968; the current version (Version 4.0) was released in 1990. HEC-HMS, originally released in 1998, is a Windows-based program that provides a Graphical User Interface (GUI) intended to simplify data input. HEC-HMS has many of the same capabilities as HEC-1.

This manual is not intended to be a user's guide for either HEC-1 or HEC-HMS. It is assumed that the reader is familiar with one or both of these programs. However, if you are unfamiliar with either program, user's manuals for both can be downloaded from HEC's homepage, http://www.hec.usace.army.mil.

HEC-1 and HEC-HMS follow the same basic procedure for hydrologic modeling described in this section. It is important to recognize that these computer programs are intended only to simulate naturally-occurring events. In many instances, the simulated processes or results can vary significantly from those that occur naturally. Whenever possible, the simulated results should be calibrated with verified results, or at a minimum, analyzed to determine whether the results are reasonable for the given meteorologic, topographic, and land use conditions.

Other hydrologic modeling software packages are available, which include, but are not limited to the following:

- TR-20 and TR-55 developed by the Natural Resources Conservation Service (NRCS)
- Storm Water Management Model (SWMM) developed by the U. S. Environmental Protection Agency (EPA)
- Hydrological Simulation Program-Fortran (HSPF), originally developed as the Stanford Watershed Model
- HYDRO from the Federal Highway Administration (FHWA)
- MIKE SHE, available from the Danish Hydraulics Institute (DHI)

Any hydrology method, including those listed above, may be used, provided the chosen method provides results consistent with those obtained using the County's hydrology procedures.

4.3 Santa Clara Valley Water District Procedures

Projects falling within the District's jurisdiction shall be coordinated with the District and shall be completed using standards and procedures established by the District.

4.4 Rainfall Simulation (Design Storm)

"Design storm" is a term used to describe the total rainfall depth, which is determined from the combination of the return period and storm duration. Chapter 2 describes the design storms to be used for hydrologic and hydraulic analysis and facility design. In Santa Clara County, rainfall is the only type of precipitation considered to cause runoff; significant snowfall is rare, and snowmelt does not contribute to runoff during flood events.

For most analyses and designs in the County, rainfall events of interest are those with 2-, 10-, and 100-year return periods. The standard storm duration for rainfall simulation is 24 hours. Figure D-1 in Appendix D provides the adopted 24-hour incremental rainfall distribution pattern for Santa Clara County. This pattern is based upon the three-day December 1955 rainfall event, still considered to be the storm of record for northern California.

The precipitation pattern has been adjusted to preserve the local rainfall statistics in Santa Clara County based on the District's TDS equations, which are consistent with the IDF curves provided in Appendix B. Thus, the incremental precipitation pattern provided in Figure D-1 is balanced so that the 24-hour storm distribution may be used even where shorter duration storms are more critical. Table D-1 provides tabulated values of precipitation as a percentage of the total 24-hour depth.

The rainfall pattern (hyetograph) can be obtained for any watershed by multiplying the incremental rainfall distribution percentages by the 24-hour rainfall depth (calculated using the methodology provided in Chapter 3).

4.5 Synthetic Unit Hydrographs

A unit hydrograph is a numerical representation of the time response of catchment runoff caused by one inch of excess rainfall applied uniformly over a unit of time. Several different techniques are available to estimate unit hydrographs for rainfall-runoff calculations. The Soil Conservation Service (SCS, now the Natural Resources Conservation Service) synthetic unit hydrograph is used in this manual. This methodology requires only an estimate of basin lag, which is the time from the beginning of excess rainfall (i.e., direct runoff) to the point in time when fifty percent of the runoff has passed the catch point.

4.6 Watershed Parameters

The following watershed parameters are used in the unit hydrograph method, and are generally required input into the computer models listed in Section 4.2.

4.6.1 Basin Area

The basin area is defined as the area enclosed by the watershed draining to the basin outlet point where the discharge is to be determined. Area should be expressed in the units required for analysis. For HEC-1, the basin area is in square miles.

4.6.2 Precipitation

Precipitation is the total depth of rainfall over the basin before any losses due to interception, infiltration, and surface storage. A value for 24-hour precipitation depth shall be obtained at each basin centroid following the procedures outlined in Chapter 3. For HEC-1, basin precipitation is input as inches.

4.6.3 Initial Abstraction

The initial abstraction represents rainfall that is absorbed by tree cover, depressions, and soil at the beginning of a storm. No runoff is calculated until the initial abstraction has been satisfied. The initial abstraction for pervious areas is set equal to 0.2S, where S = (1000/CN) - 10. For impervious or rural areas the initial abstraction is set equal to 0.05 inches.

4.6.4 SCS Curve Number

Direct runoff is estimated by subtracting soil infiltration and other losses from the rate of rainfall. The Curve Number (CN) method is an empirical methodology derived by the Soil Conservation Service (SCS). The Curve Number reflects the potential loss for a given soil and cover complex. After satisfying the initial abstraction defined above, the soil becomes saturated at a certain rate so that a higher percentage of the accumulated rainfall becomes converted into runoff.

Estimates of the CN are made based on the soil types and cover within a drainage basin. The number varies from 0 to 100, and represents the relative runoff potential for a given soil-cover complex for given antecedent moisture conditions (that is, how wet it is prior to any precipitation event).

Curve numbers for watershed modeling may be based on Table E-1 in Appendix E, which correspond to Hydrologic Soil Groups established on the maps prepared by the

SCS for Santa Clara County.[4, 5] Curve numbers for Hydrologic Soil Group A, which is rarely found in the County outside of river beds, is not included in the table. Soil cover and land use may be obtained from aerial photographs, USGS quadrangle maps, general plans, geographic information systems, and field reconnaissance surveys.

Antecedent moisture conditions (AMCs) represent prior soil saturation, depression storage conditions, and other hydrologic precursors prior to the initiation of design rainfall. The different categories of AMCs are characterized by the SCS as follows:

AMC I	dry soils
AMC II	average conditions
AMC III	heavy rainfall, light rainfall with low temperatures, or saturated soil

Antecedent moisture conditions have been established for various return periods for use with the specific rainfall distribution pattern given in Appendix D. Each AMC has been calibrated to individual flood frequency analyses of annual stream discharge data for the San Francisquito Creek, Upper Penitencia Creek, and Bodfish Creek basins. Each gage is located in an area with very little urbanization, so the calibrated AMCs are consistent with the unit hydrograph application presented here. Flood frequency analyses have been made following procedures outlined in USGS Bulletin #17B.[6]

Table 4-1 provides AMC values to be used for various return periods of interest. Engineers may modify the AMC as long as sufficient justification is provided.

Table 4-1: Antecedent Moisture Conditions for Simulation

Design Return Period	AMC
2-year (50 percent)	II¼
10-year (10 percent)	II½
100-year (1 percent)	II½

Antecedent moisture conditions given in Table 4-1 are calibrated ***specifically*** for the rainfall distribution provided in Appendix D. AMC values must be recalibrated to use any other storm distribution.

Table E-1 in Appendix E shows the Curve Numbers for different land use and soil types for AMC II. Using Table E-2, these Curve Number values can be translated to the appropriate AMC, using linear interpolation as necessary.

4.6.5 Percent Imperviousness

This is the percentage of watershed covered by an impermeable surface such as roofs, roads, sidewalks, driveways, and other hardscape. It is input into the hydrologic model as a percentage. In the absence of sufficient information to directly calculate the percentage of impervious area, ranges of typical percent imperviousness for various land use categories are provided in Table E-3.

4.6.6 Basin Lag

The SCS lag equation (a modified version of the U. S. Army Corps of Engineers basin lag equation) is given below:

$$t_{lag} = (0.862)\ 24\,N\left(\frac{L\,L_c}{\sqrt{S}}\right)^{0.38} - \frac{D}{2} \qquad (4\text{-}1)$$

Where	t_{lag}	=	SCS basin lag (hours)
	N	=	watershed roughness value (dimensionless)
	L	=	longest flow path from catchment divide to outlet (miles)
	L_c	=	length along flow path from a point perpendicular with the basin centroid to its outlet (miles)
	S	=	effective slope along main watercourse from Chapter 3 (feet/mile)
	D	=	duration of unit hydrograph (hours)

If possible, the unit hydrograph duration, D, should be selected to lie between one-fifth and one-third of the smallest basin's lag time.

Basin length and slope values may be obtained from available mapping, and are considered to be discrete, measurable parameters. Watershed roughness is the lone subjective parameter to be chosen using engineering judgment. The parameter used in Equation 4-1 should not be confused with Manning's "n". Table 4-2 provides USACE recommended "N" values based on the level of urbanization within a watershed basin.

Table 4-2: Basin Lag Urbanization Parameters

Basin Condition	N
Natural channels, little or no urbanization	0.080
Urban area with natural channels	0.050
Concrete-lined channels with ~2/3 basin urbanized	0.035
Full basin urbanization with storm drain systems	0.025

For basins with a level of urbanization that falls somewhere between the values given above, engineers may interpolate using their judgment.

4.7 Watershed Analysis

Watersheds with relatively uniform ground cover, soil type, and degree of urbanization may be analyzed as a single basin with weighted values for Curve Number and percent impervious. Watersheds with significant variations in these parameters should be broken into smaller basins based on the geographic variation in parameters, if possible, to produce separate hydrographs, which can then be arithmetically combined to produce a design hydrograph at the concentration point.

Basin separation particularly applies when there is a significant portion of soil from either Hydrologic Soil Groups A or B, and soil from either Groups C or D. Non-linearities in the SCS Curve Number methodology can introduce errors when the Curve Number estimate is weighted by area when significant variance in soil type is present. Even if other hydrologic parameters are homogeneous throughout the watershed, the catchment should be split into at least two basins (A/B soils and C/D soils) and recombined for the final design hydrograph. If the soil types are widely dispersed within the watershed, the basin may be split into areas corresponding to the percentages of each soil type, but all with the same precipitation, basin lag, and percent imperviousness.

4.8 Base Flow

Base flow from groundwater infiltration and other sources is generally not considered to significantly impact estimates of peak flow, particularly for smaller watersheds. A base flow component was not used to calibrate antecedent moisture conditions. However, if experience or data show that a considerable base flow exists due to high water table,

reservoir releases, or other circumstances, it shall be considered as an addition to the design flood hydrograph. Base flow is to be used when sizing and designing stormwater detention or retention facilities as described in Chapter 6.

4.9 Channel Routing

Once design hydrographs at concentration points are calculated using the unit hydrograph method, those hydrographs are routed to simulate the movement of a flood wave through natural or man-made channel reaches of varying shapes, sizes, and materials. Routing techniques and parameters shall be determined by the engineer, and justification of each technique shall be provided to the County for review. Common routing techniques and their typical uses are:

4.9.1 Muskingum Routing

This method is commonly used in practice to route flood hydrographs through reaches where a channel and its adjoining overbank areas provide the conveyance and storage of flood waters. Routing parameters include a storage constant (K) and a parameter, "x," which represents the relative effects of wedge and prism storage in the reach. An "x" value of zero represents a true reservoir, and could be used if there is a significant amount of spill out of a low flow channel with substantial valley storage. Most channels have "x" values between 0.0 and 0.2, the latter representing cases where the flood wave is expected to remain in the channel with no overbanking and valley storage to attenuate the hydrograph. The storage constant, K, is approximately equal to the travel time through the reach and may be determined from average flow velocity.

To ensure the numerical stability of Muskingum Routing, each routing reach is broken into multiple routing segments so that the following relationship between K and x is maintained:

$$\frac{1}{2\left(1-x\right)} \leq \frac{K}{\Delta t} \leq \frac{1}{2\,x} \qquad (4\text{-}2)$$

4.9.2 Modified Puls Routing

While the Modified Puls procedure is commonly used for reservoir routing, it may also be used in channel routing. This routing method requires the input of physically meaningful and verifiable data to perform the hydrograph transformation by

establishing storage-discharge relationships. These relationships may be established using normal depth methods or detailed backwater computations. Variables include channel geometry and roughness. If possible, routing lengths should be established so that the time of travel through a reach is more than four or five time steps on the inflow hydrograph.

4.9.3 Kinematic Wave Routing

This routing method is used almost exclusively for urban prismatic channels, and computer programs employing this method generally limit its application to predetermined channel shapes. Hydrographs routed by the kinematic wave procedure are translated downstream, but there is no attenuation.

4.9.4 Muskingum-Cunge Routing

The Muskingum-Cunge routing method is a non-linear routing technique that accounts for diffusion in the equations of motion and uses the Muskingum X-value to assess the effects of storage in the reach. The procedure estimates the X-value based on physical, rather than empirical, characteristics of the channel cross section, namely the reach length, slope, kinematic wave celerity, and the characteristic unit discharge at each time and stage increment. In practice it is believed to be a better routing technique than the kinematic wave procedure as it accounts for storage and not only translates the hydrograph but attenuates the hydrograph depending upon the computed X-values as a function of time. The major disadvantage of the Muskingum-Cunge method is that it assumes normal depth in the channel cross sections and won't account for backwater effects that produce M-1 or S-1 curves

Each of these flood routing methods is described in the HEC-1 User's Manual. In general, these methods are based on the continuity equation and some relationship between flow and storage or stage. In all of the methods, routing proceeds on an independent-reach basis from upstream to downstream without the consideration of backwater effects or discontinuities in the water surfaces. The Kinematic Wave method results primarily in the translation of the runoff hydrograph, with little to no attenuation, therefore it is only appropriate for short reaches of prismatic, concrete-lined channels.

4.10 Storage Routing

Storage routing is used to simulate the movement of a flood wave through a storage reservoir. By providing HEC-1 or HEC-HMS with information about storage (either by specifying volumes or an elevation-surface area relationship) and outflow, it models the inflow, storage, and outflow in the storage reservoir. In HEC-1, outflow information can be supplied in two ways, either by specifying actual discharges or by giving orifice and weir dimensions. In HEC-HMS, outflow information can be supplied only by the former method.

Chapter End Notes

[1]Chow, Ven Te, 1964, *Handbook of Applied Hydrology: A Compendium of Water Resources Technology* New York: McGraw-Hill.

[2]Linsley, Ray K., Max Adam Kohler, and Joseph L. H. Paulhus, 1982; *Hydrology for Engineers*. New York: McGraw-Hill.

[3]American Society of Civil Engineers. Task Committee on Hydrology Handbook of Management Group D. 1996. *Hydrology Handbook.* New York.

[4]Soil Conservation Service. 1968. *Soils of Santa Clara County*.

[5]Soil Conservation Service. 1974. *Soil Survey of Eastern Santa Clara Area, California*.

[6]U. S. Geological Survey. Interagency Committee on Water Data. 1982. *Bulletin #17B of the Hydrology Subcommittee: Guidelines for Determining Flood Flow Frequency*.

5. HYDRAULIC ANALYSIS AND DESIGN

Storm drainage and flood protection systems must be sized so that design flows calculated using one of the methodologies outlined in Chapters 3 and 4 can be collected, conveyed, and safely discharged to receiving waters while meeting the requirements discussed in Chapter 2. This chapter presents methodologies for analyzing hydraulic gradients to determine whether a particular system can meet its design requirements. Both closed conduit and open channel systems are discussed.

5.1 Closed Conduits

Conveyance systems that carry storm runoff such as pipes and culverts, regardless of shape, are closed conduit systems. These systems may carry storm water under pressure (surcharged), or with a free water surface profile exposed to the atmosphere. With a free water surface, the flow is considered to be "open channel," but the methods for calculating hydraulic gradients are described in this section.

5.1.1 Flow Regimes

Flows can be carried within closed conduits and channels either as "subcritical" flow or "supercritical" flow, often referred to as "tranquil" and "rapid" flow, respectively. It is not the velocity of flow, however, that distinguishes the flow regime; rather, the flow regime is defined by how fast the water is moving relative to the velocity of the wave that results from a small disturbance in the water surface. Disturbances in subcritical flow move upstream; disturbances in supercritical flow cannot move upstream because such waves must be swept downstream. The Froude number (F_r), which is analogous to the Mach number for gas flow, is defined as the ratio of conduit or channel velocity to wave velocity:

$$F_r = \frac{v}{\sqrt{gy}} \qquad (5\text{-}1)$$

Where: v = average flow velocity (feet per second)
g = gravitational acceleration (feet per second per second)
y = flow depth (feet)

A Froude number greater than unity signifies supercritical flow (flow velocity greater than wave velocity), while a Froude number less than one indicates subcritical flow. When the Froude number is between 0.8 and 1.2, however, the flow can be unstable, characterized by standing waves and other disturbances that may tend to propagate upstream or downstream depending upon the state of flow.

Since Froude numbers depend on conduit and channel roughness, and "n" values are expected to vary within any given reach, hydraulic gradients should be analyzed for a range of "n" as described in this chapter to protect against unexpected changes in the flow regime which could compromise the design. The high end of the "n" value range should be used to establish required freeboard, and the lower end of conduit and channel roughness are used to examine erosion potential and hydraulic stability with possibly mixed or supercritical flow conditions.

5.1.2 Hydraulic Grade Line

In non-surcharged pipe flow, the hydraulic grade line (HGL) represents the water surface profile. For surcharged pressure flow, the HGL represents a profile of the piezometric water surface; that is, the level to which water would rise along a conduit if it were allowed to do so. For subcritical flow conditions described in Section 5.1.1, the HGL is calculated as a backwater profile beginning at the most downstream point of the storm drain or outfall. For supercritical flow conditions, the HGL is calculated as a forewater profile beginning at the most upstream point of the storm drain system or inlet.

The water surface elevation at the storm drain outfall (subcritical flow) or inlet to the storm drain system (supercritical flow) must be known or calculated prior to an analysis of the conduit system since it acts as the starting elevation of the HGL. The HGL at the first junction in the pipe system can then be calculated by:

5.1.2.1 *Subcritical Flow*

$$HGL_{J1} = WSEL_{outfall} + H_f + H_L \qquad (5\text{-}2)$$

Where:
- HGL_{J1} = hydraulic grade line at Junction 1
- $WSEL_{outfall}$ = water surface elevation at system outfall
- H_f = friction loss in pipe from outfall to Junction 1
- H_L = minor losses in pipeline from outfall to Junction 1

5.1.2.2 *Supercritical Flow*

$$HGL_{J1} = WSEL_{inlet} - H_f - H_L \quad (5\text{-}3)$$

Where: $WSEL_{inlet}$ = water surface elevation at inlet
H_f = friction loss in pipe from inlet to Junction 1
H_L = minor losses in pipeline from inlet to Junction 1

The hydraulic analysis is continued in the upstream direction for subcritical flow and the downstream direction for supercritical flow until the HGL at each junction (manhole or catch basin) is calculated. Based on the requirements discussed in Chapter 2, if the 10-year HGL is contained in existing pipe with adequate freeboard and the 100-year HGL is at or below the top of curb elevation, the design is generally acceptable to the County. If either or both of these conditions are not met, the most common ways to lower the HGL are to lower the pipe inverts or to increase the pipe size or slope for one or more runs of the storm drain system. Pipes may carry flow in a non-surcharged condition at less than full depth.

5.1.3 Conduit Losses

As shown in Equations 5-2 and 5-3, estimating energy losses due to conduit friction and transitions is a key element of hydraulic grade line computation.

5.1.3.1 *Friction Losses*

Head loss due to friction is a result of the kinetic energy lost as the flow travels through the conduit. The rougher the pipe is, the greater the head loss is going to be. Head loss due to friction can be calculated using Manning's Equation:

$$V = \frac{1.49}{n} R^{2/3} S_f^{1/2} \quad \text{or} \quad Q = \frac{1.49}{n} A R^{2/3} S_f^{1/2} \quad (5\text{-}4)$$

Where: V = mean velocity (feet per second)
Q = flow rate (cubic feet per second)
A = cross sectional area (square feet)
S_f = friction slope (feet per foot)
n = coefficient of friction (dimensionless, Table F-1)
R = hydraulic radius (feet) = A/P
P = wetted perimeter (feet)

The wetted perimeter, P, represents the perimeter of the conduit (or channel) where a fixed boundary is in contact with the flow. Thus a free water surface is not included in the wetted perimeter.

Manning's Equation is an open channel flow equation used to find either the depth of flow or the velocity in an open channel or closed conduit flowing partially full where the channel roughness, slope, depth, and shape remain constant (steady, uniform flow). The depth of flow using Manning's Equation is referred to as the "normal depth" and the velocity is referred to as the "normal velocity." Table F-2 in Appendix F can be used to estimate cross-sectional area (A) and wetted perimeter (P) for a variety of channel shapes.

5.1.3.2 *Minor Losses*

In addition to friction losses along a channel or conduit, there are also local losses associated with sudden changes due to transitions, entrances, manholes, junctions, and bends. Minor losses are expressed as a loss coefficient, K, times either the velocity head or the difference in velocity head, depending on the type of loss. Minor losses are generally expressed by the following equation:

$$H_L = \frac{KV^2}{2g} \tag{5-5}$$

Where: H_L = minor head loss
K = dimensionless loss coefficient
V = average flow velocity
g = gravitational acceleration

In long reaches, where the length-to-diameter ratio is much greater than 1,000, minor losses are usually very small compared to friction losses and can be neglected. However, if the reach is very short and there are several minor losses, the sum of these losses can easily exceed the losses due to friction. The following are typical minor losses than can occur in a pipe system and methods to determine their associated head losses.

Transition Losses. Transition losses occur where the conduit or channel changes size and/or shape. The corresponding change in cross-sectional area results in a change in velocity and a loss of energy, or "head". For open channel flow conditions, including non-pressure flow pipes, the energy losses due to contraction and expansion are given by the following equations:

Contractions:

$$H_c = K_c\left(\frac{V_2^2}{2g} - \frac{V_1^2}{2g}\right) \text{for } V_2 > V_1 \quad (5\text{-}6)$$

Expansions:

$$H_e = K_e\left(\frac{V_1^2}{2g} - \frac{V_2^2}{2g}\right) \text{for } V_1 > V_2 \quad (5\text{-}7)$$

Where:

H_c	=	head loss due to contraction
H_e	=	head loss due to expansion
K_c	=	contraction loss coefficient
K_e	=	expansion loss coefficient
V_1	=	velocity upstream of the transition
V_2	=	velocity downstream of the transition

Typical values for K_c and K_e are given in Tables F-3 through F-6 in Appendix F.

Entrance Losses. Energy losses will occur at the entrance to box culverts and pipes. These entrance losses can be estimated using Equation 5-5 and the entrance loss coefficients given in Table F-7 in Appendix F.

Manhole Losses. In a straight-through manhole where there is no change in pipe size, the minor loss can be estimated by:

$$H_m = 0.05\left(\frac{V^2}{2g}\right) \quad (5\text{-}8)$$

Simple transitions in pipe size in a straight-through manhole may be analyzed by treating the transition as an expansion or contraction as previously described.

Junction Losses. Junction losses are due to either changes in the direction of flow or multiple flows entering a manhole or catch basin. When flow changes direction inside a junction, as shown in Figure 5-1, there is an associated head loss. The amount of head loss that occurs is dependent on how great the angle of deflection is. As the deflection angle between the inflow and outflow pipes is increased, the amount of head loss increases, therefore, junctions should be designed to allow the flow to come together smoothly to minimize head losses. Recommended design considerations include

minimizing the deflection angle (< 60°), minimizing the vertical difference between the two inverts (< 6 inches), and providing a semicircular channel or bench in the junction manhole.

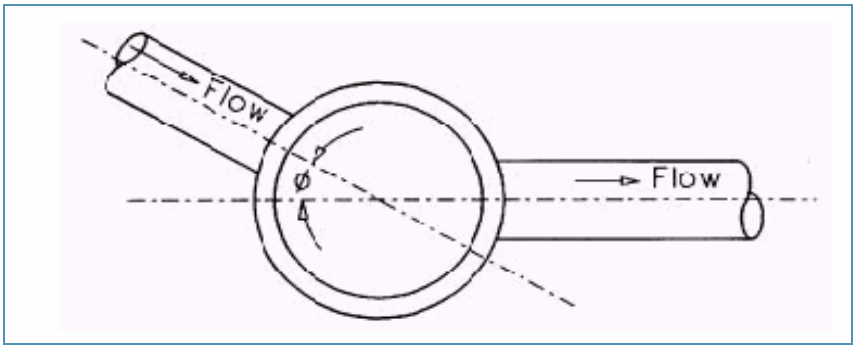


Figure 5-1: Changes in Direction of Flow at a Junction

Head losses due to changes in direction of flow can be calculated using Equation 5-5 and the loss coefficients, K_b, shown in Table 5-1.

Table 5-1: Loss Coefficients for Change in Flow Direction

Flow Change in Degrees	K_b
0	0.00
15	0.19
30	0.35
45	0.47
60	0.56
75	0.64
90 and greater	0.70

Head losses also occur when flow enters a junction from more than one pipe as shown in Figure 5-2. The head losses are dependent on the flow in each pipe and the direction that each pipe enters the junction and can be estimated with the following equation:

$$H_{mf} = \left(\frac{Q_2 V_2^2 - Q_3 V_3^2 - Q_1 V_1^2 + K_b Q_3 V_3^2}{2 g Q_2} \right) \quad (5\text{-}9)$$

Where: H_{mf} = head loss from multiple flows

K_b = head loss coefficient for change in direction (Table 5-1)

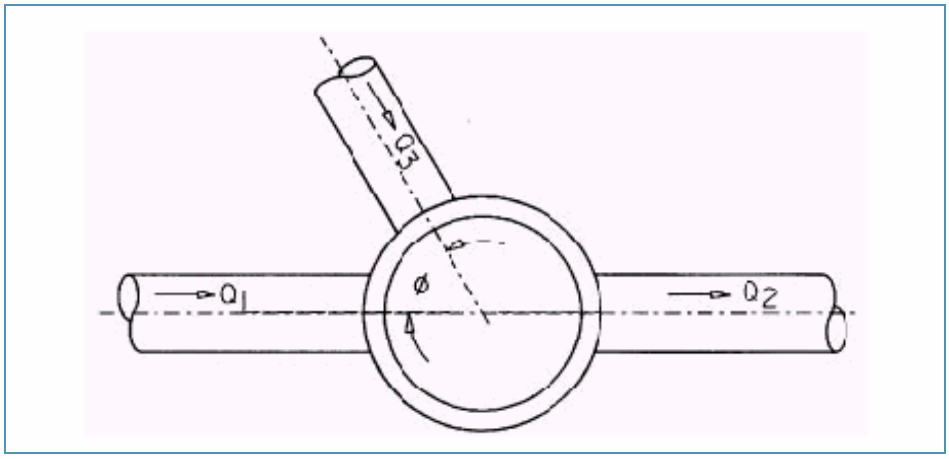


Figure 5-2: Multiple Flows Entering a Junction

Bend Losses. Bend losses in open channels are a function of the ratio of the radius of the bend, r, to the width of the channel, b. Bend losses in open channels can be estimated using Equation 5-5 and the bend loss coefficients given in Table 5-2. If r/b is equal to or greater than 3.0, the bend loss is negligible.

Table 5-2: Bend Loss Coefficients in Open Channels

r/b	K_b
2.5	0.02
2.0	0.07
1.5	0.12
1.0	0.25

Bend losses in closed conduits are a function of the deflection angle between the upstream and downstream pipes. For curved segments where the angle is less than 40° the bend loss coefficient may be estimated as:

$$K_b = 0.25\left(\frac{\Delta}{90}\right)^{0.5} \qquad (5\text{-}10)$$

Where: K_b = bend loss coefficient (dimensionless)
Δ = deflection angle (degrees)

For greater angles of deflection and bends in manholes the bend loss can be estimated by using Equation 5-5 and the bend loss coefficients shown on Figure F-8 in Appendix F.

5.1.4 Pipe Standards

Storm drain systems built under Santa Clara County Office of Development Services jurisdiction shall adhere to the following standards.

5.1.4.1 *Acceptable Pipe Sizes*

Pipe sizes shall be limited to the following standard pipe diameters for all storm drainage systems:

- 8-inch (privately maintained systems or special cases as approved by the County)
- 12-inch
- 15-inch
- 18-inch
- 21-inch
- 24-inch
- 30-inch
- A multiple of 6 inches (for pipes larger than 30 inches in diameter)

5.1.4.2 *Allowable Pipe Materials*

An engineer's evaluation and selection of materials for storm drain pipelines shall consider the following: intended use, scour or abrasion conditions, installation requirements, corrosion conditions, flow requirements, product characteristics, cost effectiveness, physical properties, and handling requirements. Given these considerations, the following pipe materials are allowed for all storm drainage systems to be maintained by Santa Clara County:

1. Plain concrete pipe (12-inch diameter driveway culvert only)
2. Reinforced concrete pipe (RCP)
3. Corrugated metal pipe (CMP)
4. Aluminum spiral rib (SRP)
5. Aluminized Type 2 corrugated steel (AASHTO M274 and M56)
6. Galvanized[1] corrugated iron or steel pipe, Treatments 1 through 6

7. Galvanized[1] steel spiral rib pipe, Treatments 1 through 6
8. Ductile iron (water supply, Class 50 or 52)
9. Lined corrugated polyethylene pipe (LCPE)[2]
10. Corrugated polyethylene pipe (CPE)[3]
11. Polyvinyl chloride (PVC)[4] sewer pipe
12. Solid wall polyethylene pipe (SWPE; also known as HDPE or HDPP)[5]

5.1.4.3 *Allowable Pipe Joints*

The use of reliable, tight pipe joints shall be used in the design of all drainage systems. The joints must be watertight, flexible, and durable. Given these considerations, the following criteria shall be met for all storm drainage systems to be maintained by Santa Clara County:

1. Concrete pipe shall be rubber-gasketed.
2. Corrugated Metal Pipe shall be rubber-gasketed and securely banded.
3. Spiral rib pipe shall be "hat-banded" with neoprene gaskets.
4. Ductile pipe joints shall be flanged, bell and spigot, or restrained mechanical joints.
5. LCPE pipe shall be joined by split corrugated couplings, with gaskets, which are at least 4 corrugations wide and exceed the soil tightness requirements of the *AASHTO Standard Specifications for Highway Bridges*, Section 23 (2.23.3).
6. CPE single wall, fully corrugated pipe shall be joined by split or snap-on couplings for 3- through 10-inch diameter pipe, and by split corrugated couplings with gasket for 12- through 24-inch diameter pipe. Couplings for 12- through 24-inch diameter pipe shall be at least 7 corrugations wide and shall exceed the soil tightness requirements of the *AASHTO Standard Specifications for Highway Bridges*, Section 23 (2.23.3).
7. PVC pipe shall be installed following the procedures outlined in ASTM D2321; joints shall conform to ASTM D3212, and gaskets shall conform to ASTM F477.
8. SWPE pipe shall be jointed by butt fusion methods or flanged.

5.1.4.4 *Pipe Alignment*

The following pipe alignment criteria shall be met for all storm drainage systems to be maintained by Santa Clara County.

1. Pipes shall be laid true to line and grade with no curves, bends, or deflections in any direction. An exception shall be allowed for vertical deflection in SWPE and ductile iron pipe with flanged restrained mechanical joint bends (< 30º) on steep slopes, provided the pipe drains.
2. A break in grade or alignment, or changes in pipe material shall only occur at catch basins or manholes.

5.1.4.5 *Changes in Pipe Size*

The following criteria shall be followed when changing pipe sizes for all storm drainage systems to be maintained by Santa Clara County:

1. Increases or decreases in pipe sizes shall be allowed only at junctions and structures. Pipe size shall not decrease in area or diameter in the downstream direction.
2. When connecting pipes at structures, the following shall be matched in descending order of preference: crowns, 80% diameters[6], or pipe inverts. Lateral pipes 12 inches in diameter and smaller are exempt from this requirement.
3. Downsizing of pipes larger than 12 inches in diameter may be allowed, provided pipe capacity is adequate for the design flows.

5.1.4.6 *Permissible Pipe Slopes and Velocities*

The following minimum and maximum pipe slopes and velocities shall be met for all storm drainage systems to be maintained by Santa Clara County:

1. A minimum velocity of 2.6 feet per second for the two-year return period shall be maintained as a self-cleaning velocity. Slopes required to maintain velocities of 2 and 3 feet per second for full and half-full flow conditions are given in Table F-9 in Appendix F. If site constraints result in velocities less than 2.6 feet per second, sedimentation impacts shall be addressed by the following: increased pipe sizes, closer spacing of structures, or sediment/debris basins. Santa Clara County may consider other sedimentation prevention measures when minimum storm drain velocities cannot be met.
2. Maximum velocities of 30 feet per second shall be allowed for CMP, Spiral Rib, PVC, and CPE[7] pipe at a maximum slope of 30 percent. Pipe anchors shall be required at slopes greater than 20 percent and shall be spaced at 1 anchor per 100 LF of pipe. Maximum velocities of 30 feet per second shall be allowed for Concrete or LCPE[7] pipe at a maximum slope of 20 percent. Pipe anchors shall be required at slopes greater than 10 percent and shall be spaced at least one anchor per 50 lineal feet of pipe. No maximum velocities or slopes shall be set for

ductile iron or SWPE pipe; however, butt-fused or flanged joints are required and above-ground installation is recommended on slopes greater than 40 percent. Pipe anchors shall be required for ductile iron pipe at slopes greater than 20 percent with at least one anchor for every pipe section. Pipe anchors shall be required for SWPE pipe at slopes greater than 20 percent with at least one anchor for every 100 lineal feet of pipe on cross-slope installations only.

5.1.4.7 *Pipe Cover*

Pipe cover, measured from the street sub-grade elevation to the top of the outside surface of the pipe, shall be 2.5 feet minimum. Under drainage easements, driveways, parking stalls, or other areas subject to light vehicular loading, pipe cover may be reduced with County approval provided the design considers the expected vehicular loading and is consistent with the pipe manufacturer's recommendations. Pipe cover in areas not subject to vehicular load may be reduced with County approval. The maximum pipe cover shall be based on the manufacturer's recommendations but shall never exceed 30 feet as measured from pipe invert to street sub-grade elevation.

5.1.4.8 *Pipe Clearance*

A minimum of one foot vertical and five feet horizontal clearance (outside surfaces) shall be provided between storm drain lines and other utility pipes and conduits. The minimum criteria for pipe clearance shall take into consideration the criteria of other local agencies and/or public utilities responsible for the utility pipes and conduits being crossed.

5.1.4.9 *Pipe Anchors*

All County-maintained CMP, Spiral Rib, PVC, and CPE storm drainage systems placed at slopes greater than twenty (20) percent shall be anchored as shown on Figures G-1 and G-2 in Appendix G. The minimum anchor spacing shall be one anchor per 100 lineal feet of pipe, or as required through a detailed load analysis. For concrete or LCPE pipe, anchors shall be required for slopes greater than ten (10) percent and shall be installed at least once for every 50 lineal feet. For anchoring specifications please also refer to Section 5.1.4.6, Criterion #2.

5.1.5 Culverts

A culvert is defined as any conduit designed to convey water through a roadway, railroad, canal, or other embankment without overtopping. Culverts are relatively short in length and may be circular, elliptical, square, rectangular, or arched in cross section.

They are usually mounted in a headwall that provides an improved entrance and may contain multiple barrels.

The hydraulic design of culverts is primarily influenced by available headwater depth. If the headwater depth is not adequate to pass the flow through the culvert, the embankment may overtop or the flow may back up and cause upstream flooding. Other factors that influence discharge through a culvert are pipe size, pipe length, pipe roughness, pipe slope, inlet geometry, and tailwater conditions.

5.1.5.1 *Hydraulic Control*

Culvert flows are classified as either under **inlet control** or **outlet control**. Under inlet control conditions, the culvert inlet size, shape, and condition restricts the amount of water passing into the culvert. Major factors influencing the discharge of a culvert flowing under inlet control are: cross sectional area of the culvert barrel, inlet geometry, and headwater depth. A modified form of the orifice equation gives culvert capacity for a culvert governed by inlet control.

Under outlet control conditions, the culvert capacity is determined by a combination of tailwater depth, barrel conditions (pipe length, slope, and roughness), and inlet geometry. The energy equation and Manning's equation are used to calculate the capacity of a culvert flowing under outlet control.

Inlet Control. Nomographs developed by the Bureau of Roads, now the Federal Highway Administration, can be used to determine the inlet control headwater depth at design flow for various types of culverts and inlet configurations. Inlet control nomographs for concrete pipe, corrugated metal pipe, and box culverts are shown on Figures H-1 through H-3 in Appendix H. Additional nomographs can be found in the FHWA publication *Hydraulic Design of Highway Culverts*, HDS No. 5 (Report No. FHWA-IP-85-15), September 1985.

Outlet Control. Nomographs have also been developed for determining outlet control headwater depths. Outlet control nomographs for concrete pipe, corrugated metal pipe, and box culverts are shown on Figures H-4 through H-6 in Appendix H. Additional nomographs can be found in FHWA, *Hydraulic Design of Highway Culverts*, previously referenced. Outlet control headwater depths for pipe systems can also be determined using a simple backwater analysis given by the following equation:

$$HW = H + TW - LS \qquad (5\text{-}11)$$

Where: HW = headwater depth, ft

H = $H_f + H_e + H_{ex}$

H_f = friction loss, ft = $(V^2n^2L)/(2.22R^{1.33})$

Note[8]: If $(H_f+TW-LS) < D$, adjust H_f such that $(H_f+TW-LS) = D$

H_e = entrance head loss, ft = $K_e (V^2/2g)$

H_{ex} = exit head loss, ft = $V^2/2g$

TW = tailwater depth above invert of culvert outlet, ft

Note[8]: If $TW < (D+d_c)/2$, set $TW = (D+d_c)/2$

L = culvert length, ft

S = culvert slope, ft/ft

D = culvert barrel diameter, ft

V = velocity, ft/s

n = Manning's coefficient, dimensionless

R = hydraulic radius, ft

K_e = entrance loss coefficient, dimensionless (Table F-7)

G = acceleration due to gravity, 32.2 ft/s^2

d_c = critical depth, ft (see Figures H-7 and H-8 in Appendix H)

5.1.5.2 *Minimum Design Criteria for Culverts*

Culverts shall be sized to pass the 25-year design flow under free outfall conditions, without an inlet head in excess of the top of culvert (that is, with H/D no greater than unity). After the culvert is sized according to this criterion, culvert sizing shall be checked under all inlet and outlet control conditions to safely pass the 100-year design flow and meet the requirements of Chapter 2.

5.1.6 Appurtenant Structures

The design of appurtenant structures for closed conduit drainage systems shall adhere to the following requirements.

5.1.6.1 *Inlets*

Storm drain inlets shall adhere to the current version of the Santa Clara County Roads and Airports Department's "Standard Details Manual." The applicable Standard Details may include, but not be limited to C1, C2, C3, C5, and C11.

5.1.6.2 *Manholes*

Storm drain manholes shall adhere to the current version of the Santa Clara County Roads and Airports Department's "Standard Details Manual." The applicable Standard Details may include, but not be limited to, C6, C7, C8, and C9.

5.1.6.3 *Outfalls*

Storm drain outfalls to natural channels, concrete channels, concrete box culverts, and other storm water conveyance pipes shall adhere to the current Santa Clara Valley Water District Standard Details or to ABAG Standard Details.

5.1.6.4 *Headwalls*

The headwalls for storm drainage passage underneath roadways shall adhere to the current version of Caltrans Standard Plans. The applicable Standard Details may include, but not be limited to, D86B, D86C, D89, and D90.

5.2 Open Channels

The detailed design of major open channel flood protection facilities is beyond the scope of this drainage manual, but drainage features such as open ditches and swales, and small channels shall be designed according to this section.

5.2.1 Flow Regimes

Flood flows can be carried within channels either as “subcritical” flow or “supercritical” flow, as defined in Section 5.1.1 for closed conduits. In subcritical flow, a disturbance caused by a channel feature can influence flow further upstream, even to the point of making previously supercritical flow subcritical. Conversely, supercritical flow cannot be influenced by downstream channel features.

Whether flow is subcritical or supercritical depends on several factors including channel shape, channel slope, channel roughness, and discharge. As illustrated in Figure 5-3, a channel features such as downstream backwater or a bridge can act as a channel control, and change the flow regime upstream of that feature.

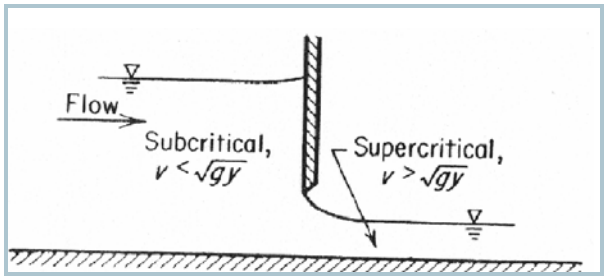


Figure 5-3: Bridges as a Generator of Subcritical Flow (Henderson, 1966)

5.2.2 Analytical Methods

Open channel analyses and designs shall be based upon commonly recognized analytical methods chosen by the engineer of record, and submitted with justification to the County for review and approval.

Depending upon project conditions, steady-state (constant flow) or unsteady (discharge varies with time) methodologies may need to be employed. Section 5.4 provides a variety of computer simulation programs for analyzing open channel flow.

Simple steady-state problems with uniform flow may be analyzed using Manning's formula (Equation 5-4) for normal depth and average velocity. Where tailwater conditions or system design are such that there are significant backwater effects, water surface profiles shall be computed using a one-dimensional open channel flow program. Two or three dimensional analytical techniques are beyond the scope of the Drainage Manual.

5.2.3 Channel Roughness

In one-dimensional open channel flow analysis can be performed using HEC-2, HEC-RAS, and other commonly available software platforms. A single parameter known as "Manning's n" is used to represent the retarding forces to flow imposed by the channel. Values for "n" are published in the literature, and in the absence of high water marks with which to calibrate stream reaches with known discharge, are often relied upon for channel design. When selecting roughness values for design, it is important to remember that in one-dimensional flow, Manning's "n" accounts for the flow resistance due to a host of hydraulic phenomena beyond boundary shear. Other factors may include the effects of eddies, cross-waves, super-elevation, bed forms, sediment and debris.

Table F-1 in Appendix F summarizes design channel roughness values of various channel elements proposed for use in hydraulic analysis.

Given the uncertainty in "n" value selection, a range of "n" values is used to examine channel performance. Lower values tend to increase channel velocity, which may be important in terms of scour or wave formation. Higher values tend to maximize water depth, and will generally set freeboard requirements. For vegetal channel linings, typical "n" values for temporary erosion control materials will be used to evaluate high flow velocities.

Manning's "n" values are contingent upon maintenance conditions within natural and artificial channels. Analyses and designs presented to the Office of Development Services for approval shall clearly indicate proposed project maintenance and the basis for selecting long-term design roughness coefficients.

5.2.4 Bridge Hydraulics

In addition to the energy required to overcome channel resistance, structures such as bridges and culverts also cause energy losses, which can result in a raised water surface profile. Methods provided by HEC-RAS for analyzing bridges and culverts under low flow and pressurized conditions should be employed in backwater computations.

Several methods are available through HEC-RAS to compute energy losses through a bridge. The "energy only" or standard step method handles a bridge section without piers in the same manner as a natural river section, except that the area between the low chord of the bridge (soffit) and the top of road is subtracted from the total cross-sectional area, and the wetted perimeter is increased where water is in contact with the bridge. Increased frictional resistance due to the added wetted perimeter is included in the energy loss through the structure.

When bridge piers are present, either conservation of momentum may be applied by using a coefficient of drag, or Yarnell's method may be used for subcritical "Class A" low flow through the bridge. Table 5-3 provides proposed drag coefficients for the momentum method and pier shape coefficients for Yarnell's low flow bridge loss calculations, respectively.

Table 5-3: Bridge Pier Coefficients

Pier Shape	Drag Coefficient	Pier Coefficient
Semicircular Nose and Tail	1.33	0.90
Multiple Cylinders	1.33	1.00
Triangular Nose and Tail	2.00	1.05
Square Nose and Tail	2.00	1.25

When the computed water surface elevation is above the bridge soffit, a "pressure/weir" feature can compute losses through the structure for pressure (orifice) flow, weir flow over the top, or a combination of both. Changes to the water surface profile resulting from the bridge are calculated based on hydraulic formulae that estimate the change in energy and water surface elevation through the bridge. Other hydraulic routines are also available to perform the same function for standard culvert shapes.

Coefficients of discharge for orifice flow during bridge pressurization are evaluated on a case-by-case basis using the guidelines outlined in Table 5-5. When an orifice is located close to the bottom or bank of the channel, the approaching flow is guided so that the orifice contraction is suppressed on those sides of the orifice close to such guides. Clear span bridges across trapezoidal or U-frame (rectangular) channels are examples of orifices contracted on three sides.

Table 5-4: Orifice Coefficients

Condition	Orifice Coefficient
Orifice in Thick Wall (typical bridge)	0.80
Submerged, square-edged	0.80

Condition	Orifice Coefficient
Submerged, well-rounded	0.90
Increase in C per number of contracted sides	0.04

5.2.5 Transition Losses

An energy loss also takes place just upstream and downstream from each structure as flow contracts and expands into and out of the bridge or culvert. The following contraction and expansion coefficients for channel transitions are used in the hydraulic models (Table 5-6). Wherever turbulent conditions create the potential for energy loss, contraction and expansion coefficients are increased. Other transitions include channel bends, resting pools, and maintenance access ramps.

Table 5-5: Expansion and Contraction Coefficients

Transition Type	Contraction	Expansion	Source
Gradual	0.1	0.3	HEC
Warped	0.1	0.2	Chow
Wedge	0.3	0.5	Chow
Square End	0.3	0.75	Chow
Abrupt	0.6	0.80	HEC

5.2.6 Channel Freeboard

As described in Chapter 2, the use of design "freeboard" provides a measure of safety that compensates for the many unknown and difficult-to-quantify parameters that affect the calculation of flood elevations. These factors include uncertainty in rainfall data, soil loss parameters, watershed urbanization, wave action, debris at bridge openings, and general uncertainties in hydrologic and hydraulic procedures. Freeboard is usually expressed in terms of feet above the design base flood elevation. To meet FEMA

standards, freeboard is necessary whenever a levee system, including structural floodwalls, is used to provide flood protection.

When mapping flood-prone areas, FEMA only recognizes those levee systems meeting their criteria, which includes a minimum three feet of freeboard whenever the design one-percent water surface elevation is carried above the natural ground elevation. An additional six inches of freeboard (3.5 feet above the water surface) is required at the upstream end of the levee/floodwall system, tapering to the minimum freeboard of 3.0 feet at the downstream end of the levee. For this reason, the Santa Clara Valley Water District has adopted a uniform freeboard criterion of 3.5 feet. An additional 0.5 foot of freeboard (4.0 feet above the water surface) must be provided within 100 feet of each side of any structure, such as a bridge or culvert.

FEMA does not impose a freeboard requirement when the base flood can be carried without the use of a levee system. The District, however, does have additional project freeboard requirements for design projects (SCVWD, 1994), which can be more restrictive than FEMA's actuarial criteria. Table 5-6 provides the respective criteria, where "D" refers to the estimated depth of flow in feet, and "V" is flow velocity in feet per second.

Table 5-6: Channel Freeboard Requirements

Situation	FEMA (Feet)	District (Feet)
Water Surface Above Natural Bank Elevation	3.0	3.5
Water Surface Above Bank within 100 feet of Structure	4.0	4.0
Water Surface Below Natural Bank Elevation	N/A	$0.2(D + V^2/2g)$
Minimum Freeboard	N/A	1.0

District criteria for water surfaces below natural bank elevation are based on NRCS guidelines. Recently the NRCS has approved the use of a slightly different formula for its freeboard criterion within incised channels; that is, where the design water surface is carried below the natural bank. The velocity head term has been omitted, leaving the required freeboard within an incised channel as the minimum of one foot or 0.2D, where D is flow depth. The NRCS has also approved a District-proposed method of weighting depth based upon flow conveyance to obtain a composite freeboard.

5.2.7 Hydraulic Jumps

Hydraulic jumps occur when the depth of flow changes rapidly from a low stage to a high stage as the flow regime changes from supercritical upstream to subcritical downstream. Where hydraulic jumps are likely to occur, their location and energy losses shall be determined and considered in the design.

5.2.8 Channel Curvature

The centrifugal force caused by flow around a curve raises the water surface on the outside wall of the curve, and depresses the water surface along the inside wall. This phenomenon is referred to as superelevation. In addition, curved channels tend to create secondary helicoidal flows that may persist downstream of the curve. Superelevation is checked with both high and low Manning's "n" values to determine the most critical case for wall freeboard. The maximum amount of superelevation above

the level water surface predicted by HEC-RAS is given by the following equation (ASCE, 1995):

$$\Delta y = C\frac{V^2 W}{gr} \tag{5-12}$$

Where:
- Δy = the rise in water surface (feet)
- C = a dimensionless coefficient (0.5 for subcritical flow in a simple circular curve in a rectangular channel; 1.0 for supercritical flow in the same type of curve)
- V = mean channel velocity (feet per second)
- W = channel width (feet)
- g = gravitational acceleration (feet per second squared)
- r = the radius of channel centerline curvature (feet)

5.2.9 Air Entrainment

At flow velocities in excess of 14 feet per second, air may be entrained, increasing the depth of flow. The increase in depth is related directly to the increase in the volume of water caused by entrainment:

$$A_o = 10\left[\frac{0.2\,V^2}{g\,R} - 1\right]^{0.5} \tag{5-13}$$

Where:
- A_o = increase in flow area attributable to air entrainment (percent)
- V = mean channel velocity (feet per second)
- R = hydraulic radius without air entrainment (feet)
- g = gravitational acceleration (feet per second squared)

5.2.9.1 *Channel Maintenance*

Regular channel maintenance is important to upholding the integrity of channel design. During preliminary and final design phases, proposed maintenance protocols shall be iteratively evaluated for their impact on design channel roughness. For instance, natural elements including grasses and emergent wetlands that are allowed to become overgrown with woody vegetation such as brush, willows, and even trees would be assumed to have higher roughness values in the 0.10 to 0.15 range. Justification regarding maintenance assumptions shall be provided to the County for review.

5.3 Coincident Analyses

Storm sewers most typically discharge into a receiving body of water. Often that body of water will not be studied or designed during the course of project work.

When discharging to another storm drain system, its design 10-year water surface elevation in the creek shall be used as the starting water surface elevation for calculating the HGL in the storm drain system. The design 100-year elevation in a receiving channel shall be used to evaluate storm drain performance and overland release in conformance with Chapter 2. When discharging directly to San Francisco Bay, the 100-year tidal elevation starting water surface elevation for calculating the 10-year HGL in the storm drain system and for discharging 100-year design flows with appropriate freeboard. Two-year return period analyses shall assume discharge to Mean Higher High Water (MHHW) tidal elevations at the nearest reporting station listed in Table I-1 and shown on Figure I-1 in Appendix I. Adopted 100-year tidal elevations shown in Figure I-2 are for South San Francisco Bay based on the U.S. Army Corps of Engineers. Engineers must be aware of datum differences and use these data accordingly. Location and datum conversion factors are shown in Table I-2.

5.4 Computer Programs

Several normal depth calculation computer programs exist for the design and analysis of pipes, ditches, open channels, weirs, orifices and inlets, including Flowmaster and StormCad by Haestad Methods. Most of these programs are capable of solving for or establishing rating relationships for any unknown variable using the Manning's, Hazen-Williams', Kutter's, Darcy-Weisbach, and Colebrook-White Formulas. Any of these computer programs shall be acceptable to the County provided that they are found to produce results consistent with the results obtained performing manual calculations using the equations provided in Chapter 5 of this manual.

The U.S. Army Corps of Engineers' **HEC-2 and HEC-RAS** are standard-step backwater program that are capable of computing water surface profiles for one-dimensional steady, gradually varied flow in rivers of any cross section. Flow may be subcritical or supercritical. Various routines are available for modifying input cross-section data, locating encroachments or inserting a trapezoidal excavation on cross sections. The water surface profile through structures such as bridges, culverts and weirs can be computed. Variable channel roughness and variable reach length between adjacent cross sections can be accommodated. HEC-RAS also has the capability of modeling mixed flow regimes and unsteady flow conditions.

Other hydraulic models are available, including WSPRO from the Federal Highway Administration (FHWA), MOUSE and MIKE-11, available from the Danish Hydraulics Institute (DHI). Any of these hydraulic models shall be acceptable to the County provided they are found to produce results consistent with those obtained performing backwater and/or forewater calculations using methodologies described in the Drainage Manual.

Chapter End Notes:

1. Galvanized metals leach zinc into the environment. High zinc concentrations can be toxic to aquatic life, therefore, where other materials are available, they should be used.
2. LCPE pipe and fittings shall be manufactured from high density polyethylene resin which shall meet or exceed the requirements of Type 111, Category 3, 4, 5, Grade P23, or P34, Class C per ASTM D1248. In addition, the pipe shall comply with all material and stiffness requirements of AASHTO M294.
3. CPE pipe (single wall, fully corrugated) is allowed only for use in temporary storm sewer systems such as downspout, footing or yard drain collectors on private property. Pipe and fittings shall comply with all of the requirements of AASHTO M252 for 3" through 10" diameter, and AASHTO M294 for 12" through 24" diameter.
4. PVC pipe is allowed only for use in privately maintained drainage systems. PVC pipe must be SDR 35 or thicker to meet the requirements of ASTM D3034.
5. SWPE pipe is normally used on steep slope installations. SWPE pipe shall comply with the requirements of Type III C5P34 as tabulated in ASTM D1248, shall have the PPI recommended designation of PE3408, and shall have an ASTM D3350 cell classification of 345534C. The pipe shall have a manufacturer's recommended hydrostatic design stress rating of 800 psi based on a material with a 1600 psi design basis determined in accordance with ASTM D2837-69. The pipe shall have a suggested design working pressure of 50 psi at 73.4° and SDR of 32.5
6. Match point is at 80% of the pipe diameter, measured from the invert of the respective pipes.
7. These materials are not allowed in landslide areas.
8. This will keep the analysis simple and still yield reasonable results that err on the conservative side.

6. STORAGE FACILITIES

Storage basins are designed to reduce the rate at which stormwater discharge must be carried in downstream facilities. Storage may also be required to mitigate increases in peak runoff rates due to watershed changes, or limited downstream facility capacities.

6.1 Detention Facilities

Detention storage is defined as an above or below ground facility, such as a pond or tank that temporarily stores stormwater runoff and subsequently releases into the receiving waterway at a slower rate than it is collected by the detention facility system. There is little or no infiltration of stored stormwater. A typical detention pond is shown in Figure J-1 in Appendix J.

One of the most common uses for detention is to limit the discharge rate from a newly developed or redeveloped site. When such a facility becomes a permanent drainage feature, assurances for the continued maintenance of its capacity must be provided to the County, another public agency, or private party through a maintenance agreement.

6.1.1 Types of Detention Basins

Several general types of detention facilities are acceptable to the County for controlling storm discharge:

- Parking lot detention may be used only for industrial and business development. If parking lot detention is utilized, a notice that the area is subject to stormwater ponding must be filed with all tenants. Parking lots shall provide for pedestrian access through ponded areas. Maximum design depths of ponding shall not exceed four inches (4").
- Conduit storage can be used by oversizing underground drainage facilities. Care shall be taken to prevent siltation problems as a result.
- Channel storage may be provided by oversizing open channel facilities. Again, care should be taken to prevent siltation problems, and allowances must be made for a minimum capacity at maximum silt buildup thresholds for maintenance.
- Multi-purpose facilities such as parks, playing fields, tennis courts, parking areas, existing ponds and wetland areas, and landscaping may also provide a design detention function; as long as the appurtenant features described in the Drainage Manual are provided, and a maintenance agreement is made.

6.1.1.1 *Flow-Through Basins*

Flow-Through (on-line) detention basins utilize the excess storage capacity available in a conveyance system. The excess storage capacity may be naturally occurring or the result of grading activities.

A flow-through system may be designed to pass dry weather flows without detention, however, during a storm event, the volume difference between the inflow and the outflow will be detained in the basin. The inlet of an on-line basin shall utilize some form of energy dissipation to reduce incoming velocities and to disperse sediment-laden water into the basin.

The outlet of an on-line basin shall be controlled by a fixed orifice, adjusted sluice gate, pump(s), and or flow regulator(s).

6.1.1.2 *Flow-By Basins*

Flow-by detention basins, also referred to as off-line detention basins, collect diverted flow from a conveyance system once the system is overloaded. During overflow conditions, the stormwater is diverted into an off-line detention basin that provides temporary storage before releasing the flow back to the conveyance system over an extended period of time.

The off-line basin may be designed to capture the first flush volume for stormwater quality control or to provide large storage volumes for flood control. Typically, off-line basins are placed in open space areas such as parks and/or playfields adjacent to the waterway. Off-line detention is becoming increasingly popular in urban areas due to right-of-way constraints and high land costs.

An off-line basin shall be designed to trigger flow diversion once the flow in the waterway exceeds an established flow rate and/or stage. The diversion shall be designed to keep the allowable release through the waterway while spilling the excess water into the off-line basin. The spills into the off-line basin shall be controlled by either electrical devices such as flow valves and pumps, or gravity-flow devices such as side spillways and/or overflow weirs. The inlet of an off-line shall utilize some form of energy dissipation to reduce incoming velocities and to disperse sediment-laden water into the basin.

6.1.2 Outlet Structures

Outlet structures for detention basins shall be designed not to exceed a predetermined release rate. The predetermined release rate guidelines may include:

1. Pre-project peak discharge
2. Critical capacity of the existing downstream drainage facility
3. Local design criteria

In practice, the design criteria, as well as the capacity of the existing drainage facilities shall be considered and the minimum discharge rate shall be used.

Outlet structures from detention basins may consist of risers, orifices, weirs and culverts, or any combination thereof. An example outlet structure is shown in Figure J-2 in Appendix J.

Typically the outlet structures consist of a small riser for low-flows, a larger riser or concrete vault for high flows, and an emergency overflow spillway. The small riser contains perforated holes and is typically wrapped with burlap or mesh to filter out sediment. The size and density of the perforations usually depends on the size of the pipe and the designated release rate. Under low flow conditions, water seeps through the perforated riser and into the outflow pipe. During high flow conditions, water spills through the top of the larger riser or concrete vault. Initially discharge through the larger riser occurs as weir flow, but as the water surface in the detention basin increases, the discharge will switch to orifice flow. The relationship between weir flow and orifice flow in a riser is shown in Figure J-3 in Appendix J. Finally, once the water surface in the detention pond exceeds a predetermined level above the top of the larger riser, water will be released through an overflow spillway at a rate determined by the size of the spillway as discussed in Section 6.1.3.

Outlet structures shall be sized so that the detained water surface returns to its original elevation within 24 hours of the cessation of a 100-year, 24-hour precipitation event over the tributary watershed.

6.1.3 Overflow Spillways

All detention facilities shall be equipped with an emergency spillway capable of accommodating the safe passage of flood flows resulting from the blockage of the primary outlet structure or the occurrence of a flood event larger than the design event. Typical details of the emergency spillway are shown in Figures J-4 through J-6 in

Appendix J. The spillway may or may not be located near the outlet works. The emergency overflow weir section shall be designed to pass the 100-year runoff for developed conditions assuming a broad-crested weir, using the following equation:

$$Q_{100} = C\,(2g)^{1/2}\,[{}^{2}/_{3}\,LH^{3/2} + {}^{8}/_{15}\,(\text{Tan}\,\theta)\,H^{5/2}] \qquad (6\text{-}1)$$

Where:	Q_{100}	=	peak flow for the 100-year runoff event, cfs
	C	=	discharge coefficient (0.6), dimensionless
	g	=	acceleration due to gravity, 32.2 ft/s^2
	L	=	length of weir, ft (Figure J-6)
	H	=	height of water over weir, ft, (Figure J-6)
	θ	=	angle of side slopes (Figure J-6)

The overflow spillway shall be directed in such a manner that overflows will be discharged in a safe manner and not directed at structures or critical public facilities.

6.1.4 Freeboard

Detention basin designs must provide at least one foot of freeboard between the elevation of the maximum design storage volume and lowest adjacent ground.

6.2 Retention Facilities

Retention facilities do not have surface outflow, but rely instead upon percolation to the groundwater and/or evaporation to the atmosphere to dispose of stored runoff.

Retention facilities should only be used in areas where winter groundwater tables and percolation rates warrant their construction, and no other method of drainage is available. Site specific data regarding winter evapotranspiration rates, groundwater levels and percolation rates must be included with retention basin design calculations and submitted to the County for approval. Retention facility approval shall be on a case-by-case basis. Before retention facilities are approved there must be a separate approval from the Santa Clara Valley Water District allowing percolation into the ground at the site proposed. The retention pond must also be reviewed for conformance to the standards and policies of Department of Environmental Health.

Provisions for providing a safe conveyance of emergency spill shall be provided for in conformance with the provisions of Section 6.1 above. Retention facilities shall contain at least 25 percent of the mean annual precipitation for the tributary watershed,

regardless of the design storm frequency used to design influent drainage facilities. Stored water in a retention facility shall percolate into the ground and return to its original elevation within 24 hours of the cessation of precipitation from a design 100-year, 24-hour rainfall event over the basin's tributary area.

Retention basin designs must provide at least one foot of freeboard between the elevation of the maximum design storage volume and lowest adjacent ground.

6.3 Detention Basin Applicability and Design

Whether storage of on-site runoff is of benefit to downstream areas depends primarily upon the overall hydrologic routing effect of a detention basin. Routing a flow hydrograph through a detention basin generally delays the peak of that hydrograph and reduces the maximum discharge. However, the delayed peak may in fact increase downstream peak discharges if the timing is such that combined discharges downstream are greater than the combined discharge that would have resulted if the detention basin did not exist.

Additionally, detention basin routing tends to increase the duration of flow at downstream locations. If downstream discharges are sufficient to cause erosion in the receiving channel, the use of detention may actually exacerbate that erosion.

For these reasons, the engineer must analyze the entire watershed and in particular, the downstream receiving waters, to determine whether the use of detention storage is applicable to a given project.

6.3.1 Very Small Watersheds

Very small watersheds are defined as having drainage areas smaller than 50 acres. Detention storage for certain smaller watersheds can be analyzed using a simplified detention basin sizing technique, such as the modified Rational Method approach defined in the 1981 APWA Special Report 49, "Urban Stormwater Management." The APWA method is shown in Appendix K. An alternate modified Rational approach is defined in the 1992 ASCE Manual of Practice No. 77, "Design and Construction of Urban Stormwater Management Systems." However, for analysis and design of storage such techniques have many drawbacks, related primarily to the fact that the intensity-duration-frequency relationships used in the analysis are intended for peak flow rate calculations, not for the calculation of runoff hydrographs or flow volumes.

The runoff coefficients used in either modified Rational Method should be based on the runoff coefficients in Table 3.1 but modified to provide a better estimate of runoff volume. For all weighted C-values using Table 3.1 less than 0.70, add 0.15 to develop a C-value for runoff volume computations. For all weighted C-values using Table 3.1 greater than 0.70, use a C-value for volume equal to (1.0 minus weighted C-value from Table 3.1) divided by 2 and added to the original weighted C-value from Table 3.1.

6.3.2 Other Watersheds

Other watersheds are defined as having drainage areas greater than 50 acres. The hydrograph method outlined in Chapter 4 is mandatory for the analysis and design of detention storage facilities in these other watersheds.

6.3.3 Design Guidelines

In the absence of other guidance from Santa Clara County, the following general procedure is suggested for detention basin design for a project:

1. Calculate the 10- and 100-year existing conditions peak discharges based on methodologies presented in the Drainage Manual. For drainage areas less than 200 acres, use the Rational Method from Chapter 3; for areas 200 acres and more, use the unit hydrograph method from Chapter 4.
2. Develop future conditions runoff hydrographs for the 10- and 100-year design precipitation event based on the unit hydrograph method (Chapter 4). Include a base flow of 5 cfs per square mile of drainage area for the 10-year hydrograph and 10 cfs per square mile of drainage area for the 100-year hydrograph. These base flow estimates are "rules of thumb" based on other hydrologic modeling efforts completed in and around Santa Clara County. (If available, recorded runoff hydrographs may also be used to separate the base flow component from direct surface runoff.) Base flow is included to contribute water to the detention pond and get the system working prior to the start of the inflow hydrograph.
3. Calculate required detention basin storage based on the 100-year hydrograph. Establish the volume in the future conditions runoff hydrograph from Step 2 that exceeds the existing conditions peak discharge (Step 1) and multiply by three. This multiplication factor (3) provides a first estimate of the storage volume needed to convey the flow passing through the basin, as well as provide enough additional volume to detain the increase between the existing and future conditions 100-year hydrographs. Using site information and topography, prepare a preliminary detention basin design and compute storage-elevation curves.

4. Size the outlet facility so that the existing conditions peak discharges for the 10- and 100-year events (Step 1) are not exceeded with the inflow hydrographs prepared in Step 2. Size the outlet pipe(s) to discharge the 10-year existing conditions peak discharge with one or two feet between the top of pipe and spillway crest. Design an overflow spillway to pass the 100-year existing peak discharge with roughly two feet of freeboard between the spillway crest and lowest adjacent grade.
5. Route the 10-year future land use conditions runoff hydrograph from Step 2 through the detention basin to verify that the maximum outflow through the outlet does not exceed the 10-year existing conditions discharge.
6. Route the 100-year future land use conditions hydrograph from Step 2 through the detention basin to verify that the maximum combined outflow through the outlet pipe and spillway does not exceed the 100-year existing conditions discharge. Ensure that the detention basin returns to the starting elevation used in the analysis within 24 hours of the end of 100-year, 24-hour rainfall.
7. Reiterate design until the requirements set forth in Steps 5 and 6 are met.

Detention basins that also provide water quality enhancements as described in Chapter 7 may be designed in similar fashion, accommodating additional low flow and hydrograph modification (HMP) criteria. Water quality basins not intended for flood flow attenuation must still provide an emergency spillway sized to safely pass a 100-year, 24-hour event.

In some cases detention basins can actually aggravate flooding conditions and lead to prolonged erosion downstream. The hydraulic routing effect of detention basins can contribute to these problems by modifying the outflow hydrograph in two ways. First, routing an inflow hydrograph through a detention basin produces an outflow hydrograph that may have a smaller peak, but the peak occurs later than the peak flow from the inflow hydrograph. Holding this smaller peak back can actually cause it to coincide with the peaks of other downstream hydrographs, such as those at stream confluences, thereby increasing the peak discharge of the combined hydrographs. Second, routing an inflow hydrograph through a detention basin prolongs the amount of time that flows occur in downstream channels. If the detention pond releases exceed the discharge levels needed to cause erosion in the receiving streams then those streams are subject to increased erosion due caused by the detention pond.

For these reason, it is crucial to evaluate the effects of detention basins not only just beyond the basin outfall, but also further downstream in the watershed. In lieu of on-site detention, regional detention facilities, modifications to off-site basins within the

watershed, improvements to upstream hydrologic conditions, improvements to proposed developments to minimize impervious cover, and the use of parking lot or rooftop detention should also be considered to improve downstream hydrologic conditions.

An example problem is shown in Appendix L for a 5-acre site undergoing subdivision and urbanization. Downstream of the site is an existing drainage problem. Three solutions are shown: using the APWA modified Rational Method, using the ASCE modified Rational Method, and using the hydrograph method.

6.4 Computer Programs

Computer programs discussed in Chapter 4 are capable of simulating storage in detention ponds through user-input stage-storage-discharge relationships. Other programs are available for sizing, designing and analyzing detention ponds, including PondPack® by Haestad Methods and Civil Design from AutoDesk.

A. APPENDIX A

Overland Flow Velocity

Mean Annual Precipitation Map

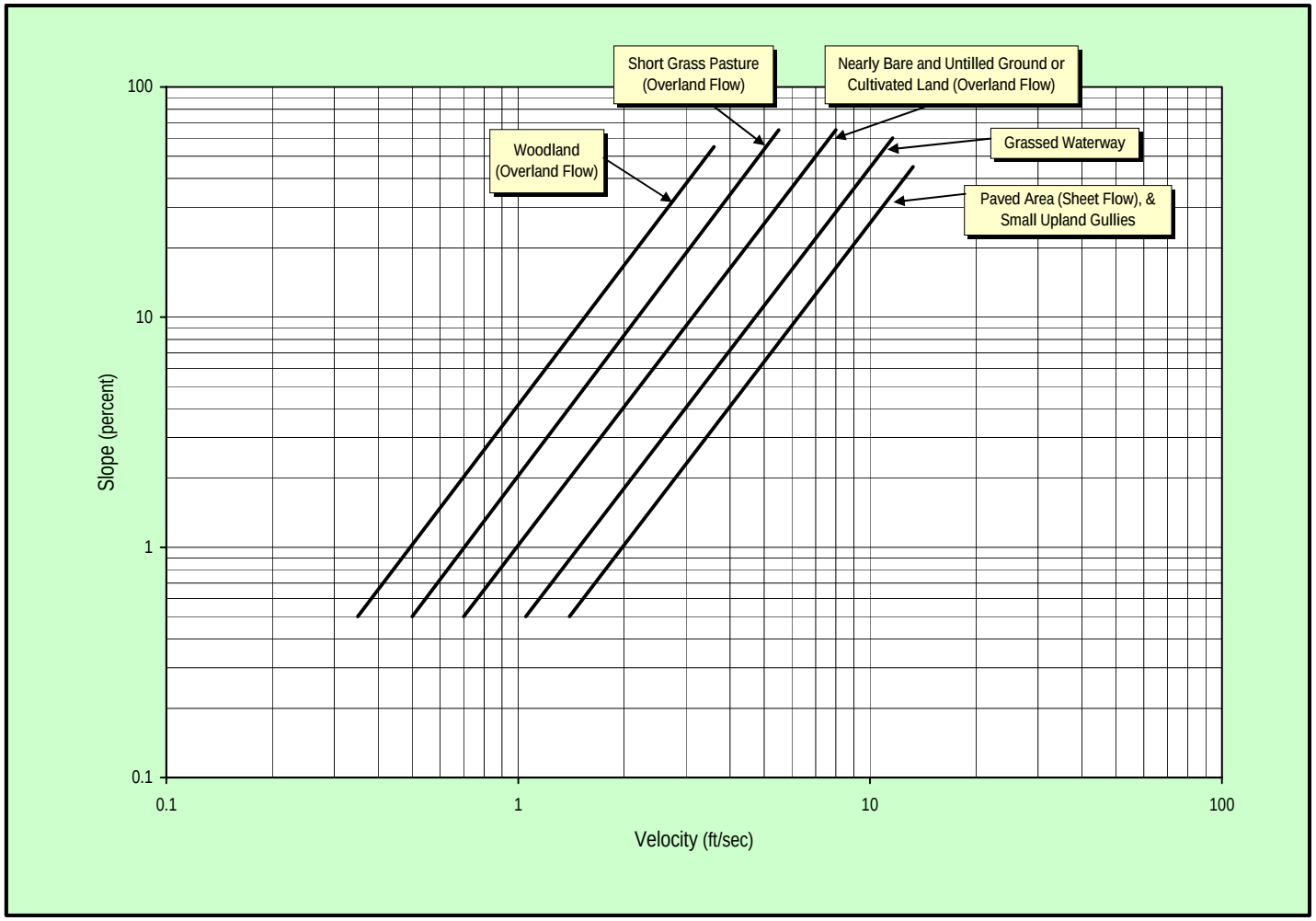
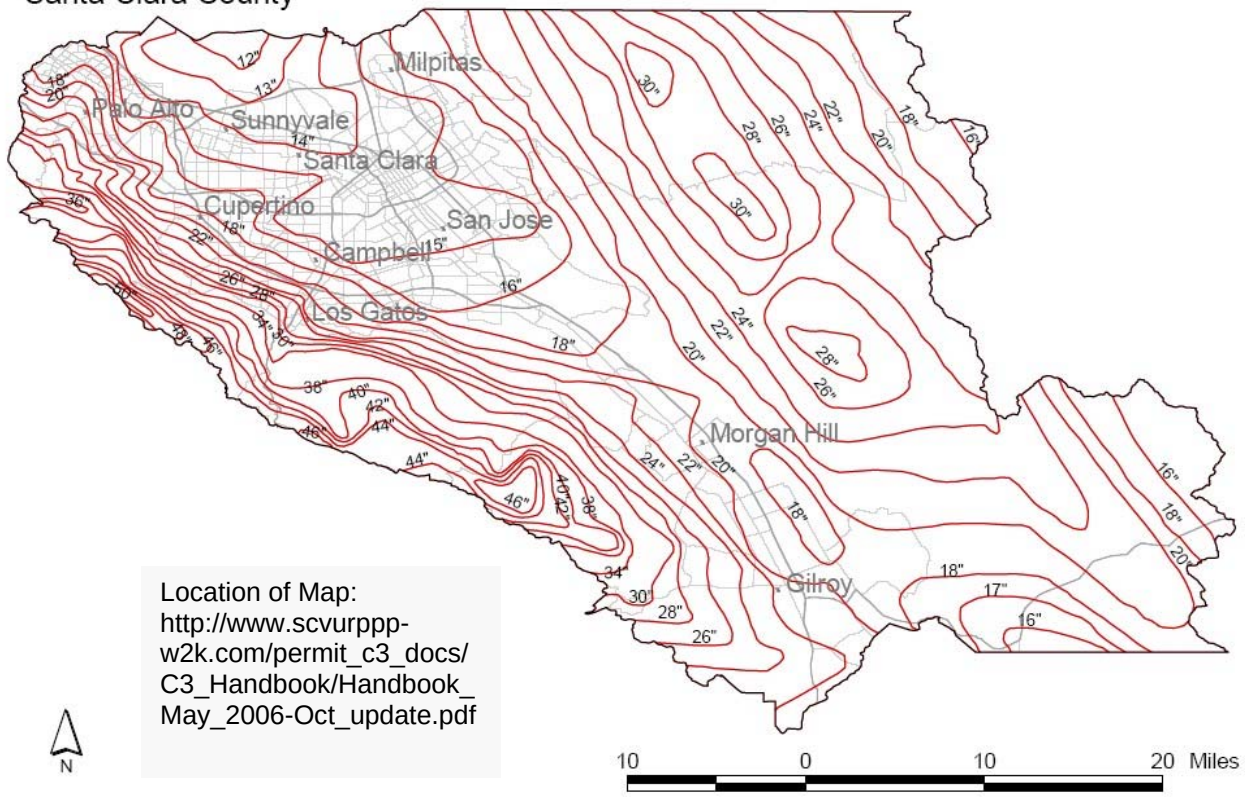


Figure A-1: Overland Flow Velocity

Figure A-2
Mean Annual Precipitation Map
Santa Clara County



SOURCE: Santa Clara Valley Water District, Mean Annual Precipitation Map, San Francisco & Monterey Bay Region, 1998

Figure A-2: Mean Annual Precipitation, Santa Clara County

B. APPENDIX B

IDF Curves

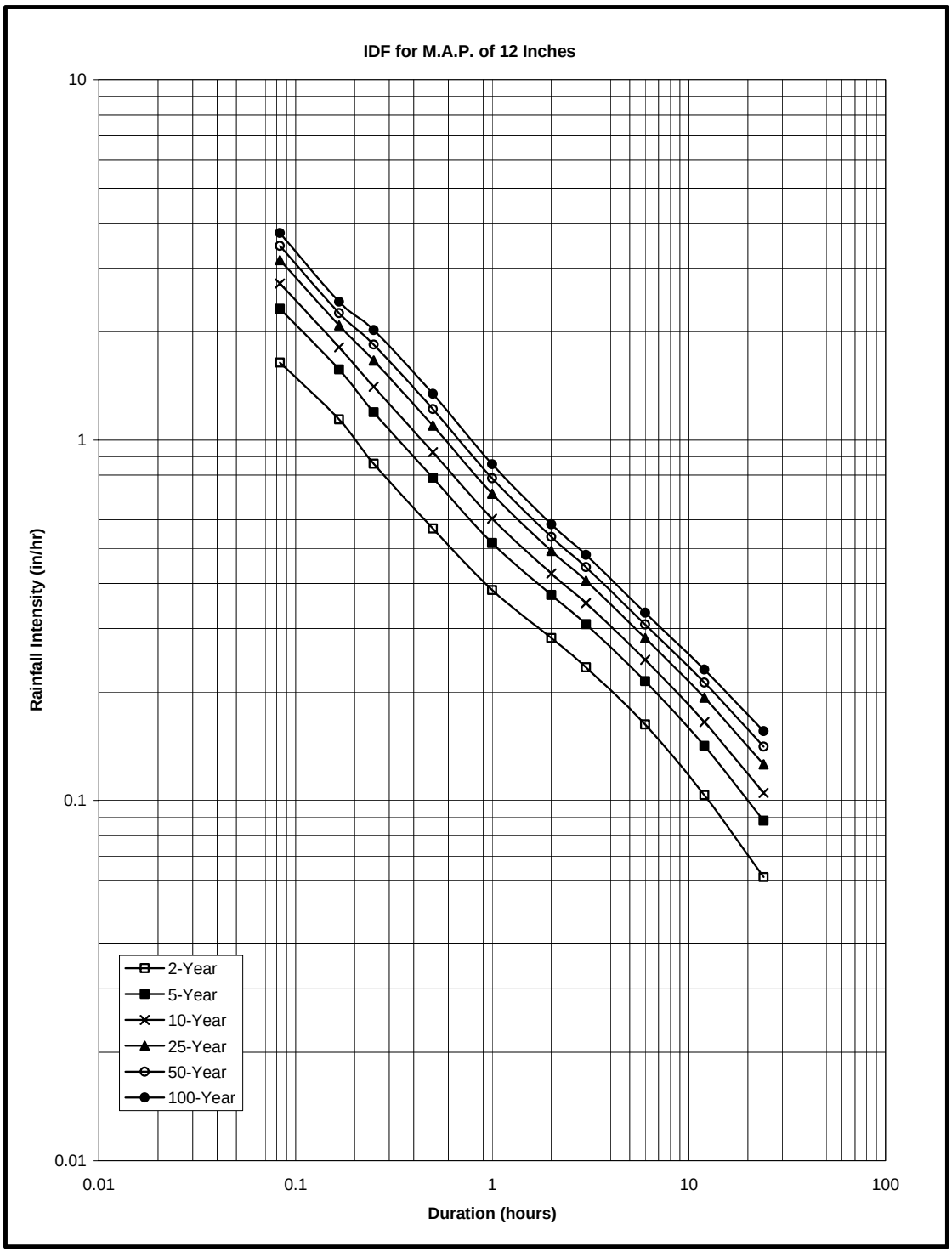


Figure B-1: IDF for M.A.P. of 12 Inches

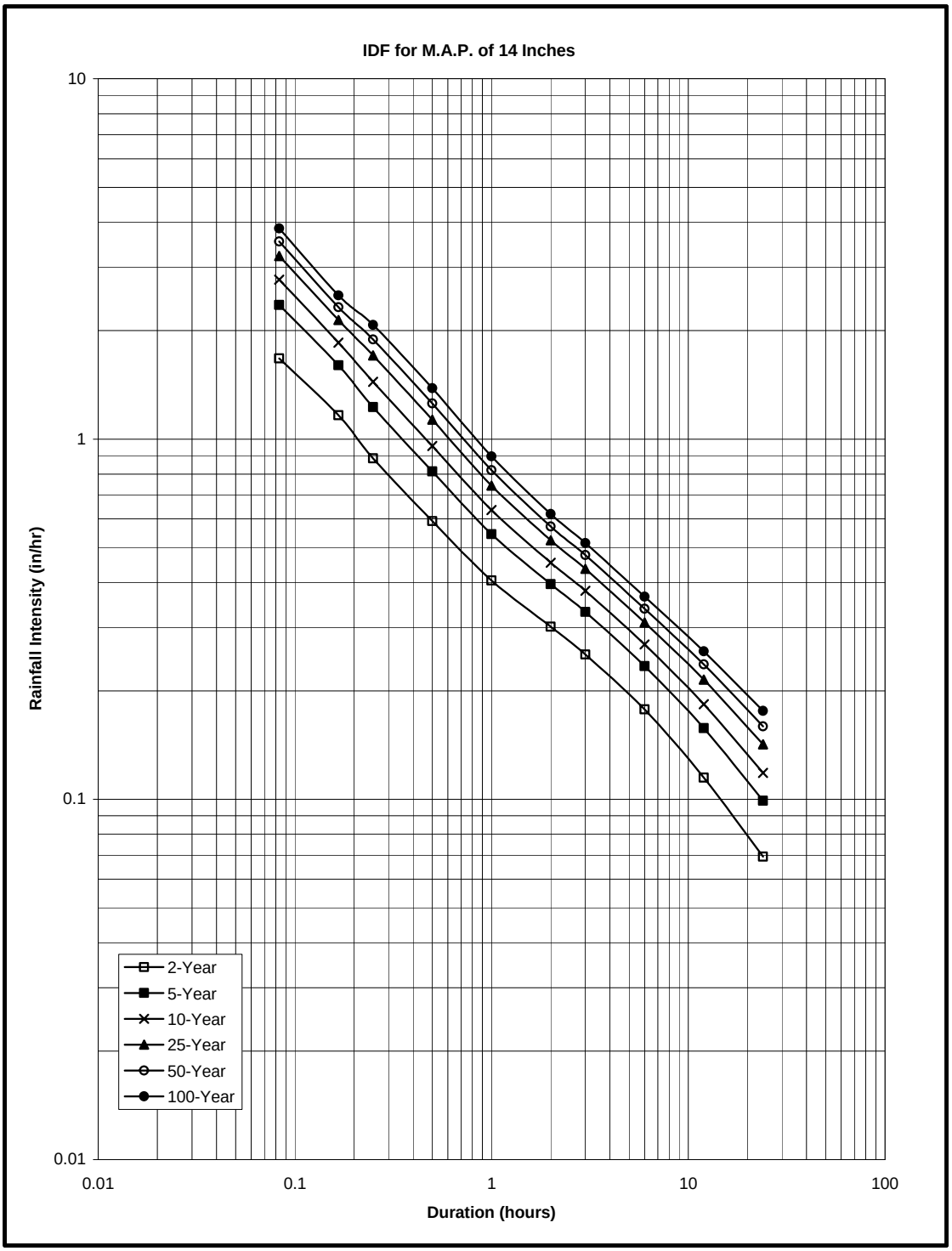


Figure B-2: IDF for M.A. P. of 14 Inches

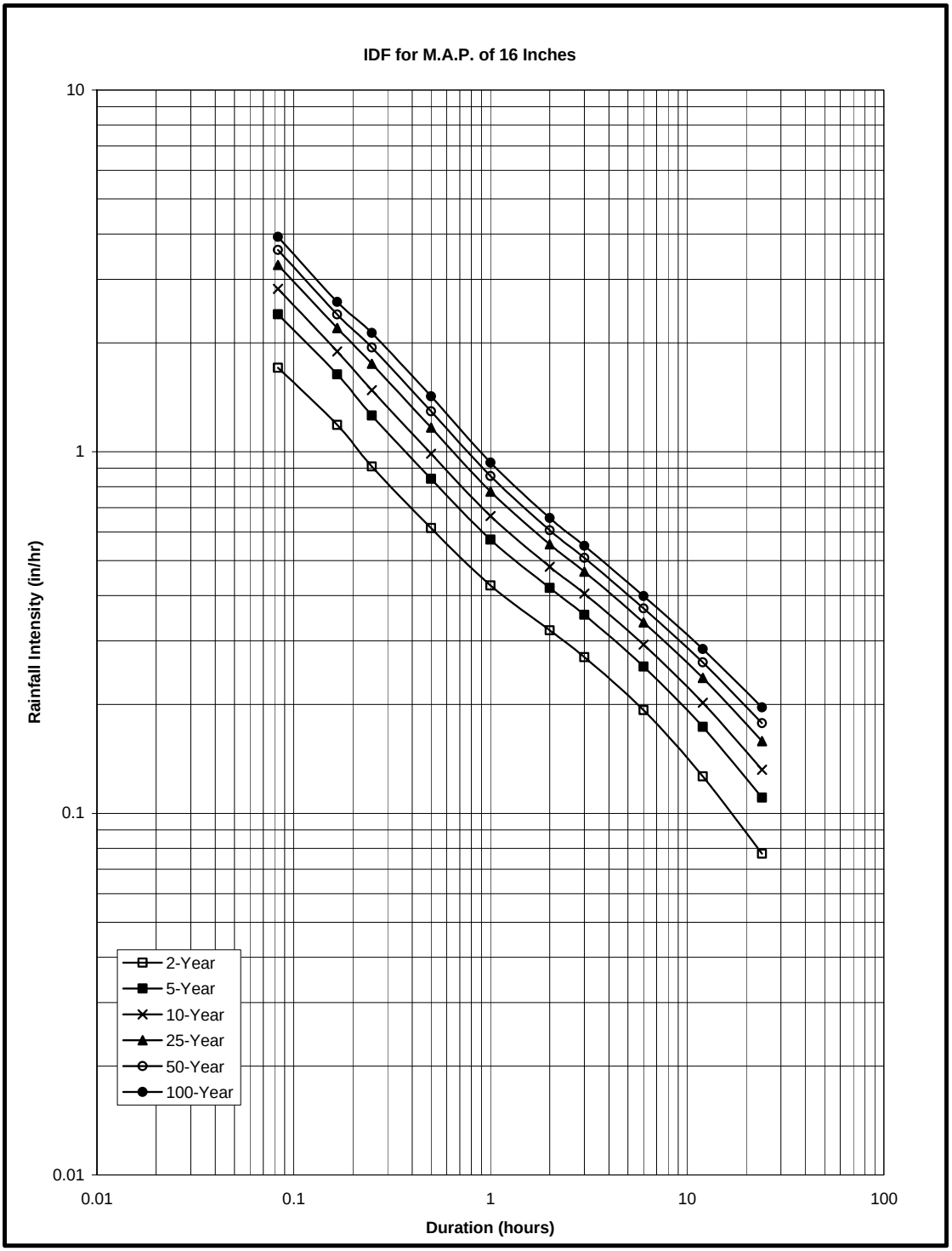


Figure B-3: IDF for M.A.P. of 16 Inches

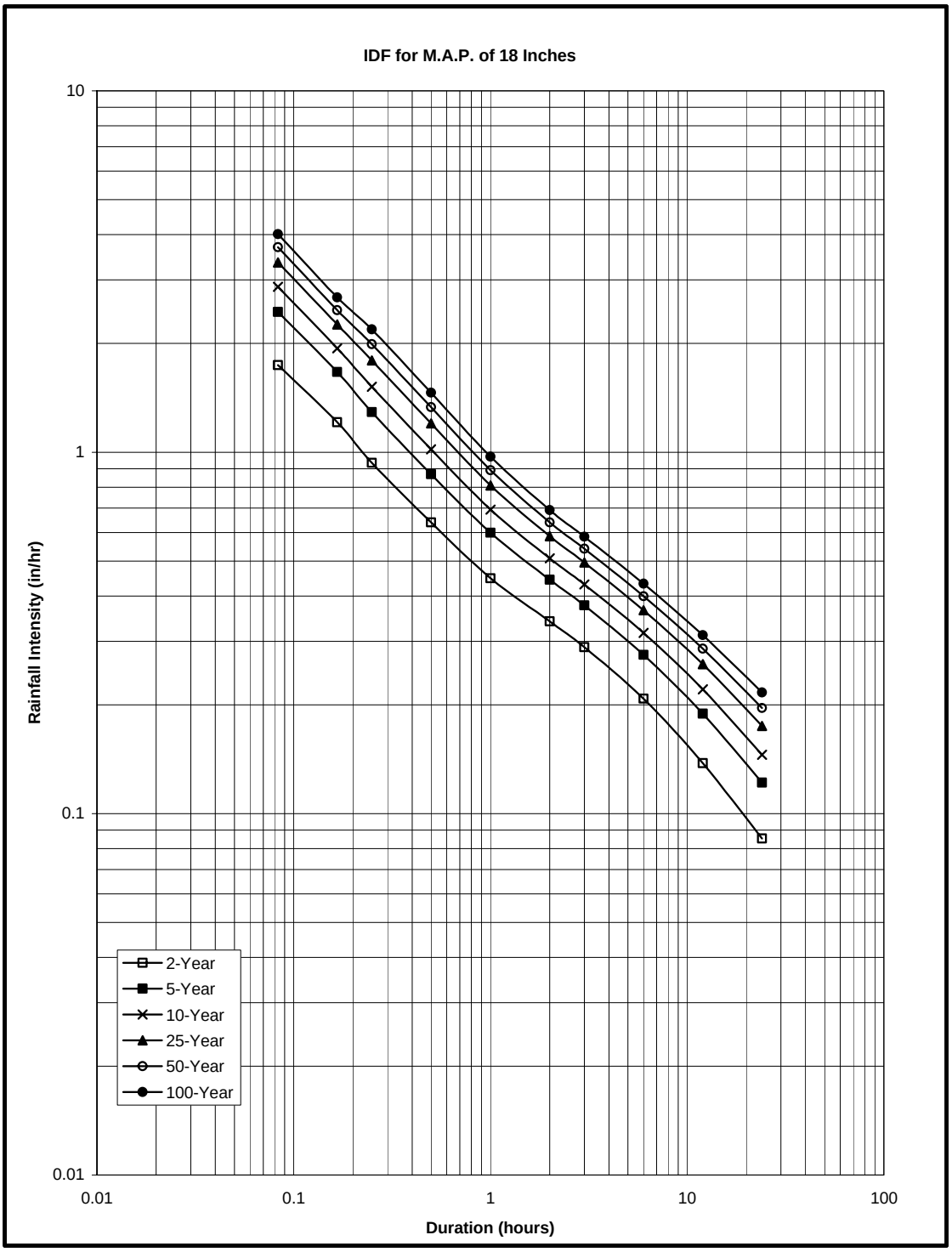


Figure B-4: IDF for M.A. P. of 18 Inches

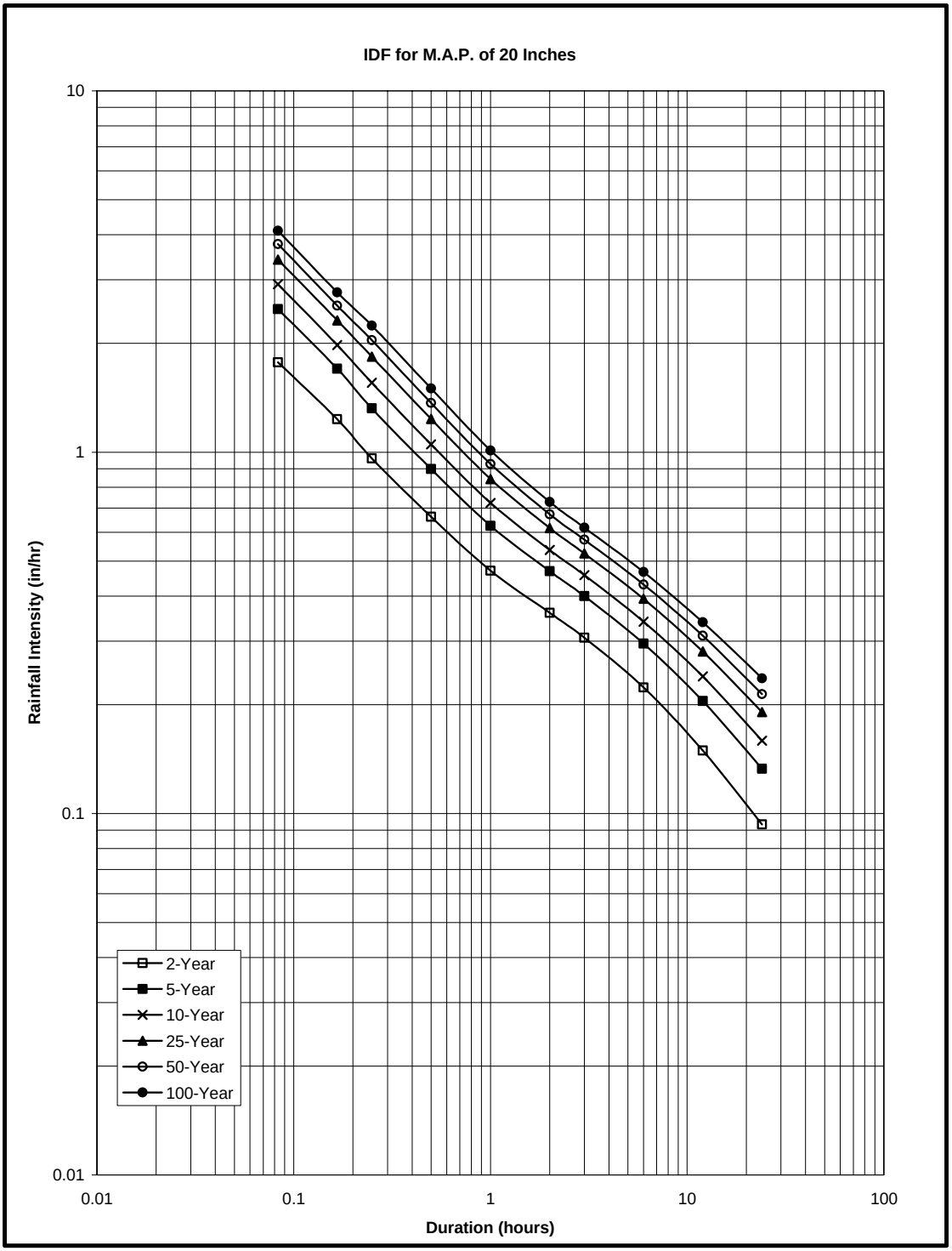


Figure B-5: IDF for M.A.P. of 20 Inches

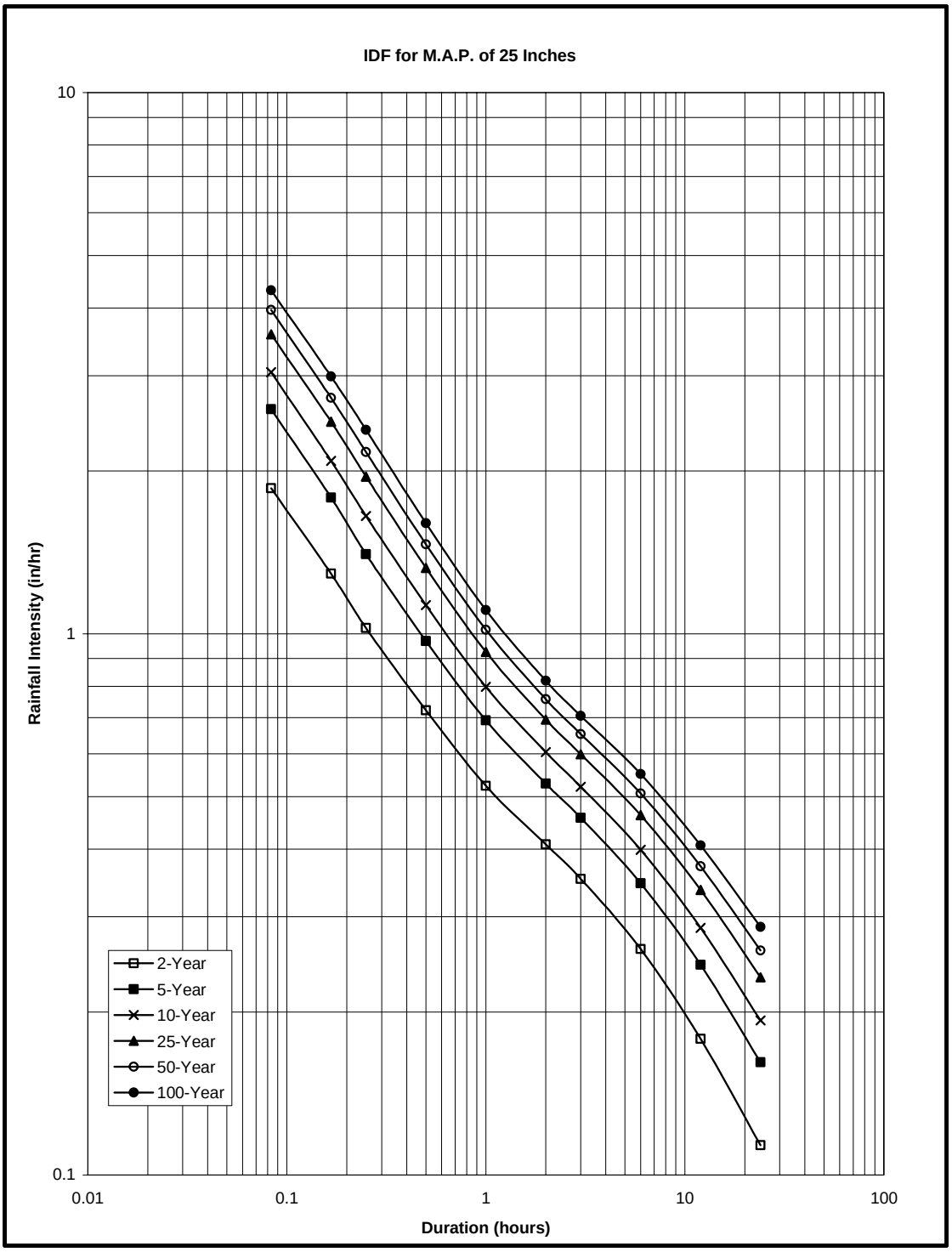


Figure B-6: IDF for M.A.P. of 25 Inches

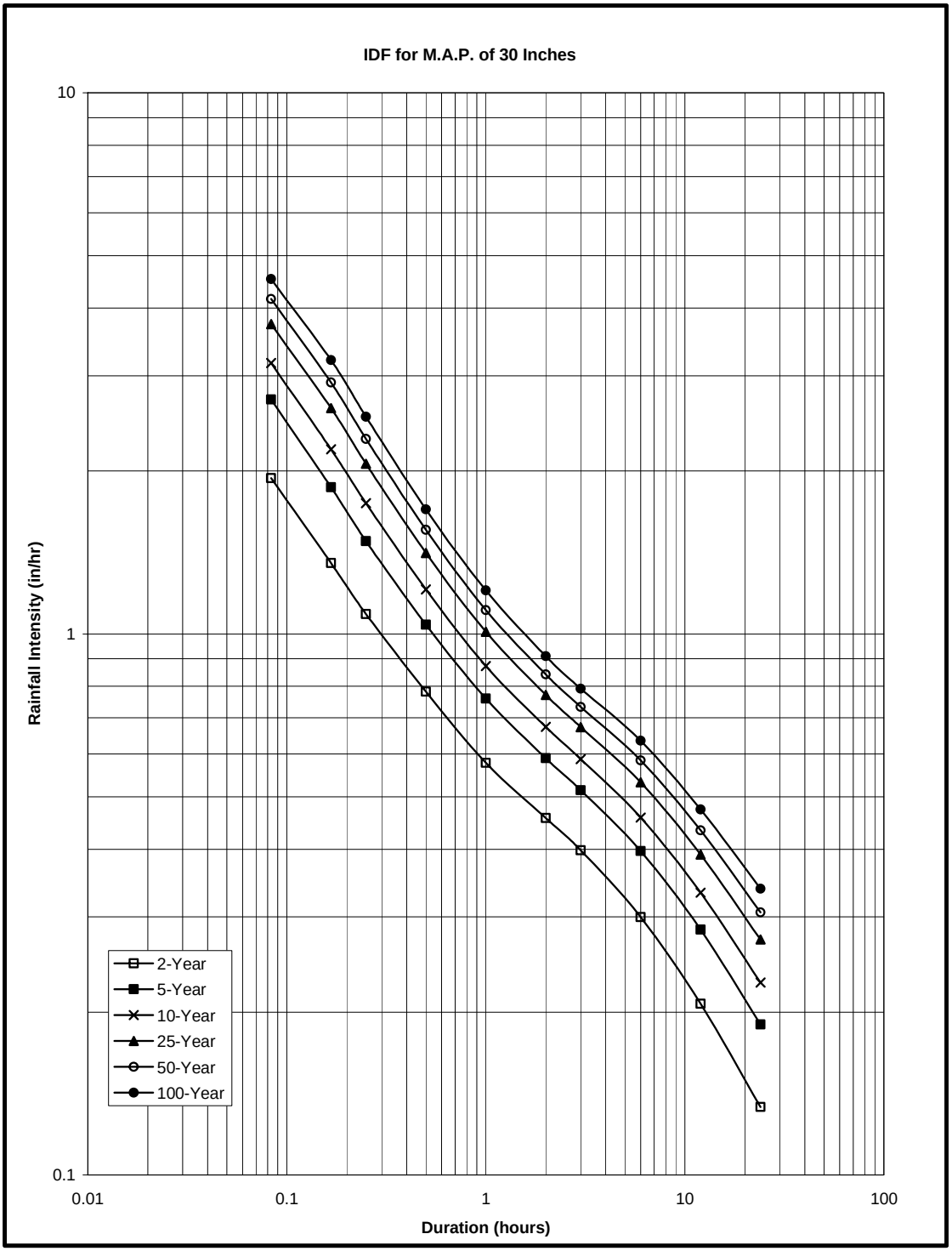


Figure B-7: IDF for M.A. P. of 30 Inches

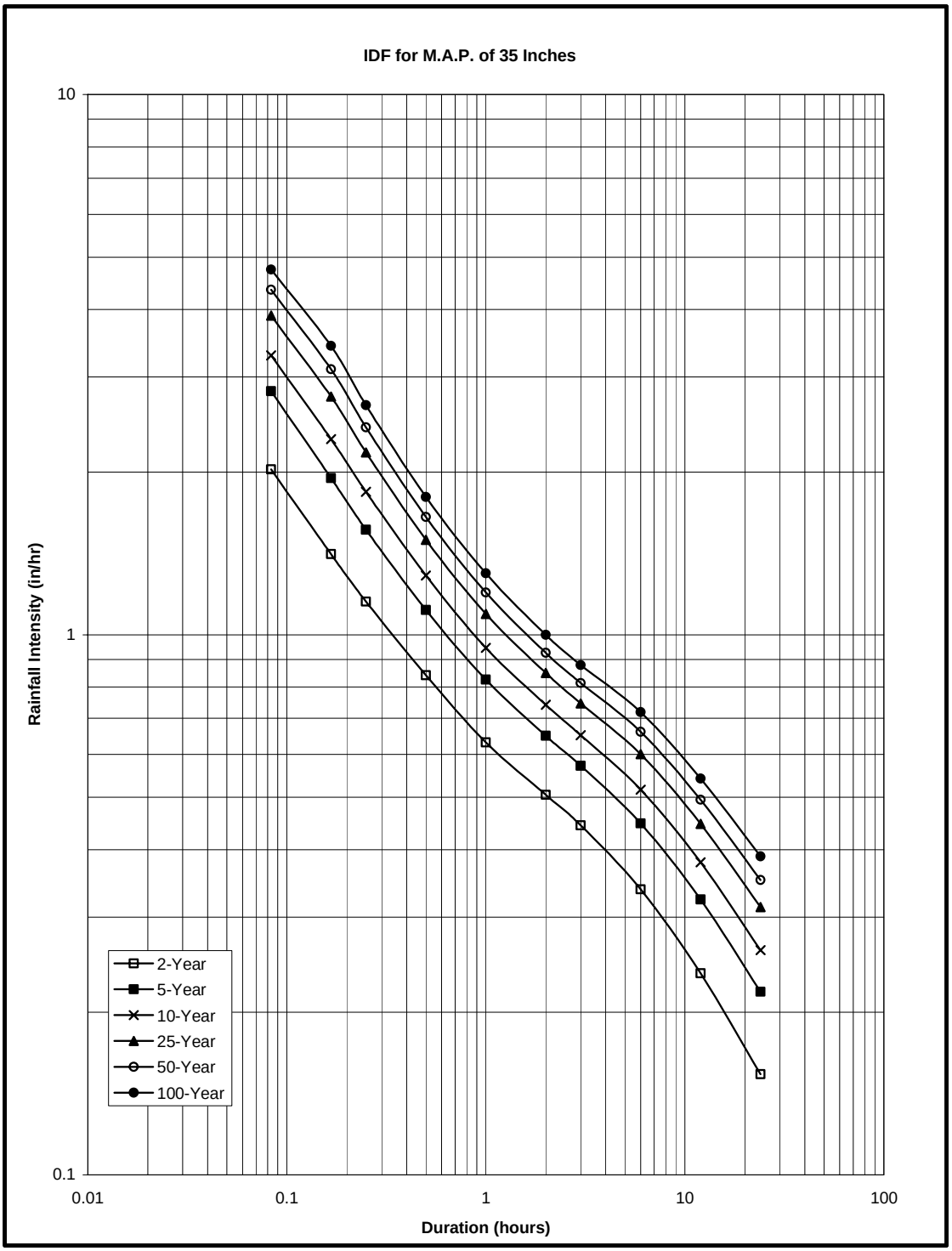


Figure B-8: IDF for M.A.P. of 35 Inches

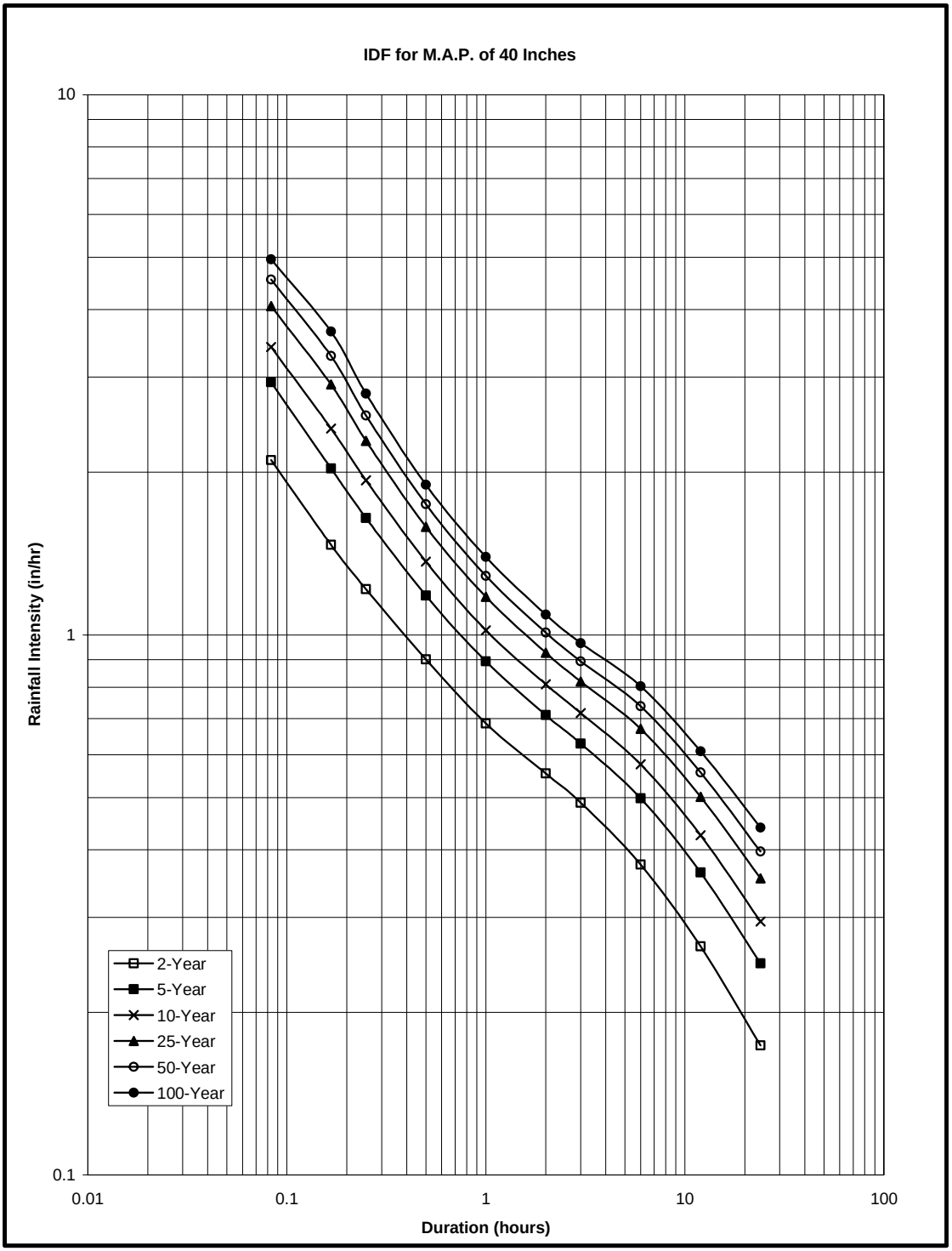


Figure B-9: IDF for M.A.P. of 40 Inches

Table B-1: Parameters $A_{T,D}$ and $B_{T,D}$ for TDS Equation

2-YR RETURN PERIOD		
5-min	0.120194	0.001385
10-min	0.166507	0.001956
15-min	0.176618	0.003181
30-min	0.212497	0.005950
1-hr	0.253885	0.010792
2-hr	0.330848	0.019418
3-hr	0.374053	0.027327
6-hr	0.425178	0.045735
12-hr	0.409397	0.069267
24-hr	0.314185	0.096343
48-hr	0.444080	0.134537
72-hr	0.447104	0.159461
5-YR RETURN PERIOD		
5-min	0.170347	0.001857
10-min	0.228482	0.002758
15-min	0.250029	0.004036
30-min	0.307588	0.007082
1-hr	0.357109	0.013400
2-hr	0.451840	0.024242
3-hr	0.512583	0.034359
6-hr	0.554937	0.060859
12-hr	0.562227	0.094871
24-hr	0.474528	0.136056
48-hr	0.692427	0.187173
72-hr	0.673277	0.224003
10-YR RETURN PERIOD		
5-min	0.201876	0.002063
10-min	0.258682	0.003569
15-min	0.294808	0.004710
30-min	0.367861	0.007879
1-hr	0.427723	0.014802
2-hr	0.522608	0.027457
3-hr	0.591660	0.038944
6-hr	0.625054	0.070715
12-hr	0.641638	0.111660
24-hr	0.567017	0.162550
48-hr	0.832445	0.221820
72-hr	0.810509	0.265469

Table B-2: Parameters $A_{T,D}$ and $B_{T,D}$ for TDS Equation

Return Period/Duration	$A_{T,D}$	$B_{T,D}$
25-YR RETURN PERIOD		
5-min	0.230641	0.002691
10-min	0.287566	0.004930
15-min	0.348021	0.005594
30-min	0.443761	0.008719
1-hr	0.508791	0.016680
2-hr	0.612629	0.031025
3-hr	0.689252	0.044264
6-hr	0.693566	0.083195
12-hr	0.725892	0.132326
24-hr	0.675008	0.195496
48-hr	0.989588	0.264703
72-hr	0.967854	0.316424
50-YR RETURN PERIOD		
5-min	0.249324	0.003241
10-min	0.300971	0.006161
15-min	0.384016	0.006315
30-min	0.496301	0.009417
1-hr	0.568345	0.017953
2-hr	0.672662	0.033694
3-hr	0.754661	0.048157
6-hr	0.740666	0.092105
12-hr	0.779967	0.147303
24-hr	0.747121	0.219673
48-hr	1.108358	0.295510
72-hr	1.075643	0.353143
100-YR RETURN PERIOD		
5-min	0.269993	0.003580
10-min	0.315263	0.007312
15-min	0.421360	0.006957
30-min	0.553934	0.009857
1-hr	0.626608	0.019201
2-hr	0.732944	0.036193
3-hr	0.816471	0.051981
6-hr	0.776677	0.101053
12-hr	0.821859	0.162184
24-hr	0.814046	0.243391
48-hr	1.210895	0.325943
72-hr	1.175000	0.389038

C. APPENDIX C

Calculation Sheets

Runoff Coefficient (based on Santa Clara County Drainage Manual):

Land Use _____ Weighted C-Value = _____

Rainfall Curves: Source _____

Return Period _____ years (County minimum standard = 10 years)

Time of Concentration (improved): Gutter Time _____ min. Total Inlet Time _____ min.

Pipe (minimum standard size = 12" diameter): Type _____ Manning's "n" _____

Minimum Velocity at Design Flow, V = _____ fps (min. standard velocity = 2.6 ft/sec.)

Attach map showing boundaries of drainage areas, runoff coefficients, inlet locations, Q's, slope, point of concentration, existing and proposed drainage facilities. If, under normal conditions, pipe will flow full at any point in the system, attach plot of hydraulic grade line.

county of santa clara

DEPARTMENT OF PUBLIC WORKS

APPROVED: _____ DIRECTOR

STANDARD FORM DM-2

Drainage Manual

STORM DRAIN DESIGN BY RATIONAL METHOD

Project Location | Computations by | Date

Type of Development | Total Area (Acres)

Remarks | Signature of Applicant

Concentration Point	Discharge (Q), cfs	Ground Slope *	Pipe Slope, S v/s	Σ Product Area * Coefficient	Duration, min. (Time of Concentration)	Rainfall Intensity, I (inches/hr.)	Runoff, cfs Q = CIA	Length, ft.	Pipe Diameter, in.	Slope, percent	Depth of Flow, Dd	Hydraulic Radius, ft.	Velocity, fps	$V^2/2g$	Flow Time, min.	Fall, ft.	Flow Time, min.	Invert Elevation, ft. In	Invert Elevation, ft. Out	Finished Grade Elevation, ft. In	Finished Grade Elevation, ft. Out	Minimum Cover, ft.	Location of Point of Minimum Contact, ft.	Cut, ft. In	Cut, ft. Out	Remarks
	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16										
			1 X 2	3 ÷ 4	5 ÷ 15		4 X 6								8/13	8 X 10										

EXAMPLE

Figure C-1: Calculation Sheet, Storm Drain Design by Rational Method

Based on Manning's Equation: $d_i = (1630Qn/\sqrt{s})^{3/8} = 16.02\ (Qn/\sqrt{S})^{3/8}$

Pipe (minimum standard size = 12" dia.): Type _____ Manning's "n" _____

Min. Velocity at Design Flow, V = _____ ft/sec (min. std. Vel. = 2.6 ft/sec)

Attach map showing boundaries of drainage areas, inlet locations; Q's, slopes, points of concentration, existing and proposed drainage facilities.

If under normal conditions, pipe will flow full at any point in the system, attach plot of hydraulic grade line.

county of santa clara
DEPARTMENT OF PUBLIC WORKS
APPROVED: _____ DIRECTOR

STANDARD FORM DM-3
Drainage Manual
PIPE SIZE AND HYDRAULIC GRADIENT COMPUTATIONS

Project Location _____ Computations by _____ Date _____

Type of Development _____ Total Area (Acres) _____

Remarks _____ Signature of Applicant _____

Concentration Point	Discharge (Q) cfs	Ground Slope *	Pipe Slope, S (ft/100ft)	√S	Θ ξ ν	Q x n / √S	(Q x n / √S)3/8	Constant (16.02)	Required Dia., di (inches)	Pipe Dia. Used, d (inches)	Required Area, A** (square feet)	Velocity, V=Q/A (feet/sec)	Velocity Head, V2/2g (feet)	Minor Losses		Length of Reach, L (feet)	Fall in Reach (feet)	Remarks
														K	KV2/2g (feet)			

EXAMPLE

Figure C-2: Pipe Size and Hydraulic Gradient Computations

D. APPENDIX D

Normalized Rainfall Pattern

Fractions of Total Rainfall

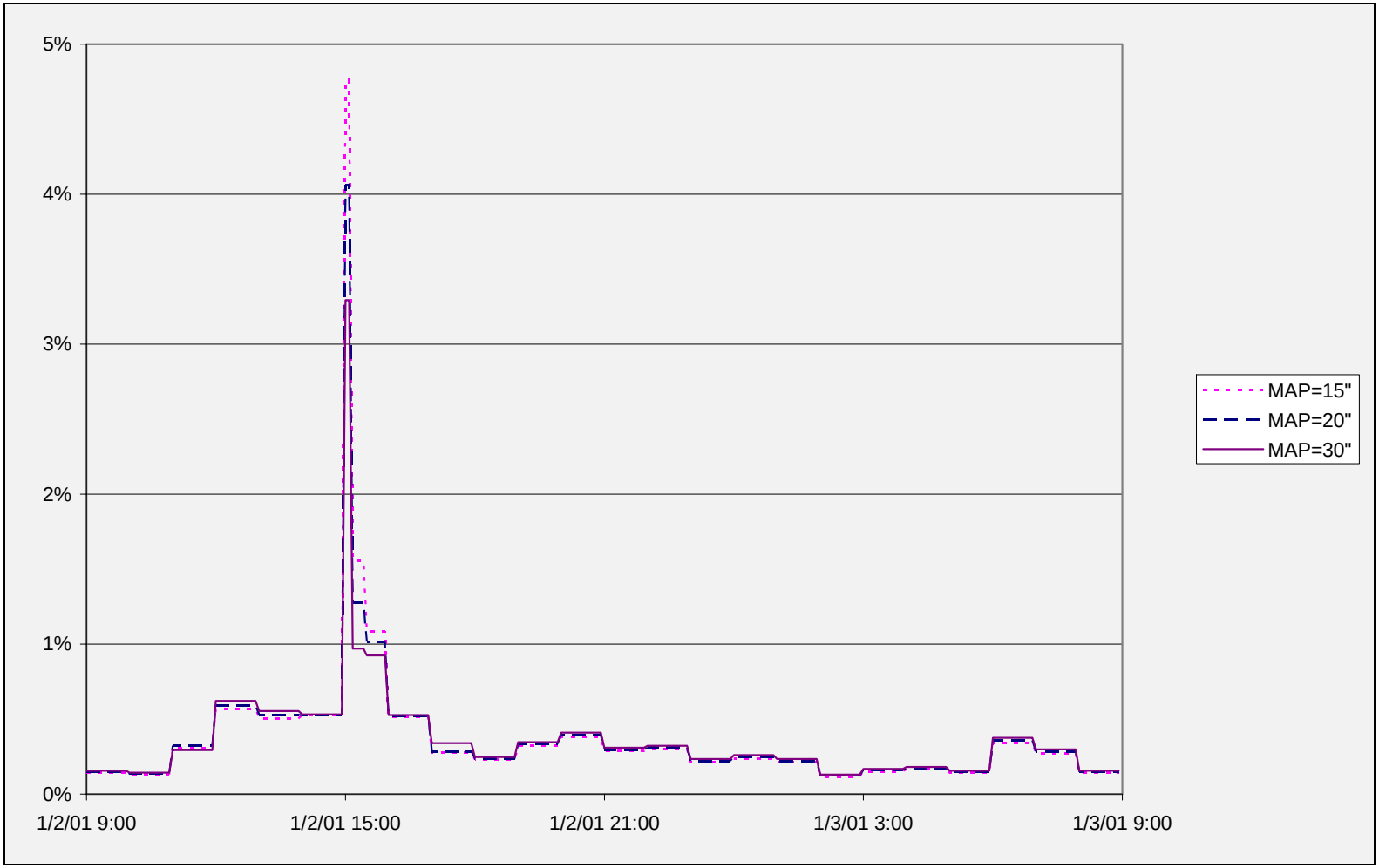


Figure D-1: Normalized Rainfall Pattern

Table D-1: Fractions of Total Rainfall for 24-Hour, 5-Minute Pattern

Time Starting	Fraction of Total Rainfall (%) *MAP=15″*	Fraction of Total Rainfall (%) *MAP=20″*	Fraction of Total Rainfall (%) *MAP=30″*
0:00	0.1412	0.1482	0.1558
1:00	0.1294	0.1358	0.1429
2:00	0.3080	0.3223	0.2945
3:00	0.5667	0.5930	0.6214
4:00	0.5051	0.5285	0.5538
5:00	0.5272	0.5266	0.5324
6:00	4.760	4.060	3.2950
6:10	1.554	1.275	0.9700
6:30	1.085	1.0169	0.9253
7:00	0.5177	0.5229	0.5263
8:00	0.2763	0.2860	0.3410
9:00	0.2302	0.2384	0.2478
10:00	0.3223	0.3337	0.3469
11:00	0.3799	0.3933	0.4089
12:00	0.2878	0.2979	0.3098
13:00	0.2993	0.3099	0.3222
14:00	0.2118	0.2223	0.2338
15:00	0.2353	0.2470	0.2597
16:00	0.2118	0.2223	0.2338
17:00	0.1177	0.1235	0.1299
18:00	0.1530	0.1605	0.1688
19:00	0.1647	0.1729	0.1818
20:00	0.1412	0.1482	0.1558
21:00	0.3412	0.3581	0.3766
22:00	0.2706	0.2840	0.2987
23:00	0.1412	0.1482	0.1558

E. APPENDIX E

Curve Numbers

Table E-1: Curve Numbers for AMC II

Land Use Type	Hydrologic Condition	Hydrologic Soil Group A	B	C	D
Open Water	good				
(100% Impervious)	fair				
	poor				
Low Density Residential	good	35	48	66	70
(25% Impervious)	fair	44	58	71	74
	poor	64	68	78	79
High Density Residential	good	35	48	65	70
(50% Impervious)	fair	44	58	71	74
	poor	64	68	78	79
Commercial/Industrial	good	35	48	65	70
(80% Impervious)	fair	44	58	71	74
	poor	64	68	78	79
Bare Rock/Sand/Clay					
(Imperviousness Varies)					
Quarries/Gravel Pits	good	0	0	0	0
(0 % Impervious)	fair	0	0	0	0
	poor	0	0	0	0
Deciduous Forest	good	27	30	41	48
(0% Impervious)	fair	35	48	57	63
	poor	48	66	74	79
Evergreen Forest	good	37	43	62	70
(0% Impervious)	fair	45	57	69	80
	poor	58	71	85	90
Mixed Forest	good	32	36	51	59
	fair	40	52	63	72
	poor	53	68	80	85
Shrub Land	good	27	43	60	68
(0% Impervious)	fair	35	51	65	72
	poor	48	62	72	78
Orchards	good	39	52	66	71
(1% Impervious)	fair	43	65	76	82
	poor	57	73	82	86
Vineyards	good	64	70	77	80
(1% Impervious)	fair	67	75	82	85
	poor	71	80	87	90
Grassland	good	38	50	69	76
(0% Impervious)	fair	48	60	74	80
	poor	58	70	80	84
Pasture/Hay	good	34	50	69	76
(0% Impervious)	fair	44	60	74	80
	poor	64	70	80	84
Row Crops	good	64	70	77	80
(1% Impervious)	fair	67	75	82	85
	poor	71	80	87	90
Small Grains	good	48	58	70	74
(0% Impervious)	fair	49	59	71	75
	poor	50	60	71	75
Fallow	good	64	68	78	79
(1% Impervious)	fair	70	77	84	86
	poor	77	86	91	94
Urban Recreational	good	34	48	66	70
(10% Impervious)	fair	44	58	71	74
	poor	64	64	78	79

Table E-2: Conversion of AMC II Curve Numbers to Other AMC Values

AMC II	AMC I	AMC III	AMC II-1/4	AMC II-1/2	AMC II	AMC I	AMC III	AMC II-1/4	AMC II-1/2
100	100	100	100	100	61	41	78	65.5	70
99	97	100	99.5	100	60	40	78	64.5	69
98	94	99	98.5	99	59	39	77	63.5	68
97	91	99	97.5	98	58	38	76	62.5	67
96	89	99	97	98	57	37	75	61.5	66
95	87	98	96	97	56	36	75	61	66
94	85	98	95	96	55	35	74	60	65
93	83	98	94.5	96	54	34	73	59	64
92	81	97	93.5	95	53	33	72	58	63
91	80	97	92.5	94	52	32	71	57	62
90	78	96	91.5	93	51	31	70	56	61
89	76	96	91	93	50	31	70	55	60
88	75	95	90	92	49	30	69	54	59
87	73	95	89	91	48	29	68	53	58
86	72	94	88	90	47	28	67	52	57
85	70	94	87.5	90	46	27	66	51	56
84	68	93	86.5	89	45	26	65	50	55
83	67	93	85.5	88	44	25	64	49	54
82	66	92	84.5	87	43	25	63	48	53
81	64	92	84	87	42	24	62	47	52
80	63	91	83	86	41	23	61	46	51
79	62	91	82	85	40	22	60	45	50
78	60	90	81	84	39	21	59	44	49
77	59	89	80	83	38	21	58	43	48
76	58	89	79.5	83	37	20	57	42	47
75	57	88	78.5	82	36	19	56	41	46
74	55	88	77.5	81	35	18	55	40	45
73	54	87	76.5	80	34	18	54	39	44
72	53	86	75.5	79	33	17	53	38	43
71	52	86	75	79	32	16	52	37	42
70	51	85	74	78	31	16	51	36	41
69	50	84	73	77	30	15	50	35	40
68	48	84	72	76	25	12	43	29.5	34
67	47	83	71	75	20	9	37	24.5	29
66	46	82	70	74	15	6	30	19	23
65	45	82	69.5	74	10	4	22	13	16
64	44	81	68.5	73	5	2	13	7	9
63	43	80	67.5	72	0	0	0	0	0

F. APPENDIX F

Manning's N-Values

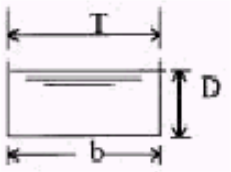
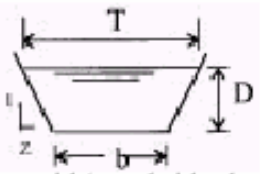
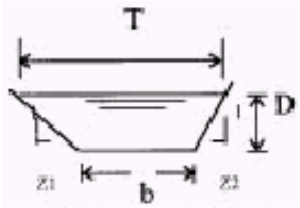
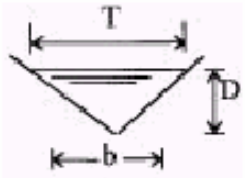
Open Channel Cross Sections

Loss Coefficients

Table F-1: Manning's Roughness Coefficients for Closed Conduits and Open Channels

Conveyance Material	Manning's n-value
Closed Conduits	
Concrete	
1. Precast or cast-in-place	0.013-0.015
2. Steel troweled or smooth-form finish	0.014-0.016
3. Wood float or broomed finish; including pneumatically applied mortar	0.014-0.017
Corrugated Metal Pipe	
1. Plain	0.022-0.026
2. Paved invert	0.018-0.022
3. Spun asphalt lined	0.011-0.015
Plastic (HDPE, PVC)	0.008-0.015
Vitrified Clay	0.011-0.015
Steel, coated	0.010-0.017
Brick	0.013-0.017
Open Channels	
Excavated or Dredged	
1. Earth, straight and uniform	0.020-0.030
2. Earth, winding and fairly uniform	0.025-0.040
3. Rock, smooth and uniform	0.025-0.033
4. Rock, jagged and irregular	0.035-0.045
5. With short grass, few weeds	0.022-0.033
6. Unmaintained, abundant vegetation as tall as flow depth	0.050-0.140
Lined	
1. Asphalt	0.013-0.017
2. Brick	0.011-0.018
3. Concrete	0.011-0.020
4. Riprap or rubble	0.020-0.035
5. Sack concrete riprap/Grouted rock riprap	0.028-0.032
6. With short grass, few weeds	0.022-0.033
7. Unmaintained, abundant vegetation as tall as flow depth	0.050-0.140
Natural Stream Channels	
1. Clean, straight bank, full stage no rifts or deep pools	0.025-0.033
2. Same as (1), but some weeds and stones	0.030-0.040
3. Clean, winding, some pools and shoals	0.033-0.045
4. Same as (3), lower stages, more ineffective slope and sections	0.040-0.055
5. Same as (3), some weeds and stones	0.035-0.050
6. Same as (5), some stony sections	0.045-0.060
7. Sluggish river reaches, rather weedy or with very deep pools	0.050-0.080
8. Very weedy reaches, trees or underbrush	0.075-0.150

Table F-2: Geometric Elements of Channel Sections

Cross Section	Area, A	Wetted Perimeter, WP
 Rectangle	bD	$b + 2D$
 Trapezoid (equal side slopes)	$(b + ZD)D$	$b + 2D\sqrt{1 + Z^2}$
 Trapezoid (unequal side slopes)	$\frac{D^2}{2}(Z_1 + Z_2) + Db$	$b + D(\sqrt{1 + Z_1^2} + \sqrt{1 + Z_2^2})$
 Triangle	ZD^2	$2D\sqrt{1 + Z^2}$

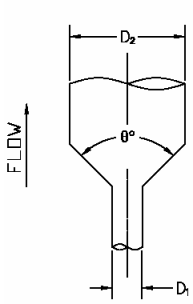
Reference: VT Chow "Open Channel Hydraulics" for a more complete table of geometric elements.

Table F-3: Storm Sewer Energy Loss Coefficients

(a) Expansion (K_c)

Θ*	$D_2/D_1 = 3$	$D_2/D_1 = 1.5$
10	0.17	0.17
20	0.40	0.40
45	0.86	1.06
60	1.02	1.21
90	1.06	1.14
120	1.04	1.07
180	1.00	1.00

* The angle is the angle in degrees between the sides of the



(b) Pipe Entrance from Reservoir

Bell Mouth	$H_L = 0.04 V^2/2g$
Square-Edge	$H_L = 0.04 V^2/2g$
Groove End U/S For Concrete Pipe	$H_L = 0.2 V^2/2g$

(c) Contractions (K_c)

D_2/D_1	K_c
0	0.5
0.4	0.4
0.6	0.3
0.8	0.1
1.0	0

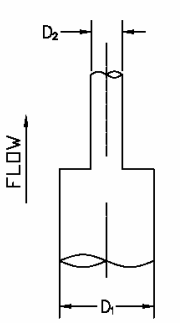


Table F-4: Values of K_e for Determining Loss of Head Due to Sudden Enlargement in Pipes, from the Equation: $H_2 = K_2(V_1^2/2g)$

d_2/d_1 = ratio of larger pipe to smaller pipe												V_1 = velocity in smaller pipe	
$\underline{d_2}$	Velocity, V_1, in feet per second												
d_1	2	3	4	5	6	7	8	10	12	15	20	30	40
1.2	.11	.10	.10	.10	.10	.10	.10	.09	.09	.09	.09	.09	.08
1.4	.26	.26	.25	.24	.24	.24	.24	.23	.23	.22	.22	.21	.20
1.6	.40	.39	.38	.37	.37	.36	.36	.35	.35	.34	.33	.32	.32
1.8	.51	.49	.48	.47	.47	.46	.46	.45	.44	.43	.42	.41	.40
2.0	.60	.58	.56	.55	.55	.54	.53	.52	.52	.51	.50	.48	.47
2.5	.74	.72	.70	.69	.68	.67	.66	.65	.64	.63	.62	.60	.58
3.0	.83	.80	.78	.77	.76	.75	.74	.73	.72	.70	.69	.67	.65
4.0	.92	.89	.87	.85	.84	.83	.82	.80	.79	.78	.76	.74	.72
5.0	.96	.93	.91	.89	.88	.87	.86	.84	.83	.82	.80	.77	.75
10.0	1.00	.99	.96	.95	.93	.92	.91	.89	.88	.86	.84	.82	.80
∞	1.00	1.00	.98	.96	.95	.94	.93	.91	.90	.88	.86	.83	.81

Table F-5: Values of K2 for Determining Loss of Head Due to Gradual Enlargement in Pipes, from the Equation: $H_2 = K_2(V_1^2/2g)$

d_2/d_1 = ratio of diameter of larger pipe to diameter of smaller pipe. Angle of cone is twice the angle between the axis of the cone and its side.														
$\underline{d_2}$	Angle of cone													
d_1	2°	4°	6°	8°	10°	15°	20°	25°	30°	35°	40°	45°	50°	60°
1.1	.01	.01	.01	.02	.03	.05	.10	.13	.16	.18	.19	.20	.21	.23
1.2	.02	.02	.02	.03	.04	.09	.16	.21	.25	.29	.31	.33	.35	.37
1.4	.02	.03	.03	.04	.06	.12	.23	.30	.36	.41	.44	.47	.50	.53
1.6	.03	.03	.04	.05	.07	.14	.26	.35	.42	.47	.51	.54	.57	.61
1.8	.03	.04	.04	.05	.07	.15	.28	.37	.44	.50	.54	.58	.61	.65
2.0	.03	.04	.04	.05	.07	.16	.29	.38	.46	.52	.56	.60	.63	.68
2.5	.03	.04	.04	.05	.08	.16	.30	.39	.48	.54	.58	.62	.65	.70
3.0	.03	.04	.04	.05	.08	.16	.31	.40	.48	.55	.59	.63	.66	.71
∞	.03	.04	.05	.06	.08	.16	.31	.40	.49	.56	.60	.64	.67	.72

Table F-6: Values of K3 for Determining Loss of Head Due to Sudden Contraction from the Equation: $H_3 = K_3(V_2^2/2g)$

d_2/d_1 = ratio of larger to smaller diameter												V_2 = velocity in smaller pipe	
$\underline{d_2}$	Velocity, V_2, in feet per second												
d_1	2	3	4	5	6	7	8	10	12	15	20	30	40
1.1	.03	.04	.04	.04	.04	.04	.04	.04	.04	.04	.05	.05	.06
1.2	.07	.07	.07	.07	.07	.07	.07	.08	.08	.08	.09	.10	.11
1.4	.17	.17	.17	.17	.17	.17	.17	.18	.18	.18	.18	.19	.20
1.6	.26	.26	.26	.26	.26	.26	.26	.26	.26	.25	.25	.25	.24
1.8	.34	.34	.34	.34	.34	.34	.33	.33	.32	.32	.31	.29	.27
2.0	.38	.38	.37	.37	.37	.37	.36	.36	.35	.34	.33	.31	.29
2.2	.40	.40	.40	.39	.39	.39	.39	.38	.37	.37	.35	.33	.30
2.5	.42	.42	.42	.41	.41	.41	.40	.40	.39	.38	.37	.34	.31
3.0	.44	.44	.44	.43	.43	.43	.42	.42	.41	.40	.39	.36	.33
4.0	.47	.46	.46	.46	.45	.45	.45	.44	.43	.42	.41	.37	.34
5.0	.48	.48	.47	.47	.47	.46	.46	.45	.45	.44	.42	.38	.35
10.0	.49	.48	.48	.48	.48	.47	.47	.46	.46	.45	.43	.40	.36
∞	.49	.49	.48	.48	.48	.47	.47	.47	.46	.45	.44	.41	.38

Table F-7: Entrance Head Loss Coefficients

Type of Structure and Design of Entrance	Coefficient K_e
• Pipe, Concrete	
Projecting from fill, socket end (groove-end)	0.2
Projecting from fill, sq. cut end	0.5
Headwall or headwall and wingwalls	
Socket end of pipe (groove-end)	0.2
Square-edge	0.5
Rounded (radius = D/12)	0.2
Mitered to conform to fill slope	0.7
* End-Section conforming to fill slope	0.5
Beveled edges, 33.7° or 45° bevels	0.2
Side- or slope-tapered inlet	0.2
• Pipe, or Pipe-Arch, Corrugated Metal	
Projecting from fill (no headwall)	0.9
Headwall or headwall and wingwalls square-edge	0.5
Mitered to conform to fill slope, paved or unpaved slope	0.7
* End-Section conforming to fill slope	0.5
Beveled edges, 33.7° or 45° bevels	0.2
Side- or slope-tapered inlet	0.2
• Box, Reinforced Concrete	
Headwall parallel to embankment (no wingwalls)	
Square-edged on 3 edges	0.5
Rounded on 3 edges to radius of D/12 or B/12 or beveled edges on 3 sides	0.2
Wingwalls at 30° to 75° to barrel	
Square-edged at crown	0.4
Crown edge rounded to radius of D/12 or beveled top edge	0.2
Wingwall at 10° to 25° to barrel	
Square-edged at crown	0.5
Wingwalls parallel (extension of sides)	
Square-edged at crown	0.7
Side- or slope-tapered inlet	0.2

*Note: "End Sections conforming to fill slope," made of either metal or concrete, are the sections commonly available from manufacturers. From limited hydraulic tests they are equivalent in operation to a headwall in both inlet and outlet control. Some end sections, incorporating a closed taper in their design have a superior hydraulic performance. These latter sections can be designed using the information given for the beveled inlet.

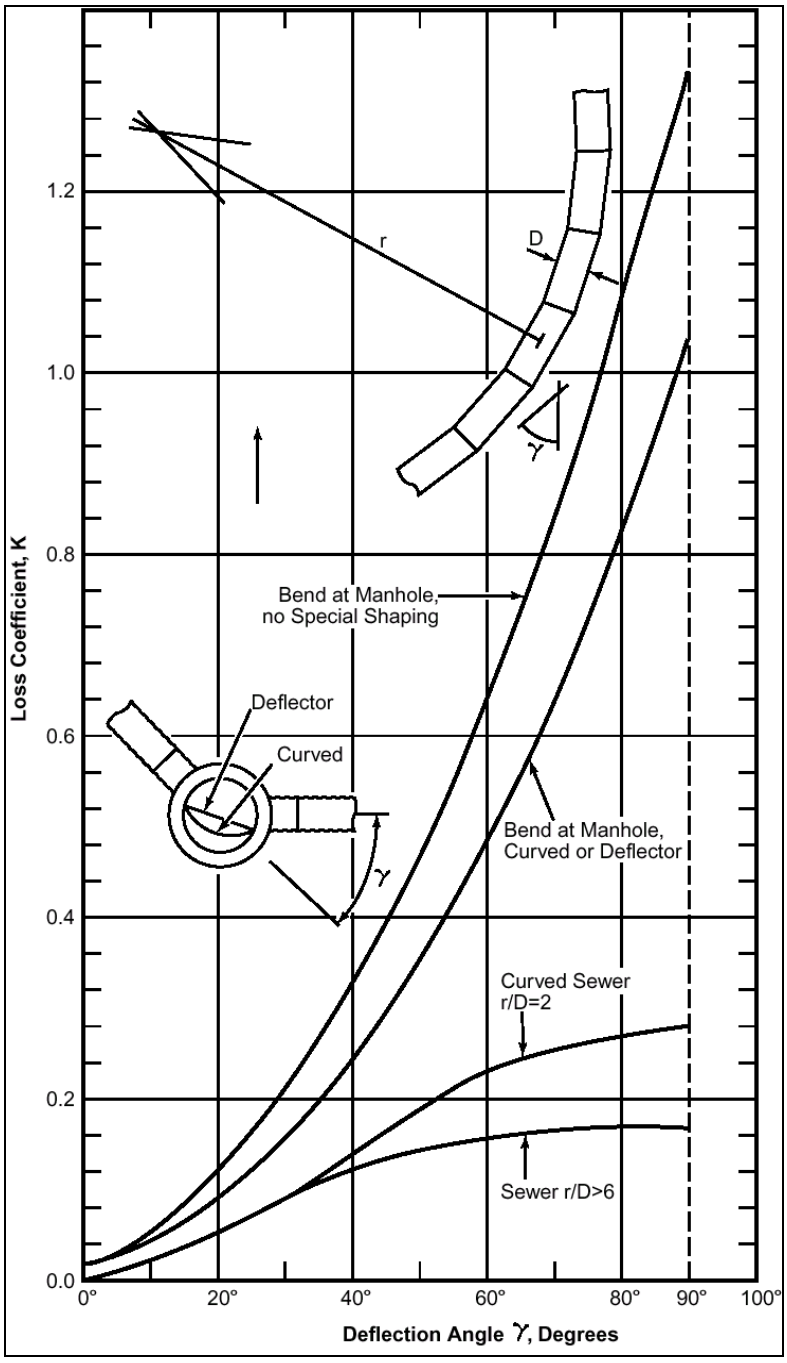


Figure F-1: Bend Head Loss Coefficients

Table F-8: Slopes Required to Maintain Minimum Velocities for Full and Half-Full Flow

Pipe Diameter, Inches	V = 2 fps Slope, Percent				V = 3 fps Slope, Percent			
	n=0.010	n=0.011	n=0.012	n=0.013	n=0.010	n=0.011	n=0.012	n=0.013
8	0.197	0.238	0.284	0.332	0.443	0.536	0.638	0.749
10	0.147	0.178	0.213	0.248	0.329	0.399	0.474	0.557
12	0.115	0.139	0.166	0.194	0.259	0.313	0.372	0.437
15	0.086	0.104	0.123	0.145	0.192	0.232	0.277	0.325
18	0.067	0.081	0.097	0.114	0.151	0.182	0.217	0.255
21	0.055	0.066	0.079	0.092	0.123	0.149	0.177	0.208
24	0.046	0.055	0.066	0.077	0.103	0.124	0.148	0.174
27	0.039	0.047	0.056	0.065	0.088	0.106	0.127	0.149
30	0.034	0.041	0.049	0.057	0.076	0.092	0.110	0.129
33	0.030	0.036	0.043	0.051	0.067	0.081	0.097	0.114
36	0.027	0.032	0.038	0.045	0.060	0.073	0.086	0.101
42	0.022	0.026	0.031	0.036	0.049	0.059	0.070	0.083
48	0.018	0.022	0.026	0.31	0.041	0.049	0.059	0.069
54	0.015	0.019	0.022	0.027	0.035	0.042	0.050	0.059
60	0.013	0.016	0.019	0.023	0.030	0.037	0.044	0.051
66	0.012	0.014	0.017	0.020	0.027	0.032	0.039	0.045
72	0.011	0.013	0.015	0.018	0.024	0.029	0.034	0.040
78	0.010	0.011	0.014	0.016	0.021	0.026	0.031	0.036
84	0.009	0.010	0.012	0.015	0.019	0.024	0.028	0.033
90	0.008	0.010	0.011	0.013	0.018	0.021	0.026	0.030
96	0.007	0.009	0.010	0.012	0.016	0.020	0.023	0.027
102	0.007	0.008	0.010	0.011	0.015	0.018	0.022	0.025
108	0.006	0.007	0.009	0.010	0.014	0.017	0.020	0.024
114	0.006	0.007	0.008	0.010	0.013	0.016	0.019	0.022
120	0.005	0.006	0.008	0.009	0.012	0.015	0.017	0.020
126	0.005	0.006	0.007	0.008	0.011	0.014	0.016	0.019
132	0.004	0.006	0.007	0.008	0.011	0.013	0.015	0.018
138	0.004	0.005	0.006	0.007	0.010	0.012	0.014	0.017
144	0.004	0.005	0.006	0.007	0.009	0.011	0.014	0.016

G. APPENDIX G

Supplemental Standard Details

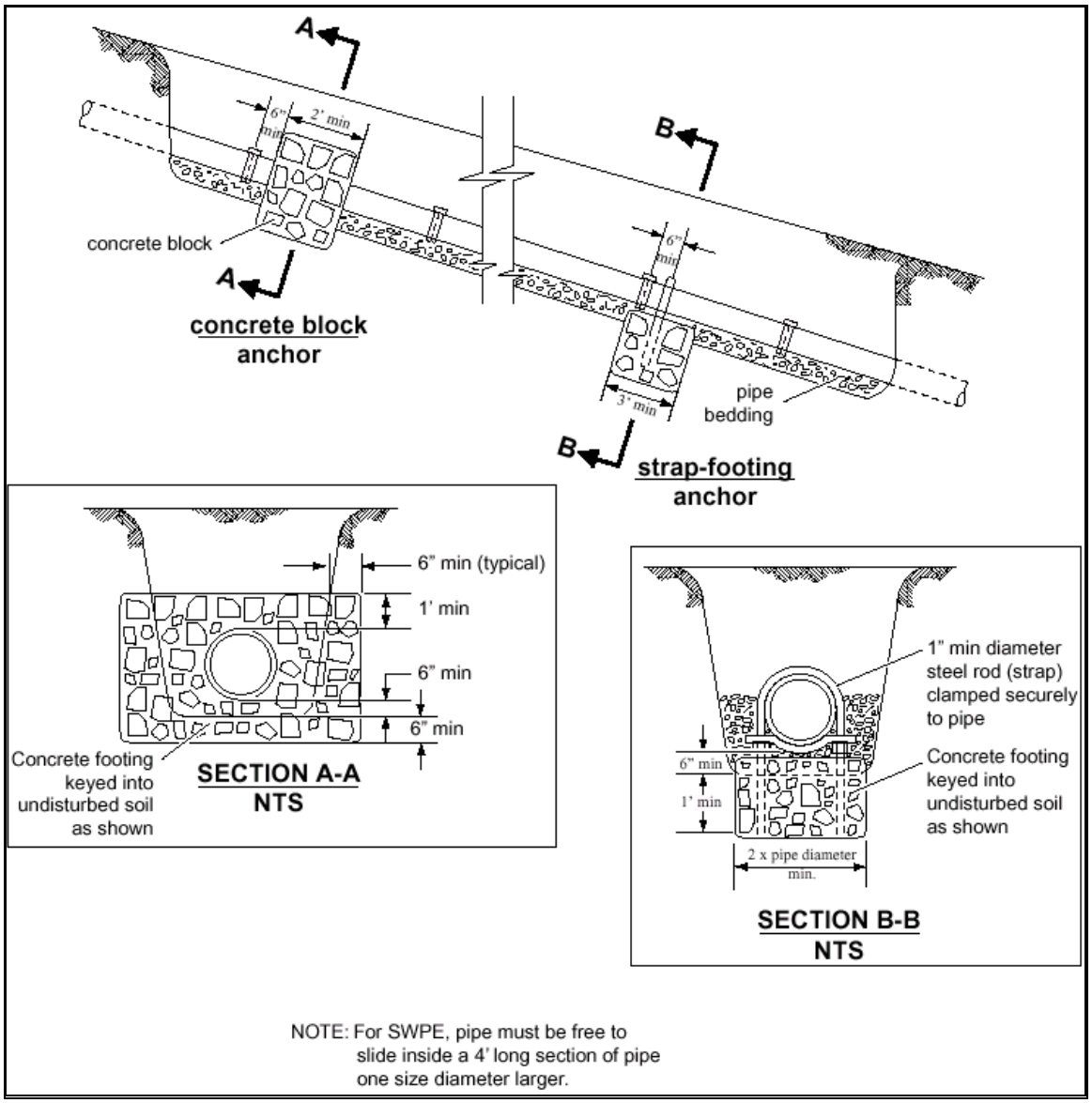


Figure G-1: Pipe Anchor Detail 1

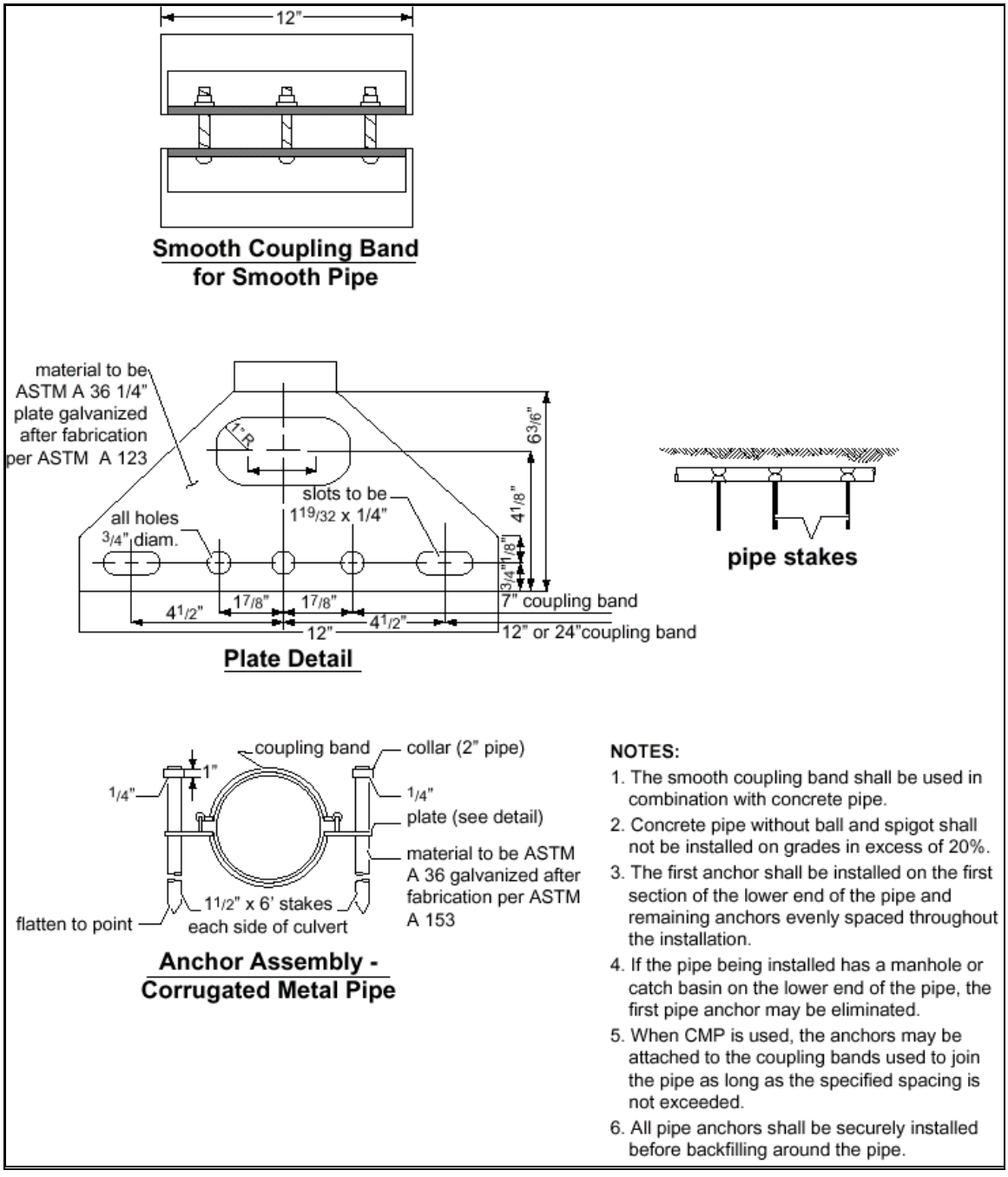


Figure G-2: Pipe Anchor Detail 2

H. APPENDIX H

Nomographs

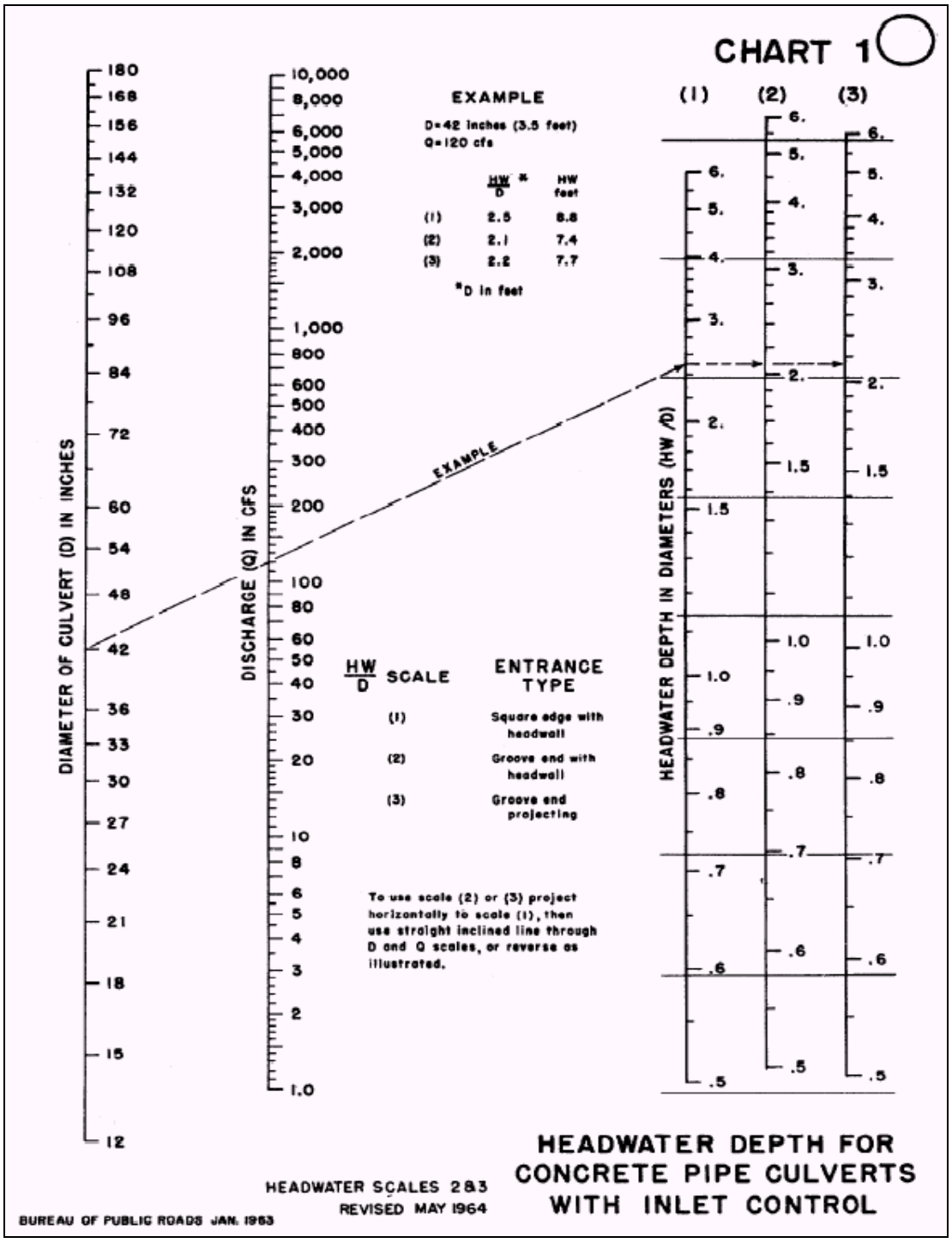


Figure H-1: Concrete Pipe Inlet Control Nomograph

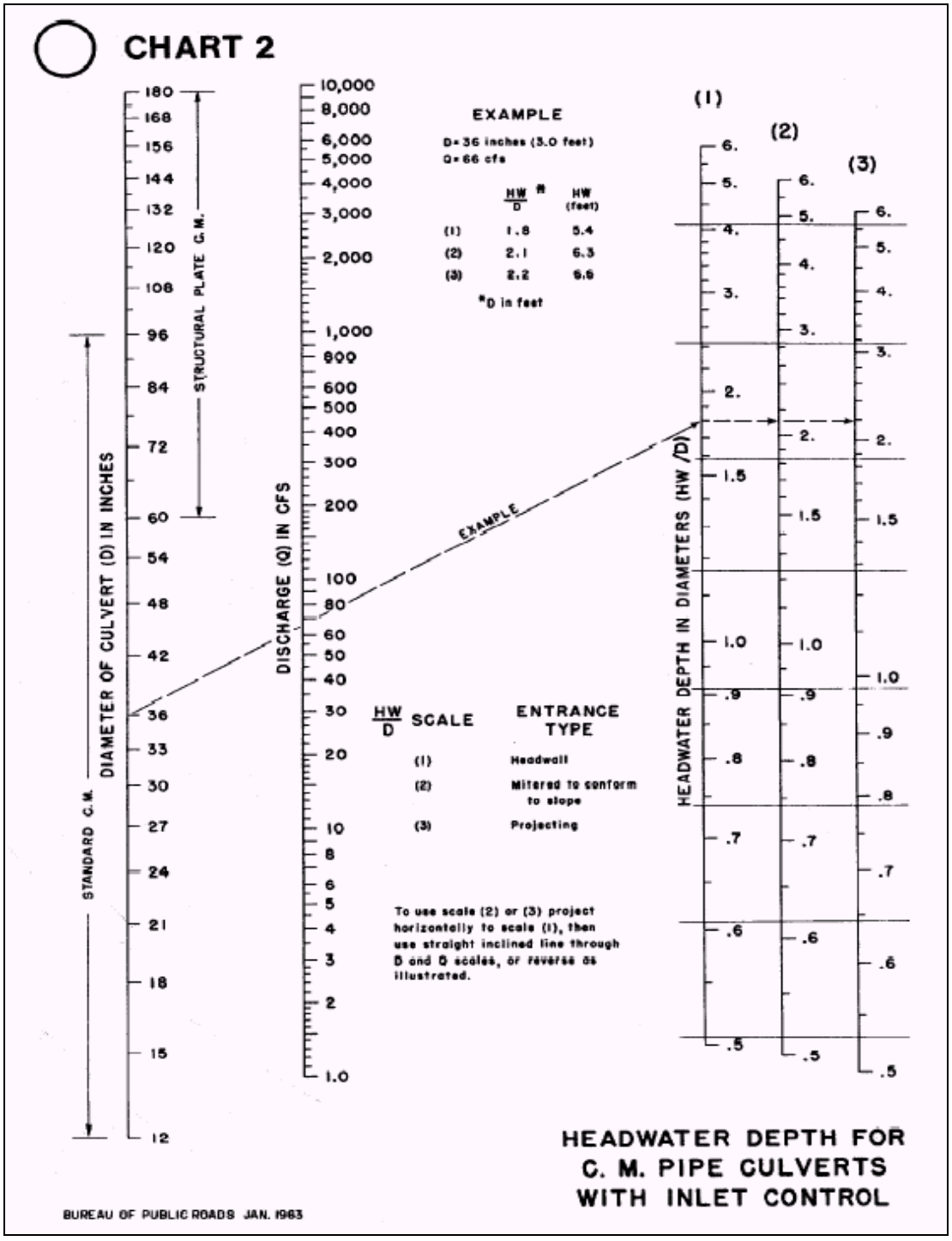


Figure H-2: Corrugated Metal Pipe Inlet Control Nomograph

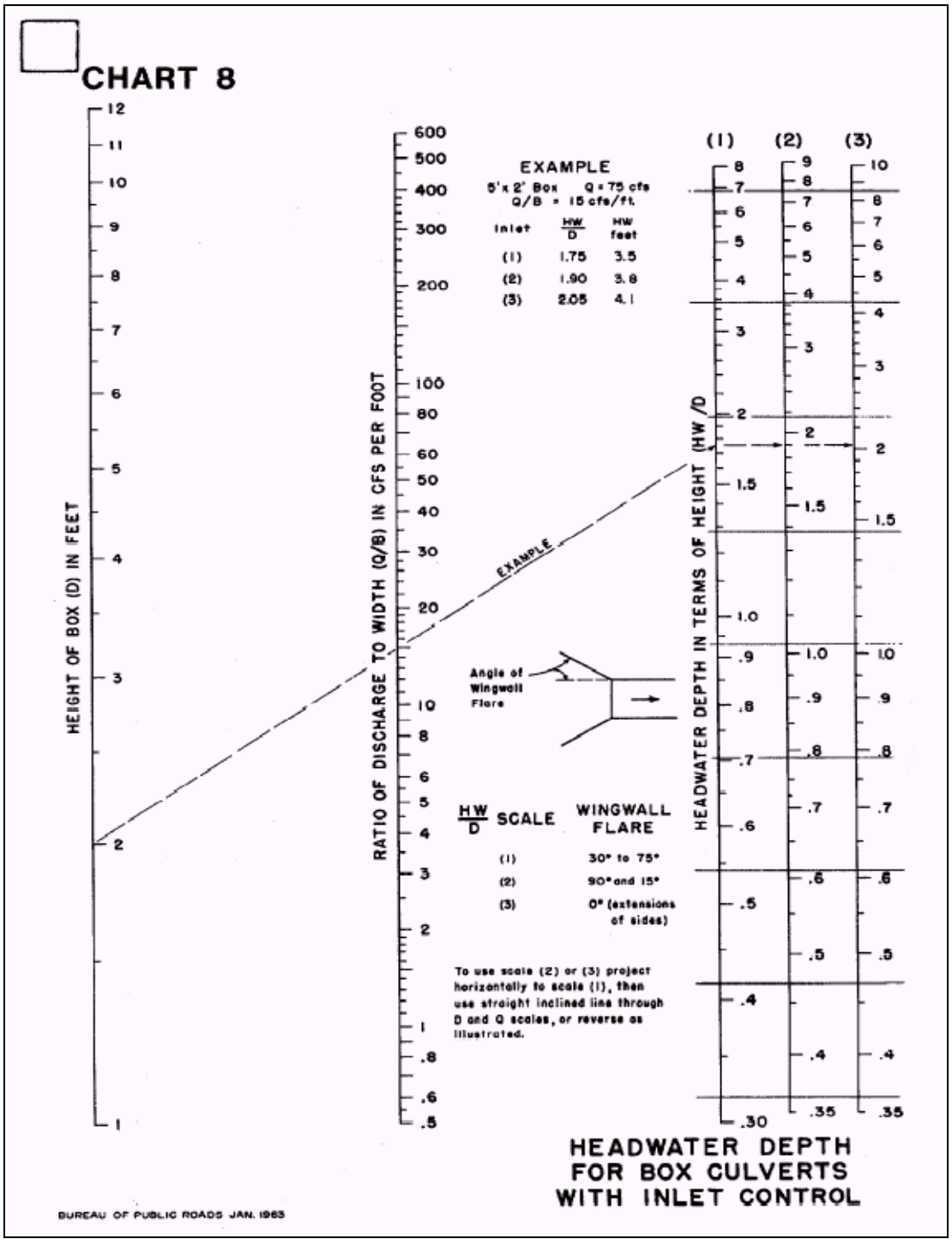


Figure H-3: Box Culvert Inlet Control Nomograph

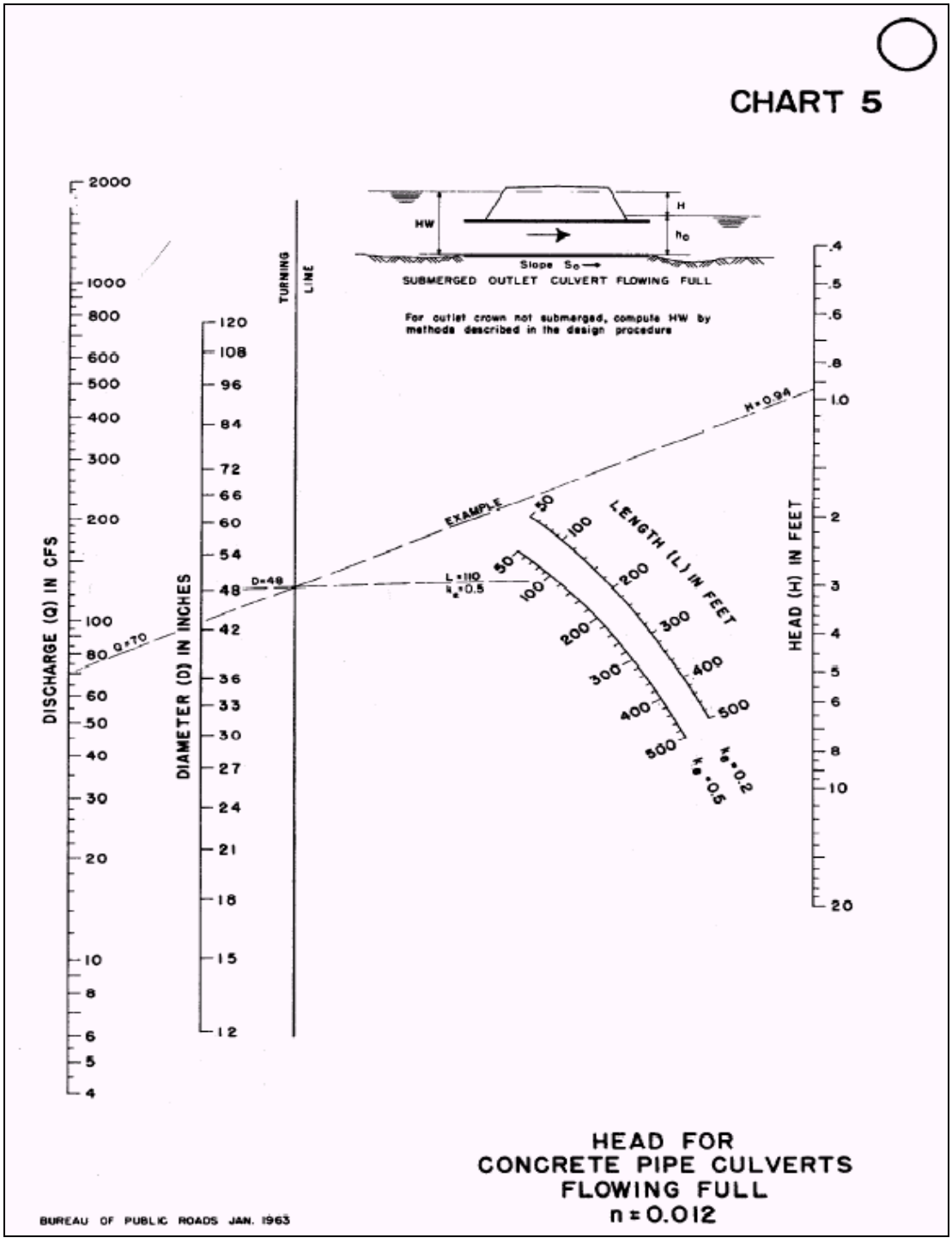


Figure H-4: Concrete Pipe Outlet Control Nomograph

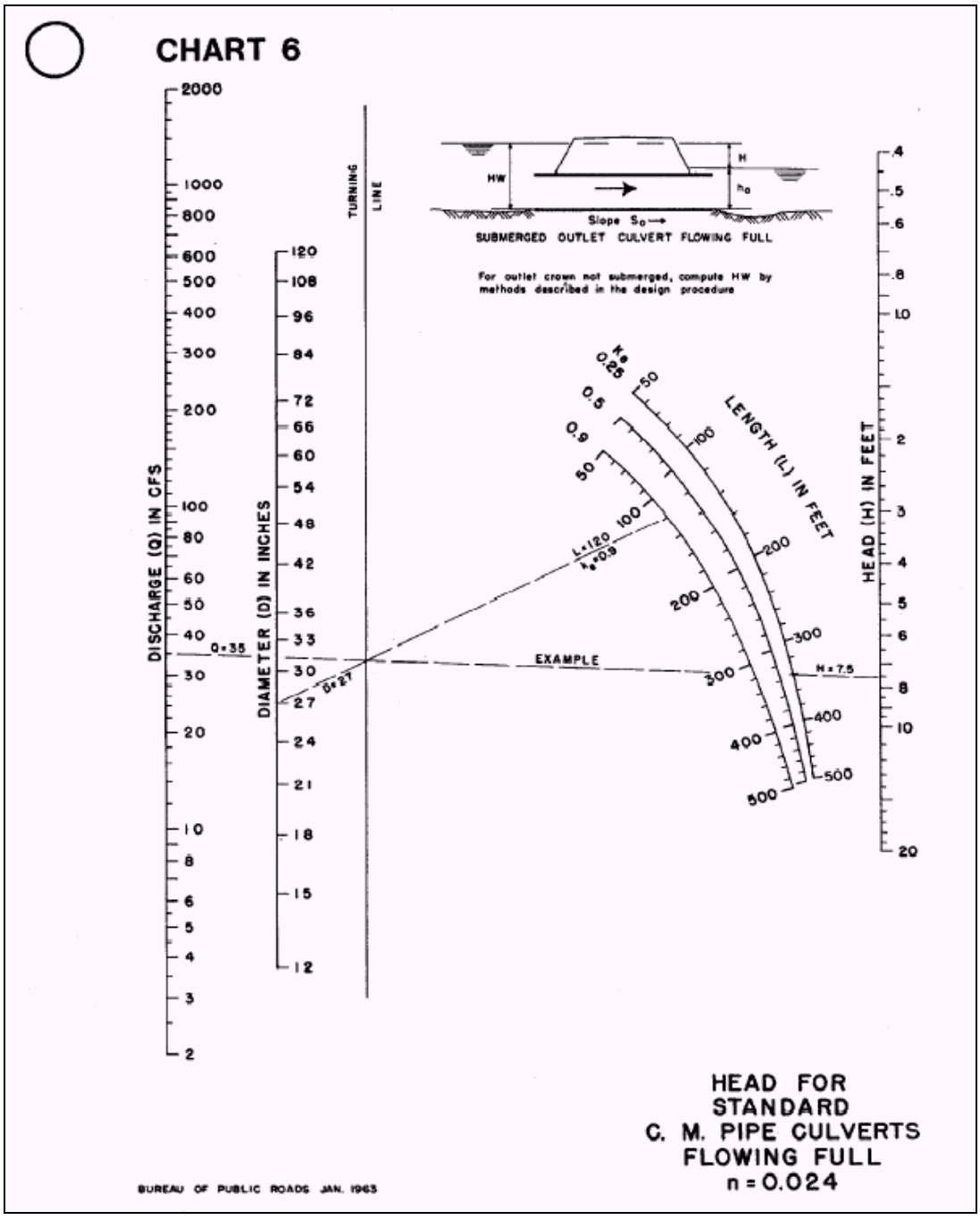


Figure H-5: Corrugated Metal Pipe Outlet Control Nomograph

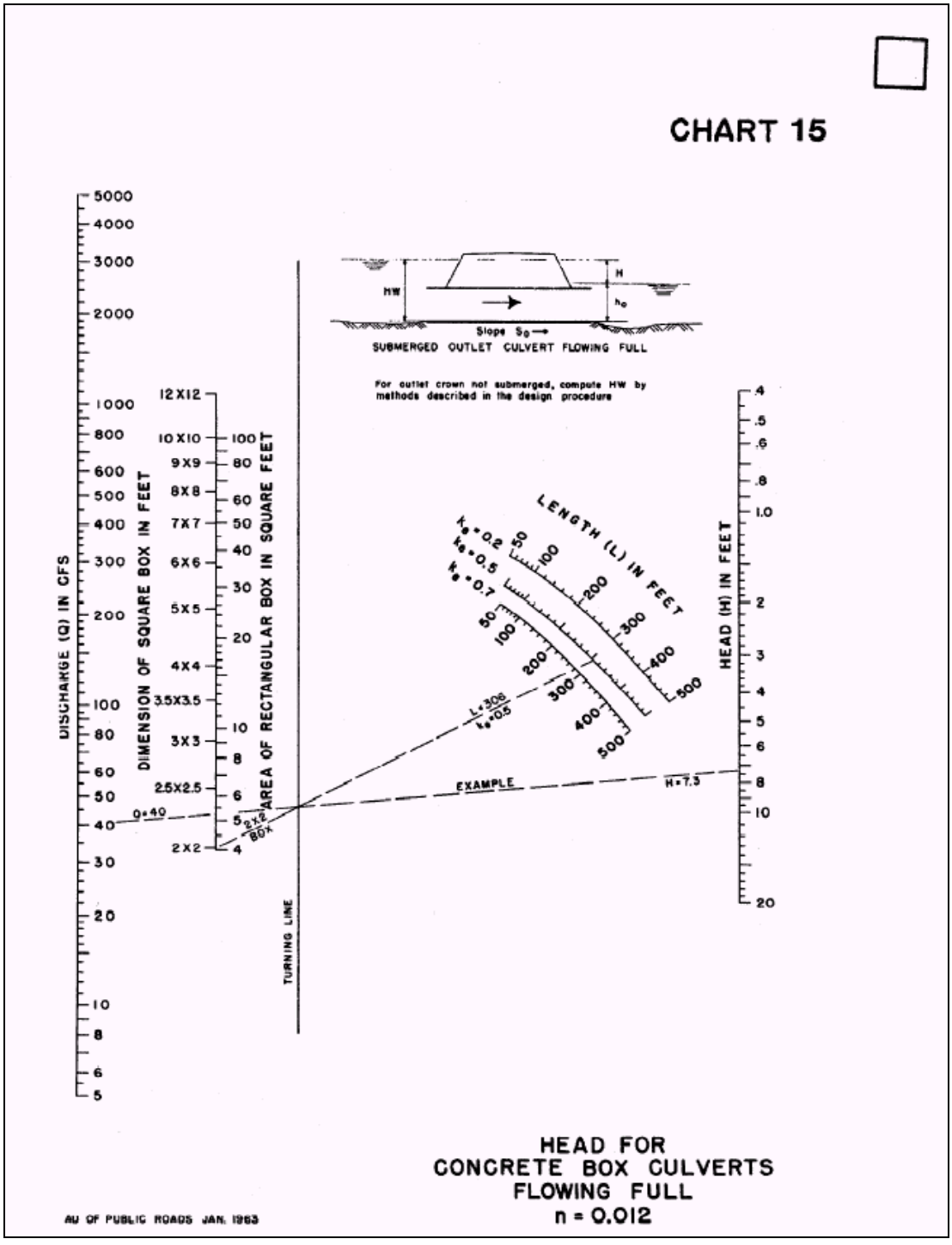


Figure H-6: Box Culvert Outlet Control Nomograph

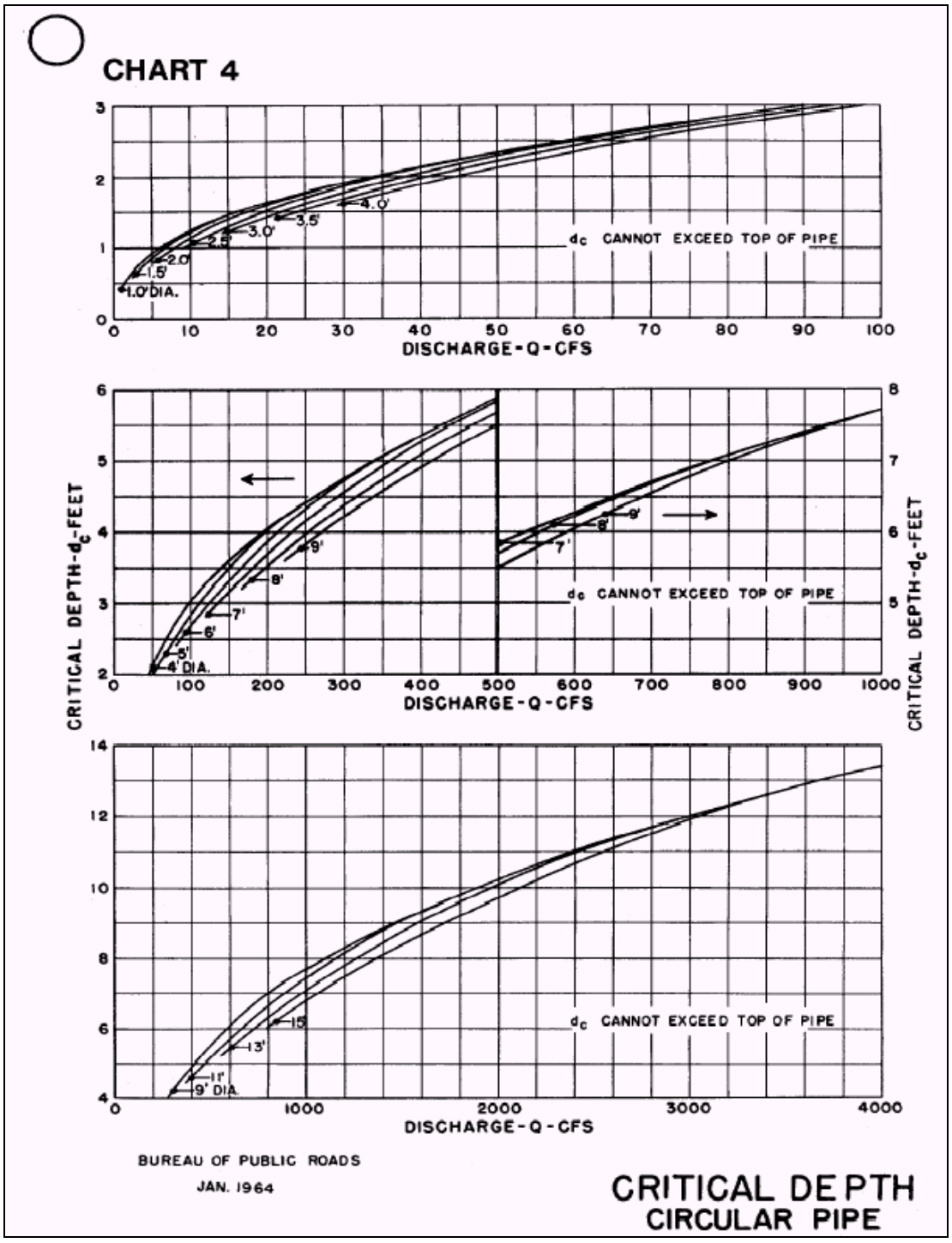


Figure H-7: Critical Depth for Circular Pipe

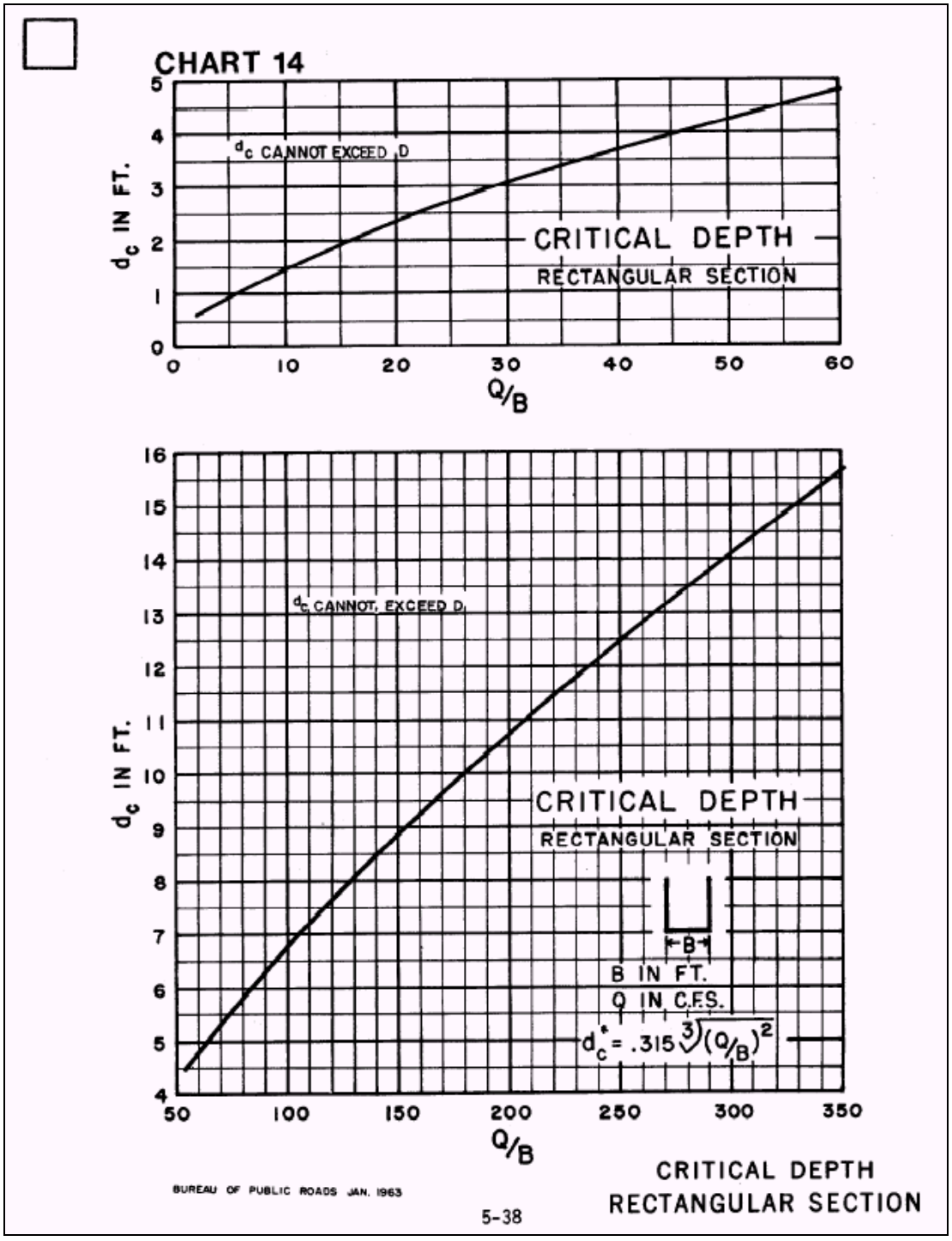


Figure H-8: Critical Depth for Rectangular Channel

I. APPENDIX I

Tidal Data

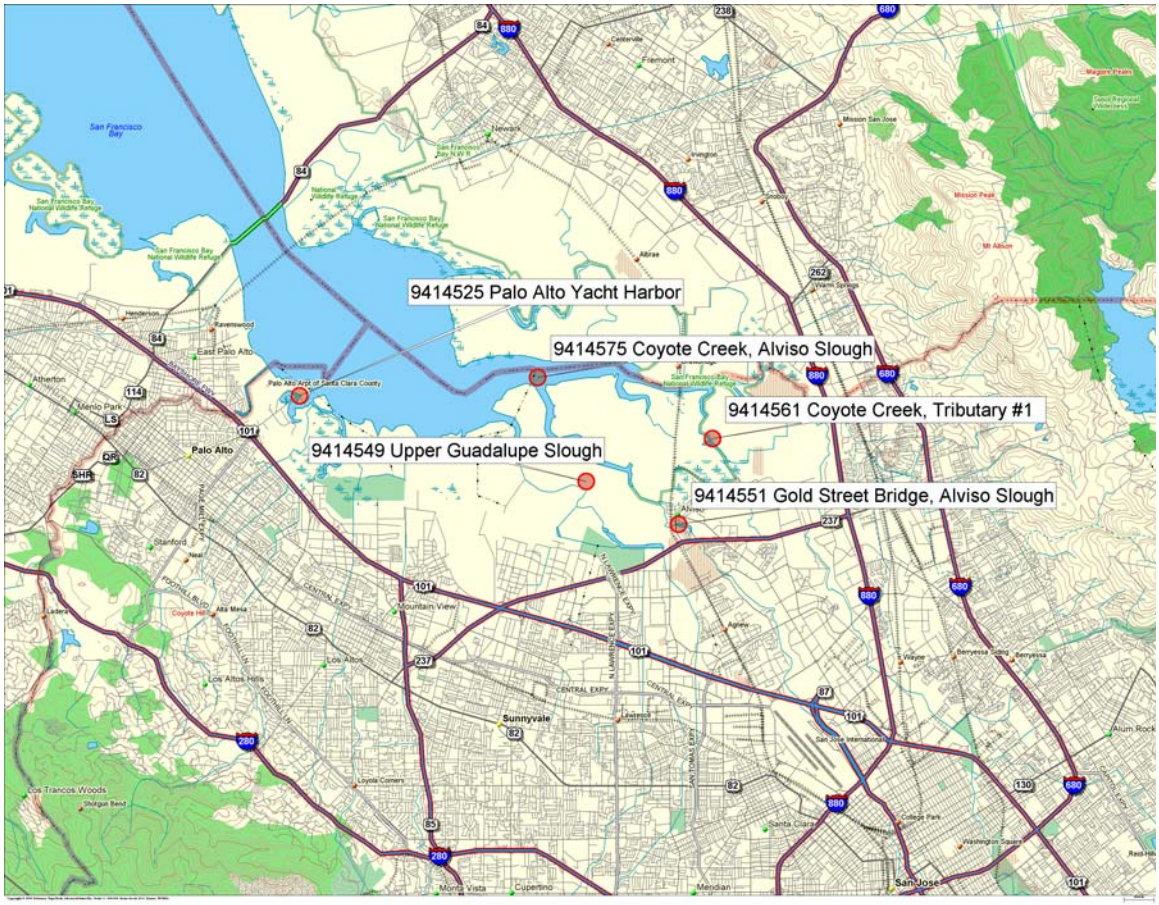


Figure I-1: National Oceanic and Atmospheric Administration (NOAA) Tidal Benchmarks in Santa Clara County

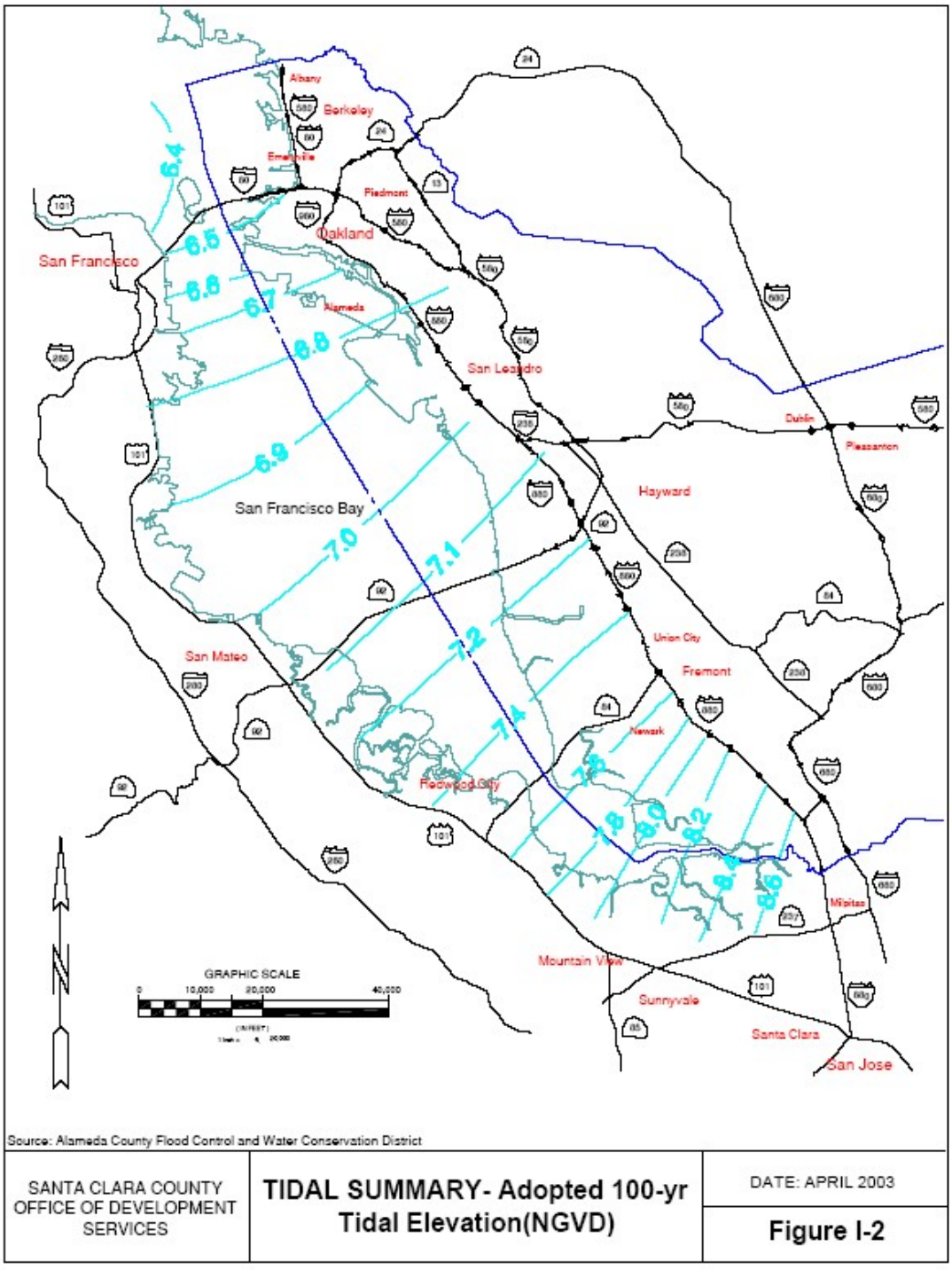


Figure I-2: Tidal Summary - Adopted 100-Year Tidal Elevation (NGVD)

Table I-1: NHHW and Adopted 100-Year Tide Data

		MHHW		***Adopted 100-Year Tide Elevation***
ID	*Bench Mark*	MLLW Datum	NGVD	NGVD
9414290	***San Francisco (Reference Station)***	*5.7*	*2.9*	*6.0*
9414525	***Palo Alto Yacht Harbor***	*8.3*	*4.3*	*7.8*
9414549	***Upper Guadalupe Slough***	*9.2*	*4.7*	*8.5*
9414551	***Gold Street Bridge, Alviso Slough***	*9.2*	*4.9*	*8.6*
9414561	***Coyote Creek, Tributary #1***	*8.4*	*4.0*	*8.6*
9414575	***Coyote Creek, Alviso Slough***	*8.9*	*4.6*	*8.2*

Source: U. S. Army Corps of Engineers, San Francisco District, San Francisco Bay Tidal Study, June 1984
all elevations in feet

Table I-2: Location and Datum Conversion Factors for NOAA Tidal Benchmarks in Santa Clara County

		LOCATION CONVERSION FACTORS FOR HHW	***DATUM CONVERSION FACTORS***		
		San Francisco (Reference Station) → Secondary Station	*MLLW Datum → NGVD*	*NGVD → NAVD*	*NGVD → NAVD*
ID	*Station*	*(ft or multiplier)*	*(ft)*	*(m)*	*(ft)*
9414525	***Palo Alto Yacht Harbor***	** 1.32*	*-4.0*	*0.819*	*2.687*
9414549	***Upper Guadalupe Slough***	** 1.66*	*-4.5*	*0.821*	*2.694*
9414551	***Gold Street Bridge, Alviso Slough***	** 1.66*	*-4.3*	*0.824*	*2.703*
9414561	***Coyote Creek, Tributary #1***	*+ 2.60*	*-4.5*	*0.824*	*2.703*
9414575	***Coyote Creek, Alviso Slough***	** 1.60*	*-4.4*	*0.820*	*2.690*

J. APPENDIX J

Supplemental Standard Details

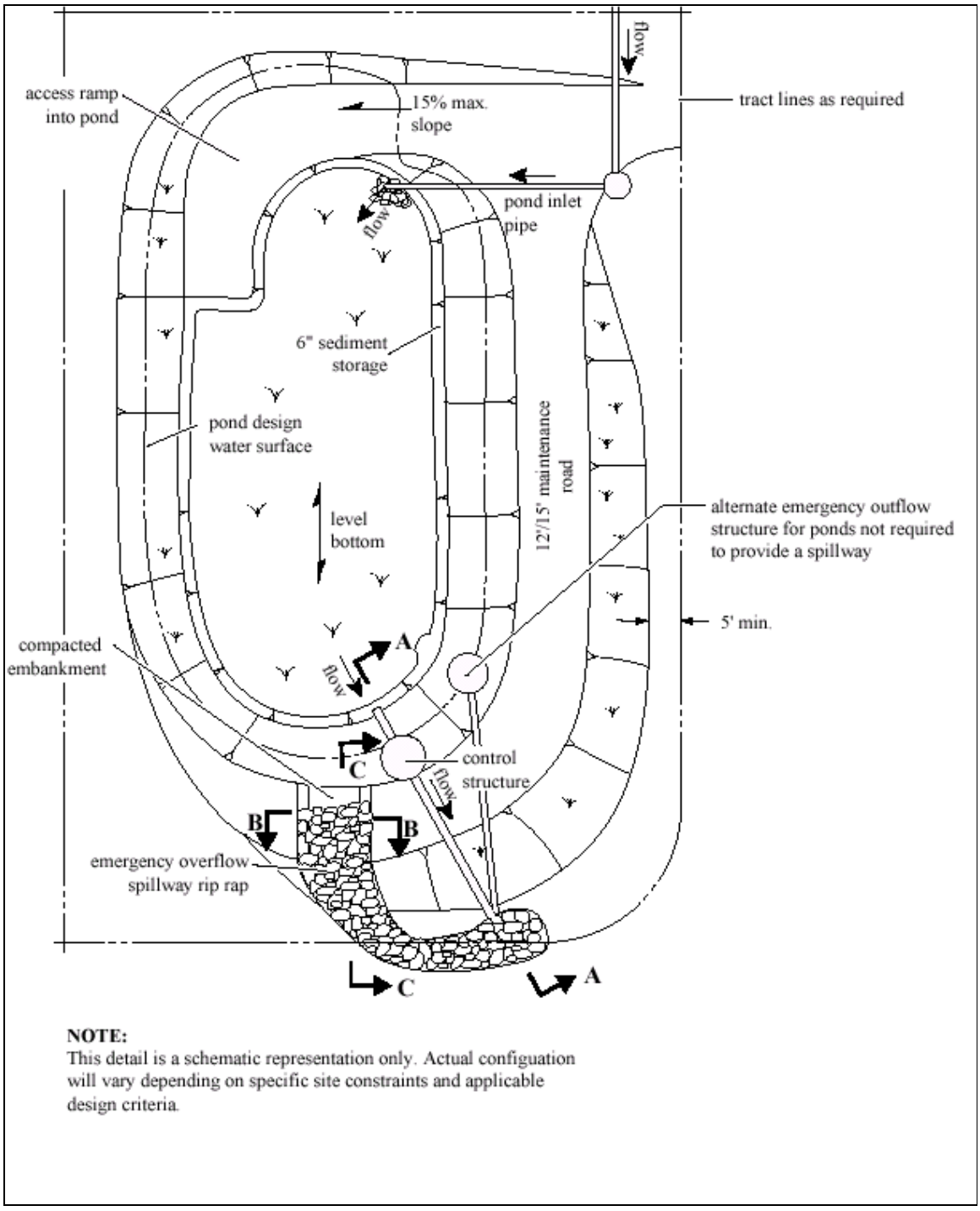


Figure J-1: Typical Detention Pond

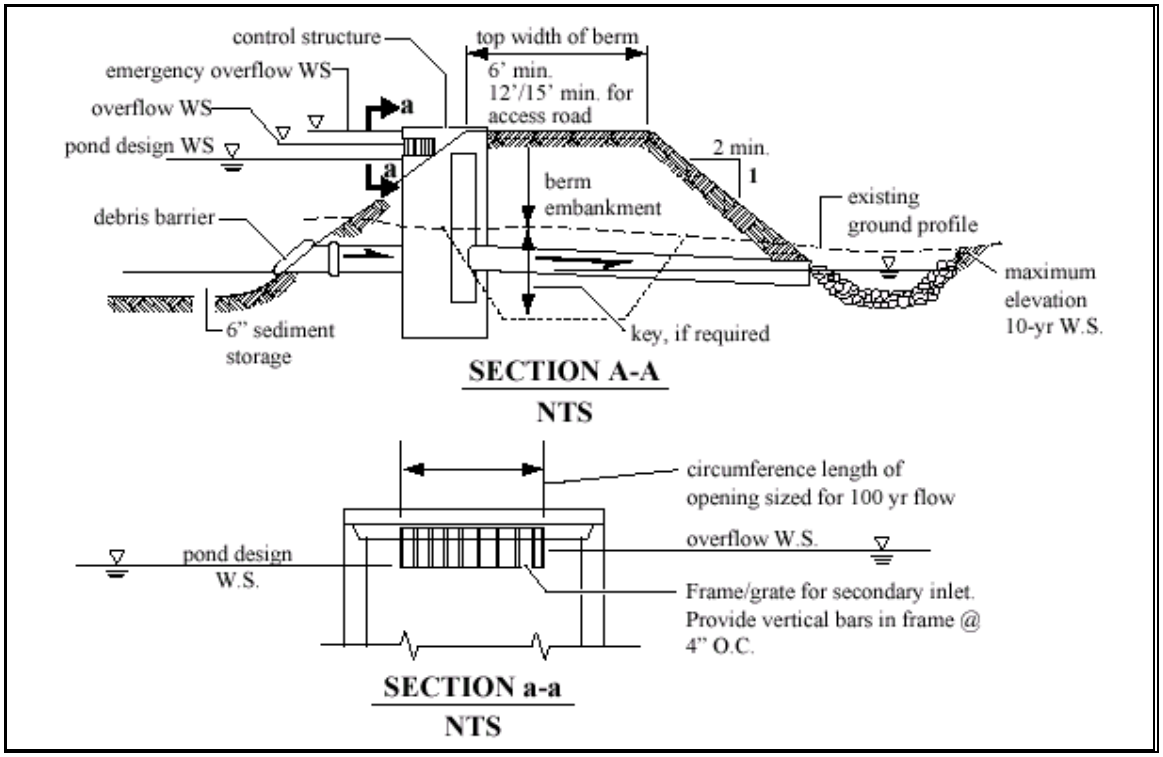


Figure J-2: Typical Detention Pond Section (Outlet Structures)

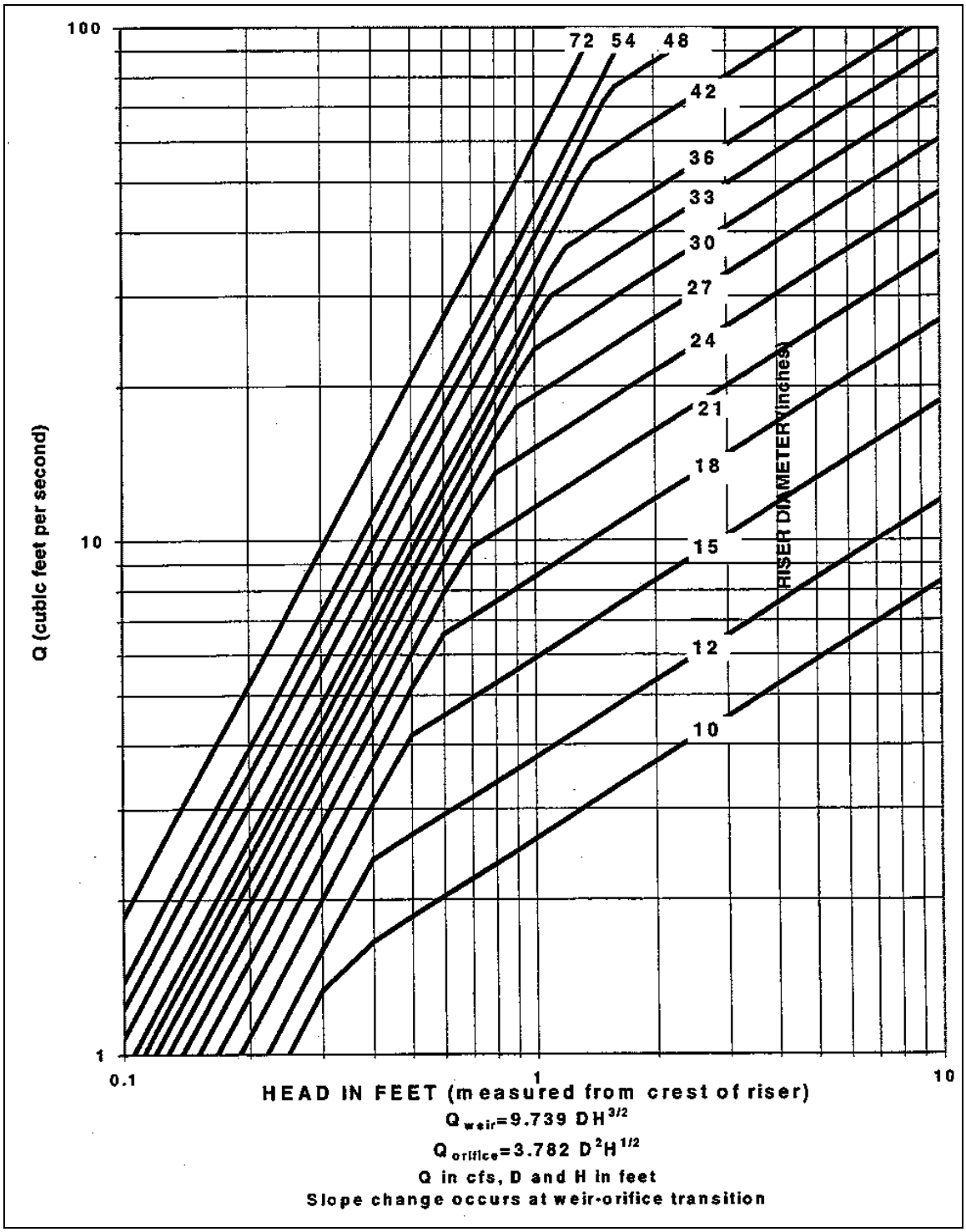


Figure J-3: Riser Inflow Curves

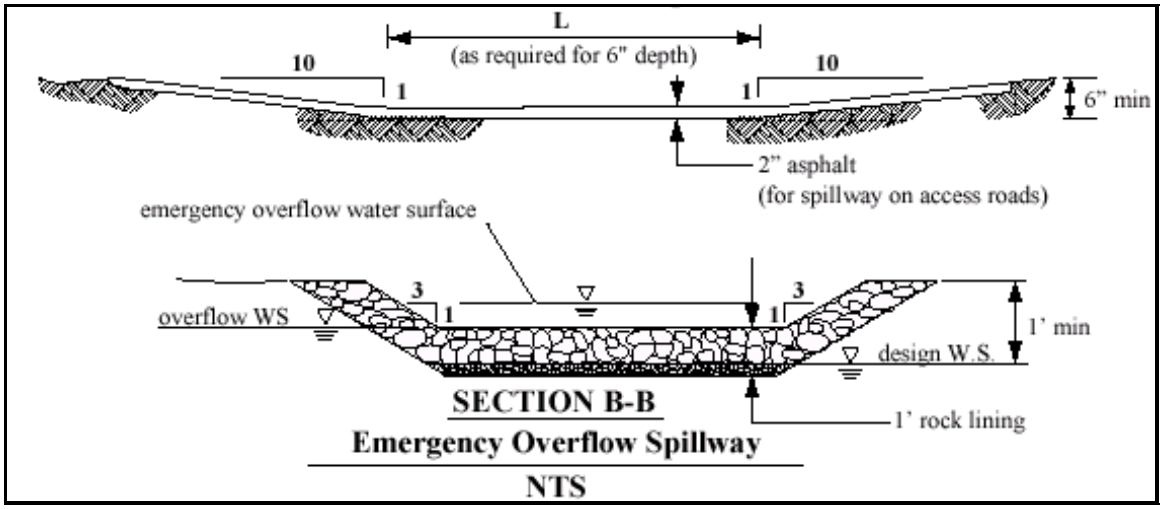


Figure J-4: Typical Detention Pond Section (Emergency Overflow Spillway) 2 Options

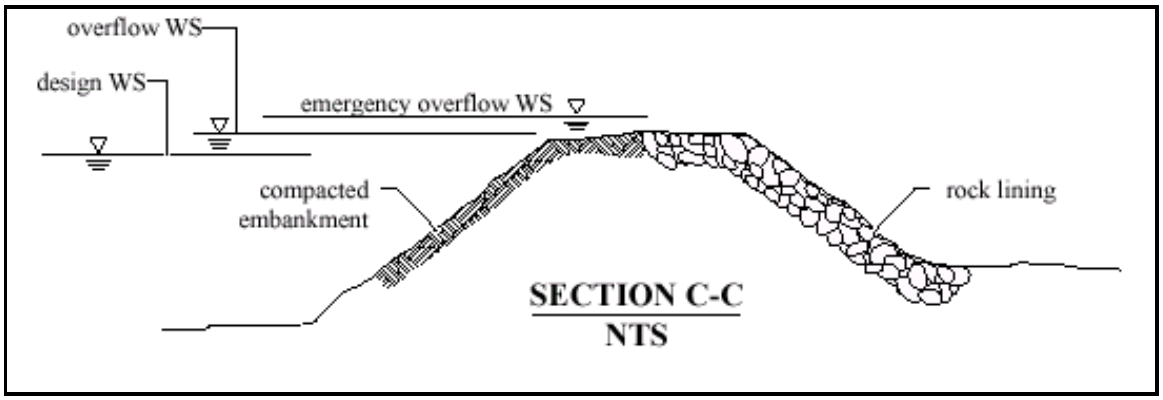


Figure J-5: Typical Detention Pond Section (Emergency Overflow Spillway)

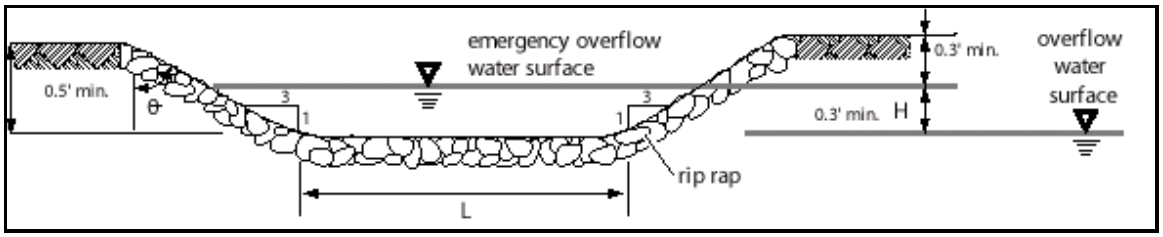


Figure J-6: Weir Section for Emergency Overflow Spillway

K. APPENDIX K

APWA Modified Rational Method
(used with permission)

Several examples of the computation of runoff flow rates and runoff volumes as applied to the design of stormwater detention facilities are given in the following pages to illustrate the use of various methods. Although examples are included that embody the use of the basic Rational Formula, it is not intended to suggest, encourage or perpetuate the use of this method. Such examples are included because they are typical of the design practices of personnel of many engineering firms and public agencies across the country.

Examples are given of the Modified Rational Method Analysis (used in Madison, Wisconsin, and in other places); a method developed by Dr. Ven Te Chow for particular application in the State of Illinois; and the Colorado Urban Hydrograph Procedure.

Modified Rational Method Analysis:[3] The term Modified Rational Method Analysis refers to a procedure for manipulating the basic Rational Method techniques to reflect the fact that storms with durations greater than the normal time of concentration for a basin will result in a larger volume of runoff even though the peak discharge is reduced. This greater volume of runoff produced by longer duration storms must be analyzed to determine the correct sizing for detention facilities.

Many limitations and shortcomings in the assumptions behind this method are evident. The approach becomes more valid on progressively smaller basins, eventually reaching a size so small that watershed modeling is approached. The procedure should, therefore, be limited to relatively small areas such as rooftops, parking lots, or other upstream areas with tributary basins less than 20 acres. This would minimize major damage which could result from overtopping or failure of the proposed detention facility.

Figure 9, Modified Rational Method Hydrographs, presents a family of curves for a theoretical basin. These hydrographs are developed by using the basic Rational Method assumptions of constant rainfall intensity (i) time of Concentration (T_c) from the most distant point, timewise, and the coefficient of runoff (C). The typical Rational Formula

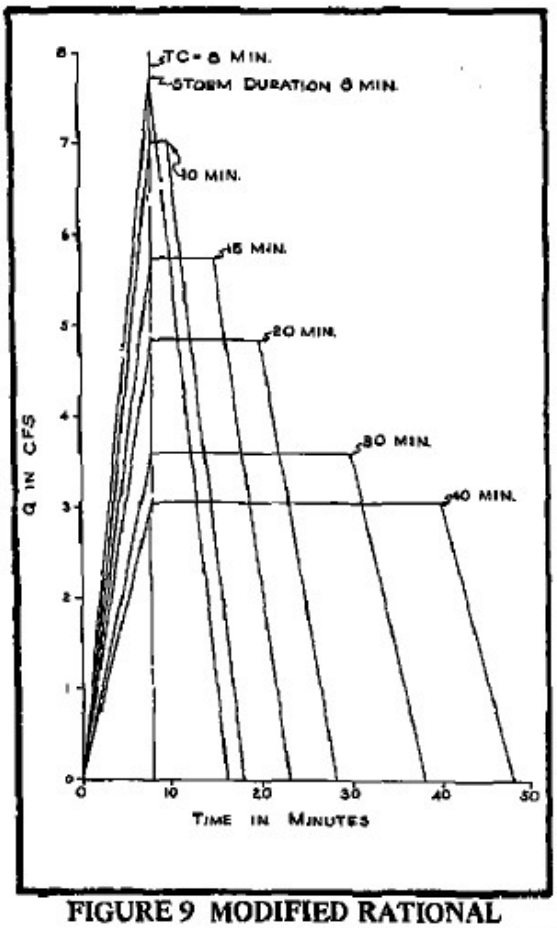


FIGURE 9 MODIFIED RATIONAL METHOD HYDROGRAPHS

hydrograph with the peak discharge coinciding with the time of concentration for the basin (T_c), is first calculated using the normal formula Q = CiA. Following this, a family of hydrographs representing storms of greater duration are developed. The peak runoff rate for each hydrograph is equal to CiA where (i) is the rainfall intensity for the storm duration in question. The rising limb and falling limb of the hydrograph are, in each case, equal to (T_c) for the basin. The basic assumption of this method is that the area under the assumed trapazoidal hydrograph equals the volume of runoff from the theoretical rainfall. The area under the hydrograph is also equal to the peak discharge rate for that particular rainfall times the duration of the rainfall.

The following example presents the calculation method for a typical two-acre basin.

Example No. 1

Given

Area: A = 2.0 acres

Type of development: commercial parking lot, fully paved, C = 0.9

Design rainfall frequency: five-year

Rainfall time–intensity–frequency curves: as indicated in Figure 10, Rainfall Time–Intensity–Frequency Chart

Time of Concentration: T_c = 8 minutes

Required:

Develop family of curves representing Modified Rational Method hydrographs for the 8, 10,15,20,30 and 40 minute rainfall durations.

Rainfall Duration (minutes)	Rainfall Intensity (in./hr.)	Peak Runoff Rate (cfs)
8	4.3	7.74
10	3.9	7.02
15	3.2	5.76
20	2.7	4.86
30	2.0	3.60
40	1.7	3.06

Answer: The resulting storm hydrographs are depicted in Figure 9.

It is recommended that a coefficient be added to the Rational Method to account for antecedent precipitation conditions for major storms with recurrence intervals greater than 25 years. Table 13, Recommended Antecedent Precipitation Factors, presents a set of recommended coefficients. Under these conditions, the Rational Formula becomes $Q = CC_aiA$. Although this approach does not totally reconcile the difficulties in representing volume of runoff by the Rational Method, it does attempt to predict more realistic hydrograph volumes characteristic of the higher frequency storms.

TABLE 13
Recommended Antecedent Precipitation Factors for the Rational Formula

Recurrence Interval (Years)	C_a
2 to 10	1.0
25	1.1
50	1.2
100	1.25

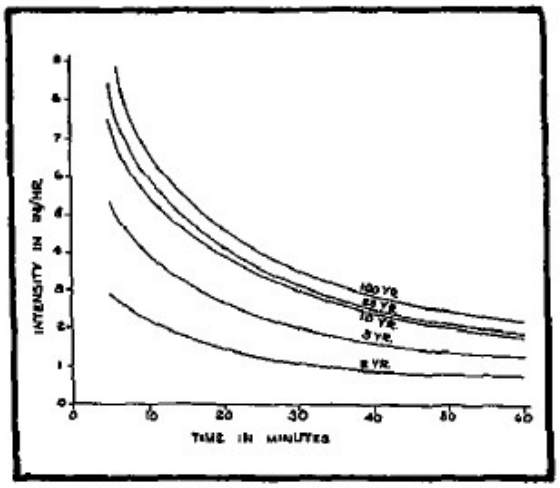


FIGURE 10 RAINFALL TIME-INTENSITY-FREQUENCY CHART

The next step in determining the necessary storage volume for the detention facility is to (1) set a release rate and determine the volume of storage necessary to accomplish this release rate or (2) determine the amount of stormwater storage volume available on the site and then determine the minimum release rate required so as to not exceed the storage volume. The first possibility, that of determining necessary storage volume when a pre-determined release rate is selected, will be dealt with first.

To determine the storage volume required, a reservoir routing procedure should be accomplished on each of the hydrographs, with the critical storm duration and required volume being determined. The importance of the particular project should govern the type of routing utilized. For small areas requiring repetitive calculations, such as in bays of a parking lot, an assumed release curve is normally satisfactory. For larger areas, such as a pond in a small park area with 20 acres of tributary area, a simple reservoir routing procedure would be in order.

Figure 11, Typical Example of On-Site Detention – Rational Method Analysis, represents the method actually utilized by Frasier & Gingery, Inc., for small area

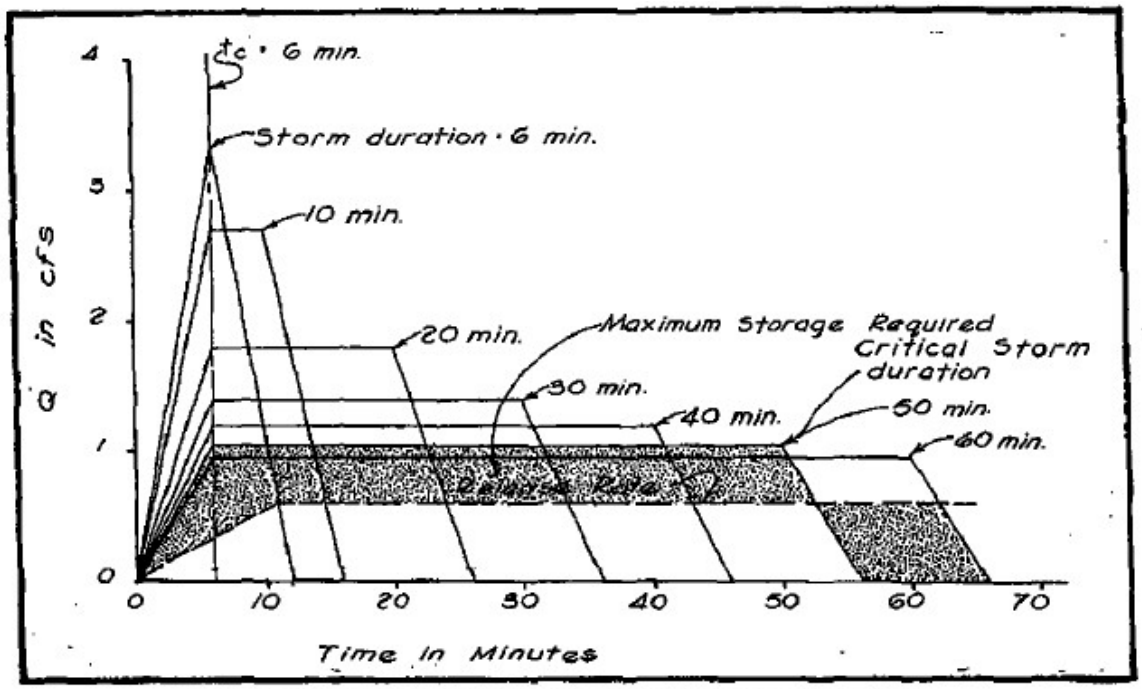


FIGURE 11 TYPICAL EXAMPLE OF ON-SITE DETENTION RATIONAL METHOD ANALYSIS

detention analyses. The assumed release curve approximates a formal reservoir routing in much the same way the Rational Method Hydrograph approximates a true storm hydrograph. The curve allows for the low release rate at the beginning of a storm and an increasing release rate as the storage volume increases. In normal flood routing, the maximum release rate will always occur at the point where the outflow hydrograph crosses the receding limb of the inflow hydrograph. For this reason, the design release rate is forced to coincide with that point on the falling limb of the hydrograph resulting from the storm of duration equal to the time of concentration for the basin. The release rate is held constant past this point. The critical storage volume is then found by determining the area between the inflow and release hydrographs. Example No. 2 continues the calculations initiated in Example No. 1 to determine the required storage volume.

Example No. 2

Given:

Drainage basin and other hydrologic information presented in Example No. 1

Allowable release rate: Q = 2.5 cfs

Required:

Determine the critical storage volume

Storm Duration (Minutes)	Storm Runoff Volume (cu ft)	Release Flow Volume (cu ft)	Required Storage Volume (cu ft)
8	3710	1200	2510
10	4206	1500	2706
15	5184	2250	2934
20	5820	3000	2820
30	6480	4500	1980
40	7344	6000	1344

The critical storage volume is then 2,934 cubic feet occurring for a 15-minute rainfall duration, or time of concentration.

Many cities and counties in the Denver Metropolitan Area require that the release rate be held to the historic level for a particular rainfall frequency. Under these circumstances the release rate is calculated utilizing the Rational Formula with a *C* value characteristic of the undeveloped conditions

and an *i* value for a storm duration equal to the time of concentration for the basin under historic development. Also, the developed basin is oftentimes different from the historic basin requiring the determination of the historic basin area for the calculation of the historic flow rate.

Because this approach is used regularly by Frasier & Gingery, Inc., for the design of on-site detention facilities, where as many as a dozen storage bays may exist within a 10-acre site, it has been programmed on an IBM 1130 computer to assist in the completion of the repetitive calculations necessary in the analysis. A sample computer output sheet for the program which condenses all previous long-hand calculations into a simple computer operation is as follows:

HISTORIC C	= 0.60	COMPOSITE C	= 0.90
HISTORIC TC	= 0. Min	DEVELOPED TC	= 8. Min
ROOF AREA	= 0.00 AC	ROOF REL. RATE	= 0.00 In/Hr
TOTAL AREA	= 2.00 AC	STORM FREQ.	= 100 Year

DURATION (Min)	INTENSITY (In/Hr)	PEAK Q (CFS)	STORM VOL (Cu Ft)	RELEASE VOL (Cu Ft)	STORAGE (Cu Ft)
8.	4.10	7.73	3,715.19	1,200.00	2,515.15
10.	3.90	7.01	4,212.60	1,500.00	2,711.99
15.	3.20	5.76	5,184.00	2,256.00	2,933.99
20.	2.70	4.35	5,831.99	3,000.00	2,831.99

MAX STORAGE	=	2,933.99 Cu Ft
CRITICAL DURATION	=	15. Min
RELEASE RATE	=	2.50 CFS

Many limitations and shortcomings in the assumptions behind this method are evident. The approach becomes more valid on progressively smaller basins eventually reaching a size so small that watershed modeling is approached. The procedure should, therefore, be limited to relatively small areas where no major damage would result from over-topping or failure of the proposed detention facility.

Method Developed by Ven Te Chow: A case study of the design of an existing detention pond located (on ground surface) in northeastern Illinois is presented to illustrate the application of a design method developed by Dr. Ven Te Chow. The method is described in detail in a 1962 publication of the University of Illinois (Urbana) titled *Hydrologic Determination of Waterway Areas for the Design of Drainage Structures in Small Drainage Basins,* Engineering Experiment Station Bulletin No. 462.

The publication describes the development and application of a scientific, simple, practical method to determine the peak discharge rate from small rural drainage basins. The method was developed for application to the design of waterway openings of minor drainage structures such as culverts and small bridges. For practical applications of the method, a design chart for climatic and physiographic conditions characteristic of Illinois is included in Bulletin No. 462.

The method was derived utilizing the concept of unit hydrographs and is based upon unit hydrograph synthesis. Reference to Bulletin No. 462 is suggested for those readers who wish to follow the example in detail.

Dr. Chow's formula for determining the direct peak discharge from a drainage basin is computed as a product of the rainfall excess and the peak discharge of a unit hydrograph. The derived formula for the direct peak discharge, Q, is given as:

$$Q = AXYZ$$

where

- Q = Peak Discharge Rate (cfs)
- A = Area of Drainage Basin (acres)
- X = Runoff Factor, determined by the design rainfall duration and frequency and the soil type, cover and surface condition (values are obtained from curves in Bulletin No. 462)
- Y = Climatic Factor dependent on rainfall, as developed for various regions in Illinois (from a chart in Bulletin No. 462)
- Z = Peak - Reduction Factor – dependent on the ratio of the design rainfall duration to the lag time. The lag time is a function of the length of the drainage basin measured along the watercourse and the average channel slope. The value of *Z* is obtained from a curve in Bulletin No. 462.

Although the curves, charts and tables presented in Bulletin No. 462 for determining the values of the three factors *X*, *Y* and *Z* are based on the particular climate and

L. APPENDIX L

Example Problems

Statement for Problem #1

“A 5-acre site is to be split into two 2-1/2 acre pieces, each with a house of 4,000 square feet and an additional 3,000 square feet of outbuildings and pavement. In addition, each lot will have 4,000 square feet of landscaping (lawns and shrubs). The remainder of the lots will be in fruit trees. The current land use is oak with a grass understory. The HSG is C. The site is located at M.A.P. 22.5 and has an average slope in the direction of flow of 3%.”

"Currently there is a flooding problem every other year (2-year flood) not far downstream from this site.”

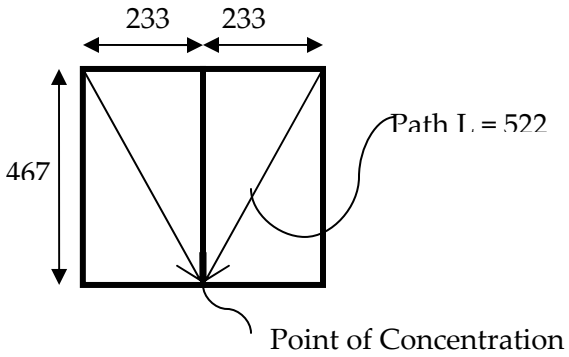
"Design a detention basin or basins that will not exacerbate the downstream flood problem and use the Santa Clara County Manual as the basis for the calculations."

Solution and Steps

1. Summarize data and identify necessary input parameters.
2. Develop flow rates for pre-development conditions.
3. Develop flow rates for post-development conditions.
4. Size detention basin.

I. Existing Conditions

Area = 5 ac (217,800 sq. ft.)
Assume square = 467 ft x 467 ft (2 parcels of 467 x 233)
Length L =



Slope = 0.03 (given)

t_c (minutes) Equation 3-2

$$t_c = 0.0078\left(\frac{L^2}{S}\right)^{0.385} + 10$$

$$= 0.0078\left(\frac{522^2}{0.03}\right)^{0.385} + 10$$

t_c = 13.7 min.

C = 0.2 from description and Table 3-1 (Shrub Land on C soil)

Find rainfall depth $x_{T,D}$ (and intensity) for the 2-year storm at 22.5 M.A.P.

From Table B-1 for the 2-year return period.

T (min)	$A_{T,D}$	$B_{T,D}$
10	0.166507	0.001956
13.7	0.173989	0.002863
15	0.176618	0.003181

Depth $x_{T,D} = A_{T,D} + \left(B_{T,D}MAP\right)$ Equation 3-3

= 0.173989 + 22.5 (0.002863)

Depth $x_{T,D}$ = 0.2384 in. at 13.7 min.

Intensity $i_{T,D} = \frac{x_{T,D}}{D}$

$$= \frac{0.2384}{\left(\frac{13.7\,\text{min}}{60\,\text{min}/hr}\right)}$$

Intensity $i_{T,D}$ = 1.04 in/hr

Peak Runoff Rate = CIA
= 0.2 (1.04)(5)
Peak Runoff Rate = 1.04 cfs

II. Future Conditions

The 522 ft of run is divided:

- Half into Short Grass Pasture
- Half into Paved Area

@ 0.03 slope from Figure A-1

261 ft @ 1.3 fps = 200.8 sec
261 ft @ 3.5 fps = 74.6 sec
275.3 sec
t_c = 4.6 min.

Proposed C: weighted average from description and Table 3-1 (Shrub Land on C soil)

Site Area		Paved/Impervious Area		Landscaped Area				Average C
				Urban Open Space		Agricultural		
	(sq. ft.)	(sq. ft.)	C	(sq. ft.)	C	(sq. ft.)	C	
1	108,900	7,000	0.85	4,000	0.35	97,900	0.35	
2	108,900	7,000	0.85	4,000	0.35	97,900	0.35	
								0.38

Find rainfall depth $x_{T,D}$ (and intensity) for the 100-year storm at 22.5 M.A.P.

From Table B-1 for the 100-year return period

T (min)	$A_{T,D}$	$B_{T,D}$
5	0.269993	0.003580

Depth $x_{T,D} = A_{T,D} + \left(B_{T,D} MAP\right)$ Equation 3-3

= 0.269993 + 22.5 (0.003580)

Depth $x_{T,D}$ = 0.3505 in. at 5 minutes

Intensity $i_{T,D} = \dfrac{x_{T,D}}{D}$

$$= \frac{0.3505}{\left(\dfrac{5\,\text{min}}{60\,\text{min}/hr}\right)}$$

Intensity $i_{T,D}$ = 4.21 in/hr

Peak Runoff Rate = CIA

= 0.38 (4.21)(5)

Peak Runoff Rate = 8.0 cfs

Must throttle back to match the 1.04 cfs existing 2-year storm condition to not increase 2-year flooding downstream.

ASCE Method (Constant Outflow – usually most applicable for pumps)

T	100-Yr Depth (in)	Volume In (ft^3)	Volume Out (ft^3)	Storage (ft^3)
5-min	0.3505	3,372	300	3,072
10-min	0.4798	4,615	600	4,015
15-min	0.5779	5,559	900	4,659
30-min	0.6431	6,186	1,800	4,386
1-hr	1.0586	10,183	3,600	6,583
2-hr	1.5473	14,884	7,200	7,684
3-hr	1.9860	19,104	10,800	**8,304**
6-hr	3.0504	29,343	21,600	7,743
12-hr	4.4710	43,009	43,200	-191

Depth $x_{T,D} = A_{T,D} + \left(B_{T,D} MAP\right)$ Equation 3-3

Volume In = A(ac) x (C + 0.15) x (Depth/12) x 43560(ft^2/ac)

(adjustment to C explained in Section 6.3.1)

Max Storage = 8,300 ft³

A basin 41′ x 41′ x 6′ deep (5′ of active storage and 1′ of freeboard)

APWA Method

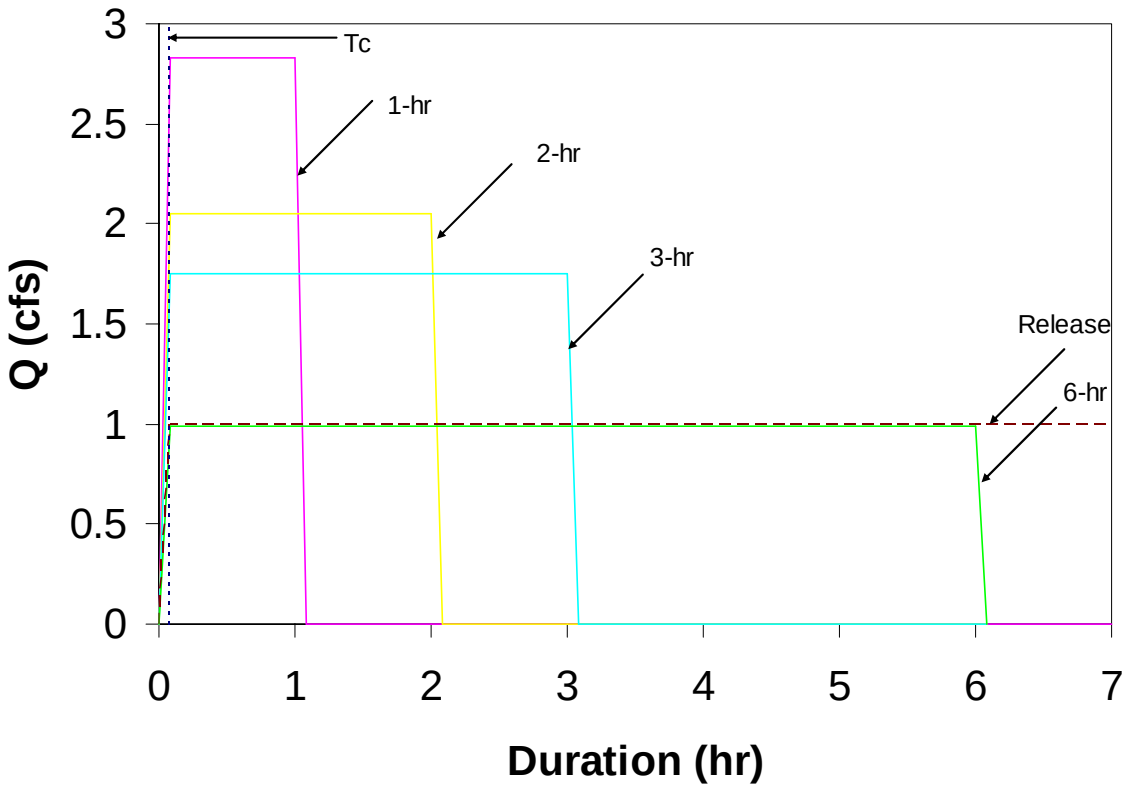
Use C = 0.38 + 0.15 = 0.53

Depth $x_{T,D} = A_{T,D} + \left(B_{T,D} MAP\right)$ Equation 3-3

Intensity $i_{T,D} = \frac{x_{T,D}}{D}$

Q = CIA

T	100-Yr Depth (in)	Intensity (in/hr)	Q (cfs)
5-min	0.3505	4.21	11.15
10-min	0.4798	2.88	7.63
15-min	0.5779	2.31	6.13
30-min	0.6431	1.29	3.41
1-hr	1.0586	1.06	2.81
2-hr	1.5473	0.77	2.05
3-hr	1.9860	0.662	1.75
6-hr	3.0504	0.51	1.35
12-hr	4.4710	0.37	0.99



Find greatest storage volume (difference between the areas under the curve of the inflow and outflows). Can be found geometrically or with a planimeter.

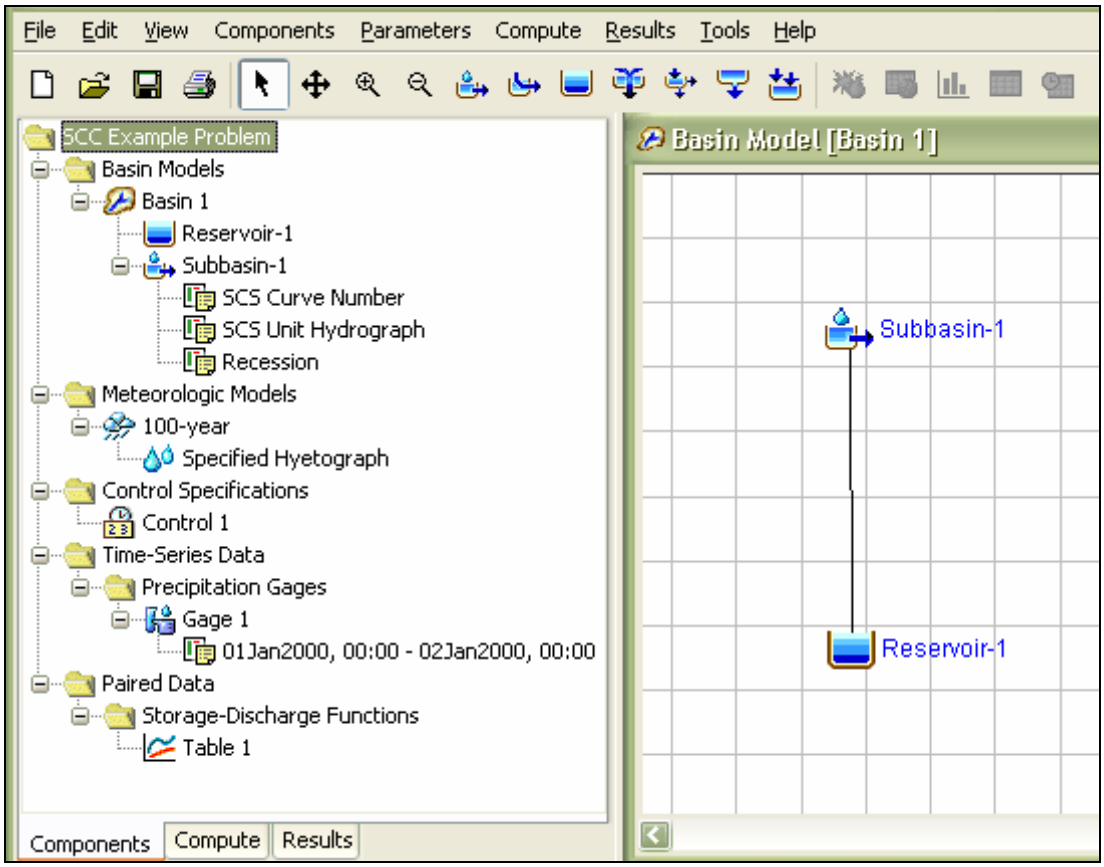
1-hr Volume = 6,588 ft^3
2-hr Volume = 7,560 ft^3
3-hr Volume = 8,100 ft^3 Max Storage
6-hr Volume = -216 ft^3

40′ x 40′ x 6′ deep (5′ of active storage and 1′ of freeboard)

HEC-HMS Computer Model Method

HMS Input

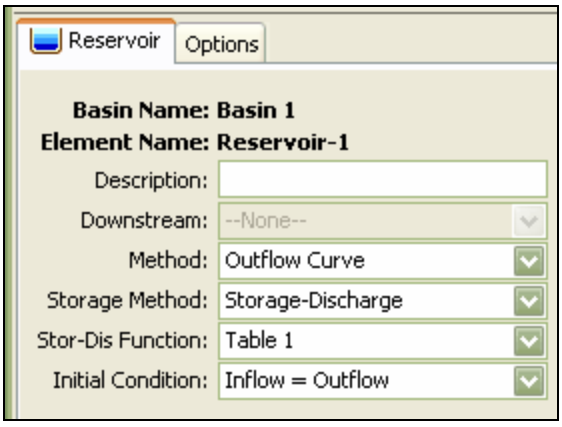
Set up your model as outlined below. Parameters follow.



Basin Model

Reservoir

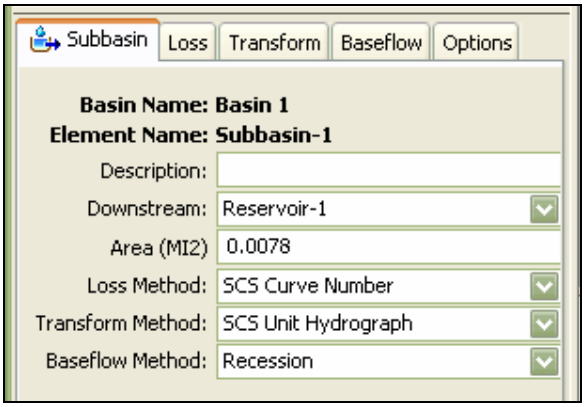
Method: Outflow Curve
Storage Method: Storage-Discharge
Stor-Dis Function: Table (create outflow table of 1 cfs in paired data)
Initial Condition: Inflow = Outflow



Options – Blank

Subbasin

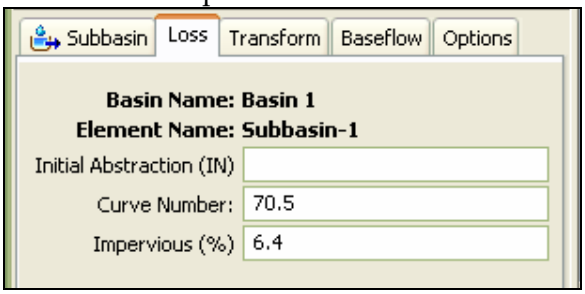
A = 0.0078 mi²



Loss Method: SCS Curve Number

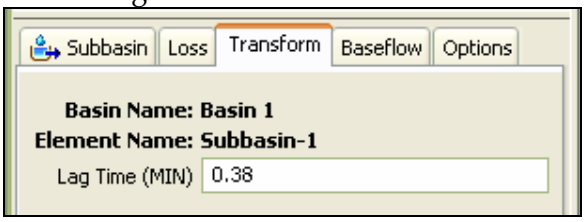
CN = 70.5

6.4% impervious



Transform Method: SCS Unit Hydrograph

Lag Time = 0.38 min



Baseflow Method: Recession

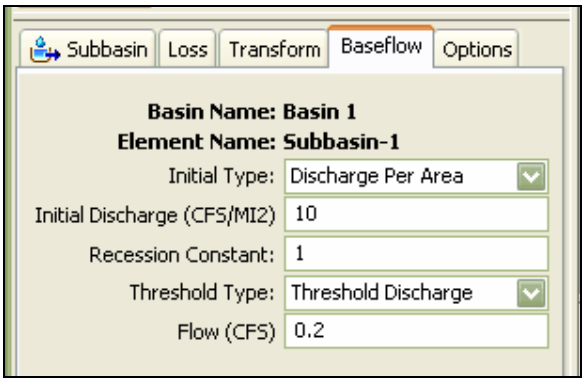
Initial Type: Discharge Per Area

Initial Discharge = 10 cfs/mi²)

Recession Constant = 1

Threshold Type = Threshold Discharge

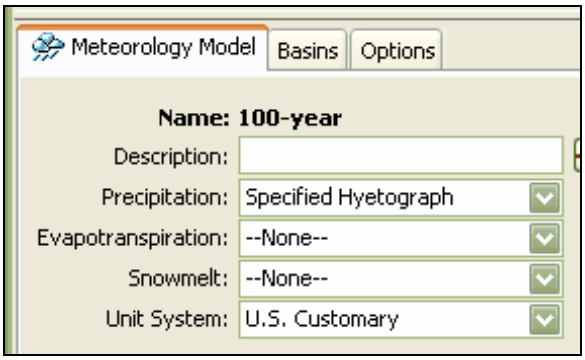
Flow = 0.2 cfs



Meteorological Model

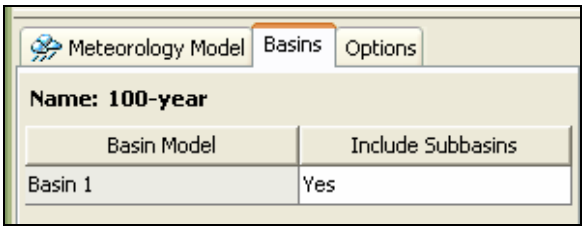
100-yr

Precipitation: Specified Hyetograph
Evapotranspiration: None
Snowmelt: None



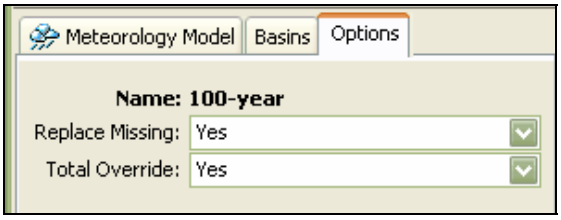
Basins

Include Subbasins: Yes



Options

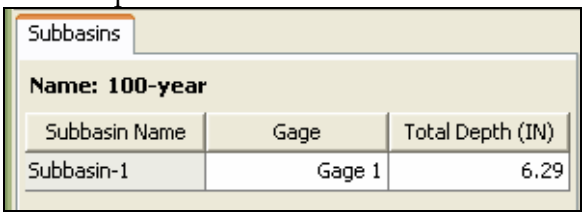
Replace Missing: Yes
Total Override: Yes



Specified Hyetograph

Gage 1

Total Depth = 6.29 in.



-
  - Compute the precipitation depths for the 100-year design return event using the TDS Regional Equation and enter it under Total Depth. Since the SCS Unit Hydrograph method has been chosen, the gage data is input as a unit hydrograph (percentages of the total flow) that will be multiplied with this total depth.

Control Specifications

Control 1

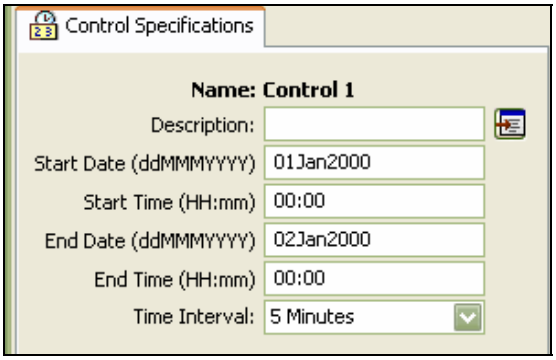
Start Date: 01 Jan 2000

Start Time: 00:00

End Date: 02 Jan 2000

End Time: 00:00

Time Interval: 5 Minutes



Time-Series Data

Precipitation Gages

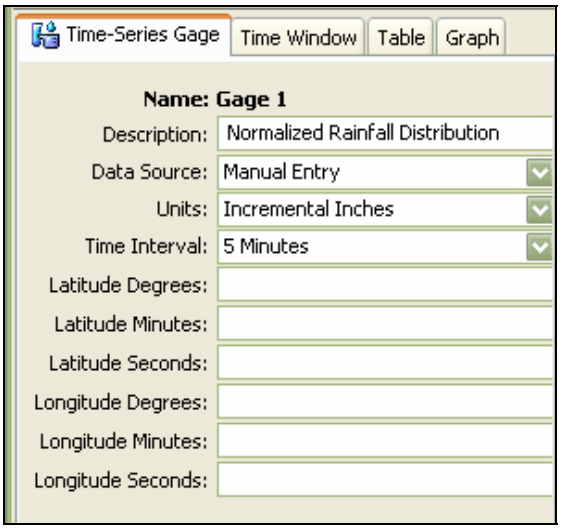
Gage 1

Time-Series Gage

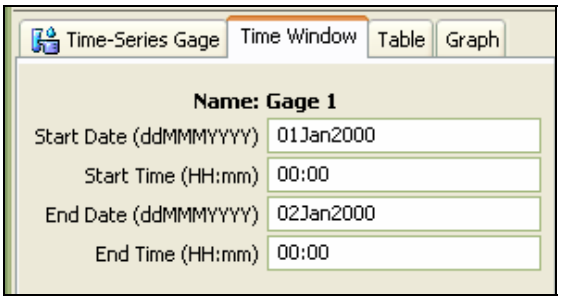
Data Source: Manual Entry

Units: Incremental Inches

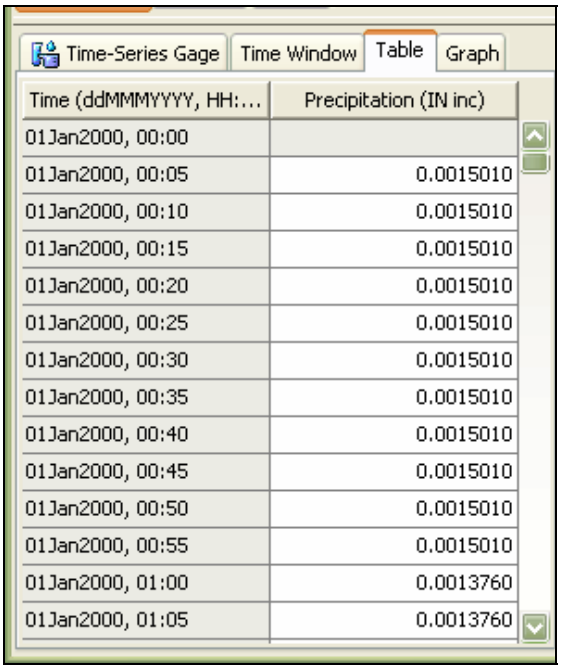
Time Interval: 5 Minutes



Time Window



Table



- Fill in full gage record. Use Table D-1 and interpolate for MAP of site. This table gives percentages of the total storm depth. The data shown gives the percentage for each 5-minute time period within each hourly interval. If a 5-minute time step is not desired, compute a fraction of total rainfall for each time interval by multiplying the 5-minute percentages by the time interval (in minutes) and dividing by 5.

Paired Data

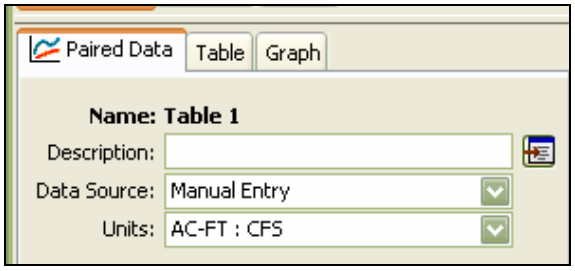
Storage-Discharge Functions

Table 1

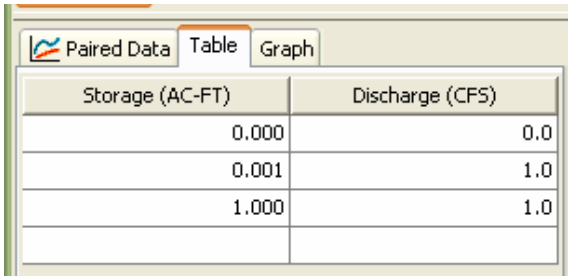
Paired Data

Data Source: Manual Entry

Units: AC-FT : CFS



Table



Create a run a compute.

Storage = 8, 093 ft^3

40′ x 40′ x 6′ deep (5′ of active storage and 1′ of freeboard)

HMS gives results in to only tenths of an acre-foot. For smaller ponds, this is inadequate. For this problem, HMS rounds the maximum storage to 0.2 ac-ft where the actual is 0.1858 ac-ft. To see more significant figures, the storage file in the DSS file may be opened and explored. Consult HEC manuals for more information on HEC-HMS and HEC-DSS.

Statement for Problem #2

The same parameters as Problem #1 – "A 5-acre site is to be split into two 2-1/2 acre pieces, each with a house of 4,000 square feet and an additional 3,000 square feet of outbuildings and pavement. In addition, each lot will have 4,000 square feet of landscaping (lawns and shrubs). The remainder of the lots will be in fruit trees. The current land use is oak with a grass understory. The HSG is C. The site is located at M.A.P. 22.5 and has an average slope in the direction of flow of 3%."

"Design a detention basin that will mitigate the peak discharge for each the 2-year, 10-year, and 100-year storm events using the Santa Clara County Manual as the basis for the calculations. Use the same size basin for each storm event and design an outlet combination structure that will mitigate all 3 discharges."

Solution and Steps

1. Summarize data and identify necessary input parameters
2. Develop discharge for pre-development conditions using the rational method
3. Develop discharge for post-development conditions using HEC-HMS
4. Size detention basin and outlet structures

Existing Conditions

2-year

Same as Example #1

Peak Runoff Rate = 1.04 cfs

10-year

Find rainfall depth $x_{T,D}$ (and intensity) for the 10-year storm at 22.5 M.A.P.

From Table B-1 for the 10-year return period

T (min)	$A_{T,D}$	$B_{T,D}$
13.7	0.285415	0.004413

Depth $x_{T,D} = A_{T,D} + \left(B_{T,D} MAP\right)$ Equation 3-3

$= 0.285415 + 22.5\ (0.004413)$

Depth $x_{T,D}$ = 0.385 in. at 13.7 min.

Intensity $i_{T,D} = \dfrac{x_{T,D}}{D}$

$$= \frac{0.385}{\left(\dfrac{13.7\,\text{min}}{60\,\text{min}/hr}\right)}$$

Intensity $i_{T,D}$ = 1.68 in/hr

Peak Runoff Rate = CIA

$= 0.2\ (1.68)(5)$

Peak Runoff Rate = 1.68 cfs

100-year

Find rainfall depth $x_{T,D}$ (and intensity) for the 100-year storm at 22.5 M.A.P.

From Table B-1 for the 100-year return period

T (min)	$A_{T,D}$	$B_{T,D}$
13.7	0.393775	0.007050

Depth $x_{T,D} = A_{T,D} + \left(B_{T,D} MAP\right)$ Equation 3-3

$= 0.393775 + 22.5\ (0.007050)$

Depth $x_{T,D}$ = 0.552 in. at 13.7 min.

Intensity $i_{T,D} = \frac{x_{T,D}}{D}$

$$= \frac{0.552}{\left(\frac{13.7\,\text{min}}{60\,\text{min}/hr}\right)}$$

Intensity $i_{T,D}$ = 2.42 in/hr

Peak Runoff Rate = CIA
= 0.2 (2.42)(5)
Peak Runoff Rate = 2.42 cfs

These peak runoff rates cannot be exceeded in the future conditions models. Detention basin outlets must be designed to restrict flows to these values.

Future Conditions

Basin Model

Basin parameters for the loss method need to be calculated. The SCS Curve Number method will be used. First, impervious areas and composite CN values need to be determined. CN values for different land uses can be found in Table E-1. These values need to be adjusted for antecedent moisture condition (AMC) according to Tables 4-1 and E-2.

Future Land Cover	% of Total Area	CN (Table E-1)	AMC Adjusted CN (Tables 4-1, E-2)	
			2-yr	10-yr,100-yr
Paved/impervious	6.43%			
Lawns & Shrubs	3.67%	66 (Urban Recreational)	70	74
Fruit Trees	89.9%	66 (Orchards)	70	74

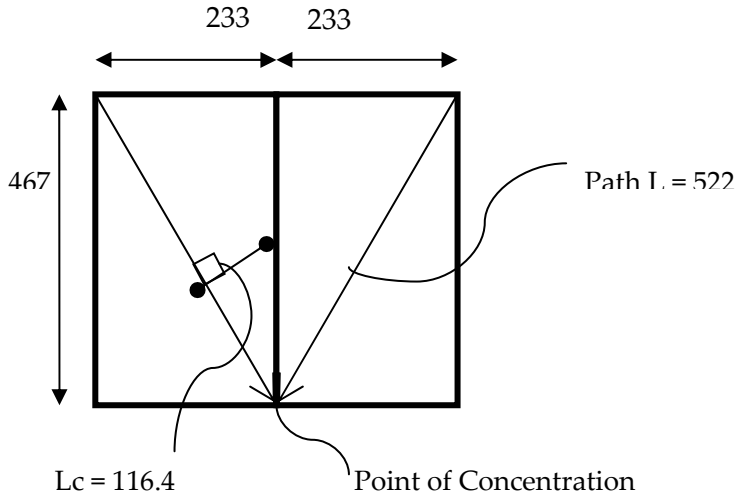
This results in a composite CN of 70 for the 2-year storm and 74 for the 10- and 100-year storms. The paved/impervious areas are not factored into the composite CN value since the CN is only applied to pervious surfaces in HMS. The impervious area is input as a percentage and subtracted from the total area during HMS calculations.

For the transform method, the SCS Unit Hydrograph method will be used. This requires the computation of a lag time. This is explained in Section 4.6.6.

$$t_{lag} = (0.862)\ 24\ N \left(\frac{L\,L_c}{\sqrt{S}} \right)^{0.38} - \frac{D}{2} \qquad (4\text{-}1)$$

Where	t_{lag}	=	SCS watershed lag (hours)
	N	=	watershed roughness value (dimensionless)
	L	=	longest flow path from catchment divide to outlet (miles)
	L_c	=	length along flow path from a point perpendicular with the basin centroid to its outlet (miles)
	S	=	effective slope along main watercourse from Chapter 3 (feet/mile)
	D	=	duration of unit hydrograph (hours)

Since the percentage of impervious area remains small even after development, an N of 0.08 for natural channels with little urbanization will be used as found in Table 4-2.



L = 522 ft = 0.099 mi
Lc = 116.4 ft = 0.022 mi
S = 0.03 ft/ft = 158.4 ft/mi
D = 5 min = 0.08333 hr

t_{lag} = 0.01989 hr = 1.19 min

For baseflow, the Recession method will be used.

Initial Type: Discharge Per Area
Initial Discharge = 10 cfs/mi²)
Recession Constant = 1

Threshold Type = Threshold Discharge
Flow = 0.2 cfs

Since the CN values vary for the different storms, three separate watershed models should be set up. They will be identical except for the CN values.

Meteorologic Model

The Meteorologic Model, Control Specifications, and Time Series Data will all be the same as in Problem 1. The only parameter that will differ will be the total precipitation depth for each storm event. Therefore, three separate meteorologic models will need to be set up – one for each storm event using different total storm depths found by using the Regression Equations. The total storm length of 24 hours will be used.

The total precipitation depths for the 24-hour storms are as follows:

Depth $x_{T,D} = A_{T,D} + \left(B_{T,D} MAP\right)$ Equation 3-3

Storm	Total Storm Depth (inches)
2-year	2.48
10-year	4.22
100-year	6.29

Detention Basin Sizing

The basin will be designed as a square with 3:1 side slopes. The outlet structure will be an open-topped standpipe with orifices. The flow through the orifices for any given height will be based on the basic orifice equation $Q = CA\sqrt{(2gh)}$ with a C of 0.6.

Since the 100-year storm will require the greatest total pond volume, we will size the basin itself based on 100-year flows. In general, a depth of approximately 5 feet is desirable. To get a basic sizing, an assumed orifice will be placed at the base of the standpipe that will not allow more than the existing 100-year peak discharge (2.42 cfs) to pass with 5 feet of head. A 6-inch diameter orifice is the largest in whole inches that will comply, passing 2.11 cfs.

A routing element will placed downstream of the basin element in the watershed model. Two sets of paired data need to be created and specified in the pond element: a storage-discharge curve and an elevation-storage curve. Create these curves using the 6 inch orifice, varying the basin area, and run the model until the chosen basin area results in a model basin depth of approximately 5 feet. After some iteration, a bottom basin base of about 15 x 15 feet is found. This storage-discharge curve will no longer be needed.

Since the 2-year flow is the smallest, the portion of the outlet structure to regulate the 2-year flow will be designed first. The required volume will likely be considerably less than the available volume, making the depth an unknown. The elevation-storage curve is now fixed, so only the discharge-storage curve needs to be altered. Choose orifices sizes, placing them at elevation = 0 so that the basin can drain fully, and experiment until one is found where the resulting depth creates an outflow less than the existing 2-year flow (1.04 cfs). This simulation run will use the Basin and Meteorologic Models for the 2-year storm.

Orifice Diameter	Max. Flow (cfs)	Depth (ft)
3″ at elev = 0′	0.3	1.7
4″ at elev = 0′	0.4	0.9
5″ at elev = 0′	**0.4**	**0.6**
6″ at elev = 0′	0.4	0.4

Notice that the maximum future outflow from the basin is already less than the existing. This is because the time step for the model is 5 minutes and the lag time is only 1.19 minutes. This causes the peak to be cut off since it occurs somewhere within the large time step, but the total volume is conserved. Therefore, since an orifice cannot be chosen based on discharge, the orifice will be chosen that releases the largest amount of flow while still creating some detention (backs up the flow above the height of the orifice).

*Orifice #1 – diameter = 5″, elevation = 0′

Next, the discharge-storage curve from this orifice will be placed in the 10-year storm model to ensure that the increased depth does not cause a flow higher than the existing 10-year (1.68cfs). This causes a discharge of 1.2 cfs at a depth of 3.1 feet. Since this is significantly below the existing discharge, another orifice will be added. It will be set at elevation = 0.6 feet so that it will only drain the 10-year discharges that are in excess of the 2-year discharge.

Orifices	Max. Flow (cfs)	Depth (ft)
5″ at elev = 0′ + 4″ at elev = 0.6′	**1.5**	**2.3**
5″ at elev = 0′ + 5″ at elev = 0.6′	1.7	2.1

The 4 inch orifice is the largest that be added without exceeded 1.68 cfs.

*Orifice #2 – diameter = 4″, elevation = 0.6′

Next, the discharge-storage curve from the combined orifices will be placed in the 100-year storm model. This causes a flow of 2.2 cfs at a depth of 4.7 feet. Since this is slightly below the existing 100-year flow of 2.42 cfs, no additional orifices need to be added.

If proper maintenance is not performed or debris is washed into the basin during a storm event, it is possible that the orifices may become clogged. To prevent overtopping of the basin should this occur, an emergency spillway will be designed. The HMS model shows that the maximum inflow of 7.4 cfs to the basin. The emergency spillway will be designed to carry this flow without significantly increasing the basin size. Since the outlet structure is designed as a riser pipe with orifices, the easiest way to accommodate overflows is to leave the top of the riser open and size the riser diameter to be able to carry the flows. Figure J-3 gives elevation-discharge curves for varying riser diameters. Set the crest of the riser at 4.7 feet above the base of the basin (maximum depth for the 100-year storm). Create elevation-discharge curves assumed that both orifices have clogged (worst case scenario). Run these in HMS with the 100-year storm to determine total basin depth. There are several diameters that will work. A 24″ riser will function well as it is average sized and creates an addition depth of only 0.5′, increasing the total basin depth requirement to 5.2′ for the worst case scenario.

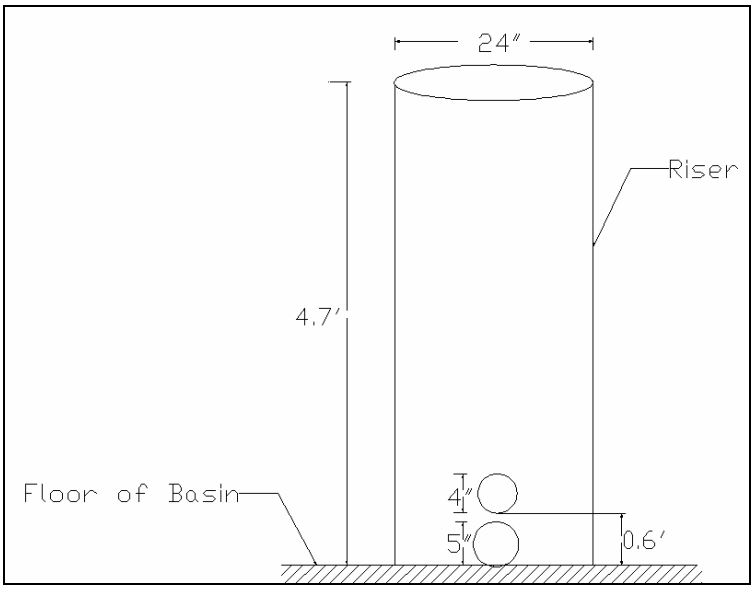
The final basin design to accommodate all 3 storm scenarios and the emergency scenario is as follows:

Detention Basin Overview

Basin Base Area	15′ x 15′ at base (225 ft²)
Basin Footprint Area	46.2′ x 46.2′ (2,134 ft²)
Side Slopes	3:1
Maximum Basin Depth	5.2′

Detention Basin Outlet Works

Outlet Description	**Diameter or Width**	**Invert above Basin Bottom**
Orifice #1	5″	0′
Orifice #2	4″	0.6′
Riser	24″	4.7′


24″
Riser
4.7′
Floor of Basin
4″
5″
0.6′