White Paper
Migrating from AHYMO'97 to HEC-HMS (and USEPA SWMM)

Prepared for

Albuquerque Metropolitan Arroyo Flood Control Authority

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Prepared by

OCCAM ENGINEERS, INC.
now
Stantec
6100 SEAGULL STREET NE
SUITE B102
ALBUQUERQUE, NM 87109

State of Practice for Hydrology
Migrating from AHYMO'97 to HEC-HMS (and USEPA SWMM)

Introduction

AMAFCA commissioned its on-call consulting engineer, Easterling Consultants LLC (now Occam Engineers, Inc.) to explore the potential, the opportunities and the obstacles to migrating from the current regionally used AHYMO_97 hydrology modeling tool and its associated DPM hydrologic methodology to US Army Corps of Engineers HEC-HMS for upland watersheds and to USEPA SWMM for the valley areas. Occam Engineers Inc. was asked to recommend a hydrologic methodology that produces reasonable and consistent results when evaluated in the context of the region's existing drainage infrastructure which was designed and built using modern analyses and design criteria. Any proposed migration away from the current DPM hydrologic methodology for larger watersheds (> 40 acres) will also require the adoption of a new methodology for those that are less than 40 acres to keep the DPM internally consistent.

Computer models should:

- Incorporate the latest technology within the state of the practice in urban hydrology
- Have very good user manuals and technical documentation
- Be in the public domain and universally available free or at minimal expense
- Be able to run on current computer operating systems and have strong and sustainable support for maintenance and updates as computers and technology progress
- Be GIS compatible
- Be operated with easily acquired and readily available data for its operation
- Be able to produce reasonable and consistent results when used in accordance with sound engineering principles and practices
- Be acceptable to the City of Albuquerque, Office of State Engineer, FEMA, NMDOT and other approving and coordinating agencies

The recommended hydrologic methods should:

- Be based on sound engineering and physical processes
- Be relatively simple to use by practitioners and to review by government engineers
- Be widely understood or at least understandable both inside and outside the local engineering community
- Be based on physical processes while allowing for the application of sound engineering judgment
- Be able to produce consistent results when used in accordance with sound engineering principles and practices
- Be performed with easily acquired and readily available data
- Give reasonable results consistent with current practices (not produce runoff values that are significantly higher or lower than those used in recent years)
- Be useable within the computer models chosen without significant modification or alteration
- Be acceptable to the City of Albuquerque, Office of State Engineer, FEMA, NMDOT and other approving and coordinating agencies

Background

Albuquerque Metropolitan Arroyo Flood Control Authority (AMAFCA) tasked Easterling Consultants LLC (now Occam Engineers Inc.) with developing this white paper to assist the community in the required migration from AHYMO_97 (Anderson-Hydro) as the local hydrologic model of choice to HEC-HMS (US Army Corps of Engineers). This white paper provides background information in support of both the need to abandon the DOS version of AHYMO_97 and the recommendation to adopt MS Windows platform HEC-HMS as the upland hydrology model of choice and USEPA SWMM as the lowlands or valley model of choice.

AHYMO_97 - AHYMO_97 is an Arid Lands HYdrology MOdel. This program was based on the USDA Agricultural Research Service (ARS) HYMO program. HYMO was introduced to New Mexico and the Albuquerque area in 1979 when it was adapted by Bohannan Houston, Inc. as AHYMO for use in the City of Albuquerque's Master Drainage Study (AMDS) of the northeast and southeast heights of Albuquerque. At that time, there were no urban hydrology models in existence suitable for modeling the urban watersheds in Albuquerque. Beginning in 1986, Cliff Anderson, P.E., then an AMAFCA employee, began to modify the computer code within the original ARS HYMO for most of the same purposes that were required for use in the AMDS. However, the basic NRCS Runoff Curve Number (CN) method used in both the original HYMO and the AMDS version of AHYMO was replaced by a set of hydrologic methods based on then available rainfall/runoff hydrologic data and some field data gathered for that purpose by AMAFCA thought to be specific to the Albuquerque region. As a result, it soon became the program of choice by the AMAFCA and the City of Albuquerque.

AHYMO_97 is a DOS operating system program which requires the 80 character input format left over from the days of punch cards. This system has become obsolete with the demise of MS Windows XP and the advent of MS Windows 7 and later versions. AHYMO_97 will not run on MS Windows 7 and later versions of the Windows operation system, thus making it effectively impossible to use. AHYMO_97 cannot be used by engineers outside New Mexico (such as FEMA reviewers), needing to test and evaluate modeling results. A newer version of AHYMO_97 (called AHYMO_S4) has been produced by Anderson-Hydro which will run on a MS Windows 7 operating platform, but that version is not currently listed by FEMA as an acceptable computer model for conducting flood plain studies. Anecdotally, recent side-by-side testing of AHYMO_97 and AHYMO_S4 by one of AMAFCA's consultants indicates that the new version appears to produce significantly different results from those of AHYMO_97 particularly in larger watersheds. The reasons and solutions to this issue have not been identified and are not included in this White Paper.

Approach and Findings

The approach used to arrive at a set of recommendations and assist in gaining their approval with the local engineering community is as follows:

A. *Determine the "state of the practice" in urban hydrology, particularly in the southwest US;* An internet survey was conducted of all available on-line information for cities in New Mexico and larger communities in surrounding states. Of those in New Mexico not using

AHYMO and the City of Albuquerque DPM or some derivative of it, the overwhelming majority use the Rational Method for small basins where no hydrograph is needed (i.e. for pond design) and NRCS "TR-55 Hydrology for Urban Watersheds" for their hydrology method when basin size exceeds 40-60 acres or a hydrograph is needed. The exception is Arizona where ADOT and some of the larger cities use Clark's Unit Hydrograph method for larger basins. NMDOT, Texas DOT, Colorado DOT and Utah DOT all either require or allow TR-55 and/or NRCS Curve Number method for hydrologic computations.
See **Appendix A,** "Other Users of CN Method," for a more complete listing of Federal, State and local TR-55 users.

B. *Review available computer models meeting the criteria.*
The internet was surveyed again for computer models meeting the desired criteria. The available models fitting the stated criteria are limited to USACE HEC-HMS for upland hydrology modeling and USEPA SWMM for valley type conditions. Both are widely used, generally taught in engineering schools, in the public domain, relatively easy to learn and use, well documented and supported, run on the latest computer operating systems and the required input data is usually readily available or easily obtainable. See **Appendix B,** "HEC-HMS Described," for more details its capabilities and use.

C. *Recommend hydrologic data inputs*
1) Rainfall distribution, and source of data – NOAA Atlas 14 data as found on NOAA's data server. A range of rainfall distributions can be generated at this site. A 25% frequency curve distribution is recommended for most applications. Runoff values are somewhat sensitive to this decision, but not as much as was assumed. Further study was performed to provide guidance in the selection of a standard rainfall distribution for modeling in the local area.
    a) Rainfall Distribution Sensitivity Analyses -**Appendix C** "Rainfall" contains the results of the sensitivity analysis performed on seven HEC-HMS watershed models previously prepared for AMAFCA, SSCAFCA and NMDOT by local consultants. Modeled watersheds range in size from 119 acres to over 15,000 acres. Six are urban watersheds and one is rural. Five different rainfall distributions (DPM AHYMO, NRCS Type II75, and 25%, 33% and 50% frequency distributions from HEC-HMS) were tested in each of the seven models in order to determine if, and to what degree the results are sensitive to rainfall distribution. See "Rainfall Distribution Sensitivity Analysis" for a more thorough discussion and for the foundation for the recommendation for adoption of the 25% distribution.
    b) NOAA's data server is a tremendous resource for collecting and portraying rainfall related data in several useful forms. Since the data is geospatially related, maps of rainfall amounts, intensities and the relationships of these to location can be readily determined. A sampling of the type and format that data can be retrieved and presented is shown in **Appendix C** as well.
2) Rainfall/Runoff transformation method – the NRCS Runoff Curve Number method as described in NRCS TR-55 is recommended as the rainfall/runoff transform. It is widely used and understood. The method has been in use in its current form for over 30 years and in some form for over 50 years and is used around the world in both rural and urban settings. See **Appendix D** "Rainfall Runoff with TR-55".
3) Unit Hydrograph Selection – there is little data available for use in determining an

appropriate unit hydrograph shape. Given that the intent is actually to develop a comprehensive and compatible hydrologic modeling approach for use in planning, design and evaluation of storm drainage and flood control facilities and not an attempt in mimicking an actual storm, the recommended approach is to incorporate the SCS unit hydrograph shape as the standard and to verify that the design rainfall distribution chosen is compatible with it. Since whichever storm temporal distribution is chosen as the standard will very likely never actually occur, nor will it be distributed over the watershed evenly as our assumptions dictate, the key is to choose a unit hydrograph/storm distribution/rainfall-runoff transform combination the produces results compatible with the community's desired level of protection.

4) Time of Concentration Determination Method – it is recommended that the upland flow method be used for determining time of concentration as described in TR-55 and the current DPM. See **Appendix E,** Tc "Time of Concentration".

D. *Compare results between AHYMO '97 and HEC-HMS using TR-55.*

1) Multiple computer simulations were conducted on a range of small basins to evaluate the differences between the current methods described in the DPM using AHYMO-97 and HEC-HMS using TR-55 hydrology. Sites were chosen as being typical of either the east heights or the west side to evaluate the effects of different rainfall amounts and soil types. In addition, two actual sites were compared – a 5.9 acre commercial site on Alameda just west of I-25 and the second, a mini-DMP on 119 acres near I-40 and Unser Blvd. The results of all these comparisons are shown in **Appendix F** "Method Comparisons". For these purposes, a 25% frequency distribution for the rainfall distribution was utilized. This distribution was chosen as a "middle of the road" parameter since at that time, no sensitivity analyses had been performed using multiple historical and available distributions.
2) In addition to the trial modeling performed on hypothetical sites in various locations around the metro area, actual side by side model results were collected and compared from the limited data available. Modeling of large complex watersheds happens infrequently in the metro area due to the cost. Watersheds that have relatively recent and comparable AHYMO '97 and HEC-HMS models covering the same areas were found for the Amole, Boca Negra, Kirtland and South Diversion watersheds. All but the South Diversion HEC-HMS model incorporated the recommendations from this White Paper on using hydrology methods described in NRCS TR-55. The South Diversion Channel HEC-HMS modeling was performed using the Stantec modification found in the Rio Rancho/SSCAFCA DPM. The model results were plotted in an attempt to find trends and differences. The plots of the modeling results are shown in **Appendix H**.

Recommended Practices for utilizing HEC-HMS for Upland Hydrologic Modeling

To update the Hydrologic modeling for the Albuquerque area to meet the current state of practices within the engineering community the following changes are recommended:

1. Rainfall – it is recommended that the source of data and temporal distribution be determined from the most current data published by NOAA. NOAA supports an online data server that allows a user to determine the rainfall amounts for a wide range of frequencies, durations and distributions for all of New Mexico. The data server allows the user to input the location of the site either by selecting the location on a map or by entering the coordinates. For planning and design purposes, the objective is to determine a design storm temporal distribution and volume that is appropriately conservative and consistent with the selected unit hydrograph and time of concentration methodology.

2. Rainfall/Runoff Transform– the CN method as described in NRCS Technical Report 55 is recommended as the preferred method for converting rainfall to runoff because throughout the US over the past 30 years it has been demonstrated to:
   - produce reasonable results,
   - be understandable,
   - produce reproducible results,
   - be relatively simple to use and review.

   The input parameters are hydrologic soil type and land use (cover). The CN method is described as a "lumped parameter" method because its use incorporates multiple factors and processes into one number.
   a. Unit Hydrograph shape- SCS Unit hydrograph is good as any here so long as basins are divided appropriately, runoff volumes are computed accurately, and Tc is calculated consistently and intelligently. The alternative is to use a unit hydrograph methodology that adjusts the unit graph shape based on basin factors (length, width, and slope). An adjustable unit hydrograph shape may have the potential to be more accurate (for mimicking actual storms), but most of the gains would be at the expense of simplicity and consistency and significantly higher data collection requirements. Data collection and related review effort would be considerably greater than is currently required and as well as much greater than using the SCS (NRCS) Unit Hydrograph and CN methods as described in TR-55.
   b. Tc – it is recommended that the NRCS upland method be utilized, almost exactly as described in the DPM (it needs to be modified somewhat for very large watershed usage).

3. Modeling- Use HEC-HMS for the uplands (East and West sides), and the EPA program SWMM for flat valley with using CN hydrology as modified by Bernalillo County for the Sanchez Farm study and by the City of Albuquerque for the Mid-Valley DMP. See Appendix I for a description of this methodology.

4. The Rational Method developed for the 2017 Update of the NMDOT Drainage Design Manual, adapted as needed to the Albuquerque metro area is recommended for calculations of peak rate only for small drainage areasNMDOT Manual allows the use of the Rational Method in watersheds of up to 160 acres. The vast majority of NMDOT

applications are rural (undeveloped), making it more likely that the watershed is reasonable homogenous and can be adequately described by one Rational Formula ‘C’ factor. In urban (developed) watersheds, the probability of a watershed being homogeneous in watersheds over 40 acres is significantly lower. The size, complexity and capital cost of proposed drainage facilities increases dramatically as watershed size increases as well. It is recommended that the current DPM limit of 40 acres be retained for urban applications.

Migrating to HEC-HMS

The Current version of HEC-HMS is 4.2.1 (as Oct. 2017). The three (3) basic parts of HMS include the following:

- Basin Models (Where the physical characteristics of the watershed are entered in the model)
- Meteorological Models (Where the rainfall/precipitation is located)
- Control Specifications (Time steps, start and stop times and dates over what temporal segment a model is run)

With these three basic building blocks, a hydrologic model can be constructed that will create and route hydrographs for use in sizing and analyzing drainage, water quality and flood control structures.
Based on the current publicly available data for the Albuquerque metropolitan area, Occam Engineers Inc. recommends these parameters and criteria when performing hydrologic analysis using HEC-HMS.

Control Specifications

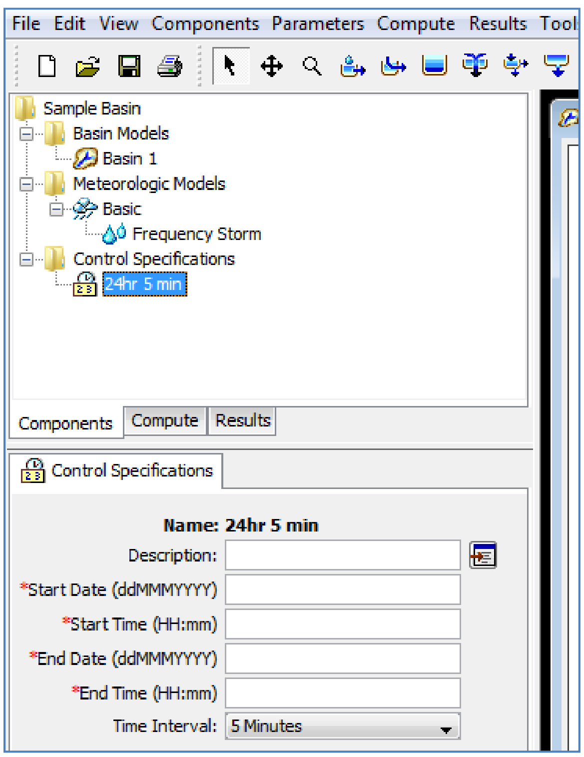


Figure 1 – Example of the Control Specifications Window

- Control specifications should always have a name that includes the duration and time step (**24hr 5-min)** and any additional details if seasonal modeling is going to occur (monsoon season vs spring).
- Start date should always be the year of the project (the month is not important unless modeling a specific event).

- Start time shall always start at **00:00.**
- End time shall always be a time step longer than duration of the model (00:05, if it was a 24hr 5 min interval storm).
- The duration of the control should always be long enough to capture the full event (for instance a modeled pond or reservoir should be completely drained before ending the control time).
- Time interval should **5 minutes or less** to ensure the peak flows are captured. Shorter times can be used when dealing with small Tc's for basins.

A note about the time steps - HEC-HMS automatically adjusts the time steps within the model if there is a need for it, however what is reported out is what the user specifies, for additional information, please see the HEC-HMS User's Manual.

Meteorological Models

With the release of NOAA Atlas 14 for New Mexico, a more refined dataset is now available than the current DPM standard. This allows for the use of a synthetic frequency storm created by HMS using actual rainfall data, creating an optimum rainfall model for any location within the Albuquerque metropolitan area.

Rainfall data can be found at NOAA's website using their frequency data server. http://hdsc.nws.noaa.gov/hdsc/pfds/pfds_map_cont.html?bkmrk=nm

The study point should be placed at the **CENTROID** of the watershed, not at the point of study. Ensure that the time series type is set at ***partial duration***.

When submitting models, be sure to create a hard copy (print or pdf) of precipitation to document the values used. Values are updated on the data server (NOAA updates the storm library from time to time).

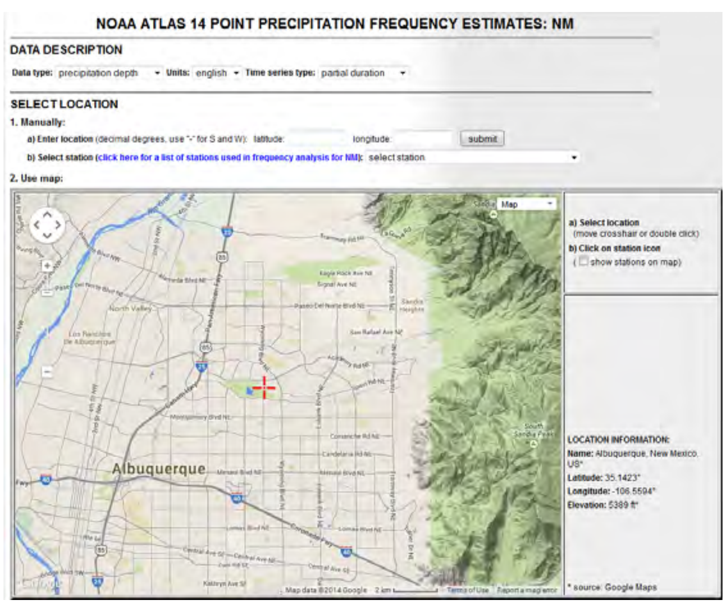


Figure 2 - NOAA Atlas 14 Window

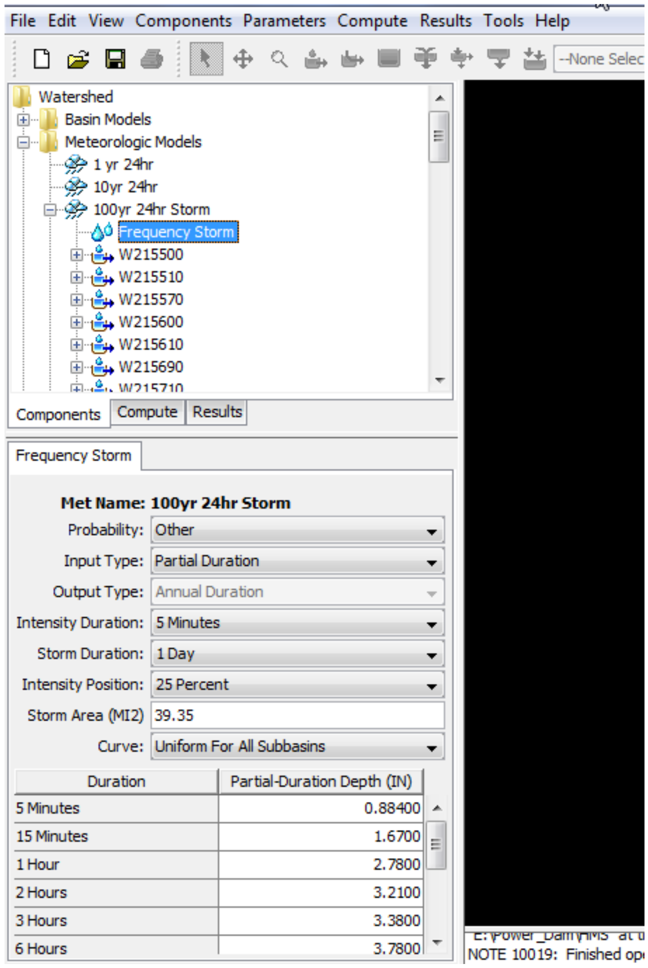


Figure 3 - Example of How Rainfall Data is Entered in HEC-HMS

- Probability will be the inverse of the Frequency storm (100 yr = 1%, 50 yr = 2%, 10 yr = 10%, 2 yr = 50%, etc.)
- Input type should be **Partial Duration**
- Intensity Duration should be **5 minutes**
- Storm Duration : **Value set by local requirements – 6 hours or 24 hours is typical for the Albuquerque metro area**
- Intensity Position: **25%.** This distribution value has been determined by a sensitivity analyses for various size watersheds within the Albuquerque metro area.
- Storm Area should be at least as large as the study watershed for most studies.
- Partial duration depth data can be retrieved from the NOAA servers (pay close attention to the duration values). Figure 4 shows an example of data retrieved from the NOAA servers for use in HMS.

PF tabular | PF graphical | Supplementary information

PDS-based precipitation frequency estimates with 90% confidence inter

Duration	Average recurrence interval (years)						
	1	2	5	10	25	50	100
5-min	0.175 (0.148-0.209)	0.226 (0.190-0.270)	0.305 (0.255-0.363)	0.365 (0.304-0.434)	0.447 (0.371-0.531)	0.512 (0.424-0.608)	0.581 (0.476-0.688)
10-min	0.266 (0.225-0.317)	0.345 (0.288-0.410)	0.464 (0.388-0.552)	0.556 (0.463-0.660)	0.680 (0.565-0.808)	0.780 (0.645-0.926)	0.884 (0.725-1.05)
15-min	0.330 (0.279-0.393)	0.427 (0.358-0.509)	0.575 (0.481-0.685)	0.689 (0.574-0.818)	0.843 (0.700-1.00)	0.966 (0.800-1.15)	1.09 (0.898-1.30)
30-min	0.444 (0.375-0.530)	0.575 (0.482-0.685)	0.774 (0.648-0.922)	0.928 (0.772-1.10)	1.14 (0.943-1.35)	1.30 (1.08-1.55)	1.48 (1.21-1.75)
60-min	0.550 (0.464-0.656)	0.712 (0.596-0.848)	0.958 (0.802-1.14)	1.15 (0.956-1.36)	1.41 (1.17-1.67)	1.61 (1.33-1.91)	1.83 (1.50-2.16)
2-hr	0.658 (0.542-0.817)	0.846 (0.697-1.05)	1.12 (0.920-1.39)	1.34 (1.10-1.65)	1.65 (1.33-2.02)	1.90 (1.53-2.33)	2.16 (1.73-2.64)
3-hr	0.704 (0.583-0.864)	0.896 (0.739-1.10)	1.18 (0.972-1.44)	1.40 (1.15-1.71)	1.71 (1.40-2.09)	1.96 (1.59-2.39)	2.23 (1.80-2.71)
6-hr	0.823 (0.686-1.00)	1.04 (0.867-1.27)	1.34 (1.12-1.63)	1.58 (1.31-1.91)	1.91 (1.58-2.31)	2.16 (1.78-2.62)	2.44 (1.99-2.94)

Figure 4 - Example of Partial Duration Depth Data from NOAA 14

Basin Models

The Basin model consists of 7 basic elements*:

1. Subbasins
2. Routing Reaches
3. Reservoirs
4. Junctions
5. Diversions
6. Sources
7. Sinks

With these 7, a simple or very-complicated model can be built to route hydrographs.

*For a detailed description of each, see the HEC-HMS User’s Manual.

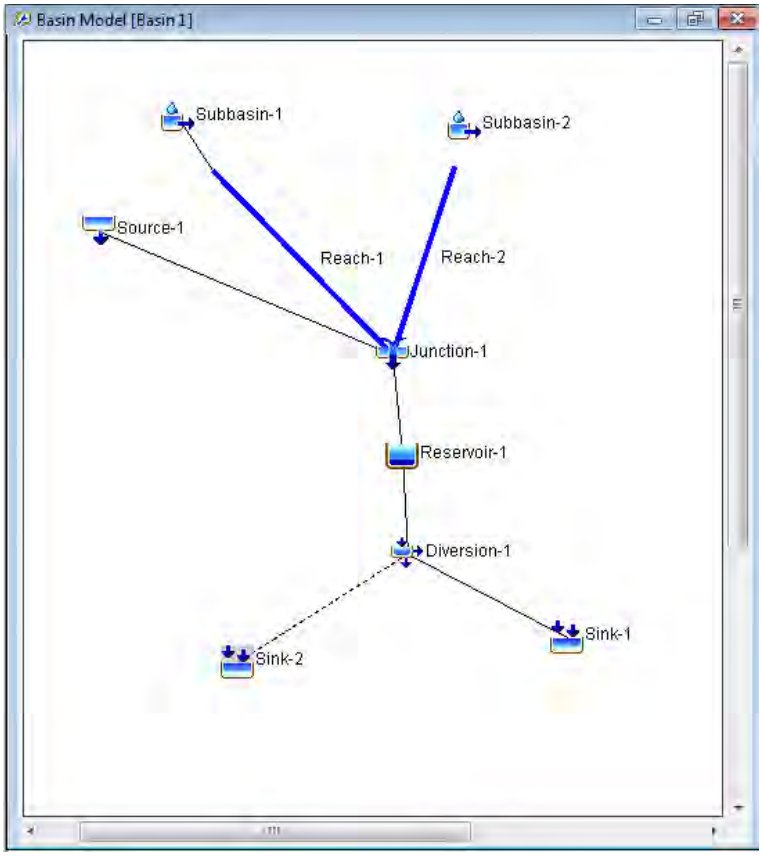


Figure 5 - Example of a Sub-Basin Model

Subbasins

Figure 6 shows the window in which input parameters are entered for each subbasin.

- Name: **Use Logical name if possible**, if recreating old models, use the same naming convention
- Downstream: Be sure that your downstream connection is correct.
- Area: expressed as **square miles**
- Lat/Long values are not used.
- Canopy Method: **None**
- Surface Method: **None**
- Loss Method: **SCS Curve Number**
- Transform Method: **SCS Unit Hydrograph**
- Baseflow Method: **None**

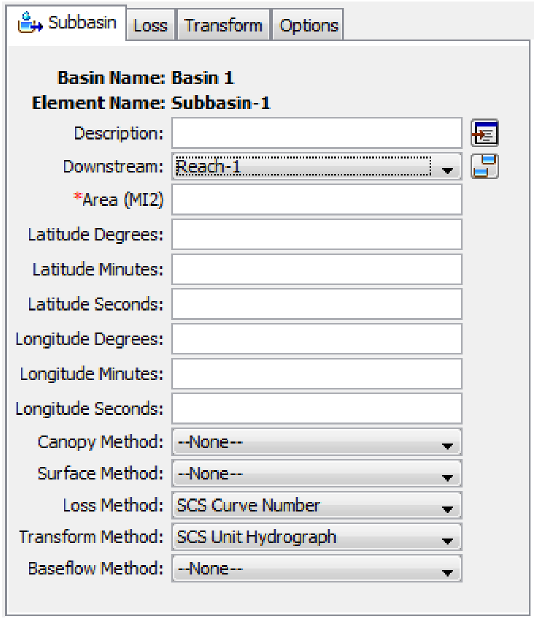


Figure 6 – HEC_HMS Subbasin Input Screen

Loss Method

Using the SCS Curve number method for determining losses has been quite successful throughout the US and even within New Mexico; however because of the soils and the typical storm systems that pass through the metro area, a modification of the Curve number method is needed.

- Curve numbers will be based on Soils and Land Use
  - Hydrologic Soils Group Information can be obtained from NRCS or elsewhere. Land Use can be determined from most current aerial photography and/or City or County Zoning documents
  - Curve numbers shall be determined from the methods prescribed in the NRCS Document **TR-55.** Pgs. 2-5 through 2-9 **(Appendix D)**
    - Curve Numbers should range from 50's on the West side to the low 90's in the clay soils of the valley and from the 60's to the 90's on the East side and mountain face watersheds.
  - Subbasins must be homogenous for the majority of the sub basin (No merging 60 acres of open space with 40 acres of parking lot!) If not practical to subdivide basins to that extent, then the "Weighted Runoff" method should be used rather than the "Weighted Curve Number" method. See **TR-55** for further guidance.
  - Curve Numbers shall be reported as whole numbers, NO TENTHS.

Transform

Use of the SCS unit hydrograph is prescribed, and the transform input as **Basin Lag**, in minutes. The graph type will be **Standard** as shown in Figure 7.

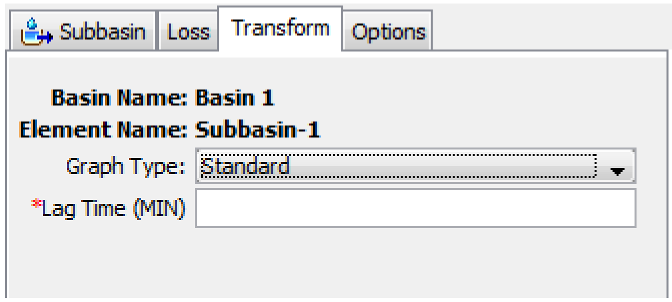


Figure 7 – HEC-HMS Transform Input Window

Basin Lag is a function of the Time of Concentration (T Lag = .6 Tc).

The time of concentration will be calculated using the method prescribed in TR-55 and the DPM.

Where Tc= t sheet + t shallow + t channel.

For drainages that are from the west face of the Sandia Mountains, a modified Tc will have to be employed to account for the bare rocky faces and the shortened Tc's. Calculated travel times can be unreasonably short in the prescribed Tc method when slopes are exceptionally steep (>10%).

Reach

Routing of the hydrographs will be performed using the **Muskingum-Cunge** Method.

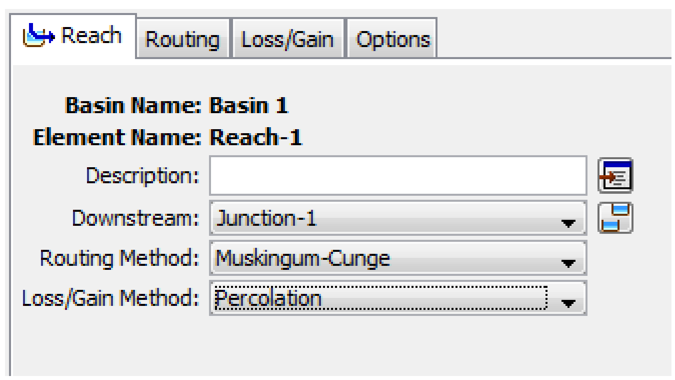


Figure 8 - HEC-HMS Reach Input Window

Routing

- Time Step Method: **Automatic Adaption**
- Length (ft): Measured
- Slope (ft/ft): **Average reach slope** (US/DS elevations) – The slope is a component to determining the velocity at which water will travel through the routing reach. Care should be given to routing reaches across grade control structures to ensure that flow velocity is accurately reflected in the model.
- Manning's 'n': **Weighted average 'n' value used in HEC-RAS model for channel and floodplain**
- Invert: (usually left blank)

- Shape: Whenever possible use the trapezoid or 8 point for natural channels (this can vary from site to site based on available data).
    - The 8 point section will need cross sections (entered as paired data into another part of the program)
- Bottom width (ft):measured
- Side Slopes: Measured, estimated from aerials/topo.

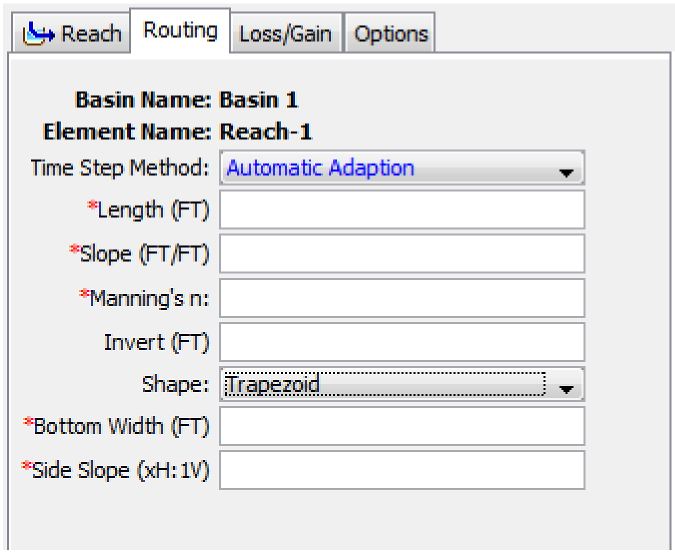


Figure 9 - HEC-HMS Routing Input Window

Loss/Gain

The Loss/Gain Method (Channel Losses) may be determined within the model as the need arises.

Reservoirs

Best practices for modeling ponds, reservoirs and dams are utilized for modeling these facilities.

Junctions

Rules for using junctions-

- Junctions should be used at all confluences and whenever 3 or more sub basins flow to a pond.
- Junctions should also be used for study points.

Diversions

Best practices for diversions include:

- fixed flow rate diversion,
- stage based diversion
- percentage based diversions.

Sources and Sinks

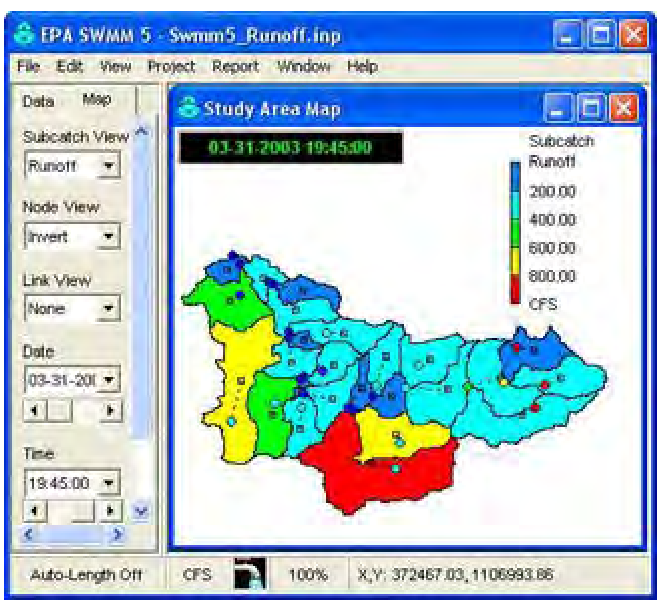
Best practices for including flow sources and flow sinks are available in the program.

AHYMO ’97 has been used inappropriately for more than 20 years for hydrologic modeling in the valley in the Albuquerque metro area. Recent modeling associated with drainage master plans in the valley (Mid Valley DPM for COA and Sanchez Farm DMP for Bernalillo County) has demonstrated that AHYMO and the DPM hydrology methodology significantly overestimate runoff volumes and flow rates and is limited as a tool for developing and evaluating alternatives. Further, hydraulic modeling with SWMM in the course of these planning efforts demonstrates that valley storm drains and channels can and may often flow two directions during a single storm event, which upland models such as AHYMO’97 and HEC-HMS are not capable of calculating.

The re-evaluation of modeling within previously prepared valley drainage master plans performed with AHYMO and current DPM hydrology has resulted in the finding of significant floodplain reductions and smaller required storm drainage facilities. Rainfall/runoff calculations may be generated within SWMM or imported from HEC-HMS (recommended).

The following short description of SWMM outlines its capabilities. See the SWMM User Manual for model building and operations.

Storm Water Management Model (SWMM)



Version 5.1.006 with Low Impact Development (LID) Controls

Description

EPA's Storm Water Management Model (SWMM) is used throughout the world for planning, analysis and design related to stormwater runoff, combined and sanitary sewers, and other drainage systems in urban areas. There are many applications for drainage systems in non-urban areas as well.

SWMM is a dynamic hydrology-hydraulic-water quality simulation model. It is used for single event or long-term (continuous) simulation of runoff quantity and quality from primarily urban areas. The runoff component operates on a collection of sub catchment areas that receive precipitation and generate runoff and pollutant loads. The routing portion transports this runoff through a system of pipes, channels, storage/treatment devices, pumps, and regulators.

SWMM tracks the quantity and quality of runoff made within each sub catchment. It tracks the flow rate, flow depth, and quality of water in each pipe and channel during a simulation period made up of multiple time steps. SWMM 5 has recently been extended to model the hydrologic performance of specific types of low impact development (LID) controls. The LID controls that the user can choose include the following seven green infrastructure practices:

- permeable pavement
- rain gardens
- green roofs
- street planters
- rain barrels
- infiltration trenches
- vegetative swales

The updated model allows engineers and planners to accurately represent any combination of LID controls within a study area to determine their effectiveness in managing stormwater and combined sewer overflows.

Running under Windows, SWMM 5 provides an integrated environment for editing study area input data; running hydrologic, hydraulic and water quality simulations; and viewing the results in a variety of formats, such as:

- color-coded drainage area and conveyance system maps,
- time series graphs and tables,
- profile plots, and
- statistical frequency analyses.

SWMM 5 was produced by USEPA in a joint development effort with CDM, Inc., a global consulting, engineering, construction, and operations firm.

Capabilities

SWMM accounts for various hydrologic processes that produce runoff from urban areas. These include:

- time-varying rainfall,
- evaporation of standing surface water,
- snow accumulation and melting,
- rainfall interception from depression storage,
- infiltration of rainfall into unsaturated soil layers,
- percolation of infiltrated water into groundwater layers,
- interflow between groundwater and the drainage system,
- dynamic routing that allows flow in opposite directions within the same conduit as a result of hydrograph timing and relative hydraulic grade line elevations within the system during the modeling of an event
- nonlinear reservoir routing of overland flow, and
- runoff reduction via Low Impact Development (LID) controls.

Spatial variability in all of these processes is achieved by dividing a study area into a collection of smaller, homogeneous sub catchment areas. Each of the areas contains its own fraction of pervious and impervious sub-areas. Overland flow can be routed between sub-areas, between sub catchments, or between entry points of a drainage system.

SWMM contains a flexible set of hydraulic modeling capabilities used to route runoff and external inflows through the drainage system network of pipes, channels, storage/treatment units and diversion structures. These include the ability to:

- handle drainage networks of unlimited size;
- use a wide variety of standard closed and open conduit shapes as well as natural channels;
- model special elements such as storage/treatment units, flow dividers, pumps, weirs, and orifices;
- apply external flows and water quality inputs from surface runoff, groundwater interflow, rainfall-dependent infiltration/inflow, dry weather sanitary flow, and user-defined inflows;
- utilize either kinematic wave or full dynamic wave flow routing methods;
- model various flow regimes, such as backwater, surcharging, reverse flow, and surface ponding; and
- apply user-defined dynamic control rules to simulate the operation of pumps, orifice openings, and weir crest levels.
-

SWMM can estimate the production of pollutant loads associated with stormwater runoff. The following processes can be modeled for any number of user-defined water quality constituents:

- Dry-weather pollutant buildup over different land uses;
- Pollutant wash-off from specific land uses during storm events;
- Direct contribution of rainfall deposition;
- Reduction in dry-weather buildup due to street cleaning;

- Reduction in wash-off load due to BMPs;
- Entry of dry weather sanitary flows and user-specified external inflows at any point in the drainage system;
- Routing of water quality constituents through the drainage system; and/or
- Reduction in constituent concentration through treatment in storage units or by natural processes in pipes and channels.

Applications

Since its release, SWMM has been used in thousands of sewer and stormwater studies throughout the world. Typical applications include the following:

- Design and sizing of drainage system components for flood control.
- Sizing of detention facilities and their appurtenances for flood control and water quality protection.
- Floodplain mapping of natural channel systems (SWMM 5 is a FEMA-approved model for NFIP studies).
- Designing control strategies for minimizing combined sewer overflows.
- Evaluating the impact of inflow and infiltration on sanitary sewer overflows.
- Generating non-point source pollutant loadings for waste load allocation studies.
- Controlling site runoff using Low Impact Development practices.
- Evaluating the effectiveness of BMPs for reducing wet weather pollutant loadings.

Support

There is no formal support offered for EPA SWMM. A SWMM users' listserv was established by the University of Guelph. This listserv allows subscribers to ask questions and exchange information. To subscribe, send an email message to listserv@listserv.uoguelph.ca with the words "subscribe swmm-users" (without the quotes) in the body followed by your name.

Rational Formula

Building a hydrology model in HEC-HMS or SWMM is not appropriate for a significant percentage of cases where only a flow rate is needed for planning, analyses or design. The well-known Rational Formula has, despite its limitations, served engineers well for many years and still has utility for small, simple project needs. As was seen in the early development of the Albuquerque DPM, the hydrology used for small projects should be compatible and reasonably consistent with that developed for larger more complex projects in terms of the runoff rates and volumes developed within each. While not directly within the scope of this White Paper, it was recognized that if the current DPM hydrologic methods are linked to AHYMO (e.g. Treatment Types and Rainfall Zones) the chosen Rational Formula method should be compatible and consistent with NRCS TR-55 hydrology methods. Research performed in association with Occam Engineers, Inc.'s recent efforts to update the NMDOT Drainage Design Manual (2017) indicates that the current NMDOT Rational Method approach meets this need. It is also compatible with recent US EPA dictates regarding the calculation of flow rates associated with NPDES water quality protection.

Therefore, the NMDOT Rational Method approach is proposed as the replacement to the current DPM Rational Formula method with one exception- ***watershed size should not exceed 40 acres for urban watersheds***. The following excerpt is taken from the introduction to the NMDOT manual. The Rational Method from NMDOT may be found in **Appendix G** or the reader may (eventually) download the manual from the NMDOT Website.

From NMDOT Drainage Design Manual Section 403 Rational Formula:

> "Hydrologic analyses performed on small (<160 acre) watersheds will normally be performed using the Rational Formula. The Rational Formula Method is a widely and long accepted procedure worldwide for estimating peak rates of runoff from small watersheds. The Rational Formula may be used on NMDOT projects for roadway drainage facilities and small drainage structures as described in **Section401 (Figure 401-1 and Figure 401-2)** of thismanual. The standard form of the equation in English units is:
>
> $$\mathbf{Q = CiA} \qquad \textbf{403-1}$$
>
> Where:
>
> Q = the peak rate of runoff, in cfs
>
> C = a dimensionless runoff coefficient
>
> i = the rainfall intensity, in inches/hour
>
> A = the watershed or drainage area, in acres
>
> The units in the Rational Formula equation do not yield cfs directly, but rather are in acre-inches/hour. However, the conversion from acre-inches/hour to cfs is 1.008 which is commonly neglected because it does not introduce a significant error. The Rational

Formula has several assumptions implicit to the method, including:

- The rainfall intensity is uniform for a duration equal to or greater than Tc.
- Peak flow occurs when the entire watershed is contributing runoff.
- The frequency of the resulting peak discharge is equal to the frequency of the rainfall event.
- Both Rational ‘C’ Coefficient and rainfall intensity (i) vary with the return period (both tend to increase as return period increases). Therefore, both must be determined separately for each design storm frequency.
- Rational ‘C’ Coefficient is dependent on the Hydrologic Soil Group (HSG) and the vegetative cover or in the case of developed watersheds, the percentage of impervious cover. HSG’s are divided into four soil groups and are described in **Section 402.4** Soils Data

Limitations for using the Rational Formula on NMDOT projects include the following:

- The total drainage area should not exceed 160 acres.
- Land use, slope, and soils are fairly consistent throughout the watershed.
- There are no diversions, detention basins, pump stations or other structures in the watershed which would require the routing of a flood hydrograph.
- The time of concentration does not exceed one hour.”

AHYMO and HEC-HMS Model Comparisons

A review of the first draft of this White Paper by a Technical Review Committee organized by AMAFCA resulted in a recommendation that as many side-by-side comparisons of modeling performed with the AHYMO model using then current Albuquerque Development Process Manual guidelines and HEC-HMS models of the same basins, using the recommendations of this White Paper as guidance. Four basins were found to have been modeled by both AHYMO and HEC-HMS. The basins for which models existed are: South Diversion Channel, Amole-Hubbell, Boca Negra and Gibson Blvd. at Kirtland.

The modeling results were compared, one against the other for several parameters considered appropriate for making such comparisons. Not all modeling results contained all the data necessary to do a side-by-side comparison for every parameter, however. The data was also accumulated into one data set and compared from basin to basin. The results of these comparisons are found in **Appendix H**.

Ideally, these comparisons would have demonstrated clear cut differences between modeling a basin with AHYMO according to the hydrologic approach presented in the DPM and HEC-HMS using the hydrologic approach recommended in this white paper. While side by side models of the same watersheds often showed significant differences between the two models and methods, some of the biggest differences were found within the individual model results themselves. As a result there were no clear patterns apparent between the AHYMO and HEC-HMS models. In other words, one did not consistently produce higher peak runoff rates or greater runoff volumes than the other.

Model results using AHYMO/DPM models showed significantly more scatter and variability of results when measured against ***peak rate/cfs***, ***runoff volume/inch of rainfall***, and ***time to peak vs watershed size***, compared to the HEC-HMS/White Paper models.

Given that the Albuquerque metropolitan area has little actual rainfall/runoff gage data against which to compare the results from either hydrologic approach the more internally consistent modeling results produced by the HEC-HMS/White Paper system is the more attractive hydrologic modeling tool.

From this analysis there is no evidence that changing models or modeling methods will cause a dramatic increase or decrease in the size of future storm drainage systems or demonstrate that existing modern systems are significantly under or over-sized.

APPENDIX A

OTHER USERS OF CN METHOD

Examples of Regional Use of NRCS Curve Number Method for Urban Hydrology per TR-55

- NRCS New Mexico (***with over 100 dams and no spillway operations***)
- FEMA
- Federal Energy Regulatory Commission (***the most conservative of all Federal agencies***)
- US Bureau of Reclamation
- USEPA "Stormwater Management for TMDL's in an Arid Climate: A Case Study Application of SUSTAIN in Albuquerque, New Mexico" EPA/600/R-13/004, March 2013 (***recent project by USEPA in Albuquerque but also recommended in most USEPA manuals***)
- FHWA
- Bernalillo County
- NMDOT
- NMOSE
- SWCD's in NM for the development and review of subdivision terrain management plans
- City of Carlsbad, NM allows both NMDOT method and DPM – AHYMO (latest version)
- City of Las Cruces, NM
- Farmington, NM
- NMSU Facilities Dept.
- Texas Commission on Environmental Quality statewide (***dams and water quality***)
- Texas DOT
- Midland, Texas
- Amarillo, Texas
- Lubbock, Texas
- Colorado DOT
- Colorado Springs, Colorado
- Scottsdale, Arizona
- La Paz County, Arizona (***very desert area, near Lake Havasu City)***
- Utah DOT
- Clark County, Nevada (Las Vegas)

NOTE: The use of NRCS Runoff Curve Number Method is universal. It is used throughout the US (From Maine to California and Oregon to Florida as well as worldwide.)

Also of note, the only viable alternate method that I am aware of- "Green-Ampt" is being used by Arizona DOT. The description on how to determine the Green Ampt Loss Method is 15 pages long and the appendix supporting it is 400 pages long. Factors to be determined to use this method are: Initial Content (dry); Initial Content (wet); Saturated Content; Suction; Conductivity; Impervious %, and this has to be developed individually for each sub-basin in a watershed.

http://www.azdot.gov/docs/default-source/roadway-engineering-library/2014_adot_hydrology_manual.pdf?sfvrsn=8

APPENDIX B

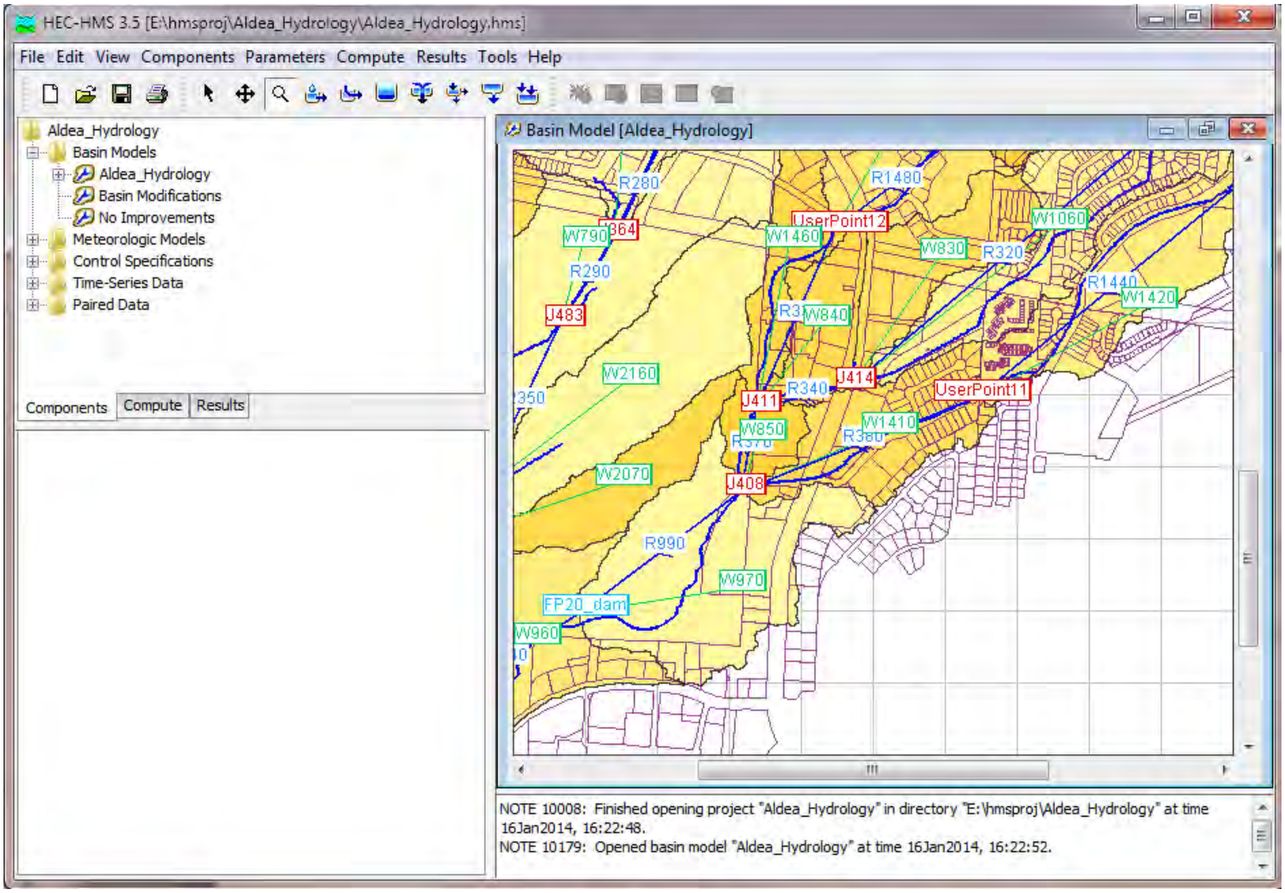
HEC-HMS DESCRIBED

HEC-HMS

The Hydrologic Modeling System (HEC-HMS) is designed to simulate the complete hydrologic processes of dendritic watershed systems. The software includes many traditional hydrologic analysis procedures such as event infiltration, unit hydrographs, and hydrologic routing. HEC-HMS also includes procedures necessary for continuous simulation including evapo-transpiration, snowmelt, and soil moisture accounting. Advanced capabilities are also provided for gridded runoff simulation using the linear quasi-distributed runoff transform (ModClark). Supplemental analysis tools are provided for parameter estimation, depth-area analysis, flow forecasting, erosion and sediment transport, and nutrient water quality.

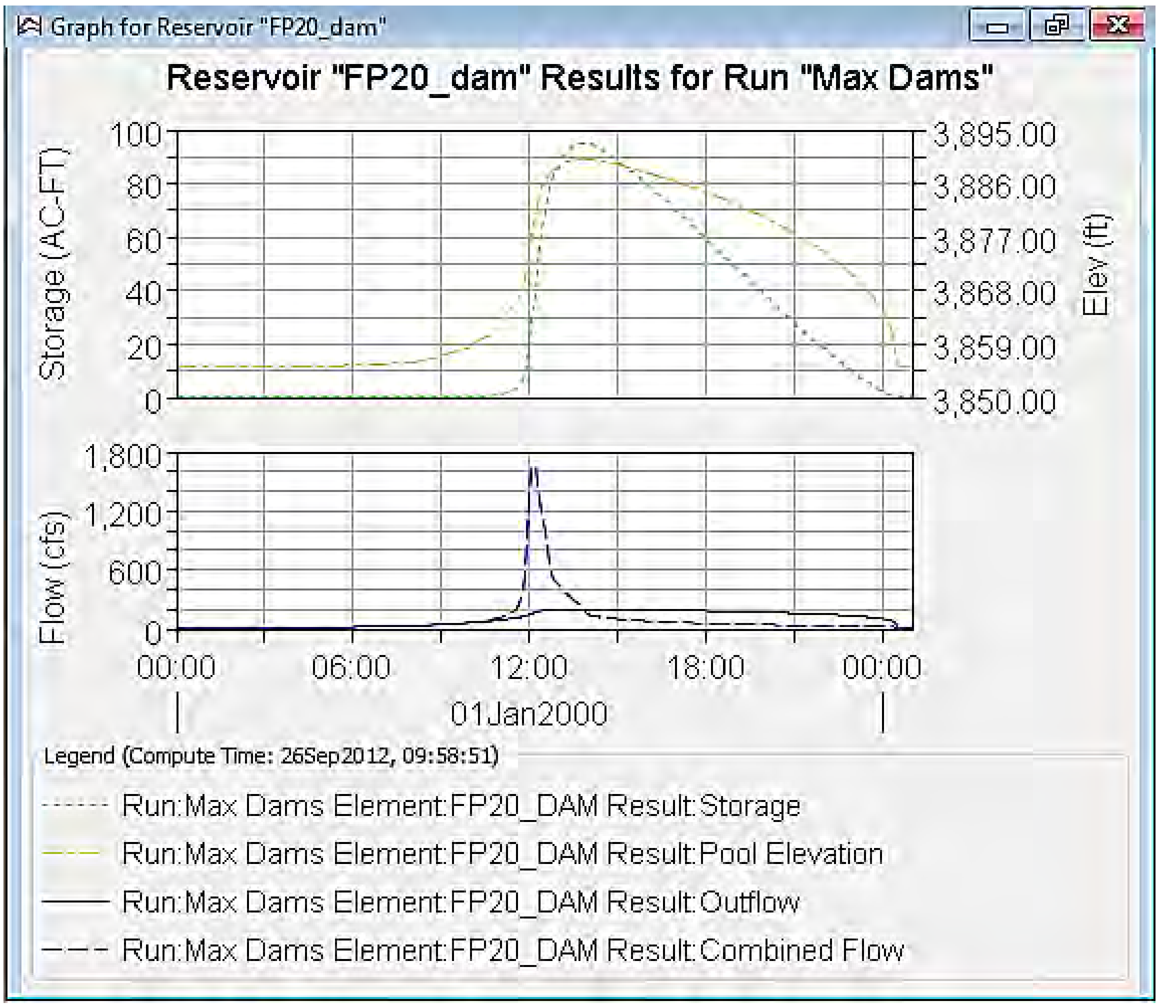
The software features a completely integrated work environment including a database, data entry utilities, computation engine, and results reporting tools. A graphical user interface allows the user seamless movement between the different parts of the software. Simulation results are stored in HEC-DSS (Data Storage System) and can be used in conjunction with other software for studies of water availability, urban drainage, flow forecasting, future urbanization impact, reservoir spillway design, flood damage reduction, floodplain regulation, and systems operation.

- **Management**
    - It runs on the latest MS Window operating systems
    - It is supported by the US Army Corps of Engineers through their Hydrologic Engineering Center and continues to be enhanced
    - It is free, and does not require extra licensing
    - It is accepted by FEMA, the EPA, the New Mexico State Engineer’s Office as well as most federal water resource agencies
    - It runs on multiple operating systems and does not require a DOS to work (no special computers required) (Windows, Solaris & Linux)
    - It is supported by an extensive and easy-to-follow set of helpful documents:
        - Quick Start Guide,
        - User’s Manual,
        - Application Guide, and
        - Technical Reference Manual
- **Training**
    - It is taught at nearly all the engineering schools for modeling hydrology
    - Classes are regularly offered for HEC-HMS by ASCE and others
    - It is based on a graphical user interface, and can be used in conjunction with GIS data files



- **Usability**
    - It is very easy to see if the model balances (all sub-basins are connected downstream)
    - Time steps can be adjusted very quickly
    - Importing and Exporting hydrographs is simple using Excel
    - Study points can be simply added or removed while modeling
    - Output files can be both tabular and graphical (which can be used in reports and excel)
- **Modeling**
    - Rainfall data can be input directly from NOAA atlas 14
    - It can run various hydrologic rainfall/runoff methods (initial constant, exponential, Green/Ampt, TR-55 (SCS Curve Number), etc) and is very easy to change canopy and percent imperviousness in sub basins (very important when modeling the smaller storms)
    - Seven methods are available for transforming excess precipitation to surface runoff (including Clark, Snyder and SCS techniques)

- It's possible to model large complex watersheds to see how hydrograph timing affects storm water facilities
- Dividing and adding hydrographs is simple to use and very intuitive
- It has a very robust (i.e. lots of options) reservoir/pond modeling routine that allows for relatively simple alternative development and testing
- It simply interfaces with USACE's Riverine Analysis System program HEC-RAS



Typical Output from HEC-HMS Dam Routing

- **Specifics to the Albuquerque Area**
    - The dam breach routine is very straight forward and is the recommended method by the NM OSE Dam Safety Bureau for use in analyses, design and inundation mapping for EAP's.
    - Channel loses can be modeled (important for natural arroyos)
    - It has 6 different methods to route hydrographs (attenuating flows), important when modeling a diverse landscape like the Albuquerque area (flat valley to the steep NE heights).
    - Pumps can be simulated for interior drainage areas and pump controls can be linked to either reservoir (pond) level or to stage in outfall channel.
- **Meteorology Description**
    - There are multiple methods for modeling both historic and simulated rainfall events
- **Hydrologic Simulation**
    - Flexible output - tables, graphs and basin map in user selected formats
- **Sediment and Water Quality**
    - Erosion estimates using MUCLE for both natural and urban environments
    - Channel erosion, deposition and sediment transport can be added reach by reach
    - Sediment settling in ponds and reservoirs and be included
    - Nutrient transformations and transport can be modeled
- **GIS Connection**
    - Companion program HEC-GeoHMS (also free and downloadable) can be used to create basin and meteorologic models for use in HMS
    - Basin and sub-basin boundaries, soils and land use data can be established by use of GIS

For more information go to: http://www.hec.usace.army.mil/software/hec-hms/

URBAN WATERSHED MODELING WITH HYMO

By Charles M. Easterling

ABSTRACT: "HYMO", the Agricultural Research Service's computer program for hydrologic modeling agricultural watersheds was modified and successfully used to model a complex urban watershed. Pervious and impervious area runoff hydrographs were computed, added together and routed overland, down streets and channels and through storm sewers. A new method for using the Soil Conservation Service's Runoff Curve Number procedure is presented.

INTRODUCTION

In February 1977 the City of Albuquerque awarded a contract for a Master Drainage Study of a twenty-two square mile area of urban Albuquerque. The goals of the study were to determine the existing storm drainage situation and to assess the adequacy of the existing storm drainage facilities. In addition, a computer model of the area was to be provided inorder that planned drainage facilities could be tested prior to construction and that the impacts of proposed storm drainage policy changes could be determined. The search for an existing computer model was begun immediately. The criteria for selection were applicability to the area, accessability of the source deck, core requirements of the program and ease of operations.

It was found that each of the urban models examined required a compromising of the selection criteria. It was therefore decided to develop a new program or modify an existing one. Of the models examined, the Agricultural Research Services (1) HYMO appeared to be the most flexible and adaptable.

THE ORIGINAL "HYMO"

HYMO was originally developed for hydrologic modeling of agricultural watersheds. The program develops incremental runoff volumes by applying the Soil Conservation Service's (SCS) "Runoff Curve Number" (2) (3) procedure to a mass rainfall table which is supplied by the user. Unit hydrographs are computed from several empirically derived equations which utilize the watershed parameters area, length, width and slope. The unit hydrographs are then combined with the derived excess rainfall to produce runoff hydrographs.

The analysis scheme follows that of other conventional upland watershed models. A runoff hydrograph from the uppermost sub-watershed is computed, then routed through a stream or channel. A runoff hydrograph is then computed for the intervening area and added to the routed hydrograph from above. That combined hydrograph is then routed and the process is continued down the watershed. Flood routing in HYMO is performed using the "Variable Storage Coefficient method" (5) which accounts for the variation

Note. -- Charles M. Easterling is currently serving as Department Head, Water Resources Department, Bohannan–Huston, Inc., Consulting Engineers, Planners and Photogrammetrists – Albuquerque, New Mexico

FIG. 1. — Study Area

Scale 1" = 3000' Approx. Contour Interval = 10' in the water surface slope during the passage of the flood wave. Routing hydraulics are based on user supplied rating tables or normal depth computations utilizing user supplied stream cross-sections.

THE STUDY AREA

The area studied is densely developed with a mixture of residential, commercial and light industrial uses. The topography typifies that of southwestern alluvial fans with many long, narrow, parallel watersheds and slopes in the 1 - 4% range (Figure I). The area contains eighteen major storm sewer systems which run generally down the slope but often transport runoff from one of the forty separate watersheds to another. Also, in some instances, storm sewers decrease in capacity in the downstream direction.

The soils in the area are deep silty-sands and are classified by the SCS as belonging to hydrologic group B. The land use in the area resulted in precent impervious figures ranging from ten to eighty percent. The area had been recently mapped on ortho photo-based topographic maps at 1" = 200' with a two foot contour interval from which a majority of the input data was collected.

MODIFICATIONS TO HYMO

The program was modified in stages to meet several specific needs. The first stage was to calibrate the hydrograph portion of the model. As there are no reliable streamflow—rainfall network gaging systems in the area the model was calibrated against other emperical methods. Of those tried, the SCS method as presented in chapter 16 of the National Engineering Handbook (2) gave the most consistent results for the small watersheds in the study area. The model's unit hydrograph was forced toward the standard unit graph shape used by the SCS. Later in the study, results were compared to those of other studies preformed in the area. In most instances, the results were in close agreement.

The second stage was to develop a means of dealing with combined storm sewer and overland flows. A new routine was added which allowed hydrographs to be divided into two parts. One part with an upper limit discharge to model storm sewer interception (Figure 2) and the remainder to be routed overland and/or down streets and channels. Each of the 300 flow rates in hydrographs is associated with its proper elapsed time.

The third stage was to redimensionalize the model to accommodate more than six hydrographs due to complex import-export of storm runoff between basins. HYMO will now accept up to fifty distinct hydrographs although only two are ever in core at a time due to extensive use of disk storage.

The final stage of modifications to HYMO was to allow the program to perform its flood routing operations on a hydrograph traveling in a storm sewer (Figure 3). This was accomplished by modifying the "Compute Rating Curve" routine to accept and utilize user input storm sewer parameters in its computation of normal depth for routing. The input parameters are pipe diameter, slope and Manning's n. Losses and surcharge can be modeled by varying the roughness coefficient or the friction slope or both at the user's descretion. In this study, the slopes were sufficiently steep to assume upstream hydraulic control in storm sewers, thus simplifying the analyses.

THE STUDY PERFORMED

Once the modifications to the model had been completed, the next task was to construct a detailed plan of operation inorder that several people could work on the study simultaneously and produce compatible results. This was necessary due to the inter-

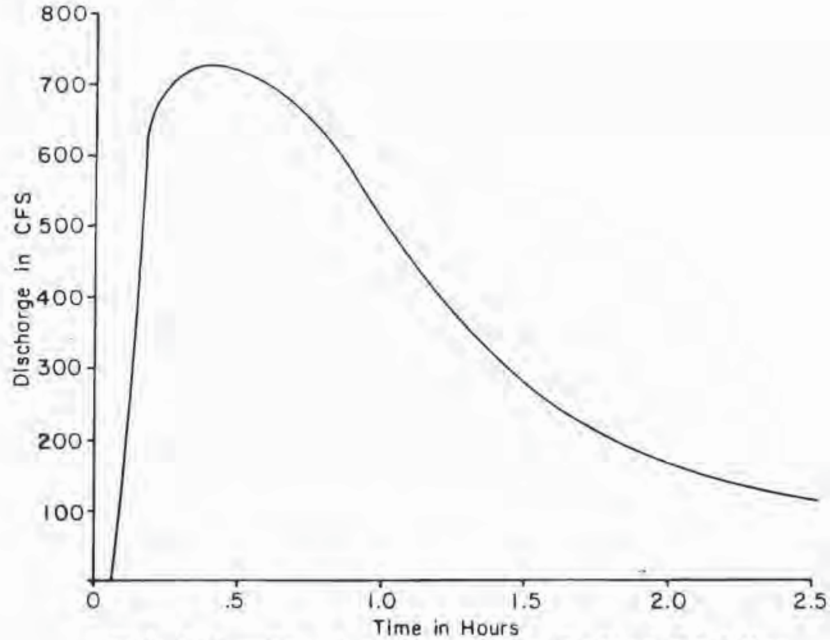


FIG. 2A.—Total Hydrograph Before Dividing

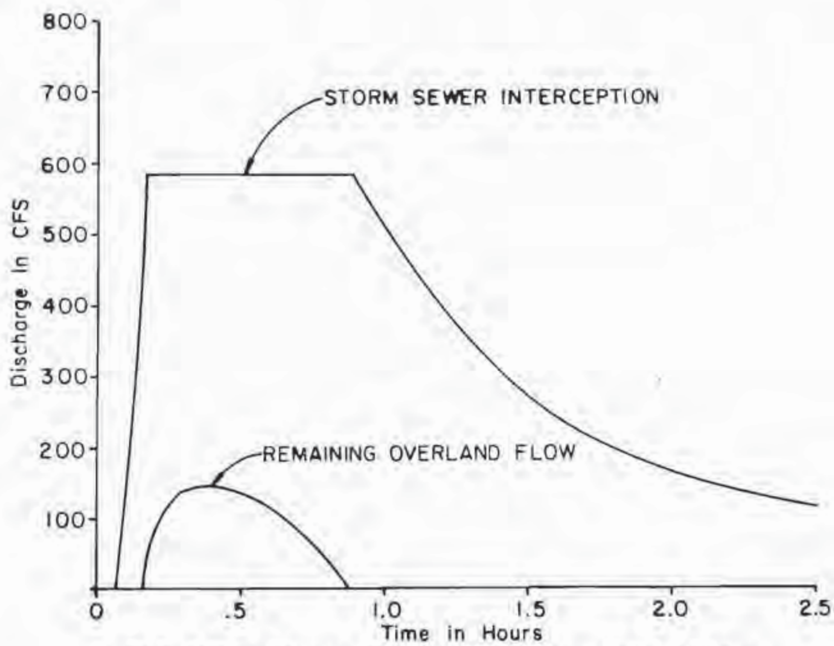


FIG. 2B.—Storm Sewer and Overland Flow Hydrographs

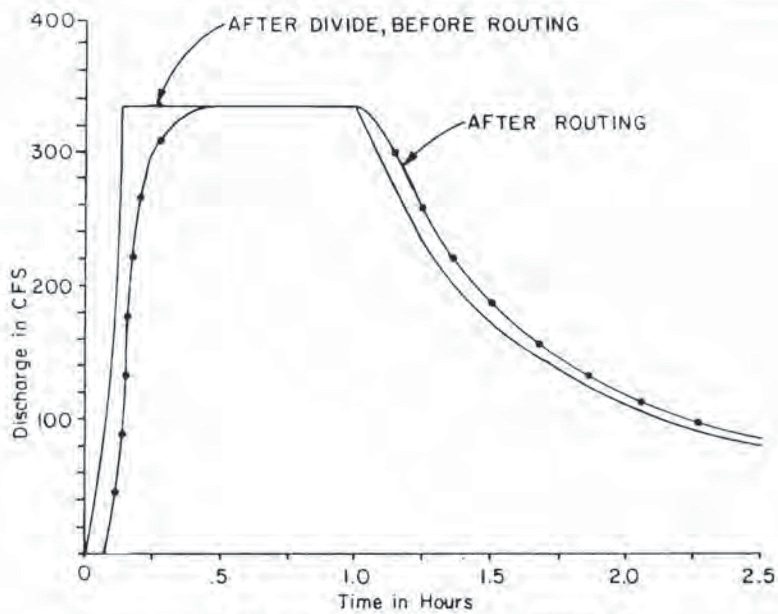


FIG. 3.—Storm Sewer Routing

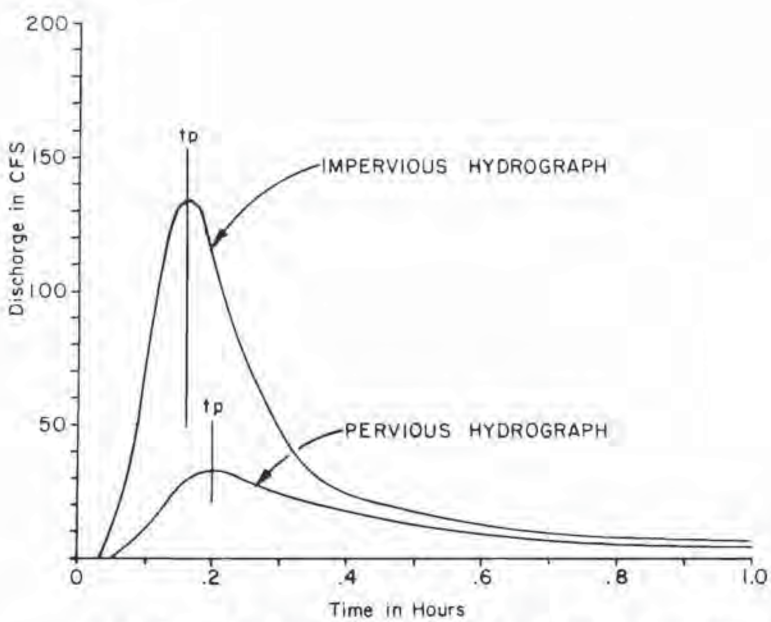


FIG. 4.—Pervious and Impervious Hydrographs

mingling of storm waters between parallel watersheds.

As mentioned above, the SCS's "Runoff Curve Number Method" was used to develop runoff volumes. Due to the flexibility of the model it was decided to try a new approach. Rather than use either of the methods proposed by the SCS to arrive at the runoff volume for a non-homogeneons watershed, each sub-basin was separated into two distinct basins, one pervious and one impervious, overlaying one another. The time of concentration used for each part was that of the composite sub-basin before separating. The orginal intent was to allow the impervious portion of the sub-basins to generate its share of the total runoff early in the design storm while the pervious portion would respond more slowly as in nature. (Figure 4) It was felt that this early developing runoff from streets and roofs has the most significant impact on storm drainage facilities. Care was taken that the total volume of runoff did not vary from that expected using the accepted "Composite Curve Number Method" (2).

As shown in Figure 5, the resultant combined hydrograph peaks sooner, as expected. However, the large difference in peak flow rates was not expected. The question of which is more nearly correct has not been satisfactorily resolved but the combined hydrograph method has a better empirical basis because of the way inwhich the runoff curve numbers were originally determined by the SCS. Further, the combined hydrographs were found to be more intuitively acceptable and thus were used throughout the study effort.

These pervious and impervious hydrographs were then combined to form the total runoff hydrograph from that sub-basin. Assuming a sub-basin near the middle hydraulically in the watershed, the next step was to add this intervening hydrograph to the hydrograph which had been routed overland and underground from upstream. Once combined, this new total hydrograph was then redivided into two or more parts. The storm sewer interception was taken directly from each hydrograph point depending on the capacity of the storm sewer. The remaining overland flow hydrograph was then divided on a percentage basis depending on the configuration of the streets and/or channels which transport it downstream to the next analysis point. These two or more hydrographs were then routed in the storm sewer and above ground respectively and recombined downstream. (Figure 6) An unexpected benefit resulted in the use of this operational scheme. In those instances where storm sewer capacity decreased in the downstream direction, that portion which the storm sewer would no longer accommodate was automatically added back to the overland flow hydrograph, thus modeling discharging inlets and manholes, and retaining the hydrograph shapes at the same time.

The entire study area was modeled and run as one watershed to predict the complex interactions of the forty separate watersheds and eighteen major storm sewer systems.

The peak flow rates and flow depths in streets throughout the study area were then tabulated and the information transfered to the photo-based maps. Areas where flows overtopped street curbs, flowed overland or ponded, were defined as flood hazard areas and were candidates for structural improvements. Again, beginning in the uppermost sub-basins in each watershed, structural improvements (mostly storm sewers) were added to the model and then tested for design adequacy. This operation progressed downstream until all flood hazard areas had been eliminated. It is interesting to note that minor improvements upstream of some significant flood hazard areas unexpectedly removed those flood hazard areas at substantial savings over treating the problem at its downstream location.

The results obtained from the modeling were checked in the field during several timely rainstorms which occurred during the study. Although the storm intensities and volumes were less than those of the design storms used in the modeling, many flood hazard areas

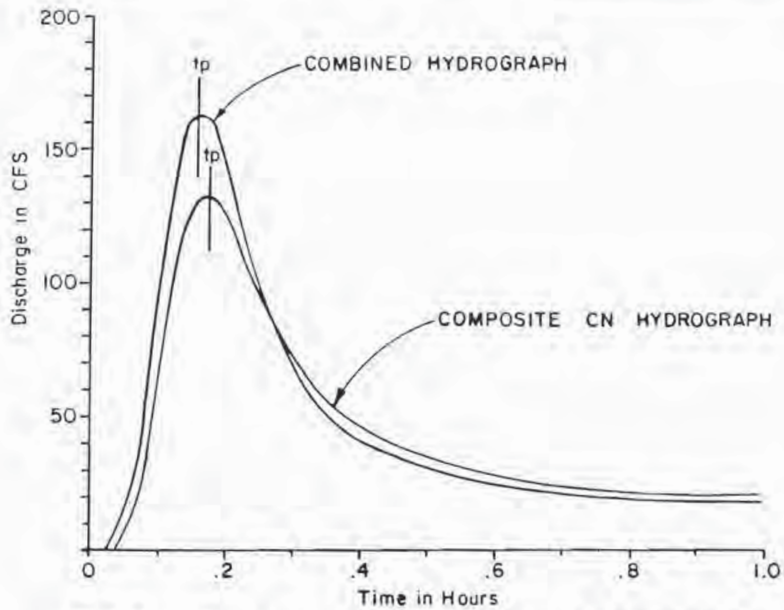


FIG. 5.—Combined Hydrograph vs. Composite CN Hydrograph

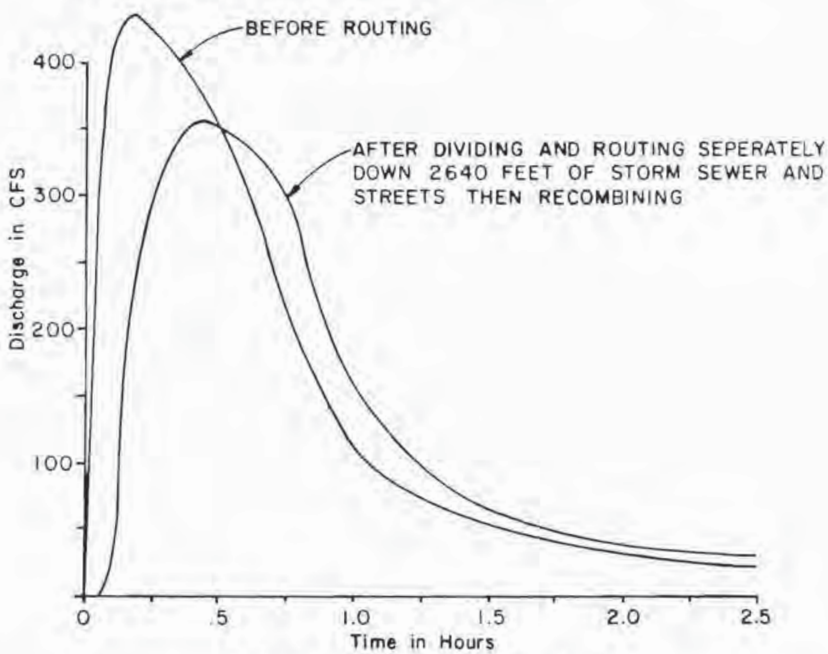


FIG. 6.—Combined Routing

were accurately predicted. The results were also checked against historical flood events and were found to be in close agreement.

SUMMARY

HYMO, modified for urban use can be easily calibrated to fit gaged or empirical hydrologic data. The program with adaptations, is simple to use, gives reasonable results and filled a specific need for modeling upland urban drainage. Such a model will be useful to owners and operators of storm drainage systems in much the same way as digital and analog models of water distribution systems are for predicting and locating problems in the design or operation of their systems and for the testing of solutions for feasibility and cost effectiveness.

APPENDIX – REFERENCES

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APPENDIX C

RAINFALL

Rainfall Distribution Sensitivity Analysis

Selecting a Distribution

Easterling Consultants LLC

September 29, 2014

Introduction

In recent years, several HEC-HMS based watershed studies have been commissioned and approved by AMAFCA and SSCAFCA. The methodology used was often designed in such a way that the results from HEC-HMS would mimic those from community standard AHYMO_97 in terms of computing initial abstractions, infiltration rates while using the traditional AHYMO rainfall distribution. Given that AHYMO '97 has been found to have serious internal flood routing routine problems, does not run with current MS Windows operating systems and is not universally used or understood, the community is contemplating migrating to TR-55 Hydrology within HEC-HMS. A group of local experienced hydrologists were assembled by AMAFCA to review this process and after considering the issues, asked to see how various rainfall distributions would affect the net results for runoff volumes and peak discharges.

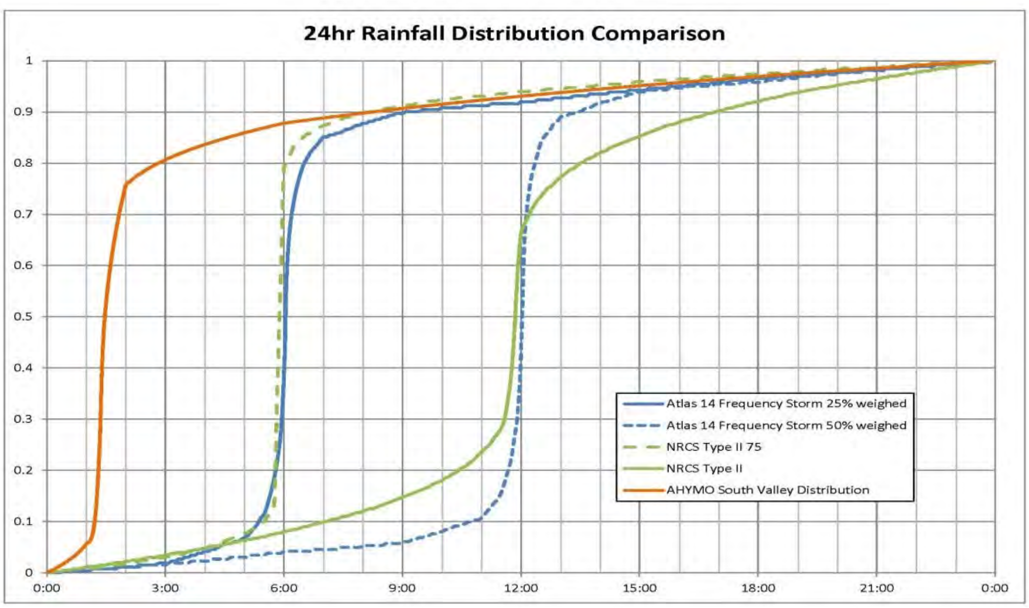
Easterling Consultants was tasked with conducting a sensitivity analysis on the net effects on runoff volume and peak discharge using NOAA Atlas 14 rainfall amounts and several of the most promising and/or familiar rainfall distributions in available HEC-HMS models. The results would provide the technical committee a firm foundation for recommending a distribution to adopt as the standard for future hydrological studies done using HEC-HMS within the community.

Methodology

6 approved HEC-HMS models were taken and run with the following distributions:

- AHYMO distribution
- SCS Type II 75 distribution
- HEC-HMS Frequency Storm at 25% Intensity position
- HEC-HMS Frequency Storm at 33% Intensity position
- HEC-HMS Frequency Storm at 50 % Intensity position

The figure below illustrates the difference in the shape of each distribution.



The table below summarizes the various models run, the respective drainage areas and rainfall depths. Rainfall depths were determined at the centroid of each watershed using the NOAA Data Server.

Watershed Model	Drainage Area (acres)	100Yr-24Hr Rainfall Depth (in.)
Pilar (US HWY 68)	5582	3.21
Boca Negra	9014	2.4
Embudo	15564	2.91
Venada	10568	2.84
South Diversion	5338	2.6
Unser Diversion	119	2.52
Pino	3825	3.59

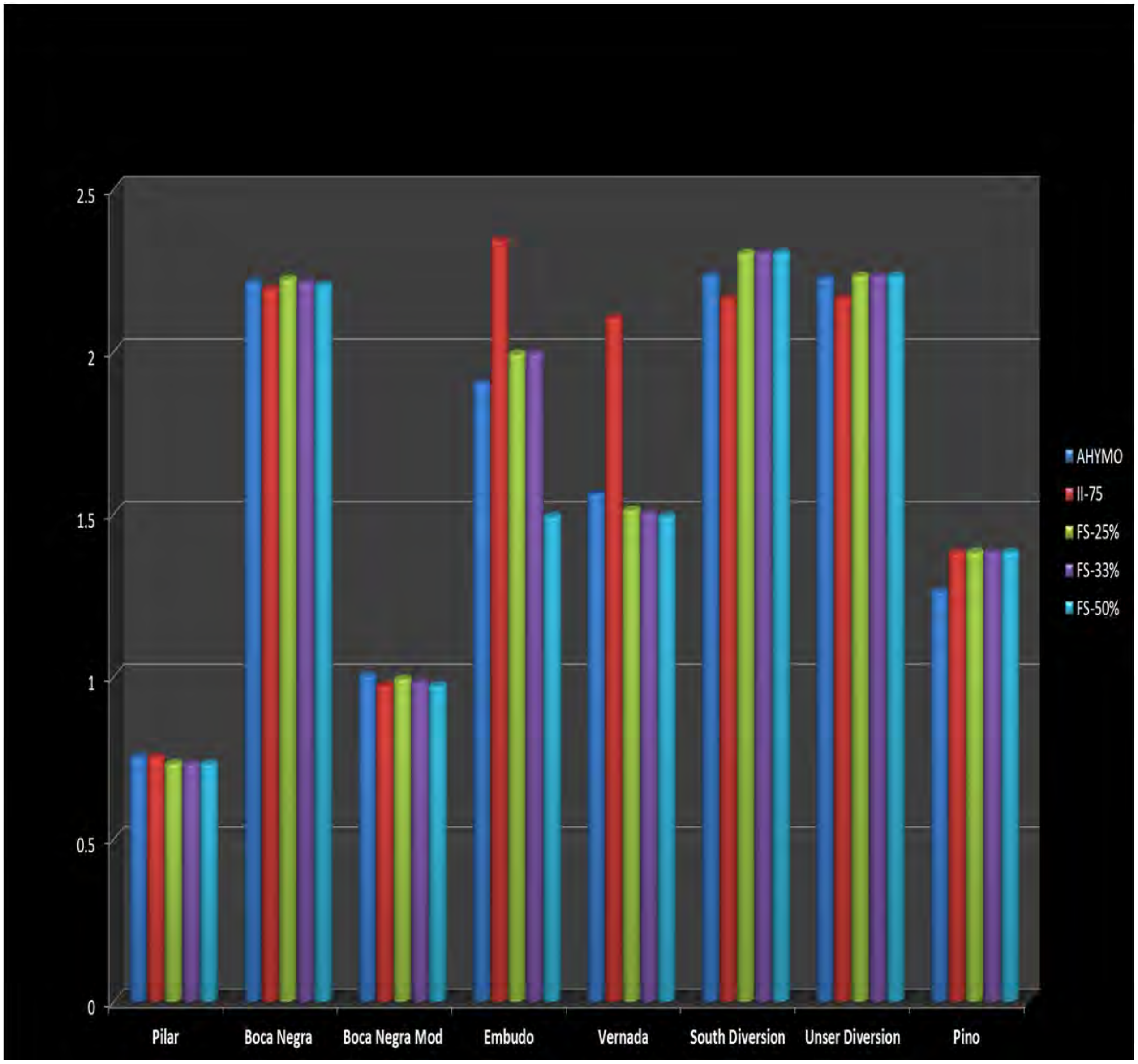
Note that the Pino Arroyo and the Boca Negra Models were developed for PMP analyses associated with dam design and evaluations. 100 year rainfall amounts from the NOAA Data Server were substituted for the PMP rainfall values in these models for this study. The Embudo, Venada and Pino models also used the unaltered SSCAFCA/City of Rio Rancho DMP hydrology method of determining Tc, while the Pilar, South Diversion Channel and Unser Diversion Channel models followed the NMDOT and Albuquerque DPM method. Additionally, the Boca Negra model, which was developed for dam spillway design and breach analyses, had an effective runoff curve number of 97, which was modified for these purposes to a more reasonable value for an urban area on the west side of Albuquerque.

Results

In general, the runoff volumes, and peak discharges were very similar. The NRCS Type II 75 distribution in the Embudo, Venada and Pino models generated results that did not fit well with the rest of the data set. That is pretty clear from the charts that are presented below and the reasons for that are addressed in the discussion section below.

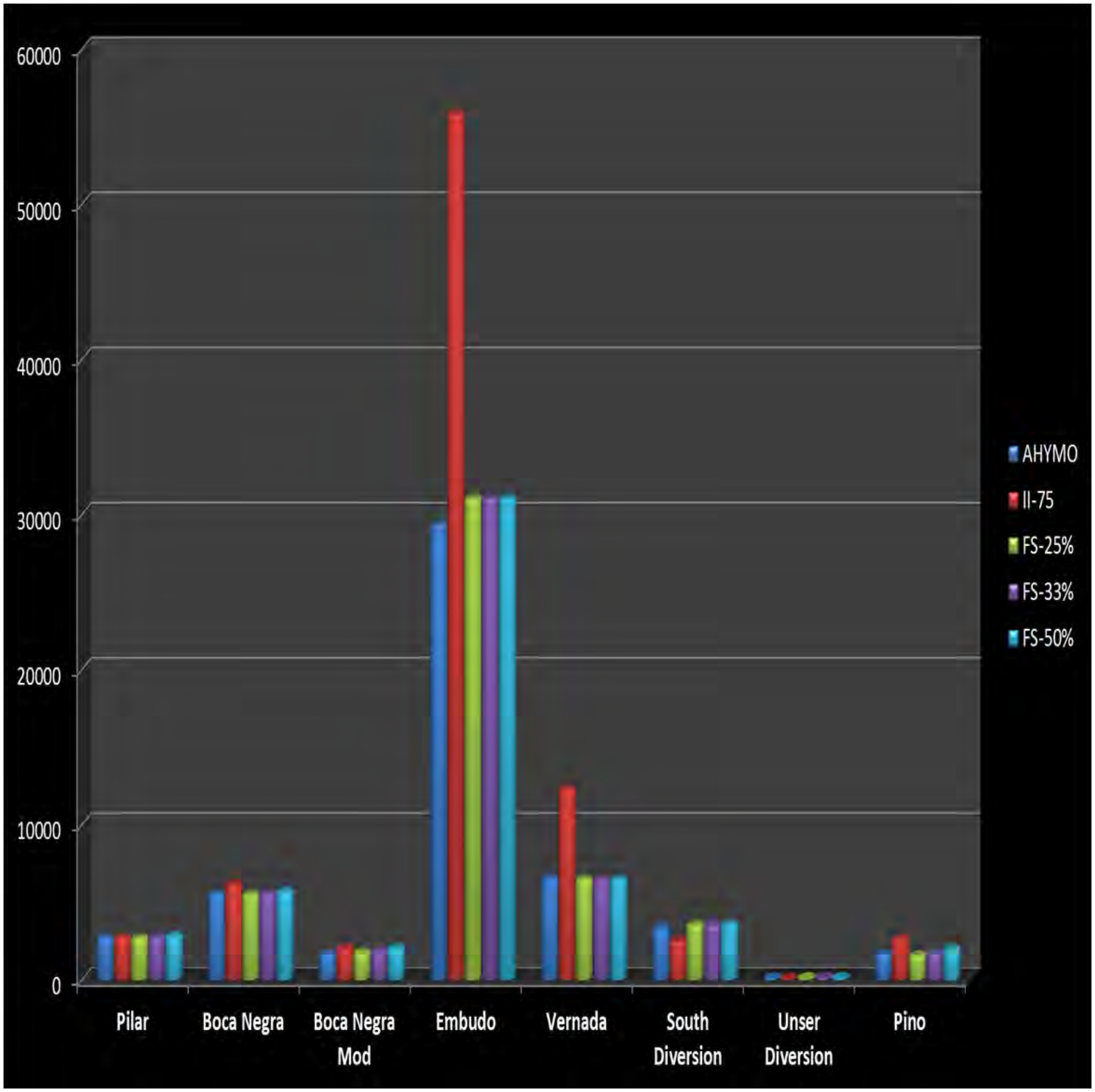
1. Runoff Volume Summary:

The chart below presents the difference in runoff volume (inches) for the various watersheds. With the exception of Embudo and Venada, the difference in volume is very small between the various distributions, as should be expected.



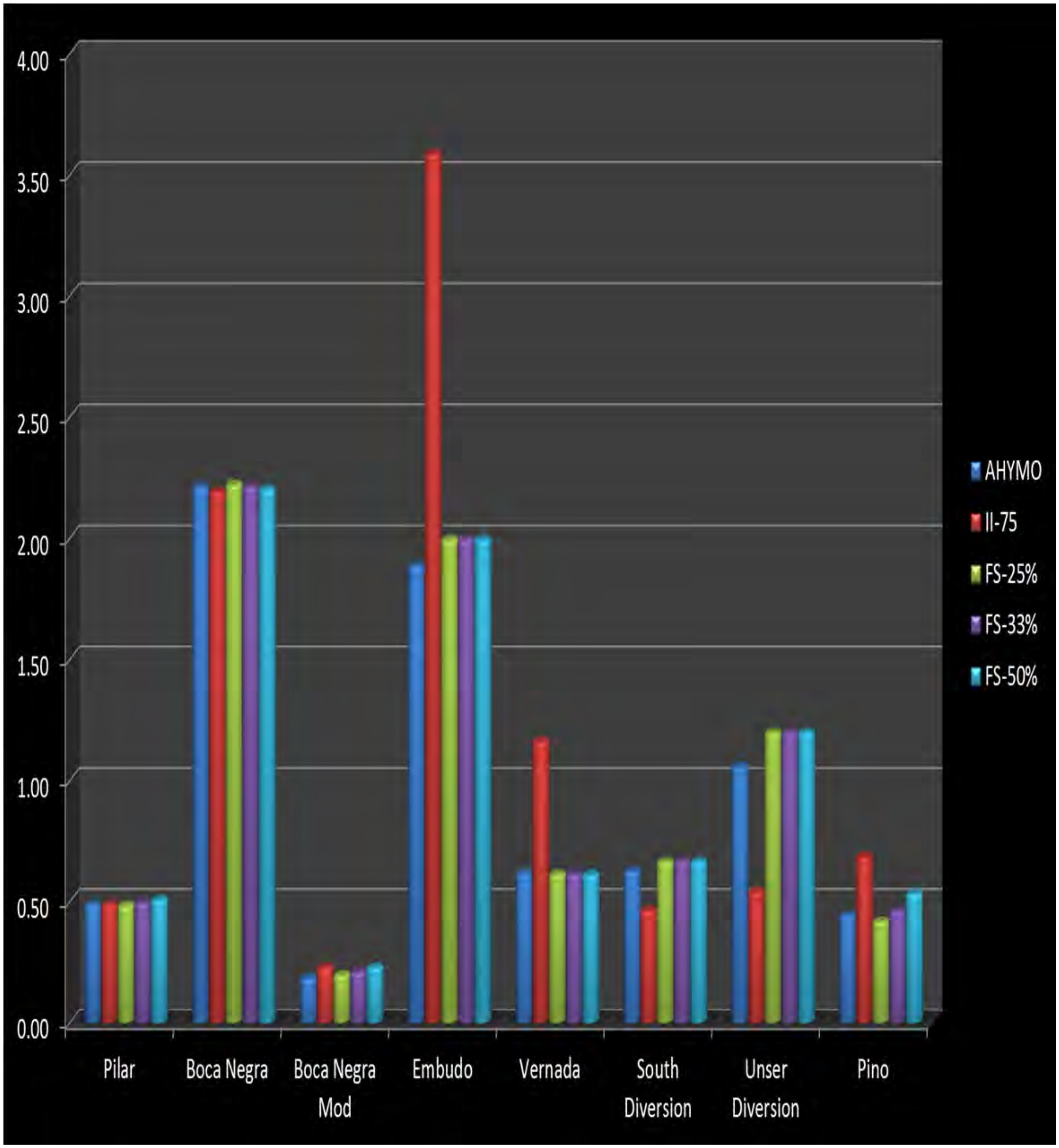
2. Peak Discharge Summary

The chart below summarizes the differences in peak discharge (cfs). Note that the watersheds vary over a relatively large range of sizes.



3. Peak Discharge/acre Summary (cfs/acre)

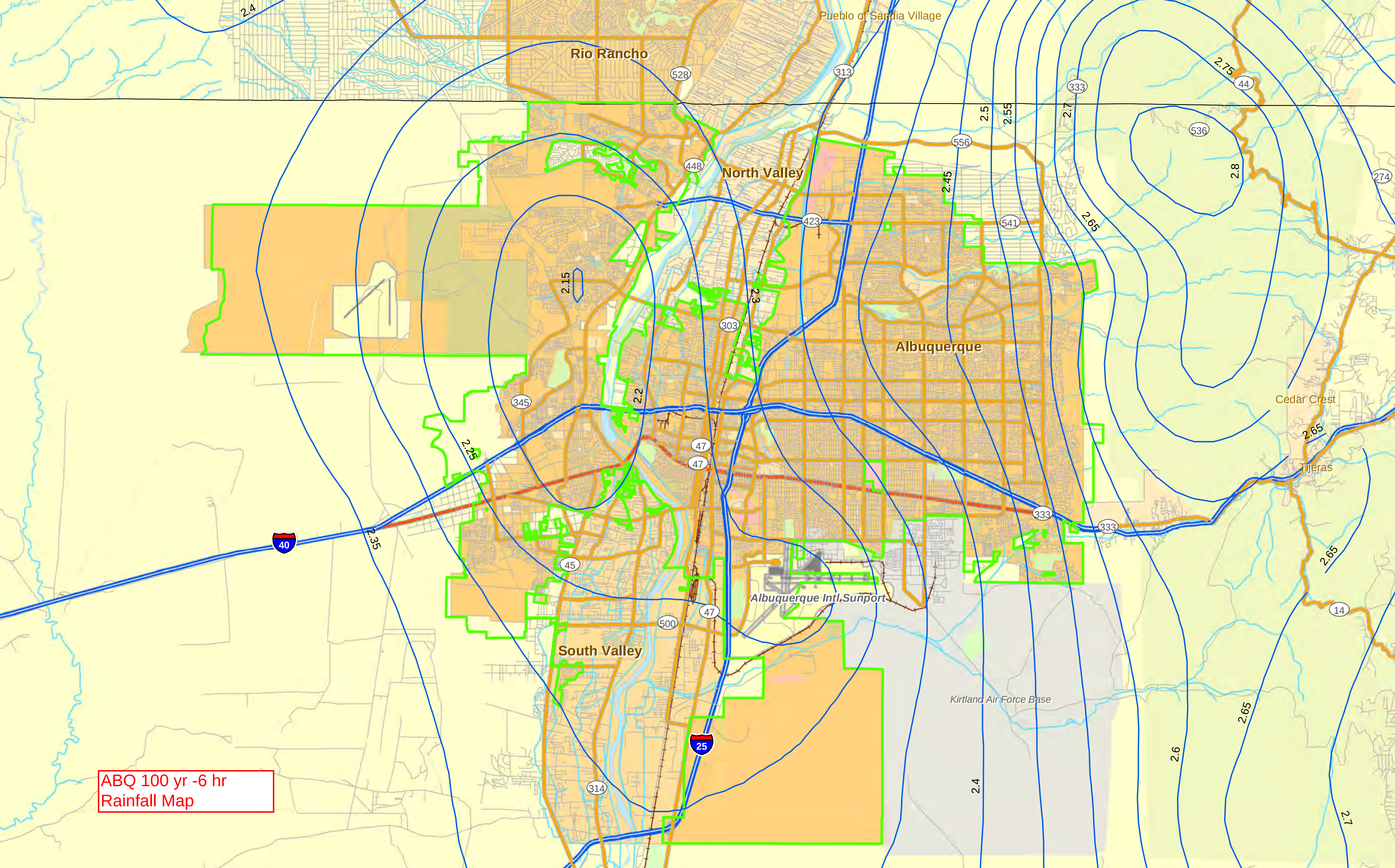
The chart below summarizes the model results for the various distributions when adjusted for size to cfs/acre.



Discussion/Conclusion

After completing the various modeling scenarios, it could be seen that, for the most part, the rainfall distribution does not make a significant impact on the runoff volumes or peak discharges. The three models that produced results that did not fit well with the rest of the data set when using the NRCS Type II 75 distribution were all based on the SSCAFCA/City of Rio Rancho DPM methodology which is based on the Clark's Unit Hydrograph Method and an adjusted definition of Tc. The peak discharges were noticeably higher than the other models. This may be largely attributed to the way certain parameters are modeled within HMS (such the infiltration rates) or the manner in which some of these parameters are computed. Upon closer examination, the Type II 75 rainfall distribution contains a 15 minute period which produces 2.37 inches of rainfall (66% of the total storm rainfall at 9.48 in/hr), an unusually high value. In comparison, the 1000 year 15 minute rainfall from NOAA Atlas 14 is 1.67 inches (6.68 in/hr) which should explain the significantly higher peak discharges in those models that used the Clark rainfall/runoff transform . Another factor that appears to be significant is that the rainfall/runoff method in AHYMO '97 and in the SSCAFCA/City of Rio Rancho DPM is based on the relationship of rainfall to runoff at a point and incorporates a infiltration loss rate that is independent of the rainfall rate, where the TR-55 Curve Number Method is watershed based and varies with rainfall rate. See accompanying papers "*Runoff Curve Number Method: Beyond the Handbook*" by USDA ARS and NRCS and "*Runoff Curve Number: Has it Reached Maturity*? By Ponce and Hawkins".

Based on these analyses and the foregoing discussion, we recommend that the community adopt the 25 % Frequency Distribution as the standard. However, as the Pino Model demonstrates, there may be watersheds that, due to their size, geometry and combination of detention and conveyance facilities that a storm distribution different from the 25% will be the most critical. Since the effort to evaluate different rainfall distributions within HEC-HMS is minimal, it is further recommended that when sizing large and capital intensive drainage and flood control infrastructure (dams and bridges for example) the 33% and 50% distribution be analyzed as well. For hydrologic evaluations related to large scale drainage master plans, CLOMR's and LOMR's, we recommend that the 25% distribution be the standard to keep things as simple in the review process as possible. In other words, use of distributions other than the 25% should be rare, circumstance driven, and only considered on a case by case basis.


ABQ 100 yr -6 hr
Rainfall Map
Rio Rancho
North Valley
Albuquerque
South Valley
Pueblo of Sandia Village
Cedar Crest
Tijeras
Albuquerque Intl Sunport
Kirtland Air Force Base
2.15
2.2
2.25
2.3
2.35
2.4
2.45
2.5
2.55
2.6
2.65
2.7
2.75
2.8
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25
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528
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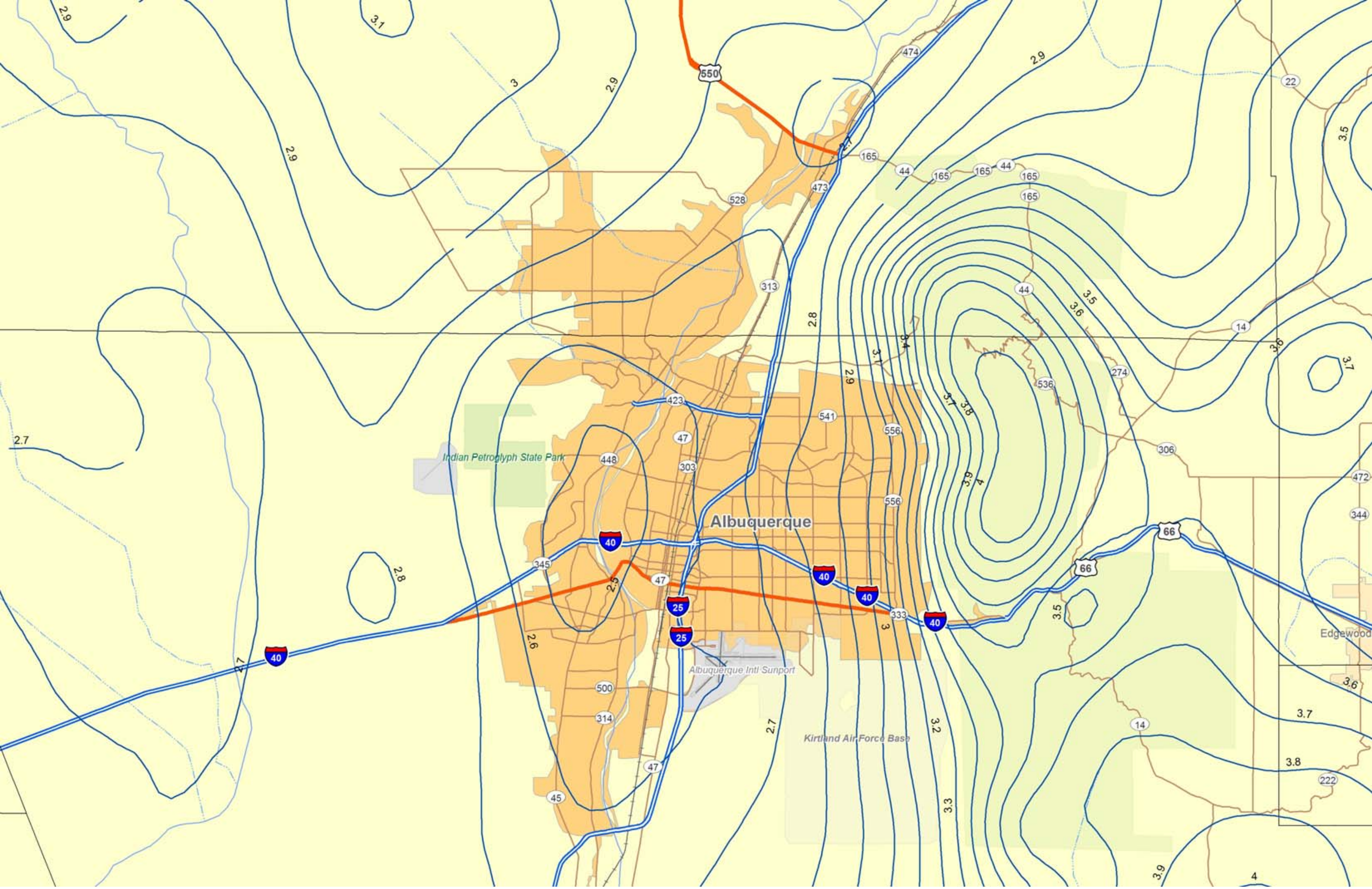
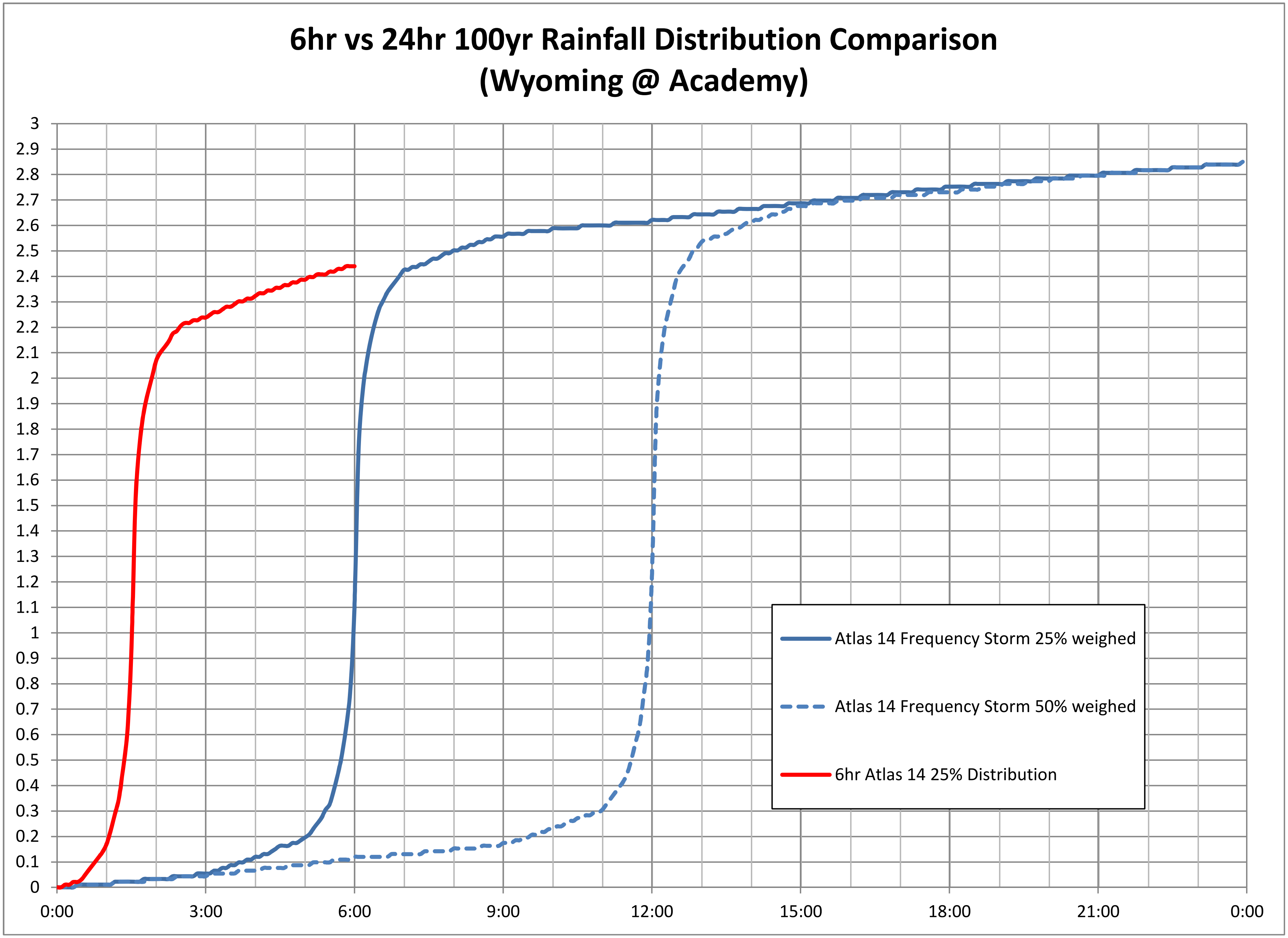
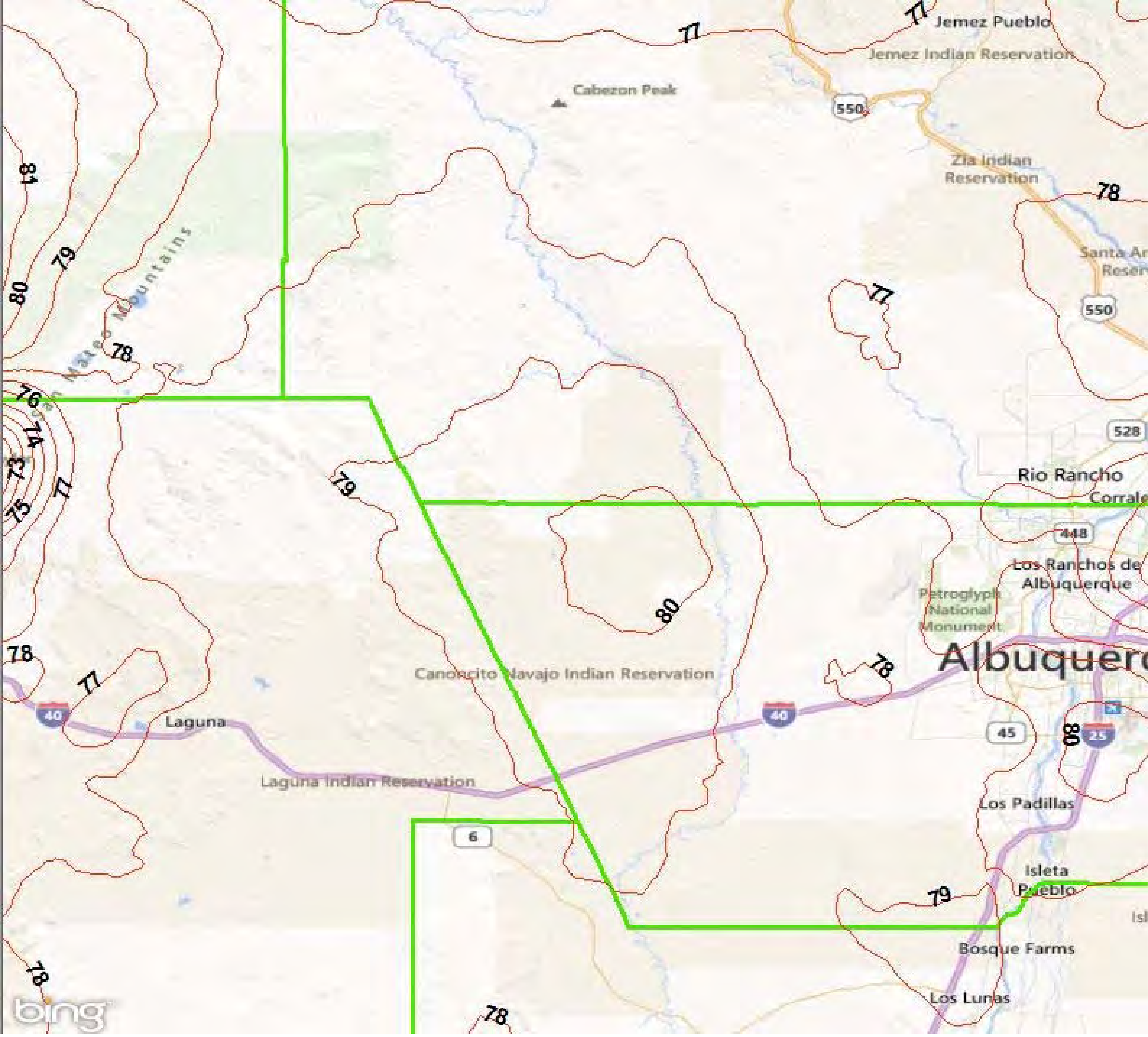
556
541
333
536
44
274
14



ABQ 100 yr - 24 hr
Rainfall


6hr vs 24hr 100yr Rainfall Distribution Comparison
(Wyoming @ Academy)
3
2.9
2.8
2.7
2.6
2.5
2.4
2.3
2.2
2.1
2
1.9
1.8
1.7
1.6
1.5
1.4
1.3
1.2
1.1
1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
0:00
3:00
6:00
9:00
12:00
15:00
18:00
21:00
0:00
Atlas 14 Frequency Storm 25% weighed
Atlas 14 Frequency Storm 50% weighed
6hr Atlas 14 25% Distribution



Relationship of 100 yr 1 hr to 6 hr rainfall

NOAA Atlas 14, Volume 1, Version 5
Location name: Albuquerque, New Mexico, US*
Coordinates: 34.9530, -106.6625
Elevation: 5024 ft*
* source: Google Maps

POINT PRECIPITATION FREQUENCY ESTIMATES

Sanja Perica, Sarah Dietz, Sarah Heim, Lillian Hiner, Kazungu Maitaria, Deborah Martin, Sandra Pavlovic, Ishani Roy, Carl Trypaluk, Dale Unruh, Fenglin Yan, Michael Yekta, Tan Zhao, Geoffrey Bonnin, Daniel Brewer, Li-Chuan Chen, Tye Parzybok, John Yarchoan

NOAA, National Weather Service, Silver Spring, Maryland

PF_tabular | PF_graphical | Maps_&_aerials

PF tabular

PDS-based point precipitation frequency estimates with 90% confidence intervals (in inches)[1]

Duration	Average recurrence interval (years)									
	1	2	5	10	25	50	100	200	500	1000
5-min	0.178 (0.154-0.207)	0.231 (0.198-0.268)	0.309 (0.265-0.359)	0.370 (0.316-0.428)	0.452 (0.385-0.523)	0.516 (0.438-0.597)	0.584 (0.491-0.675)	0.655 (0.547-0.756)	0.750 (0.621-0.868)	0.827 (0.680-0.957)
10-min	0.272 (0.234-0.315)	0.351 (0.302-0.408)	0.470 (0.404-0.546)	0.562 (0.481-0.651)	0.688 (0.586-0.796)	0.786 (0.666-0.909)	0.889 (0.748-1.03)	0.997 (0.833-1.15)	1.14 (0.945-1.32)	1.26 (1.03-1.46)
15-min	0.337 (0.290-0.390)	0.436 (0.374-0.506)	0.583 (0.500-0.677)	0.697 (0.596-0.807)	0.853 (0.727-0.987)	0.974 (0.825-1.13)	1.10 (0.927-1.27)	1.24 (1.03-1.43)	1.42 (1.17-1.64)	1.56 (1.28-1.81)
30-min	0.453 (0.390-0.525)	0.587 (0.504-0.681)	0.785 (0.674-0.912)	0.939 (0.802-1.09)	1.15 (0.978-1.33)	1.31 (1.11-1.52)	1.48 (1.25-1.72)	1.66 (1.39-1.92)	1.91 (1.58-2.21)	2.10 (1.73-2.43)
60-min	0.561 (0.483-0.650)	0.726 (0.624-0.843)	0.972 (0.834-1.13)	1.16 (0.993-1.34)	1.42 (1.21-1.64)	1.62 (1.38-1.88)	1.84 (1.54-2.12)	2.06 (1.72-2.38)	2.36 (1.95-2.73)	2.60 (2.14-3.01)
2-hr	0.641 (0.551-0.760)	0.821 (0.703-0.973)	1.08 (0.928-1.28)	1.30 (1.10-1.52)	1.59 (1.34-1.86)	1.82 (1.53-2.14)	2.07 (1.73-2.43)	2.33 (1.93-2.73)	2.70 (2.21-3.15)	3.00 (2.43-3.51)
3-hr	0.681 (0.589-0.803)	0.865 (0.746-1.02)	1.13 (0.977-1.33)	1.34 (1.15-1.58)	1.64 (1.40-1.92)	1.88 (1.59-2.19)	2.13 (1.79-2.48)	2.40 (2.00-2.80)	2.77 (2.28-3.23)	3.07 (2.50-3.59)
6-hr	0.789 (0.688-0.923)	0.994 (0.867-1.16)	1.28 (1.12-1.49)	1.50 (1.31-1.75)	1.81 (1.57-2.10)	2.05 (1.76-2.38)	2.30 (1.97-2.67)	2.56 (2.18-2.98)	2.93 (2.46-3.40)	3.22 (2.68-3.75)
12-hr	0.874 (0.768-0.999)	1.10 (0.969-1.26)	1.39 (1.22-1.59)	1.62 (1.42-1.85)	1.93 (1.69-2.20)	2.17 (1.88-2.47)	2.42 (2.09-2.75)	2.68 (2.29-3.05)	3.02 (2.57-3.45)	3.31 (2.78-3.78)
24-hr	0.986 (0.878-1.12)	1.24 (1.10-1.40)	1.54 (1.37-1.75)	1.79 (1.59-2.02)	2.12 (1.87-2.39)	2.37 (2.09-2.67)	2.63 (2.32-2.97)	2.90 (2.54-3.26)	3.26 (2.83-3.67)	3.54 (3.06-3.98)
2-day	1.04 (0.934-1.17)	1.30 (1.17-1.46)	1.62 (1.46-1.81)	1.88 (1.68-2.09)	2.21 (1.98-2.47)	2.47 (2.20-2.75)	2.74 (2.43-3.05)	3.00 (2.65-3.35)	3.36 (2.96-3.75)	3.63 (3.18-4.06)
3-day	1.13 (1.03-1.25)	1.41 (1.28-1.55)	1.74 (1.58-1.91)	1.99 (1.81-2.19)	2.34 (2.12-2.57)	2.60 (2.35-2.85)	2.87 (2.59-3.15)	3.13 (2.82-3.44)	3.48 (3.12-3.83)	3.75 (3.34-4.13)
4-day	1.22 (1.13-1.33)	1.51 (1.40-1.65)	1.85 (1.70-2.00)	2.11 (1.95-2.29)	2.46 (2.27-2.67)	2.73 (2.51-2.96)	3.00 (2.74-3.24)	3.26 (2.98-3.53)	3.60 (3.28-3.91)	3.86 (3.50-4.19)
7-day	1.41 (1.29-1.52)	1.74 (1.60-1.88)	2.11 (1.95-2.28)	2.40 (2.21-2.58)	2.77 (2.56-2.98)	3.04 (2.81-3.27)	3.32 (3.06-3.57)	3.58 (3.29-3.84)	3.90 (3.59-4.20)	4.14 (3.80-4.46)
10-day	1.54 (1.43-1.67)	1.91 (1.77-2.07)	2.33 (2.16-2.52)	2.66 (2.46-2.87)	3.09 (2.86-3.33)	3.41 (3.15-3.67)	3.73 (3.44-4.01)	4.04 (3.71-4.34)	4.44 (4.07-4.78)	4.73 (4.32-5.09)
20-day	1.95 (1.79-2.10)	2.41 (2.23-2.61)	2.92 (2.70-3.15)	3.30 (3.06-3.56)	3.79 (3.50-4.08)	4.14 (3.82-4.45)	4.47 (4.12-4.80)	4.79 (4.41-5.13)	5.17 (4.76-5.55)	5.44 (5.00-5.84)
30-day	2.31 (2.14-2.49)	2.87 (2.65-3.08)	3.44 (3.19-3.69)	3.86 (3.58-4.14)	4.38 (4.06-4.69)	4.75 (4.40-5.08)	5.10 (4.71-5.45)	5.42 (5.01-5.79)	5.80 (5.35-6.19)	6.05 (5.58-6.47)
45-day	2.81 (2.61-3.01)	3.47 (3.23-3.72)	4.12 (3.84-4.41)	4.58 (4.27-4.89)	5.13 (4.79-5.47)	5.50 (5.13-5.86)	5.83 (5.44-6.20)	6.11 (5.71-6.49)	6.40 (6.00-6.79)	6.56 (6.17-6.95)
60-day	3.24 (3.01-3.49)	4.01 (3.73-4.30)	4.76 (4.44-5.10)	5.30 (4.94-5.67)	5.95 (5.54-6.35)	6.38 (5.95-6.81)	6.77 (6.31-7.22)	7.11 (6.64-7.58)	7.48 (6.99-7.97)	7.70 (7.22-8.19)

[1] Precipitation frequency (PF) estimates in this table are based on frequency analysis of partial duration series (PDS).

Numbers in parenthesis are PF estimates at lower and upper bounds of the 90% confidence interval. The probability that precipitation frequency estimates (for a given duration and average recurrence interval) will be greater than the upper bound (or less than the lower bound) is 5%. Estimates at upper bounds are not checked against probable maximum precipitation (PMP) estimates and may be higher than currently valid PMP values.

Please refer to NOAA Atlas 14 document for more information.

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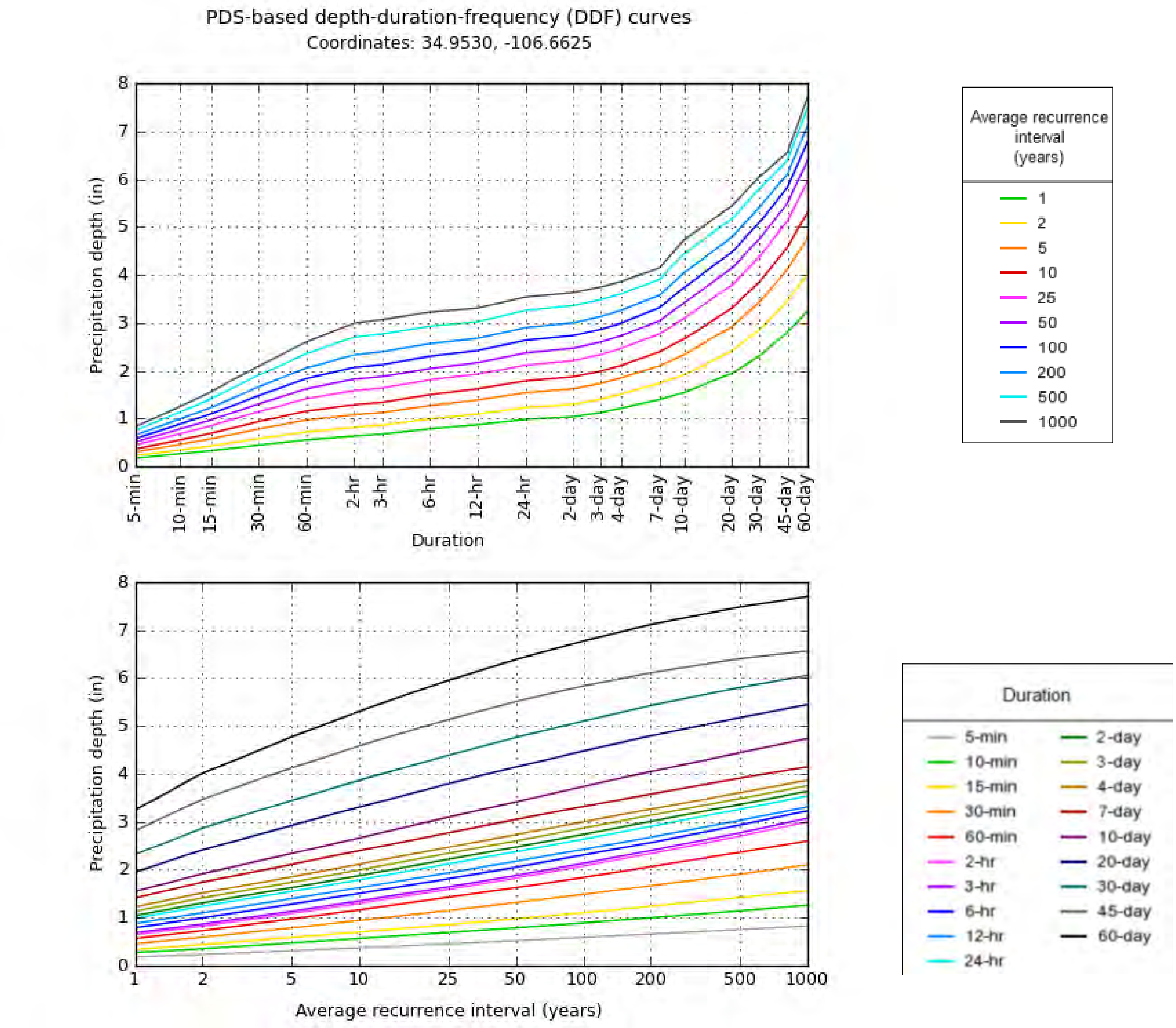
PF graphical



NOAA/NWS/OHD/HDSC Created (GMT): Mon Aug 20 22:02:28 2012

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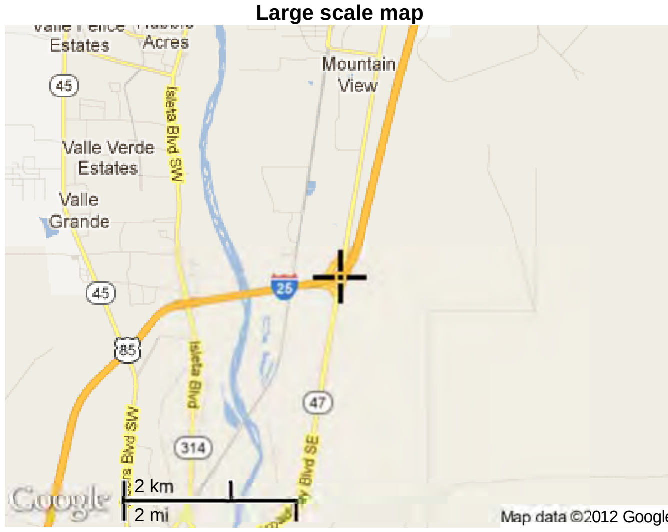
Maps & aerials

Small scale terrain

Large scale terrain

Large scale map



Large scale aerial

NOAA Atlas 14, Volume 1, Version 5
Location name: Albuquerque, New Mexico, US*
Coordinates: 35.1523, -106.7175
Elevation: 5167 ft*
* source: Google Maps

POINT PRECIPITATION FREQUENCY ESTIMATES

Sanja Perica, Sarah Dietz, Sarah Heim, Lillian Hiner, Kazungu Maitaria, Deborah Martin, Sandra Pavlovic, Ishani Roy, Carl Trypaluk, Dale Unruh, Fenglin Yan, Michael Yekta, Tan Zhao, Geoffrey Bonnin, Daniel Brewer, Li-Chuan Chen, Tye Parzybok, John Yarchoan

NOAA, National Weather Service, Silver Spring, Maryland

PF_tabular | PF_graphical | Maps_&_aerials

PF tabular

PDS-based point precipitation frequency estimates with 90% confidence intervals (in inches)[1]

Duration	Average recurrence interval (years)									
	1	2	5	10	25	50	100	200	500	1000
5-min	0.159 (0.136-0.186)	0.206 (0.176-0.241)	0.278 (0.235-0.325)	0.335 (0.284-0.391)	0.412 (0.347-0.480)	0.472 (0.397-0.549)	0.536 (0.448-0.622)	0.605 (0.501-0.702)	0.697 (0.573-0.811)	0.772 (0.629-0.897)
10-min	0.242 (0.207-0.283)	0.314 (0.268-0.367)	0.423 (0.358-0.495)	0.510 (0.433-0.594)	0.627 (0.529-0.730)	0.718 (0.605-0.835)	0.817 (0.682-0.947)	0.920 (0.763-1.07)	1.06 (0.872-1.23)	1.18 (0.957-1.36)
15-min	0.299 (0.257-0.350)	0.389 (0.332-0.455)	0.524 (0.444-0.614)	0.632 (0.537-0.737)	0.777 (0.655-0.905)	0.890 (0.749-1.03)	1.01 (0.846-1.17)	1.14 (0.946-1.32)	1.31 (1.08-1.53)	1.46 (1.19-1.69)
30-min	0.403 (0.346-0.472)	0.524 (0.447-0.612)	0.705 (0.598-0.827)	0.851 (0.723-0.992)	1.05 (0.882-1.22)	1.20 (1.01-1.39)	1.36 (1.14-1.58)	1.54 (1.27-1.78)	1.77 (1.46-2.06)	1.96 (1.60-2.28)
60-min	0.499 (0.428-0.584)	0.648 (0.553-0.758)	0.873 (0.740-1.02)	1.05 (0.895-1.23)	1.29 (1.09-1.51)	1.48 (1.25-1.73)	1.69 (1.41-1.96)	1.90 (1.58-2.21)	2.19 (1.80-2.55)	2.43 (1.98-2.82)
2-hr	0.585 (0.505-0.676)	0.748 (0.643-0.865)	0.993 (0.851-1.15)	1.19 (1.02-1.36)	1.47 (1.25-1.67)	1.69 (1.43-1.93)	1.93 (1.62-2.19)	2.18 (1.81-2.48)	2.53 (2.08-2.87)	2.82 (2.29-3.20)
3-hr	0.621 (0.546-0.717)	0.791 (0.694-0.914)	1.04 (0.914-1.20)	1.24 (1.09-1.43)	1.52 (1.31-1.74)	1.74 (1.50-1.99)	1.98 (1.70-2.26)	2.24 (1.90-2.55)	2.60 (2.17-2.96)	2.89 (2.39-3.30)
6-hr	0.718 (0.633-0.819)	0.910 (0.808-1.04)	1.18 (1.04-1.34)	1.39 (1.23-1.58)	1.68 (1.48-1.91)	1.91 (1.67-2.16)	2.15 (1.86-2.43)	2.40 (2.07-2.71)	2.75 (2.34-3.11)	3.04 (2.57-3.45)
12-hr	0.795 (0.705-0.895)	1.00 (0.891-1.13)	1.28 (1.13-1.44)	1.50 (1.32-1.69)	1.79 (1.57-2.00)	2.02 (1.76-2.26)	2.25 (1.95-2.52)	2.50 (2.15-2.80)	2.83 (2.42-3.17)	3.10 (2.63-3.49)
24-hr	0.921 (0.813-1.05)	1.16 (1.02-1.31)	1.46 (1.29-1.65)	1.69 (1.49-1.91)	2.01 (1.76-2.27)	2.25 (1.97-2.54)	2.51 (2.19-2.83)	2.77 (2.40-3.11)	3.13 (2.69-3.51)	3.40 (2.92-3.82)
2-day	0.949 (0.848-1.06)	1.19 (1.06-1.33)	1.49 (1.34-1.67)	1.73 (1.54-1.93)	2.05 (1.83-2.29)	2.30 (2.04-2.56)	2.55 (2.25-2.84)	2.81 (2.47-3.13)	3.16 (2.76-3.54)	3.44 (2.99-3.85)
3-day	1.11 (1.00-1.22)	1.38 (1.25-1.52)	1.71 (1.54-1.88)	1.97 (1.78-2.17)	2.31 (2.08-2.55)	2.58 (2.32-2.84)	2.85 (2.55-3.14)	3.12 (2.79-3.44)	3.49 (3.10-3.85)	3.77 (3.33-4.16)
4-day	1.26 (1.15-1.38)	1.57 (1.43-1.71)	1.92 (1.75-2.10)	2.20 (2.01-2.40)	2.58 (2.34-2.81)	2.86 (2.60-3.12)	3.15 (2.85-3.43)	3.43 (3.10-3.74)	3.81 (3.43-4.16)	4.10 (3.67-4.47)
7-day	1.44 (1.32-1.57)	1.79 (1.64-1.95)	2.18 (2.00-2.37)	2.48 (2.27-2.69)	2.88 (2.63-3.12)	3.17 (2.90-3.43)	3.46 (3.16-3.75)	3.74 (3.41-4.06)	4.11 (3.73-4.46)	4.37 (3.96-4.75)
10-day	1.60 (1.47-1.74)	1.98 (1.82-2.16)	2.43 (2.23-2.63)	2.77 (2.55-3.00)	3.23 (2.96-3.49)	3.57 (3.27-3.86)	3.92 (3.58-4.23)	4.25 (3.87-4.60)	4.69 (4.26-5.08)	5.01 (4.53-5.43)
20-day	2.00 (1.83-2.18)	2.48 (2.28-2.71)	3.01 (2.77-3.28)	3.41 (3.13-3.70)	3.92 (3.59-4.25)	4.29 (3.92-4.65)	4.64 (4.24-5.03)	4.97 (4.53-5.38)	5.39 (4.90-5.84)	5.68 (5.16-6.16)
30-day	2.39 (2.19-2.59)	2.96 (2.72-3.21)	3.56 (3.27-3.85)	4.00 (3.67-4.32)	4.55 (4.18-4.91)	4.95 (4.53-5.33)	5.32 (4.87-5.72)	5.66 (5.18-6.10)	6.07 (5.55-6.54)	6.36 (5.80-6.85)
45-day	2.92 (2.69-3.16)	3.61 (3.33-3.91)	4.30 (3.97-4.65)	4.79 (4.42-5.17)	5.39 (4.97-5.81)	5.79 (5.34-6.24)	6.15 (5.68-6.63)	6.47 (5.97-6.97)	6.82 (6.30-7.35)	7.03 (6.51-7.57)
60-day	3.35 (3.10-3.63)	4.15 (3.83-4.49)	4.94 (4.57-5.34)	5.51 (5.09-5.95)	6.19 (5.72-6.68)	6.65 (6.14-7.17)	7.07 (6.54-7.63)	7.44 (6.88-8.03)	7.86 (7.27-8.48)	8.11 (7.53-8.76)

[1] Precipitation frequency (PF) estimates in this table are based on frequency analysis of partial duration series (PDS).

Numbers in parenthesis are PF estimates at lower and upper bounds of the 90% confidence interval. The probability that precipitation frequency estimates (for a given duration and average recurrence interval) will be greater than the upper bound (or less than the lower bound) is 5%. Estimates at upper bounds are not checked against probable maximum precipitation (PMP) estimates and may be higher than currently valid PMP values.

Please refer to NOAA Atlas 14 document for more information.

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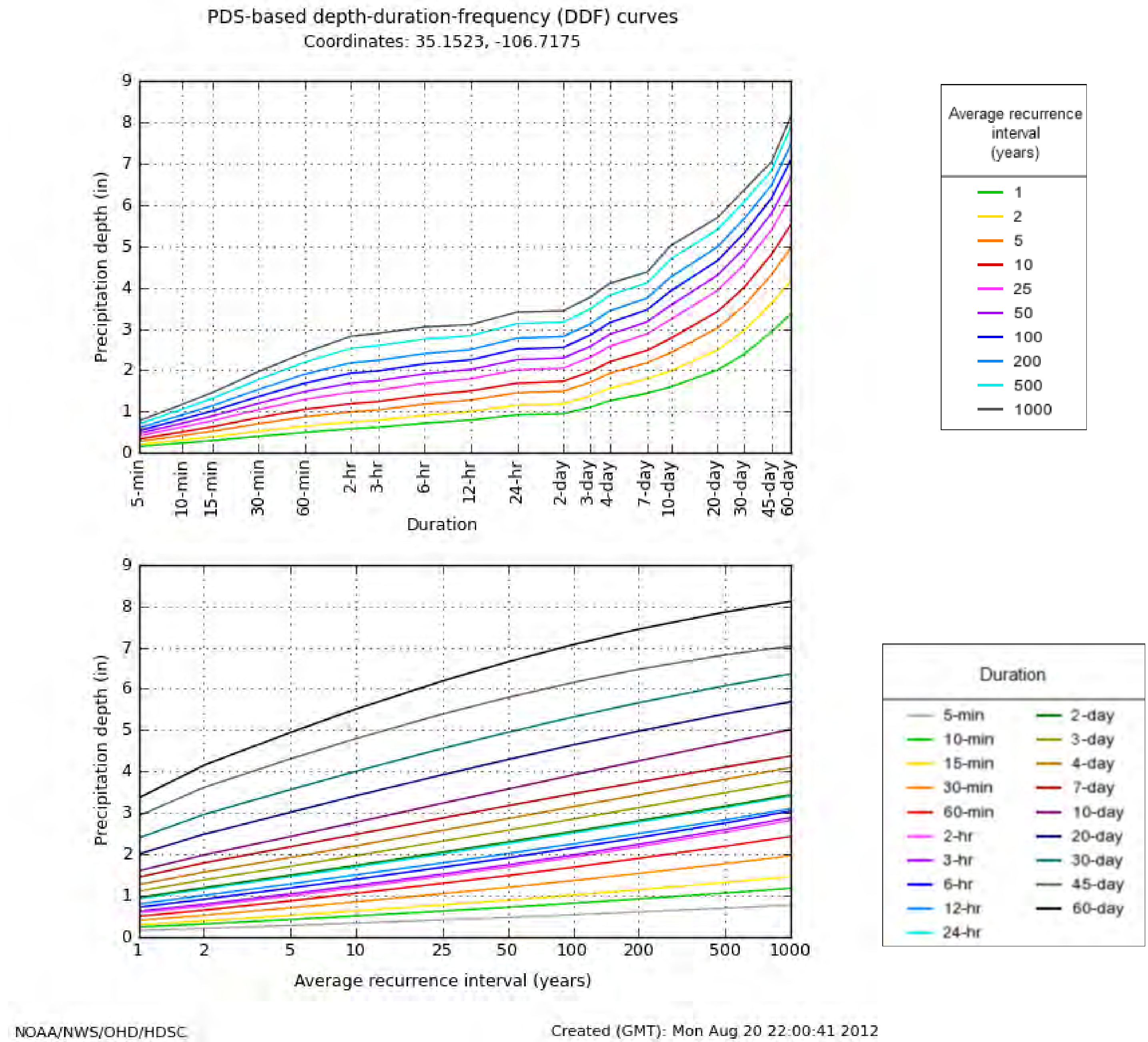
PF graphical



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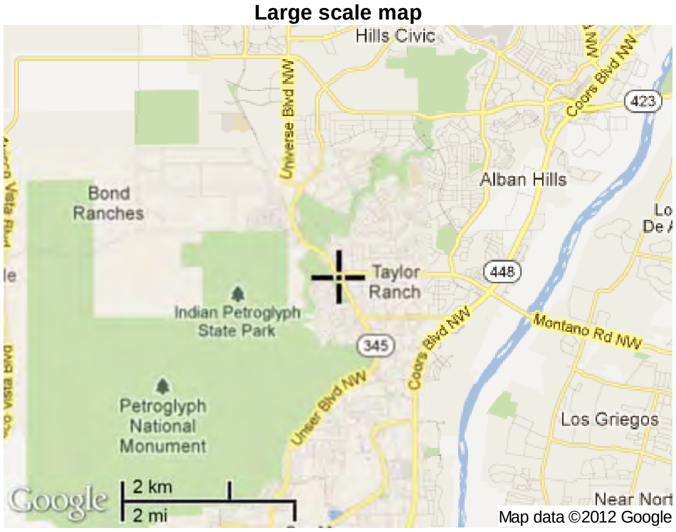
Maps & aerials

Small scale terrain

Large scale terrain

Large scale map



Large scale aerial

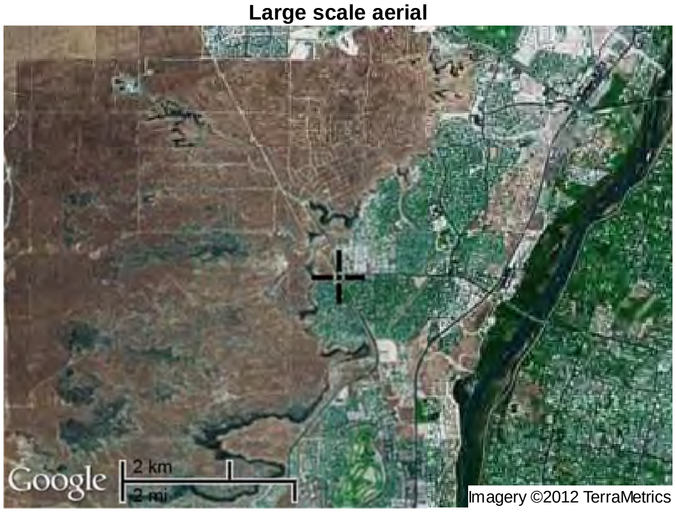
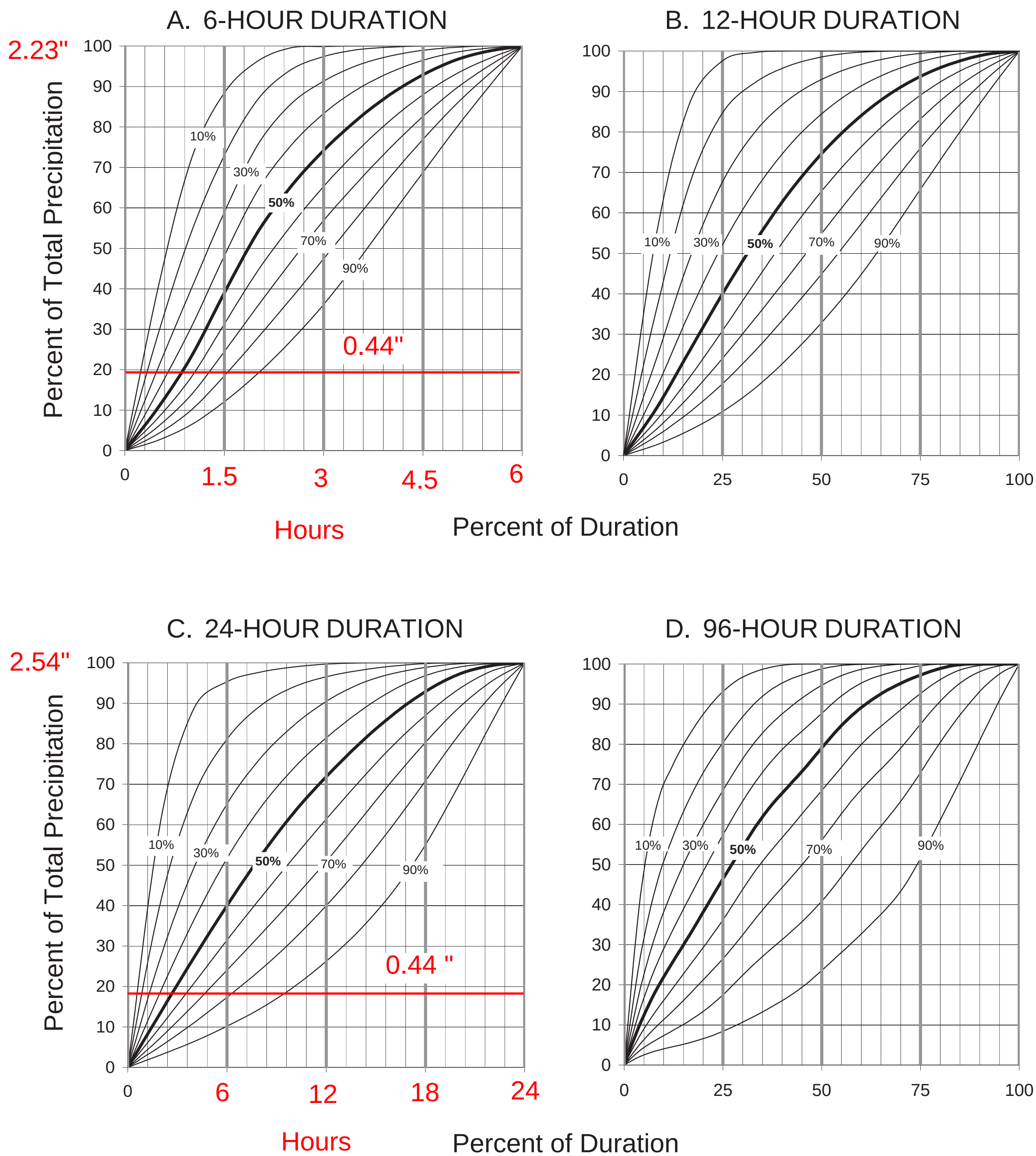
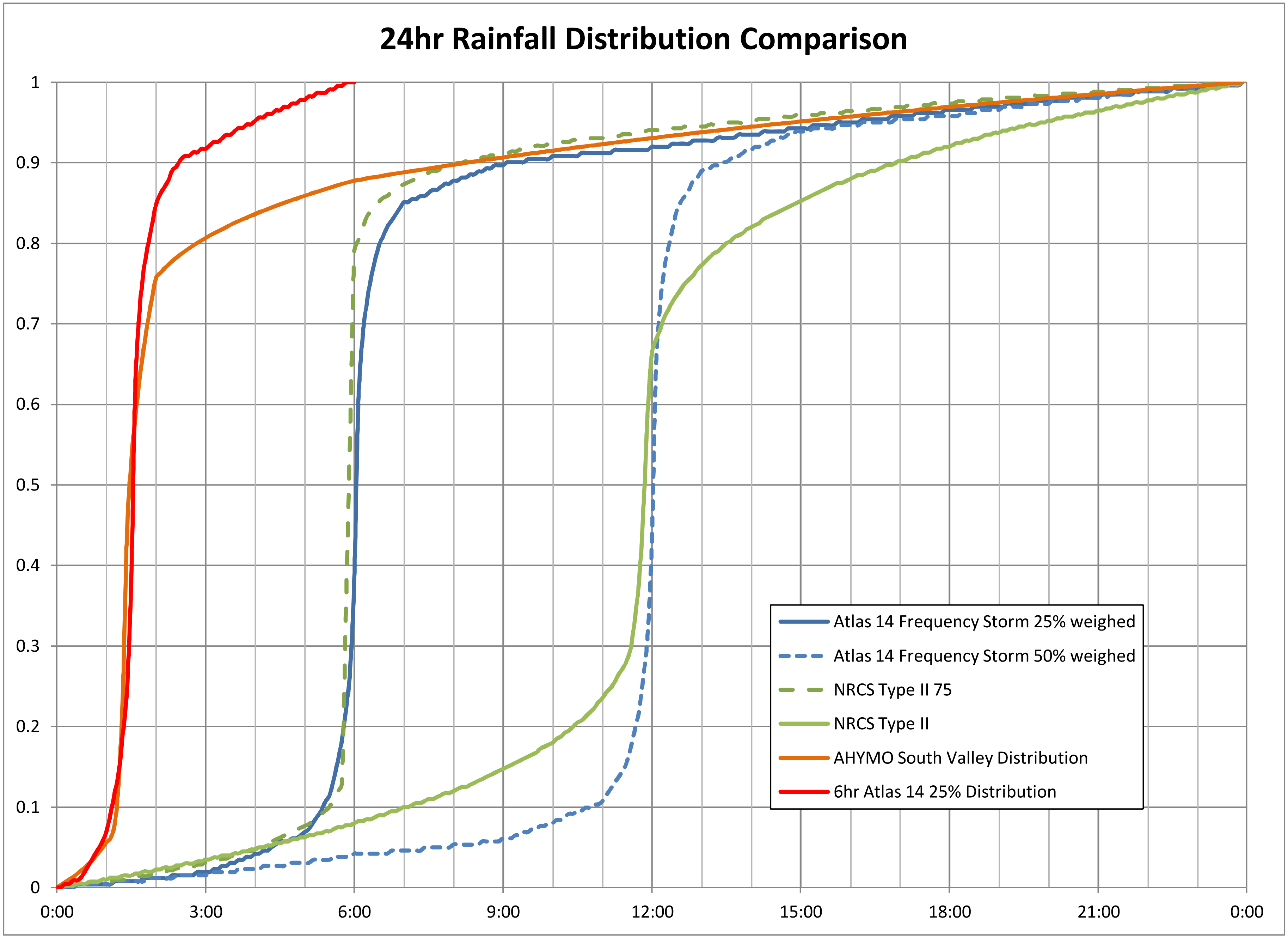
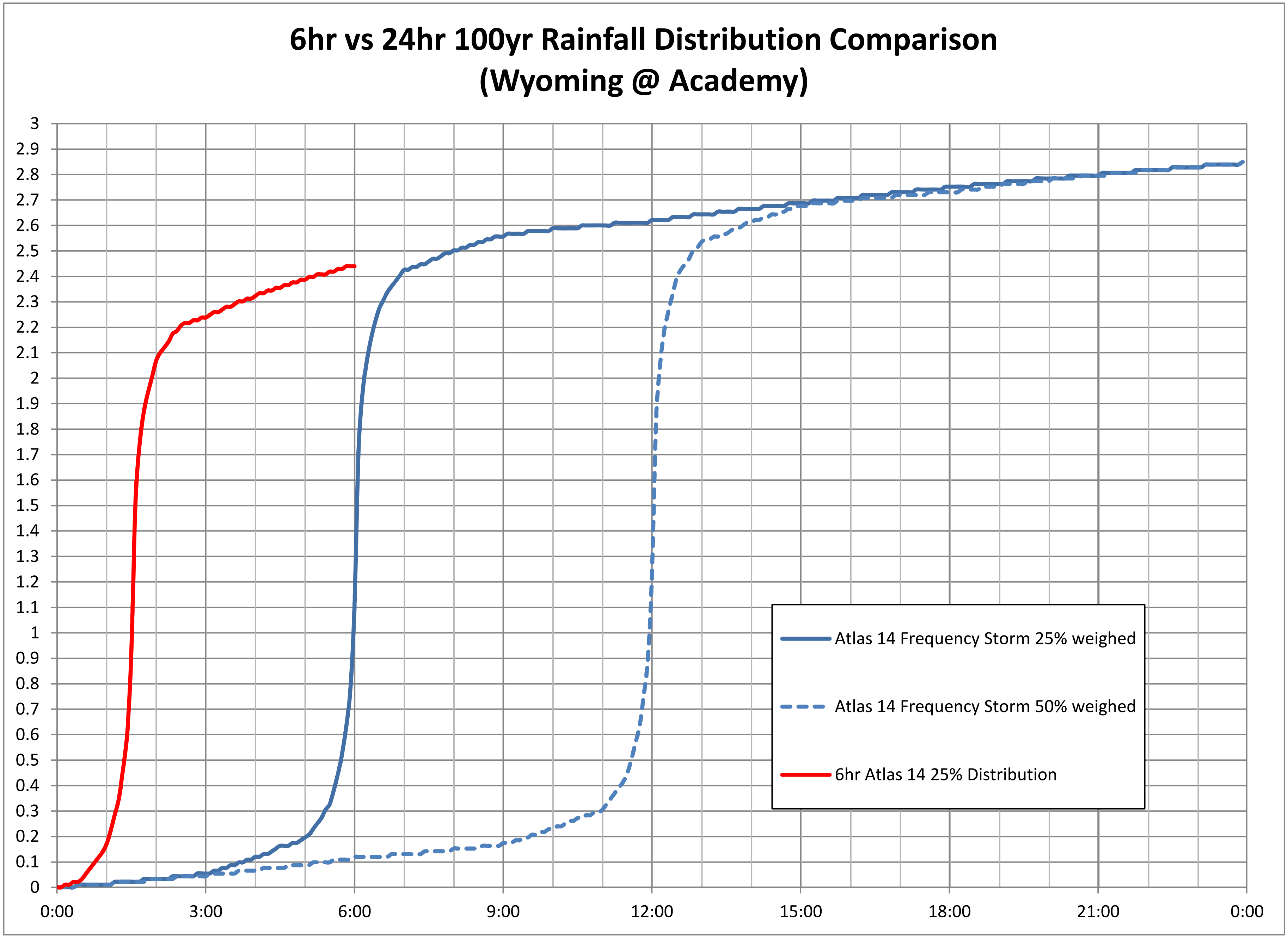


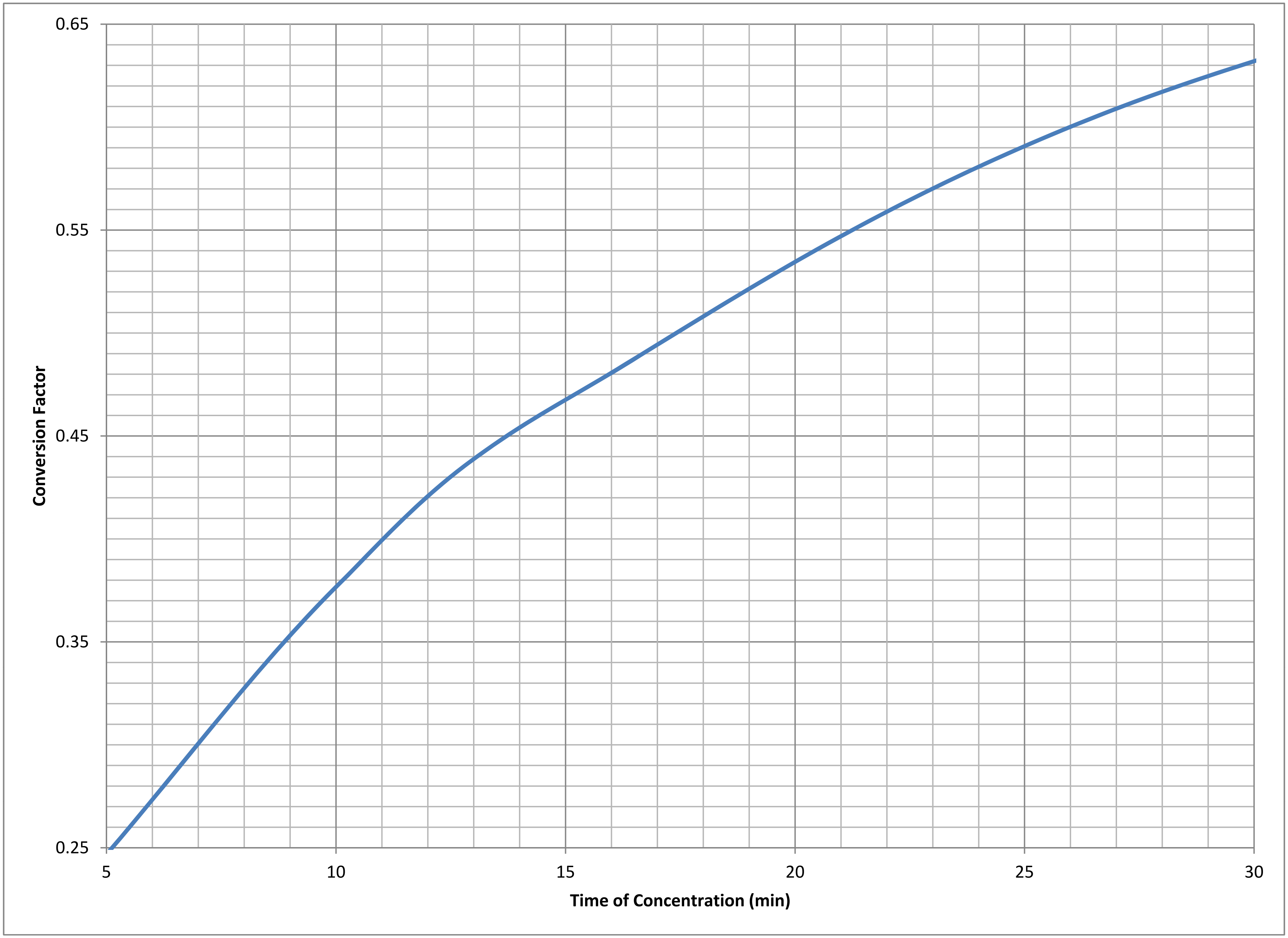
FIGURE A.1.3
TEMPORAL DISTRIBUTION: ALL CASES
CONVECTIVE PRECIPITATION AREA





6 hr vs 24 hr distributions


6hr vs 24hr 100yr Rainfall Distribution Comparison
(Wyoming @ Academy)
3
2.9
2.8
2.7
2.6
2.5
2.4
2.3
2.2
2.1
2
1.9
1.8
1.7
1.6
1.5
1.4
1.3
1.2
1.1
1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
0:00
3:00
6:00
9:00
12:00
15:00
18:00
21:00
0:00
Atlas 14 Frequency Storm 25% weighed
Atlas 14 Frequency Storm 50% weighed
6hr Atlas 14 25% Distribution



100 year Rainfall Intensity Curve for Rational Method Use

APPENDIX D

RAINFALL/RUNOFF WITH TR-55

Table 2-2a Runoff curve numbers for urban areas [1/]

Cover description: Cover type and hydrologic condition	Average percent impervious area [2/]	Curve numbers for hydrologic soil group: A	B	C	D
Fully developed urban areas (vegetation established)					
Open space (lawns, parks, golf courses, cemeteries, etc.) [3/]:					
Poor condition (grass cover < 50%)		68	79	86	89
Fair condition (grass cover 50% to 75%)		49	69	79	84
Good condition (grass cover > 75%)		39	61	74	80
Impervious areas:					
Paved parking lots, roofs, driveways, etc. (excluding right-of-way)		98	98	98	98
Streets and roads:					
Paved; curbs and storm sewers (excluding right-of-way)		98	98	98	98
Paved; open ditches (including right-of-way)		83	89	92	93
Gravel (including right-of-way)		76	85	89	91
Dirt (including right-of-way)		72	82	87	89
Western desert urban areas:					
Natural desert landscaping (pervious areas only) [4/]		63	77	85	88
Artificial desert landscaping (impervious weed barrier, desert shrub with 1- to 2-inch sand or gravel mulch and basin borders)		96	96	96	96
Urban districts:					
Commercial and business	85	89	92	94	95
Industrial	72	81	88	91	93
Residential districts by average lot size:					
1/8 acre or less (town houses)	65	77	85	90	92
1/4 acre	38	61	75	83	87
1/3 acre	30	57	72	81	86
1/2 acre	25	54	70	80	85
1 acre	20	51	68	79	84
2 acres	12	46	65	77	82
Developing urban areas					
Newly graded areas (pervious areas only, no vegetation) [5/]		77	86	91	94
Idle lands (CN's are determined using cover types similar to those in table 2-2c).					

1 Average runoff condition, and $I_a = 0.2S$.
2 The average percent impervious area shown was used to develop the composite CN's. Other assumptions are as follows: impervious areas are directly connected to the drainage system, impervious areas have a CN of 98, and pervious areas are considered equivalent to open space in good hydrologic condition. CN's for other combinations of conditions may be computed using figure 2-3 or 2-4.
3 CN's shown are equivalent to those of pasture. Composite CN's may be computed for other combinations of open space cover type.
4 Composite CN's for natural desert landscaping should be computed using figures 2-3 or 2-4 based on the impervious area percentage (CN = 98) and the pervious area CN. The pervious area CN's are assumed equivalent to desert shrub in poor hydrologic condition.
5 Composite CN's to use for the design of temporary measures during grading and construction should be computed using figure 2-3 or 2-4 based on the degree of development (impervious area percentage) and the CN's for the newly graded pervious areas.

Antecedent runoff condition

The index of runoff potential before a storm event is the antecedent runoff condition (ARC). ARC is an attempt to account for the variation in CN at a site from storm to storm. CN for the average ARC at a site is the median value as taken from sample rainfall and runoff data. The CN's in table 2-2 are for the average ARC, which is used primarily for design applications. See NEH-4 (SCS 1985) and Rallison and Miller (1981) for more detailed discussion of storm-to-storm variation and a demonstration of upper and lower enveloping curves.

Urban impervious area modifications

Several factors, such as the percentage of impervious area and the means of conveying runoff from impervious areas to the drainage system, should be considered in computing CN for urban areas (Rawls et al., 1981). For example, do the impervious areas connect directly to the drainage system, or do they outlet onto lawns or other pervious areas where infiltration can occur?

Connected impervious areas — An impervious area is considered connected if runoff from it flows directly into the drainage system. It is also considered connected if runoff from it occurs as concentrated shallow flow that runs over a pervious area and then into the drainage system.

Urban CN's (table 2-2a) were developed for typical land use relationships based on specific assumed percentages of impervious area. These CN vales were developed on the assumptions that (a) pervious urban areas are equivalent to pasture in good hydrologic condition and (b) impervious areas have a CN of 98 and are directly connected to the drainage system. Some assumed percentages of impervious area are shown in table 2-2a

If all of the impervious area is directly connected to the drainage system, but the impervious area percentages or the pervious land use assumptions in table 2-2a are not applicable, use figure 2-3 to compute a composite CN. For example, table 2-2a gives a CN of 70 for a 1/2-acre lot in HSG B, with assumed impervious area of 25 percent. However, if the lot has 20 percent impervious area and a pervious area CN of 61, the composite CN obtained from figure 2-3 is 68. The CN difference between 70 and 68 reflects the difference in percent impervious area.

Unconnected impervious areas — Runoff from these areas is spread over a pervious area as sheet flow. To determine CN when all or part of the impervious area is not directly connected to the drainage system, (1) use figure 2-4 if total impervious area is less than 30 percent or (2) use figure 2-3 if the total impervious area is equal to or greater than 30 percent, because the absorptive capacity of the remaining pervious areas will not significantly affect runoff.

When impervious area is less than 30 percent, obtain the composite CN by entering the right half of figure 2-4 with the percentage of total impervious area and the ratio of total unconnected impervious area to total impervious area. Then move left to the appropriate pervious CN and read down to find the composite CN. For example, for a 1/2-acre lot with 20 percent total impervious area (75 percent of which is unconnected) and pervious CN of 61, the composite CN from figure 2-4 is 66. If all of the impervious area is connected, the resulting CN (from figure 2-3) would be 68.

Figure 2-3 Composite CN with connected impervious area.

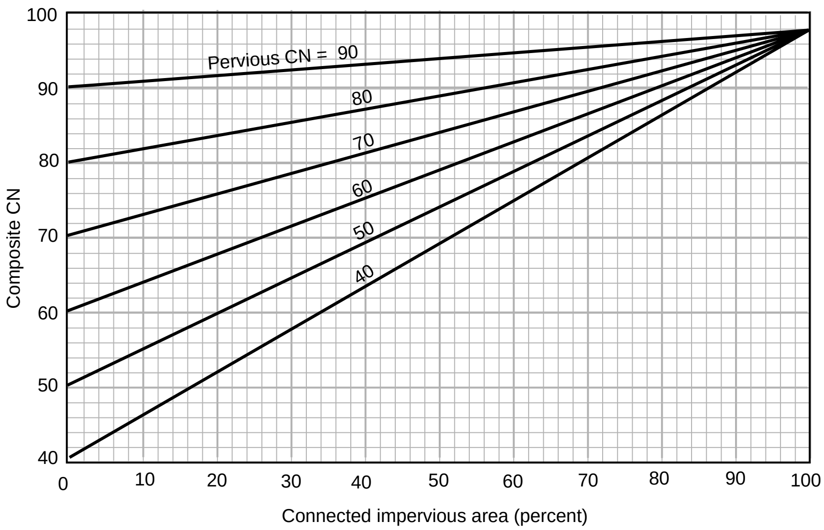


Figure 2-4 Composite CN with unconnected impervious areas and total impervious area less than 30%

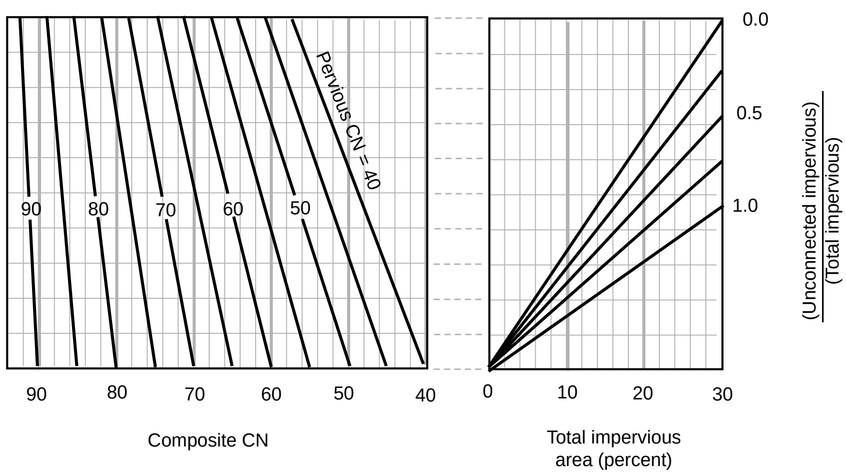


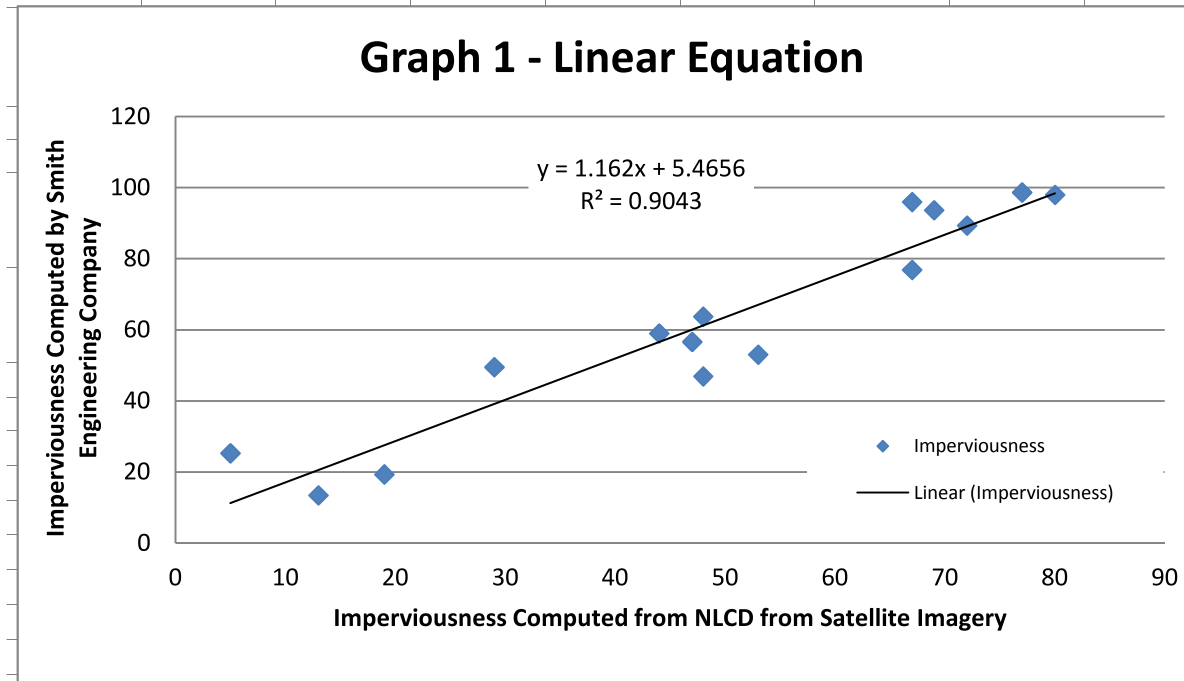
TABLE I1

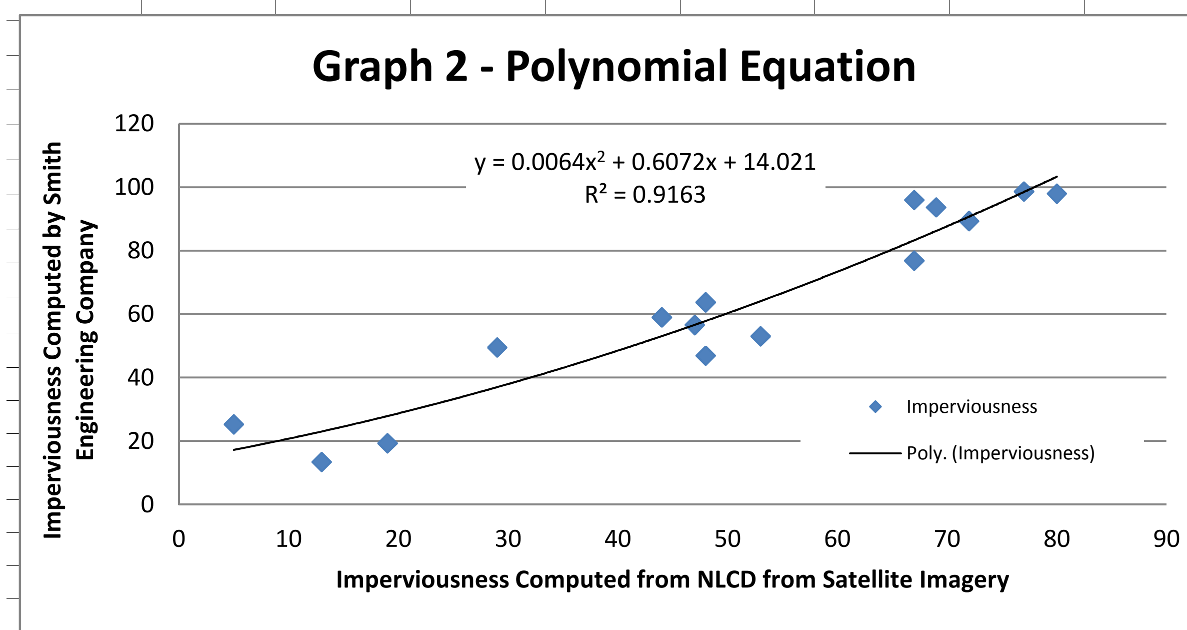
SUMMARY OF 15 SUBCATCHMENTS - Imperviousness Measurements (Actual) vs. Satellite Imagery

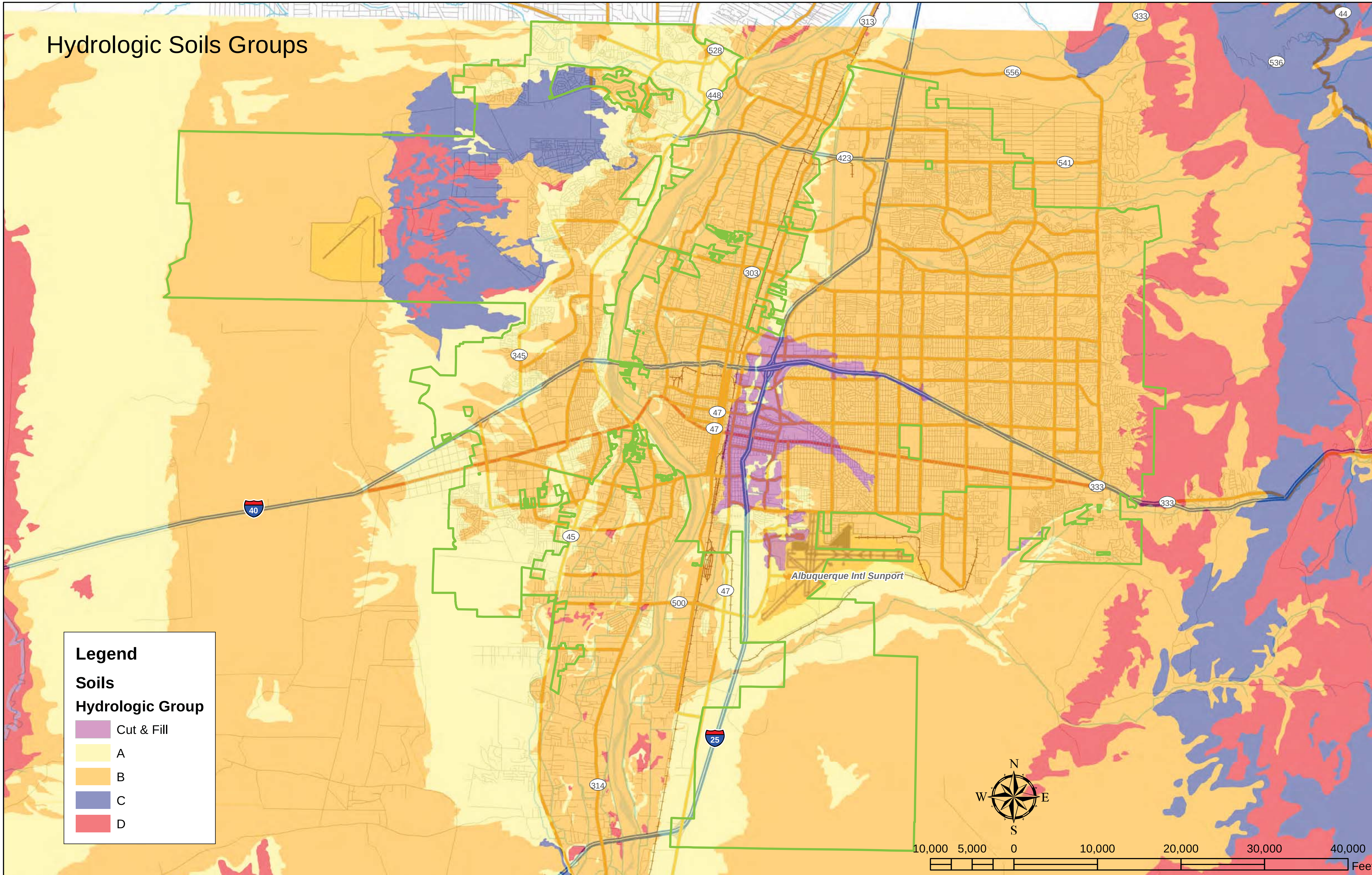
Mid Valley Drainage Management Plan

Subcatchment (Basin) No.	Imperviousness Measured by Smith Engineering Company % (a)	Imperviousness Computed from NLCD from Satellite Imagery % (b)	Imperviousness Computed by Smith Engineering Company Using a Linear Equation % (c)	Imperviousness Computed by Smith Engineering Company Using a Polynomial Equation % (d)
BR8-H2	64	48	61	58
BR14	89	72	89	91
BR16	13	13	21	23
BR5	59	44	57	53
B18	98	80	98	104
B23	99	77	95	99
B11	19	19	28	28
B41	94	69	86	86
B25	96	67	83	83
B36	77	67	83	83
B4	47	48	61	58
A4	50	29	39	37
A8	57	47	60	57
A12	53	53	67	64
A16-C	25	5	11	17

a & b - Data obtained from Table I2
c- Data obtained from linear equation derived in Graph 1
d- Data obtained from polynomial equation derived in Graph 2






Hydrologic Soils Groups
Legend
Soils
Hydrologic Group
Cut & Fill
A
B
C
D
Albuquerque Intl Sunport
40
25
333
313
528
448
556
536
44
423
541
303
345
47
47
47
45
500
314
N
S
E
W
10,000
5,000
0
10,000
20,000
30,000
40,000
Feet

RUNOFF CURVE NUMBER: HAS IT REACHED MATURITY?

By Victor M. Ponce,[1] and Richard H. Hawkins,[2] Members, ASCE

ABSTRACT: The conceptual and empirical foundations of the runoff curve number method are reviewed. The method is a conceptual model of hydrologic abstraction of storm rainfall. Its objective is to estimate direct runoff depth from storm rainfall depth, based on a parameter referred to as the "curve number." The method does not take into account the spatial and temporal variability of infiltration and other abstractive losses; rather, it aggregates them into a calculation of the total depth loss for a given storm event and drainage area. The method describes average trends, which precludes it from being perfectly predictive. The observed variability in curve numbers, beyond that which can be attributed to soil type, land use/treatment, and surface condition, is embodied in the concept of antecedent condition. The method is widely used in the United States and other countries. Perceived advantages of the method are (1) its simplicity; (2) its predictability; (3) its stability; (4) its reliance on only one parameter; and (5) its responsiveness to major runoff-producing watershed properties (soil type, land use/treatment, surface condition, and antecedent condition). Perceived disadvantages are (1) its marked sensitivity to curve number; (2) the absence of clear guidance on how to vary antecedent condition; (3) the method's varying accuracy for different biomes; (4) the absence of an explicit provision for spatial scale effects; and (5) the fixing of the initial abstraction ratio at 0.2, preempting a regionalization based on geologic and climatic setting.

INTRODUCTION

The runoff curve number method for the estimation of direct runoff from storm rainfall is well established in hydrologic engineering and environmental impact analyses. Its popularity is rooted in its convenience, its simplicity, its authoritative origins, and its responsiveness to four readily grasped catchment properties: soil type, land use/treatment, surface condition, and antecedent condition.

The method was developed in 1954 by the USDA Soil Conservation Service (Rallison 1980), and is described in the Soil Conservation Service (SCS) National Engineering Handbook Section 4: Hydrology (NEH-4) (*SCS* 1985). The first version of the handbook containing the method was published in 1954. Subsequent revisions followed in 1956, 1964, 1965, 1971, 1972, 1985, and 1993. Since its inception, the method had the full support of a federal agency and, moreover, it filled a strategic technological niche. Thus, it quickly became established in hydrologic practice, with numerous applications in the United States and other countries. Experience with the runoff curve number continues to increase to this date (Bosznay 1989; Hjelmfelt 1991; Hawkins 1993; Steenhuis et al. 1995).

The method's credibility and acceptance has suffered, however, due to its origin as agency methodology, which effectively isolated it from the rigors of peer review. Other than the information contained in NEH-4, which was not intended to be exhaustive (Rallison and Cronshey 1979), no complete account of the method's foundations is available to date, despite some recent noteworthy attempts (Rallison 1980; Chen 1982; Miller and Cronshey 1989).

In the four decades that have elapsed since the method's inception, the increased availability of computers has led to the use of complex hydrologic models, many of which incorporate the curve number method. Thus, the question naturally arises: What is the status of the curve number method in a postulated hierarchy of hydrologic abstraction models? (Miller and Cronshey 1989; Rallison and Miller 1982). Has it matured into general acceptance and usage? Or, as some of its critics suggest, is it now obsolete, a remnant of outdated technology, and in need of overhaul or outright replacement? (Smith and Eggert 1978; Van Mullem 1989).

An effective overhaul of the method would require a clearer understanding of its properties than is currently available (Woodward 1991; Woodward and Gburek 1992). An outright replacement, if one were to be developed, is likely to forego part or all of the extensive data on hydrologic soil groups and land use/treatment classes that has been assembled for most of the United States (Miller and Cronshey 1989). More than 4,000 soils in the United States have been given a hydrologic soil group (Rallison 1980). Moreover, a replacement or overhaul could not avoid relying on many of those same features that are now part of the curve number method. Therefore, it has become necessary to examine the curve number method, to shed additional light on its foundations, and to delineate its strengths and weaknesses, so that the method may continue to be used by practitioners without fear of an impending demise. Thus, the objectives of this paper are the following:

1. To critically examine the curve number method
2. To clarify its conceptual and empirical basis
3. To delineate its capabilities, limitations, and uses
4. To identify areas of research in runoff curve number methodology

Over the years, the conceptual basis of the curve number method has been the object of both support and criticism. A conceptual model shares the simplicity of empirical models and the wider applicability of the more rigorous physically based models (Dooge 1977). Being conceptual, the runoff curve number method is simple, and this is at the root of its popularity. On the other hand, it is precisely for this reason that the runoff curve number method has not fared well among the supporters of alternative models, which include the physically based models (Smith 1976). If experience is any indication, the choice between physically based and conceptual models of hydrologic abstraction is a difficult one, particularly with regard to infiltration (Branson et al. 1981; Savabi et al. 1990; Hjelmfelt 1991).

Branson et al. (1962, 1981), among others, have argued that the simpler conceptual models are not necessarily inferior to the more complex physically based models. The latter may do a good job of describing the physical processes, but this

[1]Prof., Civ. and Envir. Engrg. Dept., San Diego State Univ., San Diego, CA 92182-1324.

[2]Prof., Watershed Resour. Program, School of Renewable Natural Resour., Univ. of Arizona, Tucson, AZ 85721.

Note. Discussion open until June 1, 1996. To extend the closing date one month, a written request must be filed with the ASCE Manager of Journals. The manuscript for this paper was submitted for review and possible publication on July 10, 1995. This paper is part of the ***Journal of Hydrologic Engineering***, Vol. 1, No. 1, January, 1996. ©ASCE, ISSN 1084-0699/96/0001-0011–0019/$4.00 + $.50 per page. Paper No. 11094.

is usually at the expense of the chemical and biological aspects. In many instances, processes such as surface crusting, clay shrinkage and swelling, entrapped gases, root structure and decay, and soil macro- and microfauna may be of such importance as to largely invalidate a strictly physical approach to infiltration modeling (Le Bissonnais and Singer 1993).

LUMPED VERSUS DISTRIBUTED MODELS

The curve number method is an infiltration loss model, although it may also account for interception and surface storage losses through its initial abstraction feature. As originally developed, the method is not intended to account for evaporation and evapotranspiration (long-term losses).

An infiltration loss model can be either lumped or distributed. The lumped model aggregates spatial and temporal variations into a calculation of the total infiltration depth for a given storm depth and drainage area. The distributed model describes instantaneous and/or local infiltration rates, from which a total infiltration depth is eventually obtained by suitable integration in time and space. The curve number method was originally developed as a lumped model (spatial and temporal), used to convert storm rainfall depth into direct runoff volume. To this date, it is used primarily as a temporally lumped model in the manner specified by the NEH-4 handbook (*SCS* 1985). However, a few investigators, notably Smith (1976), Aron et al. (1977), Chen (1975, 1976, 1982), and Hawkins (1978a, 1980) have developed infiltration-capacity-equivalent formulas based directly or indirectly on the curve number method. This effectively extends the method to the domain of distributed modeling, although the instances of this type of use appear to be relatively few. Existing infiltration formulas such as Green and Ampt (1911), Horton (1933), and Philip (1957) describe instantaneous and/or local infiltration rates, and thus are directly suited for distributed modeling.

The relative advantages of distributed modeling versus lumped modeling are not easily determined. With regard to infiltration capacities, the spatial and temporal variability that prevails in almost all practical settings does not usually favor the distributed approach, unless the nature of this variability can be specifically incorporated into the model, which is not a small task (Miller and Cronshey 1989). Disregarding this variability, or not accounting for it in a realistic way, amounts in a real sense to lumping. Therefore, the lumped models owe their existence to our inability to properly account for the intrinsic variability of natural phenomena. What this means in practice is that a lumped model is not necessarily bad. Rather, that it is a practical way to substitute for the more complex distributed process while attempting to preserve the main features of the prototype.

A measurement of infiltration rate, or infiltration capacity, as accurate as it may be, can only describe the rate at the point of measure (Miller and Cronshey 1989). Extrapolation to a larger area is tantamount to lumping. In fact, a lumped infiltration depth is a statement of a spatially and temporally averaged infiltration rate (however small the sample plot), with all the advantages and disadvantages that this implies. The advantage is that the method preserves the average features of the phenomena. The disadvantage is that the method does not specifically describe the spatial and/or temporal variability. Nevertheless, a few interpretations of the curve number method in terms of the spatial distribution of loss depths have been developed (Hawkins 1982; Hawkins and Cundy 1982).

In practice, an acceptable amount of lumping is a function of problem scale. For small-scale problems, for example, plots measured in square feet or acres (square meters or hectares), an attempt to ascertain the spatial and temporal variability of infiltration capacity may be justified by detailed field measurements. However, as the scale increases to hundreds of hectares and tens of square kilometers, the practical inability to collect increasing amounts of infiltration data makes lumping an absolute necessity in infiltration modeling. Sooner or later, a certain amount of spatial averaging has to be introduced. Furthermore, considering that spatial averaging is implicit in the nature of rainfall data at any scale, a strong case is made for lumping as a de facto modeling tactic.

CONVERSION OF RAINFALL TO RUNOFF

The conversion of rainfall to runoff is the centerpiece of surface water modeling. An elementary expression of conservation of mass is

$$Q = P - L \tag{1}$$

where Q = runoff; P = rainfall; and L = abstractive losses, or hydrologic abstractions.

The quantification of hydrologic abstractions can be a complex task. These fall into five categories:

1. Interception storage in a rural setting, by vegetation foliage, stems, and litter and in an urban setting, by cultural features of the landscape
2. Surface storage in ponds, puddles, and other usually small temporary storage locations
3. Infiltration to the subsurface to feed and replenish soil moisture, interflow, and ground-water flow
4. Evaporation from water bodies such as lakes, reservoirs, streams, and rivers as well as from moisture on bare ground
5. Evapotranspiration from all types of vegetation

Of these five types of hydrologic abstractions, infiltration is the most important for storm analysis (short term). Evaporation and evapotranspiration are the most important for seasonal or annual yield evaluations (long term). The remaining two losses (interception and surface storage) are usually of secondary importance.

The curve number method is an infiltration loss model; therefore, its applicability is restricted to modeling storm losses. Barring appropriate modifications, the method should not be used to model the long-term hydrologic response of a catchment. Nevertheless, it is recognized that the method has been used in several long-term hydrologic simulation models developed in the past two decades (Williams and LaSeur 1976; Huber et al. 1976; Knisel 1980; Soni and Mishra 1985), with varying degrees of success (Woodward and Gburek 1992). Since the curve number method (as developed by SCS) does not model evaporation and evapotranspiration, its use in long-term hydrologic simulation should be restricted to modeling the storm losses and associated surface runoff (Boughton 1989).

Ponce and Shetty (1995) have recently developed a conceptual model of a catchment's annual water balance. The model accomplishes the sequential separation of (1) annual precipitation into surface runoff and wetting; and (2) wetting into baseflow and vaporization. Ponce and Shetty's model draws on a concept similar to that of the runoff curve number. However, for a given site, the value of the annual retention parameter bears no resemblance to that of the conventional curve number method.

MODES OF SURFACE RUNOFF GENERATION

To clarify the basis of the curve number method, we review here the processes of surface runoff generation. Surface runoff is generated by a variety of surface and near-surface flow processes, of which some of the most important are

1. Hortonian overland flow
2. Saturation overland flow
3. Throughflow processes
4. Partial-area runoff
5. Direct channel interception
6. Surface phenomena, such as crust development, hydrophobic soil layers, and frozen ground

Hortonian overland flow describes the process that takes place when rainfall rate exceeds infiltration capacity, usually at the beginning of a storm (or season), when the soil profile is likely to be on the dry side. The rate difference (rainfall rate minus infiltration capacity) is the effective rainfall rate that is converted to surface runoff.

Saturation overland flow describes the process that takes place after the soil profile has become saturated, either from antecedent rainfall events or from a sufficient volume of rainfall within the same event. At this point, any additional rainfall, regardless of intensity, will be converted into surface runoff. Saturation overland flow usually occurs during an infrequent storm, or toward the end of a particularly wet season, when the soil is likely to be already wet from prior storms.

Throughflow prevails in heavily vegetated areas with thick soil covers containing less permeable layers, overlying relatively impermeable unweathered bedrock (Kirkby and Chorley 1967). Strictly speaking, throughflow is not direct (surface) runoff, since the flow takes place primarily as interflow, or lateral flow immediately below the ground surface. Throughflow's relatively quick response, however, is in the same time frame as surface runoff and, thus, it is generally regarded as a mode of surface runoff generation.

The concept of partial-area runoff developed from the recognition that runoff estimates were improved by assuming that only rainfall on a small and fairly constant part of each drainage basin is able to contribute to direct runoff (Kirkby and Chorley 1967). Thus, partial-area runoff can be interpreted as a combination of throughflow in the upper hillslopes and saturation overland flow in the lower hillslopes (Chorley 1978; Branson et al. 1981; Hawkins 1981).

Direct channel interception refers to the runoff that originates from rainfall falling directly into the channels. This mode of surface runoff generation may be important in dense channel networks and certain humid bases, where direct channel interception may be the primary source of streamflow (Hawkins 1973).

Surface phenomena includes processes such as crust development, hydrophobic soil layers, and frozen ground, which render the soil surface impermeable, promoting surface runoff. For instance, a surface crust may develop following splash erosion in a denuded watershed, adversely affected by human activities or a natural hazard such as fire. Under a specific set of circumstances, including soil type and texture, the silt entrained by splash erosion may deposit on the surface and create a thin crust that eventually reduces the infiltration rate to a negligible level. Thus, any additional rainfall will be converted to surface runoff. This mode of surface runoff generation is typical of semiarid environments, where large amounts of surface runoff may take place even though the underlying soil profile, below a relatively thin veneer, remains substantially dry ("Influences" 1940; Le Bissonnais and Singer 1993).

HISTORICAL BACKGROUND

The origins of the curve number methodology can be traced back to the thousands of infiltrometer tests carried out by SCS in the late 1930s and early 1940s. The intent was to develop basic data to evaluate the effects of watershed treatment and soil conservation measures on the rainfall-runoff process. A major catalyst for the development and implementation of the runoff curve number methodology was the passage of the Watershed Protection and Flood Prevention Act of August 1954. Studies associated with small watershed project planning were expected to require a substantial improvement in hydrologic computation within SCS (Rallison 1980).

Sherman (1942, 1949) had proposed plotting direct runoff versus storm rainfall. Building on this idea, Mockus (1949) proposed that estimates of surface runoff for ungauged watersheds could be based on information on soils, land use, antecedent rainfall, storm duration, and average annual temperature. Furthermore, he combined these factors into an empirical parameter b characterizing the relationship between rainfall depth P and runoff depth Q (Rallison 1980).

$$Q = P(1 - 10^{-bP}) \tag{2}$$

Andrews (unpublished report, 1954), using infiltrometer data from Texas, Oklahoma, Arkansas, and Louisiana, developed a graphical procedure for estimating runoff from rainfall for several combinations of soil texture, type and amount of cover, and conservation practices. The association was referred to as a "soil-cover complex" (Miller and Cronshey 1989).

Mockus' empirical P-Q rainfall-runoff relationship [(2)] and Andrews' soil-cover complex were the basics of the conceptual rainfall-runoff relationship incorporated into the forerunner version of NEH-4 (*Hydrology* 1954). The method, since referred to as the runoff curve number, had the following significant features:

1. The runoff depth Q is bounded in the range $0 \leq Q \leq P$, assuring its stability.
2. As rainfall depth P grows unbounded ($P \to \infty$), the actual retention ($P - Q$) asymptotically approaches a constant value S. This constant value, referred to in NEH-4 as "potential maximum retention," and here simply as "potential retention," characterizes the watershed's potential for abstracting and retaining storm moisture and, therefore, its direct runoff potential.
3. A runoff equation relates Q to P, and a curve parameter CN, in turn, relates to S.
4. Estimates of CN are based on: (1) hydrologic soil group; (2) land use and treatment classes; (3) hydrologic surface condition; and (4) antecedent moisture condition.

RUNOFF CURVE NUMBER EQUATION

The method assumes a proportionality between retention and runoff, such that

$$\frac{F}{S} = \frac{Q}{P} \tag{3}$$

where $F = P - Q$ = actual retention; S = potential retention; Q = actual runoff; and P = potential runoff, that is, total rainfall. The values of P, Q, and S are given in depth dimensions. While the original method was developed in U.S. customary units (in.), an appropriate conversion to SI units (cm) is possible (Ponce 1989). Rainfall P is the total depth of storm rainfall. Runoff Q is the total depth of direct runoff resulting from storm rainfall P. Potential retention S is the maximum depth of storm rainfall that could potentially be abstracted by a given site.

In a typical case, a certain amount of rainfall, referred to as "initial abstraction," is abstracted as interception, infiltration, and surface storage before runoff begins. In the curve number method, this initial abstraction I_a is subtracted from rainfall P in (3) to yield

$$\frac{P - I_a - Q}{S} = \frac{Q}{P - I_a} \qquad (4)$$

Solving for Q in (4) yields

$$Q = \frac{(P - I_a)^2}{P - I_a + S} \qquad (5)$$

which is valid for $P > I_a$, that is, after runoff begins; and $Q = 0$ otherwise. With initial abstraction included in (4), the actual retention $P - Q$ asymptotically approaches a constant value $S + I_a$ as rainfall grows unbounded.

Eq. (5) has two parameters: S and I_a. To remove the necessity for an independent estimation of initial abstraction, a linear relationship between I_a and S was suggested [*SCS* (1985), and earlier versions]

$$I_a = \lambda S \qquad (6)$$

where λ = initial abstraction ratio.

Eq. (6) was justified on the basis of measurements in watersheds less than 10 acres in size (*SCS* 1985). While there was considerable scatter in the data, NEH-4 reported that 50% of the data points lay within the limits $0.095 \le \lambda \le 0.38$ [*SCS* (1985), and earlier versions]. This led SCS to adopt a standard value of the initial abstraction ratio $\lambda = 0.2$. However, values varying in the range $0.0 \le \lambda \le 0.3$ have been documented in a number of studies encompassing various geographical locations in the United States and other countries ("Estimation" 1972; Springer et al. 1980; Cazier and Hawkins 1984; Ramasastri and Seth 1985; Bosznay 1989).

With $\lambda = 0.2$ in (6), (5) becomes

$$Q = \frac{(P - 0.2S)^2}{P + 0.8S} \qquad (7)$$

subject to $P > 0.2S$; and $Q = 0$ otherwise.

Eq. (7) now contains only one parameter, potential retention S, which varies in the range $0 \le S \le \infty$. For convenience in practical applications, S is mapped into a dimensionless parameter CN, the curve number, which varies in the more appealing range $100 \ge \text{CN} \ge 0$. The chosen mapping equation is

$$S = \frac{1{,}000}{\text{CN}} - 10 \qquad (8)$$

where 1,000 and 10 are arbitrarily chosen constants having the same units as S (in.). Likewise

$$\text{CN} = \frac{1{,}000}{S + 10} \qquad (9)$$

A CN = 100 represents a condition of zero potential retention ($S = 0$), that is, an impermeable watershed. Conversely, a CN = 0 represents a theoretical upper bound to the potential retention ($S = \infty$), that is, an infinitely abstracting watershed.

Substituting (8) into (7), the equation relating direct runoff Q to storm rainfall P is obtained, with CN as the curve number, or curve parameter

$$Q = \frac{[\text{CN}(P + 2) - 200]^2}{\text{CN}[\text{CN}(P - 8) + 800]} \qquad (10)$$

subject to $P > (200/\text{CN}) - 2$; and $Q = 0$ otherwise.

Eq. (5) can be expanded to yield (Chen 1976; Hawkins 1978b)

$$Q = P - I_a - S + \frac{S^2}{P - I_a + S} \qquad (11)$$

This equation reveals that as potential runoff grows unbounded ($P - I_a \rightarrow \infty$), actual retention, excluding initial abstraction ($P - I_a - Q$), asymptotically approaches potential retention S. This is the basic tenet of the curve number method, that is, the asymptotic behavior of actual retention toward potential retention for sufficiently large values of potential runoff. Note that this behavior properly simulates the saturation overland flow mode of runoff generation. In this connection, Chen (1975, 1976, 1982) has derived an infiltration equation based on the curve number method, and related it to the Holtan infiltration equation, which explicitly accounts for available soil storage (Holtan et al. 1975).

In practice, there are some situations where the storm rainfall-runoff relationship does not follow (11) strictly. In these cases, fitting a curve number from data may prove to be a challenge (Hawkins 1993). Alternative rainfall-runoff models such as

$$Q = b(P - I_a) \qquad (12)$$

have been formulated (Fogel and Duckstein 1970; Hawkins 1992), but the problem remains to determine the empirical coefficient b, preferably as a function of runoff-producing properties. An apparent drawback of (12) is that as potential runoff grows unbounded ($P - I_a \rightarrow \infty$), actual retention also grows unbounded ($P - I_a - Q \rightarrow \infty$), simulating the capacity for infinite storage, that is, infinite potential retention. This same feature is shared by the classical infiltration formulas of Green and Ampt, Horton, and Phillip, a situation that has led to their being described as "bottomless," that is, able to simulate the Hortonian overland flow mode of runoff generation. On the other hand, the curve number method has a finite value of storage S for all curve numbers, excluding the special case of CN = 0, which in only a theoretical limit, and not used in practice.

The humble empirical beginnings of the curve number method in no way detract from its distinctive conceptual basis. Indeed, it is only under a conceptual modeling framework that we are able to discern *why* the retention and runoff ratios ought to be equal (Eq. 3). Equality of these ratios leads to a conceptual model where the curve number is the only parameter describing the process. In turn, this parameter is a surrogate for potential retention, a measure of available subsurface storage, that is, of the ability of a given site to abstract storm rainfall.

ANTECEDENT MOISTURE/RUNOFF CONDITION

A conceptual model works in the mean, implying that there is room for some variability. Early development of the runoff curve number method confirmed that this variability was indeed real, and that the same watershed could have more than one curve number, indeed, a set of curve numbers (*SCS* 1985; Hjelmfelt 1991). Among the likely sources of this variability are

1. The effect of the spatial variability of storm and watershed properties
2. The effect of the temporal variability of the storm, that is, the storm intensity
3. The quality of the measured data, that is, the *P*-*Q* sets
4. The effect of antecedent rainfall and associated soil moisture

The latter was recognized very early as the primary or tractable source of the variability, and thus, the concept of antecedent moisture condition (AMC) originated (*SCS* 1985).

More recently, the same concept has been referred to as the antecedent runoff condition (ARC) to denote a shift of emphasis from soil moisture to runoff ("Urban" 1986).

The original-handbook runoff curve numbers were developed from recorded rainfall-runoff data, where hydrologic soil group, land use/treatment class, and surface condition were known. Daily rainfall-runoff data corresponding to the annual flood series at a site were used in the method's development (Rallison and Cronshey 1979). The data was plotted as rainfall P in the abscissas and direct runoff Q in the ordinates. The CN corresponding to the curve that separated half of the plotted data from the other half was taken as the median curve number for the given site. The CN values of NEH-4 tables represent the average of median site CN values with the indicated soil, cover, and surface condition. The average condition was taken to mean average response, which was then extended to imply average soil moisture condition (Miller and Cronshey 1989). The natural scatter of points around the median CN was interpreted as a measure of the natural variability of soil moisture and associated rainfall-runoff relation.

To account for this variability, the P-Q plots were used to define enveloping or near-enveloping CN values for each site. While the theoretical bounds of curve number are CN = 0 (Q = 0) and CN = 100 ($Q = P$), the enveloping CN values reduce the limits to practical values based on site experience. These enveloping CN values are considered as the practical upper and lower limits of expected CN variability for the given soil-cover complex combination. Thus, antecedent moisture condition was used as a parameter to represent the experienced variability (Rallison and Cronshey 1979).

The curve number lying in the middle of the distribution is the median curve number, corresponding to AMC 2 (average runoff potential). This is the standard curve number given in the SCS and other applicable tables (*SCS* 1985). The low value is the dry curve number, of AMC 1 (lowest runoff potential). The high value is the wet curve number, of AMC 3 (highest runoff potential).

NEH-4 contains a conversion table (Table 10.1) listing corresponding AMC 1 and AMC 3 CN values for given AMC 2 CN values. The original values of this table, reported in the 1956 edition of NEH-4, were based on unsmoothed data. The values in the present AMC conversion table [in *SCS* (1985)] have been smoothed by fitting straight lines on normal probability paper. Capitalizing on this fact, Sobhani (1975) and Hawkins et al. (1985) developed correlations between the dry or wet potential retentions S_1 and S_3 and the average potential retention S_2. Hawkins et al. (1985) reported that

$$S_1 = 2.281S_2 \tag{13}$$

with $r^2 = 0.999$, and $S_e = 0.206$ in., and

$$S_3 = 0.427S_2 \tag{14}$$

with $r^2 = 0.994$, and $S_e = 0.088$ in.

These equations are applicable in the range $55 \leq \text{CN} \leq 95$, which encompasses most estimated or experienced curve numbers.

Substitution of (13) and (14) into (8) leads to

$$\text{CN}_1 = \frac{\text{CN}_2}{2.281 - 0.01281\text{CN}_2} \tag{15}$$

with $r^2 = 0.996$, and S_e = 1.0CN, and

$$\text{CN}_3 = \frac{\text{CN}_2}{0.427 + 0.00573\text{CN}_2} \tag{16}$$

with $r^2 = 0.994$, and S_e = 0.7CN.

The one-to-one relationship between CN and S [(8) and (9)] renders the latter intrinsically related to antecedent moisture. Thus, potential retention is a measure of the ability of a given site to abstract and retain storm rainfall, provided the level of antecedent moisture has been factored into the analysis. In other words, potential retention and its corresponding curve number are intended to reflect not only the capacity of a given site to abstract and retain storm rainfall, but also (1) the recent history of antecedent rainfall, or lack of it, which may have caused the soil moisture to depart from an average level; (2) seasonal variations in runoff properties; and (3) unusual storm conditions.

In this role, site moisture per se acts as a surrogate for all other sources of variability, beyond that which could be attributed to soil, land use/treatment, and surface condition. Hjelmfelt et al. (1982) found that the AMC conversion table described the 90% (AMC 1), 50% (AMC 2), and 10% (AMC 3) cumulative probabilities of exceedence of runoff depth for a given rainfall. In other words, they found that AMC 2 represented the central tendency, while AMC 1 and AMC 3 accounted for dispersion in the data. A similar analysis was performed by Gray et al. (1982) using data from Indiana, Kentucky, and Tennessee, and by Hawkins (1983), using data from Arizona and Utah. Hawkins et al. (1985) interpreted the AMC categories as "error bands" or envelopes indicating the experienced variability in rainfall-runoff data.

What level of AMC should be used in a given case? For this purpose, NEH-4 (*SCS* 1985) shows the appropriate AMC level based on the total 5-day antecedent rainfall, for dormant and growing season (Table 4.2: "Seasonal Rainfall Limits for AMC"). This table was developed using data from an unspecified location, and subsequently was adopted for general use (Miller and Cronshey 1989). Unfortunately, the table does not account for regional differences or scale effects. An antecedent period longer than 5 days would probably be required for larger watersheds. Echoing this concern, SCS has recently deleted Table 4.2 from the new version of Chapter 4, NEH-4, released in 1993.

In practice, a determination of AMC is left to the user, who must evaluate whether a certain design situation warrants either AMC 1, AMC 2, or AMC 3. It is understood that AMC 2 represents a typical design situation. A choice of AMC 1 results in lesser runoff volume, whereas greater runoff results from a choice of AMC 3. Design manuals specify the AMC choice as a function of return period, with AMC level increasing with return period. For example, the *Hydrology Manual* (1986) of Orange County, California, specifies AMC 1 for 2- and 5-yr storms, AMC 2 for 10-, 25-, and 50-yr storms, and AMC 3 for 100-yr storms. Likewise, the *Hydrology Manual* (1985) of San Diego County, California, specifies AMC values varying between 1.5 and 3.0 (in increments of 0.5) for a range of design frequencies (5–150 yr) and four climate regions: coast, foothills, mountains, and desert. While SCS does not endorse the use of fractional AMC levels (Rallison and Cronshey 1979), the practice exists and should be acknowledged.

RUNOFF CURVE NUMBERS EVALUATED FROM DATA

Since the method's inception, several investigators have attempted to determine runoff curve numbers from small watershed rainfall-runoff data. The objective has been either to verify the CN values given in the standard tables, or to extend the methodology to soil-cover complexes and geographic locations not covered in the NEH-4 handbook. An established procedure solves for S in (7), leading to (Hawkins 1973; 1979)

$$S = 5[P + 2Q - (4Q^2 + 5PQ)^{1/2}] \tag{17}$$

For a given P and Q pair, the potential retention S is calculated with this equation, and the corresponding CN is calculated using (9).

There are several ways to select the P-Q pairs for analysis. The standard method, referred to as the "annual flood series," is to select daily rainfall P and its corresponding runoff volume Q (both in inches) for the annual floods at a site (Rallison and Cronshey 1979; Springer et al. 1980). This procedure has the advantage that it results in a considerable range in rainfall and runoff values. Perceived disadvantages are that (1) this type of data is not readily available; (2) the return periods of corresponding rainfall and runoff events are not necessarily the same; and (3) there is only one data point per year of measurement.

In the absence of a long annual flood series, particularly in semiarid regions, some investigators have chosen to use less selective criteria for candidate storm events, including events of return period less than 1 yr (Woodward 1973; Hawkins 1984). This choice results in considerably more data for analysis, as well as slightly different CN values compared to those obtained using an annual flood series (Springer et al. 1980). The choice of frequency for candidate storm events is the subject of continuing research (Woodward and Gburek 1992).

Another approach to determine curve numbers from data is the frequency-matching method (Hjelmfelt 1980). The storm rainfall and direct runoff depths are sorted separately, and then realigned on a rank-order basis to form seemingly desirable P-Q pairs of equal return period. However, the individual runoff values are not necessarily associated with the causative rainfall values (Hawkins 1993).

OTHER EXPRESSIONS OF THE CURVE NUMBER EQUATION

The SCS runoff curve number has been applied in many countries throughout the world. Therefore, its expression in SI units is necessary. Likewise, geographic and other differences may dictate that the initial abstraction ratio λ be relaxed to the range validated by local experience, say $0.0 \le \lambda \le 0.3$.

In SI units, (10) converts to

$$Q = \frac{R\{\mathrm{CN}[(P/R) + 2] - 200\}^2}{\mathrm{CN}\{\mathrm{CN}[(P/R) - 8] + 800\}} \tag{18}$$

where P (cm) is divided by $R = 2.54$ (cm/in.), and the result of the computation is multiplied by R, to give Q in cm. Being dimensionless, the curve number CN remains the same in both U.S. customary and SI units. Eq. (18) is subject to the restriction that $P > R[(200/\mathrm{CN}) - 2]$; and $Q = 0$ otherwise.

To obtain the runoff curve number equation for a variable λ, (6) and (8) are substituted into (5) to yield (Ponce 1989)

$$Q = \frac{[\mathrm{CN}(P + 10\lambda) - 1{,}000\lambda]^2}{\mathrm{CN}\{\mathrm{CN}[P - 10(1 - \lambda)] + 1{,}000(1 - \lambda)\}} \tag{19}$$

which is subject to the restriction that $P > (1{,}000\lambda/\mathrm{CN}) - 10\lambda$; and $Q = 0$ otherwise.

Eq. (17) is applicable only for the standard value of initial abstraction $\lambda = 0.2$. For $\lambda = 0$

$$S = (P/Q)(P - Q) \tag{20}$$

In general, for $\lambda > 0$ (Chen 1982)

$$S = (\lambda^{-1})\{P + (0.5\lambda^{-1})[(1 - \lambda)Q - [(1 - \lambda)^2Q^2 + 4\lambda PQ]^{1/2}]\} \tag{21}$$

CRITIQUE OF RUNOFF CURVE NUMBER

There is a growing body of literature on the curve number method (Bosznay 1989; Hjelmfelt 1991; Hawkins 1993; Steenhuis et al. 1995). It will suffice here to enumerate the method's advantages and disadvantages. The advantages are

1. It is a simple, predictable, and stable conceptual method for the estimation of direct runoff depth based on storm rainfall depth, supported by empirical data.
2. It relies on only one parameter, the runoff curve number CN, which varies as a function of four major runoff-producing watershed properties:
   - Hydrologic soil group: A, B, C, and D
   - Land use and treatment classes: agricultural, range, forest, and, more recently, urban ("Urban" 1986)
   - Hydrologic surface condition of native pasture: poor, fair, and good
   - Antecedent moisture condition, a surrogate for other sources of variability, including soil moisture: 1, 2, and 3
3. It is the only agency methodology that features readily grasped and reasonably well-documented environmental inputs (soil, land use/treatment, surface condition, and antecedent moisture condition).
4. It is a well established method, having been widely accepted for use in the United States and other countries.

While it is theoretically possible for the curve numbers to span the range 0–100, practical design values validated by experience are more likely to be in the range 40–98, with few exceptions (Van Mullem 1989). This is a significant advantage, because it restricts the method's only parameter to a relatively narrow range. Viewed in this light, it is seen that estimating a design CN is indeed an empirical exercise within a conceptual modeling framework. Such an exercise is not unlike that of estimating Chezy's C or Manning's n in open-channel flow (Hawkins 1975).

Perceived disadvantages are

1. The method was originally developed using regional data, mostly from the midwestern United States, and has since been extended by way of practice to the entire United States and other countries. Some caution is recommended for its use in other geographic or climatic regions. Local studies and related experience should be substituted for the U.S. nationwide CN tables where appropriate.
2. In some instances, particularly for the lower curve numbers and/or rainfall depths, the method may be very sensitive to curve number and antecedent condition (Hawkins 1975; Bondelid et al. 1982; Ponce 1989). This is not necessarily a weak point, since it may be a reflection of the natural variability. There is, however, a lack of clear guidance on how to vary antecedent condition.
3. The method does best in agricultural sites, for which it was originally intended. Its applicability has since been extended to urban sites ("Urban" 1986). The method rates fairly in applications to range sites, and generally does poorly in applications to forest sites (Hawkins 1984, 1993). The implication here is that the runoff curve number (as developed by SCS) is better suited for storm rainfall-runoff estimates in streams with negligible baseflow, that is, those for which the ratio of direct runoff to total runoff is close to one. Typically, this is the case of streams of first and second order in subhumid and humid regions, and of ephemeral streams in arid and semiarid regions.
4. The method has no explicit provision for spatial scale effects. For example, Simanton et al. (1973) have shown that curve numbers for areas less than 560 acres (227

ha) in southeastern Arizona tend to decrease with increasing watershed size, reflecting the substantial role of channel transmission losses in this semiarid region. In the absence of clear guidelines, the runoff curve number is assumed to apply to small and midsize catchments, comparable in size to those that would normally fall within SCS scope. Without catchment subdivision and associated channel routing, its application to large catchments (say, greater than 100 sq mi, or 250 sq km) should be viewed with caution.
5. The method fixes the initial abstraction ratio at $\lambda = 0.2$. At first this appears to be an advantage, since it effectively reduces the number of parameters to one. In general, however, λ could be interpreted as a regional parameter to enhance the method's responsiveness to a diversity of geologic and climatic settings (Bosznay 1989; Ramasastri and Seth 1985). Additional research is needed to shed light on this issue.

RUNOFF CURVE NUMBER: HAS IT REACHED MATURITY?

Having reviewed its foundations, its conceptual/empirical basis, and its range of applicability, we now address the central issue of this paper: Has the runoff curve number method reached its maturity? Maturity implies usefulness, acceptance with faults acknowledged, understanding of its capabilities, and continued growth with possible eventual refinements.

We believe the method has now reached maturity on these counts:

1. The method is widely understood and accepted for what it is: a conceptual model supported with empirical data to estimate direct runoff volume from infrequent storm rainfall depth, lumped to circumvent the often cumbersome description of spatial and temporal variability of infiltration and other losses.
2. It is the method of choice by practicing engineers and hydrologists for soil and water conservation planning and design, and flood control design. The method is featured in most of the hydrologic computer models in current use, in the United States and abroad. Its practicality as a design method is beyond doubt.
3. A replacement method, if one is developed, would have to clearly prove its superiority. None of the existing point infiltration formulas, such as those of Horton, Philip, or Green and Ampt, are beyond reproach. An apparent limitation is that they allow an infinite amount of soil moisture storage. More importantly, however, is the criticism that none of these methods can claim a holistic approach, that is, one that accounts for the physical, chemical, and biological aspects of the phenomena, and that includes all relevant hydrologic processes. In many instances, the biological aspects of infiltration may be subject to such spatial diversity (the effect of vegetative subsurface features such as roots and root decay, and soil macro- and microfauna) as to defy description by even the most complex of models.

SUMMARY

The runoff curve number method owes its popularity among hydrology practitioners to its simplicity, predictability, and stability, and to its support by a major U.S. federal agency. In the four decades that have elapsed since its inception, questions have arisen as to its nature and beginnings. Its adoption and use throughout the United States and other countries, far beyond the scope intended by its original developers, have demanded that the method be subject to close scrutiny.

The method is a conceptual model of hydrologic abstraction of storm rainfall, supported by empirical data. Its objective is to estimate direct runoff volume from storm rainfall depth, based on a curve number CN. The curve number, which varies in the convenient range $100 \geq CN \geq 0$, is a surrogate for potential retention, a conceptual parameter varying in the range $0 \leq S \leq \infty$. The method does not take into account the spatial and temporal variability of infiltration and other abstractive losses; rather, it aggregates these into a calculation of the total depth loss for a given storm event and drainage area. The method works in the mean, by describing average trends, which precludes it from being perfectly predictive. The observed variability in curve numbers, beyond that which can be attributed to soil type, land use/treatment, and surface condition, is embodied in the concept of antecedent condition.

The advantages of the method are (1) its simplicity; (2) its predictability; (3) its stability; (4) its reliance on only one parameter; and (5) its responsiveness to major runoff-producing watershed properties. Perceived disadvantages are (1) its marked sensitivity to the choice of curve number; (2) the absence of clear guidance on how to vary antecedent moisture; (3) the method's varying accuracy for different biomes; (4) the absence of an explicit provision for spatial scale effects; and (5) the fixing of the initial abstraction ratio at $\lambda = 0.2$, preempting a regionalization based on geologic and climatic setting.

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APPENDIX II. NOTATION

The following symbols are used in this paper:

b = exponent in eq. (2), coefficient in eq. (12);

CN = runoff curve number;
CN_1 = dry curve number (AMC 1);
CN_2 = average curve number (AMC 2);
CN_3 = wet curve number (AMC 3);
F = actual retention;
I_a = initial abstraction;
L = abstractive losses;
P = rainfall, potential runoff;
$P - Q$ = actual retention;
Q = runoff, actual runoff;
R = unit conversion factor;
r = correlation coefficient;
S = potential retention;
S_1 = dry potential retention;
S_2 = average potential retention;
S_3 = wet potential retention;
S_e = standard error of estimate; and
λ = initial abstraction ratio.

APPENDIX E

TIME OF CONCENTRATION

Chapter 3 Time of Concentration and Travel Time

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

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

Factors affecting time of concentration and travel time

Surface roughness

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

Channel shape and flow patterns

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

Slope

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

Computation of travel time and time of concentration

Water moves through a watershed as sheet flow, shallow concentrated flow, open channel flow, or some combination of these. The type that occurs is a function of the conveyance system and is best determined by field inspection.

Travel time (T_t) is the ratio of flow length to flow velocity:

$$T_t = \frac{L}{3600V} \qquad \text{[eq. 3-1]}$$

where:

T_t = travel time (hr)
L = flow length (ft)
V = average velocity (ft/s)
3600 = conversion factor from seconds to hours.

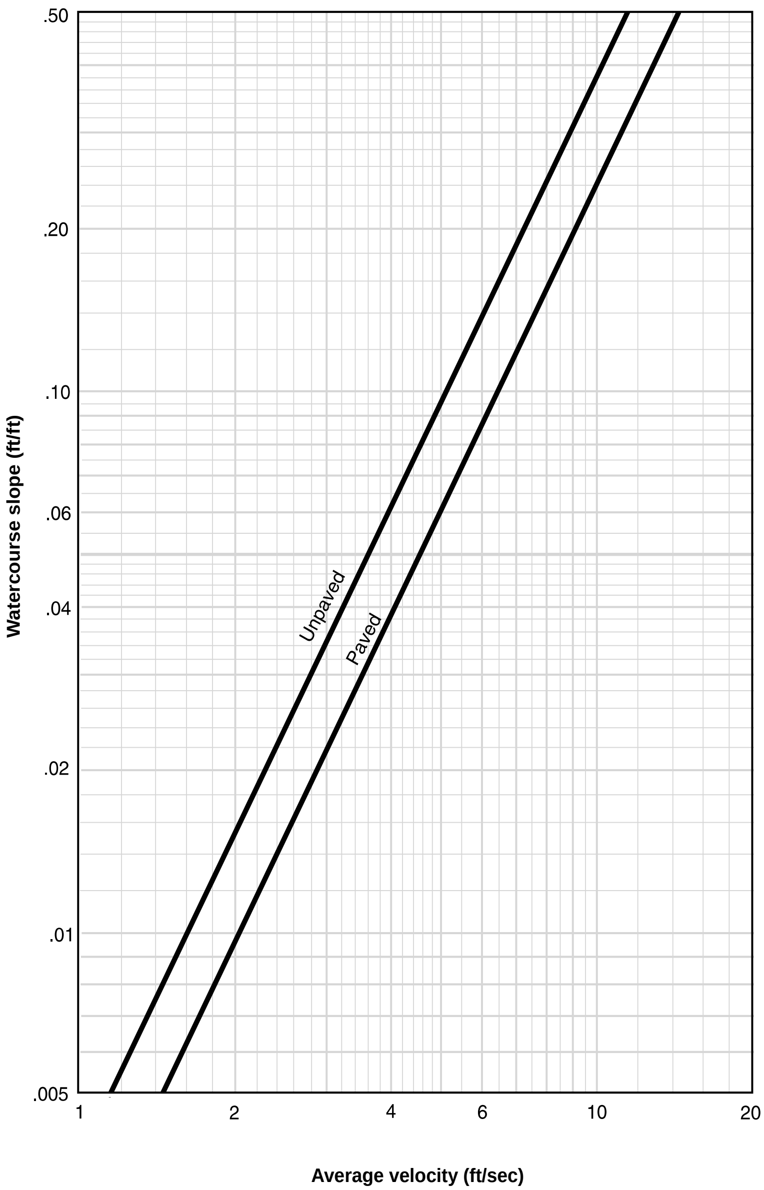
Time of concentration (T_c) is the sum of T_t values for the various consecutive flow segments:

$$T_c = T_{t_1} + T_{t_2} + K\ T_{t_m} \qquad \text{[eq. 3-2]}$$

where:

T_c = time of concentration (hr)
m = number of flow segments

Figure 3-1 Average velocities for estimating travel time for shallow concentrated flow



Sheet flow

Sheet flow is flow over plane surfaces. It usually occurs in the headwater of streams. With sheet flow, the friction value (Manning's n) is an effective roughness coefficient that includes the effect of raindrop impact; drag over the plane surface; obstacles such as litter, crop ridges, and rocks; and erosion and transportation of sediment. These n values are for very shallow flow depths of about 0.1 foot or so. Table 3-1 gives Manning's n values for sheet flow for various surface conditions.

Table 3-1 Roughness coefficients (Manning's n) for sheet flow

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

1 The n values are a composite of information compiled by Engman (1986).
2 Includes species such as weeping lovegrass, bluegrass, buffalo grass, blue grama grass, and native grass mixtures.
3 When selecting n , consider cover to a height of about 0.1 ft. This is the only part of the plant cover that will obstruct sheet flow.

For sheet flow of less than 300 feet, use Manning's kinematic solution (Overtop and Meadows 1976) to compute T_t:

$$T_t = \frac{0.007(nL)^{0.8}}{(P_2)^{0.5} s^{0.4}} \qquad \text{[eq. 3-3]}$$

where:

T_t = travel time (hr),
n = Manning's roughness coefficient (table 3-1)
L = flow length (ft)
P_2 = 2-year, 24-hour rainfall (in)
s = slope of hydraulic grade line (land slope, ft/ft)

This simplified form of the Manning's kinematic solution is based on the following: (1) shallow steady uniform flow, (2) constant intensity of rainfall excess (that part of a rain available for runoff), (3) rainfall duration of 24 hours, and (4) minor effect of infiltration on travel time. Rainfall depth can be obtained from appendix B.

Shallow concentrated flow

After a maximum of 300 feet, sheet flow usually becomes shallow concentrated flow. The average velocity for this flow can be determined from figure 3-1, in which average velocity is a function of watercourse slope and type of channel. For slopes less than 0.005 ft/ft, use equations given in appendix F for figure 3-1. Tillage can affect the direction of shallow concentrated flow. Flow may not always be directly down the watershed slope if tillage runs across the slope.

After determining average velocity in figure 3-1, use equation 3-1 to estimate travel time for the shallow concentrated flow segment.

Open channels

Open channels are assumed to begin where surveyed cross section information has been obtained, where channels are visible on aerial photographs, or where blue lines (indicating streams) appear on United States Geological Survey (USGS) quadrangle sheets. Manning's equation or water surface profile information can be used to estimate average flow velocity. Average flow velocity is usually determined for bank-full elevation.

Manning's equation is:

$$V = \frac{1.49 r^{\frac{2}{3}} s^{\frac{1}{2}}}{n} \qquad \text{[eq. 3-4]}$$

where:

V = average velocity (ft/s)
r = hydraulic radius (ft) and is equal to a/p_w
a = cross sectional flow area (ft^2)
p_w = wetted perimeter (ft)
s = slope of the hydraulic grade line (channel slope, ft/ft)
n = Manning's roughness coefficient for open channel flow.

Manning's n values for open channel flow can be obtained from standard textbooks such as Chow (1959) or Linsley et al. (1982). After average velocity is computed using equation 3-4, T_t for the channel segment can be estimated using equation 3-1.

Reservoirs or lakes

Sometimes it is necessary to estimate the velocity of flow through a reservoir or lake at the outlet of a watershed. This travel time is normally very small and can be assumed as zero.

Limitations

- Manning's kinematic solution should not be used for sheet flow longer than 300 feet. Equation 3-3 was developed for use with the four standard rainfall intensity-duration relationships.

- In watersheds with storm sewers, carefully identify the appropriate hydraulic flow path to estimate T_c. Storm sewers generally handle only a small portion of a large event. The rest of the peak flow travels by streets, lawns, and so on, to the outlet. Consult a standard hydraulics textbook to determine average velocity in pipes for either pressure or nonpressure flow.

- The minimum T_c used in TR-55 is 0.1 hour.

- A culvert or bridge can act as a reservoir outlet if there is significant storage behind it. The procedures in TR-55 can be used to determine the peak flow upstream of the culvert. Detailed storage routing procedures should be used to determine the outflow through the culvert.

Example 3-1

The sketch below shows a watershed in Dyer County, northwestern Tennessee. The problem is to compute T_c at the outlet of the watershed (point D). The 2-year 24-hour rainfall depth is 3.6 inches. All three types of flow occur from the hydraulically most distant point (A) to the point of interest (D). To compute T_c, first determine T_t for each segment from the following information:

Segment AB: Sheet flow; dense grass; slope (s) = 0.01 ft/ft; and length (L) = 100 ft. Segment BC: Shallow concentrated flow; unpaved; s = 0.01 ft/ft; and L = 1,400 ft. Segment CD: Channel flow; Manning's n = .05; flow area (a) = 27 ft^2; wetted perimeter (p_w) = 28.2 ft; s = 0.005 ft/ft; and L = 7,300 ft.

See figure 3-2 for the computations made on worksheet 3.

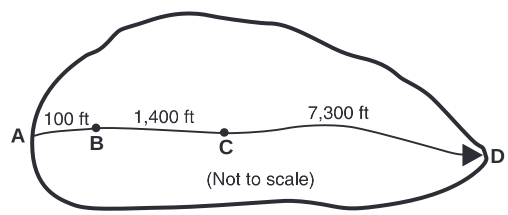


Figure 3-2 Worksheet 3 for example 3-1

Worksheet 3: Time of Concentration (T_c) or travel time (T_t)

Project *Heavenly Acres*	By *DW*	Date *10/6/85*
Location *Dyer County, Tennessee*	Checked *NM*	Date *10/8/85*

Check one: ☐ Present ☒ Developed

Check one: ☒ T_c ☐ T_t through subarea

Notes: Space for as many as two segments per flow type can be used for each worksheet.
Include a map, schematic, or description of flow segments.

Sheet flow (Applicable to T_c only)

	Segment ID	AB		
1.	Surface description (table 3-1)	Dense Grass		
2.	Manning's roughness coefficient, n (table 3-1)	0.24		
3.	Flow length, L (total L ≤ 300 ft) ... ft	100		
4.	Two-year 24-hour rainfall, P_2 ... in	3.6		
5.	Land slope, s ... ft/ft	0.01		
6.	$T_t = \frac{0.007\ (nL)^{0.8}}{P_2^{\ 0.5}\ s^{0.4}}$ Compute T_t ... hr	0.30	+	= 0.30

Shallow concentrated flow

	Segment ID	BC		
7.	Surface description (paved or unpaved)	Unpaved		
8.	Flow length, L ... ft	1400		
9.	Watercourse slope, s ... ft/ft	0.01		
10.	Average velocity, V (figure 3-1) ... ft/s	1.6		
11.	$T_t = \frac{L}{3600\ V}$ Compute T_t ... hr	0.24	+	= 0.24

Channel flow

	Segement ID	CD		
12.	Cross sectional flow area, a ... ft^2	27		
13.	Wetted perimeter, p_w ... ft	28.2		
14.	Hydraulic radius, $r = \frac{a}{p_w}$ Compute r ... ft	0.957		
15	Channel slope, s ... ft/ft	0.005		
16.	Manning's roughness coefficient, n ...	0.05		
17.	$V = \frac{1.49\ r^{2/3}\ s^{1/2}}{n}$ Compute V ... ft/s	2.05		
18.	Flow length, L ... ft	7300		
19.	$T_t = \frac{L}{3600\ V}$ Compute T_t ... hr	0.99	+	= 0.99
20.	Watershed or subarea T_c or T_t (add T_t in steps 6, 11, and 19) ... Hr			1.53

TR 55 Worksheet 3: Time of Concentration (T_c) or Travel Time (T_t)

Project: ____________________ Designed By: ____________ Date: ________

Location: ____________________ Checked By: ____________ Date: ________

Check one: ☐ Present ☐ Developed

Check one: ☐ T_c ☐ T_t ☐ through subarea ____________________

NOTES: Space for as many as two segments per flow type can be used for each worksheet. Include a map, schematic, or description of flow segments.

<u>Sheet Flow</u> (Applicable to T_c only)

Segment ID				
1. Surface description (Table 3-1)				
2. Manning's roughness coeff., n (Table 3-1)				
3. Flow length, L (total L $\leq$ 100 ft) .... ft				
4. Two-year 24-hour rainfall, P_2 .... in				
5. Land slope, s .... ft/ft				
6. $T_t = \dfrac{0.007\,(nL)^{0.8}}{P_2^{0.5}\, s^{0.4}}$ Compute T_t .... hr		+		=

<u>Shallow Concetrated Flow</u>

Segment ID				
7. Surface description (paved or unpaved)				
8. Flow length, L .... ft				
9. Watercourse slope, s .... ft/ft				
10. Average velocity, V (Figure 3-1) .... ft/s				
11. $T_t = \dfrac{L}{3600\,V}$ Compute T_t .... hr		+		=

<u>Channel Flow</u>

Segment ID				
12. Cross sectional flow area, a .... ft^2				
13. Wetted perimeter, P_w .... ft				
14. Hydraulic radius, $r = \dfrac{a}{P_w}$ Compute r .... ft				
15. Channel Slope, s .... ft/ft				
16. Manning's Roughness Coeff., n				
17. $V = \dfrac{1.49\, r^{2/3}\, s^{1/2}}{n}$ Compute V .... ft/s				
18. Flow length, L .... ft				
19. $T_t = \dfrac{L}{3600\,V}$ Compute T_t .... hr		+		=

20. Watershed or subarea T_c or T_t (add T_t in steps 6, 11, and 19 .... hr ☐

Print Form | Reset Form | Save Form

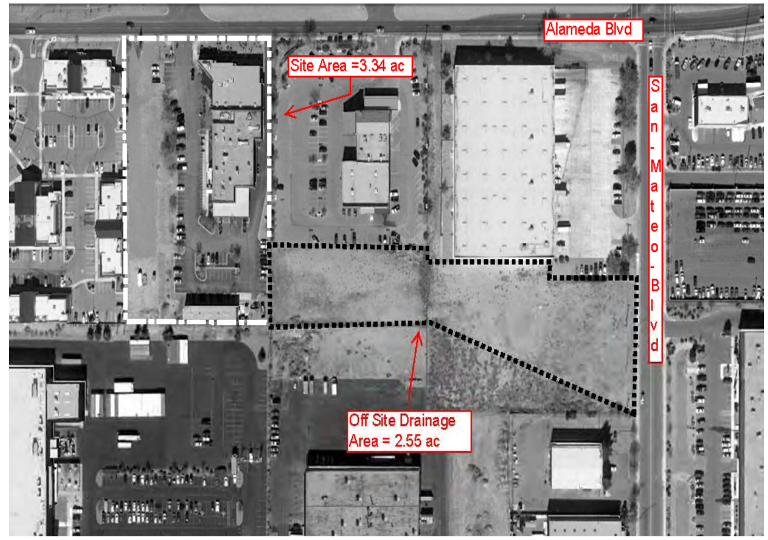
APPENDIX F

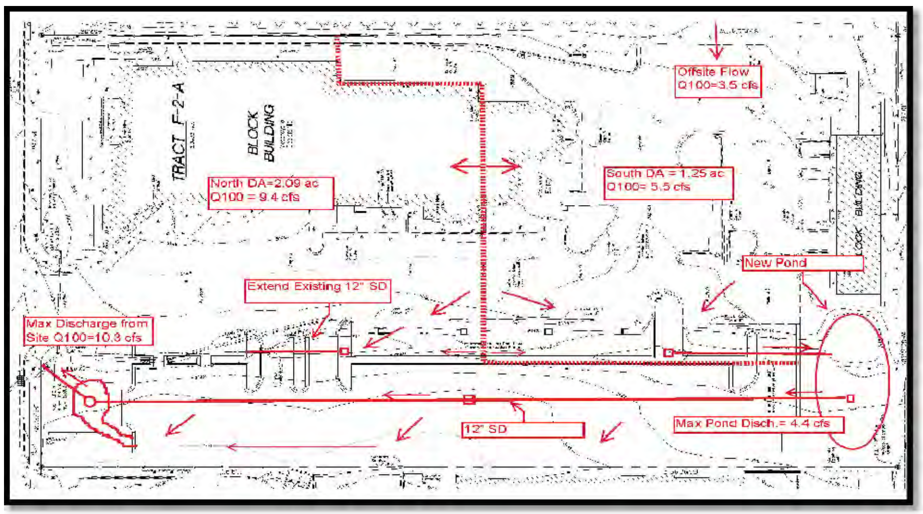
METHOD COMPARISONS

Thunderbird Harley Davidson Site - Hydrology Comparisons (DPM/AHYMO v Preposed HMS)

Rainfall (in.)		DPM	NOAA 14
1 hr		2.01	1.77
6 hr		2.35	2.37
24 hr		2.75	NA

Basin/Component	Area	Q_{100} AHYMO	Q_{100} HMS	V_{100} AHYMO	V_{100} HMS
Offsite	2.55 ac	3.54 cfs	3.4 cfs	4922 ft^3	6109 ft^3
South Basin	1.25 ac	5.49 cfs	5.8 cfs	10,484 ft^3	9574 ft^3
Pond	3.8 ac	4.43 cfs	4.1 cfs	15,403 ft^3	15,449 ft^3
North Basin	2.09 ac	9.04 cfs	9.7 cfs	17,298 ft^3	16,008 ft^3
Total at Alameda	5.89 ac	10.3 cfs	11.0 cfs	32,696 ft^3	31,430 ft^3





EC-HMS & AHYMO Summary

	Model Method			HEC-HMS								AHYMO								
	Name	Acres	Mi^2	Rainfall (100yr 6hr)	Landuse	CN (Western desert urban areas) pervious areas	% Impervious	Tc (min)	Peak Discharge (cfs)	Time of Peak	Volume (ac-ft)	Rainfall (100yr 6hr)	Precept Zone	Land Treatment A	Land Treatment B	Land Treatment C	Land Treatment D	Tc (min)	Peak (cfs)	Volume (ac-ft)
Commercial	16-1A West Side	1	0.0015625	2.26	Commercial	**63**	90	5.00	5	01Jul2012, 01:34	0.171	2.2	1	0%	10%	0%	90%	12	3.95	0.153
	17-2A West Side	2	0.003125	2.26	Commercial	**63**	90	5.00	11	01Jul2012, 01:34	0.341	2.2	1	0%	10%	0%	90%	12	7.89	0.306
	18-5A West Side	5	0.0078125	2.26	Commercial	**63**	90	5.00	26	01Jul2012, 01:34	0.854	2.2	1	0%	10%	0%	90%	12	19.69	0.7649
	19-10A West Side	10	0.015625	2.26	Commercial	**63**	90	5.02	53	01Jul2012, 01:34	1.7	2.2	1	0%	10%	0%	90%	12	39.37	1.5298
	20-20A West Side	20	0.03125	2.26	Commercial	**63**	90	6.16	100	01Jul2012, 01:35	3.4	2.2	1	0%	10%	0%	90%	12	78.72	3.0595
Multi Use	22-20A West Side	20	0.03125	2.26	75% Residential (4DU) & 25% Commercial	**63**	54	6.16	61	01Jul2012, 01:35	2.15	2.2	1	0%	29%	17%	54%	12	63.16	2.3645
	1-40A West Side	40	0.0625	2.26	75% Residential (4DU) & 25% Commercial	**63**	54	8	110	01Jul2012, 01:37	4.3	2.2	1	0%	29%	17%	54%	12	129.66	4.7259
	2-80A West Side	80	0.125	2.26	75% Residential (4DU) & 25% Commercial	**63**	54	10	209	01Jul2012, 01:38	8.6	2.2	1	0%	29%	17%	54%	12	259.15	9.4518
	3-120A West Side	120	0.1875	2.26	75% Residential (4DU) & 25% Commercial	**63**	54	10	304	01Jul2012, 01:38	12.9	2.2	1	0%	29%	17%	54%	12	388.54	14.1777
	4-160A West Side	160	0.25	2.26	75% Residential (4DU) & 25% Commercial	**63**	54	11	393	01Jul2012, 01:39	17.3	2.2	1	0%	29%	17%	54%	12	517.89	18.9035
	5-200A West Side	200	0.3125	2.26	75% Residential (4DU) & 25% Commercial	**63**	54	12	481	01Jul2012, 01:39	21.6	2.2	1	0%	10%	0%	90%	12	647.21	23.6294
Commercial	11-1A East Side	1	0.0015625	2.44	Commercial	**77**	90	5.00	6	01Jul2012, 01:34	0.1883	2.6	3	0%	10%	0%	90%	12	4.57	0.1843
	12-2A East Side	2	0.003125	2.44	Commercial	**77**	90	5.00	11	01Jul2012, 01:34	0.376	2.6	3	0%	10%	0%	90%	12	9.11	0.3686
	13-5A East Side	5	0.0078125	2.44	Commercial	**77**	90	5.00	28	01Jul2012, 01:34	0.941	2.6	3	0%	10%	0%	90%	12	22.75	0.9215
	14-10A East Side	10	0.015625	2.44	Commercial	**77**	90	5.02	56	01Jul2012, 01:34	1.9	2.6	3	0%	10%	0%	90%	12	45.48	1.8431
	15-20A East Side	20	0.03125	2.44	Commercial	**77**	90	6.16	107	01Jul2012, 01:35	3.8	2.6	3	0%	29%	17%	54%	12	90.94	3.6862
Multi Use	21-20A East Side	20	0.03125	2.44	75% Residential (4DU) & 25% Commercial	**77**	54	6.16	77	01Jul2012, 01:35	2.7	2.6	3	0%	29%	17%	54%	12	76.42	2.9198
	6-40A East Side	40	0.0625	2.44	75% Residential (4DU) & 25% Commercial	**77**	54	8	138	01Jul2012, 01:37	5.5	2.6	3	0%	29%	17%	54%	12	152.81	5.8397
	7-80A East Side	80	0.125	2.44	75% Residential (4DU) & 25% Commercial	**77**	54	10	263	01Jul2012, 01:38	10.9	2.6	3	0%	29%	17%	54%	12	307.04	11.6793
	8-120A East Side	120	0.1875	2.44	75% Residential (4DU) & 25% Commercial	**77**	54	10	380	01Jul2012, 01:38	16.4	2.6	3	0%	29%	17%	54%	12	462.86	17.519
	9-160A East Side	160	0.25	2.44	75% Residential (4DU) & 25% Commercial	**77**	54	11	493	01Jul2012, 01:39	21.8	2.6	3	0%	29%	17%	54%	12	620.4	23.3587
	10-200A East Side	200	0.3125	2.44	75% Residential (4DU) & 25% Commercial	**77**	54	12	600	01Jul2012, 01:39	27.3	2.6	3	0%	0%	0%	0%	12	779.84	29.1983

Model Comparison

% Difference HMS vs AHYMO

			HMS Values			Ahymo Values		Rainfall (in)		Peak Discharge		Volume (ac-ft)	
Sub Basin	acres	Rainfall (in)	Peak Discharge (cfs)	Volume (ac-ft)	Rainfall (in)	Peak (cfs)	Volume (ac-ft)	Depth (in)	Percent	(cfs)	Percent	Ac-ft	Percent
16-1A West Side	1	2.26	5	0.171	2.2	3.95	0.153	-0.06	3%	-1	25%	-0.018	11%
17-2A West Side	2	2.26	11	0.341	2.2	7.89	0.306	-0.06	3%	-3	25%	-0.035	10%
18-5A West Side	5	2.26	26	0.854	2.2	19.69	0.7649	-0.06	3%	-7	25%	-0.0891	10%
19-10A West Side	10	2.26	53	1.7	2.2	39.37	1.5298	-0.06	3%	-13	25%	-0.1702	10%
20-20A West Side	20	2.26	100	3.4	2.2	78.72	3.0595	-0.06	3%	-22	22%	-0.3405	10%
22-20A West Side	20	2.26	61	2.15	2.2	63.16	2.3645	-0.06	3%	3	4%	0.2145	10%
1-40A West Side	40	2.26	110	4.3	2.2	129.66	4.7259	-0.06	3%	20	18%	0.4259	10%
2-80A West Side	80	2.26	209	8.6	2.2	259.15	9.4518	-0.06	3%	51	24%	0.8518	10%
3-120A West Side	120	2.26	304	12.9	2.2	388.54	14.1777	-0.06	3%	85	28%	1.2777	10%
4-160A West Side	160	2.26	393	17.3	2.2	517.89	18.9035	-0.06	3%	125	32%	1.6035	9%
5-200A West Side	200	2.26	481	21.6	2.2	647.21	23.6294	-0.06	3%	166	35%	2.0294	9%
11-1A East Side	1	2.44	6	0.1883	2.6	4.57	0.1843	0.16	-7%	-1	18%	-0.004	2%
12-2A East Side	2	2.44	11	0.376	2.6	9.11	0.3686	0.16	-7%	-2	19%	-0.0074	2%
13-5A East Side	5	2.44	28	0.941	2.6	22.75	0.9215	0.16	-7%	-5	19%	-0.0195	2%
14-10A East Side	10	2.44	56	1.9	2.6	45.48	1.8431	0.16	-7%	-11	19%	-0.0569	3%
15-20A East Side	20	2.44	107	3.8	2.6	90.94	3.6862	0.16	-7%	-16	15%	-0.1138	3%
21-20A East Side	20	2.44	77	2.7	2.6	76.42	2.9198	0.16	-7%	0	0%	0.2198	8%
6-40A East Side	40	2.44	138	5.5	2.6	152.81	5.8397	0.16	-7%	15	11%	0.3397	6%
7-80A East Side	80	2.44	263	10.9	2.6	307.04	11.6793	0.16	-7%	44	17%	0.7793	7%
8-120A East Side	120	2.44	380	16.4	2.6	462.86	17.519	0.16	-7%	83	22%	1.119	7%
9-160A East Side	160	2.44	493	21.8	2.6	620.4	23.3587	0.16	-7%	127	26%	1.5587	7%
10-200A East Side	200	2.44	600	27.3	2.6	779.84	29.1983	0.16	-7%	180	30%	1.8983	7%

Comparison of All Methods

1 min time step

		(new 5 min tc)	HMS Values			Ahymo Values			Rational Method DPM	Rational Method New Method	% Difference AHYMO vs DPM Rational	% Difference HMS vs New Method
	Sub Basin	acres	Rainfall (in)	Peak Discharge (cfs)	Volume (ac-ft)	Rainfall (in)	Peak (cfs)	Volume (ac-ft)	Peak (cfs)	Peak (cfs)		
Commercial	16-1A West Side	1	2.26	**5.3**	0.171	2.2	**3.95**	0.153	**4**	**5**	5%	-7%
Commercial	17-2A West Side	2	2.26	**10.5**	0.341	2.2	**7.89**	0.306	**8**	**10**	5%	-6%
Commercial	18-5A West Side	5	2.26	**26.3**	0.854	2.2	**19.69**	0.7649	**21**	**25**	5%	-6%
Commercial	19-10A West Side	10	2.26	**52.5**	1.7	2.2	**39.37**	1.5298	**41**	**49**	5%	-7%
Commercial	20-20A West Side	20	2.26	**100.3**	3.4	2.2	**78.72**	3.0595	**83**	**86**	5%	-14%
Multi Use	22-20A West Side	20	2.26	**60.6**	2.15	2.2	**63.16**	2.3645	**69**	**51**	9%	-16%
Multi Use	1-40A West Side	40	2.26	**109.5**	4.3	2.2	**129.66**	4.7259	**137**	**90**	6%	-17%
Multi Use	2-80A West Side	80	2.26	**208.6**	8.6	2.2	**259.15**	9.4518	**275**	**181**	6%	-13%
Multi Use	3-120A West Side	120	2.26	**303.7**	12.9	2.2	**388.54**	14.1777	**412**	**249**	6%	-18%
Multi Use	4-160A West Side	160	2.26	**393.3**	17.3	2.2	**517.89**	18.9035	**550**	**331**	6%	-16%
Multi Use	5-200A West Side	200	2.26	**480.8**	21.6	2.2	**647.21**	23.6294	**687**	**411**	6%	-15%
Commercial	11-1A East Side	1	2.44	**5.6**	0.1883	2.6	**4.57**	0.1843	**5**	**5**	4%	-4%
Commercial	12-2A East Side	2	2.44	**11.3**	0.376	2.6	**9.11**	0.3686	**10**	**11**	4%	-4%
Commercial	13-5A East Side	5	2.44	**28.2**	0.941	2.6	**22.75**	0.9215	**24**	**27**	5%	-4%
Commercial	14-10A East Side	10	2.44	**56.2**	1.9	2.6	**45.48**	1.8431	**48**	**54**	5%	-4%
Commercial	15-20A East Side	20	2.44	**107.4**	3.8	2.6	**90.94**	3.6862	**95**	**95**	5%	-12%
Multi Use	21-20A East Side	20	2.44	**76.5**	2.7	2.6	**76.42**	2.9198	**81**	**59**	6%	-23%
Multi Use	6-40A East Side	40	2.44	**138**	5.5	2.6	**152.81**	5.8397	**161**	**104**	6%	-24%
Multi Use	7-80A East Side	80	2.44	**262.7**	10.9	2.6	**307.04**	11.6793	**323**	**209**	5%	-20%
Multi Use	8-120A East Side	120	2.44	**379.7**	16.4	2.6	**462.86**	17.519	**484**	**287**	5%	-24%
Multi Use	9-160A East Side	160	2.44	**493.1**	21.8	2.6	**620.4**	23.3587	**646**	**382**	4%	-23%
Multi Use	10-200A East Side	200	2.44	**599.7**	27.3	2.6	**779.84**	29.1983	**807**	**474**	4%	-21%

Summary of Pond Routings based on 25% Intensity Position

Pond	Model Description	Design Volume	100 Yr-24 Hr Peak Storage Volume	100 Yr-24 Hr Inflow Volume	100 Yr-24 Hr Outflow Volume	100 Yr-24 Hr Inflow	100 Yr-24 Hr Outflow	Elevation of Emergency Spillway	100 Yr-24 Hr Peak Water Surface Elevation	Freeboard from Emergency Spillway	Available Storage	Comments
		ac-ft	ac-ft	ac-ft	cfs	cfs	cfs	ft	ft	ft	ac-ft	
		a						a		b		
Pond 4	*Smith DEVEX Conditions Results from Report*	*8.51*	*4.50*	--	--	--	--	*5155.1*	*5152.9*	*2.2*	*4.01*	*All values reported on this table are taken directly from The Master Drainage Plan for the West Side Transit Facility by Smith Engineering Company, 2001.*
Pond 6	*"*	*9.01*	*6.20*	--	--	--	--	*5177.9*	*5176.7*	*1.2*	*2.81*	
Pond 4	DEVEX Option 1	8.51	4.90	23.5	23.5	135.3	96.1	5155.1	5153.31	1.8	3.61	Watershed modeled as fully developed commercial/business site at 90% impervious, using latest NOAA 14 100-Yr-24Hr rainfall depth of 2.52 in. Basin C-2D.2 drains to Pond 5 with modified outfall restricting discharge using a 12" outlet pipe as principal spillway
Pond 5	DEVEX Option 1	4.73	3.10	5.43	5.43	68.2	10.5	5168.8	5167.66	1.1	1.63	
Pond 6	DEVEX Option 1	9.01	6.89	13.8	13.8	224.6	82.9	5177.9	5177.5	0.4	2.12	

a- All values reported on this table are taken directly from The Master Drainage Plan for the West Side Transit Facility
100 Yr 24 Hr rainfall depth based on lates NOAA Atlas 14 data
b - Freeboard = Elevation of Emergency Spillway - Peak Water Surface Elevation

APPENDIX G

NMDOT RATIONAL FORMULA

403 Rational Formula Method

Hydrologic analyses performed on small (<160 acre) watersheds will normally be performed using the Rational Formula. The Rational Formula Method is a widely and long accepted procedure worldwide for estimating peak rates of runoff from small watersheds. The Rational Formula may be used on NMDOT projects for roadway drainage facilities and small drainage structures as described in **Section 401** (**Figure 401-1 and Figure 401-2**) of this manual. The standard form of the equation in English units is:

$$\mathbf{Q = CiA} \qquad \textbf{403-1}$$

Where:

Q = the peak rate of runoff, in cfs

C = a dimensionless runoff coefficient

i = the rainfall intensity, in inches/hour

A = the watershed or drainage area, in acres

The units in the Rational Formula equation do not yield cfs directly, but rather are in acre-inches/hour. However, the conversion from acre-inches/hour to cfs is 1.008 which is commonly neglected and because it does not introduce a significant error. The Rational Formula has several assumptions implicit to the method, including:

- The rainfall intensity is uniform for a duration equal to or greater than Tc
- Peak flow occurs when the entire watershed is contributing runoff
- The frequency of the resulting peak discharge is equal to the frequency of the rainfall event
- Both Rational ‘C’ Coefficient and rainfall intensity (i) vary with the return period (both tend to increase as return period increases). Therefore, both must be determined separately for each design storm frequency.
- Rational ‘C’ Coefficient is dependent on the Hydrologic Soil Group (HSG) and the vegetative cover or in the case of developed watersheds, the percentage of impervious cover. HSG’s are divided into four soil groups and are described in **Section 402.4** Soils Data:

Limitations for using the Rational Formula on NMDOT projects include the following:

- The total drainage area should not exceed 160 acres
- Land use, slope, and soils are fairly consistent throughout the watershed
- There are no diversions, detention basins, pump stations or other structures in the watershed which would require the routing of a flood hydrograph
- The time of concentration does not exceed one hour

403.1 Time of Concentration–Tc for use in the Rational Formula

The assumptions within the Rational Formula are that the rainfall intensity is uniform for a duration equal to or greater than Tc and that the entire watershed is contributing runoff when the peak occurs. Therefore, in order to determine the appropriate rainfall intensity "i" for the watershed, the Tc must be determined. For NMDOT projects, Tc shall be calculated using the Kirpich Equation (or a derivation of it) or Upland Method depending on specific circumstances.

The Upland Method is used to estimate travel times for overland flow and shallow concentrated flow conditions. Originally developed by the Soil Conservation Service (SCS now Natural Resource Conservation Service – NRCS), the upland method is limited to use in watersheds less than 2000 acres in size, or to the upper reaches of larger watersheds. For NMDOT projects the Upland Method may be used for computing the time of concentration when using the Rational Method or the Simplified Peak Flow method on an ***un-gullied*** watershed. The use of Upland Method is described in **Section 402.8**.

When using the Rational Method the Kirpich Equation should be used in watersheds ***when gullying is evident in more than 10% of the primary watercourse***. Gullying can be assumed if a blue line appears on the watercourse shown on the USGS quadrangle topographic map or is apparent from field reconnaissance or from inspection of aerial photography. The Kirpich Equation is given as:

$$\mathrm{Tc} = 0.007\mathrm{L}^{0.77}\mathrm{S}^{-0.385} \qquad \textbf{403-2}$$

Where:

T_c = time of concentration (minutes)

L = Maximum length of water travel (ft)

S = surface slope, given by H/L (ft/ft)

H = difference in elevation between the most hydraulically remote point in the drainage basin and the outlet (ft)

In small watersheds where the slope is very flat and the flow path of the hydraulically longest flow path is dominated by overland flow (> 300 ft), the Kerby Equation should be considered for the overland flow portion and Kerby for the channelized portion.

The Kerby Equation is given as:

$$\mathrm{Tc} = [\frac{2.2\mathrm{nl}}{\mathrm{S}^{0.5}}]^{0.324} \qquad 403\text{-}3$$

Where:

Tc = time of concentration, minutes

l = length of flow path from headwater to outlet, ft

S = average slope, ft/ft

n = Manning's roughness coefficient

When Kirpich and Kerby are combined (Kirpich-Kerby) the watershed should be divided between the channelized and the overland flow portions and the travel time across each reach calculated and then added together for the total Tc.

- if the calculations (with either Kirpich or with Kirpich-Kerby) yield a Tc less than 10 minutes, use 10 minutes.
- IF THE RESULTING Tc IS GREATER THAN 1 HOUR, DO NOT USE THE RATIONAL METHOD - SELECT ANOTHER HYDROLOGIC ANALYSIS METHOD.

403.2 Rainfall

Developing IDF Curves and Depth Duration Values for Rational Formula from NOAA Precipitation Frequency Data Server

The following approach is provided to develop the Intensity Duration Frequency Curves from which rainfall intensity "i" for the design frequency storm required for using the Rational Formula.

1. Go to NOAA Precipitation Frequency Data Server (PFDS) http://hdsc.nws.noaa.gov/hdsc/pfds/pfds_map_cont.html?bkmrk=nm
   a. Click on New Mexico on the Map
   b. Data Description – use defaults
   c. Get Location Options
      i. Use navigation tools to either:
         1. Enter latitude and longitude or
         2. Select Station or
         3. Selection Location on map
   d. Data Description
      i. Data Type: Select "***precipitation intensity***"
      ii. Units: Select "***english***"
      iii. Time series type: Select "***partial duration***"
   e. Scroll down to Depth/Duration/Frequency table below map
   f. Scroll to bottom of table and in the "Estimates from the table in csv format" box select "***precipitation frequency estimates***".
   g. Open in MS Excel and do a "save as" to your workspace as a .txt file
   h. Open .txt file (it should open in Excel)
   i. Insert Chart into the Excel spreadsheet (see **Table 403-1** example spreadsheet below)
      i. Insert a column adjacent to the durations and fill in with time values (*Excel doesn't recognize "5-min" as a value*)
      ii. Select X Y Scatter Chart Type
      iii. Select Data with duration (in minutes) on the x axis, intensity (in inches/hr) on the y axis for each frequency (1 year, 2 year, 5 year, 10 year, 25 year, 50 year, 100 year) as needed for project analyses.
   j. Format x axis to allow reading duration in 1 minute increments and y axis to read intensity in 0.1 in/hr increments. (See **Table 403-1**)
   k. Read rainfall intensity that matches basin Tc for the storm frequency required.
   l. DO NOT USE A RAINFALL INTENSITY FOR A Tc LESS THAN 10 MINUTES!

Table 403-1 NOAA Data Server Sample IDF Spreadsheet – Lemitar NM

Point precipitation frequency estimates (inches/hour)
NOAA Atlas 14 Volume 1 Version 5
Data type: Precipitation intensity
Time series type: Partial duration
Project area: Southwest
Location name: Lemitar, New Mexico, US*
Station Name: -
Latitude: 34.1580°
Longitude: -106.9181°
Elevation: 4712 ft*
* source: Google Maps

PRECIPITATION FREQUENCY ESTIMATES
by
duration

for ARI:		1	2	5	10	25	50	100	200	500	1000 years
5-min:	5	2.45	3.18	4.26	5.09	6.23	7.1	8.04	9	10.31	11.35
10-min:	10	1.87	2.42	3.24	3.88	4.74	5.41	6.11	6.85	7.84	8.64
15-min:	15	1.54	2	2.68	3.2	3.92	4.46	5.05	5.66	6.48	7.14
30-min:	30	1.04	1.34	1.8	2.16	2.64	3.01	3.4	3.81	4.36	4.81
60-min:	60	0.64	0.83	1.11	1.33	1.63	1.86	2.1	2.36	2.7	2.98
2-hr:		0.37	0.48	0.64	0.76	0.95	1.11	1.29	1.49	1.8	2.06
3-hr:		0.27	0.34	0.45	0.54	0.67	0.78	0.9	1.04	1.25	1.43
6-hr:		0.16	0.2	0.25	0.3	0.36	0.42	0.48	0.55	0.66	0.75
12-hr:		0.08	0.11	0.13	0.16	0.19	0.22	0.25	0.29	0.34	0.38
24-hr:		0.05	0.06	0.08	0.09	0.11	0.12	0.14	0.16	0.18	0.2
2-day:		0.03	0.03	0.04	0.05	0.06	0.07	0.07	0.08	0.1	0.11
3-day:		0.02	0.02	0.03	0.03	0.04	0.05	0.05	0.06	0.07	0.08
4-day:		0.02	0.02	0.02	0.03	0.03	0.04	0.04	0.05	0.05	0.06
7-day:		0.01	0.01	0.02	0.02	0.02	0.02	0.03	0.03	0.03	0.04
10-day:		0.01	0.01	0.01	0.01	0.02	0.02	0.02	0.02	0.03	0.03
20-day:		0	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.02	0.02
30-day:		0	0	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01
45-day:		0	0	0	0.01	0.01	0.01	0.01	0.01	0.01	0.01
60-day:		0	0	0	0	0.01	0.01	0.01	0.01	0.01	0.01

Date/time (GMT): Fri Nov 13 22:14:03 2015
pyRunTime: 0.127875804901

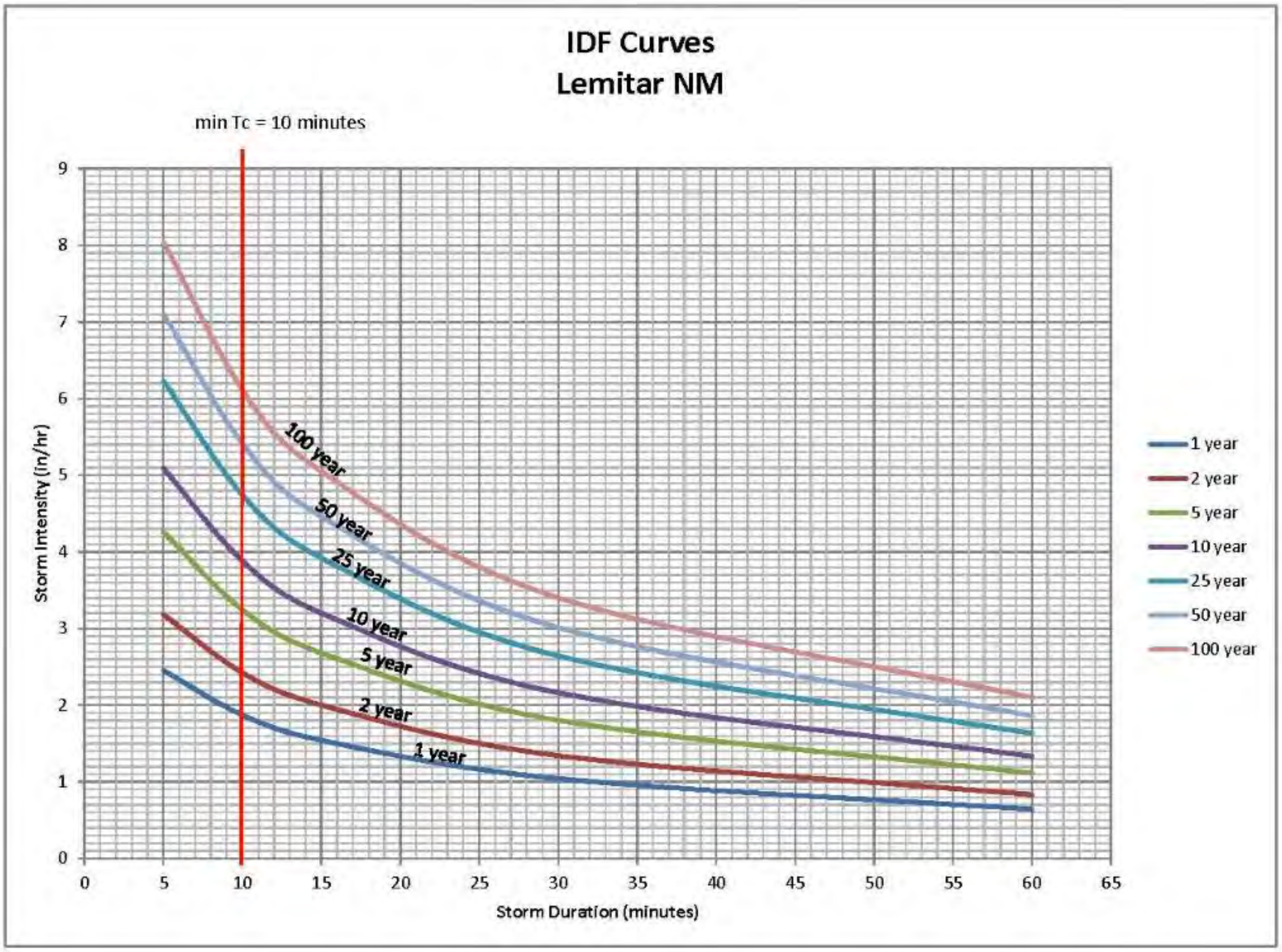


Figure 403-1 IDF Curves from NOAA Data Server – Lemitar, NM

To produce the **Depth** Duration 1- Hour Precipitation values for use in determining the Rational 'C' Factor, return to the NOAA Data Server for the same location as for the IDF Curve development. http://hdsc.nws.noaa.gov/hdsc/pfds/pfds_map_cont.html?bkmrk=nm

a. Data Description
    i. Data Type: Select "***precipitation depth***"
    ii. Units: Select "***english***"
    iii. Time series type: Select "***partial duration***"
b. Scroll down to Depth/Duration/Frequency table below map
c. Scroll to bottom of table and in the "Estimates from the table in csv format" box select "***precipitation frequency estimates***".
d. Open in MS Excel and do a "save as" to your workspace as a .txt file
e. Open .txt file (it should open in Excel)
f. Read point rainfall value for 1- hour design storm

Table 403-2 Depth Duration Frequency Table from NOAA Data Server

Point precipitation frequency estimates (inches)
NOAA Atlas 14 Volume 1 Version 5
Data type: Precipitation depth
Time series type: Partial duration
Project area: Southwest
Location name: Lemitar, New Mexico, US*
Station Name: -
Latitude: 34.1584°
Longitude: -106.9189°
Elevation: 4713 ft*
* source: Google Maps

PRECIPITATION FREQUENCY ESTIMATES

by duratior	1	2	5	10	25	50	100	200	500	1000 years
5-min:	0.2	0.27	0.35	0.42	0.52	0.59	0.67	0.75	0.86	0.95
10-min:	0.31	0.4	0.54	0.65	0.79	0.9	1.02	1.14	1.31	1.44
15-min:	0.39	0.5	0.67	0.8	0.98	1.12	1.26	1.41	1.62	1.78
30-min:	0.52	0.67	0.9	1.08	1.32	1.5	1.7	1.9	2.18	2.4
60-min:	0.64	0.83	1.11	1.33	1.63	1.86	2.1	2.36	2.7	2.98
2-hr:	0.75	0.96	1.27	1.52	1.9	2.22	2.58	2.98	3.59	4.13
3-hr:	0.81	1.03	1.35	1.61	2	2.33	2.7	3.12	3.75	4.3
6-hr:	0.93	1.18	1.51	1.78	2.18	2.52	2.9	3.31	3.93	4.48
12-hr:	1.01	1.28	1.63	1.91	2.31	2.65	3.03	3.44	4.06	4.59
24-hr:	1.16	1.45	1.82	2.12	2.55	2.9	3.29	3.72	4.35	4.88
2-day:	1.27	1.59	1.98	2.3	2.76	3.13	3.54	3.99	4.64	5.2
3-day:	1.36	1.7	2.12	2.46	2.94	3.34	3.78	4.25	4.95	5.56
4-day:	1.45	1.81	2.25	2.61	3.12	3.55	4.01	4.51	5.25	5.89
7-day:	1.67	2.08	2.57	2.96	3.52	3.97	4.46	4.99	5.77	6.41
10-day:	1.84	2.3	2.84	3.29	3.91	4.41	4.96	5.56	6.43	7.17
20-day:	2.33	2.9	3.54	4.03	4.71	5.25	5.81	6.39	7.2	7.89
30-day:	2.81	3.5	4.23	4.78	5.53	6.11	6.7	7.3	8.12	8.81
45-day:	3.41	4.23	5.08	5.7	6.51	7.12	7.71	8.29	9.11	9.78
60-day:	3.9	4.84	5.8	6.52	7.44	8.13	8.81	9.47	10.33	10.98

Date/time (GMT): Mon Nov 16 19:12:46 2015
pyRunTime: 0.126244068146

403.3 Rational Formula 'C' Factor

The runoff coefficient, C, is selected from **Figure 403-2 through Figure 403-7**, depending on the ground cover, hydrologic soil group, type of development, and 1-hour rainfall depth for the design return period. Hydrologic soil groups are defined in **Section 403** above. **Figure 403-2 through Figure 403-7** show how C varies with 1-hour rainfall depth. This is because C is a function of infiltration and other hydrologic abstractions, relating the peak discharge to the theoretical peak discharge produced by 100% runoff.

When land use or other factors vary significantly throughout the watershed, an area weighted C value should be used. The weighted C value is computed by the equation:

$$Weighted\ C = \frac{C1xA1 + C2xA2 + C3xA3 \ldots}{\sum A} \qquad \textbf{403-4}$$

Where:

C1 = C Factor for subbasin 1, etc.

A1 = area in acres of subbasin 1, etc.

∑A = total area of watershed in acres

The designer should select the appropriate **Figure 403-2** through **Figure 403-7**, depending on the watershed location (desert, upland range, mountain or urban) and the predominant vegetation type (cactus, brush, grasses, juniper, pine). Enter each Figure with the design 1-hour rainfall depth. Move vertically up through the Figure until the appropriate curve is found, then move horizontally to find the design C value. The appropriate curve is selected based on the Hydrologic Soil Group (HSG) and the percent ground cover of the vegetation or percent imperviousness. When a value falls between two curves, interpolate linearly between the two nearest ones to the required percentage of cover or imperviousness.

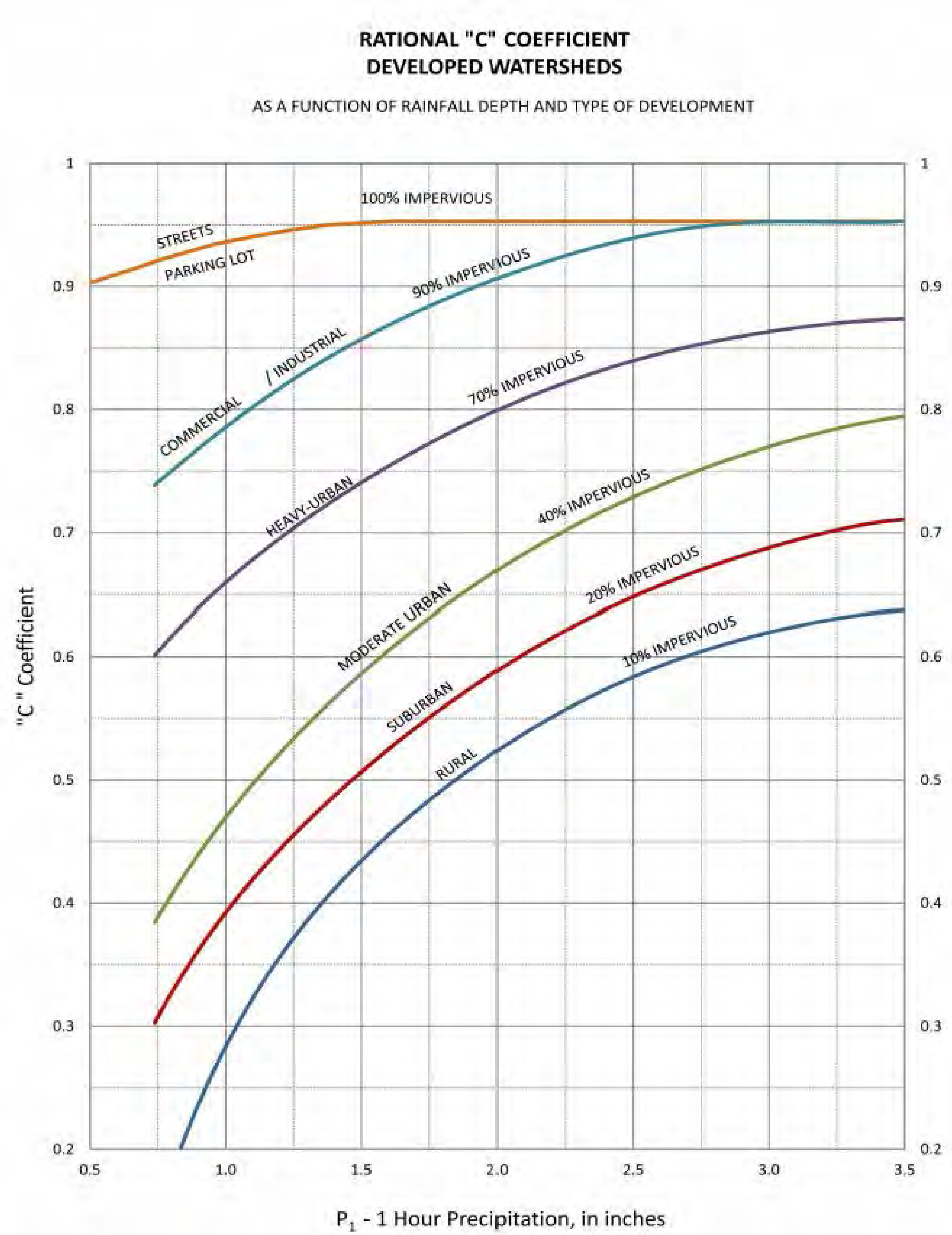


Figure 403-2 Rational 'C' Coefficient Developed Watersheds
Adapted from Arizona DOT Highway Drainage Design Manual
Volume 2 Hydrology Second Edition, 2014

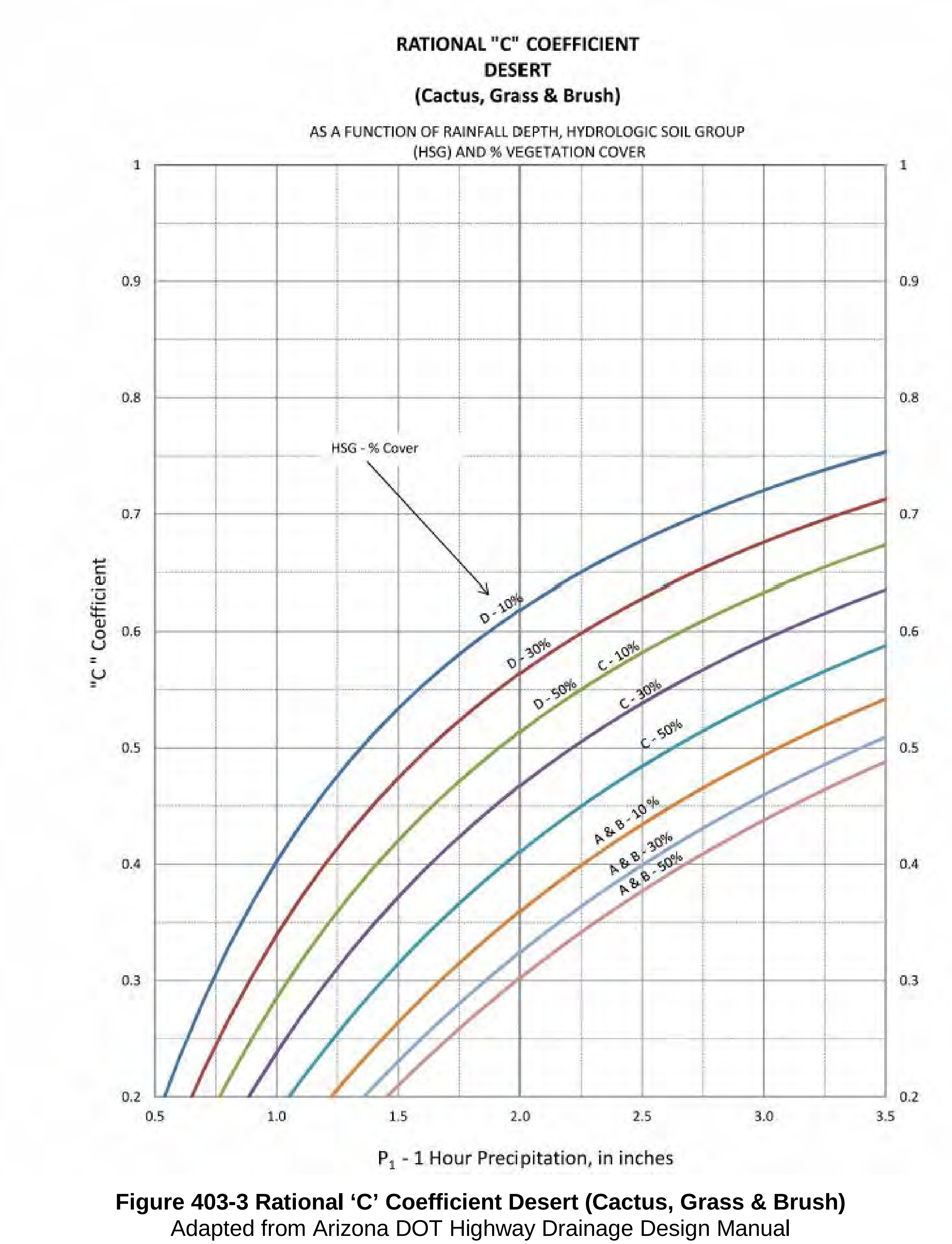


Figure 403-3 Rational 'C' Coefficient Desert (Cactus, Grass & Brush)
Adapted from Arizona DOT Highway Drainage Design Manual
Volume 2 Hydrology Second Edition, 2014

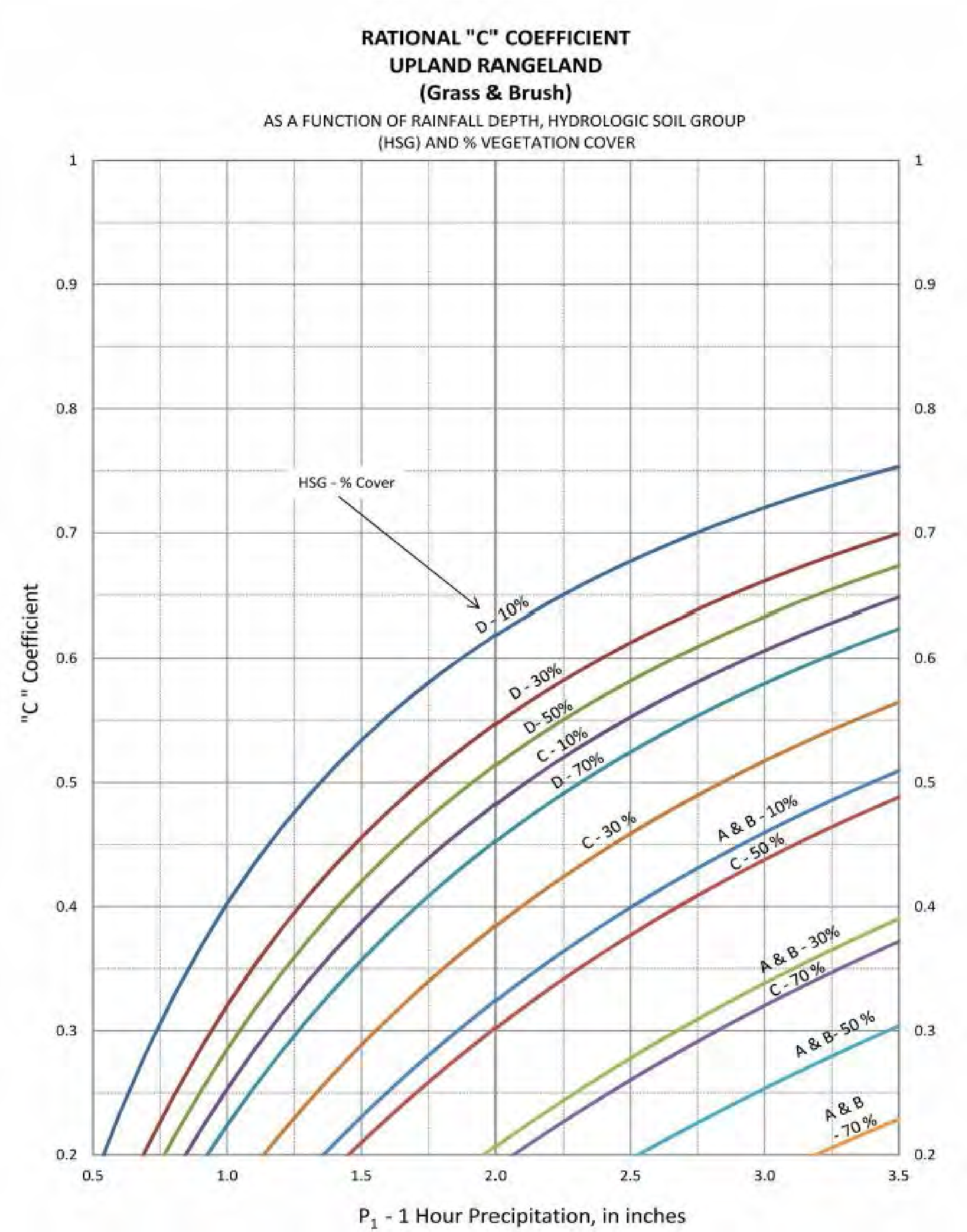


Figure 403-4 Rational ‘C’ Coefficient Upland Rangeland (Grass & Brush)
Adapted from Arizona DOT Highway Drainage Design Manual
Volume 2 Hydrology Second Edition, 2014

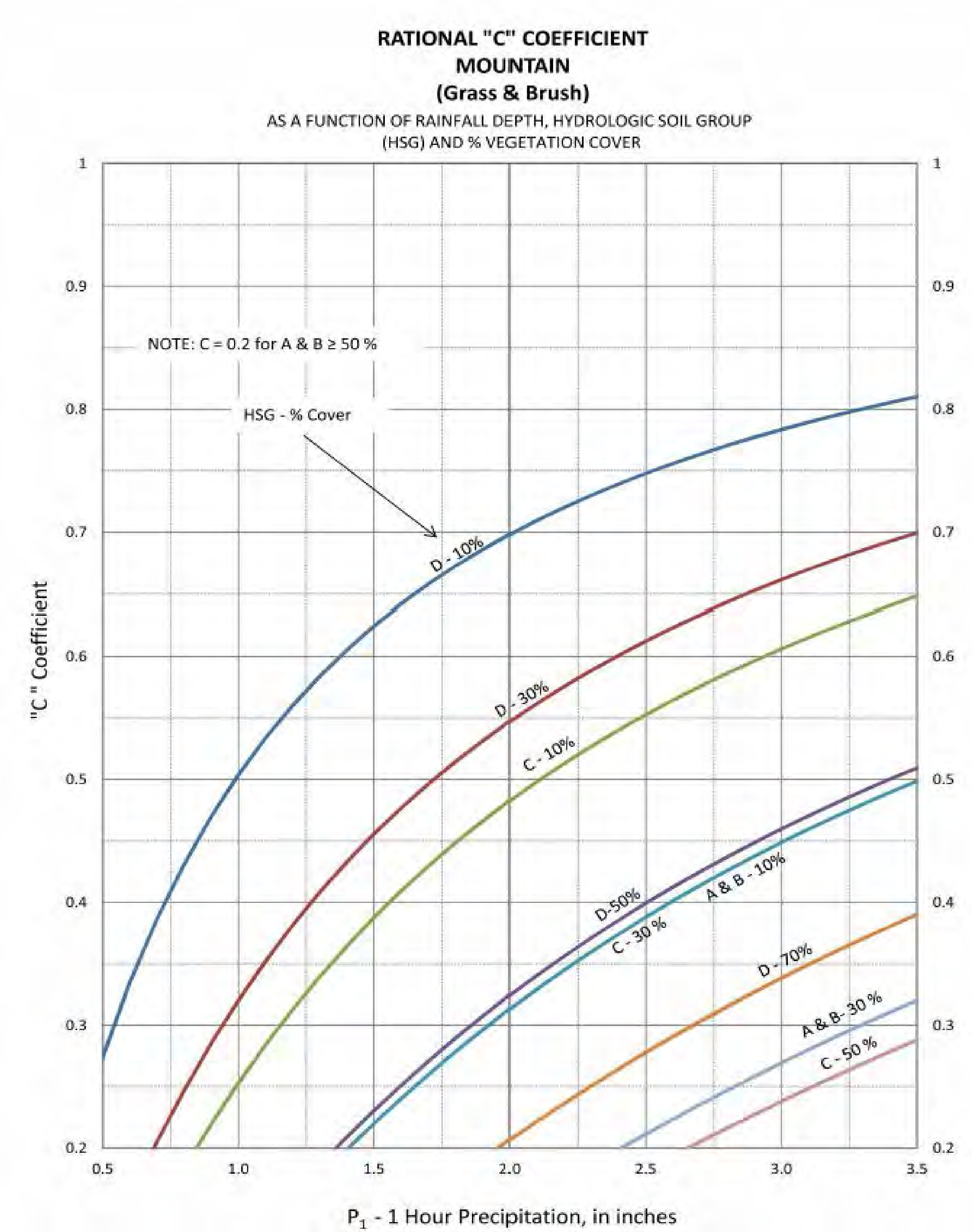


Figure 403-5 Rational 'C' Coefficient Mountain (Grass and Brush)
Adapted from Arizona DOT Highway Drainage Design Manual
Volume 2 Hydrology Second Edition, 2014

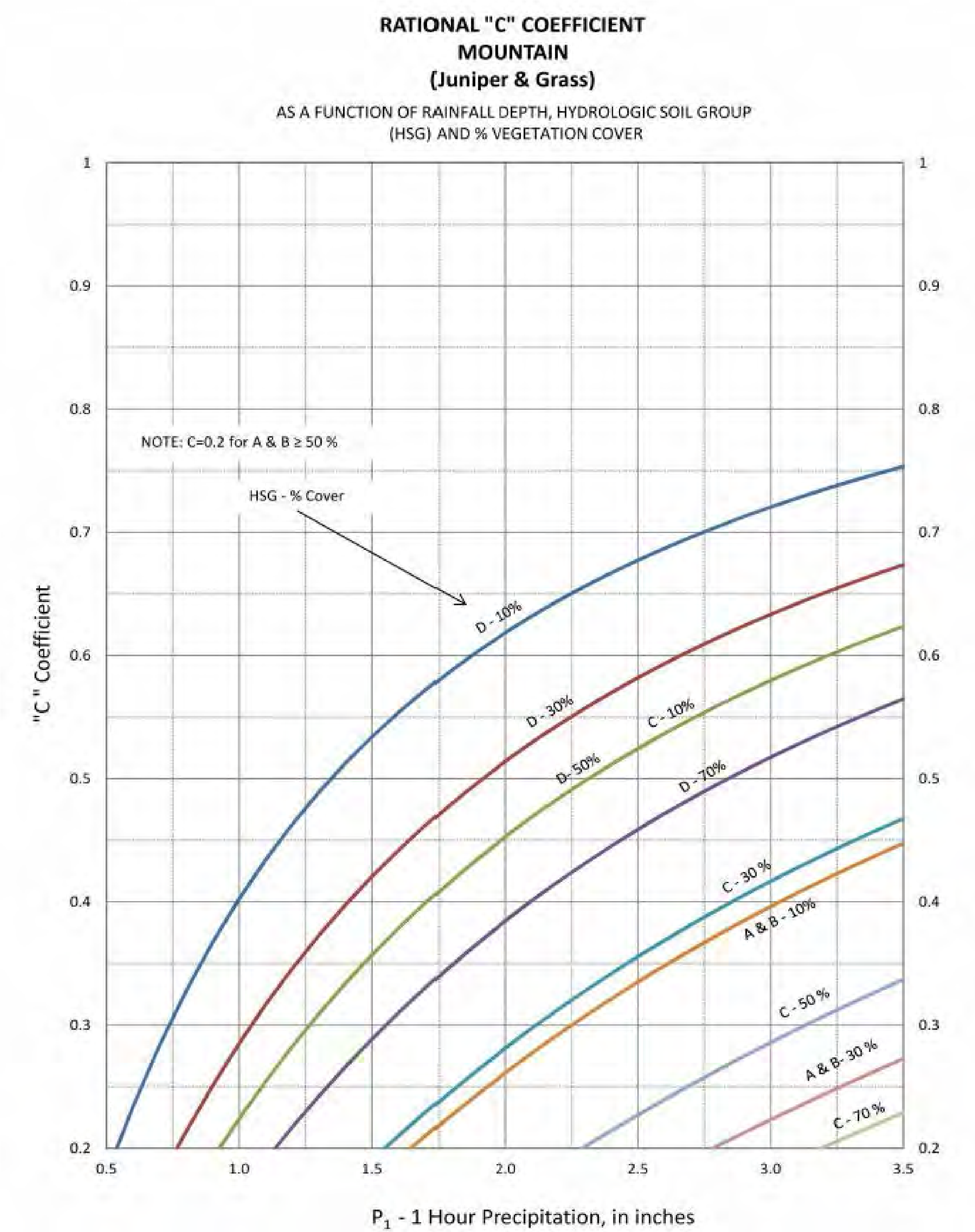


Figure 403-6 Rational 'C' Coefficient Mountain (Pinion, Juniper & Grass)
Adapted from Arizona DOT Highway Drainage Design Manual
Volume 2 Hydrology Second Edition, 2014

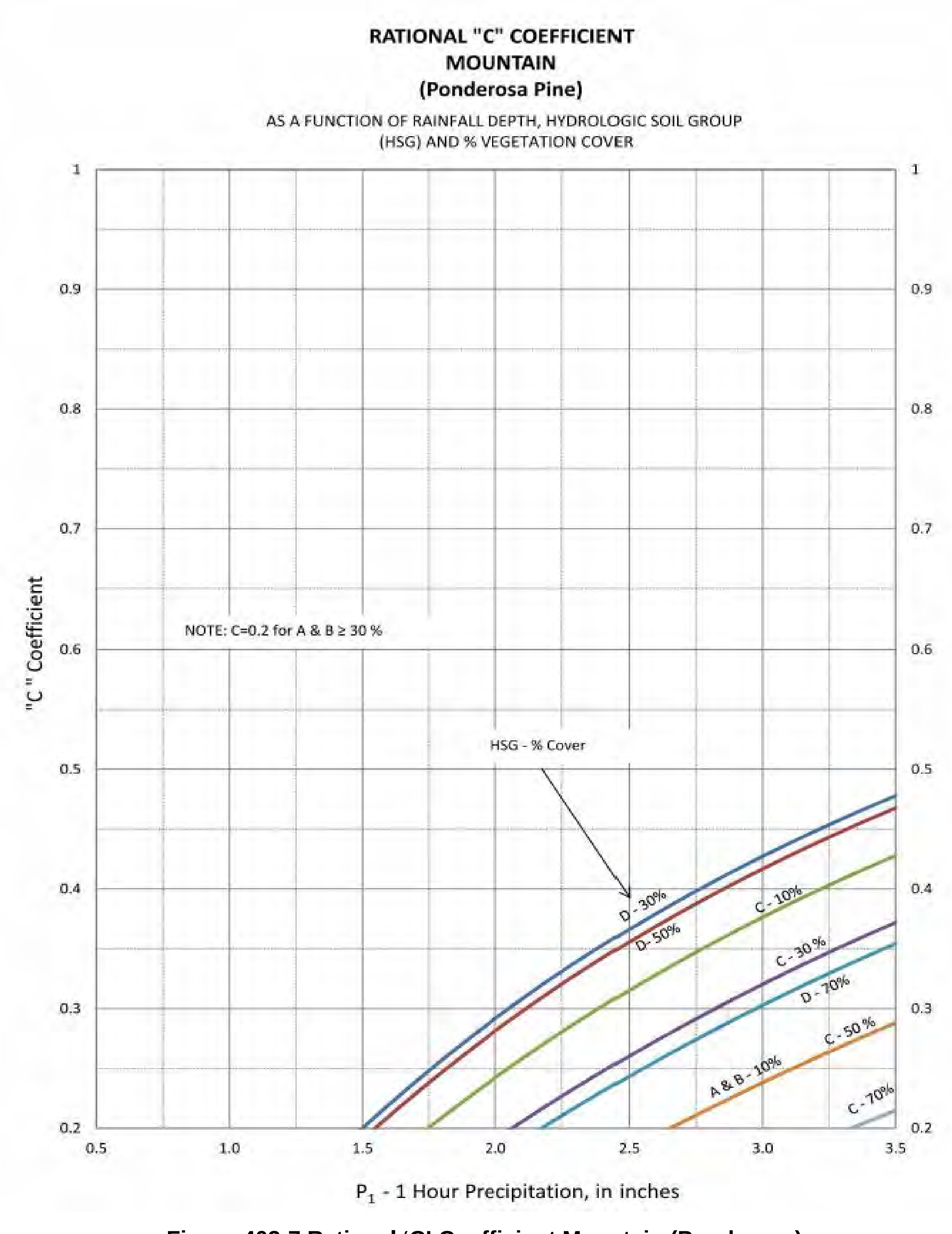


Figure 403-7 Rational 'C' Coefficient Mountain (Ponderosa)
Adapted from Arizona DOT Highway Drainage Design Manual
Volume 2 Hydrology Second Edition, 2014

403.4 Rational Formula Example Problems

Example problems are found in Appendix 7

403.5 References

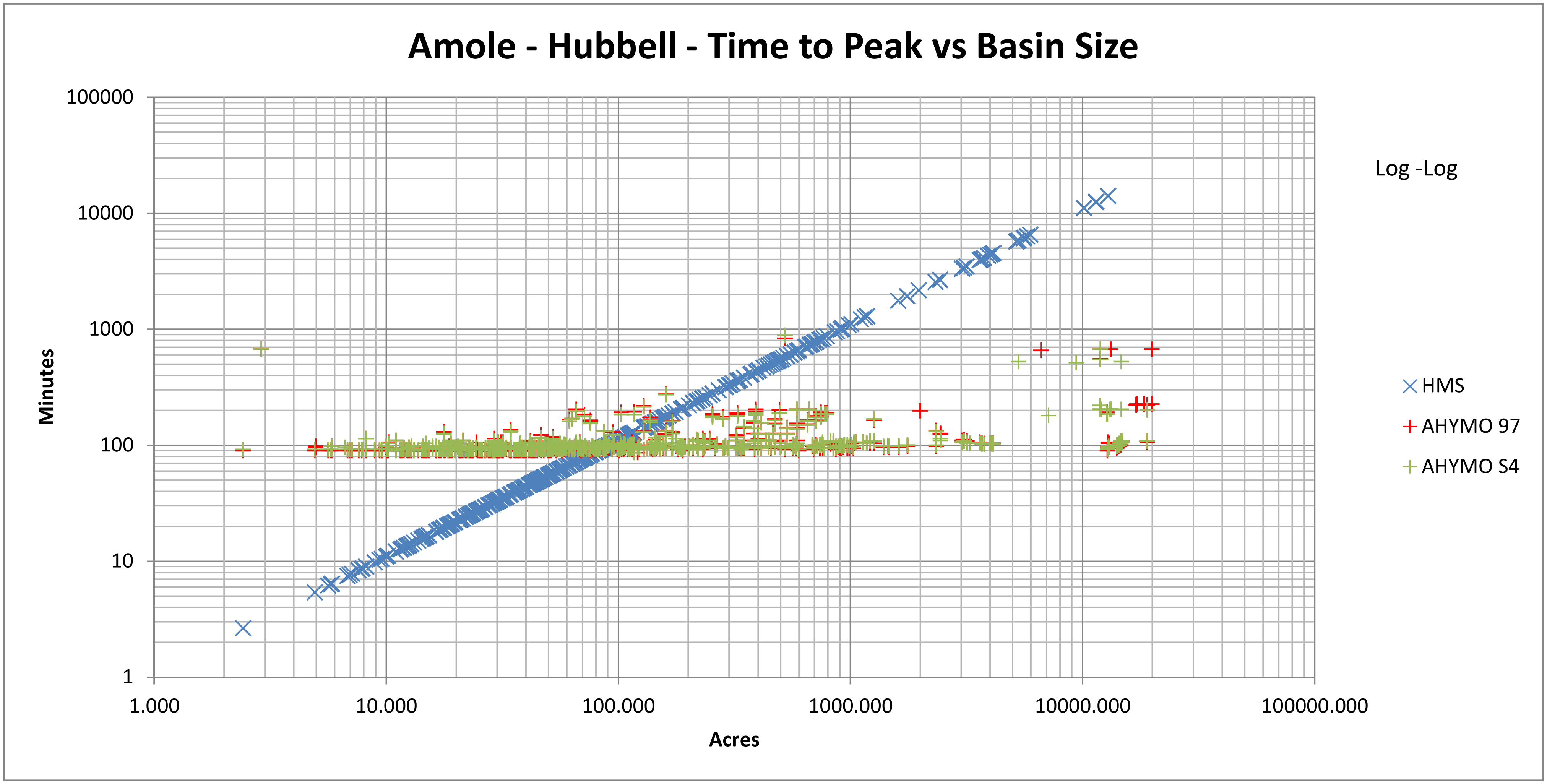
NRCS, 2007, Part 630 Hydrology, National Engineering Handbook Chapter 7 Hydrologic Soils Groups.
http://directives.sc.egov.usda.gov/OpenNonWebContent.aspx?content=17757.wba

NRCS, 2010, Part 630 Hydrology, National Engineering Handbook Chapter 15 Time of Concentration.
http://directives.sc.egov.usda.gov/OpenNonWebContent.aspx?content=27002.wba

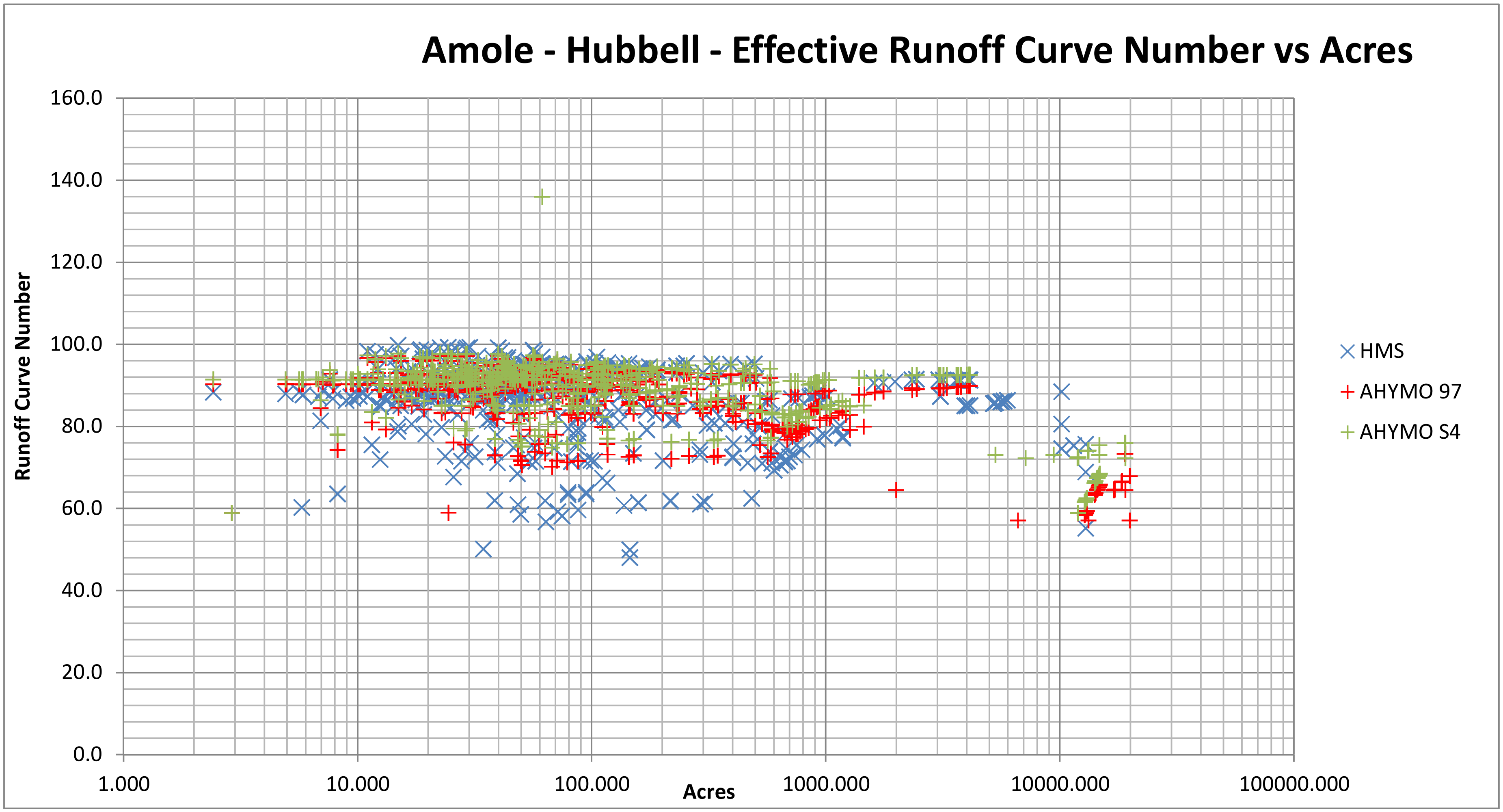
Arizona Department of Transportation Highway, 2014, Drainage Design Manual Hydrology Second Edition.
http://azdot.gov/docs/business/highway-drainage-design-manual-hydrology

APPENDIX H

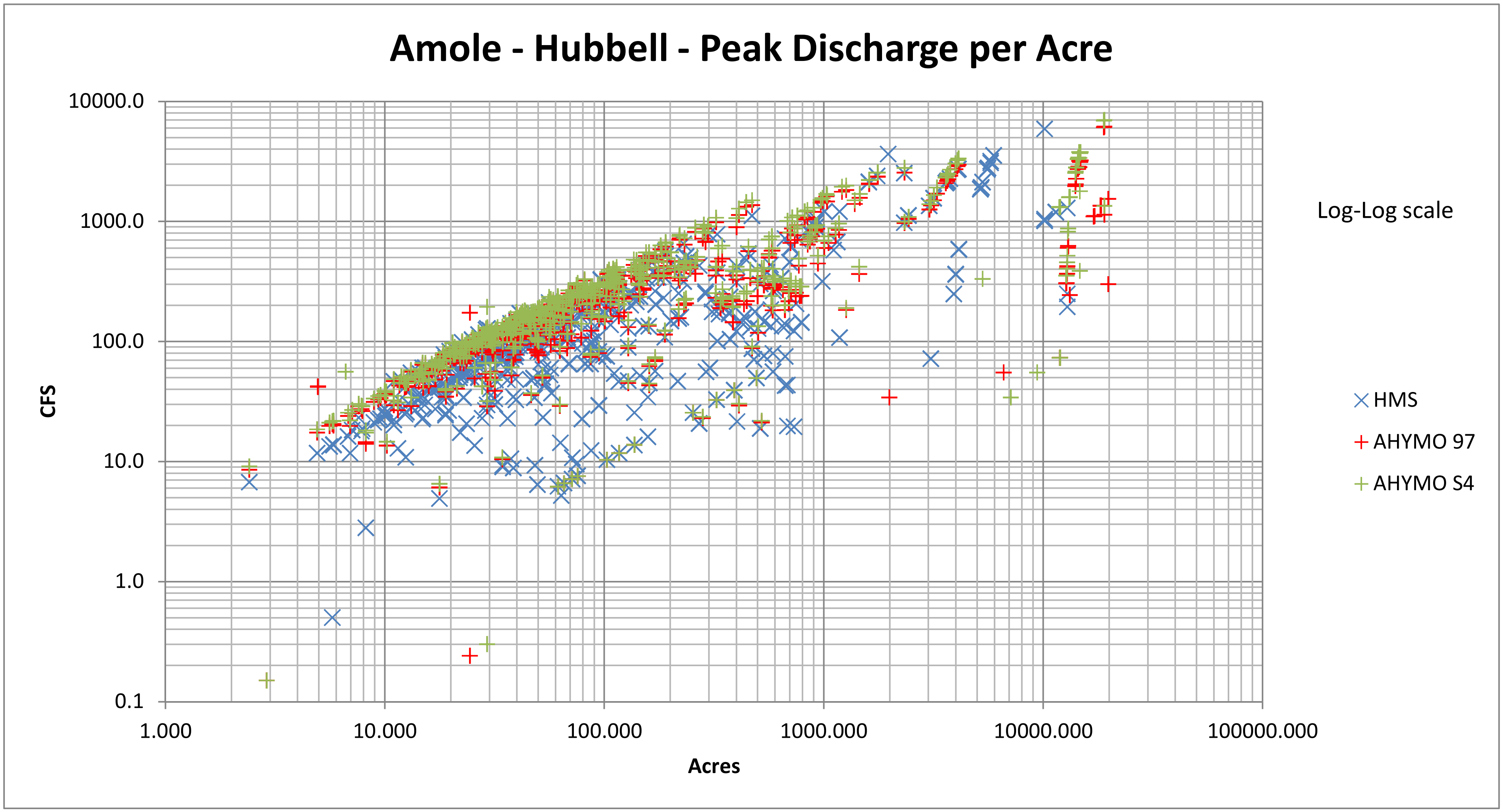
MODEL COMPARISONS



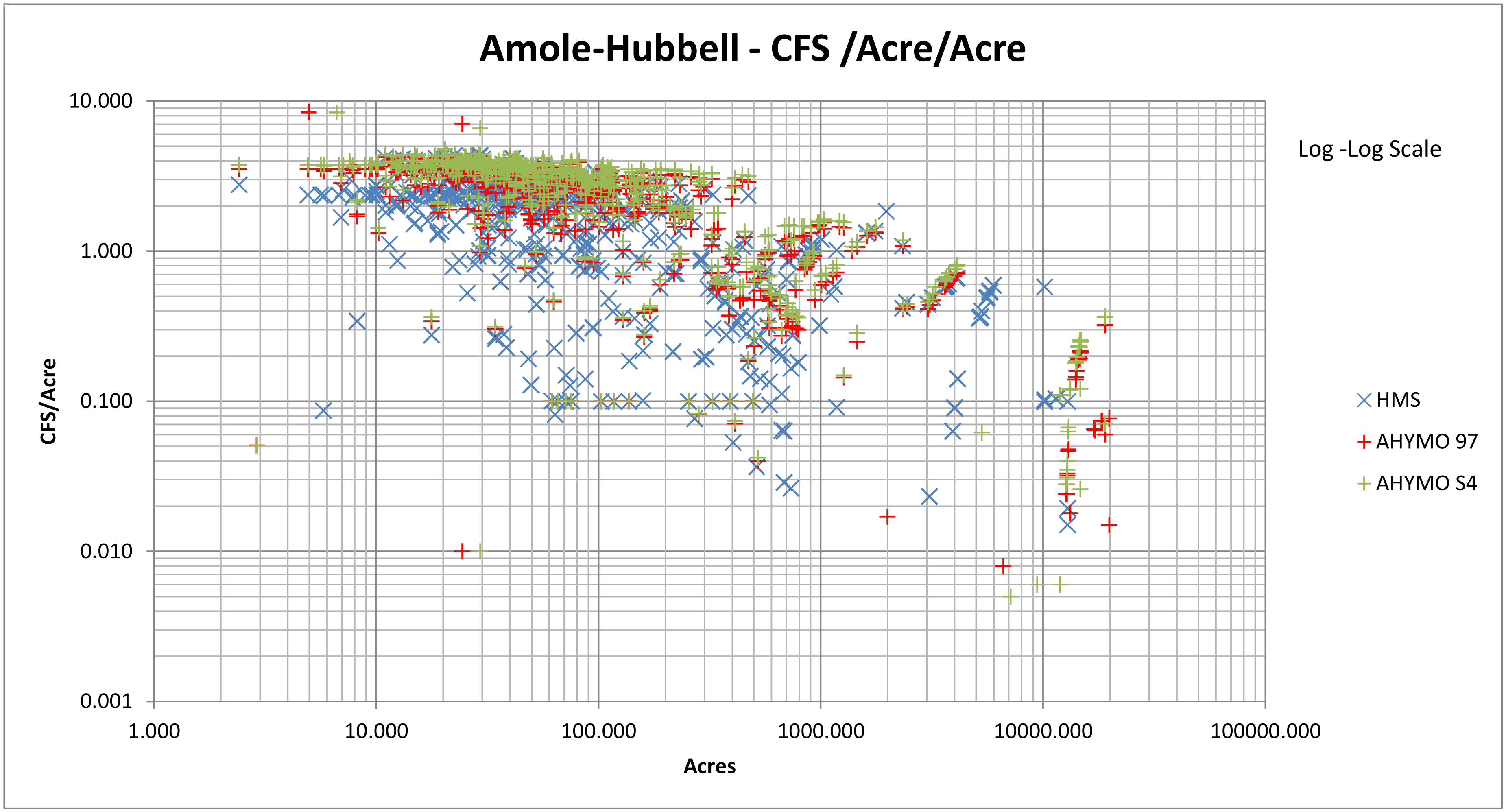
Amole - Hubbell - Time to Peak vs Basin Size
Log -Log
Minutes
100000
10000
1000
100
10
1
1.000
10.000
100.000
1000.000
10000.000
100000.000
Acres
HMS
AHYMO 97
AHYMO S4



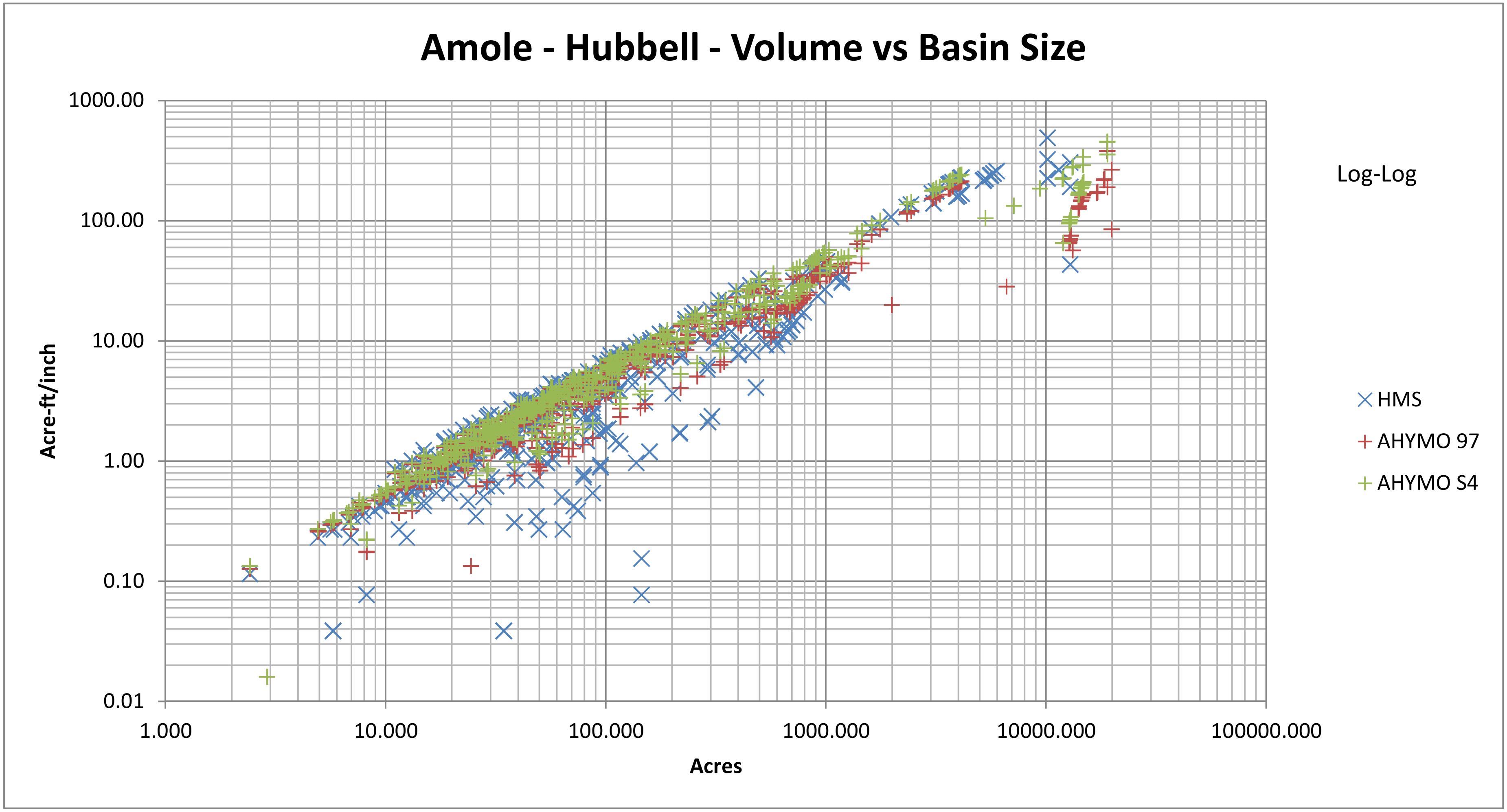
Amole - Hubbell - Effective Runoff Curve Number vs Acres
Runoff Curve Number
160.0
140.0
120.0
100.0
80.0
60.0
40.0
20.0
0.0
1.000
10.000
100.000
1000.000
10000.000
100000.000
Acres
HMS
AHYMO 97
AHYMO S4



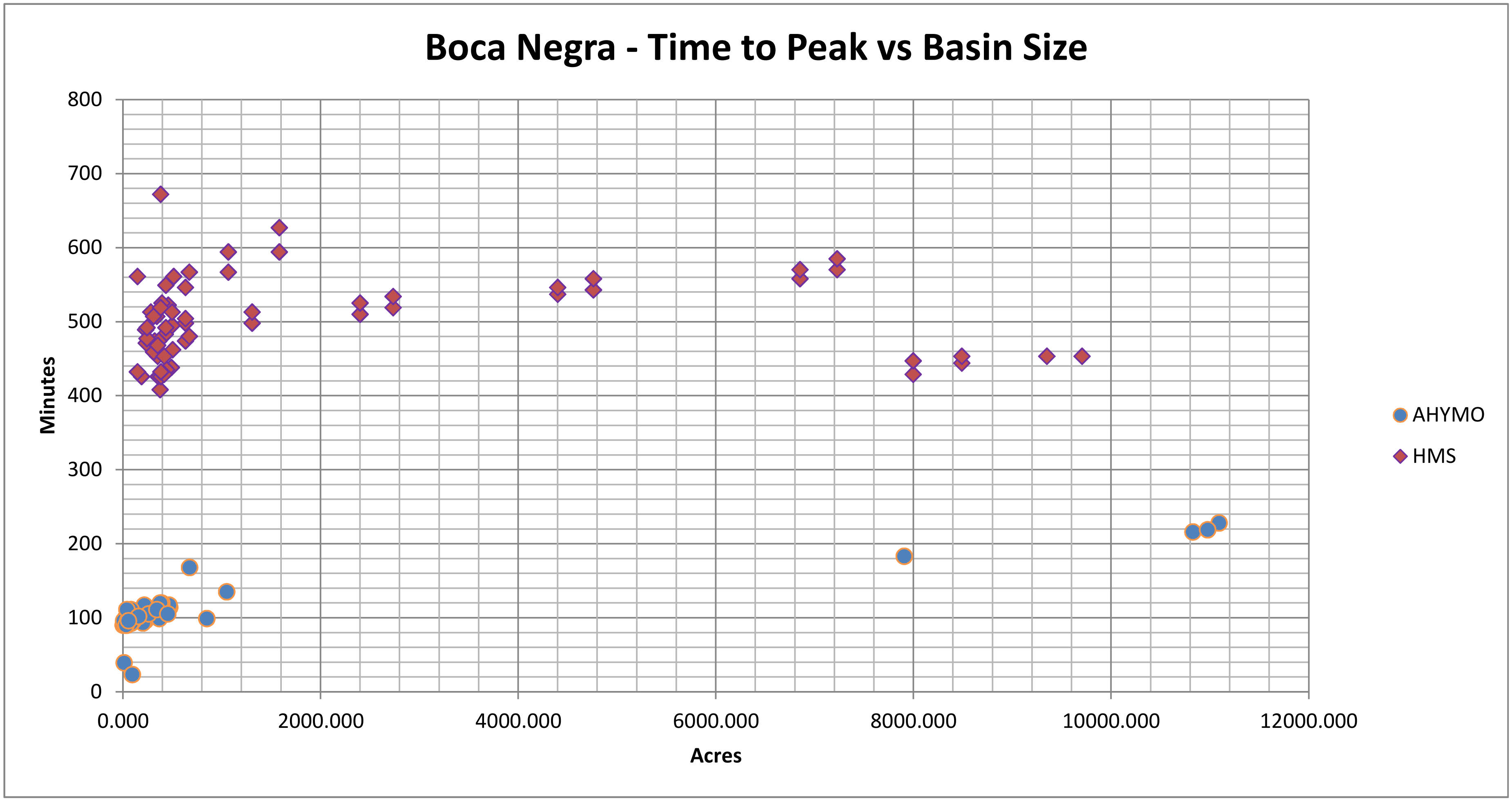
Amole - Hubbell - Peak Discharge per Acre
10000.0
1000.0
100.0
10.0
1.0
0.1
CFS
1.000
10.000
100.000
1000.000
10000.000
100000.000
Acres
Log-Log scale
HMS
AHYMO 97
AHYMO S4



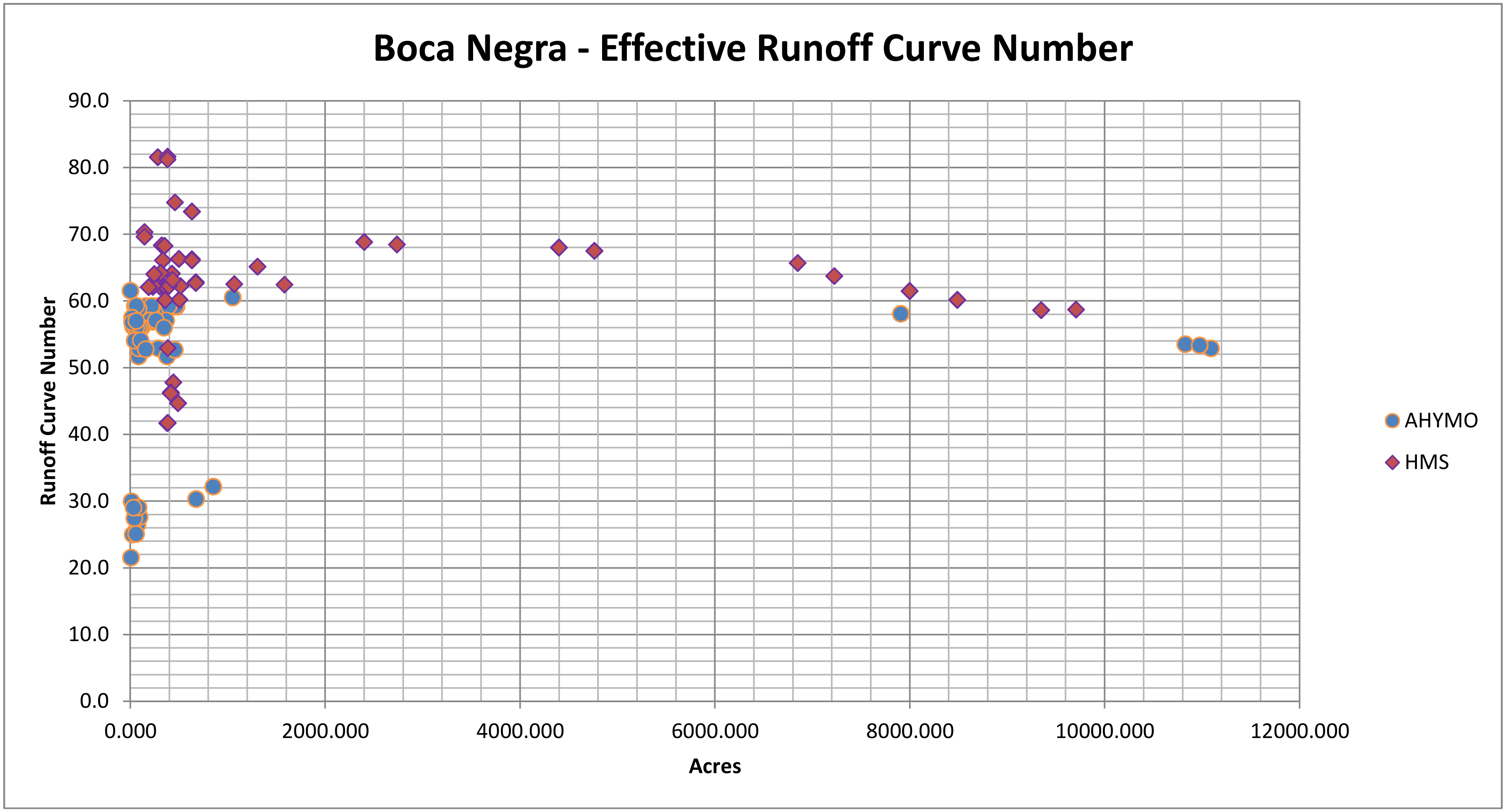
Amole-Hubbell - CFS /Acre/Acre
Log -Log Scale
CFS/Acre
10.000
1.000
0.100
0.010
0.001
1.000
10.000
100.000
1000.000
10000.000
100000.000
Acres
HMS
AHYMO 97
AHYMO S4



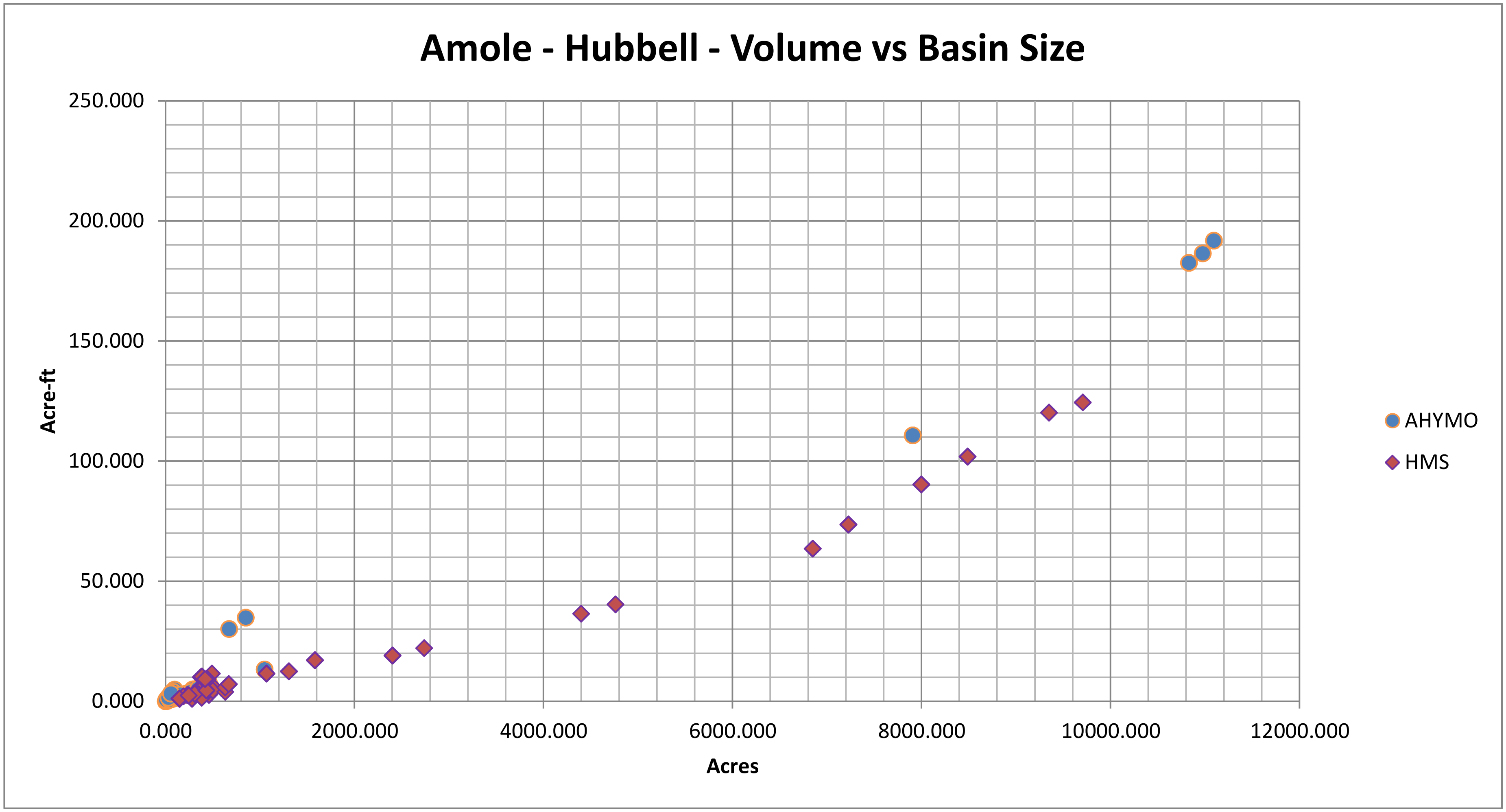
Amole - Hubbell - Volume vs Basin Size
1000.00
100.00
10.00
1.00
0.10
0.01
Acre-ft/inch
1.000
10.000
100.000
1000.000
10000.000
100000.000
Acres
Log-Log
HMS
AHYMO 97
AHYMO S4



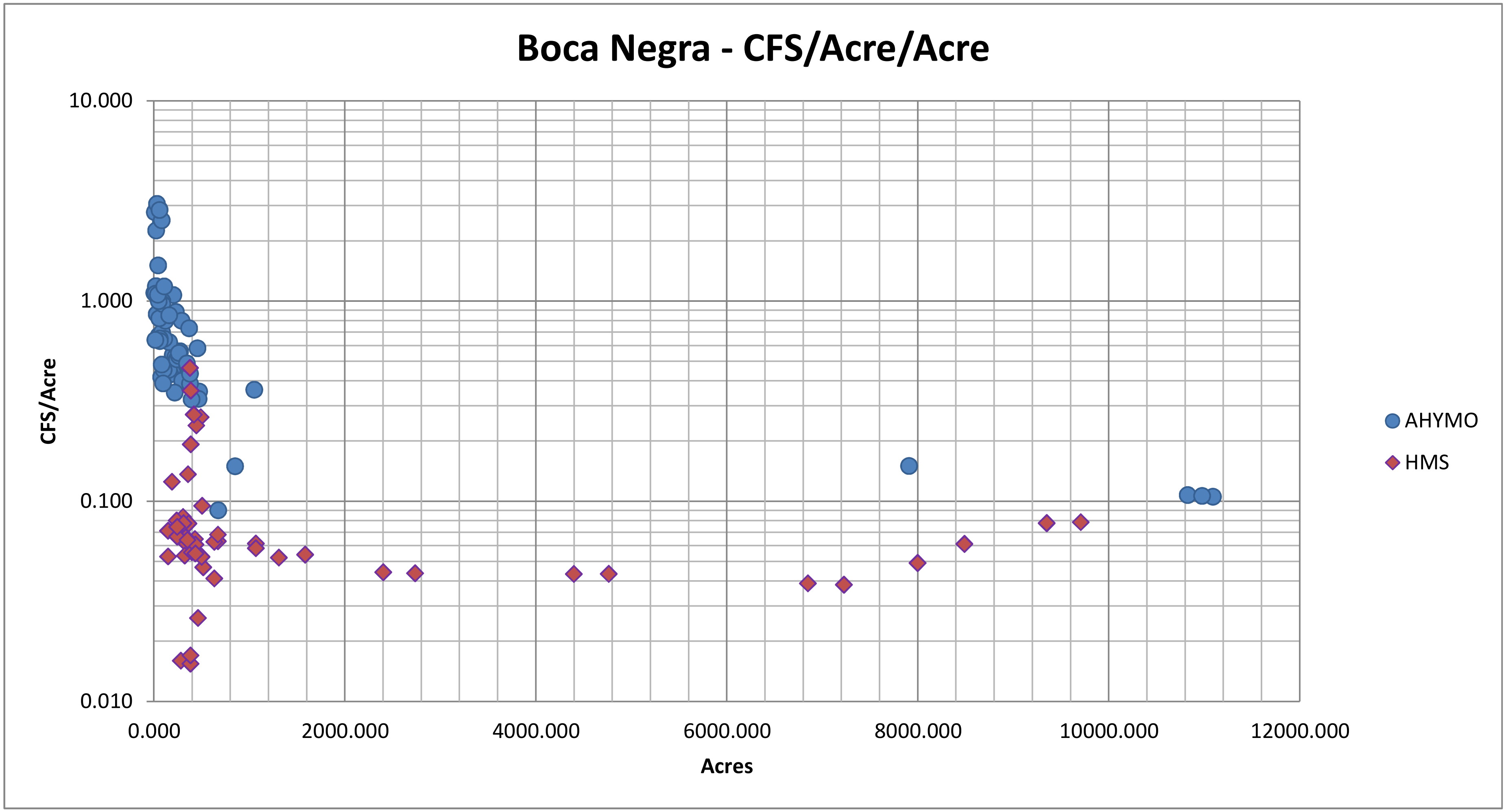
Boca Negra - Time to Peak vs Basin Size
Minutes
800
700
600
500
400
300
200
100
0
0.000
2000.000
4000.000
6000.000
8000.000
10000.000
12000.000
Acres
AHYMO
HMS

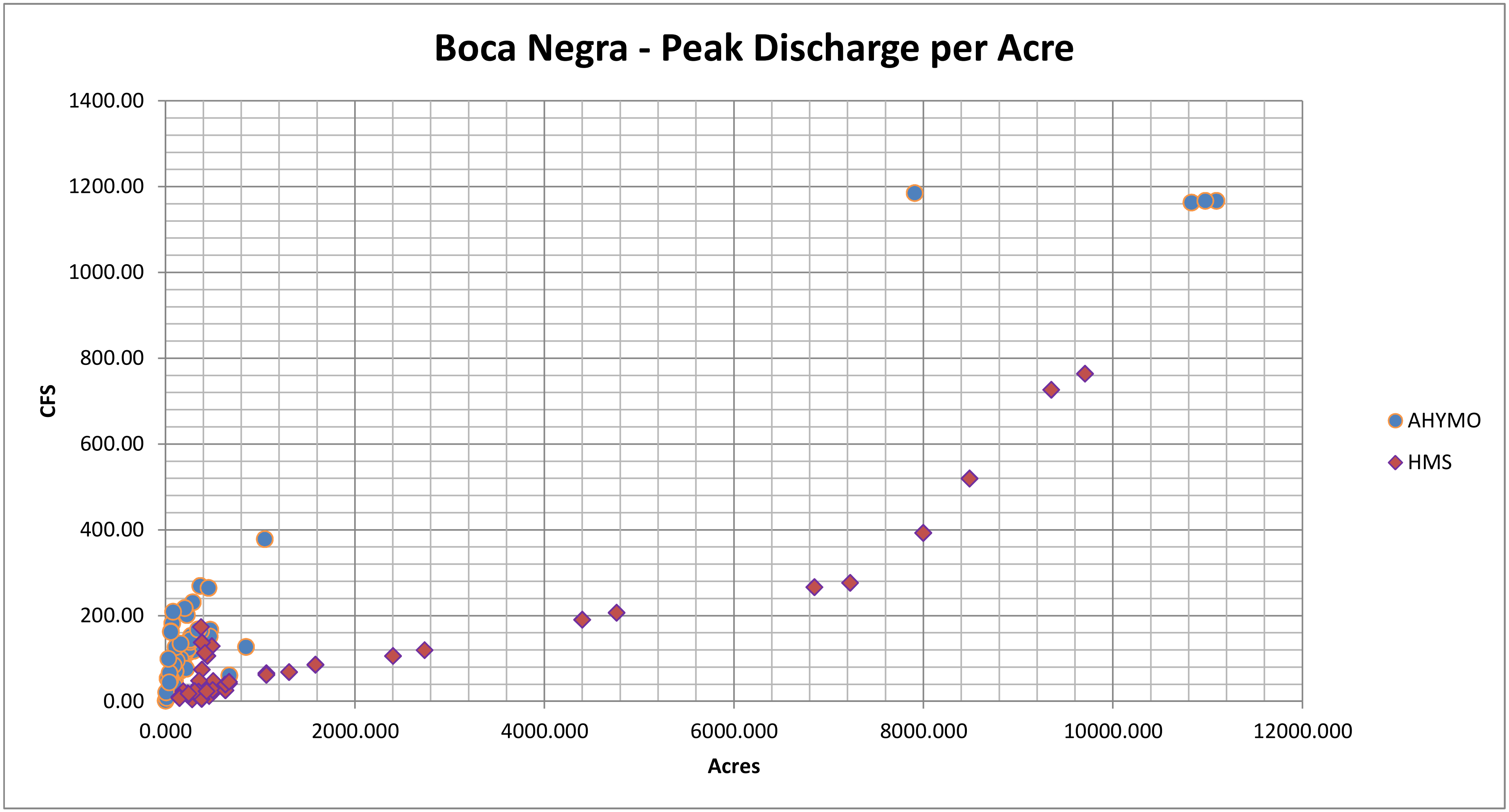


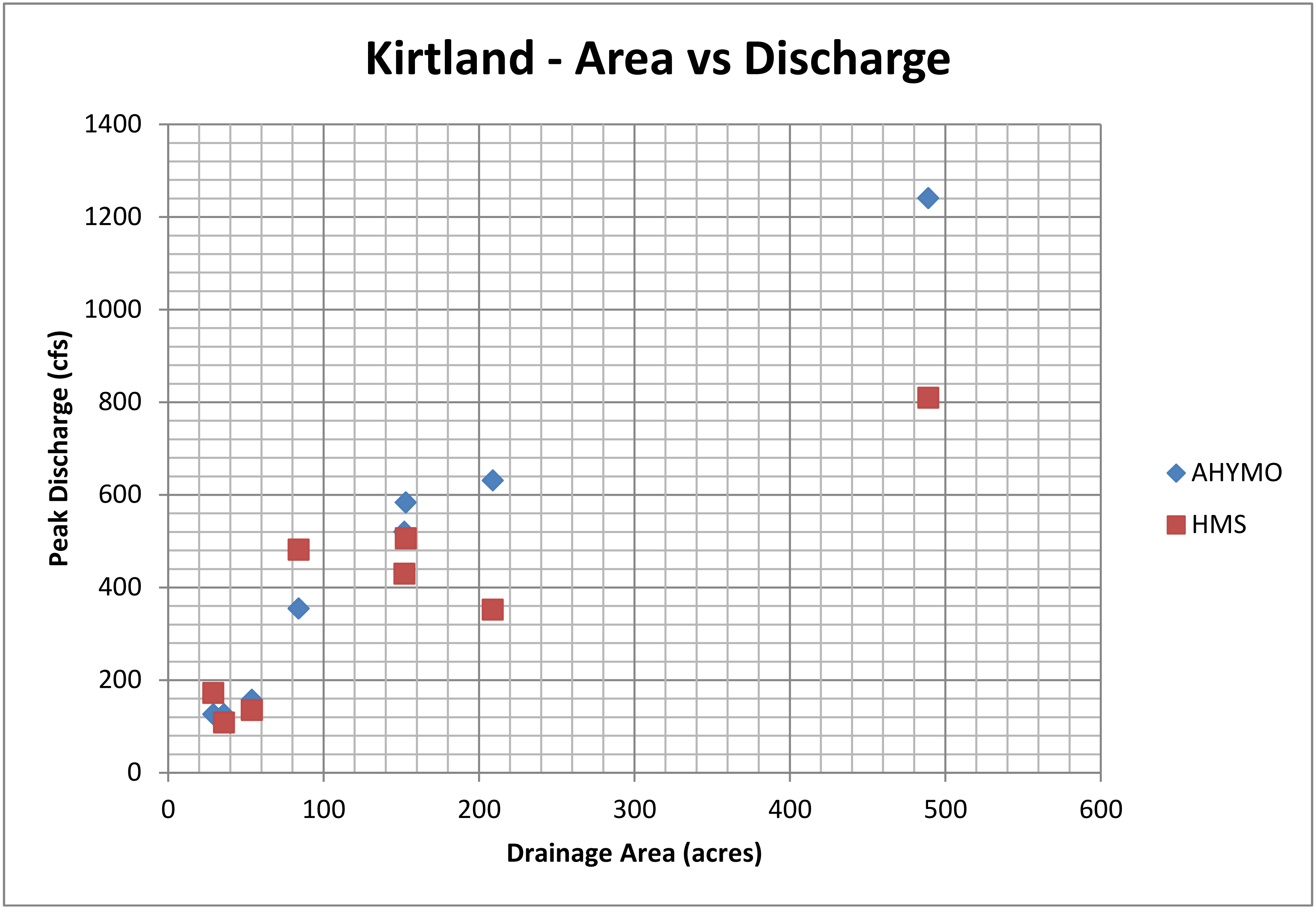
Boca Negra - Effective Runoff Curve Number
Runoff Curve Number
Acres
90.0
80.0
70.0
60.0
50.0
40.0
30.0
20.0
10.0
0.0
0.000
2000.000
4000.000
6000.000
8000.000
10000.000
12000.000
AHYMO
HMS

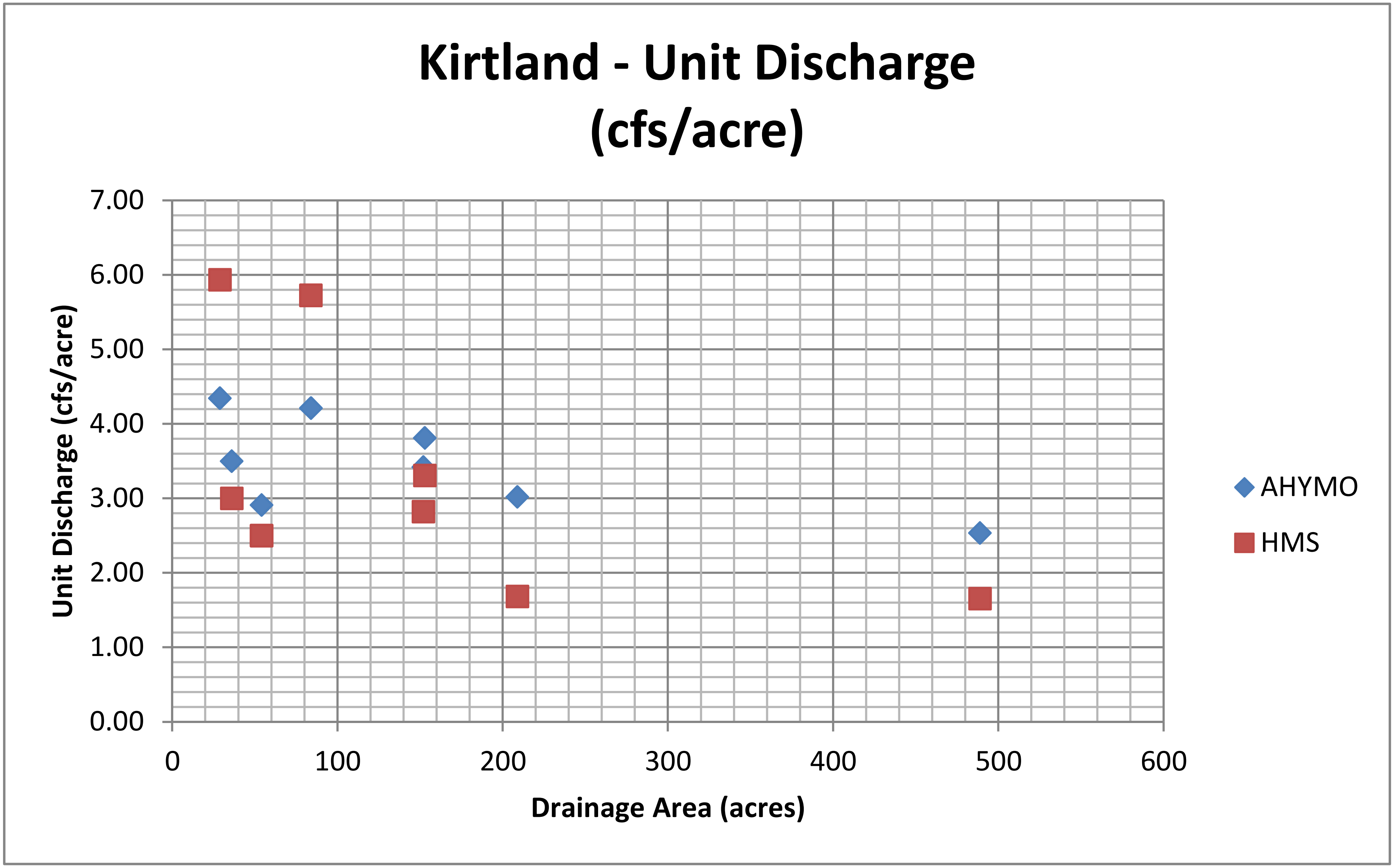


Amole - Hubbell - Volume vs Basin Size
250.000
200.000
150.000
100.000
50.000
0.000
Acre-ft
0.000
2000.000
4000.000
6000.000
8000.000
10000.000
12000.000
Acres
AHYMO
HMS

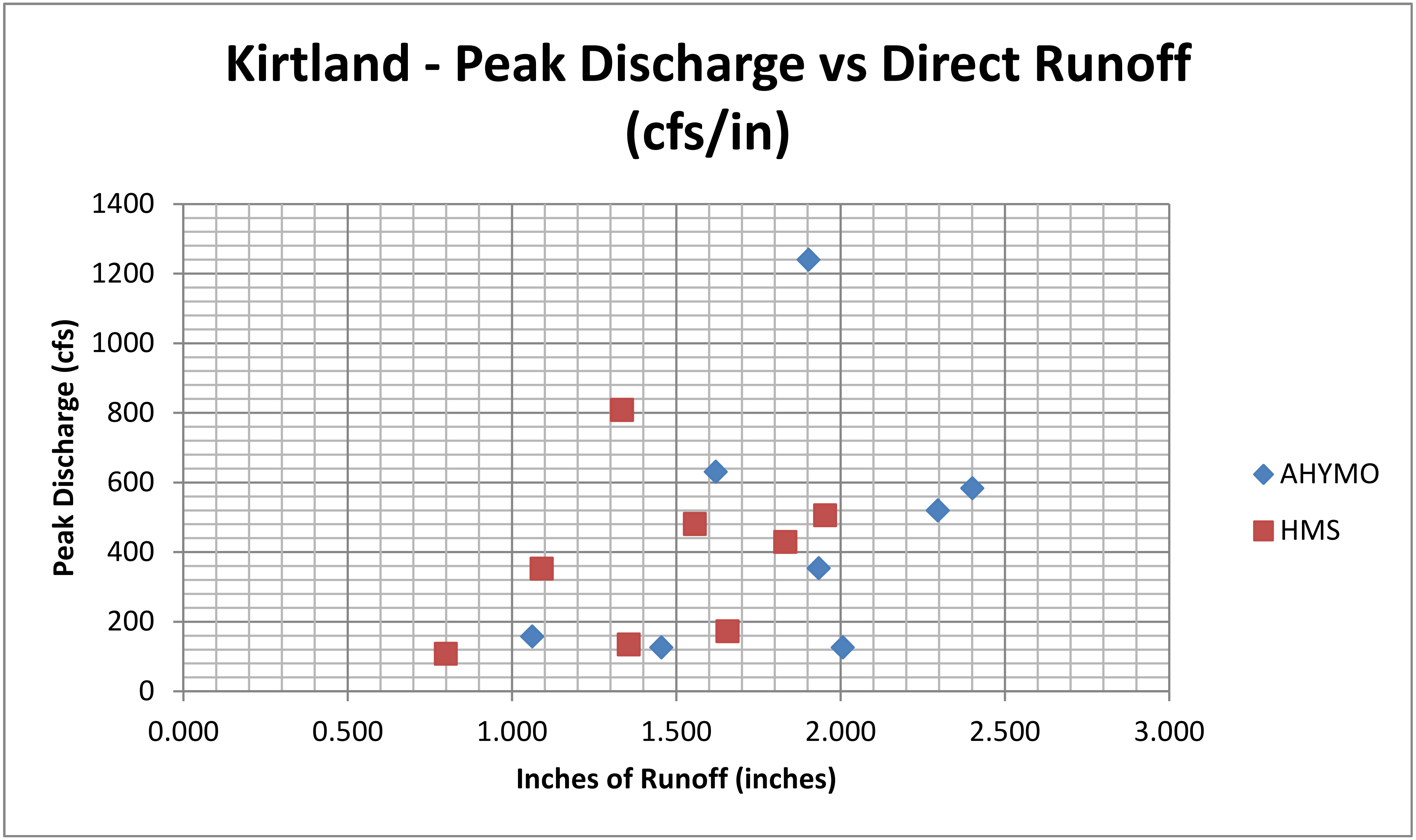

Boca Negra - CFS/Acre/Acre
10.000
1.000
0.100
0.010
CFS/Acre
0.000
2000.000
4000.000
6000.000
8000.000
10000.000
12000.000
Acres
AHYMO
HMS

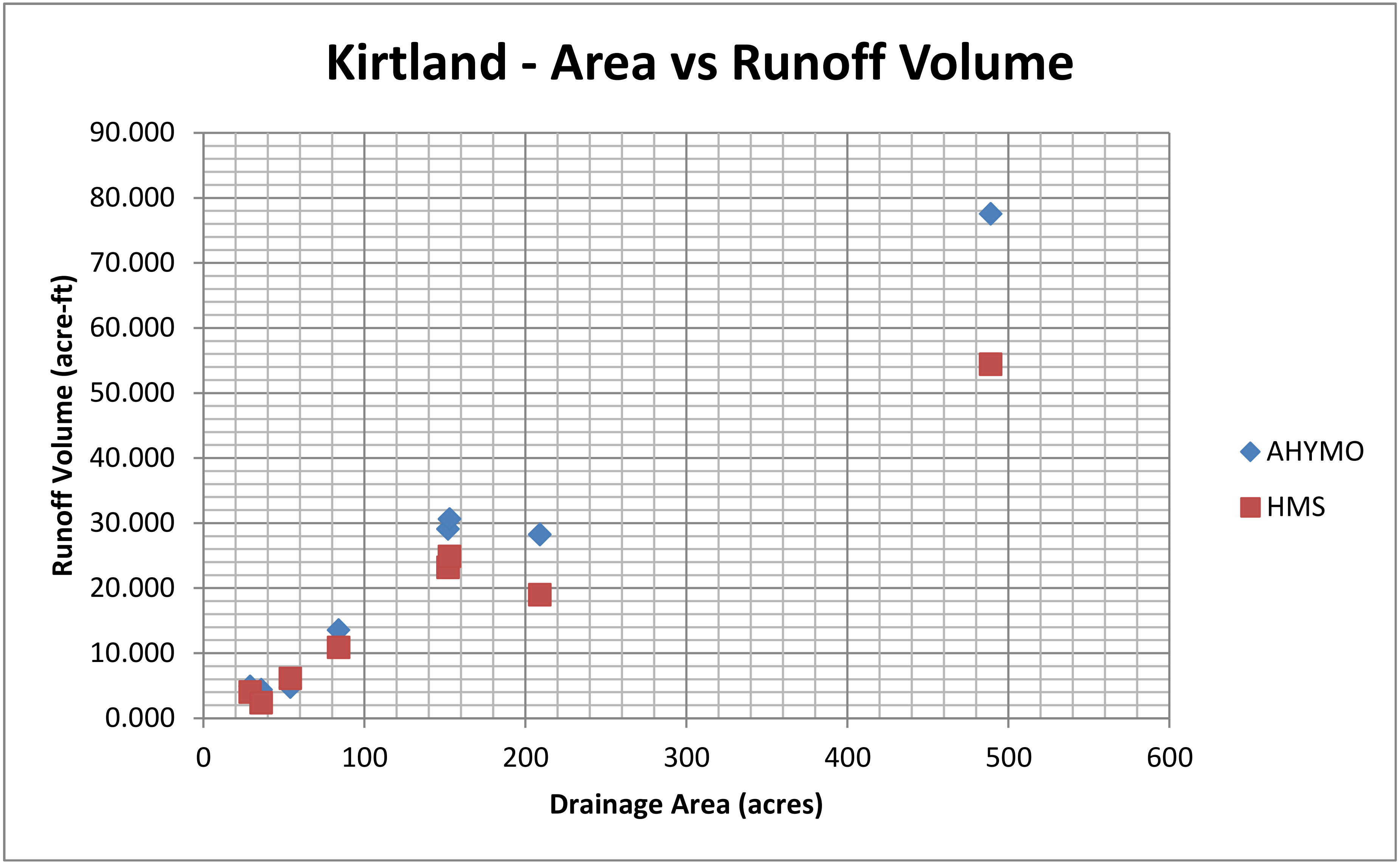

Boca Negra - Peak Discharge per Acre
CFS
1400.00
1200.00
1000.00
800.00
600.00
400.00
200.00
0.00
0.000
2000.000
4000.000
6000.000
8000.000
10000.000
12000.000
Acres
AHYMO
HMS


Kirtland - Area vs Discharge
Peak Discharge (cfs)
1400
1200
1000
800
600
400
200
0
0
100
200
300
400
500
600
Drainage Area (acres)
AHYMO
HMS

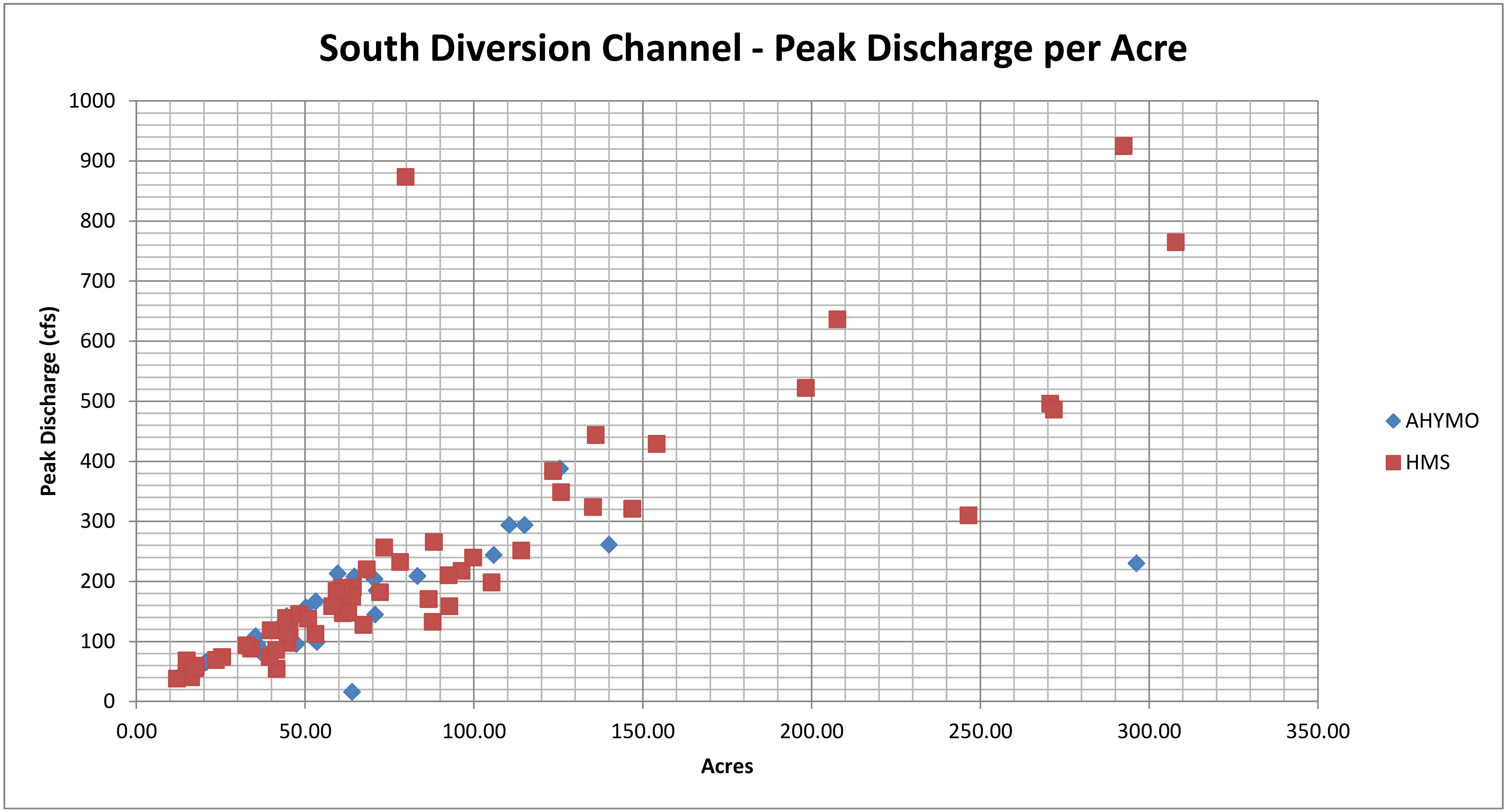


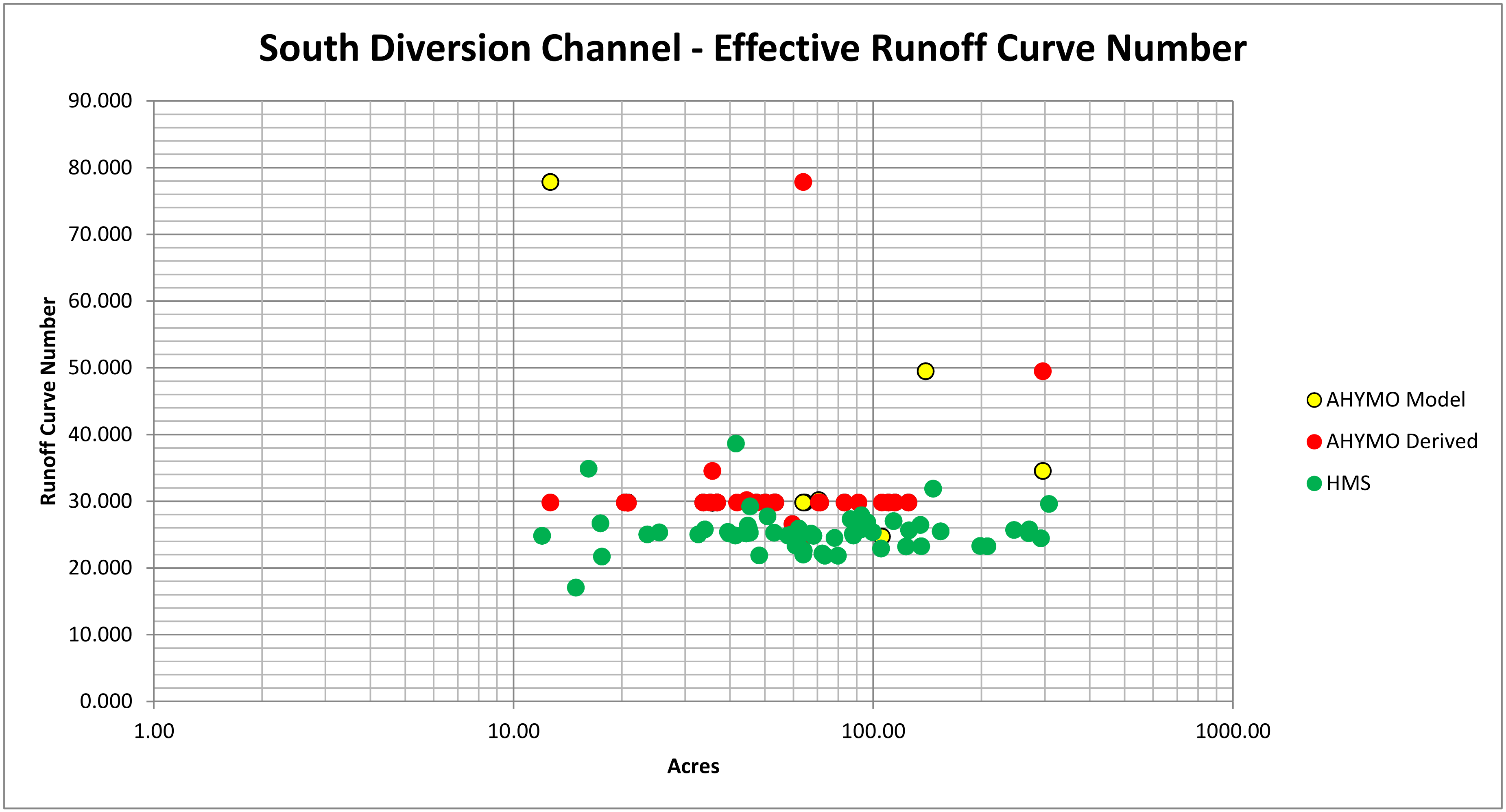
Kirtland - Unit Discharge
(cfs/acre)
Unit Discharge (cfs/acre)
7.00
6.00
5.00
4.00
3.00
2.00
1.00
0.00
0
100
200
300
400
500
600
Drainage Area (acres)
AHYMO
HMS

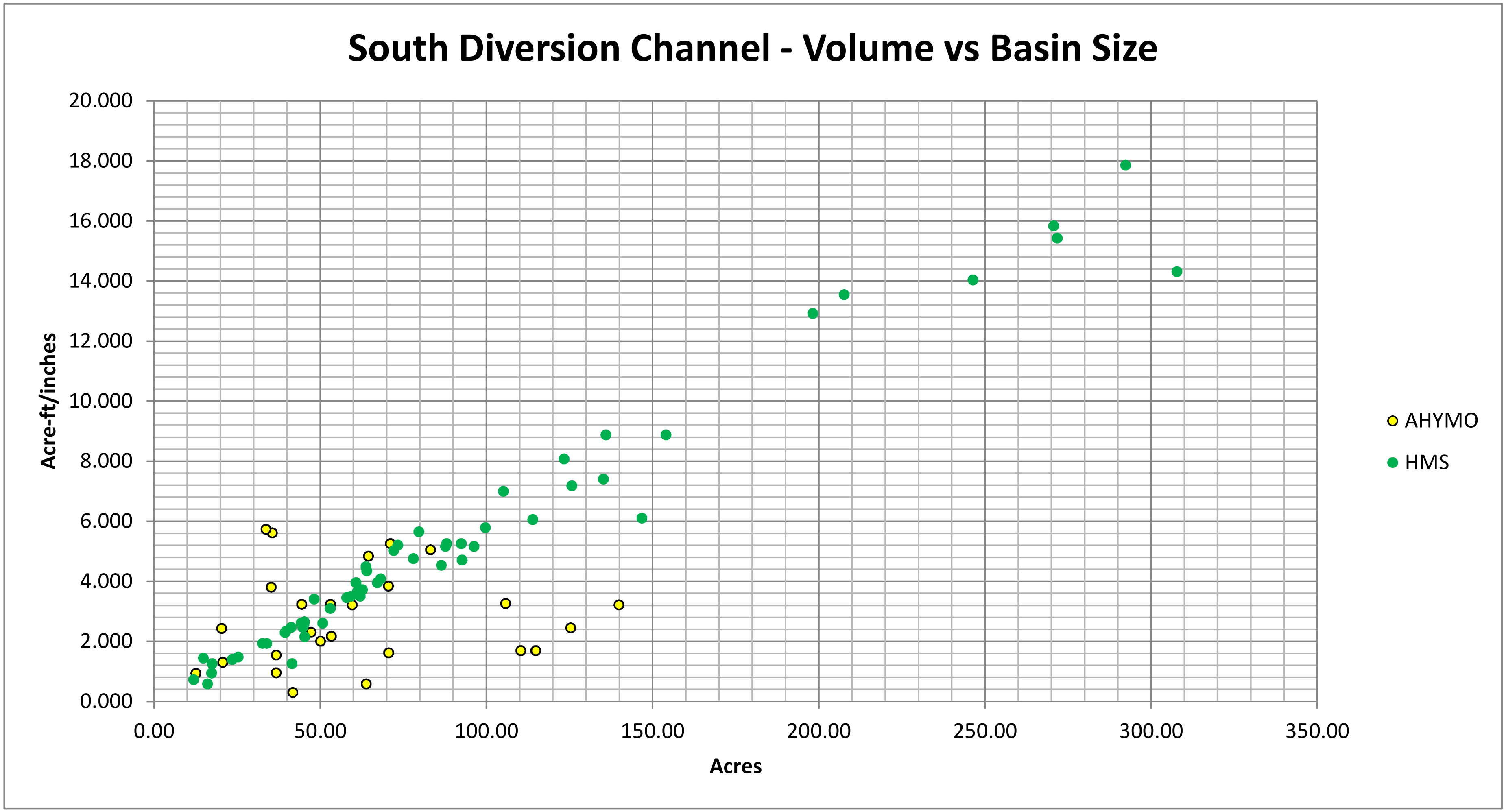

Kirtland - Peak Discharge vs Direct Runoff (cfs/in)
1400
1200
1000
800
600
400
200
0
Peak Discharge (cfs)
0.000
0.500
1.000
1.500
2.000
2.500
3.000
Inches of Runoff (inches)
AHYMO
HMS

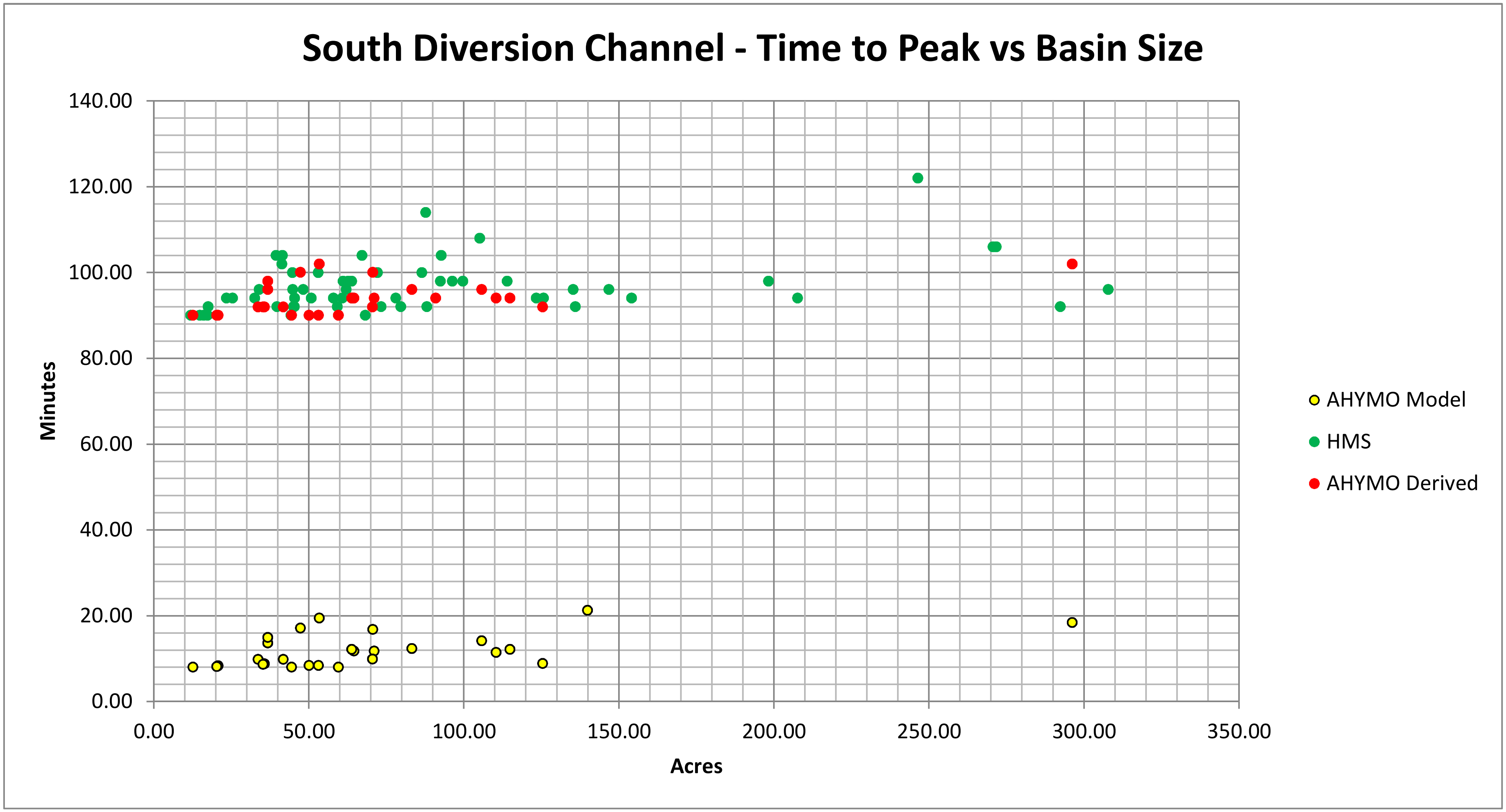


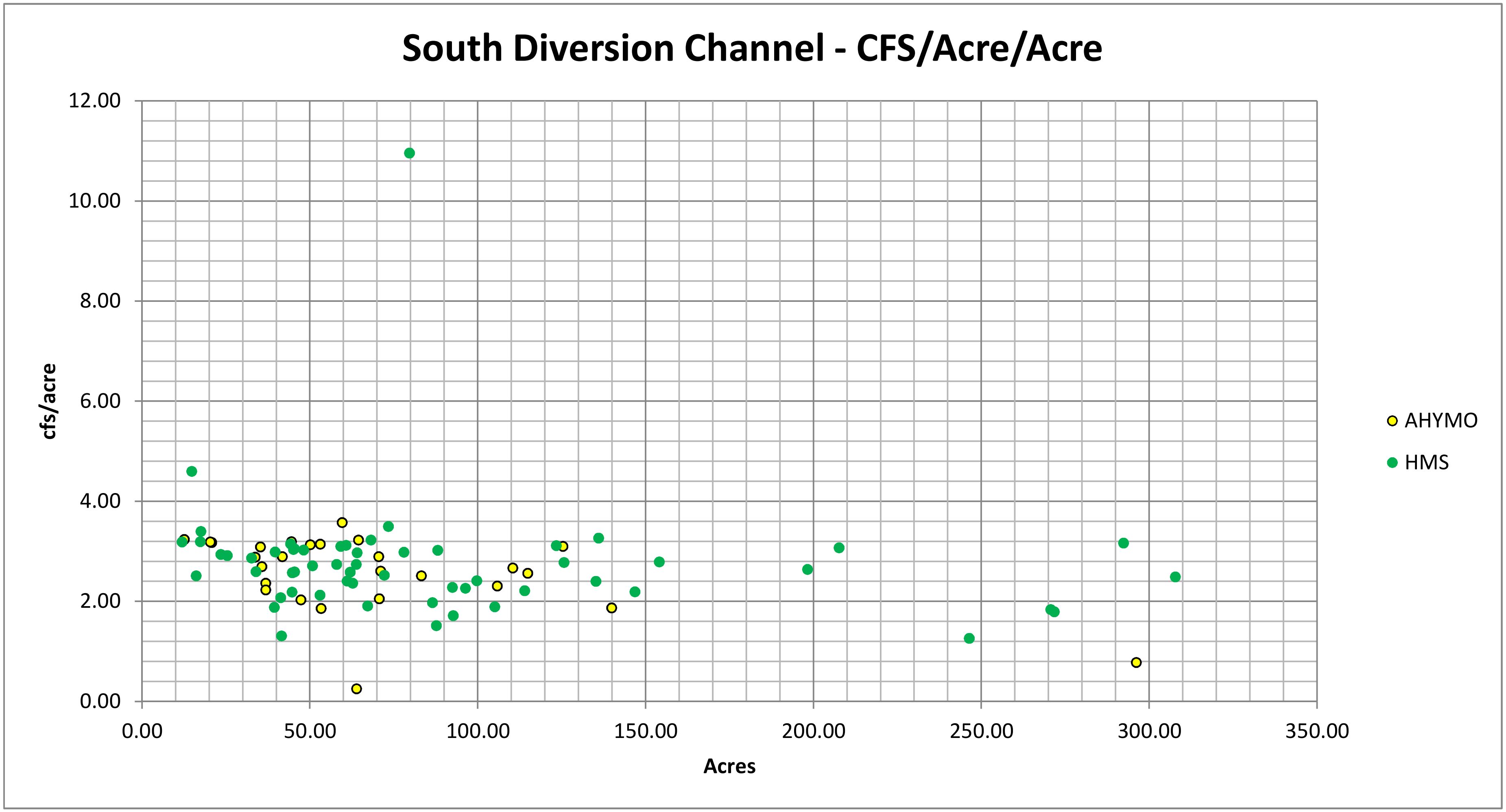
Kirtland - Area vs Runoff Volume
90.000
80.000
70.000
60.000
50.000
40.000
30.000
20.000
10.000
0.000
Runoff Volume (acre-ft)
0
100
200
300
400
500
600
Drainage Area (acres)
AHYMO
HMS

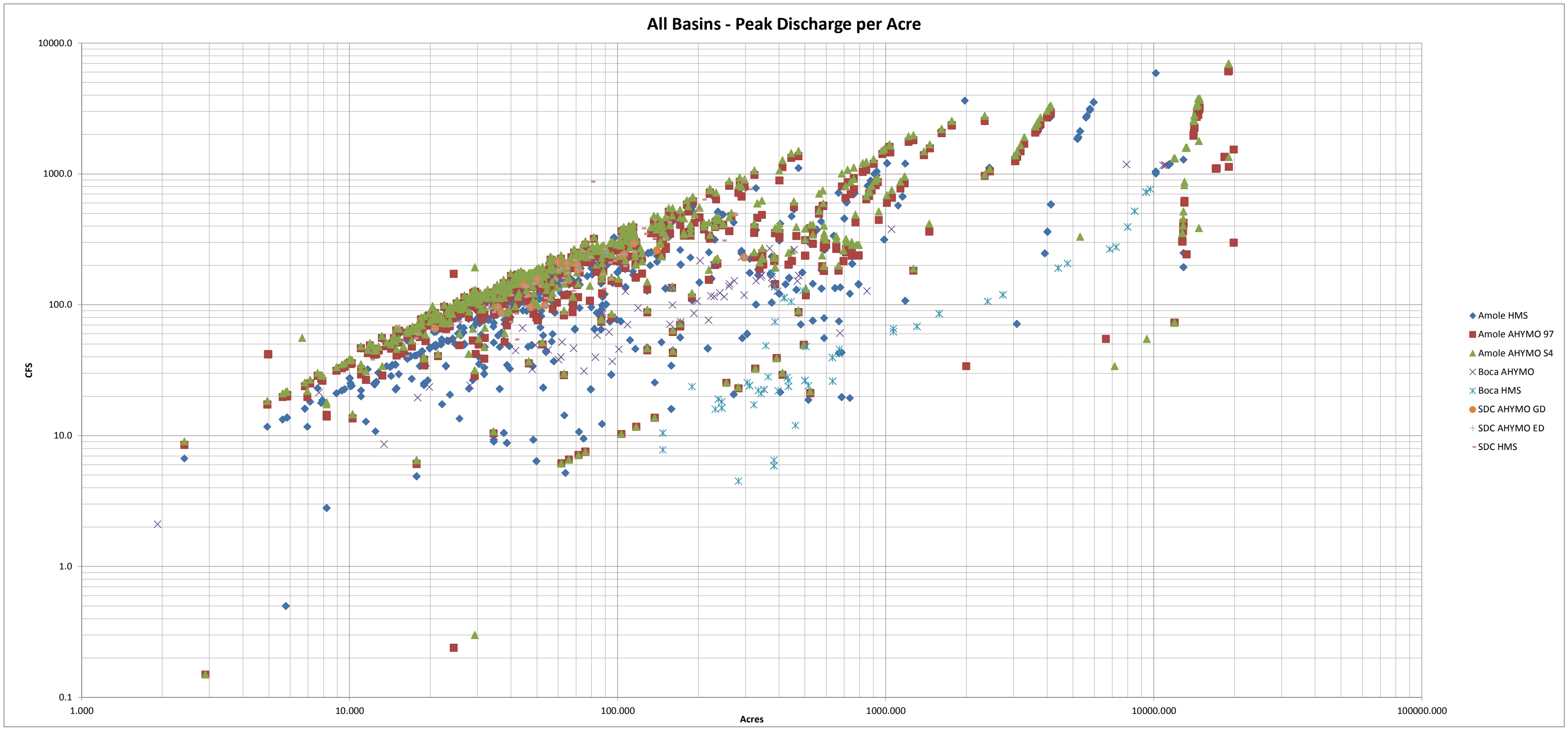

South Diversion Channel - Peak Discharge per Acre
Peak Discharge (cfs)
1000
900
800
700
600
500
400
300
200
100
0
0.00
50.00
100.00
150.00
200.00
250.00
300.00
350.00
Acres
AHYMO
HMS


South Diversion Channel - Effective Runoff Curve Number
Runoff Curve Number
90.000
80.000
70.000
60.000
50.000
40.000
30.000
20.000
10.000
0.000
1.00
10.00
100.00
1000.00
Acres
AHYMO Model
AHYMO Derived
HMS

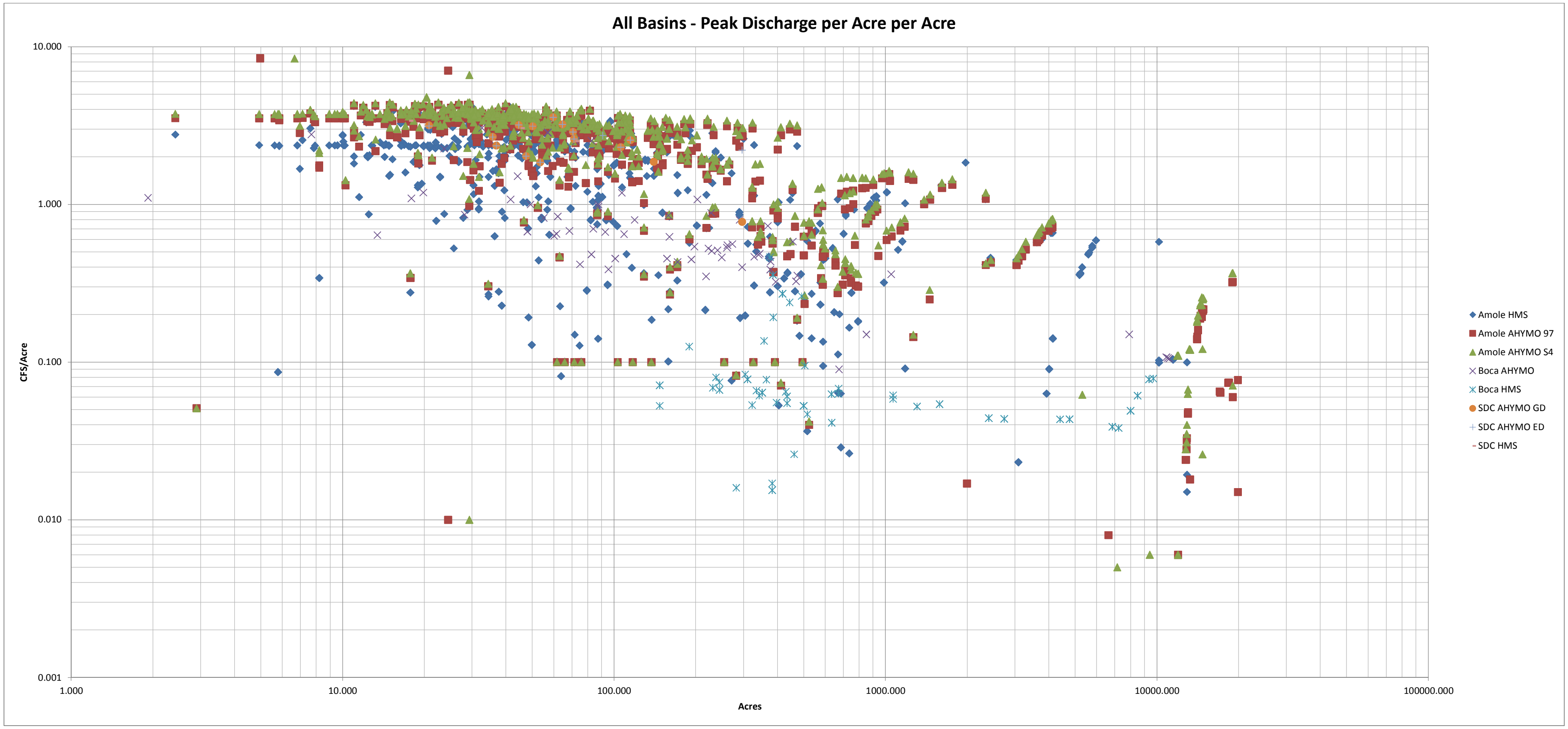

South Diversion Channel - Volume vs Basin Size
Acre-ft/inches
20.000
18.000
16.000
14.000
12.000
10.000
8.000
6.000
4.000
2.000
0.000
0.00
50.00
100.00
150.00
200.00
250.00
300.00
350.00
Acres
AHYMO
HMS

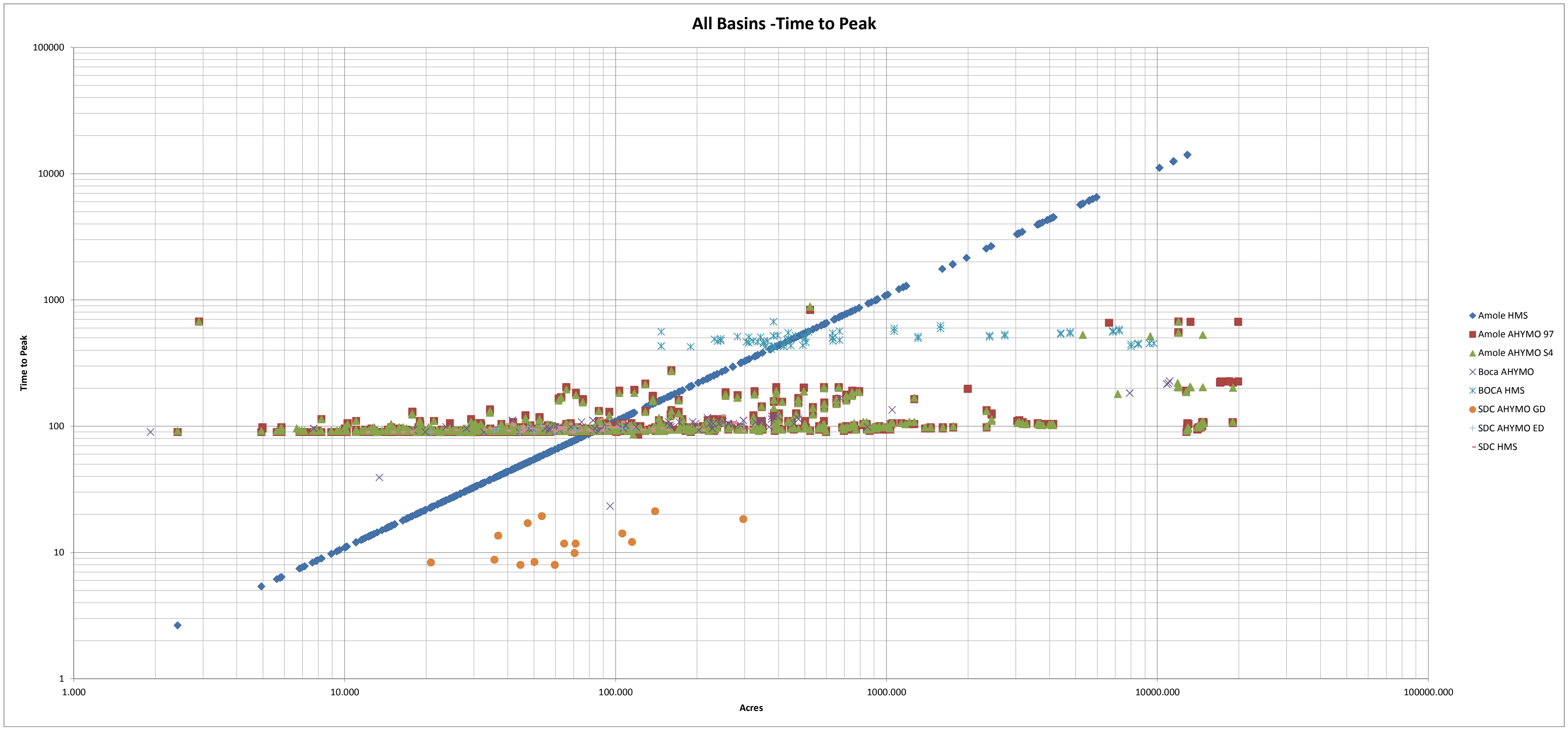

South Diversion Channel - Time to Peak vs Basin Size
Minutes
Acres
140.00
120.00
100.00
80.00
60.00
40.00
20.00
0.00
0.00
50.00
100.00
150.00
200.00
250.00
300.00
350.00
AHYMO Model
HMS
AHYMO Derived

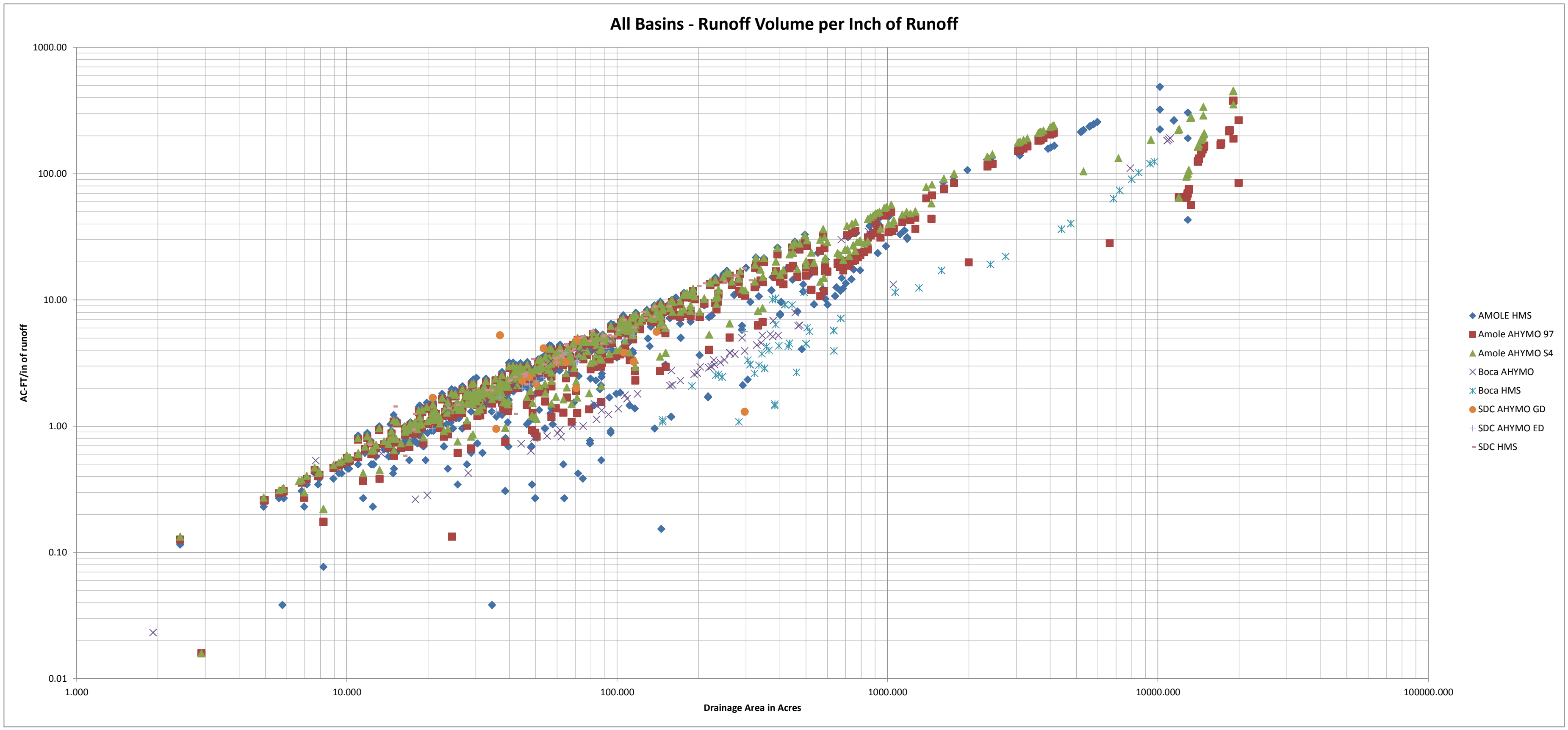

South Diversion Channel - CFS/Acre/Acre
cfs/acre
12.00
10.00
8.00
6.00
4.00
2.00
0.00
0.00
50.00
100.00
150.00
200.00
250.00
300.00
350.00
Acres
AHYMO
HMS

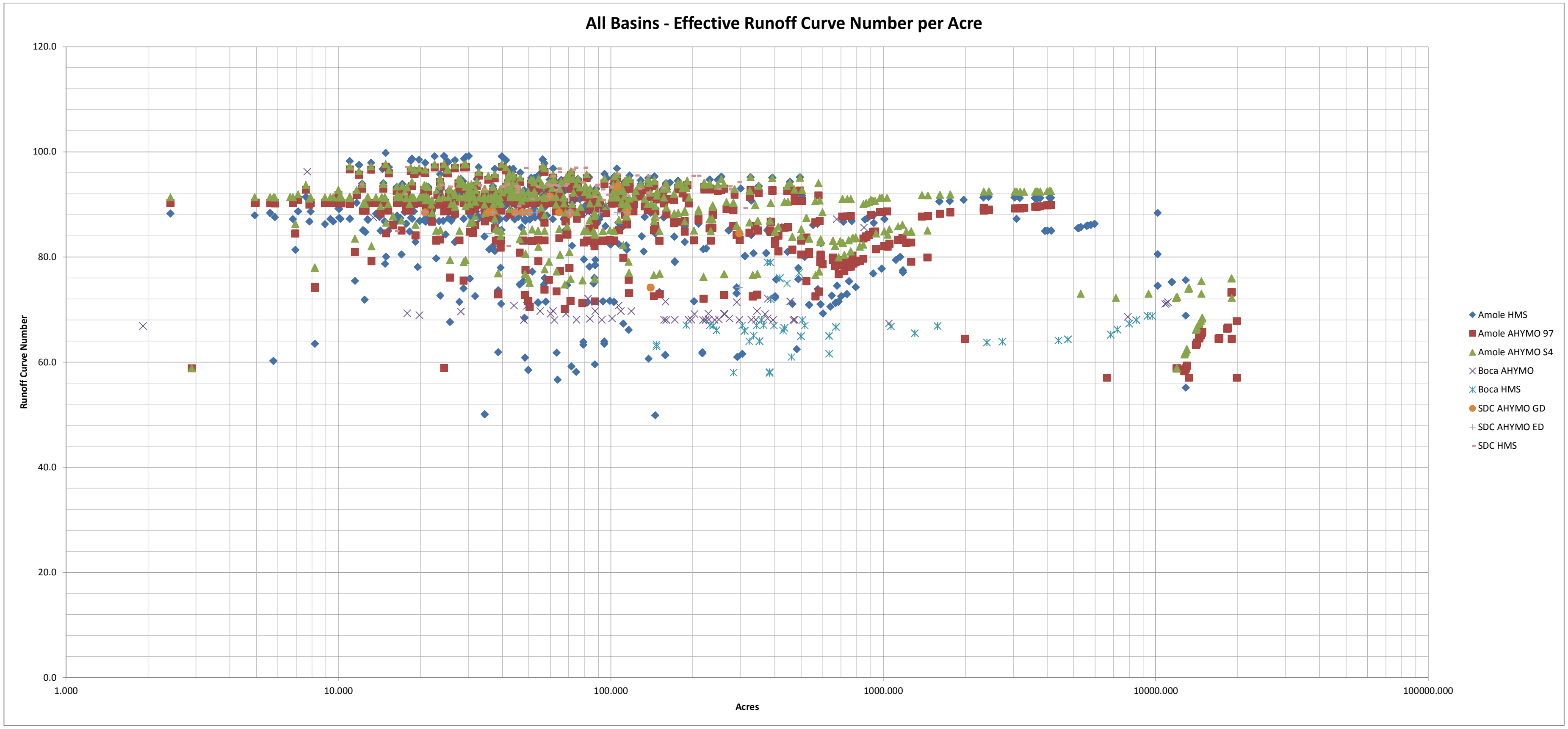


All Basins - Peak Discharge per Acre
10000.0
1000.0
100.0
10.0
1.0
0.1
CFS
1.000
10.000
100.000
1000.000
10000.000
100000.000
Acres
Amole HMS
Amole AHYMO 97
Amole AHYMO S4
Boca AHYMO
Boca HMS
SDC AHYMO GD
SDC AHYMO ED
SDC HMS


All Basins - Peak Discharge per Acre per Acre
10.000
1.000
0.100
0.010
0.001
CFS/Acre
1.000
10.000
100.000
1000.000
10000.000
100000.000
Acres
Amole HMS
Amole AHYMO 97
Amole AHYMO S4
Boca AHYMO
Boca HMS
SDC AHYMO GD
SDC AHYMO ED
SDC HMS


All Basins -Time to Peak
100000
10000
1000
100
10
1
Time to Peak
1.000
10.000
100.000
1000.000
10000.000
100000.000
Acres
Amole HMS
Amole AHYMO 97
Amole AHYMO S4
Boca AHYMO
BOCA HMS
SDC AHYMO GD
SDC AHYMO ED
SDC HMS

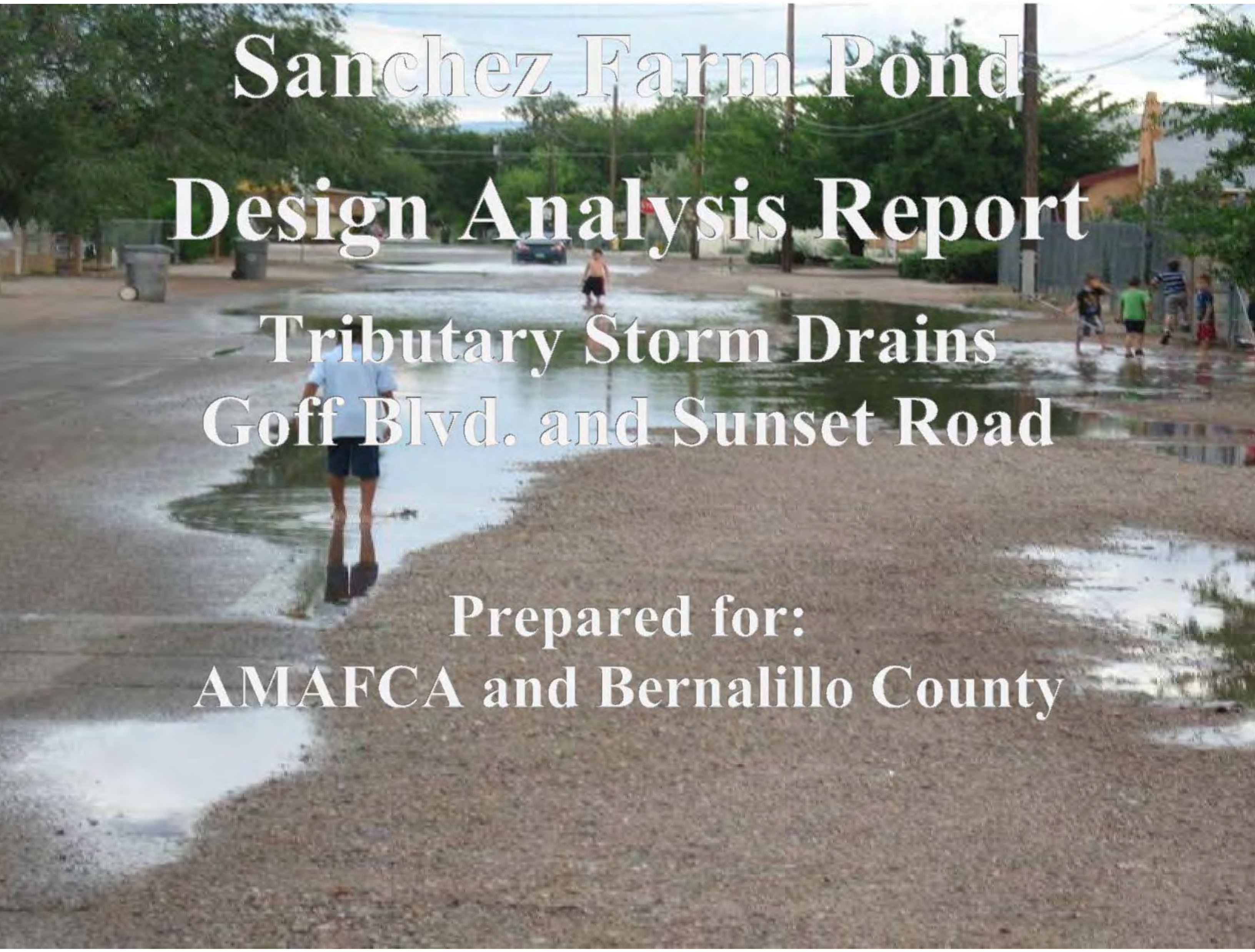

All Basins - Runoff Volume per Inch of Runoff
AC-FT/in of runoff
Drainage Area in Acres
1000.00
100.00
10.00
1.00
0.10
0.01
1.000
10.000
100.000
1000.000
10000.000
100000.000
AMOLE HMS
Amole AHYMO 97
Amole AHYMO S4
Boca AHYMO
Boca HMS
SDC AHYMO GD
SDC AHYMO ED
SDC HMS



All Basins - Effective Runoff Curve Number per Acre
Runoff Curve Number
Acres
120.0
100.0
80.0
60.0
40.0
20.0
0.0
1.000
10.000
100.000
1000.000
10000.000
100000.000
Amole HMS
Amole AHYMO 97
Amole AHYMO S4
Boca AHYMO
Boca HMS
SDC AHYMO GD
SDC AHYMO ED
SDC HMS

APPENDIX I

VALLEY HYDROLOGY METHOD



Sanchez Farm Pond

Design Analysis Report

Tributary Storm Drains
Goff Blvd. and Sunset Road

Prepared for:
AMAFCA and Bernalillo County

Prepared by:

Thompson Engineering Consultants, Inc.

&

Easterling Consultants LLC

Final Submittal July 6, 2012

Table of Contents

Tables

Figures

Plates

Appendices

IV. Methodology

Overview of Methodology

During the study it was determined that much of the information from previous hydrology models completed within the project area was not appropriate for inclusion in this model. In the many previous studies, a variety of methodologies to account for the flat valley terrain were used, while some previous studies did not attempt to account for the flatness at all. It was determined that, in order to develop the most accurate representation of the hydrology, a consistent methodology that recognized the unique valley characteristics should be utilized for those areas draining to Sunset Road and Goff Boulevard and areas north of Bridge Boulevard.

AMAFCA has recently authorized the use of the Army Corps of Engineer's HEC-HMS software in place of AHYMO that has been the standard for the Albuquerque area. This project used HEC-HMS as prescribed in SSCAFCA's DPM (which was developed in an attempt to be compatible with AHYMO methods). The SSCAFA DPM recommends a 2/3 reduction of the time of concentration (t_c) once it has been calculated using commonly used methods. The reason for this reduction was an attempt to exactly match an AHYMO hydrograph. For the purposes of this study it was deemed inappropriate to change a traditional and well understood term such as the t_c.

The SSCAFCA DPM methodology requires the calculation of rainfall distribution curves for input into HEC-HMS. The equations for distributing the rainfall can be found on page 22-52 of the SSCAFCA DPM. The calculation of the distribution curves was performed in MathCad and the calculations are shown in Appendix A.2. The rainfall depths input into the equations are from the NOAA 14 Precipitation Frequency Data Server web site for a point at the centroid of the project area. The output from the website is included in Appendix A.2.

The SSCAFCA DPM begins its estimation of rainfall losses by classifying the land into the land treatment classifications (A, B, C, and D) that have been used in AHYMO models. The option in HEC-HMS which is prescribed by the SSCAFCA DPM uses the Initial and Constant Loss Method. Therefore, the Treatment Types are each assigned an Initial Abstraction and Infiltration Rate. These values are set to the corresponding parameters that are used within AHYMO. However, there is a difference. Per the DPM, "For the Initial and Constant Loss Method as employed in HEC-HMS, it is assumed that there are no losses associated with impervious area (land treatment type D) and rainfall over the impervious area is converted directly to rainfall excess." There are three factors that will be entered into HEC-HMS, one is the initial abstraction (IA) called the Initial Loss in HEC-HMS. The second is the infiltration rate, called the Constant Loss in HEC-HMS. The third is the percentage impervious. In HEC-HMS the rainfall for the impervious area is translated directly into excess i.e. runoff.

Similar to Albuquerque's DPM, the SSCAFCA DPM calls for estimating treatment types based on the zoned dwelling units per acre. The Initial Abstraction and Infiltration Rate assigned to the Land Treatment Types (See Table F, page 22-60 of the SSCAFCA DPM) are multiplied by their respective values to arrive at a weighted IA and INF value for each sub-basin.

For this study, it was determined that the standard IA was not appropriate due to the fact that the lots are lower than the surrounding roadways. For this circumstance the values presented in

<u>"Analysis of the AHYMO Program for Flat Valley Areas"</u> <u>Bohannan-Huston, Inc.</u> February 1995 were used in place of the standard values. The following table presents the AHYMO Program default values and recommended values for <u>flat valley areas</u> in Albuquerque.

TABLE 1 – Comparison of Default and Recommended Initial Abstraction Values

Land Treatment Type	Default Initial Abstraction	Recommend Initial Abstraction	Recommended and Default and Uniform Infiltration Rate
	(inches)	(inches)	(inches / hour)
A	0.65	1.20	1.67
B	0.50	1.05	1.25
C	0.35	0.90	0.83
D	0.10	0.85	0.04

The <u>Average</u> Initial Abstraction for Land Treatment Types A, B, C, D in Table 1 computed by Smith Engineering Company = 1.0 inches

The IA adjustment is not applied to those recent developments (primarily commercial) which are built above the adjacent roadway elevation and have obviously been graded to drain efficiently.

<u>Percentage Impervious</u>

A new data collection technique implemented for this study was the use of satellite remote sensing data for the estimation of the impervious area in each sub-basin. This technique was originally used in the Albuquerque area in a restudy of the South Diversion Channel for AMAFCA performed by Easterling Consultants. It was further refined by Smith Engineering in their Mid Valley Drainage Master Plan for the City of Albuquerque, and is being investigated by the City of Albuquerque for use in Water Quality Analyses.

The initial imperviousness for each sub-basin was obtained using data from the National Land Cover Database (NLCD) that was developed from the Landsat Satellite 2001 Imagery. This NLCD data was converted using the ArcMap "Spatial Analyst Extension" for the developing zonal statistics. The basin boundaries created in AutoCad were converted to a shape file. Those boundaries were applied in the analysis to obtain % imperviousness per sub-basin.

The raw satellite data has a consistent error that must be taken into account. Smith Engineering Company in their application used direct measurements of percent impervious areas within test sub-basins using orthophotography to calibrate the satellite data. The equations developed by Smith Engineering are shown in Appendix A.3 as well as the relevant section of their report. Of the equations developed by Smith Engineering, the parabolic equation was chosen for use in the Sanchez Farm Report. The error in this equation occurs as the equation approaches 100% impervious, with the equation showing adjusted values greater than 100%. This equation was chosen because this error can be easily corrected and the equation makes accurate adjustments over the other ranges of imperviousness. As with any data the final adjusted values should be reviewed. In the case of the satellite data, the area of the "blocks" of reported imperviousness can be large relative to small sub-basins. These sub-basins should be reviewed to ensure that one block has not skewed the result for that sub-basin.

The impervious percentages, the adjusted values, and the values after a thorough visual check against the orthophoto are shown in Table 2.

Treatment Types

The percentage of imperviousness is then translated directly to Percentage of Treatment Type D, for the calculations of the initial abstraction and R values. Then a value of Treatment Type A is measured or estimated based on the aerial photography and the remaining areas are split between Treatment Type B and C unless visual examination showed that it should be otherwise. The Treatment Types are shown in Table 2.

Time of Concentration

Time of Concentration is the term that defines the hydraulic response of a sub-basin in the calculation of a runoff hydrograph. The unit peak discharge is inversely proportional to the basin time of concentration. Basins with short times of concentration tend to have higher unit peak discharges than do basins with longer times of concentration. The SSCAFCA DPM recommends the same procedure for the calculation of the Time of Concentration (t_c) for sub-basins as is found in the Albuquerque DPM except it is reduced by 2/3. The 2/3 reduction was a modification to make the resultant hydrographs from HEC-HMS more closely agree with AHYMO hydrographs. However, the t_c is a standard term in hydrology and is tied to the physical time it takes for flows to collect to a common discharge point. For this study the standard t_c was employed rather than the modified version.

The t_c is calculated by using the reach lengths of the "longest flow path" within that sub-basin. Each reach length is multiplied by a k factor to determine an estimated velocity of water flow for that reach. The k factor values range from 0.7 to 4. A k value of 0.7 is used when runoff is in sheet flow, before flows have concentrated. A k value of 1 is also used for sheet flow but on a hardened surface, values of 2 thru 4 are for increasing levels of concentration and thus increasing velocities. Generally the use of a 0.7 factor is limited to the first 400 feet of the longest flow path. This limit assumes that the runoff will be collected into some type of concentrated drainage feature or conveyance by this point i.e. a curb, inverted street, swale, or channel. In the southwest valley this is generally not the case, so it becomes a key issue in any analysis in this area. Excess rainfall that does not pond generally flows as unconcentrated sheet flow.

As shown in the Background section, various studies of this area have employed a modification to the standard procedure to acknowledge the fact that runoff in the project area remains in a sheet flow state longer than in other areas and are likely to re-disperse into a sheet flow condition even after having concentrated. This same methodology is used in this study, with the exception of those commercially developed lots that have been clearly graded to drain more effectively. The input t_c values are shown in Table 2, and the calculations are shown in Appendix A.4.

The t_c and the Treatment Types are used to calculate the Initial Loss (i.e. IA), Constant Loss (Infiltration Rate) and R values which will be input into HEC-HMS. The R value is a factor which affects the shape of the resultant hydrograph. The IA, INF, and R values are shown in Table 2. Further explanation can be found in the SSCAFCA DPM and its supporting documents. These calculations are shown in Appendix A.5.

The full HEC-HMS model created from this input is located electronically in Appendix A.6. HEC-HMS is a freely distributed software package and is a graphically based software facilitating efficient review of models. Summary results from the HEC-HMS model are shown in Table 2.

EXISTING CONDITION - CALCULATIONS FOR IA, INF, AND R-VALUE

$$\begin{pmatrix} NAME \\ PERCENT_A \\ PERCENT_B \\ PERCENT_C \\ PERCENT_D \end{pmatrix} :=$$

BASINS	A	B	C	D
101	0	38	38	24
102	64	9	9	18
103	25	25	25	25
104	5	30	30	35
105	0	20	20	60
106	5	33	33	30
107	0	37	37	26
108	0	25	25	50
109	0	32	32	36
110	0	31	31	39
111	0	27	27	45
114	0	33	33	34
115	0	15	15	70
116	0	34	34	32
118	0	32	32	36
120	0	42	42	16
121	0	25	25	50
122	0	35	35	30
123	0	33	33	34
124	16	31	31	23
125	0	36	36	28
126	0	35	35	30
127	0	41	41	17
128	32	24	24	19
129	0	34	34	32
130	50	25	25	16
131A	60	0	25	15
131B	0	0	15	85
132	5	37	37	21
133	50	16	16	9
134	0	8	8	85
135	100	0	0	0
136	0	38	38	24
137	0	45	45	10
138A	0	36	36	28
138B	0	36	36	28
139	0	30	30	40
140	22	24	24	30
141	5	31	31	33
142	0	37.5	37.5	25

85

Typically, define the Initial Abstraction Values for Each Treatment Type- From Table E-4 of SSCAFCA DPM which match those IAs used as the standard in AHYMO. Several reports for the South Valley have increased the IAs based on the very flat slopes in this area. Increased IAs are used here based on "Analysis of the AHYMO Program for Flat Areas" by BHI, 1995. It should be noted that using the SSCAFCAA methodology means that the IA for treatment type D is not factored into the equation.

$IA_A := 1.20 \qquad IA_B := 1.05 \qquad IA_C := 0.9 \qquad IA_D := 0.40$

$j := 0..39$

$$IA_j := \frac{IA_A \cdot PERCENT_{A_j} + IA_B \cdot PERCENT_{B_j} + IA_C \cdot PERCENT_{C_j}}{PERCENT_{A_j} + PERCENT_{B_j} + PERCENT_{C_j}}$$

CALCULATION OF IA, INF, AND R-VALUE FOR ROADS AND COMMERCIAL SITES

$$\begin{pmatrix} NAME \\ PERCENT_A \\ PERCENT_B \\ PERCENT_C \\ PERCENT_D \end{pmatrix} :=$$

BASINS	A	B	C	D
112	0	0	10	90
113	0	8	8	84
117	0	15	15	70
119	0	8	8	84
BR 1	0	0	15	85
BR 2	0	0	15	85
GOFF 1	0	0	15	85
GOFF2	0	0	15	85
GOFF 3	0	0	15	85
GOFF 4	0	0	15	85
SUN 1	0	0	15	85
SUN 2	0	0	15	85
SUN 3	0	0	15	85
SUN 4	0	0	15	85
SUN 5	0	0	15	85
SUN 6	0	0	15	85
ISLETA 1	0	0	10	90
ISLETA 2	0	0	10	90
143	0	0	15	85
ARENAL 1	0	0	10	90
ARENAL 2	0	0	10	90
BR 3	0	0	10	90
BR 4	0	0	10	90
BR 5	0	0	10	90
BR 6	0	0	10	90
BR 7	0	0	10	90
BR 8	0	0	10	90
BR 9	0	0	10	90

CALCULATIONS FOR DEVELOPED SUB-BASINS THAT REQUIRE MODIFIED IA

$$\begin{pmatrix} NAME \\ PERCENT_A \\ PERCENT_B \\ PERCENT_C \\ PERCENT_D \end{pmatrix} :=$$

BASINS	A	B	C	D
101	0	38	38	24
102	0	35	35	30
103	0	35	35	30
104	5	30	30	35
105	0	20	20	60
106	5	33	33	30
107	0	37	37	26
108	0	25	25	50
109	0	32	32	36
110	0	31	31	39
111	0	27	27	45
114	0	33	33	34
115	0	15	15	70
116	0	34	34	32
118	0	32	32	35
120	0	35	35	30
121	0	25	25	50
122	0	35	35	30
123	0	33	33	34
124A	0	35	35	30
125	0	36	36	28
126	0	35	35	30
127	0	35	35	30
128	0	35	35	30
129	0	34	34	32
130	0	35	35	30
131A	0	35	35	30
131B	0	0	15	85
132	5	37	37	21
133	50	16	16	9
134	0	8	8	85
135	0	35	35	30
136	0	38	38	24
137	0	45	45	10
138A	0	36	36	28
138B	0	36	36	28
139	0	30	30	40
140	0	35	35	30
141	5	31	31	33
142	0	37.5	37.5	25
124B	10	32	32	25
S100	0	36	36	28
S101	5	34	34	27
S102	0	36	36	28
S103	0	30	30	40
S114	5	33	33	29
S116	0	36	36	28

85

Typically, define the Initial Abstraction Values for Each Treatment Type- From Table E-4 of SSCAFCA DPM which match those IAs used as the standard in AHYMO. Several reports for the South Valley have increased the IAs based on the very flat slopes in this area. Increased IAs are used here based on "Analysis of the AHYMO Program for Flat Areas" by BHI, 1995. It should be noted that using the SSCAFCAA methodology means that the IA for treatment type D is not factored into the equation.

$IA_A := 1.20$ $IA_B := 1.05$ $IA_C := 0.9$ $IA_D := 0.40$

$j := 0 .. 46$

$$IA_j := \frac{IA_A \cdot PERCENT_{A_j} + IA_B \cdot PERCENT_{B_j} + IA_C \cdot PERCENT_{C_j}}{PERCENT_{A_j} + PERCENT_{B_j} + PERCENT_{C_j}}$$

CALCULATION OF IA, INF, AND R-VALUE FOR ROADS AND COMMERCIAL SITES

$$\begin{pmatrix} NAME \\ PERCENT_A \\ PERCENT_B \\ PERCENT_C \\ PERCENT_D \end{pmatrix} :=$$

BASINS	A	B	C	D
112	0	0	10	90
113	0	8	8	84
117	0	15	15	70
119	0	8	8	84
BR 1	0	0	15	85
BR 2	0	0	15	85
GOFF 1	0	0	15	85
GOFF2	0	0	15	85
GOFF 3	0	0	15	85
GOFF 4	0	0	15	85
SUN 1	0	0	15	85
SUN 2	0	0	15	85
SUN 3	0	0	15	85
SUN 4	0	0	15	85
SUN 5	0	0	15	85
SUN 6	0	0	15	85
ISLETA 1	0	0	10	90
ISLETA 2	0	0	10	90
143	0	0	15	85
ARENAL 1	0	0	10	90
ARENAL 2	0	0	10	90
BR 3	0	0	10	90
BR 4	0	0	10	90
BR 5	0	0	10	90
BR 6	0	0	10	90
BR 7	0	0	10	90
BR 8	0	0	10	90
BR 9	0	0	10	90

CALCULATIONS FOR DEVELOPED SUB-BASINS THAT REQUIRE MODIFIED IA

$$\begin{pmatrix} NAME \\ PERCENT_A \\ PERCENT_B \\ PERCENT_C \\ PERCENT_D \end{pmatrix} :=$$

BASINS	A	B	C	D
101	0	38	38	24
102	0	35	35	30
103	0	35	35	30
104	5	30	30	35
105	0	20	20	60
106	5	33	33	30
107	0	37	37	26
108	0	25	25	50
109	0	32	32	36
110	0	31	31	39
111	0	27	27	45
114	0	33	33	34
115	0	15	15	70
116	0	34	34	32
118	0	32	32	35
120	0	35	35	30
121	0	25	25	50
122	0	35	35	30
123	0	33	33	34
124A	0	35	35	30
125	0	36	36	28
126	0	35	35	30
127	0	35	35	30
128	0	35	35	30
129	0	34	34	32
130	0	35	35	30
131A	0	35	35	30
131B	0	0	15	85
132	5	37	37	21
133	50	16	16	9
134	0	8	8	85
135	0	35	35	30
136	0	38	38	24
137	0	45	45	10
138A	0	36	36	28
138B	0	36	36	28
139	0	30	30	40
140	0	35	35	30
141	5	31	31	33
142	0	37.5	37.5	25
124B	10	32	32	25
S100	0	36	36	28
S101	5	34	34	27
S102	0	36	36	28
S103	0	30	30	40
S114	5	33	33	29
S116	0	36	36	28

85

Typically, define the Initial Abstraction Values for Each Treatment Type- From Table E-4 of SSCAFCA DPM which match those IAs used as the standard in AHYMO. Several reports for the South Valley have increased the IAs based on the very flat slopes in this area. Increased IAs are used here based on "Analysis of the AHYMO Program for Flat Areas" by BHI, 1995. It should be noted that using the SSCAFCAA methodology means that the IA for treatment type D is not factored into the equation.

$IA_A := 1.20 \qquad IA_B := 1.05 \qquad IA_C := 0.9 \qquad IA_D := 0.40$

$j := 0 .. 46$

$$IA_j := \frac{IA_A \cdot PERCENT_{A_j} + IA_B \cdot PERCENT_{B_j} + IA_C \cdot PERCENT_{C_j}}{PERCENT_{A_j} + PERCENT_{B_j} + PERCENT_{C_j}}$$

EXISTING CONDITION - CALCULATIONS FOR IA, INF, AND R-VALUE

$$\begin{pmatrix} NAME \\ PERCENT_A \\ PERCENT_B \\ PERCENT_C \\ PERCENT_D \end{pmatrix} :=$$

BASINS	A	B	C	D
101	0	38	38	24
102	64	9	9	18
103	25	25	25	25
104	5	30	30	35
105	0	20	20	60
106	5	33	33	30
107	0	37	37	26
108	0	25	25	50
109	0	32	32	36
110	0	31	31	39
111	0	27	27	45
114	0	33	33	34
115	0	15	15	70
116	0	34	34	32
118	0	32	32	35
120	0	42	42	16
121	0	25	25	50
122	0	35	35	30
123	0	33	33	34
124	16	31	31	23
125	0	36	36	28
126	0	35	35	30
127	0	41	41	17
128	32	24	24	19
129	0	34	34	32
130	50	25	25	16
131A	60	0	25	15
131B	0	0	15	85
132	5	37	37	21
133	50	16	16	9
134	0	8	8	85
135	100	0	0	0
136	0	38	38	24
137	0	45	45	10
138A	0	36	36	28
138B	0	36	36	28
139	0	30	30	40
140	22	24	24	30
141	5	31	31	33
142	0	37.5	37.5	25

85

Typically, define the Initial Abstraction Values for Each Treatment Type- From Table E-4 of SSCAFCA DPM which match those IAs used as the standard in AHYMO. Several reports for the South Valley have increased the IAs based on the very flat slopes in this area. Increased IAs are used here based on "Analysis of the AHYMO Program for Flat Areas" by BHI, 1995. It should be noted that using the SSCAFCAA methodology means that the IA for treatment type D is not factored into the equation.

$IA_A := 1.20$ $\quad IA_B := 1.05$ $\quad IA_C := 0.9$ $\quad IA_D := 0.40$

$j := 0 .. 39$

$$IA_j := \frac{IA_A \cdot PERCENT_{A_j} + IA_B \cdot PERCENT_{B_j} + IA_C \cdot PERCENT_{C_j}}{PERCENT_{A_j} + PERCENT_{B_j} + PERCENT_{C_j}}$$

CALCULATIONS FOR DEVELOPED SUB-BASINS THAT REQUIRE MODIFIED IA

$$\begin{pmatrix} NAME \\ PERCENT_A \\ PERCENT_B \\ PERCENT_C \\ PERCENT_D \end{pmatrix} :=$$

BASINS	A	B	C	D	
101	0	38	38	24	
102	0	35	35	30	
103	0	35	35	30	
104	5	30	30	35	
105	0	20	20	60	
106	5	33	33	30	
107	0	37	37	26	
108	0	25	25	50	
109	0	32	32	36	
110	0	31	31	39	
111	0	27	27	45	
114	0	33	33	34	
115	0	15	15	70	
116	0	34	34	32	
118	0	32	32	35	
120	0	35	35	30	
121	0	25	25	50	
122	0	35	35	30	
123	0	33	33	34	
124A	0	35	35	30	
125	0	36	36	28	
126	0	35	35	30	
127	0	35	35	30	
128	0	35	35	30	
129	0	34	34	32	
130	0	35	35	30	
131A	0	35	35	30	
131B	0	0	15	85	85
132	5	37	37	21	
133	50	16	16	9	
134	0	8	8	85	
135	0	35	35	30	
136	0	38	38	24	
137	0	45	45	10	
138A	0	36	36	28	
138B	0	36	36	28	
139	0	30	30	40	
140	0	35	35	30	
141	5	31	31	33	
142	0	37.5	37.5	25	
124B	10	32	32	25	
S100	0	36	36	28	
S101	5	34	34	27	
S102	0	36	36	28	
S103	0	30	30	40	
S114	5	33	33	29	
S116	0	36	36	28	

Typically, define the Initial Abstraction Values for Each Treatment Type- From Table E-4 of SSCAFCA DPM which match those IAs used as the standard in AHYMO. Several reports for the South Valley have increased the IAs based on the very flat slopes in this area. Increased IAs are used here based on "Analysis of the AHYMO Program for Flat Areas" by BHI, 1995. It should be noted that using the SSCAFCAA methodology means that the IA for treatment type D is not factored into the equation.

$IA_A := 1.20 \qquad IA_B := 1.05 \qquad IA_C := 0.9 \qquad IA_D := 0.40$

$j := 0..46$

$$IA_j := \frac{IA_A \cdot PERCENT_{A_j} + IA_B \cdot PERCENT_{B_j} + IA_C \cdot PERCENT_{C_j}}{PERCENT_{A_j} + PERCENT_{B_j} + PERCENT_{C_j}}$$

EXISTING CONDITION - CALCULATIONS FOR IA, INF, AND R-VALUE

$$\begin{pmatrix} NAME \\ PERCENT_A \\ PERCENT_B \\ PERCENT_C \\ PERCENT_D \end{pmatrix} :=$$

BASINS	A	B	C	D
101	0	38	38	24
102	64	9	9	18
103	25	25	25	25
104	5	30	30	35
105	0	20	20	60
106	5	33	33	30
107	0	37	37	26
108	0	25	25	50
109	0	32	32	36
110	0	31	31	39
111	0	27	27	45
114	0	33	33	34
115	0	15	15	70
116	0	34	34	32
118	0	32	32	35
120	0	42	42	16
121	0	25	25	50
122	0	35	35	30
123	0	33	33	34
124	16	31	31	23
125	0	36	36	28
126	0	35	35	30
127	0	41	41	17
128	32	24	24	19
129	0	34	34	32
130	50	25	25	16
131A	60	0	25	15
131B	0	0	15	85
132	5	37	37	21
133	50	16	16	9
134	0	8	8	85
135	100	0	0	0
136	0	38	38	24
137	0	45	45	10
138A	0	36	36	28
138B	0	36	36	28
139	0	30	30	40
140	22	24	24	30
141	5	31	31	33
142	0	37.5	37.5	25

85

Typically, define the Initial Abstraction Values for Each Treatment Type- From Table E-4 of SSCAFCA DPM which match those IAs used as the standard in AHYMO. Several reports for the South Valley have increased the IAs based on the very flat slopes in this area. Increased IAs are used here based on "Analysis of the AHYMO Program for Flat Areas" by BHI, 1995. It should be noted that using the SSCAFCAA methodology means that the IA for treatment type D is not factored into the equation.

$IA_A := 1.20 \quad IA_B := 1.05 \quad IA_C := 0.9 \quad IA_D := 0.40$

$j := 0..39$

$$IA_j := \frac{IA_A \cdot PERCENT_{A_j} + IA_B \cdot PERCENT_{B_j} + IA_C \cdot PERCENT_{C_j}}{PERCENT_{A_j} + PERCENT_{B_j} + PERCENT_{C_j}}$$

Define the Infiltration Value for Each Treatment Type - From Table E-4 of SSCAFCA DPM

$INF_A := 1.67$ $INF_B := 1.25$ $INF_C := 0.83$

$$INF_j := \frac{INF_A \cdot PERCENT_{A_j} + INF_B \cdot PERCENT_{B_j} + INF_C \cdot PERCENT_{C_j}}{PERCENT_{A_j} + PERCENT_{B_j} + PERCENT_{C_j}}$$

Initial Abstraction Results

SUB-BASIN	IA	INF
101	0.975	1.040
102	1.151	1.533
103	1.050	1.251
104	0.992	1.089
105	0.975	1.040
106	0.991	1.085
107	0.975	1.040
108	0.975	1.040
109	0.975	1.040
110	0.975	1.040
111	0.975	1.040
114	0.975	1.040
115	0.975	1.040
116	0.975	1.040
118	0.975	1.040
120	0.975	1.040
121	0.975	1.040
122	0.975	1.040
123	0.975	1.040
124	1.022	1.171
125	0.975	1.040
126	0.975	1.040
127	0.975	1.040
128	1.064	1.290
129	0.975	1.040
130	1.088	1.355
131A	1.112	1.423
131B	0.900	0.830
132	0.989	1.080
133	1.111	1.420
134	0.975	1.040
135	1.200	1.670
136	0.975	1.040
137	0.975	1.040
138A	0.975	1.040
138B	0.975	1.040
139	0.975	1.040
140	1.046	1.238
141	0.992	1.087
142	0.975	1.040

(NAME IA INF)

This starts the portion of the sheet to calculate the Storage Coefficient (R).

Bring in the calculated Tc values from other files.

$t_c :=$

Sub-basin	Tc
101	1.23
102	2.00
103	3.58
104	2.23
105	0.17
106	1.97
107	0.42
108	0.13
109	0.75
110	0.13
111	1.07
114	0.49
115	0.28
116	2.37
118	1.28
120	0.58
121	0.23
122	0.27
123	0.30
124	1.42
125	0.31
126	0.66
127	0.33
128	0.47
129	0.39
130	0.28
131A	0.45
131B	0.13
132	0.32
133	0.26
134	0.13
135	0.63
136	0.89
137	0.19
138A	0.31
138B	0.24
139	0.68
140	1.80
141	1.62
142	0.13

NAME

Per the SSCAFCA DPM an adjustment is required to the traditional tc in order to make the hydrograph mimic a hydrograph computed in AHYMO. The adjustment is to multiply the tc by 2/3.

$$t_{c\_mod_j} := \begin{cases} 0.1333 & \text{if } t_{c_j} \le 0.20 \\ \left(t_{c_j} \cdot \frac{2}{3}\right) & \text{otherwise} \end{cases}$$

Now the Storage Coefficient (R) can be calculated

$$R_j := 1.165 \cdot t_{c\_mod_j} \left[\left(INF_j\right)^{0.45} - \left(IA_j\right)^{1.4} \left(\frac{PERCENT_{D_j}}{100}\right)^{0.4} \right]$$

Alternative Method for Calculating R based on Stantec's "Technical Documentation for use of HEC-HMS with the Development Process Manual

$$R_{stan_j} := 1.165 \cdot t_{c_j} \cdot \left[\left(INF_j\right)^{0.45} - \left(IA_j\right)^{1.4} \cdot \left(\frac{PERCENT_{D_j}}{100}\right)^{0.4} \right]$$

Per Conversations with Stantec and SSCAFCA this is not their intended methodology. The strict DPM method should be used, not what is shown in Stantec's background manual.

SUMMARY TABLE with R values

Sub-basin	IA	INF	t_c	t_{c_mod}	R	R_{stan}
101	0.98	1.04	1.23	0.82	0.447	0.670
102	1.15	1.53	2.00	1.34	0.928	1.392
103	1.05	1.25	3.58	2.39	1.356	2.034
104	0.99	1.09	2.23	1.49	0.669	1.003
105	0.98	1.04	0.17	0.13	0.036	0.047
106	0.99	1.08	1.97	1.32	0.656	0.984
107	0.98	1.04	0.42	0.28	0.149	0.223
108	0.98	1.04	0.13	0.13	0.044	0.042
109	0.98	1.04	0.75	0.50	0.218	0.328
110	0.98	1.04	0.13	0.13	0.056	0.053
111	0.98	1.04	1.07	0.71	0.261	0.392
114	0.98	1.04	0.49	0.33	0.149	0.223
115	0.98	1.04	0.28	0.19	0.040	0.060
116	0.98	1.04	2.37	1.58	0.743	1.114
118	0.98	1.04	1.28	0.85	0.379	0.569
120	0.98	1.04	0.58	0.39	0.249	0.373
121	0.98	1.04	0.23	0.15	0.051	0.077
122	0.98	1.04	0.27	0.18	0.087	0.131
123	0.98	1.04	0.30	0.20	0.092	0.137
124	1.02	1.17	1.42	0.95	0.554	0.830
125	0.98	1.04	0.31	0.20	0.105	0.157
126	0.98	1.04	0.66	0.44	0.216	0.325
127	0.98	1.04	0.33	0.22	0.140	0.210
128	1.06	1.29	0.47	0.31	0.203	0.305
129	0.98	1.04	0.39	0.26	0.122	0.183
130	1.09	1.36	0.28	0.18	0.131	0.197
131A	1.11	1.42	0.45	0.30	0.220	0.330
131B	0.90	0.83	0.13	0.13	0.017	0.017
132	0.99	1.08	0.32	0.21	0.127	0.190
133	1.11	1.42	0.26	0.17	0.146	0.220
134	0.98	1.04	0.13	0.13	0.018	0.018
135	1.20	1.67	0.63	0.42	0.617	0.925
136	0.98	1.04	0.89	0.59	0.329	0.494
137	0.98	1.04	0.19	0.13	0.098	0.142
138A	0.98	1.04	0.31	0.21	0.105	0.158
138B	0.98	1.04	0.24	0.16	0.082	0.122
139	0.98	1.04	0.68	0.46	0.185	0.277
140	1.05	1.24	1.80	1.20	0.621	0.931
141	0.99	1.09	1.62	1.08	0.510	0.764
142	0.98	1.04	0.13	0.13	0.072	0.072

$(NAME\ IA\ INF\ t_c\ t_{c\_mod}\ R\ R_{stan})$

Typically, define the Initial Abstraction Values for Each Treatment Type- From Table E-4 of SSCAFCA DPM which match those IAs used as the standard in AHYMO. Several reports for the South Valley have increased the IAs based on the very flat slopes in this area. Increased IAs are used for most of the sub-basins based on "Analysis of the AHYMO Program for Flat Areas" by BHI, 1995. However, for these 16 sub-basins the standard IAs are used due to the high level of Treatment Type D. It should be noted that using the SSCAFCAA methodology means that the IA for treatment type D is not factored into the equation.

$IA_A := 0.65 \quad IA_B := 0.50 \quad IA_C := 0.35 \quad IA_D := 0.10$

$j := 0..27$

$$IA_j := \frac{IA_A \cdot PERCENT_{A_j} + IA_B \cdot PERCENT_{B_j} + IA_C \cdot PERCENT_{C_j}}{PERCENT_{A_j} + PERCENT_{B_j} + PERCENT_{C_j}}$$

Define the Infiltration Value for Each Treatment Type - From Table E-4 of SSCAFCA DPM

$INF_A := 1.67 \quad INF_B := 1.25 \quad INF_C := 0.83$

$$INF_j := \frac{INF_A \cdot PERCENT_{A_j} + INF_B \cdot PERCENT_{B_j} + INF_C \cdot PERCENT_{C_j}}{PERCENT_{A_j} + PERCENT_{B_j} + PERCENT_{C_j}}$$

Initial Abstraction and Infiltration Rate Results

SUB-BASIN	IA	INF
112	0.350	0.830
113	0.425	1.040
117	0.425	1.040
119	0.425	1.040
BR 1	0.350	0.830
BR 2	0.350	0.830
GOFF 1	0.350	0.830
GOFF2	0.350	0.830
GOFF 3	0.350	0.830
GOFF 4	0.350	0.830
SUN 1	0.350	0.830
SUN 2	0.350	0.830
SUN 3	0.350	0.830
SUN 4	0.350	0.830
SUN 5	0.350	0.830
SUN 6	0.350	0.830
ISLETA 1	0.350	0.830
ISLETA 2	0.350	0.830
143	0.350	0.830
ARENAL 1	0.350	0.830
ARENAL 2	0.350	0.830
BR 3	0.350	0.830
BR 4	0.350	0.830
BR 5	0.350	0.830
BR 6	0.350	0.830
BR 7	0.350	0.830
BR 8	0.350	0.830
BR 9	0.350	0.830

(NAME IA INF)

This starts the portion of the sheet to calculate the Storage Coefficient (R).

Bring in the calculated Tc values from other files.

$t_c :=$

Sub-basin	Tc
112	0.13
113	2.00
117	3.58
119	2.23
BR 1	0.13
BR 2	0.13
GOFF 1	0.42
GOFF2	0.13
GOFF 3	0.75
GOFF 4	0.13
SUN 1	1.07
SUN 2	0.49
SUN 3	0.28
SUN 4	2.37
SUN 5	0.91
SUN 6	0.58
ISLETA 1	0.43
ISLETA 2	0.15
143	0.16
ARENAL 1	0.13
ARENAL 2	0.13
BR 3	0.13
BR 4	0.13
BR 5	0.13
BR 6	0.13
BR 7	0.14
BR 8	0.13
BR 9	0.13

NAME

Per the SSCAFCA DPM an adjustment is required to the traditional tc in order to make the hydrograph mimic a hydrograph computed in AHYMO. The adjustment is to multiply the tc by 2/3.

$$t_{c\_mod_j} := \begin{cases} 0.1333 & \text{if } t_{c_j} \le 0.20 \\ \left(t_{c_j} \cdot \frac{2}{3}\right) & \text{otherwise} \end{cases}$$

Now the Storage Coefficient (R) can be calculated

$$R_j := 1.165 \cdot t_{c\_mod_j} \left[\left(INF_j\right)^{0.45} - \left(IA_j\right)^{1.4} \left(\frac{PERCENT_{D_j}}{100}\right)^{0.4} \right]$$

Alternative Method for Calculating R based on Stantec's "Technical Documentation for use of HEC-HMS with the Development Process Manual

$$R_{stan_j} := 1.165 \cdot t_{c_j} \cdot \left[\left(INF_j\right)^{0.45} - \left(IA_j\right)^{1.4} \cdot \left(\frac{PERCENT_{D_j}}{100}\right)^{0.4} \right]$$

Per Conversations with Stantec and SSCAFCA this is not their intended methodology. The strict DPM method should be used, not what is shown in Stantec's background manual.

SUMMARY TABLE with R values

Sub-basin	IA	INF	t_c	t_{c_mod}	R	R_{stan}
112	0.35	0.83	0.13	0.13	0.109	0.106
113	0.43	1.04	2.00	1.34	1.147	1.720
117	0.43	1.04	3.58	2.39	2.105	3.157
119	0.43	1.04	2.23	1.49	1.278	1.917
BR 1	0.35	0.83	0.13	0.13	0.109	0.107
BR 2	0.35	0.83	0.13	0.13	0.109	0.107
GOFF 1	0.35	0.83	0.42	0.28	0.229	0.343
GOFF2	0.35	0.83	0.13	0.13	0.109	0.103
GOFF 3	0.35	0.83	0.75	0.50	0.408	0.612
GOFF 4	0.35	0.83	0.13	0.13	0.109	0.104
SUN 1	0.35	0.83	1.07	0.71	0.584	0.875
SUN 2	0.35	0.83	0.49	0.33	0.268	0.402
SUN 3	0.35	0.83	0.28	0.19	0.155	0.232
SUN 4	0.35	0.83	2.37	1.58	1.295	1.942
SUN 5	0.35	0.83	0.91	0.61	0.496	0.744
SUN 6	0.35	0.83	0.58	0.39	0.318	0.477
ISLETA 1	0.35	0.83	0.43	0.29	0.235	0.353
ISLETA 2	0.35	0.83	0.15	0.13	0.109	0.119
143	0.35	0.83	0.16	0.13	0.109	0.135
ARENAL 1	0.35	0.83	0.13	0.13	0.109	0.108
ARENAL 2	0.35	0.83	0.13	0.13	0.109	0.108
BR 3	0.35	0.83	0.13	0.13	0.109	0.108
BR 4	0.35	0.83	0.13	0.13	0.109	0.108
BR 5	0.35	0.83	0.13	0.13	0.109	0.108
BR 6	0.35	0.83	0.13	0.13	0.109	0.108
BR 7	0.35	0.83	0.14	0.13	0.109	0.112
BR 8	0.35	0.83	0.13	0.13	0.109	0.108
BR 9	0.35	0.83	0.13	0.13	0.109	0.108
			0.00			

$$\left(NAME\ IA\ INF\ t_c\ t_{c\_mod}\ R\ R_{stan}\right)$$

CALCULATION OF IA, INF, AND R-VALUE FOR ROADS AND COMMERCIAL SITES

$$\begin{pmatrix} NAME \\ PERCENT_A \\ PERCENT_B \\ PERCENT_C \\ PERCENT_D \end{pmatrix} :=$$

BASINS	A	B	C	D
112	0	0	10	90
113	0	8	8	84
117	0	15	15	70
119	0	8	8	84
BR 1	0	0	15	85
BR 2	0	0	15	85
GOFF 1	0	0	15	85
GOFF2	0	0	15	85
GOFF 3	0	0	15	85
GOFF 4	0	0	15	85
SUN 1	0	0	15	85
SUN 2	0	0	15	85
SUN 3	0	0	15	85
SUN 4	0	0	15	85
SUN 5	0	0	15	85
SUN 6	0	0	15	85
ISLETA 1	0	0	10	90
ISLETA 2	0	0	10	90
143	0	0	15	85
ARENAL 1	0	0	10	90
ARENAL 2	0	0	10	90
BR 3	0	0	10	90
BR 4	0	0	10	90
BR 5	0	0	10	90
BR 6	0	0	10	90
BR 7	0	0	10	90
BR 8	0	0	10	90
BR 9	0	0	10	90

Define the Infiltration Value for Each Treatment Type - From Table E-4 of SSCAFCA DPM

$INF_A := 1.67$ $INF_B := 1.25$ $INF_C := 0.83$

$$INF_j := \frac{INF_A \cdot PERCENT_{A_j} + INF_B \cdot PERCENT_{B_j} + INF_C \cdot PERCENT_{C_j}}{PERCENT_{A_j} + PERCENT_{B_j} + PERCENT_{C_j}}$$

Initial Abstraction Results

SUB-BASIN	IA	INF
101	0.975	1.040
102	0.975	1.040
103	0.975	1.040
104	0.992	1.089
105	0.975	1.040
106	0.991	1.085
107	0.975	1.040
108	0.975	1.040
109	0.975	1.040
110	0.975	1.040
111	0.975	1.040
114	0.975	1.040
115	0.975	1.040
116	0.975	1.040
118	0.975	1.040
120	0.975	1.040
121	0.975	1.040
122	0.975	1.040
123	0.975	1.040
124A	0.975	1.040
125	0.975	1.040
126	0.975	1.040
127	0.975	1.040
128	0.975	1.040
129	0.975	1.040
130	0.975	1.040
131A	0.975	1.040
131B	0.900	0.830
132	0.989	1.080
133	1.111	1.420
134	0.975	1.040
135	0.975	1.040
136	0.975	1.040
137	0.975	1.040
138A	0.975	1.040
138B	0.975	1.040
139	0.975	1.040
140	0.975	1.040
141	0.992	1.087
142	0.975	1.040
124B	1.005	1.125
S100	0.975	1.040
S101	0.990	1.083
S102	0.975	1.040
S103	0.975	1.040
S114	0.991	1.084
S116	0.975	1.040

(NAME IA INF)

This starts the portion of the sheet to calculate the Storage Coefficient (R).

Bring in the calculated Tc values from other files.

$t_c :=$

Sub-basin	Tc
101	1.23
102	2.00
103	3.58
104	2.23
105	0.17
106	1.97
107	0.42
108	0.13
109	0.75
110	0.13
111	1.07
114	0.49
115	0.28
116	2.37
118	1.28
120	0.58
121	0.23
122	0.27
123	0.30
124A	1.42
125	0.31
126	0.66
127	0.33
128	0.47
129	0.39
130	0.28
131A	0.45
131B	0.13
132	0.32
133	0.26
134	0.13
135	0.63
136	0.89
137	0.19
138A	0.31
138B	0.24
139	0.68
140	1.80
141	1.62
142	0.13
124B	1.42
S100	0.39
S101	0.57
S102	0.97
S103	1.08
S114	0.56
S116	0.52

NAME

Per the SSCAFCA DPM an adjustment is required to the traditional tc in order to make the hydrograph mimic a hydrograph computed in AHYMO. The adjustment is to multiply the tc by 2/3.

$$t_{c\_mod_j} := \begin{cases} 0.1333 & \text{if } t_{c_j} \le 0.20 \\ \left(t_{c_j} \cdot \frac{2}{3}\right) & \text{otherwise} \end{cases}$$

Now the Storage Coefficient (R) can be calculated

$$R_j := 1.165 \cdot t_{c\_mod_j} \left[\left(INF_j\right)^{0.45} - \left(IA_j\right)^{1.4} \left(\frac{PERCENT_{D_j}}{100} \right)^{0.4} \right]$$

Alternative Method for Calculating R based on Stantec's "Technical Documentation for use of HEC-HMS with the Development Process Manual

$$R_{stan_j} := 1.165 \cdot t_{c_j} \cdot \left[\left(INF_j\right)^{0.45} - \left(IA_j\right)^{1.4} \cdot \left(\frac{PERCENT_{D_j}}{100} \right)^{0.4} \right]$$

Per Conversations with Stantec and SSCAFCA this is not their intended methodology. The strict DPM method should be used, not what is shown in Stantec's background manual.

SUMMARY TABLE with R values

Sub-basin	IA	INF	t_c	t_{c_mod}	R	R_{stan}
101	0.98	1.04	1.23	0.82	0.447	0.670
102	0.98	1.04	2.00	1.34	0.656	0.984
103	0.98	1.04	3.58	2.39	1.173	1.760
104	0.99	1.09	2.23	1.49	0.669	1.003
105	0.98	1.04	0.17	0.13	0.036	0.047
106	0.99	1.08	1.97	1.32	0.656	0.984
107	0.98	1.04	0.42	0.28	0.149	0.223
108	0.98	1.04	0.13	0.13	0.044	0.042
109	0.98	1.04	0.75	0.50	0.218	0.328
110	0.98	1.04	0.13	0.13	0.056	0.053
111	0.98	1.04	1.07	0.71	0.261	0.392
114	0.98	1.04	0.49	0.33	0.149	0.223
115	0.98	1.04	0.28	0.19	0.040	0.060
116	0.98	1.04	2.37	1.58	0.743	1.114
118	0.98	1.04	1.28	0.85	0.379	0.569
120	0.98	1.04	0.58	0.39	0.191	0.286
121	0.98	1.04	0.23	0.15	0.051	0.077
122	0.98	1.04	0.27	0.18	0.087	0.131
123	0.98	1.04	0.30	0.20	0.092	0.137
124A	0.98	1.04	1.42	0.95	0.464	0.696
125	0.98	1.04	0.31	0.20	0.105	0.157
126	0.98	1.04	0.66	0.44	0.216	0.325
127	0.98	1.04	0.33	0.22	0.109	0.163
128	0.98	1.04	0.47	0.31	0.154	0.231
129	0.98	1.04	0.39	0.26	0.122	0.183
130	0.98	1.04	0.28	0.18	0.091	0.136
131A	0.98	1.04	0.45	0.30	0.147	0.221
131B	0.90	0.83	0.13	0.13	0.017	0.017
132	0.99	1.08	0.32	0.21	0.127	0.190
133	1.11	1.42	0.26	0.17	0.146	0.220
134	0.98	1.04	0.13	0.13	0.018	0.018
135	0.98	1.04	0.63	0.42	0.206	0.310
136	0.98	1.04	0.89	0.59	0.329	0.494
137	0.98	1.04	0.19	0.13	0.098	0.142
138A	0.98	1.04	0.31	0.21	0.105	0.158
138B	0.98	1.04	0.24	0.16	0.082	0.122
139	0.98	1.04	0.68	0.46	0.185	0.277
140	0.98	1.04	1.80	1.20	0.591	0.886
141	0.99	1.09	1.62	1.08	0.510	0.764
142	0.98	1.04	0.13	0.13	0.072	0.072
124B	1.01	1.13	1.42	0.95	0.525	0.787
S100	0.98	1.04	0.39	0.26	0.133	0.199
S101	0.99	1.08	0.57	0.38	0.200	0.300
S102	0.98	1.04	0.97	0.65	0.330	0.495
S103	0.98	1.04	1.08	0.72	0.293	0.439
S114	0.99	1.08	0.56	0.37	0.189	0.284
S116	0.98	1.04	0.52	0.35	0.177	0.265

0.133

$(NAME\ IA\ INF\ t_c\ t_{c\_mod}\ R\ R_{stan})$

CALCULATIONS FOR DEVELOPED SUB-BASINS THAT REQUIRE MODIFIED IA

$$\begin{pmatrix} NAME \\ PERCENT_A \\ PERCENT_B \\ PERCENT_C \\ PERCENT_D \end{pmatrix} :=$$

BASINS	A	B	C	D
101	0	38	38	24
102	0	35	35	30
103	0	35	35	30
104	5	30	30	35
105	0	20	20	60
106	5	33	33	30
107	0	37	37	26
108	0	25	25	50
109	0	32	32	36
110	0	31	31	39
111	0	27	27	45
114	0	33	33	34
115	0	15	15	70
116	0	34	34	32
118	0	32	32	35
120	0	35	35	30
121	0	25	25	50
122	0	35	35	30
123	0	33	33	34
124A	0	35	35	30
125	0	36	36	28
126	0	35	35	30
127	0	35	35	30
128	0	35	35	30
129	0	34	34	32
130	0	35	35	30
131A	0	35	35	30
131B	0	0	15	85
132	5	37	37	21
133	50	16	16	9
134	0	8	8	85
135	0	35	35	30
136	0	38	38	24
137	0	45	45	10
138A	0	36	36	28
138B	0	36	36	28
139	0	30	30	40
140	0	35	35	30
141	5	31	31	33
142	0	37.5	37.5	25
124B	10	32	32	25
S100	0	36	36	28
S101	5	34	34	27
S102	0	36	36	28
S103	0	30	30	40
S114	5	33	33	29
S116	0	36	36	28

85

Typically, define the Initial Abstraction Values for Each Treatment Type- From Table E-4 of SSCAFCA DPM which match those IAs used as the standard in AHYMO. Several reports for the South Valley have increased the IAs based on the very flat slopes in this area. Increased IAs are used here based on "Analysis of the AHYMO Program for Flat Areas" by BHI, 1995. It should be noted that using the SSCAFCAA methodology means that the IA for treatment type D is not factored into the equation.

$IA_A := 1.20$ $\quad IA_B := 1.05$ $\quad IA_C := 0.9$ $\quad IA_D := 0.40$

$j := 0..46$

$$IA_j := \frac{IA_A \cdot PERCENT_{A_j} + IA_B \cdot PERCENT_{B_j} + IA_C \cdot PERCENT_{C_j}}{PERCENT_{A_j} + PERCENT_{B_j} + PERCENT_{C_j}}$$